CITY OF SACRAMENTO
COMBINED WASTEWATER CONTROL SYSTEM

APPENDICES TO
PHASE 3
DETAILED TECHNICAL REPORT

TECHNICAL MEMORANDA

Submitted to the

California Regional Water Quality Control Board
Central Valley Region

Submitted by the

City of Sacramento
Department of Utilities

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August 13, 1992

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TECHNICAL MEMORANDA

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APPENDIX A

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 2, TASK 1

ANALYTICAL APPROACH TO OUTFLOW ANALYSIS

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Paula M. Arsenault
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August 13, 1992

TECHNICAL MEMORANDUM

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN

PHASE 2, TASK 1

(Supplemental Agreement #3, Phase 2 Additional Work)

PREPARED BY: Paula M. Arsenault, HDR Engineering, Inc.
Jerrold J. Troyan, HDR Engineering, Inc.

DATE: February 13, 1992

SUBJECT: Analytical Approach to Outflow Analysis

A. INTRODUCTION

A.1. Definitions

For the purpose of this memorandum and for Phase 3 reports, the following definitions will apply. Figure 1 illustrates the components of the combined sewer system and its contributions, including street floodwater.

Outflow: Combined sewage, containing raw sanitary sewage, which backs up and flows out of the combined system into low-lying areas, streets, or below-grade structures and basements. When outflows occur in low-lying streets, the combined sewage generally comes out through street drop inlets (DIs), which normally function as street drains to convey stormwater runoff into the combined sewers. Outflows into below-grade structures usually enter through the structure's service lateral and private sewer tap connection to the combined sewer system. Whether or not outflows to a low-lying structure or basement occur is highly site specific and dependent upon the configuration and elevation of the individual private sewer tap connection at the combined sewer, and the relative elevations of the combined sewer pipes and the low-lying structure or basement.

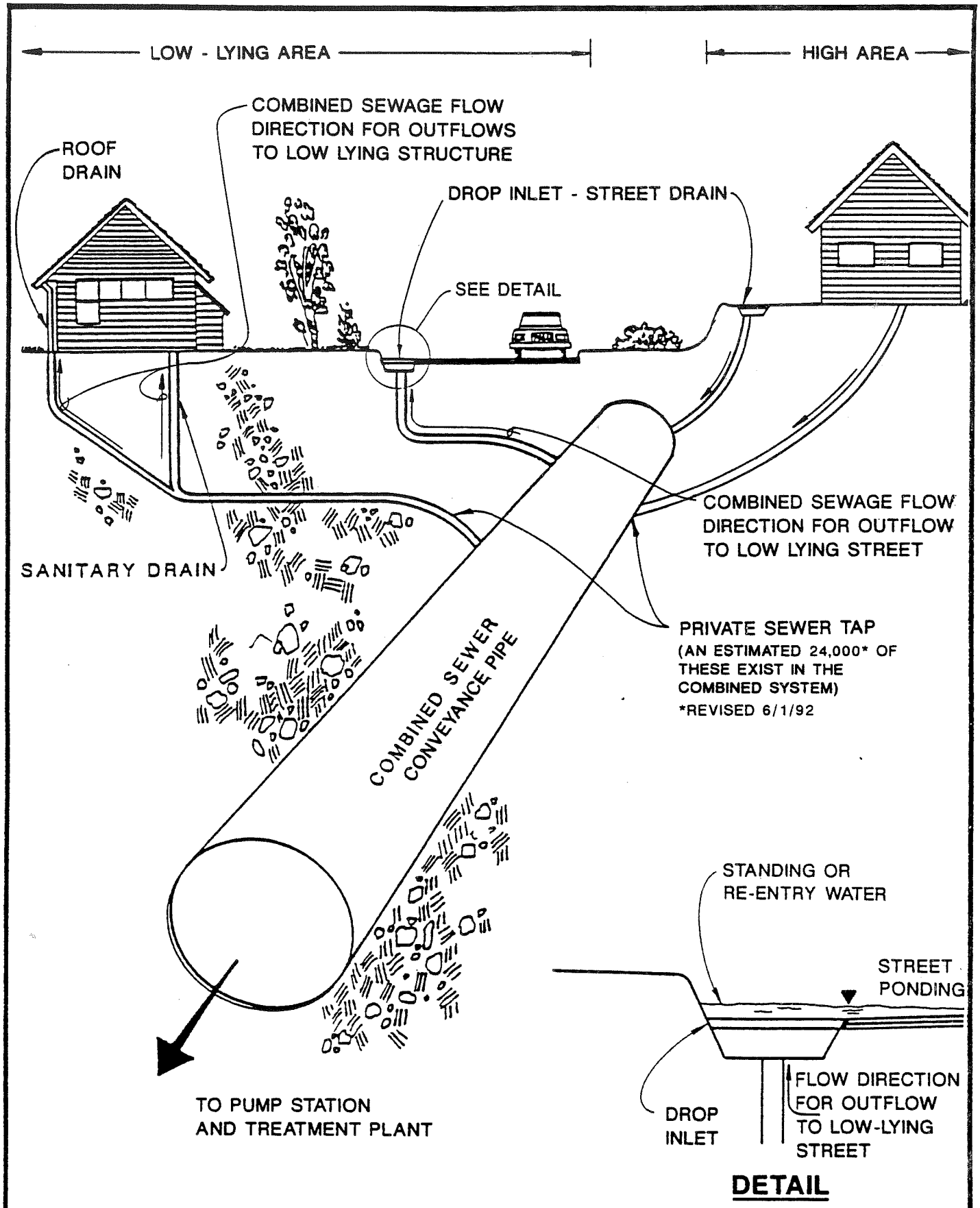

Figure 1. COMBINED SEWER SYSTEM AND FLOW COMPONENTS

Outflows are generally caused by periods of intense rainfall, and by differences in water levels throughout the combined sewer system. Combined sewage is forced out of the sewer system in low lying areas when neighboring higher areas are flooded with excessive amounts of stormwater, and the combined sewage water level (i.e. hydraulic grade line) in the sewers is high. Figure 2 illustrates this phenomenon, indicating how flooding occurs when the hydraulic grade line in the system rises above a low-lying manhole. The relationships of hydraulic grade lines and structures in the figure show, for example, that if inflow continued to enter Manhole B during a 5-year or 10-year storm event, the water level would not only rise (surcharge) in Manhole B, but also cause outflow from Manhole A upstream.

Ponding: Ponding is stormwater runoff that stands in the streets either because of inadequate drainage inlet capacity (rainfall intensity is higher than the rate at which inlet grates can pass the rate of runoff) or because the sewer system is too full to accept the rate of runoff entering through DIs. Ponded stormwater, by this definition, does not include outflow.

Floodwater: Floodwater is composed of an uncontained accumulation of ponded storm water and/or outflows from the combined sewer system.

Surcharging: Surcharging occurs when more water enters the sewers than can be conveyed by the pipes under gravity flow (open channel) conditions. When inflow into the sewers exceeds the pipeline carrying capacity, water is stored in the system. This storage effect causes water to back up in the sewer pipes. The sewers pressurize in this case, and water levels rise in manholes above the crowns of the incoming and outgoing pipes. Surcharging does not always cause outflows from the system,

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T6-FIG48.DWG
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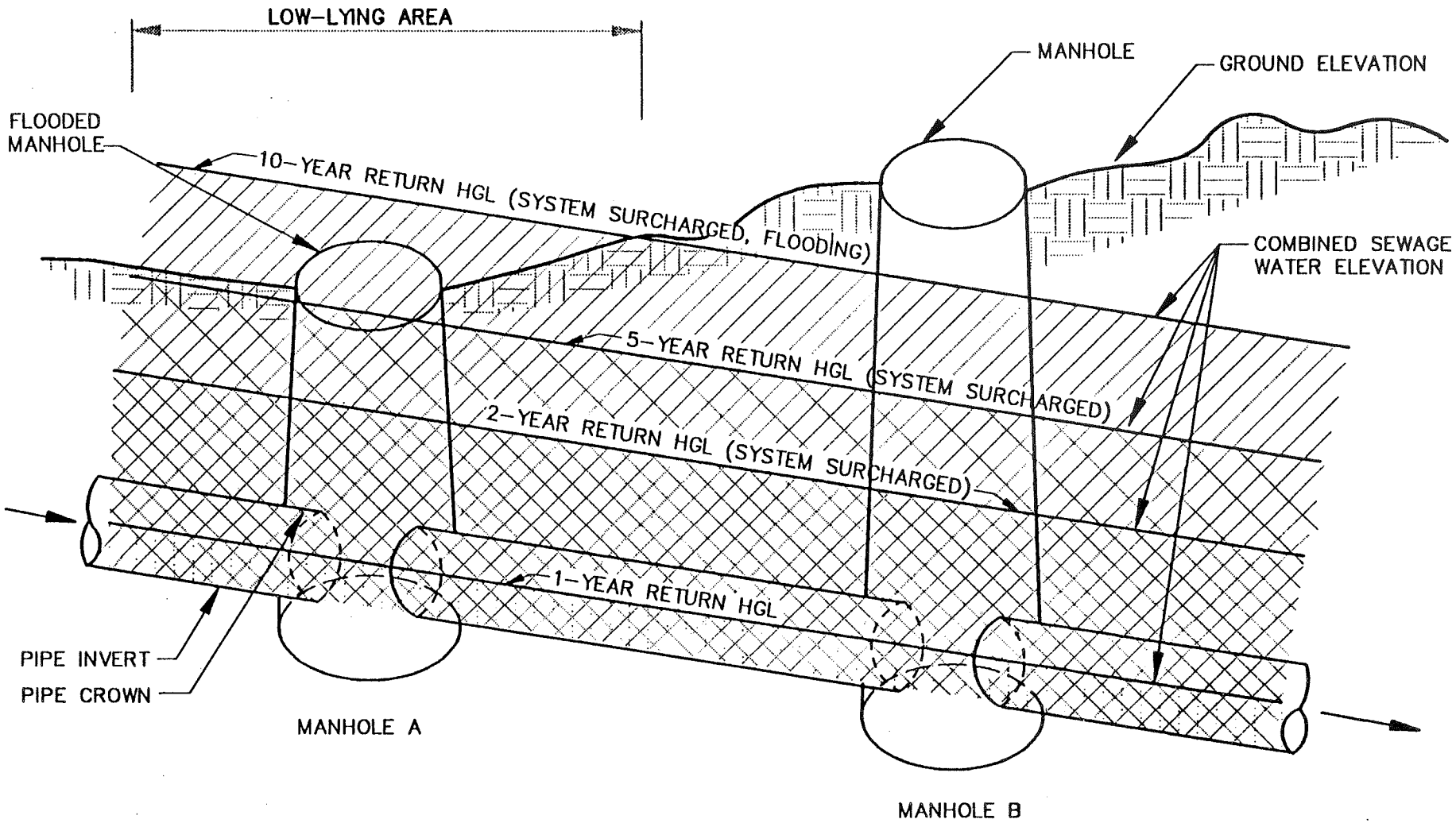

HGL = HYDRAULIC GRADE LINE OR WATER SURFACE ELEV FOR COMBINED SEWER

COMBINED SEWER DESIGN STORMS AND FLOODING DUE TO A SURCHARGED SYSTEM

HDR

Figure 2

but can in low-lying areas, when the hydraulic grade is above street level, as shown in Figure 2.

CSO: Combined sewage overflow, which for the City of Sacramento system, is a discharge of combined sewage to the Sacramento River from the Sump 2 facility or Pioneer Reservoir.

A.2. Objective

Following the submittal of the *Phase 2 Detailed Technical Report*, the Regional Water Quality Control Board (Regional Board) requested that the City determine the quantity/amount of outflows from the combined system and differentiate between floodwater caused by stormwater (ponding) and floodwater caused by outflows. The Regional Board also requested that the City include estimated costs for correcting only combined system outflows and differentiate between the costs for correcting floodwater caused by ponding and floodwater caused by outflows. Specifically, the Regional Board in its comments (dated August 16, 1991) on the *Phase 2 Detailed Technical Report*, stated that the Cease and Desist Order issued to the City does not require the City to address street flooding (ponding) or CSOs. Therefore, the City has been asked to develop costs and improvements associated only with the control of system outflows.

This task was developed to specifically address the Regional Board's request to determine the quantity/amount of outflows and differentiate between floodwater due to ponding versus floodwater due to system outflows. The objective of this task is to determine if such a determination and differentiation is possible and if so, how the analysis could be completed. In particular, this task must determine if such an analysis is possible given the analytical tools available to the City, or through the development of new analytical tools. If such an evaluation is possible, the result would be a description of the analytical steps involved, the costs, and the schedule required to complete the analysis.

This memorandum describes the evaluation needed in general terms, identifies the detailed work effort involved in separating outflows for the purpose of defining

improvement projects and allocating costs, and presents conclusions on the feasibility of completing such an analysis.

B. THEORETICAL ANALYSIS PROCEDURE

Separating costs associated with ponding and CSO control from outflow costs, and minimizing outflows, would require the following basic analytical process:

1) Quantify the total volume of flooding predicted to occur for a given design storm.

2) Differentiate outflow flooding volumes from ponding flooding volumes.

3) Optimize the improvements to minimize (or eliminate, if possible) outflows.

4) Determine the portion of a given alternative plan that reduces outflows, the portion that reduces ponding, the portion that reduces CSO's, and the portion that has multiple benefits (both ponding and outflow reduction).

5) Estimate the costs for the improvements determined in number 4 above.

C. THEORETICAL HYDRAULIC MODEL

The City of Sacramento's existing version of SWMM can accomplish Step 1 of the theoretical analysis procedure defined above, i.e. quantify the volume of flooding predicted to occur. To do this however, it uses an approximation of the complete combined sewer system, like any hydrologic and hydraulic model does when applied to a large, complex system. There are, for example, numerous existing DIs, connections and pipelines which are not explicitly included in the model. Instead, a single inlet or junction included in the model may represent a combination of several DIs. With this simplification, EXTRAN (one block or

portion of the SWMM model) assumes that all water generated by the RUNOFF block is initially taken into the sewer system in its entirety. The model then proceeds through hydraulic calculations to determine water elevations and conveyance flows at the end of an iteration time step. In conjunction with these hydraulic calculations, the excess water (floodwater) that cannot be conveyed by the sewer system is calculated at each junction. The excess water represents the amount of water caused by system outflow, ponding, or a combination of both.

Because outflows are only a portion of the floodwater volumes calculated by the existing version of SWMM, differentiating outflow volumes from ponding volumes (Step 2), as well as optimizing system improvements (Step 3), and determining the components of an alternative plan related to controlling each type of flooding (Step 4), would require the development of an extremely detailed hydraulic model. In fact, such a model would need to keep track of each unit of flow in the system including its specific point of entry into the system. The model would, therefore, need to determine, for a given iteration time step, whether the unit of flow entered a DI, backed out of a DI, was conveyed through the system, ponded on the street, was conveyed down an overflow channel, backed into a basement, or was discharged as CSO.

To correctly carry out these calculations, all individual connections to the sewers and all DIs would have to be specified separately. In addition, for each DI in the system, the individual runoff basin, or watershed, tributary to that inlet would also have to be delineated and specified in the model. Finally, an extensive flow metering and street floodwater sampling program would have to be implemented to provide data to calibrate the model.

Theoretically, if all of the system components were added to the model, and if the basins tributary to all of the drainage inlets were surveyed and added, the existing system could be analyzed. Using Steps 2 through 5 of the theoretical analysis procedure described above, it would then be possible to estimate costs for improvements related directly to control of outflows.

D. FEASIBILITY OF THE ANALYSIS

The entire analysis of outflows for the Sacramento combined sewer system would depend on the ability to develop a very detailed model of the system. Required modifications to the Sacramento SWMM would be extensive, time-consuming, and costly. Even then, the feasibility of such modifications for tracking all of the elements of combined sewer routing in a system as complex as the City's is questionable.

The effort involved to 1) locate and define all pipes and connections in the system; 2) include approximately 7,000 drainage inlets; 3) conduct a detailed aerial and ground survey of approximately 6,500 acres of the combined system; 4) purchase and install calibration flow meters throughout the entire system; 5) sample and analyze for bacteria in street floodwaters as verification of the existence of outflows; and 6) to modify, debug, and calibrate model input files can only be roughly approximated at this time. We believe, however, that it could take up to 3 years to complete, including up to 2 years for surveying and mapping, and up to 15 person years of effort, at a cost of more than $2 million. This effort would then have to be added to the City's other multiple objectives for flood control, rehabilitation, and CSO control as parts of the overall improvement plan for the combined system.

E. CONCLUSION

Based on our review of the Regional Board's request and the analysis tools available for allocating costs to outflow control, meeting the request for information by the Board is definitely not possible by July 1, 1992, when the Phase 3 Detailed Technical Report must be submitted. Beyond that date, we believe that the time and expense necessary to attempt to develop the theoretical SWMM model and cost allocation procedures are unreasonable, and do not appear to be justified by the Board's objective. Finally, it is also possible that after several years of effort, one or more aspects of the modelling effort may prove to be insurmountable or practically impossible to carry out satisfactorily. This same view of the modelling effort is shared by James M. Montgomery, Consulting engineers (JMM), who are special consultants to the City regarding the Sacramento

SWMM model, and who have made major improvements to the model in the past few years. A letter from JMM is attached for reference.

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Environmental Needs

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JMM James M. Montgomery
Consulting Engineers, Inc.

15 November 1991

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Attn: Ms Paula Arsenault

Dear Paula:

In accordance with your request, we have reviewed your draft memorandum dated November 8, 1991 concerning the analytical approach to outflow analysis.* We concur with your statement in section B.2.a that it is practically impossible to separately estimate "ponding water" and "outflow" using the existing City's version of SWMM. To clarify, we would like to provide the following information.

Any hydrologic and hydraulic modeling requires an approximation of the system being modeled. This is especially true for the combined sewer system for the City of Sacramento. There are numerous drainage inlets (DIs), connections, and pipelines which are not explicitly included in the model. A single inlet or junction specified in the model may represent a combination of several drainage inlets. With this simplification, Extran assumes that all water generated by the Runoff block is initially taken into the sewer system in its entirety. The model then proceeds through hydraulic calculations to determine hydraulic heads and conveyance flows at the end of an iteration timestep. In conjunction with these hydraulic calculations, the excess water or floodwater that cannot be conveyed by the sewer system is calculated at each junction. The excess water may represent the amount of water caused by system outflow, ponding water, or a combination of both.

Theoretically, the separation of outflow and ponding water may be possible by making model modifications, however, the results would have no practical meaning. The amount of ponding water theoretically calculated by the model may actually represent storm runoff that entered upstream DIs not separately included in the model and is coming out of the system as outflow. To correctly distinguish between outflows and ponding water, all individual connections to the sewers and drainage inlets have to be specified separately. In addition, an individual watershed draining to each inlet has to be delineated and specified in the model. This effort would require a detailed survey of the entire sewer system and modification of the existing SWMM model, which is considered to be insurmountable.

If you have any questions, feel free to give me a call and we can discuss the problem further.

Sincerely,

James M. Montgomery,
Consulting Engineers, Inc.

Young S. Yoon, PE
Vice President

*Note: The outline of the final memorandum (February 13, 1992) is different than the draft memorandum this letter refers to, but the ideas expressed are the same.

APPENDIX B

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 4

EVENT REPORT FOR THE DECEMBER 7, 1991, SAMPLING EVENT

Paula M. Arsenault
HDR Engineering, Inc.

Philip K. Walsack
Beak Consultants, Inc.

September 1, 1992

CITY OF SACRAMENTO
COMBINED SEWER SYSTEM
IMPROVEMENT AND IMPLEMENTATION PLAN

PHASE 3 STUDY

CWCS SAMPLING PROGRAM FOR 1991/1992

WATER QUALITY AND TOXICITY RESULTS FOR THE DECEMBER 7, 1991 SAMPLING EVENT

Prepared for

City of Sacramento
Utilities Department
Engineering Division

Prepared by

HDR Engineering, Inc.
El Dorado Hills, CA

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September 1, 1992

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SECTION 1
PROGRAM OVERVIEW AND OBJECTIVES

A. INTRODUCTION

This report summarizes the results of the December 7, 1991 sampling event. As background for reviewing these results, this section provides a brief overview of the sampling and analysis plan for the Combined Wastewater Collection and Treatment System (CWCS) and Sacramento River for the 1991/1992 wet season. This program parallels, but has been conducted separately from the "routine" monitoring program specified in the City's NPDES Permit No. CA0079111 for the CWCS. This program was conducted as part of the Phase 3 study to develop an improvement and implementation plan for the Sacramento combined system.

Four key elements were used to develop a water quality sampling program for the 1991/1992 wet weather season: 1) the results of the 1990/1991 wet weather sampling for two storm/combined sewer overflow (CSO) events; 2) the NPDES permit for the CWCS, which requires that biotoxicity tests be performed on CWCS "effluent" samples, including Combined Wastewater Treatment Plant (CWTP) effluent and CSOs from Pioneer Reservoir and Sump 2; 3) a request by the RWQCB staff to conduct a matrix detection limit study for lead; and 4) a request by the RWQCB to include a screen for the pesticides and PCBs included in the *California Inland Surface Waters Plan*, "Objectives for Protection of Freshwater Aquatic Life". (See Section 4 for details on these objective concentrations.)

B. OBJECTIVES

Based on the elements described above, the objectives for the 1991/1992 CWCS Sampling Program were as follows:

1) Determine the chronic toxicity of CWTP effluent, Sump 2 overflows and Pioneer Reservoir overflows. Determine the seasonal variation of toxicity, if possible, by sampling an early-season CWTP "operation-only" event, as well as two additional storm events that are large enough to both require

operation of CWTP and produce CSOs to the Sacramento River. The objective was to sample CSOs from each of the outfall locations, even if overflows were not necessarily simultaneous at Sump 2 and Pioneer Reservoir. Determine the No Observable Effect Concentration (NOEC) and Lowest Observable Effect Concentration (LOEC) for CWTP effluent by performing toxicity tests on dilutions of effluent for all three of the events sampled. For the CSO samples (Sump 2 and Pioneer Reservoir), determine toxicity on a "screening level" basis, by testing the undiluted (100 percent) samples. If toxicity was found in the 100-percent CSO samples, then dilution series chronic tests would be performed during the subsequent 1992/1993 wet season.

2) Sample local street floodwater and analyze for bacteria content, in order to estimate the source of bacteria contamination and, if possible, to identify locations having consistent sewage outflows onto streets during storm events. It was intended that City staff would complete the work related to this objective.

3) Determine the impact of trace metal pollutant discharges on the Sacramento River. Collect and analyze samples from the Sacramento River at appropriate locations during two storm events large enough to require CWTP operation and overflows from Pioneer Reservoir and/or Sump 2. If overflows from a particular event did not occur simultaneously at Sump 2 and Pioneer Reservoir, samples would be collected for the one CSO location. (Ultimately, the objective was to capture two CSO's at each location.) These river sampling events would be concurrent with the CWTP discharge sampling events, for at least two CSO events. Combined wastewater would be characterized for Pioneer Reservoir overflows, Sump 2 overflows (total station flow greater than 190 mgd), and CWTP effluent.

4) Determine the presence of organochlorine pesticides and polychlorinated biphenyls (PCBs) in CWCS discharge samples, using standard screening analysis methods. In addition, determine the presence of tributyltin. Tributyltin is an organic compound included in the previously mentioned objective concentrations for aquatic life, and requires special analytical

methods. Testing for these constituents in the CSO and effluent samples rather than the river would yield "worst case" results with respect to river impacts, because the river dilutes the discharge.

5) Perform a matrix detection limit (MDL) study for lead concentrations in the four unique matrices being sampled in the CWCS. The four matrices involved are: the Sacramento River in the vicinity of CSO discharges, CWTP effluent, Sump 2 overflow, and Pioneer Reservoir overflow. The objective was to determine the repeatable reporting limit concentrations that can be achieved for these matrices using standard analysis methods.

C. OBJECTIVES MET BY THE CWTP "OPERATION ONLY" SAMPLING EVENT

This report documents the findings of the CWTP "operation-only" event that occurred on December 7, 1991, and is the first in a series of event reports to be compiled for events sampled during the 1991/1992 wet weather season. During this event, CWTP discharged effluent to the Sacramento River; no CSOs occurred from either Sump 2 or Pioneer Reservoir.

The primary objective of sampling this early-season CWTP "operation only" event was to provide three sets of chronic toxicity test data for CWTP effluent for the 1991/1992 wet season. The other two sets (one being provided by this event) would be provided by the two CSO events sampled later in the season. In addition, it was intended that sampling this early-season event, along with test results from later-season events, would allow for an evaluation of seasonal variation, if any, in the chronic toxicity of CWTP effluent. It was also intended that the assessment of impacts to the Sacramento River from CWCS discharges, including CWTP effluent, would be conducted during the two storm/CSO events captured during the 1991/1992 wet season. This approach, as identified in the Phase 3 Scope of Work, is identical to that implemented during the previous, 1990/1991 wet season; two storm/CSO events were sampled to evaluated potential river impacts following discharges. For these reasons, samples were not collected from the river downstream of the CWTP outfall.

C.1. Chronic Toxicity

Provision 5 of the City's NPDES permit requires that a biotoxicity study be completed on 100-percent effluent samples during the 1991/1992 wet season. If toxicity is present, the City would be required to perform a dilution series as part of its 1992/1993 monitoring plan. The City voluntarily accelerated this schedule in the case of the CWTP effluent biotoxicity tests, by performing dilution series tests on CWTP effluent samples under the 1991/1992 monitoring program.

The toxicity of CWTP effluent dilutions, which define the LOEC and NOEC points, were analyzed in accordance with EPA protocols that specify the following dilution percentages: 100, 50, 25, 12.5 and 6.2. As per the EPA protocols, dilution water was collected from the river upstream of the CWTP outfall at Captain's Table dock, and was analyzed for chronic toxicity. This analysis was conducted to provide baseline or "control" information on Sacramento River water used for the dilution series.

Procedures for determining if toxicity is present are defined in EPA 600/4-89/001 (*Short-term Methods for Estimating the Chronic Toxicity of Effluent and Receiving Water to Freshwater Organisms*) and EPA 440/4-85/032 (*Technical Support Document for Water Quality Based Toxic Control*). EPA's three-species freshwater chronic toxicity test procedures were used, as specified in the CWCS NPDES permit and in Table 1-1.

TABLE 1-1. BIOTOXICITY TEST PARAMETERS

Test Specie	Type of Test	EPA Method
Fathead Minnow	Survival/Growth	1000
Ceriodaphnia (water flea)	Survival/Reproduction	1002
Selenastrum (algae)	Growth	1003

The biotoxicity analyses completed for these CWTP effluent and river water samples fulfills part of Objective 1 for the 1991/1992 CWCS Sampling Program. However, any conclusions on the toxicity of CWTP effluent and system CSOs can only be made from the analysis of multiple events. A summary report for all three events sampled during the 1991/1992 wet season presents conclusions, if any, on the consistent and/or seasonal presence of chronic toxicity.

C.2. Bacteria Content of Street Floodwaters

The December 6-7 storm event was generally low in intensity, with several short periods of high intensity rainfall. Because of these storm characteristics, local street flooding was minimal. Therefore City staff did not collect samples of local street floodwaters as part of this CWTP "operation only" event.

C.3. Trace Metal Pollutants

The collection and analysis of water quality samples was conducted to determine the presence of trace metal pollutants in the CWTP discharge. Water quality samples were also collected and analyzed for the dilution water collected from the river at Captain's Table dock.

The trace metal analysis fulfilled the requirements of Objective 3 for the 1991/1992 CWCS Sampling Program.

C.4. Organochlorine Pesticides/PCBs

A CWTP effluent discharge sample was analyzed to determine the presence of organochlorine pesticides and PCBs. During this sampling event, samples were not collected to determine the presence of tributyltin. This analysis was conducted as part of the CSO events later in the wet season.

Although not intended to be part of the program, the field crew at Captain's Table dock on the Sacramento River collected a grab sample in glass bottles for analysis of organochlorine pesticides and PCBs, which were analyzed by the laboratory. This was not done during the two, later-season events. The results

were evaluated and are included in this report.

The organochlorine pesticides and PCBs analysis fulfilled the requirements of Objective 4 for the 1991/1992 CWCS Sampling Program.

C.5. Lead MDL Study

Samples for the lead MDL study were not collected during this event. This study was completed as part of the February 12, 1992, CSO event. The findings of the study will be submitted in a separate report.

C.6. Other Constituents

In addition, Sacramento River water and CWTP effluent were analyzed for bacteriological and inorganic constituents. The chemistry of the dilution water (obtained from the river) was also analyzed from grab samples collected during the storm event.

D. REPORT OVERVIEW

This report is organized into seven major sections. Section 1 summarizes the CWCS sampling program for the 1991/1992 wet weather season. Section 2 describes the event sampling program, including the field methods, sample collection and handling procedures, and laboratory quality assurance procedures. In addition, this section describes, in tabular form, the general analysis matrix and the specific constituents analyzed during the CWTP "operation only" event. Section 3 summarizes the storm event, field data results, and the results of the quality assurance analysis conducted by the laboratory. Section 4 describes the derivation of the water quality comparative standards. Section 5 summarizes the water quality sample results for the December 7, 1991, storm event. Section 6 provides a description of the toxicity program, test results for the toxicity analysis, and quality assurance results. Section 7 then summarizes the results for the sampling event.

SECTION 2
EVENT SAMPLING PROGRAM

This section describes the field procedures used to collect and handle the water quality samples obtained from the CWTP effluent discharge and the Sacramento River.

A. FIELD METHODS

The *Field Manual* prepared for Phase 3 work provides a detailed description of the sampling procedures and instructions for all locations and constituents involved in the sampling program. Because the timing, movement, and intensity of each storm event differs, the *Field Manual* (final version dated January 30, 1992), was intended to provide guidelines for a theoretical storm pattern. Storm event descriptions are presented in Section 3 of this report.

A summary of the sampling and analysis matrix for this event is listed within Table 2-1. The table identifies the types of analyses and groups of constituents of interest for this CWTP "operation only" event. Table 2-2 lists the constituents analyzed during this storm event.

A.1. Sacramento River

The Sacramento River water obtained for preparing effluent dilutions was collected at Captain's Table dock. Inorganic chemistry (including bacteria and trace metals) analyses were completed for this sample. River water was collected directly from the river into plastic receptacles and transferred into the sample containers. The water for oil and grease analysis was collected in glass bottles filled directly from the river. The laboratory composited these in a glass container upon delivery from the field. As mentioned in Section 1, the field crew also obtained a grab sample in glass bottles for analysis of organochlorine pesticides and PCBs Water temperature was measured at the time of sample collection.

A.2. CWTP

At the CWTP, effluent was collected at 30-minute intervals from the spigot in the plant's wet laboratory. This spigot is plumbed into the effluent stream following the dechlorination stage of treatment. Water for the inorganic analyses was composited into 48-liter polyethylene carboys at 30-minute intervals. The volumes added to the carboys were time-dependent and are specified in the *Field Manual*.

The samples for oil and grease analysis and organochlorine pesticides/PCBs analysis were collected in glass bottles filled directly from the spigot every 30 minutes. These sample bottles were then composited into one glass container at the laboratory.

Water temperature and dissolved oxygen were measured at the time of each 30-minute interval collection at CWTP.

B. SAMPLE COLLECTION AND HANDLING PROCEDURES

All containers, ice chests, and packing materials were prepared and provided by the laboratory. All filtering, necessary for analyzing dissolved metals and phosphorous, was performed by the laboratory upon delivery of the samples. Exposure to sunlight was avoided by keeping sample bottles in covered ice chests. Exposure to sunlight was negligible for this event because samples were collected under heavy cloud cover during the night and early morning hours of the storm event. All samples were packed on ice, without freezing, prior to transport to the laboratory.

B.1. Total Kjeldahl Nitrogen, Chemical Oxygen Demand, and Total Phosphorus

The container used for the collection of these constituents was a glass bottle preserved with sulfuric acid. The sample bottle was filled leaving a small air space at the top.

TABLE 2-1. SAMPLING AND ANALYSIS MATRIX FOR CWTP "OPERATION-ONLY" EVENT[1]		
Location/Description	**Analysis/Constituents**	**Sample Type**
Sacramento River Site		
Captain's Table		
"Dilution"	Dilution Water for CWTP Chronic Toxicity Test	Grab
For background of river water quality used to prepare chronic toxicity test dilutions of CWTP effluent	Chemistry[2] (of dilution water)	Grab
	Oil and Grease	Grab[5]
	Pesticides/PCBs[6]	Grab[5]
	MS/MSD[8]	Grab
	Duplicate[7]	Grab
	Chronic Toxicity (of dilution water, 100% sample test for control compared to CWTP effluent toxicity)	Grab
CWCS Discharge Site		
CWTP Effluent		
"CWTP"	Chemistry[2]	Composite[3]
	Oil and Grease	Composite[4,5]
	Pesticides/PCBs	Composite[4,5]
	MS/MSD	Composite[3]
	Chronic Toxicity (dilution series)	Composite[3]

1 No CSO discharges released.
2 Includes inorganics, bacteria and trace metals. See Table 2-2.
3 Composites consisted of discrete samples collected at 30-minute intervals for the duration of the discharge.
4 Discrete samples were obtained during each 30-minute interval for the duration of the discharge. Laboratory prepared composite sample in a glass container.
5 Enough material collected to allow for MS/MSD analyses.
6 For this event only.
7 This was a sample split rather than a field duplicate.
8 Matrix spike/matrix spike duplicate.

TABLE 2-2. CONSTITUENTS ANALYZED FOR DECEMBER 7, 1991 CWTP "OPERATION ONLY" SAMPLING EVENT

Constituents	EPA Method Number	Reporting Limit[9]	Container Type	Sample Size
Bacteria				
Total Coliform	908A	2 MPN	Plastic	2 X 250 ml
Fecal Coliform	908C	2 MPN	Plastic	
Fecal Streptococci	908C	2 MPN	Plastic	
Inorganics				
pH	150.1	N.A.	Plastic	500 ml[1]
Temperature	Field	-	-	-
Turbidity	180.1	N.A.	Plastic	500 ml[1]
Dissolved Oxygen	Field	-	-	-
Hardness as $CaCO_3$	314A	3.3	Glass w/ HNO_3	500 ml[2]
Total Kjeldahl Nitrogen	351.2	0.5 mg/l	Glass w/ H_2SO_4	1 liter[2]
Phosphorus (total)	365.3	0.5 mg/l	Glass w/ H_2SO_4	1 liter[2]
Phosphorus (dissolved)	365.3	0.5 mg/l	Plastic	500 ml[3]
Nitrate	300.0	0.5 mg/l	Plastic	500 ml[1]
Nitrite	300.0	0.5 mg/l	Plastic	500 ml[1]
Chemical Oxygen Demand	410.4	5.0 mg/l	Glass w/ H_2SO_4	1 liter[2]
Biochemical Oxygen Demand	405.1	2.0 mg/l	Plastic	500 ml[2]
Total Suspended Solids	160.2	5 mg/l	Plastic	500 ml[4]
Total Dissolved Solids	160.1	10 mg/l	Plastic	500 ml[4]
Oil and Grease	413.2	1.0 mg/l	Glass	2 liters[5]
Organics				
Organochlorine Pesticides/PCBs	608	0.05-1.0 µg/l	Glass	2 liters[5]
Metals (Total, Recoverable, and Dissolved)				
Arsenic	206.3 (HAA)	1.0 µg/l[6]	Plastic	--[3,7]
Cadmium	213.2 (GFAA)	0.4 µg/l	Plastic	--
Chromium	218.2 (GFAA)	0.3 µg/l	Plastic	--
Copper	220.2 (GFAA)	0.5 µg/l	Plastic	--
Lead	239.2 (GFAA)	1.7 µg/l	Plastic	--
Mercury[8]	245.1 (CVAA)	0.2 µg/l	Plastic	--
Selenium	270.3 (HAA)	0.3 µg/l[6]	Plastic	--
Silver	272.2 (CVAA)	0.1 µg/l	Plastic	--
Zinc	200.7 (ICP)	8.5 µg/l	Plastic	--

1 Analyses for pH, turbidity, nitrate and nitrite conducted from one sample container.
2 Analyses for total Kjeldahl nitrogen, COD, and total phosphorus collected from one sample container.
3 Analysis for dissolved phosphorous and dissolved metals conducted from one 500-ml container. The laboratory filtered the sample and then preserved with HNO_3 before analysis.
4 TDS and TSS collected in one container, and separated by laboratory for analysis.
5 Total amount required for analysis. Discrete samples collected at CWTP and composited by the laboratory. Grab sample also collected from the river only for this event.
6 Analysis of recoverable concentrations for arsenic and selenium is by GFAA, which results in a higher reporting limit of 5 µg/l. Analysis for total and dissolved concentrations for arsenic and selenium is by HAA, which results in reporting limits of 1.0 µg/l and 0.3 µg/l, respectively.
7 Analysis for total and recoverable concentrations, and hardness conducted from 2 500-ml sample containers, preserved with HNO_3.
8 No method available for analyzing recoverable mercury.
9 Actual results vary from these in some cases. See Sections 3 and 5.

B.2. Turbidity, pH, Nitrate and Nitrite

The container used for the collection of these constituents was a polyethylene bottle, which was filled leaving a small air space at the top of the bottle.

B.3. Total Suspended Solids and Total Dissolved Solids

A single polyethylene bottle was used to collect water for both of these analyses. The sample was split into two batches at the laboratory and analyzed for each constituent.

B.4. Bacteriological and Biological Oxygen Demand

The containers for collecting the bacteriological samples were preserved with sodium thiosulfate to neutralize the effects of residual chlorine on the organisms in the sampled waters. The container for BOD analysis was a polyethylene bottle with no preservatives.

B.5. Total Metals, Recoverable Metals and Hardness

The containers used for the collection of these constituents were polyethylene bottles preserved with nitric acid.

B.6. Dissolved Metals and Dissolved Phosphorus

The containers used for the collection of these constituents were unpreserved polyethylene bottles. Upon delivery to the laboratory, the sampled waters were filtered and subsequently preserved with nitric acid.

B.7. Oil and Grease, and Organochlorine Pesticides/PCBs

The containers used for the collection of these constituents were glass bottles. The sampled waters did not contact plastic containers or implements in their compositing or storage.

C. LABORATORY QUALITY ASSURANCE METHODS

Enseco Laboratory performed various quality assurance procedures in the analysis of the CWCS and river samples to ensure the quality of the test results. The *Enseco Incorporated Quality Assurance Program* requires the analysis of several types of control samples as standard elements in their Laboratory Performance Quality Control process. Duplicate control samples, single control samples, method blanks, and equipment blanks are used to identify any procedural errors. These are a part of Enseco's in-house QA/QC procedures.

Duplicate control samples (DCS) consist of a pair of standard, control matrix samples (reagent water is used for aqueous samples) spiked with surrogate compounds representative of the constituents to be tested. Accuracy and precision data are compared to the control limits established for the tested constituents. This is a standard procedure conducted on all sets of samples.

A single control sample (SCS) is also a control matrix sample, spiked with a surrogate compound appropriate to the testing method. This test is used with smaller batches of samples. The recovery data are compared to the control limits established for the tested compound.

Method blanks (MB) are used to determine the level of interference or contamination in the analytical system that could elevate or diminish concentration results. The concentration of target constituents in the MB should be below the reporting limit. However, common solvents, which are present in the laboratory, may be acceptable at levels up to 5 times the reporting limit. Because certain metals may also be difficult to reduce to the extremely low limits used in this study, concentrations may be accepted up to 2 times the reporting limit.

D. FIELD QUALITY CONTROL METHODS

Matrix spikes (MS) and matrix spike duplicates (MSD) are environmental samples collected in the field. The MS and MSD samples are spiked with known concentrations of constituents, and from the analysis of these controls, the

percent recovery of the added constituents is determined. Based upon the results of this analysis, the effect of the matrix on the accuracy of the analysis procedure is determined. Both the MS and the MSD results indicate the precision of the analysis for a particular constituent.

As part of the quality assurance program, the laboratory split each water quality sample in half upon receipt from the field. (These were the "Dilution" samples, collected from the river at Captain's Table dock.) These two sets of samples were then analyzed. The parent samples were labeled with an "SA" suffix in the analytical results. The quality control samples were labeled with a "DU" suffix in the analytical results. This quality assurance methodology was used to verify the laboratory's ability to replicate precise and accurate analytical results.

In the initial sampling plan, field duplicates (as opposed to sample splits), were intended to be used as the field quality control samples. However, the sample split results proved to be a more stringent measure of laboratory precision and accuracy. Therefore, the sample splits rather than field duplicates were used as the field quality control check on the laboratory.

SECTION 3

STORM EVENT DESCRIPTIONS, FIELD DATA RESULTS AND QUALITY ASSURANCE RESULTS

This section describes the event during which sampling took place, presents the field data, and discusses the quality control assurance analytical results.

A. STORM EVENT AND DISCHARGES

During the December 7, 1992 storm event, 0.64 inches of rainfall were measured at the National Weather Service gauge located at 801 I Street in Sacramento. The 6.5-hour storm began at 9:00 p.m. on December 6 and lasted until 3:30 a.m. December 7. Effluent discharged from CWTP was recorded and sampled during this storm event. No discharges occurred from Sump 2 or Pioneer Reservoir during this storm event.

The CWTP discharge began at 3:00 a.m. and lasted until 5:15 a.m. on December 7. Over the course of this treatment period (2.25 hours), 6.4 million gallons (MG) of effluent was discharged into the Sacramento River.

The Sacramento River was flowing at a rate of approximately 4,994 million gallons per day, or 7,726 cubic feet per second, past the USGS gauge at Freeport Bridge.

These storm and discharge event descriptions and data were prepared by City staff.

Assuming this river flow rate was the average rate that occurred during the discharge period, then the dilution factor, based on volume, was as follows:

$$\text{CWTP Dilution} = \frac{\text{River Volume}}{\text{Effluent Volume}} = \frac{4{,}994 \text{ mgd} * 2.25 \text{ hrs}}{6.4 \text{ MG}} \frac{1}{24 \text{ hrs}} = 73:1$$

This river dilution factor is used to estimate the potentially detectable concentrations of organic compounds in the river, from the test results for CWTP effluent. Section 5 presents this evaluation.

B. FIELD DATA RESULTS

Water temperature, dissolved oxygen (DO) concentration measurements, and time of collection for each of these samples are presented in Table 3-1. Water temperatures recorded from samples collected within the system were stable at 10.0 °C. DO concentrations greater than 11.0 mg/l were recorded for the majority of the discharge period.

TABLE 3-1. FIELD DATA RESULTS FOR DECEMBER 7, 1991 EVENT

Sample Identification	Time	Temp (°C)	DO (mg/l)
CWTP Effluent	0315	10.0	12.0
	0345	10.0	11.8
	0415	10.0	12.4
	0445	10.0	11.6
	0525	10.0	10.8
Captain's Table I	0755	10.0	No Data[1]

1 The dissolved oxygen meter produced unreliable data in the rain.

C. EVENT-SPECIFIC QUALITY ASSURANCE ANALYSIS RESULTS

As described in Section 2, a series of in-house laboratory and project-specific quality assurance procedures were applied in the analysis of the CWCS samples. Laboratory quality assurance procedures, designed to assess laboratory operations and to establish method-specific control limits, included the analysis of method blanks, matrix duplicates, single control samples, and duplicate control samples. Matrix spike (MS) and matrix spike duplicate (MSD) analyses of field samples were conducted to assess potential sample matrix effects.

The quality assurance results indicate that the laboratory testing procedures produced reliable results for the various analytes within the water quality samples. Most percent recoveries, relative percent differences, and matrix interferences were within the accepted control limits. However, several specific complications resulting from matrix interferences, excessive relative percent differences, and laboratory contaminants influenced the interpretation of the results.

All reporting limits for organochlorine pesticide and PCB concentrations were raised for the CWTP effluent samples due to matrix interferences. Following the initial analysis, the results did not match the calibration curve for these analytes. The sample was then diluted to obtain more accurate results requiring that the reporting limits be raised. With a Method 608 screen analysis, it is not possible to exactly determine the cause of matrix interferences. Possible sources of interference are presence of pthalitic esters, sulfur or plasticisers. With regard to the latter, even items such as plastic cups in stormwater entering the sewers can cause a problem in the analysis. Because of this potential, the field and laboratory procedures required that sample water for organochlorine pesticide/PCB analysis be handled only in glass. For the river water sample, no such interferences were found.

The duplicate control sample for the oil and grease analysis did not meet the relative percent difference (RPD) quality control limit, according to the Environmental Protection Agency's *Solid Waste Analytical Methods Manual 846, Third Edition*. The RPD is a measure of precision, determined by comparing the two percent recovery tests of a controlled spike sample. For the oil and grease analysis, the calculated RPD was 28 percent, which exceeded the quality control goal of 18 percent. The RPDs, however, for the matrix spike and matrix spike duplicate field samples were acceptable.

Sample matrix interferences were observed in the analyses of nitrite, nitrate, arsenic and silver. Reporting limits were raised to compensate for the interferences. Laboratory quality control personnel indicated that the matrix interferences could have been caused by acids used to preserve samples in the containers, or acids used in specific analyte preparation.

The reporting limit for the dissolved mercury analysis of the CWTP effluent sample was raised from 0.00020 mg/l to 0.00040 mg/l. This reporting limit was raised to reflect limitations in sample volume. Because the initial analytical results did not meet the in-house laboratory QA/QC guidelines, the laboratory re-analyzed the sample. In doing so, a significant portion of the sample volume was consumed. The laboratory split the sample volume into smaller fractions in order to conserve the remaining sample water. Therefore, the reporting limits were raised to reflect this volume reduction in the analyzed sample. At the revised reporting limit, dissolved mercury was not detected.

Two metal analytical results, chromium and copper, may have been influenced by laboratory contaminants in the preparation of the samples. Total chromium was found in one laboratory sample blank at a concentration of 0.00074 mg/l, using a reporting limit of 0.00050 mg/l. Total copper was also found in one laboratory sample blank at a concentration of 0.00091 mg/l, using a reporting limit of 0.00050 mg/l. Quality control procedures adopted from the Environmental Protection Agency's *Solid Waste Analytical Methods Manual 846, Third Edition*, states that sample blank contamination must be less than two times the MDL. The chromium and copper concentrations in the sample blanks were within these guidelines. For the trace metal analytes, the MS and MSD results were acceptable.

SECTION 4
COMPARATIVE STANDARDS

This section describes the derivation of numerical objectives for pollutant concentrations used to provide a basis for comparison to the laboratory results for the storm event. The impact on the receiving water, i.e. the Sacramento River, is estimated through the use of water quality objectives.

A. INLAND SURFACE WATERS PLAN

The *California Inland Surface Waters Plan - Water Quality Control Plan for Inland Surface Waters of California - 91-12 WQ* (Water Resources Control Board, April 1991), provides a comparative standard with which the laboratory analytical results can be evaluated. (This document is herein referred to as the *Inland Surface Waters Plan*.) Table 1 in this document presents the "Water Quality Objectives for Protection of Freshwater Aquatic Life", including 4-day average, daily average, 1-hour average, and instantaneous maximum pollutant concentration objectives.

Of the four durations (4-day, daily, 1-hour, instantaneous) for which water quality objectives are identified in Table 1 of the *Inland Surface Waters Plan*, an appropriate duration must be selected for comparison to the storm/CSO event results in this report. The duration of effluent discharges from the CWTP during this event was 2.25 hours, which exceeds 1 hour but is far shorter than 4 days. It would therefore be most appropriate to compare the event-specific results to a comparative standard for a duration between 1 hour and 4 days, consistent with the duration of the CWTP discharge. Ideally, comparable standards could be made available that account for the variability of storm and discharge event durations. However, no such published standards exist. Therefore, the storm event results were compared to <u>both</u> the 4-day and the 1-hour numerical objectives. Specific results for the storm event sampled, and a comparison to calculated numerical objective concentrations, are presented in Section 5.

B. DERIVATION OF NUMERICAL OBJECTIVES

B.1. Receiving Water Hardness

The *Inland Surface Waters Plan* (specifically, Table 1) identifies specific concentrations for 21 constituents. However, numerical objectives for the nine trace metals listed are based on the hardness (as $CaCO_3$) found in the receiving water. These objectives will therefore differ between water bodies. The hardness of a given water body, like other constituents, varies over time. A baseline hardness for the river at Captain's Table dock, upstream of CWTP effluent discharge, was chosen to represent the Sacramento River during this storm event. Analytical results of river water collected from the Captains' Table location showed a hardness concentration of 56.1 mg/l. This value in conjunction with the river water's pH of 7.6, was used to derive the water quality objective concentrations.

B.2. Pollutant Concentration Objectives

The objectives listed in the *Inland Surface Waters Plan* were converted to pollutant concentrations to which the river sampling results were compared. These are listed in Table 4-1, including the objectives for specific organics. Equations for calculating concentration can be found on Page 4 of the *Inland Surface Waters Plan* document.

Although the concentration objectives are applied to the Sacramento River sample results, discharges to the river cannot further degrade water quality when the river is at the objective concentration. If, for instance, the river concentration for a trace metal reaches the objective concentration, then the concentration of that metal in the discharge must be at or below the limit.

It is important to note that the aquatic life water quality objectives for cadmium, chromium, copper, lead, nickel and zinc are based on total recoverable (or "recoverable") concentrations rather than "total" metal concentration. Total recoverable concentration is theoretically lower than total concentration for trace metals. The analysis results presented in Section 5, however, compares all

metal concentration results for the sample analyses, including total, recoverable and dissolved, to the objective concentrations listed in Table 4-1.

TABLE 4-1. WATER QUALITY OBJECTIVES FOR PROTECTION OF FRESHWATER AQUATIC LIFE FOR COMPARISON TO DECEMBER 7, 1991 SAMPLING EVENT RESULTS

Values used in computations:
Hardness, in mg/l 56.1 (Sacramento River at Captain's Table dock)
pH 7.6

Constituent	Units	Concentration Objectives			
		4-Day Average	Daily Average	1-Hour Average	Instantaneous Maximum
arsenic	ug/l	190.0	-----	360.0	-----
cadmium	ug/l	0.7	-----	2.0	-----
chlordane	ng/l	-----	4.3		-----
chromium(VI)	ug/l	11.0	-----	16.0	-----
copper	ug/l	7.2	-----	10.3	-----
DDT	ng/l	-----	1.0	-----	-----
dieldrin	ng/l	-----	1.9	-----	-----
endosulfan	ng/l	-----	56.0	-----	220.0
endrin	ng/l	-----	2.3	-----	180.0
heptachlor	ng/l	-----	3.8	-----	-----
hexachlorocyclo-	ng/l	-----	80.0	-----	-----
hexanegamma		-----	-----	-----	-----
lead	ug/l	1.5	-----	39.1	-----
mercury	ug/l	-----	-----	2.4	-----
nickel	ug/l	96.7	-----	869.7	-----
PCBs	ng/l	-----	14.0	-----	-----
pentachlorophenol	ng/l	10.5	-----	16.6	-----
selenium	ug/l	5.0	-----	20.0	-----
silver	ug/l	-----	-----	-----	1.5
toxaphene	ng/l	0.2	-----	730.0	-----
tributyltin	ng/l	20.0	40.0	-----	60.0
zinc	ug/l	64.9	-----	71.7	-----

NOTES:

mg/l = milligram(s) per liter; ug/l = micrograms per liter; ng/l = nanograms per liter

Objectives for cadmium, chromium, copper, lead, nickel, silver and zinc are based on total recoverable metal concentration.

Dischargers may, at their option, meet the total chromium limitation.

The 4-day average for tributyltin is the six month median.

Source: California Inland Surface Waters Plan, Water Resources Control Board, April 1991.

SECTION 5
WATER QUALITY SAMPLE RESULTS

This section summarizes the laboratory results for the storm and sampling event of December 7, 1991, and presents a comparison of water quality for CWTP effluent and the Sacramento River to the *Inland Surface Waters Plan* objective concentrations. Results for the chronic biotoxicity tests are presented in Section 6.

A. LABORATORY RESULTS

The detailed laboratory analysis data for this event are presented in Appendices A and B, separate from this report. Summaries of the laboratory results for detectable analytes are presented according to sampling location in Tables 5-1 and 5-2. Although samples were analyzed for all constituents listed in Table 2-2, only the detected constituents are included in the results tables. Table 5-1 presents the analysis results for CWTP effluent ("CWTP"), and Table 5-2 presents the results for dilution water collected from the Sacramento River at Captain's Table dock ("Dilution"). Results for Method 608 analyses (organochlorine pesticides and PCBs) are presented separately in Tables 5-3 and 5-4.

A.1. General Inorganics

The analyses of general inorganics constituents for the CWTP effluent sample resulted in concentrations above the reporting limit for all constituents except dissolved phosphorus. As expected, the CWTP effluent constituents were detected at higher concentrations than the river water collected at Captain's Table dock. The pH of the CWTP effluent was found to be slightly more acidic than the sampled river water. Measured hardness as calcium carbonate ($NaCO_3$) in CWTP effluent was 49.5 mg/l, compared to river water which was measured at 56.1 mg/l.

A.2. Bacteria

Bacteria counts from the river water sample collected at Captain's Table dock showed the presence of 13,000 MPN/100 ml of total coliform, and 1,300 MPN/100 ml

of fecal coliform at a reporting limit of 2 MPN/100 ml. Fecal streptococcus were present at 140 MPN/100 ml, using a reporting limit of 20 MPN/100 ml.

Bacteria counts from the effluent water sample collected at CWTP showed no detectable concentrations of fecal coliform or fecal streptococcus at a reporting limit of 20 MPN/100 ml. Total coliform were measured at a concentration of 20 MPN/100 ml, using a reporting limit of 20 MPN/100 ml. The samples for the total and fecal coliform analyses were diluted by the laboratory upon arrival. The laboratory suspected high concentrations of bacteria due to the high suspended solid content and high turbidity of the samples. As a result of the dilution, the laboratory raised the reporting limit from 2 MPN/100 ml to 20 MPN/100 ml. However, upon analysis, the sample was found to have a bacterial count lower than the 20 MPN/100 ml reporting limit.

Bacteria counts at CWTP were considerably lower than in the Sacramento River, which reflects the effective chlorine treatment at the facility.

A.3. Organochlorine Pesticides/PCBs

As described in the previous sections, a CWTP effluent sample and a Sacramento River dilution water sample (for preparing effluent dilutions) were collected for analysis of organochlorine pesticides and PCBs. This was the only event in which samples were collected directly from the river for this type of analysis. Tables 5-3 and 5-4 present the results and a comparison to objectives in the *Inland Surface Waters Plan*. It is important to note that, unlike the trace metals, objective concentrations are mostly only available for daily average values, as listed in Table 4-1.

A.3.a. Discharge and River Results--

None of the analyzed constituents were detected in either CWTP effluent or the Sacramento River water, using the reporting limits presented in Appendix B, which are commensurate with a Method 608 screening analysis. As noted in Section 3, all of the organochlorine pesticide/PCB reporting limits for the CWTP effluent sample were raised due to matrix interferences. The river water sample analysis was not similarly affected.

For some of the Method 608 analysis constituents, conclusions can be made relative to meeting the *Inland Surface Waters Plan* objective concentrations directly from the analysis results, without considering dilution factors.

Table 5-4 presents the results for the Sacramento River sample analysis. The instantaneous maximum objective concentrations for endrin and endosulfan are 180 ng/l and 220 ng/l, respectively. Based on the "ND" result for a reporting limit of 100 ng/l and 50 ng/l for the river sample, it is concluded that these constituents are less than the objective concentration. Similarly, the concentrations for endosulfan I and hexachlorocyclo-hexanegamma (also Lindane or gamma-BHC) in the river was less than the daily average objective concentration, based on a reporting limit of 50 ng/l. For the CWTP effluent, the results are not conclusive, since the reporting limits were raised.

A.3.b. Estimated River Results--

The organochlorine pesticide/PCB screen analysis was performed on the effluent discharge sample in order to evaluate a "worst-case" condition. However, the reportable and detectable concentrations that would have occurred in the river can be approximated from the discharge sample and dilution factor calculated in Section 3. For these estimation purposes, dilution factor was rounded to 75:1 for the CWTP discharge to the river. The results are presented in Table 5-3.

For instance, heptachlor was not detected in the CWTP discharge, using a reporting limit of 0.25 μg/l. Using the dilution ratio, the reporting limit for the downstream river section would be 3.3 ng/l. At this reporting limit, the result would be "ND". Neither a 4-day average nor 1-hour average objective concentration is available for heptachlor; the daily average value is 3.3 ng/l. Therefore, for this constituent, the estimated river concentration downstream of the CWTP outfall would be less than the daily average objective concentration.

As indicated by the Table 5-3, many of these compounds would not have been detected in the river as compared to the daily average and instantaneous objective concentrations in the *Inland Surface Waters Plan.* For the majority of constituents, however, conclusions cannot be made due to the low values of the objectives and the raising of reporting limits bacause of matrix interferences.

One-hour and 4-day objective concentrations are provided for toxaphene. Results for the CWTP effluent sample was "ND". However, conclusions relative to Table 4-1 objectives cannot be determined for either the sample or the estimated "diluted" river concentration, since the reporting limit is higher.

A.4. Oil and Grease

Oil and grease concentrations were not detected in samples from the Sacramento River. Oil and grease concentrations were detected at 28 mg/l in CWTP effluent. However, as previously mentioned in Section 3, the RPD goal for this analysis was exceeded, which may influence the precision of this analytical result.

A.5. Trace Metals

The analytical results for trace metals in the Captain's Table river sample collected upstream of the CWTP effluent discharge showed detectable concentrations for 3 of the 9 total metal analytes, 2 of the 8 recoverable metal analytes, and 3 of the 9 dissolved metal analytes as shown in Table 5-1. However, the total chromium analytical results may have been influenced by contamination from laboratory sample preparation. This metal was found in the quality assurance sample blanks, as described in Section 3.

The analytical results for trace metals in the CWTP effluent sample showed detectable concentrations for 8 of the 9 total metal analytes, 6 of the 8 recoverable metal analytes, and 7 of the 9 dissolved metal analytes as shown in Table 5-2. However, the total chromium and copper analytical results may have been influenced by contamination from laboratory sample preparation. These metals were found in the quality assurance sample blanks, as described in Section 3. The reporting limit for the dissolved mercury analysis was adjusted due to sample volume restrictions, also as described in Section 3.

In addition, the reporting limits for two analyses were raised due to matrix interferences. The reporting limit for recoverable arsenic was raised from 0.0050 mg/l to 0.010 mg/l. Using this reporting limit, no detectable concentration of arsenic was found in the sample. The reporting limit for recoverable silver was

raised from 0.00010 mg/l to 0.00030 mg/l. Using this reporting limit, silver was detected at a concentration of 0.0067 mg/l in the sample.

A comparison of trace metal concentrations in the Sacramento River and CWTP effluent yields, for the most part, a trend of higher analyte concentrations in the effluent than in the river water samples. Dissolved arsenic is the exception to this trend. Dissolved arsenic concentrations were measured as 0.0027 mg/l in the Sacramento River sample, while dissolved arsenic in the CWTP effluent was detected at 0.0025 mg/l.

A comparison of the total, recoverable and dissolved metal concentrations detected in the Sacramento River to the 1-hour and 4-day objective concentrations listed in Table 4-1 shows that no metals exceeded the objectives.

A comparison of the total and recoverable metal concentrations in the CWTP effluent to the 1-hour and 4-day objective shows that several metals exceeded the objectives. Total and recoverable chromium, copper, and zinc exceeded both of these objective concentrations. In addition, total and recoverable cadmium and lead exceeded the 4-day objective. Total and recoverable silver concentrations exceeded the instantaneous maximum objective.

Dissolved copper and zinc exceeded both the 1-hour and 4-day objective concentrations. In addition, dissolved lead exceeded the 4-day objective. Dissolved silver concentrations exceeded the instantaneous maximum objective.

A.6. Quality Control Results

The laboratory split the Sacramento River water samples collected at Captain's Table dock into two batches and were analyzed separately (i.e. duplicates) as a part of the project's quality control procedures. Table 5-5 compares and notes variations between the parent sample results and the quality control (sample split) results. The quality control results for the analysis compared favorably with those of their parent sample.

TABLE 5-1. RESULTS FOR SACRAMENTO RIVER AT CAPTAIN'S TABLE FOR DECEMBER 7, 1991 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
General Inorganics					
Nitrate as N	mg/l	.05	0.10		
Total Dissolved Solids	mg/l	10.0	95.0		
Total Suspended Solids	mg/l	5.0	6.0		
pH	units	N/A	7.6		
Turbidity	NTU	1.0	5.8		
Hardness	mg/l	3.3	56.1		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0023	0.360	0.190
Chromium	mg/l	.0005	0.0025	0.0160	0.0110
Copper	mg/l	.0005	0.002120	0.0103	0.0072
Trace Metals (Recoverable)					
Chromium	mg/l	.0005	0.0020	0.0160	0.0110
Copper	mg/l	.0005	0.0017	0.0103	0.0072
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0027	0.360	0.190
Chromium	mg/l	.0005	0.00063	0.0160	0.110
Copper	mg/l	.0005	0.0017	0.0103	0.0072

TABLE 5-1. RESULTS FOR SACRAMENTO RIVER AT CAPTAIN'S TABLE FOR DECEMBER 7, 1991 EVENT					
Constituents	**Units**	**Reporting Limit**	**Sample Results**	**Objectives[1] (1-Hour Average)**	**Objectives[1] (4-Day Average)**
Bacteria					
Total Coliform	MPN/100ml	2	13,000		
Fecal Coliform	MPN/100ml	2	1,300		
Fecal Streptococcus	MPN/100ml	20	140		

1 Objective concentrations for trace metals based on recoverable concentrations.

TABLE 5-2. RESULTS FOR CWTP EFFLUENT FOR DECEMBER 7, 1991 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[3] (1-Hour Average)	Objectives[3] (4-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	50	183		
Biological Oxygen Demand	mg/l	12	180		
Nitrate as N	mg/l	.5	1.2		
Total Kjeldahl Nitrogen as N	mg/l	.50	10.1		
Phosphorus, Total as P	mg/l	.05	0.37		
Total Dissolved Solids	mg/l	10.0	302		
Total Suspended Solids	mg/l	5.0	122		
pH	units	N/A	6.2		
Turbidity	NTU	10.0	87.3		
Hardness	mg/l	3.3	49.5		
Oil and Grease	mg/l	.20	28		
Trace Metals (Total)					
Arsenic	mg/l	.0010	0.0034	0.360	0.190
Cadmium	mg/l	.0004	0.0012	0.0020	0.0007
Chromium	mg/l	.0010	0.016	0.0160	0.0110
Copper	mg/l	.0025	0.045	0.0103	0.0072
Lead	mg/l	.0017	0.036	0.0391	0.0015
Mercury	mg/l	.00020	0.00034	0.0024	-[2]
Silver	mg/l	.00030	0.0060	0.0015[1]	0.0015[1]
Zinc	mg/l	.0085	0.28	0.0717	0.0649
Trace Metals (Recoverable)					
Cadmium	mg/l	.0004	0.0011	0.0020	0.0007
Chromium	mg/l	.0010	0.016	0.0160	0.0110
Copper	mg/l	.0025	0.043	0.0103	0.0072
Lead	mg/l	.0017	0.031	0.0391	0.0015
Silver	mg/l	.0003	0.0067	0.0015[1]	0.0015[1]

TABLE 5-2. RESULTS FOR CWTP EFFLUENT FOR DECEMBER 7, 1991 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[3] (1-Hour Average)	Objectives[3] (4-Day Average)
Zinc	mg/l	.0085	0.25	0.0717	0.0649
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0025	0.360	0.190
Cadmium	mg/l	.0004	0.00061	0.0020	0.0007
Chromium	mg/l	.0005	0.0027	0.0160	0.0110
Copper	mg/l	.0010	0.024	0.0103	0.0072
Lead	mg/l	.0017	0.0041	0.0391	0.0015
Silver	mg/l	.0001	0.0018	0.0015[1]	0.0015[1]
Zinc	mg/l	.0085	0.22	0.0717	0.0649
Bacteria					
Total Coliform	MPN/100ml	20	20		
Fecal Coliform	MPN/100ml	20	<20		
Fecal Streptococcus	MPN/100ml	20	<20		

1 The objective concentration for silver is calculated as the instantaneous maximum.

2 No standard available.

3 Objective concentrations for trace metals based on recoverable concentration.

TABLE 5-3. ORGANOCHLORINE PESTICIDE/PCB SUMMARY AND COMPARISON[4] FOR DILUTED CWTP EFFLUENT					
Constituent	Reporting Limit and "ND" Concentration[1] (ng/l)[3]	Objective Concentrations (ng/l)[3]		Exceedence Summary for Objective Concentrations[2]	
		Daily Average	Instantaneous Maximum	Daily Average	Instantaneous Maximum
chlordane	33.3	4.3	-----	U	-----
DDT[5]	6.7	1.0	-----	U	-----
dieldrin	3.3	1.9	-----	U	-----
endosulfan	6.7[6]	56.0	220.0	N	N
endrin	3.3	2.3	180.0	U	N
heptachlor	3.3	3.8	-----	N	-----
hexachlorocyclo-hexanegamma	8.4	80.0	-----	N	-----
PCBs	66.7[7]	14.0	-----	U	-----

1 For Sacramento River downstream of CWTP outfall, assumes 73:1 dilution of CWTP effluent sample reporting limit, for which all Method 608 screen results were "ND".

2 Key: N = ND value for reporting limit is less than the objective concentration.
U = Unknown as to whether objective concentration would be exceeded.

3 ng/l = nanograms per liter or parts per trillion.

4 See Table 4-1 analytes.

5 Results identical for DDD. DDE estimate is 3.3 ng/l.

6 Estimate is for endosulfan II and endosulfan sulfate. Endosulfan I estimate is 3.3 ng/l.

7 PCBs consist of 7 compounds.

TABLE 5-4. ORGANOCHLORINE PESTICIDE/PCB SUMMARY AND COMPARISON[4] FOR SACRAMENTO RIVER SAMPLE					
Constituent	Reporting Limit and "ND" Concentration[1] (ng/l)[3]	Objective Concentrations (ng/l)[3]		Exceedence Summary for Objective Concentrations[2]	
		Daily Average	Instantaneous Maximum	Daily Average	Instantaneous Maximum
chlordane	50	4.3	-----	U	-----
DDT[5]	100	1.0	-----	U	-----
dieldrin	100	1.9	-----	U	-----
endosulfan	50[6]	56.0	220.0	N	N
endrin	100	2.3	180.0	U	N
heptachlor	50	3.8	-----	U	-----
hexachlorocyclo-hexanegamma	50	80.0	-----	N	-----
PCBs	500[7]	14.0	-----	U	-----

1 For Sacramento River upstream of CWTP outfall; assumes no dilution, since sample collected directly from the river.

2 Key: N = ND value is less than the objective concentration.
U = Unknown as to whether objective concentration would be exceeded.

3 ng/l = nanograms per liter or parts per trillion.

4 See Table 4-1 analytes.

5 Results identical for DDD and DDE.

6 Estimate is for endosulfan I. Endosulfan II and endosulfan sulfate estimates are 100 ng/l.

7 PCBs consist of 7 compounds. Estimate is for Aroclor 1016, 1221, 1232, 1242 and 1248; Aroclor 1254 and 1260 have 1000 ng/l reporting limit.

TABLE 5-5. QUALITY CONTROL RESULTS FOR DECEMBER 7, 1991 EVENT

Constituents	Units	Reporting Limit	Parent Sample Results	QC Sample Results	Variance[1]
General Inorganics					
Biological Oxygen Demand	mg/l	3.0	ND	5.0	42.0%[2]
Oil and Grease	mg/l	0.20	ND	ND	NC
Hardness	mg/l	3.3	56.1	52.7	6.1%
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0023	0.0022	4.3%
Cadmium	mg/l	.0004	ND	ND	NC
Chromium	mg/l	.0005	0.0025	0.0018	28.0%
Copper	mg/l	.0005	0.00212	0.00219	3.3%
Lead	mg/l	.0017	ND	ND	NC
Mercury	mg/l	.0002	ND	ND	NC
Selenium	mg/l	.001	ND	ND	NC
Silver	mg/l	.0001	ND	ND	NC
Zinc	mg/l	.0085	ND	ND	NC
Trace Metals (Recoverable)					
Arsenic	mg/l	.005	ND	ND	NC
Cadmium	mg/l	.0004	ND	ND	NC
Chromium	mg/l	.0005	0.0020	0.0020	0%
Copper	mg/l	.0005	0.0017	0.0017	0%
Lead	mg/l	.0017	ND	ND	NC
Selenium	mg/l	.005	ND	ND	NC
Silver	mg/l	.0001	ND	ND	NC
Zinc	mg/l	.0085	ND	ND	NC
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0027	0.0026	3.7%
Cadmium	mg/l	.0004	ND	ND	NC
Chromium	mg/l	.0005	0.00063	0.00060	0.5%

TABLE 5-5. QUALITY CONTROL RESULTS FOR DECEMBER 7, 1991 EVENT

Constituents	Units	Reporting Limit	Parent Sample Results	QC Sample Results	Variance[1]
Copper	mg/l	.0005	0.0017	0.0014	17.6%
Lead	mg/l	.0017	ND	ND	NC
Mercury	mg/l	.0002	ND	ND	NC
Selenium	mg/l	.001	ND	ND	NC
Silver	mg/l	.0001	ND	ND	NC
Zinc	mg/l	.0085	ND	ND	NC
Bacteria					
Total Coliform	MPN/100ml	2	13,000	16,000	23.1%
Fecal Coliform	MPN/100ml	2	1,300	700	46.2%
Fecal Streptococcus	MPN/100ml	20	140	220	57.1%

1 Calculation for variance is as follows: absolute value of the difference between the parent and QC sample results, divided by the parent sample result, expressed as a percentage.
2 Calculation of variance is based on a parent sample value of 2.9 mg/l.
ND Not detected.
NC Variance cannot be calculated for this pair of analytical results.

SECTION 6
CHRONIC TOXICITY SAMPLE RESULTS

This section summarizes the toxicity sampling and analysis program, sample test results, and quality assurance/quality control results for the storm event of December 7, 1991.

A. BIOTOXICITY TESTING PROGRAM OVERVIEW

Provision 5 of the City's NPDES Permit requires that a biotoxicity study be completed for CWCS discharges, using 100-percent solutions of CWTP effluent and CSO discharges and the EPA's three-species test for chronic toxicity. If chronic toxicity was found in the 100-percent samples collected during the 1991/1992 wet weather season, then the City would be required perform a dilution series analysis for the corresponding sampling locations during the 1992/1993 wet season.

During this storm event, chronic toxicity analyses were performed on a 100-percent solution of CWTP effluent. Neither Pioneer Reservoir or Sump 2 overflowed during this event, so sample material was not collected for these discharge.

The City voluntarily accelerated the toxicity testing program on CWTP effluent. During this CSO event, the City performed a chronic toxicity dilution series of CWTP effluent. Dilutions were prepared for the following effluent percentages in accordance with EPA protocols: 100, 50, 25, 12.5, 6.2. These dilutions of the CWTP effluent were analyzed to define the No Observable Effect Concentration (NOEC) and the Lowest Observable Effect Concentration (LOEC) points for each of the biological test species. The dilution series was created by proportioning CWTP effluent and Sacramento River water (collected at Captain's Table dock during the storm event and prior to upstream discharges). In order to determine baseline chronic toxicity for the CWTP dilution series, 100-percent river water was also analyzed for toxicity.

The following tests were performed on the 100-percent effluent discharge and Sacramento river samples, and on the CWTP effluent dilutions:

1. A 7-day, chronic fathead minnow (*Pimephales promelas*), static renewal, larval survival and growth test.
2. A 3-brood, 7-day, chronic cladoceran (*Ceriodaphnia dubia*), static renewal, survival and reproduction test.
3. A 4-day, chronic, algal (*Selenastrum capricornutum*), static, growth test.

Procedures for determining biotoxicity are defined in EPA 600/4-89/001 (*Short-term Methods for Estimating the Chronic Toxicity of Effluent and Receiving Water to Freshwater Organisms*) and EPA 440/4-85/032 (*Technical Support Document for Water Quality Based Toxic Control*). The EPA's three-species freshwater chronic toxicity tests were performed as specified in these protocals and NPDES permit. Methods numbers are summarized in Table 6-1.

TABLE 6-1. CHRONIC TOXICITY TEST PARAMETERS AND METHOD NUMBERS

Test Specie	Type of Test	EPA Method
Fathead Minnow	Survival/Growth	1000
Ceriodaphnia (water flea)	Survival/Reproduction	1002
Selenastrum (algae)	Growth	1003

B. SAMPLE COLLECTION AND HANDLING

Toxicity sample collection for the CWTP effluent discharge was conducted similar to the inorganic constituent sample collection. Discrete samples were composited in 30-minute periods, such that each period throughout the discharge would be represented by an equal volume. The sample collected from the river was collected

as a grab. The samples were refrigerated during transport to Aquatic Testing Laboratories in Ventura. All tests were initiated within the required holding time limits.

C. ANALYTICAL RESULTS

Aquatic Testing Laboratories analyzed the biotoxicity of the effluent in the dilution percentages as previously described. Data collected from laboratory test results were entered into a data base for statistical analysis. EPA's TOXSTAT computer program was used to conduct statistical tests and to estimate the NOEC and LOEC endpoints. The three-species freshwater chronic toxicity test results are summarized in the following sections and tables.

C.1. Sacramento River Sample Results

The chronic toxicity of Sacramento River water was tested to provide baseline data for the CWTP dilution series tests. Test results were compared to the laboratory control for determining whether chronic toxicity was present in the river during this storm event at the Captain's Table dock location that could influence the results of the dilution series tests. Fathead minnow survival and growth was not significantly adversely affected by the 100-percent solution of Sacramento River water collected at Captain's Table dock, as shown in Tables 6-2 and 6-3. It was therefore concluded that the river sample was not chronically toxic to any of the three species. The river water was then used as the control for analyzing the CWTP dilution series results.

TABLE 6-2. CHRONIC TOXICITY OF SACRAMENTO RIVER WATER ON FATHEAD MINNOWS

Sample	Percent Survival	Larval Mean Dry Weight
Control-Lab	96.9%	0.67 mg
Sacramento River	97.5%	0.65 mg

Control-Lab is a laboratory control water (20% DMW).

Sacramento River water collected from the Captain's Table dock.

TABLE 6-3. SUMMARY OF SACRAMENTO RIVER CHRONIC TOXICITY ENDPOINTS FOR FATHEAD MINNOWS		
Endpoint	**Survival**	**Growth**
NOEC	NS	NS
NS is not significantly less than control.		

C.2. CWTP Effluent Sample Results

Tables 6-4 and 6-5 show that fathead minnow survival was siginificantly affected in a 100-percent effluent solution, therefore, a survival NOEC of 50-percent effluent was determined. The analysis also showed that in a 6.2-percent effluent solution, fathead minnow reproduction was affected. Since the 6.2-percent solution was the smallest dilution analyzed, a definitive NOEC point below this dilution could not be accurately determined. Therefore, it is estimated that the 6.2-percent effluent sample was chronically toxic to fathead minnows.

Ceriodaphnia survival and growth were both significantly affected in a 50-percent effluent solution, therefore, survival and growth NOECs at 25-percent effluent were determined for this specie. The results are shown in Tables 6-6 and 6-7. Based on the these results, the 50-percent effluent event sample was chronically toxic to *Ceriodaphnia*.

Selenastrum growth rates were not significantly affected in a 100-percent solution, therefore, the NOEC was determined at 100-percent for this specie, as shown in Table 6-8.

TABLE 6-4. CHRONIC TOXICITY OF CWTP EFFLUENT ON FATHEAD MINNOWS		
Sample Concentration	Percent Survival	Larval Mean Dry Weight
Control (river water)	97.5%	0.65 mg
6.2% Effluent	97.5%	0.56 mg*
12.5% Effluent	97.5%	0.55 mg*
25.0% Effluent	95.0%	0.53 mg*
50.0% Effluent	87.5%	0.52 mg*
100.0% Effluent	0%*	NA

Control is the dilution water control collected from the river at the Captain's Table dock.

NA is not applicable.

* Significantly lower than control.

TABLE 6-5. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY ENDPOINTS FOR FATHEAD MINNOWS		
Endpoints	Survival	Growth
NOEC	50.0%	<6.2%
LOEC	100%	6.2%

TABLE 6-6. CHRONIC TOXICITY OF CWTP EFFLUENT ON CERIODAPHNIA		
Sample	Percent Survival	Mean Number of Young per Female
Control	100.0%	25.0
6.2% Effluent	90.0%	19.7
12.5% Effluent	100.0%	16.0
25.0% Effluent	90.0%	18.3
50.0% Effluent	30.0%	1.1*
100.0% Effluent	0%*	0*

Control is the dilution water control collected from the river at the Captain's Table dock.

* Significantly less than control.

TABLE 6-7. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY ENDPOINTS FOR CERIODAPHNIA		
Endpoint	Survival	Reproduction
NOEC	25.0%	25.0%
LOEC	50.0%	50.0%

TABLE 6-8. CHRONIC TOXICITY OF CWTP EFFLUENT ON SELENASTRUM	
Effluent Concentration	Mean Fluorescence
Control	227.8
6.2% Effluent	219.8
12.5% Effluent	438.2
25.0% Effluent	864.5
50.0% Effluent	518.2
100.0% Effluent	517.5

Relative fluorescence values are in-vivo relative content measurements taken with a fluorometer equipped with a chlorophyll filter.

Fluorescence value of 200 is approximately 270,000 cells/ml based on actual cell counts of the control test solution.

Control is dilution water collected from the river at the Captain's Table dock.

TABLE 6-9. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY RESULTS FOR SELENASTRUM	
NOEC = 100.0%	LOEC = NA

NA is not applicable

D. QUALITY ASSURANCE/QUALITY CONTROL

The three test species were tested with reference toxicants to determine whether the cultured species exhibited normal toxicity. Reference toxicant data are included in Appendix B, submitted as a separate document. Fathead minnows were tested with a reference toxicant of sodium dodecyl sulfate (SDS). The SDS NOECs for fathead minnow survival and growth were 1.2 mg/l and 2.5 mg/l, respectively.

Ceriodaphnia were tested with a reference toxicant of sodium chloride (NaCl). The NaCl NOEC point for Ceriodaphnia was 1,000 mg/l for both survival and reproduction. Selenastrum were tested with a reference toxicant of copper sulfate. The chronic toxicity NOEC point for Selenastrum was 0.010 mg/l. The reference toxicant analyses indicated that the three species exhibited a "normal" toxic response to their respective toxicants.

The control solution used for the biotoxicity analysis was Sacramento River water collected from Captain's Table dock during this CSO event. None of the species used in the analysis were adversely effected by conditions in the control sample waters.

SECTION 7
SUMMARY

This section provides a summary of the results for the December 7, 1991 storm event. During this event, CWTP discharged effluent to the Sacramento River.

A. OBJECTIVES MET BY THE CSO EVENT

Based on the goals established for the 1991/1992 CWCS Sampling Program, the following objectives were met by the CSO event:

1) The chronic toxicity of Sacramento River water and CWTP effluent was determined for event samples on three test species (fathead minnows, *Ceriodaphnia*, and *Selenastrum*). These test samples were undiluted, i.e. 100 percent river water or discharge samples. Toxicity tests for dilutions of the CWTP effluent were conducted for the following percentages of effluent: 100, 50, 25, 12.5 and 6.2. River water was used for preparing the dilution series test samples. The objective was to determine the significant impact, from a statistical standpoint, of sample waters on each of the three species on critical life stages. The NOEC and LOEC represent this statistical significance.

2) The concentrations of pollutants (inorganics, bacteria and metals) discharged to the Sacramento River from CWTP were measured by collecting and analyzing samples of these matrices.

3) The presence of organochlorine pesticides and PCBs in CWTP effluent and river water samples was measured by collecting and analyzing samples during this CSO event.

The primary objective of sampling this early-season CWTP "operation only" event was to provide three sets of chronic toxicity test data for CWTP effluent for the 1991/1992 wet season. The other two sets (one being provided by this event) would be provided by the two CSO events sampled later in the season. In addition, it

was intended that sampling this early-season event, along with test results from later-season events, would allow for an evaluation of seasonal variation, if any, in the chronic toxicity of CWTP effluent. It was also intended that the assessment of impacts to the Sacramento River from CWCS discharges, including CWTP effluent, would be conducted during the two storm/CSO events captured during the 1991/1992 wet season. This approach, as identified in the Phase 3 Scope of Work, is identical to that implemented during the previous, 1990/1991 wet season; two storm/CSO events were sampled to evaluated potential river impacts following discharges. For these reasons, samples were not collected from the river downstream of the CWTP outfall.

B. SUMMARY

B.1. Chronic Toxicity

EPA's three species, freshwater chronic toxicity tests were used as specified in the NPDES permit. EPA's TOXSTAT computer program was used to perform the statistical computations on the laboratory results.

B.1.a. Sacramento River Sample--

An analysis of biotoxicity of the Sacramento River water sample was conducted to determine a baseline for the control waters used in the CWTP effluent dilution series. Fathead minnow survival and growth were not significantly affected in a 100-percent Sacramento River water sample collected at the Captain's Table dock. Therefore, chronic toxicity was not present. The river water therefore served as the control for evaluating CWTP dilution series results.

B.1.c. CWTP Effluent Sample--

The analysis showed that fathead minnow reproduction was affected in a 6.2-percent effluent solution. Since the 6.2-percent solution was the smallest dilution series run, the NOEC for this event was between the 0 and 6.2 percent effluent dilutions.

Ceriodaphnia survival and reproduction was significantly affected in a 50-percent effluent solution. Therefore, an NOEC for both survival and reproduction was

determined to be at the 25-percent effluent solution.

Selenastrum was not significantly affected by the effluent. The NOEC for *Selenastrum* growth was in 100-percent effluent.

The safe concentration for these particular samples, species and critical life stages is between the NOEC and LOEC. This concentration is biologically rather than statistically significant. Assuming fathead minnows to be the most sensitive specie, a minimum of 16:1 river dilution would be needed, conservatively, to ensure normal propagation. During this event, the river provided approximately 75:1 dilution for CWTP effluent.

B.2. General Inorganics

The CWTP effluent analytes were detected at higher concentrations than the Sacramento River water collected at Captain's Table dock. The pH of the CWTP effluent was found to be slightly more acidic than the sampled river water. Measured hardness as calcium carbonate in CWTP effluent was 49.5 mg/l, as compared to river water which was 56.1 mg/l.

B.3 Bacteria

Bacteria counts at CWTP were considerably lower than in the Sacramento River which reflects the CWTP's effective chlorination treatment.

B.4. Organochlorine Pesticides/PCBs

None of the analyzed constituents were detected in either CWTP effluent or Sacramento River water.

B.5. Oil and Grease

Oil and grease were not detected in samples from the Sacramento River, but were detected at 28 mg/l in CWTP effluent.

B.6. Trace Metals

A comparison of metal concentrations in the Sacramento River and CWTP effluent yields, for the most part, a trend of higher analyte concentrations in the effluent than in the river water samples. Dissolved arsenic is the exception to this trend.

A comparison of the total, recoverable and dissolved metal concentrations detected in the Sacramento River to the 1-hour and 4-day objective concentrations listed in Table 4-1 shows that no metals exceeded the objectives.

A comparison of the total and recoverable metal concentrations in the CWTP effluent to the 1-hour and 4-day objective shows that several metals exceeded the objectives. Total and recoverable chromium, copper, and zinc exceeded both of these objective concentrations. In addition, total and recoverable cadmium and lead exceeded the 4-day objective. Total and recoverable silver concentrations exceeded the instantaneous maximum objective.

Dissolved copper and zinc exceeded both the 1-hour and 4-day objective concentrations. In addition, dissolved lead exceeded the 4-day objective. Dissolved silver concentrations exceeded the instantaneous maximum objective.

APPENDIX C

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 4

EVENT REPORT FOR THE FEBRUARY 12, 1992, SAMPLING EVENT

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September 1, 1992

CITY OF SACRAMENTO
COMBINED SEWER SYSTEM
IMPROVEMENT AND IMPLEMENTATION PLAN

PHASE 3 STUDY

CWCS SAMPLING PROGRAM FOR 1991/1992

WATER QUALITY AND TOXICITY RESULTS FOR THE FEBRUARY 12, 1992 SAMPLING EVENT

Prepared for

City of Sacramento
Utilities Department
Engineering Division

Prepared by

HDR Engineering, Inc.
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Sacramento, CA

September 1, 1992

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SECTION 1
PROGRAM OVERVIEW AND OBJECTIVES

A. INTRODUCTION

This report summarizes the results for the February 12, 1992 sampling event. As background for reviewing these results, this section provides a brief overview of the sampling and analysis plan for the Combined Wastewater Collection and Treatment System (CWCS) and Sacramento River for the 1991/1992 wet season. This program parallels, but has been conducted separately from the "routine" monitoring program specified in the City's NPDES Permit No. CA0079111 for the CWCS. This program was conducted as part of the Phase 3 study to develop an improvement and implementation plan for the Sacramento combined sewer system.

Four key elements were used to develop a water quality sampling program for the 1991/1992 wet weather season: 1) the results of the 1990/1991 wet weather sampling for two storm/combined sewer overflow (CSO) events; 2) the NPDES permit for the CWCS, which requires that biotoxicity tests be performed on CWCS "effluent" samples, including Combined Wastewater Treatment Plant (CWTP) effluent and CSOs from Pioneer Reservoir and Sump 2; 3) a request by the RWQCB staff to conduct a matrix detection limit study for lead; and 4) a request by the RWQCB to include a screen for the pesticides and PCBs included in the *California Inland Surface Waters Plan*, "Objectives for Protection of Freshwater Aquatic Life". (See Section 4 for details on these objective concentrations.)

B. OBJECTIVES

Based on the elements described above, the objectives for the 1991/1992 CWCS Sampling Program were as follows:

1) Determine the chronic toxicity of CWTP effluent, Sump 2 overflows and Pioneer Reservoir overflows. Determine the seasonal variation of toxicity, if possible, by sampling an early-season CWTP "operation-only" event, as well as two additional storm events that are large enough to both require

operation of CWTP and produce CSOs to the Sacramento River. The objective was to sample CSOs from each of the outfall locations, even if overflows were not necessarily simultaneous at Sump 2 and Pioneer Reservoir. Determine the No Observable Effect Concentration (NOEC) and Lowest Observable Effect Concentration (LOEC) for CWTP effluent by performing toxicity tests on dilutions of effluent for all three of the events sampled. For the CSO samples (Sump 2 and Pioneer Reservoir), determine toxicity on a "screening level" basis, by testing the undiluted (100-percent) samples. If toxicity was found in the 100-percent CSO samples, then dilution series chronic tests would be performed during the subsequent 1992/1993 wet season.

2) Sample local street floodwater and analyze for bacteria content, in order to estimate the source of bacteria contamination and, if possible, to identify locations having consistent sewage outflows onto streets during storm events. It was intended that City staff would complete the work related to this objective.

3) Determine the impact of trace metal pollutant discharges on the Sacramento River. Collect and analyze samples from the Sacramento River at appropriate locations during two storm events large enough to require CWTP operation and overflows from Pioneer Reservoir and/or Sump 2. If overflows from a particular event did not occur simultaneously at Sump 2 and Pioneer Reservoir, samples would be collected for the one CSO location. (Ultimately, the objective was to capture two CSO's at each location.) The sampling related to CSO impacts would be concurrent with the CWTP discharge sampling events, for at least two CSO events. Combined wastewater would be characterized for Pioneer Reservoir overflows, Sump 2 overflows (total station flow greater than 190 mgd), and CWTP effluent.

4) Determine the presence of organochlorine pesticides and polychlorinated biphenyls (PCBs) in CWCS discharge samples, using standard screening analysis methods. In addition, determine the presence of tributyltin. Tributyltin is an organic compound included in the previously mentioned objective concentrations for aquatic life, and requires special analytical

methods. Testing for these constituents in the CSO and effluent samples rather than the river would yield "worst case" results with respect to river impacts, because the river dilutes the discharge.

5) Perform a matrix detection limit (MDL) study for lead concentrations in the four unique matrices being sampled in the CWCS. The four matrices involved are: the Sacramento River in the vicinity of CSO discharges, CWTP effluent, Sump 2 overflow, and Pioneer Reservoir overflow. The objective was to determine the repeatable reporting limit concentrations that could be achieved for these matrices using standard analysis methods.

C. OBJECTIVES MET BY THE CSO SAMPLING EVENT

This report documents the findings of the CSO event that occurred on February 12, 1992, and is the second in a series of three reports for storm/CSO events sampled during the 1991/1992 wet weather season. During this event, CWTP discharged effluent to the Sacramento River and a CSO occurred from Sump 2. No discharge occurred from Pioneer Reservoir.

C.1. Chronic Toxicity

Provision 5 of the City's NPDES permit requires that a biotoxicity study be completed on 100-percent effluent samples during the 1991/1992 wet season. If toxicity is present, the City would be required to perform a dilution series as part of its 1992/1993 monitoring plan. The City voluntarily accelerated this schedule in the case of the CWTP effluent biotoxicity tests, by performing dilution series tests on CWTP effluent samples under the 1991/1992 monitoring program. The toxicity of CWTP effluent dilutions, which define the LOEC and NOEC dilutions, were analyzed in accordance with EPA protocols that specify the following dilution percentages: 100, 50, 25, 12.5 and 6.2. Two additional dilutions of 3.2 and 1.6 percent effluent were also analyzed for all three test species. As per the EPA protocols, dilution water was collected from the river upstream of the CWTP outfall at Captain's Table dock, and tested for chronic toxicity.

The 100-percent solution of combined wastewater discharged from Sump 2 was analyzed for chronic toxicity. This analysis was conducted to provide baseline toxicity information for this CWCS sample matrix.

The Sacramento River water collected at Captain's Table dock for preparing the CWTP effluent dilution series was analyzed separately for chronic toxicity. This analysis was conducted to provide baseline or "control" information on Sacramento River water used for the dilution series.

Procedures for determining if toxicity is present are defined in EPA 600/4-89/001 (*Short-term Methods for Estimating the Chronic Toxicity of Effluent and Receiving Water to Freshwater Organisms*) and EPA 440/4-85/032 (*Technical Support Document for Water Quality Based Toxic Control*). EPA's three-species freshwater chronic toxicity test procedures were used, as specified in the CWCS NPDES permit and in Table 1-1.

TABLE 1-1. BIOTOXICITY TEST PARAMETERS

Test Specie	Type of Test	EPA Method
Fathead Minnow	Survival/Growth	1000
Ceriodaphnia (water flea)	Survival/Reproduction	1002
Selenastrum (algae)	Growth	1003

The biotoxicity analysis completed for these effluent, CSO and river samples fulfills part of Objective 1 for the 1991/1992 CWCS Sampling Program. Any conclusions on the toxicity of CWTP effluent and system CSOs can only be made from the analysis of multiple events. A summary report for all three sampling events conducted during the 1991/1992 wet season will present conclusions, if any, on the consistent and/or seasonal presence of chronic toxicity.

C.2. Bacteria Content of Street Floodwaters

The February 12, 1992 storm event was, in general, a high-intensity storm but localized flooding was minimal. Therefore, the City staff did not collect samples of local street floodwaters as part of this sampling event.

C.3. Trace Metal Pollutants

The collection and analysis of water quality samples was conducted to determine the presence of trace metal pollutants in the Sacramento River during a CSO/CWTP effluent discharge event. These were collected from the river at the Captain's Table dock, Miller Park dock, and the "Wooden Stairs" location. The Miller Park dock samples provided the river background chemistry for estimating CWCS discharge impacts. Water quality samples were collected and analyzed for the dilution water collected from the Captain's Table dock. Analyses were also completed for discharges from Sump 2, and for CWTP effluent.

The trace metal analysis fulfilled the requirements of Objective 3 for the 1991/1992 CWCS Sampling Program.

C.4. Organochlorine Pesticides/PCBs and Tributyltin

The collection and analysis of samples was conducted to determine the presence of organochlorine pesticides and PCBs in the Sump 2 CSO and CWTP effluent discharges. Samples were also collected from these discharges to determine the presence of tributyltin.

The organochlorine pesticides/PCBs and tributyltin analysis fulfilled the requirements of Objective 4 for the 1991/1992 CWCS Sampling Program. However, the holding time for these samples was exceeded. The impacts of this laboratory error are described in Section 3.

C.5. Lead MDL Study

Samples for conducting the lead MDL study were collected during the CSO event. The findings of the study will be submitted in a separate report.

C.6. Other Constituents

Sacramento River waters (including water collected for CWTP effluent dilutions), and Sump 2 CSO and CWTP effluent discharges were analyzed for bacteriological and inorganic constituents.

D. REPORT OVERVIEW

This report is organized into seven major sections. This section summarized the objectives for 1991/1992 CWCS Sampling Program, conducted during the 1991/1992 wet weather season. Section 2 describes the event sampling program, including the field methods, sample collection and handling procedures, and laboratory quality assurance procedures. In addition, this section presents the general sample and analysis matrix and the specific constituents analyzed for the CSO event samples. Section 3 summarizes the storm event, sample collection timing, field data results, and the results of the quality assurance analyses conducted by the laboratory. Section 4 describes the derivation of the water quality objective concentrations based on the *California Inland Surface Waters Plan*. Section 5 summarizes the water quality sample results for the February 12, 1992, storm/CSO event. Section 6 provides a description of the chronic toxicity testing program, test results, and quality assurance results. Section 7 then summarizes the results for the sampling event.

SECTION 2
EVENT SAMPLING PROGRAM

This section describes the field procedures used to collect and handle the water quality samples obtained from the Sacramento River, and Sump 2 CSO and CWTP effluent discharges during the February 12, 1992 storm event.

A. FIELD METHODS FOR SAMPLE COLLECTION

The *Field Manual* prepared for the Phase 3 study provides a detailed description of the sampling procedures and instructions for all locations and constituents involved in the sampling program. Because the timing, movement, and intensity of each storm event differs, the *Field Manual* (final version dated January 30, 1992), was intended to provide guidelines for a theoretical storm pattern. Storm event descriptions are presented in Section 3 of this report.

A summary matrix of the sampling groups and analyses for this event is presented in Table 2-1. The table identifies the types of analyses and groups of constituents of interest for this CSO/CWTP effluent discharge event. Table 2-2 lists the constituents analyzed during this storm event.

A.1. Sacramento River

The Sacramento River water obtained for inorganic chemistry (including trace metals) and bacteria analyses was collected at Captain's Table dock, Miller Park dock, and the Wooden Stairs. Dilution water for CWTP chronic toxicity tests was obtained from Captain's Table dock. Water was collected directly from the river into plastic receptacles and transferred into the sample containers. The water for oil and grease analysis was collected in glass bottles filled directly from the river. River water for the lead MDL study obtained during this event was collected directly in plastic receptacles and transferred into the sample containers. Water temperature and dissolved oxygen were measured at the time of sample collection.

TABLE 2-1. SAMPLING AND ANALYSIS MATRIX FOR CSO EVENTS		
Location/Description	**Analyses/Constituents**	**Sample Type**
Sacramento River Sites		
Captain's Table		
"Captain's Table I"		
After discharge from CWTP, but before Sump 2 discharge, for "pre-discharge" background of river water quality	Dilution Water for CWTP Chronic Toxicity Test	Grab
	Chemistry[1] (of dilution water)	Grab
	Oil and Grease	Grab[4]
	MS/MSD[7]	Grab
	Duplicate[6]	Grab
	Chronic Toxicity (of dilution water, 100% sample test for control compared to CWTP effluent toxicity)	Grab
	Lead MDL Study	Grab
"Captain's Table II"		
After Sump 2 discharge	Chemistry[1]	Grab
Miller Park		
"Miller Park I"		
After discharge from Sump 2, but before Pioneer discharge, for "pre-discharge" background of river water quality[6]	Chemistry[1]	Grab
Wooden Stairs		
"Wooden Stairs"		
After discharge from CWTP	Chemistry[1]	Grab

TABLE 2-1. SAMPLING AND ANALYSIS MATRIX FOR CSO EVENTS		
Location/Description	Analyses/Constituents	Sample Type
CWCS Discharge Sites		
Sump 2 CSO		
"Sump 2"	Chemistry[1]	Composite[2]
	Tributyltin	Grab
	Lead MDL Study	Grab
	Oil and Grease	Grab[3]
	Pesticides/PCBs	Grab[3,4]
	MS/MSD	Composite[2]
	Chronic Toxicity (100% sample test)	Composite[2]
CWTP Effluent		
"CWTP"	Chemistry[1]	Composite[2]
	Tributyltin	Grab
	Lead MDL Study	Grab
	Oil and Grease	Composite[3]
	Pesticides/PCBs	Composite[3,4]
	MS/MSD[7]	Composite[2]
	Chronic Toxicity (dilution series)	Composite[2]

1 Includes inorganics, bacteria, and trace metals. See Table 2-2.
2 Composites consisted of discrete samples collected at 30-minute intervals over the duration of the discharge.
3 Discrete samples were obtained during each 30-minute interval over the duration of the discharge. Laboratory prepared composite sample in a galss container.
4 Enough material was collected to allow for MS/MSD analyses.
5 Miller Park site used for river background chemistry for Sump 2 discharge.
6 This was a sample split rather than a field duplicate.
7 Matrix spike/matrix spike duplicate.

TABLE 2-2. CONSTITUENTS TO BE ANALYZED FOR THE CSO SAMPLING EVENT

Constituents	EPA Method Number	Reporting Limit	Container Type	Sample Size
Bacteria				
Total Coliform	SM 908A	2 MPN	Plastic	2 X 250 ml
Fecal Coliform	SM 908C	2 MPN	Plastic	
Fecal Streptococci	SM 908C	2 MPN	Plastic	
Inorganics				
pH	150.1	N.A.	Plastic	500 ml[1]
Temperature	Field	-	-	-
Turbidity	180.1	N.A.	Plastic	500 ml[1]
Dissolved Oxygen	Field	-	-	-
Hardness as $CaCO_3$	314A	3.3	Glass w/ HNO_3	500 ml
Total Kjeldahl Nitrogen	351.2	0.5 mg/l	Glass w/ H_2SO_4	1 liter[2]
Phosphorus (total)	365.3	0.5 mg/l	Glass w/ H_2SO_4	1 liter[2]
Phosphorus (dissolved)	365.3	0.5 mg/l	Plastic	500 ml[3]
Nitrate	300.0	0.5 mg/l	Plastic	500 ml[1]
Nitrite	300.0	0.5 mg/l	Plastic	500 ml
Chemical Oxygen Demand	410.4	5.0 mg/l	Glass w/ H_2SO_4	1 liter[2]
Biochemical Oxygen Demand	405.1	2.0 mg/l	Plastic	500 ml
Total Suspended Solids	160.2	5 mg/l	Plastic	500 ml[4]
Total Dissolved Solids	160.1	10 mg/l	Plastic	500 ml[4]
Oil and Grease	413.2	1.0 mg/l	Glass	2 liters[5]
Organics				
Organochlorine Pesticides/PCBs	608	0.05-1.0 µg/l	Glass	2 liters[5]
Tributyltin	-	0.001 µg/l	Glass	2 liters
Metals (Total, Recoverable, and Dissolved)				
Arsenic	206.3 (HAA)	1.0 µg/l[6]	Plastic	--[3,7]
Beryllium[8]	200.7 (ICP)	5.0 µg/l	Plastic	--
Cadmium	213.2 (GFAA)	0.4 µg/l	Plastic	--
Chromium	218.2 (GFAA)	0.3 µg/l	Plastic	--
Cobalt[8]	200.7 (ICP)	10.0 µg/l	Plastic	--
Copper	220.2 (GFAA)	0.5 µg/l	Plastic	--
Lead	239.2 (GFAA)	1.7 µg/l	Plastic	--
Mercury[9]	245.1 (CVAA)	0.2 µg/l	Plastic	--
Nickel[10]	249.2 (GFAA)	2.5 µg/l	Plastic	--
Selenium	270.3 (HAA)	0.3 µg/l[6]	Plastic	--
Silver	272.2 (CVAA)	0.1 µg/l	Plastic	--
Zinc	200.7 (ICP)	8.5 µg/l	Plastic	--
Lead MDL Study[11]	239.2 (GFAA)	12	Plastic	--

1 Analyses for pH, turbidity, nitrate and nitrite conducted from one sample container.
2 Analyses for total Kjeldahl nitrogen, COD, and total phosphorus collected from one sample container.
3 Analysis for dissolved phosphorous and dissolved metals conducted from one 500-ml container. The laboratory filtered the sample and then preserved with HNO_3 before analysis.
4 TDS and TSS collected in one container, and separated by laboratory for analysis.
5 Total amount required for analysis. Discrete samples collected in the field and composited by the laboratory.
6 Analysis of recoverable concentrations for arsenic and selenium is by GFAA, which results in a higher reporting limit of 5 µg/l. Analysis for total and dissolved concentrations for arsenic and selenium is by HAA, which results in reporting limits of 1.0 µg/l and 0.3 µg/l, respectively.
7 Analysis for total and recoverable concentrations, and hardness conducted from 2 500-ml sample containers, preserved with HNO_3.
8 Total, recoverable, and dissolved analysis for both beryllium and cobalt was not required or requested for this study. However, in the ICP process, the analytical results for these metals became available at no charge to the project, therefore it was incorporated in this CSO event report.
9 No method available for analyzing recoverable mercury.
10 The analysis of nickel was not required or requested by this study. However, because of a laboratory log-in error, an analysis for dissolved nickel was conducted at no charge to the project. The data were incorporated into this CSO event report.
11 Samples collected and analyzed from all discharge locations and from one river site for determining recoverable and dissolved reporting limits. Laboratory filtered and preserved the samples.
12 To be determined by study.

A.2. Sump 2

At Sump 2, water was collected at 30-minute intervals from the spigot located on the discharge piping of Pump No.11. Combined wastewater from the Sump 2 wet well is pumped by Pump No. 11 to Chamber 3 of the Flow Control Structure, then discharged into the river. Water for the oil and grease analysis and the organochlorine pesticides/PCBs analysis were collected in glass bottles filled directly from the spigot every 30 minutes. These sample bottles were then composited into one glass container at the laboratory. Water for the tributyltin analysis was collected in glass bottles filled directly from the spigot during the first 30-minute period only. Discharge for the lead MDL study was collected in polyethylene bottles filled directly from the spigot during the first 30-minute period only.

Discrete samples for all other analyses (other inorganics, trace metals, bacteria) were composited into 48-liter polyethylene carboys at 30-minute intervals. The volumes added to the carboys were time-dependent and are specified in the *Field Manual*. Water temperature and dissolved oxygen were measured at the time of each 30-minute interval collection at Sump 2.

A.3. CWTP

At the CWTP, effluent was collected at 30-minute intervals from a spigot in the wet laboratory. This spigot is plumbed into the effluent stream following the dechlorination stage of treatment. Discrete effluent samples for the inorganic analyses (except oil and grease) were composited into 48-liter polyethylene carboys at 30-minute intervals. The volumes added to the carboys were time-dependent and are specified in the *Field Manual*.

The samples for oil and grease analysis and organochlorine pesticides/PCBs analysis were collected in glass bottles filled directly from the spigot every 30 minutes. These sample bottles were then composited into one glass container at the laboratory. Water for tributyltin analysis was collected in glass bottles filled directly from the spigot during the first 30-minute period only. Effluent for the lead MDL study was collected in polyethylene bottles filled directly from

the spigot during the first 30-minute period only.

Discrete samples for all other analyses (other inorganics, trace metals, bacteria) were composited into 48-liter polyethylene carboys at 30-minute intervals. The volumes added to the carboys were time-dependent and are specified in the *Field Manual*. Water temperature and dissolved oxygen were measured at the time of each 30-minute interval collection at CWTP.

B. SAMPLE HANDLING PROCEDURES

All containers, ice chests, and packing materials were prepared and provided by the laboratory. All filtering, necessary for analyzing dissolved metals and phosphorous, was performed by the laboratory upon delivery of the samples. Exposure to sunlight was avoided by keeping sample bottles in covered ice chests. Exposure to sunlight was negligible for this event because samples were collected under heavy cloud cover during the night and early morning hours of the storm event. All samples were packed on ice, without freezing, prior to transport to the laboratory.

B.1. Total Kjeldahl Nitrogen, Chemical Oxygen Demand, and Total Phosphorus

The container used for the collection of these constituents was a glass bottle preserved with sulfuric acid. The sample bottle was filled leaving a small air space at the top.

B.2. Turbidity, pH, Nitrate, and Nitrite

The container used for the collection of these constituents was a polyethylene bottle, which was filled leaving a small air space at the top of the bottle.

B.3. Total Suspended Solids and Total Dissolved Solids

A single polyethylene bottle was used to collect water for both of these analyses. The sample was split into two batches at the laboratory and analyzed for each constituent.

B.4. Bacteriological and Biological Oxygen Demand

The containers for collecting the bacteriological samples were preserved with sodium thiosulfate to neutralize the effects of residual chlorine on the organisms in the sampled waters. The container for BOD analysis was a polyethylene bottle with no preservatives.

B.5. Total Metals, Recoverable Metals, and Hardness

The containers used for the collection of these constituents were polyethylene bottles preserved with nitric acid.

B.6. Dissolved Metals and Dissolved Phosphorus

The containers used for the collection of these constituents were unpreserved polyethylene bottles. Upon delivery to the laboratory, the sampled waters were filtered and subsequently preserved with nitric acid.

B.7. Oil and Grease, and Organochlorine Pesticides/PCBs

The containers used for the collection of these constituents were glass bottles. The sampled waters did not contact plastic containers or implements in their compositing or storage.

C. LABORATORY QUALITY ASSURANCE METHODS

Enseco Laboratory performed various quality assurance procedures in the analysis of the CWCS and river samples to ensure the quality of test results. The *Enseco Incorporated Quality Assurance Program* requires the analysis of several types of control samples as standard elements in their Laboratory Performance Quality Control process. Duplicate control samples, single control samples, method blanks, and equipment blanks are used to identify any procedural errors. These are a part of Enseco's in-house QA/QC procedures.

Duplicate control samples (DCS) consist of a pair of standard, control matrix samples (reagent water is used for aqueous samples) spiked with surrogate compounds representative of the constituents to be tested. Accuracy and precision data are compared to the control limits established for the tested constituents. This is a standard procedure conducted on all sets of samples.

A single control sample (SCS) is also a control matrix sample, spiked with a surrogate compound appropriate to the testing method. This test is used with smaller batches of samples. The recovery data are compared to the control limits established for the tested compound.

Method blanks (MB) are used to determine the level of interference or contamination in the analytical system that could elevate or diminish concentration results. The concentration of target constituents in the MB samples should be below the reporting limit. However, common solvents, which are present in the laboratory, may be acceptable at levels up to 5 times the reporting limit. Because certain metals may also be difficult to reduce to the extremely low reporting limits used in this study, concentrations are accepted up to 2 times the reporting limit.

D. FIELD QUALITY CONTROL METHODS

Matrix spikes (MS) and matrix spike duplicates (MSD) are environmental samples collected in the field. The MS and MSD samples are spiked with known concentrations of constituents, and from the analysis of these controls, the percent recovery of the added constituents is determined. Based upon the results of this analysis, the effect of the matrix on the accuracy of the analysis procedure is determined. Both the MS and the MSD results indicate the precision of the analysis for a particular constituent and matrix.

As part of the quality assurance program, the laboratory split the river water samples collected upstream of the CWTP effluent discharge in half. (These were the "Captain's Table I" samples, collected upstream of the CWTP outfall, prior to the CSOs from Sump 2). These two sets of samples were then analyzed. One set of samples was labeled with an "SA" suffix in the analytical results and is

referred to as the parent set of samples in this report. The other set of samples was labeled with a "DU" suffix in the analytical results and is referred to as the quality control sample set. This quality assurance methodology was used to verify the laboratory's ability to replicate precise and accurate analytical results.

A complete set of duplicate water samples was also collected from the Captain's Table dock, upstream of the CWTP outfall and prior to the CSO discharge from Sump 2. These were labeled "Dup" in the laboratory results (see Appendix A). In the initial sampling plan, field duplicates (as opposed to sample splits), were intended to be used as the field quality control samples. However, the sample split results proved to be a more stringent measure of laboratory precision and accuracy. Therefore, the sample splits rather than the "Dup" samples results were used as the field quality control check on the laboratory.

SECTION 3
STORM EVENT DESCRIPTIONS, SAMPLE COLLECTION TIMING, FIELD DATA RESULTS, AND QUALITY ASSURANCE RESULTS

This section describes the event during which sampling took place, presents the field data, and discusses the quality assurance analytical results.

A. STORM EVENT AND DISCHARGES

During the February 12, 1992 storm event, 1.47 inches of rainfall were measured at the National Weather Service gauge located at 801 I Street in Sacramento. The 13.5-hour storm began at 8:30 p.m. on February 11 and lasted until 10:00 a.m. on February 12. Discharges from Sump 2 and CWTP were recorded and sampled during the storm event. No discharge occurred from Pioneer Reservoir during this storm event.

The CWTP discharge began at 2:30 a.m. and lasted until 10:30 a.m. on February 12. Over the course of this treatment period (8 hours), 37.3 million gallons (MG) of effluent was discharged into the Sacramento River.

Two distinct discharges were made into the Sacramento River from Sump 2 during the storm event. A 4.1-MG discharge was recorded between 2:30 a.m. and 3:45 a.m. on February 12. The second discharge of 14.4 MG began at 4:15 a.m. and lasted until 8:45 a.m. on February 12. Therefore, a total CSO volume of 18.5 MG was discharged over a total of 5.75 hours.

The Sacramento River was flowing at a rate of approximately 10,865 million gallons per day, or 16,810 cubic feet per second, past the USGS gauge at Freeport Bridge.

These storm and discharge event descriptions and data were prepared by City staff.

Assuming this river flow rate was the average rate that occurred during the discharge periods, then the dilution factors based on volume were as follows:

$$\text{CWTP Dilution} = \frac{\text{River Volume}}{\text{Effluent Volume}} = \frac{10{,}865 \text{ mgd} * 8 \text{ hrs}}{37.3 \text{ MG}} \frac{1}{24 \text{ hrs}} = 97:1$$

$$\text{CSO Dilution} = \frac{\text{River Volume}}{\text{CSO Volume}} = \frac{10{,}865 \text{ mgd} * 5.75 \text{ hrs}}{18.5} \frac{1}{24 \text{ hrs}} = 141:1$$

These river dilution factors are used to estimate the potentially detectable concentrations of organic compounds in the river from the test results for CWTP effluent and Sump 2 CSO samples. Section 5 presents this evaluation.

B. SAMPLE COLLECTION TIMING

During this CSO event, samples were collected from the Sacramento River to determine the effects of Sump 2 discharges and CWTP effluent on river water quality. This objective was accomplished by sampling river water for establishing "pre-discharge" conditions, and then sampling as the plume of discharge or effluent, respectively, passed the sampling location.

At the time of the CSO from Sump 2, Sacramento River velocities were measured at the Captain's Table dock and Miller Park dock. The river velocity was measured at 0.50 feet per second (fps), using a Montedoro-Whitney electromagnetic velocity meter. A time-of-travel for the Sump 2 discharge plume was calculated using the measured river velocity and the distance between Sump 2 and the Captain's Table dock (4,800 feet). Later in the storm event, river water samples were obtained as the Sump 2 discharge plume passed the Captain's Table dock, as shown in Table 3-1. These samples, labeled "Captain's Table II", were analyzed to determine the effects of the discharge on river water quality.

TABLE 3-1.TIME TABLE OF SAMPLE COLLECTION AND DISCHARGE/EFFLUENT PLUME			
Sample Identification	**Sample Collection Period (Clock Hour)**	**Sump 2 Plumes Past Captain's Table Dock (Clock Hour)**	**CWTP Plume Time (Clock Hour)**
Captain's Table I	0315 - 0500	0525 - 0630 0700-1100	NA
Captain's Table II	0700 - 0740	0525 - 0630 0700 - 1100	NA
Miller Park I	0340 - 0520	NA	NA
Wooden Stairs	0545 - 0640	NA	0505 - after 1030

River water samples from the Miller Park dock, upstream of the Sump 2 discharge, were collected as the discharge was occurring. These samples, labeled "Miller Park I", were analyzed to determine the background river water quality prior to the Sump 2 CSO discharge.

The Sacramento River velocity of 0.50 fps and the distance between CWTP and the Wooden Stairs dock (4,400 feet) were used to calculate the time-of-travel for the CWTP effluent plume. Later in the storm event, river water samples were collected as the CWTP effluent plume passed the Wooden Stairs location, as indicated in Table 3-1. These samples were analyzed to determine the effects of the effluent discharge on river water quality.

River water samples from the Captain's Table dock, upstream of the CWTP discharge, were collected as the CWTP discharge was occurring but before the Sump 2 discharge plume arrived at this location. These samples, labeled "Captain's Table I", were analyzed to determine the background river water quality prior to CWTP effluent discharge.

C. FIELD DATA RESULTS

Water temperature, dissolved oxygen (DO) concentration measurements, and time of collection for each of these samples are presented in Table 3-2. Water temperatures recorded from samples collected in the Sacramento River ranged from 11.7°C to 12.2°C. Water temperatures recorded for samples collected within the wastewater system ranged from 10.0°C to 11.0°C in the CWTP effluent, and from 14.7°C to 15.1°C in the Sump 2 CSO. DO concentrations were generally higher in Sacramento River waters, ranging from 10.0 mg/l to 10.4 mg/l at the sampling sites, than in the discharge samples. DO concentrations ranged from 9.0 mg/l to 10.8 mg/l in the Sump 2 discharge, and from 7.8 mg/l to 9.6 mg/l in CWTP effluent.

TABLE 3-2. FIELD DATA RESULTS FOR FEBRUARY 12, 1992 EVENT			
Sample Identification	Time	Temp (°C)	DO (mg/l)
Captain's Table I	0432	11.7	10.4
Captain's Table II[1]	0733	12.2	10.0
Miller Park I[2]	0340	12.2	10.0
Wooden Stairs	0635	12.2	10.2
Sump 2 Discharge	0245	15.1	9.0
	0315	15.0	9.1
	0345	15.2	9.0
	0445	15.0	No Data[3]
	0515	15.0	No Data[3]
	0545	15.1	10.8
	0615	15.1	10.2
	0648	14.7	9.8
	0720	15.0	9.9
	0750	14.9	9.6
CWTP Effluent	0245	11.0	8.2
	0315	11.0	9.1
	0345	11.0	9.2
	0415	10.0	9.2
	0445	10.0	9.6
	0515	10.0	9.6
	0545	10.0	9.0
	0615	10.0	9.2
	0645	10.0	9.4
	0715	10.0	7.8
	0745	10.0	8.6
	0815	10.0	8.4
	0845	10.0	7.8

1 Samples collected during the passing of the Sump 2 discharge plume.
2 "Background" river site, upstream of all other discharges, for this event.
3 The dissolved oxygen meter malfunctioned during this period.

D. EVENT-SPECIFIC QUALITY ASSURANCE ANALYSIS RESULTS

As described in Section 2, a series of in-house laboratory and project-specific quality assurance procedures were applied in the analysis of the CWCS samples. Laboratory quality assurance procedures, designed to assess laboratory operations and to establish method-specific control limits, included the analysis of method blanks, matrix duplicates, single control samples, and duplicate control samples. Matrix spike (MS) and matrix spike duplicate (MSD) analyses of field samples were conducted to assess potential sample matrix effects.

The quality assurance results indicate that the laboratory testing procedures produced reliable results for the various analytes within the water quality samples. All of the percent recoveries and relative percent differences were within the accepted control limits. However, several specific complications resulting from matrix interferences and laboratory contaminants influenced the interpretation of the results.

The organochlorine pesticide and PCB analyses were conducted outside of the two-week holding time, due to a laboratory log-in error. Sample extraction was performed 30 days after delivery to the laboratory from the field. During the extended holding time, the samples may have metabolized, breaking down certain constituents into their sub-compounds. The organochlorine pesticides and PCB list for an EPA Method 608 analysis screen has some compounds that are metabolites of more complex analytes on the list. DDT is one such analyte where the laboratory also searched for DDD and DDE. This would provide information on the possible presence of the parent molecule. In all cases, no metabolites were detected. The chromatographs of each sample were carefully studied by the laboratory for any possible detections below the reporting limits. Although the matrix examined caused chromatography interferences, laboratory staff found no confirmed detections for any of the "target" analytes (i.e. those from the *Inland Surface Waters Plan* aquatic life objectives). Quality control samples with analyte spikes reported acceptable recoveries, giving the laboratory confidence in their ability to detect pesticides and PCBs in this matrix.

As mentioned, matrix interferences were observed in the analysis of organochlorine pesticides and PCBs for Sump 2 and CWTP samples. For the Sump 2 sample analysis, reporting limits for 3 of the 19 pesticides and 5 of the 7 PCB constituents were raised to compensate for the interferences. For the CWTP effluent sample analysis, reporting limits for 19 pesticides and the 7 PCB constituents were raised to compensate for matrix interferences. Following the initial analysis, the results did not match the calibration curve for these analytes. The sample was then diluted to obtain more accurate results. With a Method 608 screen analysis, it is not possible to exactly determine the cause of matrix interferences. Possible sources of interference are presence of pthalitic esters, sulfur or plasticisers. With regard to the latter, even items such as plastic cups in stormwater entering the sewers can cause a problem in the analysis. Because of this potential, the field and laboratory procedures required that sample water for organochlorine pesticide/PCB analysis be handled only in glass.

Sample matrix interferences were also observed in the analysis of recoverable copper for Sump 2. The reporting limit was raised from 0.00050 mg/l to 0.015 mg/l to compensate for the interferences. Laboratory personnel indicated that the matrix interferences may have been caused by an excess of nitric acid used in the sample container preservation or by other acids used in the sample preparation.

The reporting limits for total and dissolved mercury analysis were raised from 0.00020 mg/l to 0.00040 mg/l to reflect limitations in sample volume. The samples affected were Captain's Table I (total mercury) and CWTP effluent (total and dissolved mercury). Because the initial analytical results did not meet the in-house laboratory QA/QC guidelines, the laboratory re-analyzed the samples. In doing so, a significant portion of the sample volume was consumed. The laboratory split the sample volume into smaller fractions in order to conserve the remaining sample water. Therefore, the reporting limits were raised to reflect this volume reduction in the analyzed sample.

Two metal analytical results for sample blanks were influenced by laboratory contamination in sample preparation, resulting in detectable levels of chromium and zinc. Total chromium was found in two sample blanks at concentrations of

0.00053 mg/l and 0.87 mg/l, using a reporting limit of 0.00050 mg/l. Recoverable chromium was found in one sample blank at a concentration of 0.00088 mg/l, using a reporting limit of 0.00050 mg/l. Total zinc was found in one sample blank at a concentration of 0.012 mg/l, using a reporting limit of 0.0085 mg/l.

Under laboratory QA/QC procedures (adopted from the Environmental Protection Agency's *Solid Waste Analytical Methods Manual 846, Third Edition*), sample blank contamination must be less than two times the Minimum Detection Limit (MDL). The total chromium sample blank of 0.87 mg/l does <u>not</u> meet this guideline. This concentration in the sample blank may influence both the precision and accuracy of the analytical results within the QA/QC batch run on March 19, 1992, which included the following samples: Sump 2-Quality Control Sample, Sump 2-Matrix Spike, Sump 2-Matrix Spike Duplicate, CWTP-Quality Control Sample, and CWTP-Matrix Spike Duplicate.

SECTION 4
COMPARATIVE STANDARDS

This section describes the derivation of numerical objectives for pollutant concentrations used to provide a basis for comparison to the laboratory results for the storm event. The impact on the receiving water, i.e. the Sacramento River, is estimated through the use of these water quality objectives.

A. INLAND SURFACE WATERS PLAN

The *California Inland Surface Waters Plan - Water Quality Control Plan for Inland Surface Waters of California - 91-12 WQ* (Water Resources Control Board, April 1991), provides a comparative standard with which the laboratory analytical results can be evaluated. (This document is herein referred to as the *Inland Surface Waters Plan.*) Table 1 in this document presents the "Water Quality Objectives for Protection of Freshwater Aquatic Life", including 4-day average, daily average, 1-hour average, and instantaneous maximum pollutant concentration objectives.

Of the four durations (4-day, daily, 1-hour, instantaneous) for which water quality objectives are identified in Table 1 of the *Inland Surface Waters Plan*, an appropriate duration must be selected for comparison to the storm/CSO event results in this report. The duration of effluent discharges from the CWTP during this event was 8 hours, which exceeds 1 hour but is far shorter than 4 days. It would therefore be most appropriate to compare the event-specific results to a comparative standard for a duration between 1 hour and 4 days, consistent with the duration of the CWTP discharge. Ideally, comparable standards could be made available that account for the variability of storm and discharge event durations. However, no such published standards exist. Therefore, the storm event results were compared to <u>both</u> the 4-day and the 1-hour numerical objectives. Specific results for the storm event sampled, and a comparison to calculated numerical objective concentrations, are presented in Section 5.

B. DERIVATION OF NUMERICAL OBJECTIVES

B.1. Receiving Water Hardness

The *Inland Surface Waters Plan* (specifically, Table 1) identifies objective concentrations for 21 constituents. Numerical objectives for the 9 trace metals listed are based on the hardness (as $CaCO_3$) found in the receiving water during the event. These objectives will therefore differ between water bodies. The hardness of a given water body, like other constituents, also varies over time. A baseline hardness for the river at Miller Park dock, which is upstream of the Sump 2 outfall, was chosen to represent the Sacramento River during this storm event. Analytical results of river water collected from the Miller Park location showed a hardness concentration of 72.6 mg/l. This value, in conjunction with the river water's pH of 7.5, was used to derive the water quality objective concentrations.

B.2. Pollutant Concentration Objectives

The objectives listed in the *Inland Surface Waters Plan* were converted to pollutant concentrations to which the river sampling results were compared. These are listed in Table 4-1, including the objectives for specific organics. Equations for calculating concentrations can be found on Page 4 of the *Inland Surface Waters Plan* document.

Although the objective concentrations are compared to the Sacramento River sample results, discharges to the river cannot further degrade water quality when the river is at the objective concentration. If, for instance, the river concentration for a metal reaches the objective concentration, then the concentration of that metal in the discharge must be at or below the limit.

TABLE 4-1. WATER QUALITY OBJECTIVES FOR PROTECTION OF FRESHWATER AQUATIC LIFE FOR COMPARISON TO FEBRUARY 12, 1992 SAMPLING EVENT RESULTS

Values used in computations:
Hardness, in mg/l 72.6 (Sacramento River at Miller Park)
pH 7.5

Constituent	Units	Concentration Objectives			
		4-Day Average	Daily Average	1-Hour Average	Instantaneous Maximum
arsenic	ug/l	190.0	—	360.0	—
cadmium	ug/l	0.9	—	2.7	—
chlordane	ng/l	—	4.3		—
chromium(VI)	ug/l	11.0	—	16.0	—
copper	ug/l	9.0	—	13.1	—
DDT	ng/l	—	1.0	—	—
dieldrin	ng/l	—	1.9	—	—
endosulfan	ng/l	—	56.0	—	220.0
endrin	ng/l	—	2.3	—	180.0
heptachlor	ng/l	—	3.8	—	—
hexachlorocyclo-	ng/l	—	80.0	—	—
hexanegamma		—	—	—	—
lead	ug/l	2.1	—	54.3	—
mercury	ug/l	—	—	2.4	—
nickel	ug/l	120.3	—	1081.7	—
PCBs	ng/l	—	14.0	—	—
pentachlorophenol	ng/l	9.5	—	15.0	—
selenium	ug/l	5.0	—	20.0	—
silver	ug/l	—	—	—	2.3
toxaphene	ng/l	0.2	—	730.0	—
tributyltin	ng/l	20.0	40.0	—	60.0
zinc	ug/l	80.8	—	89.2	—

NOTES:

mg/l = milligram(s) per liter; ug/l = micrograms per liter; ng/l = nanograms per liter

Objectives for cadmium, chromium, copper, lead, nickel, silver and zinc are based on total recoverable metal concentration.

Dischargers may, at their option, meet the total chromium limitation.

The 4-day average for tributylin is the six month median.

Source: California Inland Surface Waters Plan, Water Resources Control Board, April 1991.

SECTION 5
WATER QUALITY SAMPLE RESULTS

This section summarizes the laboratory results for the storm/CSO event of February 12, 1992, and presents a comparison of water quality results for the Sacramento River, Sump 2 overflow, and CWTP effluent discharge to the *Inland Surface Waters Plan* objective concentrations. Results for chronic biotoxicity tests are presented in Section 6.

A. LABORATORY RESULTS

The detailed laboratory analysis data for this event are presented in Appendices A and B, separate from this report. Summaries of the laboratory results for detectable analytes are presented according to sampling locations in Tables 5-1 through 5-6. Although samples were analyzed for all constituents listed in Table 2-2, only the detected constituents are included in the results tables. Tables 5-1 through 5-6 present the analysis results from: Captain's Table I (on the river upstream of the CWTP outfall and prior to effluent discharge), Captain's Table II (on the river in the plume of Sump 2 CSO), Miller Park I (on the river upstream of Sump 2 CSO discharge), Wooden Stairs (on the river in the plume of CWTP effluent discharge), Sump 2 CSO, and CWTP effluent, respectively. All tables are located at the end of this section.

A.1. General Inorganics

The analyses of general inorganics constituents for Sacramento River samples (Captain's Table I, Captain's Table II, Miller Park I, and Wooden Stairs) resulted in concentrations higher than the reporting limit for all analytes except chemical oxygen demand (COD), total Kjeldahl nitrogen, and nitrite. Comparison between the Miller Park I sample, collected upstream of the Sump 2 discharge, and the Captain's Table II sample, collected in the effluent discharge plume of Sump 2, indicates comparable analytical results. This suggests that Sump 2 effluent discharge had an insignificant effect on the Sacramento River inorganic water quality.

The results from the Wooden Stairs river sample collected in the effluent discharge plume from CWTP are comparable to the upstream Captain's Table I sample. This suggests that CWTP effluent also had an insignificant overall effect on the Sacramento River's inorganic water chemistry.

Comparing the results from the Captain's Table I sample, collected upstream of the effluent discharge from CWTP, and Captain's Table II sample, collected after the effluent discharge from Sump 2, indicates a very slight increase in all of the detected inorganic constituents in the latter sample. This comparison suggests that Sump 2 CSO, over the discharge period may have had a very slight effect on the Sacramento River's inorganic water chemistry at the Captain's Table dock location.

The analysis of general inorganics constituents for the two discharge locations, Sump 2 and CWTP, indicates concentrations above the reporting limit for all analytes except nitrite. The inorganic characteristics of two discharges are comparable, with the exception of total dissolved solids, which was 56.0 mg/l in the Sump 2 discharge and 139.0 mg/l in the CWTP effluent. The pH results indicate that CWTP effluent was slightly more acidic than Sump 2 discharge, 6.3 and 6.6, respectively. Measured hardness as calcium carbonate ($NaCO_3$) in CWTP effluent was 25.3 mg/l, while Sump 2 effluent was 28.3 mg/l.

A.2. Bacteria

Bacteria counts from the Miller Park I river sample showed the presence of 5,000 MPN/100 ml of total coliform and fecal coliform. Fecal streptococcus were present at 3,000 MPN/100 ml upstream of all effluent discharges. Bacteria counts from Captain's Table I and II river samples showed the presence of 300,000 MPN/100 ml of total coliform and fecal coliform. Fecal streptococcus were present at 24,000 MPN/100 ml in the Captain's Table I river sample, prior to the Sump 2 CSO, and 17,000 MPN/100 ml in the Captain's Table II river sample, in the Sump 2 CSO plume. Analytical results for the Wooden Stair's sample, collected in the CWTP's effluent plume, showed a biological oxygen demand of 4.0 mg/l and the presence of 17,000 MPN/100 ml of total coliform and fecal coliform. Fecal streptococcus were present at 3,000 MPN/100 ml in this sample.

The bacteria results of the river samples do not suggest a uniform trend that can be correlated to pre- and post-discharges of either Sump 2 or the CWTP.

Analytical results from the Sump 2 discharge sample showed a biological oxygen demand (BOD) of 92.0 mg/l and the presence of total coliform, fecal coliform, and fecal streptococcus at 16,000,000 MPN/100 ml. Analytical results from the CWTP effluent sample showed a BOD concentration oxygen demand of 37.0 mg/l and the presence of total coliform and fecal coliform at 1,700 MPN/100 ml. Fecal streptococcus were not found at a reporting limit of 200 MPN/100 ml.

Bacteria counts in the CWTP samples were considerably lower than Sump 2 discharge samples, reflecting the effective chlorination treatment at the CWTP facility.

A.3. Organochlorine Pesticides/PCBs and Tributyltin

A.3.a. Discharge Results--

The organochlorine pesticide and PCB analytes were not analyzed within the required two-week holding time due to a laboratory log-in error, as described in Section 3. Organochlorine pesticides and PCBs were not detected in the samples of Sump 2 discharge. However, 3 of 19 pesticide and 5 of 7 PCB reporting limits for the Sump 2 samples were raised due to matrix interferences. None of the analyzed constituents were detected in the CWTP effluent. However, all of the pesticide and PCB reporting limits for this sample were raised due to matrix interferences.

The *Inland Surface Waters Plan* identifies instantaneous maximum concentrations for endrin (180 ng/l) and endosulfan (220 ng/l). Based on the "ND" result for a reporting limit of 100 ng/l for the Sump 2 CSO sample, it is concluded that these constituents are less than the objective concentration. For the CWTP effluent, the results are less conclusive. Endrin and endosulfan-I (or -alpha) were reported as "ND" at a reporting limit of 120 ng/l; these results indicate concentrations below the instantaneous maximum objective. Because the reporting limit was raised to 250 ng/l for endosulfan-II (or -beta) and endosulfan sulfate, no definitive conclusions can be drawn on the presence of these constituents as compared to the instantaneous maximum objective concentration.

Hexachlorocyclo-hexanegamma (also Lindane or gamma-BHC) results were "ND" for a reporting limit of 62 ng/l in both the Sump 2 and CWTP samples. Based on a daily average objective concentration of 50 ng/l, it is concluded that these sample concentrations were less than the objective.

Tributyltin was detected in the Sump 2 discharge at 0.024 µg/l, using a reporting limit of 0.001 µg/l (1 ng/l). Tributyltin was also detected in CWTP effluent at 0.040 µg/l using a reporting limit of 0.001 µg/l. A comparison of the tributyltin concentrations detected in the Sump 2 CSO and CWTP effluent discharges to the 4-day objective concentration listed in Table 4-1 shows that this analyte exceeded the objective concentration for both samples. (Tributyltin does not have a 1-hour objective concentration.) Neither concentration exceeded the instantaneous maximum objective.

A.3.b. Estimated River Results--

The organochlorine pesticide/PCB screen analysis was performed on only the discharge samples, rather than the river samples, in order to evaluate a "worst-case" condition. However, the reportable and detectable concentrations that would have occurred in the river samples can be approximated from the discharge samples and dilution factors calculated in Section 3. For these estimation purposes, dilution factors are rounded to 100:1 and 150:1, for CWTP and Sump 2 discharges to the river, respectively.

For instance, dieldrin was not detected in the Sump 2 CSO discharge, using a reporting limit of 0.10 µg/l. Using the dilution ratio, the reporting limit for the downstream river section would be 0.7 ng/l. At this reporting limit, the result would be "ND". Neither a 4-day average or 1-hour average objective concentration is available for dieldrin; the daily average value is 1.9 ng/l. Therefore, for this constituent, the estimated river concentration downstream of Sump 2 would be less than the daily average objective concentration.

The results of this analysis using dilution factors are summarized in Tables 5-8 and 5-9. As indicated by the tables, most of these compounds would not have been detected in the river as compared to the objective concentrations in the *Inland Surface Waters Plan*.

One-hour and 4-day objective concentrations are provided for toxaphene. Results for both CWTP effluent and Sump 2 CSO samples were "ND". However, conclusions relative to Table 4-1 objectives cannot be determined for either the samples or the estimated "diluted" river concentration, since the reporting limit is higher.

A.4. Oil and Grease

Oil and grease concentrations were not detected in any of the river samples at a reporting limit of 1.0 mg/l. The oil and grease were detected at 5.4 mg/l and 3.0 mg/l, using a reporting limit of 1.0 mg/l, in Sump 2 discharge and CWTP effluent, respectively.

A.5. Trace Metals

As described in Section 3, chromium and zinc (total and/or recoverable, or a combination) were found in all of the quality assurance sample blanks for these event sample batches. Blanks are analyzed coincident with sample batches to detect possible instrument contamination. The low reporting limits required by this study, which are close to detection limits and are necessary for comparison to the *Inland Surface Waters Plan* objective concentrations, increase the possibility that low-level contamination will be detected. This is particularly true for zinc, which is a common contaminant in the laboratory. If contamination is found in a blank, it is not possible to determine the amount of contamination that may have occurred in the sample of interest, unless special sample preparation and followup analyses are conducted. Such special sample handling was beyond the scope of this study.

The analysis for trace metals in the Captain's Table I river sample, collected upstream of the CWTP outfall, detected concentrations for six total metal, four recoverable metal, and three dissolved metal analytes, as shown in Table 5-1. The total zinc and total recoverable chromium analytical results may have been influenced by contamination from laboratory sample preparation. Although the reporting limit for total mercury was raised due to sample volume restrictions, it was not detected at the higher limit.

The analysis for trace metals in the Captain's Table II sample, collected in the plume of Sump 2 discharge, resulted in detectable concentrations for six total metal, four recoverable metal, and three dissolved metal analytes, as shown in Table 5-2. Total chromium, recoverable chromium, and total zinc analytical results may have been influenced by contamination. The reporting limit for the total mercury analysis was adjusted due to sample volume restrictions.

The analysis for trace metals in the Miller Park I sample, collected upstream of the Sump 2 discharge, detected concentrations for five total metal, two recoverable metal, and three dissolved metal analytes, as shown in Table 5-3. Total and recoverable chromium results may have been influenced by contamination from laboratory sample preparation.

In the Wooden Stair's sample, collected in the plume of CWTP effluent, five total metal, three recoverable metal, and three dissolved metal analytes were detected, as shown in Table 5-4. The total chromium and total zinc analytical results may have been influenced by contamination from laboratory sample preparation.

Seven total metal, five recoverable metal, and five dissolved metal analytes were detected in the Sump 2 discharge as shown in Table 5-5. Total chromium, recoverable chromium, and total zinc were found in the sample blanks for this location.

The analysis for trace metals in the CWTP effluent sample detected seven total metal, five recoverable metal, and four dissolved metal analytes, as shown in Table 5-6. However, the total chromium, recoverable chromium, and total zinc analytical results may have been influenced by contamination from laboratory sample preparations. The reporting limit for the total and dissolved mercury analysis was adjusted due to sample volume restrictions.

A.5.a. Sump 2 Discharge Effects--

A comparison of trace metal concentrations prior to and after Sump 2 discharge did not indicate that there was a significant impact on the Sacramento River water quality during this event. Of the five <u>total</u> metal concentrations detected in both pre-discharge and post-discharge samples, three analyte concentrations

were higher in the pre-discharge (Miller Park I) sample, while two analyte concentrations were higher in the post-discharge (Captain's Table II) sample. Total lead was detected at 0.0028 mg/l in the post-discharge river sample, but was not found above the reporting limit in the pre-discharge sample.

Of the two recoverable metal concentrations detected in both river water samples for pre-discharge and post-discharge of Sump 2, one analyte concentration was higher in the pre-discharge (Miller Park I) sample and one was equal in concentration at both sample locations. Recoverable lead and zinc were detected in the post-discharge (Captain's Table II) sample at concentrations of 0.0026 mg/l and 0.012 mg/l, respectively, but were not found above the reporting limit in the pre-discharge (Miller Park I) sample.

Of the three dissolved metal concentrations detected in both sample pre- and post-discharge of Sump 2, one analyte concentration was higher in pre-discharge (Miller Park I) sample, while two were higher in the post-discharge (Captain's Table II) sample. Dissolved copper was detected in the post-discharge (Captain's Table II) sample at 0.0029 mg/l and in the pre-discharge (Miller Park I) sample at 0.0011 mg/l.

A.5.b. CWTP Effluent Discharge Effects--

A comparison of metal concentrations prior to and after CWTP effluent discharge also did not indicate a significant impact on the Sacramento River water quality due to the discharge. Of the five total metal concentrations detected in both pre-discharge and post-discharge river water sample samples, three analyte concentrations were higher in post-discharge (Wooden Stairs), one was detected at the same concentration in both pre-discharge and post-discharge samples, while one was higher in the pre-discharge (Captain's Table I) sample. Total chromium was detected in the pre-discharge (Captain's Table I) sample at 0.0038 mg/l and in the post-discharge (Wooden Stairs) sample at 0.0019 mg/l. Total silver was detected at 0.00013 mg/l in the pre-discharge sample, but was not found above the reporting limit in the post-discharge sample.

Of the three recoverable metal concentrations detected in both of CWTP's pre-discharge and post-discharge river water samples, all three analyte

concentrations were higher in the pre-discharge (Captain's Table I) sample. Recoverable zinc was also detected in the pre-discharge (Captain's Table I) sample at a concentration of 0.011 mg/l, but was not found above the reporting limit in the post-discharge (Wooden Stairs) sample.

Of the three dissolved metal concentrations detected in both of CWTP's pre-discharge and post-discharge river water samples, two analyte concentrations were higher in the post-discharge (Wooden Stairs) sample, while one was higher in the pre-discharge (Captain's Table I). Dissolved copper was detected in the pre-discharge (Captain's Table I) sample at 0.0015 mg/l and in the post-discharge (Wooden Stairs) sample at 0.00075 mg/l.

A.5.c. Comparison of Sump 2 and CWTP--

A comparison of the analyzed trace metal constituents detected in both discharge locations, Sump 2 and CWTP, indicates strong similarities. The analysis for total metals in both the Sump 2 discharge and CWTP effluent samples detected seven total trace metals. Of these seven concentrations, six concentrations were higher in the Sump 2 sample and one analyte (silver) was detected at the same concentration. The analysis for recoverable metals in both samples detected concentrations for five trace metals. Of these five concentrations, one analyte (lead) concentration was higher in the Sump 2 sample, and four analyte concentrations were higher in the CWTP sample. The analytical results for dissolved metals in the Sump 2 discharge and CWTP effluent samples show detectable concentrations for five and four of the analytes, respectively. Of the four dissolved metal concentrations detected in both samples, one analyte concentration was higher in the Sump 2 sample and three analyte concentrations were higher in CWTP effluent. Dissolved arsenic was also detected in the Sump 2 sample at a concentration of 0.0029 mg/l, but was not found above the reporting limit in the CWTP sample.

A.5.d. Comparison to Water Quality Objectives--

Analytical results for trace metals in the river and discharge samples, presented in Tables 5-1 to 5-6, were compared to the *Inland Surface Waters Plan* objective concentrations listed in Table 4-1. Although the objectives represent recoverable concentration, comparison was also made to total and dissolved metal

concentrations detected in the event samples.

A comparison of trace metal concentrations (total, recoverable and dissolved) detected in the Sacramento River to the 1-hour averge objective concentrations listed in Table 4-1 shows that no trace metal exceeded those objectives. A comparison of the trace metal concentrations detected in the Sacramento River to the 4-day average objective concentration shows that total and recoverable lead exceeded the 4-day objective at various sampling locations: recoverable lead in Captain's Table I sample; total and recoverable lead in the Captain's Table II sample; and total lead in the Wooden Stairs sample.

A comparision of the total metal concentrations in the Sump 2 CSO sample to the 1-hour and 4-day objective concentrations show that four metals: total chromimum, copper, lead and zinc exceeded both objectives. A comparison of the recoverable metal concentration detected in the Sump 2 CSO to the 1-hour and 4-day objectives indicates that two metals, recoverable copper and zinc, exceeded both objectives while recoverable lead exceeded the 4-day objective. No dissolved trace metal concentrations exceeded the 1-hour or 4-day objectives for the Sump 2 CSO sample.

Total metal concentration in the CWTP effluent discharge were compared to the 1-hour and 4-day objectives. Two metals, total copper and zinc, exceeded both objectives while total chromimum and lead exceeded the 4-day objective. This pattern of exceedances was identical for the recoverable metal concentrations. Recoverable copper and zinc exceeded the objectives for both durations, and recoverable chromimum and lead exceeded only the 4-day objectives. Only dissolved zinc exceeded the 4-day objective for the CWTP effluent sample.

A.6. Quality Control Results

The laboratory split the Sacramento River water samples collected from Captain's Table dock into two batches that were analyzed separately (i.e. duplicates) as a part of the project's quality control procedures. These samples were labeled "Dup" in the laboratory results. (See Appendix A.) In the initial sampling plan, field duplicates (as opposed to sample splits) were intended to be used as the field quality control samples. However, the sample split results proved to be a

more stringent measure of laboratory precision and accuracy. Within the laboratory, the analyses were conducted "blind", i.e. the laboratory technicians were not apprised of the sample location identification names during the analysis of the samples.

Table 5-7 compares and notes variances between the parent sample results and the quality control (duplicate) results. The variance was calculated by dividing the absolute value of the difference between the parent and QC sample results by the parent sample results (expressed as a percentage). Although several of the variances appear to be significant it should be noted that these variance represent minute differences in actual analytical results. For example, dissolved copper was detected at 0.0015 mg/l in the parent sample and 0.00074 mg/l in the quality control sample using a reporting limit of 0.0005 mg/l. The calculated variance of 50.7 percent for this constituent represents a 0.00076 mg/l difference in actual analytical results. To determine if this variance was significant, a survey of several laboratories was conducted and a general guideline was established. For sample split analytical results that are detected at less than five times the reporting limit, a variance range between 50 and 75 percent is acceptable. Using this guideline, the quality control results for the analysis compared favorably with those of their parent sample. One notable exception exists. Fecal streptococcus was detected at 24,000 MPN/100 ml in the parent sample and detected at 1,300 MPN/100 ml in the quality control sample. This difference in analytical results represents a significant variance.

TABLE 5-1. RESULTS FOR SACRAMENTO RIVER AT CAPTAIN'S TABLE I FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
General Inorganics					
Nitrate as N	mg/l	.05	0.19		
Phosphorus, Total as P	mg/l	.05	0.17		
Phosphorus, Dissolved as P	mg/l	.05	0.14		
Total Dissolved Solids	mg/l	10.0	92.0		
Total Suspended Solids	mg/l	5.0	26.0		
pH	units	N/A	7.3		
Turbidity	NTU	2.0	18.3		
Hardness	mg/l	3.3	68.4		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0016	0.360	0.190
Chromium	mg/l	.0005	0.0038	0.016	0.110
Copper	mg/l	.0005	0.0036	0.0131	0.0090
Lead	mg/l	.0017	0.0020	0.0543	0.0021
Silver	mg/l	.0001	0.00013	0.0023[2]	0.0023[2]
Zinc	mg/l	.0085	0.011	0.0892	0.0808
Trace Metals (Recoverable)					
Chromium	mg/l	.0005	0.0043	0.0160	0.0110
Copper	mg/l	.0005	0.0041	0.0131	0.0090
Lead	mg/l	.0017	0.0024	0.0543	0.0021
Zinc	mg/l	.0085	0.011	0.0892	0.0808
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0016	0.360	0.190
Chromium	mg/l	.0005	0.00088	0.016	0.110
Copper	mg/l	.0005	0.0015	0.0131	0.0090
Bacteria					
Total Coliform	MPN/100ml	200	300,000		
Fecal Coliform	MPN/100ml	200	300,000		
Fecal Streptococcus	MPN/100ml	200	24,000		

1 Objective concentrations for trace metals are based on recoverable concentrations.
2 Objective is for instantaneous maximum.

TABLE 5-2. RESULTS FOR SACRAMENTO RIVER AT CAPTAIN'S TABLE II FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
General Inorganics					
Nitrate as N	mg/l	.05	0.21		
Phosphorus, Total as P	mg/l	.05	0.55		
Phosphorus, Dissolved as P	mg/l	.05	0.24		
Total Dissolved Solids	mg/l	10.0	130		
Total Suspended Solids	mg/l	5.0	38.5		
pH	units	N/A	7.5		
Turbidity	NTU	2.0	28.6		
Hardness	mg/l	3.3	70.3		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0019	0.360	0.190
Chromium	mg/l	.0005	0.0026	0.016	0.110
Copper	mg/l	.0005	0.0042	0.0131	0.0090
Lead	mg/l	.0017	0.0028	0.543	0.0021
Silver	mg/l	.0001	0.00010	0.0023[2]	0.0023[2]
Zinc	mg/l	.0085	0.016	0.0892	0.0808
Trace Metals (Recoverable)					
Chromium	mg/l	.0005	0.0024	0.0160	0.0110
Copper	mg/l	.0005	0.0045	0.0131	0.0090
Lead	mg/l	.0017	0.0026	0.0543	0.0021
Zinc	mg/l	.0085	0.012	0.0892	0.0808
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0022	0.360	0.190
Chromium	mg/l	.0005	0.00078	0.016	0.110
Copper	mg/l	.0005	0.0029	0.0131	0.0090
Bacteria					
Total Coliform	MPN/100ml	200	300,000		
Fecal Coliform	MPN/100ml	200	300,000		
Fecal Streptococcus	MPN/100ml	200	17,000		

1 Objective concentrations for trace metals are based on recoverable concentrations.
2 Objective is for instantaneous maximum.

TABLE 5-3. RESULTS FOR SACRAMENTO RIVER AT MILLER PARK I FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
General Inorganics					
Nitrate as N	mg/l	.05	0.23		
Phosphorus, Total as P	mg/l	.05	0.76		
Phosphorus, Dissolved as P	mg/l	.05	0.067		
Total Dissolved Solids	mg/l	10.0	134		
Total Suspended Solids	mg/l	5.0	29.0		
pH	units	N/A	7.5		
Turbidity	NTU	2.0	15.6		
Hardness	mg/l	3.3	72.6		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0022	0.360	0.190
Chromium	mg/l	.0005	0.0031	0.016	0.110
Copper	mg/l	.0005	0.0036	0.0131	0.0090
Silver	mg/l	.0001	0.00013	0.0023[2]	0.0023[2]
Zinc	mg/l	.0085	0.010	0.0892	0.0808
Trace Metals (Recoverable)					
Chromium	mg/l	.0005	0.0032	0.0160	0.0110
Copper	mg/l	.0005	0.0045	0.0131	0.0090
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0020	0.360	0.190
Chromium	mg/l	.0005	0.00089	0.016	0.110
Copper	mg/l	.0005	0.0011	0.0131	0.0090
Bacteria					
Total Coliform	MPN/100ml	200	5,000		
Fecal Coliform	MPN/100ml	200	5,000		
Fecal Streptococcus	MPN/100ml	200	3,000		

1 Objective concentrations for trace metals are based on recoverable concentrations.
2 Objective is for instantaneous maximum.

TABLE 5-4. RESULTS FOR SACRAMENTO RIVER AT WOODEN STAIRS FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
General Inorganics					
Biological Oxygen Demand	mg/l	3.0	4.0		
Nitrate as N	mg/l	.05	0.22		
Phosphorus, Total as P	mg/l	.05	0.64		
Phosphorus, Dissolved as P	mg/l	.05	0.60		
Total Dissolved Solids	mg/l	10.0	127		
Total Suspended Solids	mg/l	5.0	18.0		
pH	units	N/A	7.6		
Turbidity	NTU	1.0	13.4		
Hardness	mg/l	3.3	65.7		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0023	0.360	0.190
Chromium	mg/l	.0005	0.0019	0.016	0.110
Copper	mg/l	.0005	0.0040	0.0131	0.0090
Lead	mg/l	.0017	0.0022	0.0543	0.0021
Zinc	mg/l	.0085	0.011	0.0892	0.0808
Trace Metals (Recoverable)					
Chromium	mg/l	.0005	0.0024	0.0160	0.0110
Copper	mg/l	.0005	0.0037	0.0131	0.0090
Lead	mg/l	.0017	0.0019	0.0543	0.0021
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0024	0.360	0.190
Chromium	mg/l	.0005	0.00097	0.016	0.110
Copper	mg/l	.0005	0.00075	0.0131	0.0090
Bacteria					
Total Coliform	MPN/100ml	200	17,000		
Fecal Coliform	MPN/100ml	200	17,000		
Fecal Streptococcus	MPN/100ml	200	3,000		

1 Objective concentrations for trace metals are based on recoverable concentrations.

TABLE 5-5. RESULTS FOR SUMP 2 DISCHARGE FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[3] (1-Hour Average)	Objectives[3] (4-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	20.5		
Biological Oxygen Demand	mg/l	6.0	92.0		
Nitrate as N	mg/l	.05	0.35		
Total Kjeldahl Nitrogen as N	mg/l	.50	3.0		
Phosphorus, Total as P	mg/l	.10	0.63		
Phosphorus, Dissolved as P	mg/l	.05	0.18		
Total Dissolved Solids	mg/l	10.0	56.0		
Total Suspended Solids	mg/l	5.0	186		
pH	units	N/A	6.6		
Turbidity	NTU	20.0	203		
Hardness	mg/l	3.3	28.3		
Oil and Grease	mg/l	4.6	5.4		
Organics					
Tributyltin	µg/l	.001	0.024	NA[1]	0.020
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0038	0.360	0.190
Cadmium	mg/l	.0004	0.00060	0.0027	0.00090
Chromium	mg/l	.0005	0.016	0.0160	0.0110
Copper	mg/l	.0025	0.048	0.0131	0.0090
Lead	mg/l	.0034	0.068	0.0543	0.0021
Silver	mg/l	.0001	0.0018	0.0023	0.0023
Zinc	mg/l	.0085	0.18	0.0892	0.0808
Trace Metals (Recoverable)					
Chromium	mg/l	.0005	0.0059	0.0160	0.0110
Copper	mg/l	.0015	0.019	0.0131	0.0090
Lead	mg/l	.0017	0.041	0.0543	0.0021
Silver	mg/l	.0001	0.00067	0.0023[2]	0.0023[2]
Zinc	mg/l	.0085	0.11	0.0892	0.0808

TABLE 5-5. RESULTS FOR SUMP 2 DISCHARGE FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[3] (1-Hour Average)	Objectives[3] (4-Day Average)
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0029	0.360	0.190
Chromium	mg/l	.0003	0.0034	0.0160	0.0110
Copper	mg/l	.0005	0.0047	0.0131	0.0090
Silver	mg/l	.0001	0.00017	0.0023	0.0023
Zinc	mg/l	.0085	0.026	0.0892	0.0808
Bacteria					
Total Coliform	MPN/100ml	200	16,000,000		
Fecal Coliform	MPN/100ml	200	16,000,000		
Fecal Streptococcus	MPN/100ml	200	16,000,000		

1 A one-hour comparative standard for tributyltin does not exist.
2 Objective is for instantaneous maximum.
3 Objective concentrations for trace metals are based on recoverable concentration.

TABLE 5-6. RESULTS FOR CWTP EFFLUENT FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives (1-Hour Average)	Objectives (4-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	16.1		
Biological Oxygen Demand	mg/l	6.0	37.0		
Nitrate as N	mg/l	.05	0.48		
Total Kjeldahl Nitrogen as N	mg/l	.50	2.4		
Phosphorus, Total as P	mg/l	.10	0.73		
Phosphorus, Dissolved as P	mg/l	.05	0.35		
Total Dissolved Solids	mg/l	10.0	139		
Total Suspended Solids	mg/l	5.0	213		
pH	units	N/A	6.3		
Turbidity	NTU	20.0	184		
Hardness	mg/l	3.3	25.3		
Oil and Grease	mg/l	1.0	3.0		
Organics					
Tributyltin	µg/l	.001	0.040	NA[1]	0.020
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0033	0.360	0.190
Cadmium	mg/l	.0004	0.00047	0.0027	0.00090
Chromium	mg/l	.0005	0.014	0.0160	0.0110
Copper	mg/l	.0010	0.022	0.0131	0.0090
Lead	mg/l	.0017	0.034	0.0543	0.0021
Silver	mg/l	.0001	0.0018	0.0023	0.0023
Zinc	mg/l	.0085	0.15	0.0892	0.0808
Trace Metals (Recoverable)					
Chromium	mg/l	.0005	0.012	0.0160	0.0110
Copper	mg/l	.0010	0.024	0.0131	0.0090
Lead	mg/l	.0017	0.031	0.0543	0.0021
Silver	mg/l	.0001	0.0019	0.0023[2]	0.0023[2]
Zinc	mg/l	.0085	0.15	0.0892	0.0808

TABLE 5-6. RESULTS FOR CWTP EFFLUENT FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives (1-Hour Average)	Objectives (4-Day Average)
Trace Metals (Dissolved)					
Chromium	mg/l	.0005	0.0017	0.0160	0.0110
Copper	mg/l	.0005	0.0078	0.0131	0.0090
Silver	mg/l	.0001	0.00091	0.0543	0.0021
Zinc	mg/l	.0085	0.082	0.0892	0.0808
Bacteria					
Total Coliform	MPN/100ml	200	1,700		
Fecal Coliform	MPN/100ml	200	1,700		
Fecal Streptococcus	MPN/100ml	200	ND		

1 A one-hour comparative standard for tributyltin does not exist.
2 The comparative standard for silver is calculated as the instantaneous maximum.

TABLE 5-7. QUALITY CONTROL RESULTS FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Parent Sample Results	QC Sample Results	Variance[1]
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	ND	ND	NC
Biological Oxygen Demand	mg/l	3.0	ND	ND	NC
Nitrate as N	mg/l	0.05	0.19	0.19	0%
Nitrite as N	mg/l	0.05	ND	ND	NC
Total Kjeldahl Nitrogen	mg/l	0.5	ND	ND	NC
Phosphorus, Total as P	mg/l	0.05	0.17	0.17	0%
Phosphorus, Dissolved as P	mg/l	0.05	0.14	0.14	0%
Total Dissolved Solids	mg/l	5.0	26.0	26.0	0%
Total Suspended Solids	mg/l	10.0	92.0	116.0	26.1%
Turbidity	NTU	2.0	18.3	18.8	2.7%
pH	Units	NA	7.3	7.5	2.7%
Hardness	mg/l	3.3	68.4	69.5	1.6%
Oil & Grease	mg/l	1.0	ND	ND	NC
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0016	0.0018	12.5%
Beryllium	mg/l	.005	ND	ND	NC
Cadmium	mg/l	.0004	ND	ND	NC
Chromium	mg/l	.0005	0.0038	0.0042	10.5%
Cobalt	mg/l	.01	ND	ND	NC
Copper	mg/l	.0005	0.0036	0.0039	8.3%
Lead	mg/l	.0017	0.0020	0.0024	20%
Mercury	mg/l	.0004	ND	ND	NC
Selenium	mg/l	.001	ND	ND	NC
Silver	mg/l	.0001	0.00013	ND	30.8[2]
Zinc	mg/l	.0085	0.011	0.011	0%
Trace Metals (Recoverable)					
Arsenic	mg/l	.005	ND	ND	NC
Beryllium	mg/l	.002	ND	ND	NC

TABLE 5-7. QUALITY CONTROL RESULTS FOR FEBRUARY 12, 1992 EVENT

Constituents	Units	Reporting Limit	Parent Sample Results	QC Sample Results	Variance[1]
Cadmium	mg/l	.0004	ND	ND	NC
Chromium	mg/l	.0005	0.0043	0.0030	30.2%
Cobalt	mg/l	.01	ND	ND	NC
Copper	mg/l	.0005	0.0041	0.0039	4.9%
Lead	mg/l	.0017	0.0024	0.0021	12.5%
Selenium	mg/l	.005	ND	ND	NC
Silver	mg/l	.0001	ND	ND	NC
Zinc	mg/l	.0085	0.011	0.0098	10.9%
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0016	0.0015	6.3%
Beryllium	mg/l	.002	ND	ND	NC
Cadmium	mg/l	.0004	ND	ND	NC
Chromium	mg/l	.0005	0.00088	0.0012	36.4%
Cobalt	mg/l	.01	ND	ND	NC
Copper	mg/l	.0005	0.0015	0.00074	50.7%
Lead	mg/l	.0017	ND	ND	NC
Mercury	mg/l	.0002	ND	ND	NC
Nickel	mg/l	.0025	ND	ND	NC
Selenium	mg/l	.001	ND	ND	NC
Silver	mg/l	.0001	ND	ND	NC
Zinc	mg/l	.0085	ND	ND	NC
Bacteria					
Total Coliform	MPN/100ml	200	300,000	300,000	0%
Fecal Coliform	MPN/100ml	200	300,000	300,000	0%
Fecal Streptococcus	MPN/100ml	200	24,000	1,300	94.6%

1 Calculation for variance is as follows: absolute value of the difference between the parent and QC sample results divided by the parent sample result, expressed as a percentage.
2 Calculation of variance is based on a parent sample value of 0.00009 mg/l.
ND Not detected.
NC Variance cannot be calculated for this pair of analytical results.

TABLE 5-8. ORGANOCHLORINE PESTICIDE/PCB SUMMARY AND COMPARISON[4] FOR SUMP 2 CSO SAMPLE					
Constituent	Reporting Limit and "ND" Concentration[1] (ng/l)[3]	Objective Concentrations (ng/l)[3]		Exceedence Summary for Objective Concentrations[2]	
		Daily Average	Instantaneous Maximum	Daily Average	Instantaneous Maximum
chlordane	3.3	4.3	-----	N	-----
DDT[5]	0.7	1.0	-----	N	-----
dieldrin	0.7	1.9	-----	N	-----
endosulfan	0.7[6]	56.0	220.0	N	N
endrin	0.7	2.3	180.0	N	N
heptachlor	0.8	3.8	-----	N	-----
hexachlorocyclo-hexanegamma	0.41	80.0	-----	N	-----
PCBs	16.7[7]	14.0	-----	U	-----

1 For Sacramento River downstream of Sump 2 outfall, assumes 150:1 dilution of Sump 2 CSO sample reporting limit, for which all results were "ND".

2 Key: N = ND value is less than the objective concentration.
U = Unknown as to whether objective concentration would be exceeded.

3 ng/l = nanograms per liter or parts per trillion.

4 See Table 4-1 analytes. See text for discussion of tributyltin results.

5 Results identical for DDD and DDE.

6 Estimate is for endosulfan II and sulfate. Endosulfan I estimate is 0.3 ng/l.

7 PCBs consist of 7 compounds; highest estimate used.

TABLE 5-9. ORGANOCHLORINE PESTICIDE/PCB SUMMARY AND COMPARISON[4] FOR CWTP EFFLUENT SAMPLE					
Constituent	Reporting Limit and "ND" Concentration[1] (ng/l)[3]	Objective Concentrations (ng/l)[3]		Exceedence Summary for Objective Concentrations[2]	
		Daily Average	Instantaneous Maximum	Daily Average	Instantaneous Maximum
chlordane	12.0	4.3	-----	U	-----
DDT[5]	2.5	1.0	-----	U	-----
dieldrin	1.2	1.9	-----	N	-----
endosulfan	2.5[6]	56.0	220.0	N	N
endrin	1.2	2.3	180.0	N	N
heptachlor	1.2	3.8	-----	N	-----
hexachlorocyclo-hexanegamma	0.62	80.0	-----	N	-----
PCBs	25.0[7]	14.0	-----	U	-----

1 For Sacramento River downstream of CWTP outfall, assumes 150:1 dilution of CWTP effluent CSO sample reporting limit, for which all results were "ND".

2 Key: N = ND value is less than the objective concentration.
U = Unknown as to whether objective concentration would be exceeded.

3 ng/l = nanograms per liter or parts per trillion.

4 See Table 4-1 analytes. See text for discussion of tributyltin results.

5 Results identical for DDD. DDE estimate is 1.2 ng/l.

6 Estimate is for endosulfan II and sulfate. Endosulfan I estimate is 1.2 ng/l.

7 PCBs consist of 7 compounds.

SECTION 6
CHRONIC TOXICITY SAMPLE RESULTS

This section summarizes the toxicity sampling and analysis program, sample test results, and quality assurance/quality control results for the storm event of February 12, 1992.

A. BIOTOXICITY TESTING PROGRAM OVERVIEW

Provision 5 of the City's NPDES Permit requires that a biotoxicity study be completed for CWCS discharges, using 100-percent solutions of CWTP effluent and CSO discharges and the EPA's three-species test for chronic toxicity. If chronic toxicity was found in the 100-percent samples collected during the 1991/1992 wet weather season, then the City would be required perform a dilution series analysis for the corresponding sampling locations during the 1992/1993 wet season.

During this storm event, chronic toxicity analyses were performed on 100-percent solutions of Sump 2 CSO and CWTP effluent discharges. Pioneer Reservoir did not overflow during this event, so sample material was not collected for this discharge.

The City voluntarily accelerated the toxicity testing program on CWTP effluent. During this CSO event, the City performed a chronic toxicity dilution series of CWTP effluent. Dilutions were prepared for the following effluent percentages in accordance with EPA protocols: 100, 50, 25, 12.5, 6.2. Dilutions of 3.2 and 1.6 percent were added to the series, based on the December 7, 1991 sampling event toxicity test results and the anticipated need for lower dilutions. These dilutions of the CWTP effluent were analyzed to define the No Observable Effect Concentration (NOEC) and the Lowest Observable Effect Concentration (LOEC) points for each of the biological test species. The dilution series was created by proportioning CWTP effluent and Sacramento River water (collected at Captain's Table dock during the storm event and prior to upstream discharges). In order to determine baseline chronic toxicity for the CWTP dilution series, 100-percent

river water was also analyzed for toxicity.

The following tests were performed on the 100-percent discharge (Sump 2 and CWTP) and Sacramento river samples, and on the CWTP effluent dilutions:

1. A 7-day, chronic fathead minnow (*Pimephales promelas*), static renewal, larval survival and growth test.
2. A 3-brood, 7-day, chronic cladoceran (*Ceriodaphnia dubia*), static renewal, survival and reproduction test.
3. A 4-day, chronic, algal (*Selenastrum capricornutum*), static, growth test.

Procedures for determining biotoxicity are defined in EPA 600/4-89/001 (*Short-term Methods for Estimating the Chronic Toxicity of Effluent and Receiving Water to Freshwater Organisms*) and EPA 440/4-85/032 (*Technical Support Document for Water Quality Based Toxic Control*). The EPA's three-species freshwater chronic toxicity tests were performed as specified in these protocals and NPDES permit. Methods numbers are summarized in Table 6-1.

TABLE 6-1. CHRONIC TOXICITY TEST PARAMETERS AND METHOD NUMBERS

Test Specie	Type of Test	EPA Method
Fathead Minnow	Survival/Growth	1000
Ceriodaphnia (water flea)	Survival/Reproduction	1002
Selenastrum (algae)	Growth	1003

B. SAMPLE COLLECTION AND HANDLING

Toxicity sample collection for the Sump 2 and CWTP effluent discharges was conducted similar to the inorganic constituent sample collection. Discrete samples were composited in 30-minute periods, such that each period throughout the discharge would be represented by an equal volume. The samples were

refrigerated during transport to Aquatic Testing Laboratories in Ventura. All tests were initiated within the required holding time limits.

C. ANALYTICAL RESULTS

Aquatic Testing Laboratories analyzed the biotoxicity of the effluent in the dilution percentages as previously described. Data collected from laboratory test results were entered into a data base for statistical analysis. EPA's TOXSTAT computer program was used to conduct statistical tests and to estimate the NOEC and LOEC endpoints. The three-species freshwater chronic toxicity test results are summarized in the following sections and tables.

C.1. Sacramento River Sample Results

The chronic toxicity of Sacramento River water was tested to provide baseline data for the CWTP dilution series tests. Test results were compared to the laboratory control for determining whether chronic toxicity was present in the river during this storm event at the Captains Table dock location that could influence the results of the dilution series tests. Fathead minnow survival and growth, *Ceriodaphnia* survival and reproduction, and *Selenastrum* growth rates were not significantly adversely affected by the 100-percent solution of Sacramento River water collected from the Captain's Table dock, as shown in Tables 6-2 through 6-7. It was therefore concluded that the river sample was not chronically toxic to any of the three species. The river water was then used as the control for analyzing the CWTP dilution series results.

TABLE 6-2. CHRONIC TOXICITY OF SACRAMENTO RIVER WATER ON FATHEAD MINNOWS

Sample	Percent Survival	Larval Mean Dry Weight
Control-Lab	100.0%	0.45 mg
Sacramento River	100.0%	0.40 mg

Control-Lab is a laboratory control water (20% DMW).

Sacramento River water collected from the Captain's Table dock, prior to upstream CSO discharges.

TABLE 6-3. SUMMARY OF SACRAMENTO RIVER CHRONIC TOXICITY ENDPOINTS FOR FATHEAD MINNOWS		
Endpoint	**Survival**	**Growth**
NOEC	NS	NS
NS is not significantly less than control.		

TABLE 6-4. CHRONIC TOXICITY OF SACRAMENTO RIVER WATER ON CERIODAPHNIA		
Sample	**Percent Survival**	**Mean Number of Young per Female**
Control-Lab	100.0%	15.9
Sacramento River	90.0%	16.0
Control Lab is laboratory control water (20% DMW). Sacramento River water collected from the Captain's Table dock.		

TABLE 6-5. SUMMARY OF SACRAMENTO RIVER WATER CHRONIC TOXICITY ENDPOINTS FOR CERIODAPHNIA		
Endpoint	**Survival**	**Reproduction**
NOEC	NS	NS
NS is not significantly less than control.		

TABLE 6-6. CHRONIC TOXICITY OF SACRAMENTO RIVER WATER ON SELENASTRUM	
Sample	**Mean Fluorescence**
Control-Lab	141.5
Sacramento River	369.8
Relative fluorescence values are in-vivo relative content measurements taken with a fluorometer equipped with a chlorophyll filter. Fluorescence value of 210 is approximately 206,000 cells/ml based on actual cell counts of the control test solution. Control-Lab is laboratory control (20% DMW). Sacramento River water collected from the Captain's Table dock.	

TABLE 6-7. SUMMARY OF SACRAMENTO RIVER WATER CHRONIC TOXICITY FOR SELENASTRUM	
NOEC = NS	LOEC = NA
NS is not significantly less than control. NA is not applicable.	

C.2. Sump 2 CSO Discharge Results

Laboratory water was used as the control to assess the chronic toxicity for the Sump 2 CSO discharge sample. Fathead minnow survival and growth were not significantly affected in the 100-percent solution of Sump 2 discharge as shown in Tables 6-8 and 6-9. *Ceriodaphnia* survival and reproduction were both significantly affected in the 100-percent discharge solution, as summarized in Tables 6-10 and 6-11. Results for *Selenastrum* growth (Tables 6-12 and 6-13) corresponded to that of the fathead minnow test; no significant, adverse impacts occurred for the specie. Therefore, the 100-percent Sump 2 discharge collected during this event was chronically toxic to *Ceriodaphnia* growth and reproduction, but not to fathead minnows or *Selenastrum*.

TABLE 6-8. CHRONIC TOXICITY OF SUMP 2 DISCHARGE ON FATHEAD MINNOWS		
Sample	Percent Survival	Larval Mean Dry Weight
Control-Lab	100.0%	0.45 mg
Sump 2 Discharge	92.5%	0.39 mg
Control-Lab is a laboratory control water (20% DMW). Sump 2 Discharge is a 100% CSO discharge solution.		

TABLE 6-9. SUMMARY OF SUMP 2 DISCHARGE CHRONIC TOXICITY ENDPOINTS FOR FATHEAD MINNOWS		
Endpoint	Survival	Growth
NOEC	NS	NS
NS is not significantly less than control.		

TABLE 6-10. CHRONIC TOXICITY OF SUMP 2 DISCHARGE ON CERIODAPHNIA		
Sample	**Percent Survival**	**Mean Number of Young per Female**
Control-Lab	100.0%	15.9
Sump 2 Discharge	20.0%*	11.0*

Control-Lab is laboratory control water (20% DMW)

Sump 2 Discharge is a 100% CSO discharge solution.

* Significantly less than control.

TABLE 6-11. SUMMARY OF SUMP 2 DISCHARGE CHRONIC TOXICITY ENDPOINTS FOR CERIODAPHNIA		
Endpoint	**Survival**	**Reproduction**
NOEC	SL	SL

SL is significantly less than control.

TABLE 6-12. CHRONIC TOXICITY OF SUMP 2 DISCHARGE ON SELENASTRUM	
Effluent Concentration	**Mean Fluorescence**
Control-Lab	141.5
Sump 2 Discharge	406.2

Relative fluorescence values are in-vivo relative content measurements taken with a fluorometer equipped with a chlorophyll filter.

Fluorescence value of 210 is approximately 206,000 cells/ml based on actual cell counts of the control test solution.

Control-Lab is laboratory control (20% DMW)

Sump 2 Discharge is a 100% CSO discharge solution.

TABLE 6-13. SUMMARY OF SUMP 2 DISCHARGE CHRONIC TOXICITY FOR SELENASTRUM	
NOEC = NS	LOEC = NA

NS is not significantly less than control.

NA is not applicable.

C.3. CWTP Effluent Sample Results

Tables 6-14 and 6-15 show that fathead minnow survival and growth were not siginificantly affected in a 100-percent effluent solution, therefore, an NOEC of 100-percent effluent was determined.

Ceriodaphnia survival was significantly affected in a 100-percent effluent solution, therefore, a survival NOEC of 50-percent effluent was determined for this specie. Reproduction was significantly affected in 50-percent effluent solution, therefore, a reproduction NOEC of 25-percent effluent was determined for this specie, as shown in Tables 6-16 and 6-17. Based on the reproduction results, the 50-percent effluent event sample was chronically toxic to *Ceriodaphnia*.

Selenastrum growth rates were significantly affected in a 12.6-percent solution of CWTP effluent but were not a 6.2-percent solution as shown in Table 6-19. The NOEC was determined at 6.2-percent and the LOEC was determined at 12.5-percent for this specie as shown in Table 6-18.

TABLE 6-14. CHRONIC TOXICITY OF CWTP EFFLUENT ON FATHEAD MINNOWS

Sample Concentration	Percent Survival	Larval Mean Dry Weight
Control (river water)	97.5%	0.43 mg
1.6% Effluent	100.0%	0.40 mg
3.2% Effluent	100.0%	0.42 mg
6.2% Effluent	100.0%	0.46 mg
12.5% Effluent	100.0%	0.40 mg
25.0% Effluent	100.0%	0.45 mg
50.0% Effluent	100.0%	0.42 mg
100.0% Effluent	100.0%	0.39 mg

Control is the dilution water control collected from the river at the Captain's Table dock.

TABLE 6-15. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY ENDPOINTS FOR FATHEAD MINNOWS		
Endpoints	**Survival**	**Growth**
NOEC	100.0%	100.0%
LOEC	NA	NA
NA is not applicable.		

TABLE 6-16. CHRONIC TOXICITY OF CWTP EFFLUENT ON CERIODAPHNIA		
Sample	**Percent Survival**	**Mean Number of Young per Female**
Control	100.0%	12.8
1.6% Effluent	100.0%	17.7
3.2% Effluent	100.0%	18.6
6.2% Effluent	90.0%	20.3
12.5% Effluent	100.0%	15.5
25.0% Effluent	90.0%	14.0
50.0% Effluent	90.0%	4.7*
100.0% Effluent	0%*	0*
Control is the dilution water control collected from the river at the Captain's Table dock.		
* Significantly less than control.		

TABLE 6-17. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY ENDPOINTS FOR CERIODAPHNIA		
Endpoint	**Survival**	**Reproduction**
NOEC	50.0%	25.0%
LOEC	100.0%	50.0%

TABLE 6-18. CHRONIC TOXICITY OF CWTP EFFLUENT ON SELENASTRUM	
Effluent Concentration	Mean Fluorescence
Control	447.5
1.6% Effluent	500.2
3.2% Effluent	385.8
6.2% Effluent	399.2
12.5% Effluent	310.0*
25.0% Effluent	315.8*
50.0% Effluent	279.5*
100.0% Effluent	300.0*

Relative fluorescence values are in-vivo relative content measurements taken with a fluorometer equipped with a chlorophyll filter.

Fluorescence value of 210 is approximately 206,000 cells/ml based on actual cell counts of the control test solution.

Control is dilution water collected from the river at the Captain's Table dock.

* Significantly less than control.

TABLE 6-19. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY RESULTS FOR SELENASTRUM	
NOEC = 6.2%	LOEC = 12.5%

D. QUALITY ASSURANCE/QUALITY CONTROL

The three test species were tested with reference toxicants to determine whether the cultured species exhibited normal toxicity. Reference toxicant data are included in Appendix B, submitted as a separate document. Fathead minnows were tested with a reference toxicant of sodium dodecyl sulfate (SDS). The SDS NOEC for fathead minnow survival and growth was 2.5 mg/l. Ceriodaphnia were tested with a reference toxicant of sodium chloride (NaCl). The NaCl NOEC point for Ceriodaphnia was 1,000 mg/l for both survival and reproduction. Selenastrum were tested with a reference toxicant of copper sulfate. The chronic toxicity NOEC

point for Selenastrum was 0.010 mg/l. The reference toxicant analyses indicated that the three species exhibited a "normal" toxic response to their respective toxicants.

The control solution used for the biotoxicity analysis was Sacramento River water collected from Captain's Table dock during this CSO event. None of the species used in the analysis were adversely affected by conditions in the control sample waters.

SECTION 7
SUMMARY

This section provides a summary of the results for the February 12, 1992 storm/CSO event. During this event, CWTP discharged effluent and Sump 2 overflowed to the Sacramento River.

A. OBJECTIVES MET BY THE CSO EVENT

Based on the goals established for the 1991/1992 CWCS Sampling Program, the following objectives were met by the CSO event:

1) The chronic toxicity of Sacramento River water, the Sump 2 discharge, and CWTP effluent was determined for event samples on three test species (fathead minnows, *Ceriodaphnia*, and *Selenastrum*). These test samples were undiluted, i.e. 100 percent river water or discharge samples. Toxicity tests for dilutions of the CWTP effluent were conducted for the following percentages of effluent: 100, 50, 25, 12.5, 6.2, 3.2, and 1.2. River water was used for preparing the dilution series test samples. The objective was to determine the significant impact, from a statistical standpoint, of sample waters on each of the three species on critical life stages. The NOEC and LOEC represent this statistical significance.

2) The effect of pollutants (inorganics, bacteria and metals) discharged to the Sacramento River from Sump 2 and CWTP were measured by collecting and analyzing river water samples collected in the respective discharge plumes. The "effect" or impact was determined by comparing upstream, background river sample constituent concentrations to those downstream of the discharges, and by comparing analytical results for the river to the *Inland Surface Waters Plan.* The concentrations of pollutants in the Sump 2 discharge and CWTP effluent were measured by collecting and analyzing samples of these matrices.

3) The presence of organochlorine pesticides and PCBs in Sump 2 and CWTP discharges was measured by collecting and analyzing samples during this

CSO event. A special analysis was performed for tributyltin.

B. SUMMARY OF RESULTS

B.1. Chronic Toxicity

EPA's three-species, freshwater chronic toxicity tests were used as specified in the NPDES permit. EPA's TOXSTAT computer program was used to perform the statistical computations on the laboratory results.

B.1.a. Sacramento River Sample--

An analysis of biotoxicity of the Sacramento River water sample was conducted to determine a baseline for the control waters used in the CWTP effluent dilution series. Fathead minnow survival and growth, *Ceriodaphnia* survival and reproduction, and *Selenastrum* growth rates were not significantly affected in a 100-percent Sacramento River water sample collected at the Captain's Table dock. Therefore, chronic toxicity was not present. The river water therefore served as the control for evaluating CWTP dilution series results.

B.1.b. Sump 2 Sample--

Fathead minnow survival and growth, and *Selenastrum* growth rates were not affected in a 100-percent solution of Sump 2 discharge. *Ceriodaphnia* survival and reproduction were both affected in a 100-percent discharge solution, therefore, survival and reproduction were determined to be significantly less than the control for this specie.

B.1.c. CWTP Effluent Sample--

Toxicity tests were conducted for specified dilutions of the CWTP effluent. Fathead minnow survival and growth were not affected in a 100-percent CWTP effluent solution, therefore, the NOEC for both survival and growth was determined to be at the 100-percent effluent level.

Ceriodaphnia survival was significantly affected in a 100-percent effluent solution, therefore, a survival NOEC at 50-percent effluent and an LOEC at 100-percent were determined. *Ceriodaphnia* reproduction was significantly affected in

a 50-percent effluent solution, therefore, a reproduction NOEC at 25-percent effluent and an LOEC at 50-percent effluent were determined. *Selenastrum* growth rates were significantly affected in a 12.6-percent solution of CWTP effluent. An NOEC at 6.2 percent effluent and LOEC at 12.6 percent effluent were determined.

The safe concentration for these particular samples, species and critical life stages is between the NOEC and LOEC. This concentration is biologically rather than statistically significant. Assuming *Selenastrum* to be the most sensitive specie in the CWTP effluent dilution series tests, a 16:1 river dilution of effluent would be needed, conservatively, to ensure normal propagation. During this event, the river provided approximately 100:1 dilution for CWTP effluent.

Since a dilution series analysis was not performed on the Sump 2 CSO samples, the safe concentration in the river cannot be determined. In this sample analysis, it can only be concluded that *Ceriodaphnia* were significantly affected by the 100-percent solution collected during this event. At Sump 2, the river provided a dilution ratio of approximately 150:1.

B.2. General Inorganics

Comparison between the sample collected upstream of the Sump 2 discharge (Miller Park I) and the sample collected in the effluent discharge plume of Sump 2 (Captain's Table II) indicates comparable analytical results. The results from the Wooden Stairs sample collected in the effluent discharge plume from CWTP are comparable to the upstream samples results collected from Captain's Table I. This suggests that the Sump 2 discharge and CWTP effluent had an insignificant overall effect on the Sacramento River inorganic water chemistry.

Comparing the results from the Captain's Table I sample, collected upstream of the effluent discharge from CWTP, and Captain's Table II sample, collected in the Sump 2 discharge plume, indicates a very slight increase in all of the detected inorganic constituents in the latter sample.

B.3. Bacteria

The bacteria results of the river samples do not suggest a uniform trend that can be correlated to pre- and post-discharges of either Sump 2 or the CWTP. This may be the result of stormwater discharges to the river that enter upstream of the sampling locations.

Bacteria counts in the CWTP samples were considerably lower than Sump 2 discharge samples which reflects the effective chlorination treatment at the CWTP facility.

B.4. Organochlorine Pesticides/PCBs and Tributyltin

None of the organochlorine pesticide/PCB constituents were detected in samples of Sump 2 discharge or CWTP effluent, using standard EPA method analyses. However, the constituents were not analyzed within the required two-week holding time.

A comparison of the tributyltin concentrations detected in the Sump 2 CSO and CWTP effluent discharge to the 4-day objective concentration derived from the *Inland Surface Waters Plan*, shows that this analyte exceeded the objective. (Tributyltin does not have a 1-hour objective concentration.)

B.5. Oil and Grease

Oil and grease concentrations were not detected in any river samples, but were found to be 5.4 mg/l and 3.0 mg/l, in the Sump 2 discharge and CWTP effluent, respectively.

B.6. Trace Metals

Analytical results for the river and discharge samples were compared to the *Inland Surface Waters Plan* objective concentrations for trace metals, as presented in Table 4-1. Although the objectives represent recoverable concentration, comparison was also made to total and dissolved metal concentrations detected in the event samples.

A comparison of trace metal concentrations (total, recoverable and dissolved) detected in the Sacramento River to the 1-hour averge objective concentrations listed in Table 4-1 shows that no trace metal exceeded those objectives. A comparison of the trace metal concentrations detected in the Sacramento River to the 4-day average objective concentration shows that total and recoverable lead exceeded the 4-day objective at various sampling locations: recoverable lead in Captain's Table I sample; total and recoverable lead in the Captain's Table II sample; and total lead in the Wooden Stairs sample.

A comparision of the total metal concentrations in the Sump 2 CSO sample to the 1-hour and 4-day objective concentrations show that four metals: total chromimum, copper, lead and zinc exceeded both objectives. A comparison of the recoverable metal concentration detected in the Sump 2 CSO to the 1-hour and 4-day objectives indicates that two metals, recoverable copper and zinc, exceeded both objectives while recoverable lead exceeded the 4-day objective. No dissolved trace metal concentrations exceeded the 1-hour or 4-day objectives for the Sump 2 CSO sample.

Total metal concentration in the CWTP effluent discharge were compared to the 1-hour and 4-day objectives. Two metals, total copper and zinc, exceeded both objectives while total chromimum and lead exceeded the 4-day objective. This pattern of exceedances was identical for the recoverable metal concentrations. Recoverable copper and zinc exceeded the objectives for both durations, and recoverable chromimum and lead exceeded only the 4-day objectives. Only dissolved zinc exceeded the 4-day objective for the CWTP effluent sample.

APPENDIX D

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 4

EVENT REPORT FOR THE FEBRUARY 14, 1992, SAMPLING EVENT

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September 1, 1992

CITY OF SACRAMENTO
COMBINED SEWER SYSTEM
IMPROVEMENT AND IMPLEMENTATION PLAN

PHASE 3 STUDY

CWCS SAMPLING PROGRAM FOR 1991/1992

WATER QUALITY AND TOXICITY RESULTS FOR THE FEBRUARY 14, 1992 SAMPLING EVENT

Prepared for

City of Sacramento
Utilities Department
Engineering Division

Prepared by

HDR Engineering, Inc.
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Sacramento, CA

September 1, 1992

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SECTION 1
PROGRAM OVERVIEW AND OBJECTIVES

A. INTRODUCTION

This report summarizes the results for the February 14, 1992 sampling event. As background for reviewing these results, this section provides a brief overview of the sampling and analysis plan for the Combined Wastewater Collection and Treatment System (CWCS) and Sacramento River for the 1991/1992 wet season. This program parallels, but has been conducted separately from the "routine" monitoring program specified in the City's NPDES Permit No. CA0079111 for the CWCS. This program was conducted as part of the Phase 3 study to develop an improvement and implementation plan for the Sacramento combined sewer system.

Four key elements were used to develop a water quality sampling program for the 1991/1992 wet weather season: 1) the results of the 1990/1991 wet weather sampling for two storm/combined sewer overflow (CSO) events; 2) the NPDES permit for the CWCS, which requires that biotoxicity tests be performed on CWCS "effluent" samples, including Combined Wastewater Treatment Plant (CWTP) effluent and CSOs from Pioneer Reservoir and Sump 2; 3) a request by the RWQCB staff to conduct a matrix detection limit study for lead; and 4) a request by the RWQCB to include a screen for the pesticides and PCBs included in the *California Inland Surface Waters Plan*, "Objectives for Protection of Freshwater Aquatic Life". (See Section 4 for details on these objective concentrations.)

B. OBJECTIVES

Based on the elements described above, the objectives for the 1991/1992 CWCS Sampling Program were as follows:

1) Determine the chronic toxicity of CWTP effluent, Sump 2 overflows and Pioneer Reservoir overflows. Determine the seasonal variation of toxicity, if possible, by sampling an early-season CWTP "operation-only" event, as well as two additional storm events that are large enough to both require

operation of CWTP and produce CSOs to the Sacramento River. The objective was to sample CSOs from each of the outfall locations, even if overflows were not necessarily simultaneous at Sump 2 and Pioneer Reservoir. Determine the No Observable Effect Concentration (NOEC) and Lowest Observable Effect Concentration (LOEC) for CWTP effluent by performing toxicity tests on dilutions of effluent for all three of the events sampled. For the CSO samples (Sump 2 and Pioneer Reservoir), determine toxicity on a "screening level" basis, by testing the undiluted (100-percent) samples. If toxicity was found in the 100-percent CSO samples, then dilution series chronic tests would be performed during the subsequent 1992/1993 wet season.

2) Sample local street floodwater and analyze for bacteria content, in order to estimate the source of bacteria contamination and, if possible, to identify locations having consistent sewage outflows onto streets during storm events. It was intended that City staff would complete the work related to this objective.

3) Determine the impact of trace metal pollutant discharges on the Sacramento River. Collect and analyze samples from the Sacramento River at appropriate locations during two storm events large enough to require CWTP operation and overflows from Pioneer Reservoir and/or Sump 2. If overflows from a particular event did not occur simultaneously at Sump 2 and Pioneer Reservoir, samples would be collected for the one CSO location. (Ultimately, the objective was to capture two CSO's at each location.) The sampling related to CSO impacts would be concurrent with the CWTP discharge sampling events, for at least two CSO events. Combined wastewater would be characterized for Pioneer Reservoir overflows, Sump 2 overflows (total station flow greater than 190 mgd), and CWTP effluent.

4) Determine the presence of organochlorine pesticides and polychlorinated biphenyls (PCBs) in CWCS discharge samples, using standard screening analysis methods. In addition, determine the presence of tributyltin. Tributyltin is an organic compound included in the previously mentioned objective concentrations for aquatic life, and requires special analytical

methods. Testing for these constituents in the CSO and effluent samples rather than the river would yield "worst case" results with respect to river impacts, because the river dilutes the discharge.

5) Perform a matrix detection limit (MDL) study for lead concentrations in the four unique matrices being sampled in the CWCS. The four matrices involved are: the Sacramento River in the vicinity of CSO discharges, CWTP effluent, Sump 2 overflow, and Pioneer Reservoir overflow. The objective was to determine the repeatable reporting limit concentrations that could be achieved for these matrices using standard analysis methods.

C. OBJECTIVES MET BY THE CSO SAMPLING EVENT

This report documents the findings of the CSO event that occurred on February 14, 1992, and is the last in a series of three reports for storm/CSO events sampled during the 1991/1992 wet weather season. During this event, CWTP discharged effluent to the Sacramento River and a CSO occurred from Sump 2. No discharge occurred from Pioneer Reservoir.

C.1. Chronic Toxicity

Provision 5 of the City's NPDES permit requires that a biotoxicity study be completed on 100-percent effluent samples during the 1991/1992 wet season. If toxicity is present, the City would be required to perform a dilution series as part of its 1992/1993 monitoring plan. The City voluntarily accelerated this schedule in the case of the CWTP effluent biotoxicity tests, by performing dilution series tests on CWTP effluent samples under the 1991/1992 monitoring program. The toxicity of CWTP effluent dilutions, which define the LOEC and NOEC dilutions, were analyzed in accordance with EPA protocols that specify the following dilution percentages: 100, 50, 25, 12.5 and 6.2. Two additional dilutions of 3.2 and 1.6 percent effluent were also analyzed for all three test species. As per the EPA protocols, dilution water was collected from the river upstream of the CWTP outfall at Captain's Table dock, and tested for chronic toxicity.

The 100-percent solution of combined wastewater discharged from Sump 2 was analyzed for chronic toxicity. This analysis was conducted to provide baseline toxicity information for this CWCS sample matrix.

The Sacramento River water collected at Captain's Table dock for preparing the CWTP effluent dilution series was analyzed separately for chronic toxicity. This analysis was conducted to provide baseline or "control" information on Sacramento River water used for the dilution series.

Procedures for determining if toxicity is present are defined in EPA 600/4-89/001 (*Short-term Methods for Estimating the Chronic Toxicity of Effluent and Receiving Water to Freshwater Organisms*) and EPA 440/4-85/032 (*Technical Support Document for Water Quality Based Toxic Control*). EPA's three-species freshwater chronic toxicity test procedures were used, as specified in the CWCS NPDES permit and in Table 1-1.

TABLE 1-1. BIOTOXICITY TEST PARAMETERS

Test Specie	Type of Test	EPA Method
Fathead Minnow	Survival/Growth	1000
Ceriodaphnia (water flea)	Survival/Reproduction	1002
Selenastrum (algae)	Growth	1003

The biotoxicity analyses completed for these effluent, CSO and river samples fulfills part of Objective 1 for the 1991/1992 CWCS Sampling Program. Any conclusions on the toxicity of CWTP effluent and system CSOs can only be made from the analysis of multiple events. A summary report for all three sampling events conducted during the 1991/1992 wet season will present conclusions, if any, on the consistent and/or seasonal presence of chronic toxicity.

C.2. Bacteria Content of Street Floodwaters

The February 14, 1992 storm event was, in general, a low-intensity storm with several periods of high-intensity rainfall but localized flooding was minimal. Therefore, the City staff did not collect bacteria count samples of local street floodwaters as part of this sampling event.

C.3. Trace Metal Pollutants

The collection and analysis of water quality samples was conducted to determine the presence of trace metal pollutants in the Sacramento River during a CSO/CWTP effluent discharge event. These were collected from the river at the Captain's Table dock, Miller Park dock, and the "Wooden Stairs" location. The Miller Park dock samples provided the river background chemistry for estimating CWCS discharge impacts. Water quality samples were collected and analyzed for the dilution water collected from the Captain's Table dock. Analyses were also completed for discharges from Sump 2, and for CWTP effluent.

The trace metal analysis fulfilled the requirements of Objective 3 for the 1991/1992 CWCS Sampling Program.

C.4. Organochlorine Pesticides/PCBs and Tributyltin

The collection and analysis of samples was conducted to determine the presence of organochlorine pesticides and PCBs in the Sump 2 CSO and CWTP effluent discharges. Samples were also collected from these discharges to determine the presence of tributyltin.

The organochlorine pesticides/PCBs and tributyltin analysis fulfilled the requirements of Objective 4 for the 1991/1992 CWCS Sampling Program. However, the holding time for these samples was exceeded. The impacts of this laboratory error are described in Section 3.

C.5. Lead MDL Study

Samples for conducting the lead MDL study were not collected during this event. This study was completed as part of the February 12, 1992 event. The findings of the study will be submitted in a separate report.

C.6. Other Constituents

Sacramento River waters (including water collected for CWTP effluent dilutions), and Sump 2 CSO and CWTP effluent discharges were analyzed for bacteriological and inorganic constituents.

D. REPORT OVERVIEW

This report is organized into seven major sections. This section summarized the objectives for 1991/1992 CWCS Sampling Program, conducted during the 1991/1992 wet weather season. Section 2 describes the event sampling program, including the field methods, sample collection and handling procedures, and laboratory quality assurance procedures. In addition, this section presents the general sample and analysis matrix and the specific constituents analyzed for the CSO event samples. Section 3 summarizes the storm event, sample collection timing, field data results, and the results of the quality assurance analyses conducted by the laboratory. Section 4 describes the derivation of the water quality objective concentrations based on the *California Inland Surface Waters Plan*. Section 5 summarizes the water quality sample results for the February 12, 1992, storm/CSO event. Section 6 provides a description of the chronic toxicity testing program, test results, and quality assurance results. Section 7 then summarizes the results for the sampling event.

SECTION 2
EVENT SAMPLING PROGRAM

This section describes the field procedures used to collect and handle the water quality samples obtained from the Sacramento River, and Sump 2 CSO and CWTP effluent discharges during the February 14, 1992 storm event.

A. FIELD METHODS FOR SAMPLE COLLECTION

The *Field Manual* prepared for the Phase 3 study provides a detailed description of the sampling procedures and instructions for all locations and constituents involved in the sampling program. Because the timing, movement, and intensity of each storm event differs, the *Field Manual* (final version dated January 30, 1992), was intended to provide guidelines for a theoretical storm pattern. Storm event descriptions are presented in Section 3 of this report.

A summary matrix of the sampling groups and analyses for this event is presented in Table 2-1. The table identifies the types of analyses and groups of constituents of interest for this CSO/CWTP effluent discharge event. Table 2-2 lists the constituents analyzed during this storm event.

A.1. Sacramento River

The Sacramento River water obtained for inorganic chemistry (including trace metals) and bacteria analyses was collected at Captain's Table dock, Miller Park dock, and the Wooden Stairs. Dilution water for CWTP chronic toxicity tests was obtained from Captain's Table dock. Water was collected directly from the river into plastic receptacles and transferred into the sample containers. The water for oil and grease analysis was collected in glass bottles filled directly from the river. Water temperature and dissolved oxygen were measured at the time of sample collection.

TABLE 2-1. SAMPLING AND ANALYSIS MATRIX FOR CSO EVENTS		
Location/Description	**Analyses/Constituents**	**Sample Type**
Sacramento River Sites		
Captain's Table		
"Captain's Table I"		
After discharge from CWTP, but before Sump 2 discharge, for "pre-discharge" background of river water quality	Dilution Water for CWTP Chronic Toxicity Test	Grab
	Chemistry[1] (of dilution water)	Grab
	Oil and Grease	Grab[4]
	MS/MSD[7]	Grab
	Duplicate[6]	Grab
	Chronic Toxicity (of dilution water, 100% sample test for control compared to CWTP effluent toxicity)	Grab
"Captain's Table II"		
After Sump 2 discharge	Chemistry[1]	Grab
Miller Park		
"Miller Park I"		
After discharge from Sump 2, but before Pioneer discharge, for "pre-discharge" background of river water quality[6]	Chemistry[1]	Grab
Wooden Stairs		
"Wooden Stairs"		
After discharge from CWTP	Chemistry[1]	Grab

TABLE 2-1. SAMPLING AND ANALYSIS MATRIX FOR CSO EVENTS		
Location/Description	**Analyses/Constituents**	**Sample Type**
CWCS Discharge Sites		
Sump 2 CSO		
"Sump 2"	Chemistry[1] Tributyltin Oil and Grease Pesticides/PCBs MS/MSD Chronic Toxicity (100% sample test)	Composite[2] Grab Grab[3] Grab[3,4] Composite[2] Composite[2]
CWTP Effluent		
"CWTP"	Chemistry[1] Tributyltin Oil and Grease Pesticides/PCBs MS/MSD[7] Chronic Toxicity (dilution series)	Composite[2] Grab Composite[3] Composite[3,4] Composite[2] Composite[2]

1 Includes inorganics, bacteria, and trace metals. See Table 2-2.
2 Composites consisted of discrete samples collected at 30-minute intervals over the duration of the discharge.
3 Discrete samples were obtained during each 30-minute interval over the duration of the discharge. Laboratory prepared composite sample in a galss container.
4 Enough material was collected to allow for MS/MSD analyses.
5 Miller Park site used for river background chemistry for Sump 2 discharge.
6 This was a sample split rather than a field duplicate.
7 Matrix spike/matrix spike duplicate.

TABLE 2-2. CONSTITUENTS TO BE ANALYZED FOR THE CSO SAMPLING EVENT

Constituents	EPA Method Number	Reporting Limit	Container Type	Sample Size
Bacteria				
Total Coliform	SM 908A	2 MPN	Plastic	2 X 250 ml
Fecal Coliform	SM 908C	2 MPN	Plastic	
Fecal Streptococci	SM 908C	2 MPN	Plastic	
Inorganics				
pH	150.1	N.A.	Plastic	500 ml[1]
Temperature	Field	-	-	-
Turbidity	180.1	N.A.	Plastic	500 ml[1]
Dissolved Oxygen	Field	-	-	-
Hardness as $CaCO_3$	314A	3.3	Glass w/ HNO_3	500 ml[2]
Total Kjeldahl Nitrogen	351.2	0.5 mg/l	Glass w/ H_2SO_4	1 liter[2]
Phosphorus (total)	365.3	0.5 mg/l	Glass w/ H_2SO_4	1 liter[2]
Phosphorus (dissolved)	365.3	0.5 mg/l	Plastic	500 ml[3]
Nitrate	300.0	0.5 mg/l	Plastic	500 ml[1]
Nitrite	300.0	0.5 mg/l	Plastic	500 ml[1]
Chemical Oxygen Demand	410.4	5.0 mg/l	Glass w/ H_2SO_4	1 liter[2]
Biochemical Oxygen Demand	405.1	2.0 mg/l	Plastic	500 ml[2]
Total Suspended Solids	160.2	5 mg/l	Plastic	500 ml[4]
Total Dissolved Solids	160.1	10 mg/l	Plastic	500 ml[4]
Oil and Grease	413.2	1.0 mg/l	Glass	2 liters[5]
Organics				
Organochlorine Pesticides/PCBs	608	0.05-1.0 µg/l	Glass	2 liters[5]
Tributyltin	-	0.001 µg/l	Glass	2 liters
Metals (Total, Recoverable, and Dissolved)				
Arsenic	206.3 (HAA)	1.0 µg/l[6]	Plastic	--[3,7]
Beryllium[8]	200.7 (ICP)	5.0 µg/l	Plastic	--
Cadmium	213.2 (GFAA)	0.4 µg/l	Plastic	--
Chromium	218.2 (GFAA)	0.3 µg/l	Plastic	--
Cobalt[8]	200.7 (ICP)	10.0 µg/l	Plastic	--
Copper	220.2 (GFAA)	0.5 µg/l	Plastic	--
Lead	239.2 (GFAA)	1.7 µg/l	Plastic	--
Mercury[9]	245.1 (CVAA)	0.2 µg/l	Plastic	--
Nickel[10]	249.2 (GFAA)	2.5 µg/l	Plastic	--
Selenium	270.3 (HAA)	0.3 µg/l[6]	Plastic	--
Silver	272.2 (CVAA)	0.1 µg/l	Plastic	--
Zinc	200.7 (ICP)	8.5 µg/l	Plastic	--

1 Analyses for pH, turbidity, nitrate and nitrite conducted from one sample container.
2 Analyses for total Kjeldahl nitrogen, COD, and total phosphorus collected from one sample container.
3 Analysis for dissolved phosphorous and dissolved metals conducted from one 500-ml container. The laboratory filtered the sample and then preserved with HNO_3 before analysis.
4 TDS and TSS collected in one container, and separated by laboratory for analysis.
5 Total amount required for analysis. Discrete samples collected in the field and composited by the laboratory.
6 Analysis of recoverable concentrations for arsenic and selenium is by GFAA, which results in a higher reporting limit of 5 µg/l. Analysis for total and dissolved concentrations for arsenic and selenium is by HAA, which results in reporting limits of 1.0 µg/l and 0.3 µg/l, respectively.
7 Analysis for total and recoverable concentrations, and hardness conducted from 2 500-ml sample containers, preserved with HNO_3.
8 Total, recoverable, and dissolved analysis for both beryllium and cobalt was not required or requested for this study. However, in the ICP process, the analytical results for these metals became available at no charge to the project, therefore it was incorporated in this CSO event report.
9 No method available for analyzing recoverable mercury.
10 The analysis of nickel was not required or requested by this study. However, because of a laboratory log-in error, an analysis for dissolved nickel was conducted at no charge to the project. The data were incorporated into this CSO event report.

A.2. Sump 2

At Sump 2, water was collected at 30-minute intervals from the spigot located on the discharge piping of Pump No.11. Combined wastewater from the Sump 2 wet well is pumped by Pump No. 11 to Chamber 3 of the Flow Control Structure, then discharged into the river. Water for the oil and grease analysis and the organochlorine pesticides/PCBs analysis were collected in glass bottles filled directly from the spigot every 30 minutes. These sample bottles were then composited into one glass container at the laboratory. Water for the tributyltin analysis was collected in glass bottles filled directly from the spigot during the first 30-minute period only.

Discrete samples for all other analyses (other inorganics, trace metals, bacteria) were composited into 48-liter polyethylene carboys at 30-minute intervals. The volumes added to the carboys were time-dependent and are specified in the *Field Manual*. Water temperature and dissolved oxygen were measured at the time of each 30-minute interval collection at Sump 2.

A.3. CWTP

At the CWTP, effluent was collected at 30-minute intervals from a spigot in the wet laboratory. This spigot is plumbed into the effluent stream following the dechlorination stage of treatment. Discrete effluent samples for the inorganic analyses (except oil and grease) were composited into 48-liter polyethylene carboys at 30-minute intervals. The volumes added to the carboys were time-dependent and are specified in the *Field Manual*.

The samples for oil and grease analysis and organochlorine pesticides/PCBs analysis were collected in glass bottles filled directly from the spigot every 30 minutes. These sample bottles were then composited into one glass container at the laboratory. Water for tributyltin analysis was collected in glass bottles filled directly from the spigot during the first 30-minute period only. Effluent for the lead MDL study was collected in polyethylene bottles filled directly from the spigot during the first 30-minute period only.

Discrete samples for all other analyses (other inorganics, trace metals, bacteria) were composited into 48-liter polyethylene carboys at 30-minute intervals. The volumes added to the carboys were time-dependent and are specified in the *Field Manual*. Water temperature and dissolved oxygen were measured at the time of each 30-minute interval collection at CWTP.

B. SAMPLE HANDLING PROCEDURES

All containers, ice chests, and packing materials were prepared and provided by the laboratory. All filtering, necessary for analyzing dissolved metals and phosphorous, was performed by the laboratory upon delivery of the samples. Exposure to sunlight was avoided by keeping sample bottles in covered ice chests. Exposure to sunlight was negligible for this event because samples were collected either under heavy cloud cover or during the night and early morning hours of the storm event. All samples were packed on ice, without freezing, prior to transport to the laboratory.

B.1. Total Kjeldahl Nitrogen, Chemical Oxygen Demand, and Total Phosphorus

The container used for the collection of these constituents was a glass bottle preserved with sulfuric acid. The sample bottle was filled leaving a small air space at the top.

B.2. Turbidity, pH, Nitrate, and Nitrite

The container used for the collection of these constituents was a polyethylene bottle, which was filled leaving a small air space at the top of the bottle.

B.3. Total Suspended Solids and Total Dissolved Solids

A single polyethylene bottle was used to collect water for both of these analysis. The sample was split into two batches at the laboratory and analyzed for each constituent.

B.4. Bacteriological and Biological Oxygen Demand

The containers for collecting the bacteriological samples were preserved with sodium thiosulfate to neutralize the effects of residual chlorine on the organisms in the sampled waters. The container for BOD analysis was a polyethylene bottle with no preservatives.

B.5. Total Metals, Recoverable Metals, and Hardness

The containers used for the collection of these constituents were polyethylene bottles preserved with nitric acid.

B.6. Dissolved Metals and Dissolved Phosphorus

The containers used for the collection of these constituents were unpreserved polyethylene bottles. Upon delivery to the laboratory, the sampled waters were filtered and subsequently preserved with nitric acid.

B.7. Oil and Grease, and Organochlorine Pesticides/PCBs

The containers used for the collection of these constituents were glass bottles. The sampled waters did not contact plastic containers or implements in their compositing or storage.

C. LABORATORY QUALITY ASSURANCE METHODS

Enseco Laboratory performed various quality assurance procedures in the analysis of the CWCS and river samples to ensure the quality of test results. The *Enseco Incorporated Quality Assurance Program* requires the analysis of several types of control samples as standard elements in their Laboratory Performance Quality Control process. Duplicate control samples, single control samples, method blanks, and equipment blanks are used to identify any procedural errors. These are a part of Enseco's in-house QA/QC procedures.

Duplicate control samples (DCS) consist of a pair of standard, control matrix samples (reagent water is used for aqueous samples) spiked with surrogate compounds representative of the constituents to be tested. Accuracy and precision data are compared to the control limits established for the tested constituents. This is a standard procedure conducted on all sets of samples.

A single control sample (SCS) is also a control matrix sample, spiked with a surrogate compound appropriate to the testing method. This test is used with smaller batches of samples. The recovery data are compared to the control limits established for the tested compound.

Method blanks (MB) are used to determine the level of interference or contamination in the analytical system that could elevate or diminish concentration results. The concentration of target constituents in the MB samples should be below the reporting limit. However, common solvents, which are present in the laboratory, may be acceptable at levels up to 5 times the reporting limit. Because certain metals may also be difficult to reduce to the extremely low reporting limits used in this study, concentrations are accepted up to 2 times the reporting limit.

D. FIELD QUALITY CONTROL METHODS

Matrix spikes (MS) and matrix spike duplicates (MSD) are environmental samples collected in the field. The MS and MSD samples are spiked with known concentrations of constituents, and from the analysis of these controls, the percent recovery of the added constituents is determined. Based upon the results of this analysis, the effect of the matrix on the accuracy of the analysis procedure is determined. Both the MS and the MSD results indicate the precision of the analysis for a particular constituent and matrix.

As part of the quality assurance program, the laboratory split the river water samples collected upstream of the CWTP effluent discharge in half. (These were the "Captain's Table I" samples, collected upstream of the CWTP outfall, prior to the CSOs from Sump 2). These two sets of samples were then analyzed. One set of samples was labeled with an "SA" suffix in the analytical results and is

referred to as the parent set of samples in this report. The other set of samples was labeled with a "DU" suffix in the analytical results and is referred to as the quality control sample set. This quality assurance methodology was used to verify the laboratory's ability to replicate precise and accurate analytical results.

A complete set of duplicate water samples was also collected from the river at Captain's Table dock, upstream of the CWTP outfall and prior to the CSO discharge from Sump 2. In the initial sampling plan, field duplicates (as opposed to sample splits), were intended to be used as the field quality control samples. However, the sample split results proved to be a more stringent measure of laboratory precision and accuracy. Therefore, the sample splits were used as the field quality control check on the laboratory.

SECTION 3
STORM EVENT DESCRIPTIONS, SAMPLE COLLECTION TIMING, FIELD DATA RESULTS, AND QUALITY ASSURANCE RESULTS

This section describes the event during which sampling took place, presents the field data, and discusses the quality control assurance results.

A. STORM EVENT

During the February 14, 1992 storm event, 1.24 inches of rainfall were measured at the National Weather Service gauge located at 801 I Street in Sacramento. The 17.5-hour storm began at 9:00 a.m. on February 14 and lasted until 2:30 a.m. on February 15. Discharges from Sump 2 and CWTP were recorded and sampled during the storm event. No discharge occurred from Pioneer Reservoir during this storm event.

The CWTP discharge began at 5:30 p.m. and lasted until 3:00 a.m. on February 15. Over the course of this 9.5-hour treatment period, 39.5 million gallons (MG) of effluent were discharged into the Sacramento River.

A 3-hour, 16.0-MG discharge from Sump 2 into the Sacramento River was recorded between 6:00 p.m. and 9:00 p.m. on February 14.

The Sacramento River was flowing at a rate of approximately 25,040 million gallons per day, or 38,740 cubic feet per second, past the USGS gauge at Freeport Bridge.

These storm and discharge event descriptions and data were prepared by City staff.

Assuming this river flow rate was the average rate that occurred during the discharge periods, then the dilution factors based on volume were present during this storm event:

$$\text{CWTP Dilution} = \frac{\text{River Volume}}{\text{Effluent Volume}} = \frac{25{,}040 \text{ mgd} * 9.5 \text{ hrs}}{39.5 \text{ MG}} \frac{1}{24 \text{ hrs}} = 251:1$$

$$\text{CSO Dilution} = \frac{\text{River Volume}}{\text{CSO Volume}} = \frac{25{,}040 \text{ mgd} * 3 \text{ hrs}}{16.0 \text{ MG}} \frac{1}{24 \text{ hrs}} = 196:1$$

These river dilution factors are used to estimate the potentially detectable concentrations of organic compounds in the river from the test results for CWTP effluent and Sump 2 CSO samples. Section 5 presents this evaluation.

B. SAMPLE COLLECTION TIMING

During this CSO event, samples were collected from the Sacramento River to determine the effects of Sump 2 discharge and CWTP effluent on river water quality. This objective was accomplished by sampling river water as the plume of discharge/effluent passed the sampling location.

At the time of the CSO from Sump 2, Sacramento River velocities were measured at the Captain's Table dock and Miller Park dock. The river velocity was measured at 0.50 feet per second (fps) using a Montedoro-Whitney electromagnetic velocity meter. A time-of-travel for the discharge plume was calculated using the measured river velocity and the distance between Sump 2 and the Captain's Table dock (4,800 feet). Later in the storm event, river water samples were obtained as the Sump 2 discharge plume passed the Captain's Table dock, as shown in Table 3-1. These samples, labeled "Captain's Table II", were analyzed to determine the effects of the discharge on river water quality. River water samples from the Miller Park dock, upstream of the Sump 2 discharge, were collected as the discharge was occurring. These samples, labeled "Miller Park I", were analyzed to determine the background river water quality prior to the Sump 2 CSO discharge.

TABLE 3-1. TIME TABLE OF SAMPLE COLLECTION AND DISCHARGE/EFFLUENT PLUME TRAVEL			
Sample Identification	**Sample Collection Period (Clock Hour)**	**Sump 2 Plumes Past Captain's Table Dock (Clock Hour)**	**CWTP Plume Past Wooden Stairs Dock (Clock Hour)**
Captain's Table I	1845 - 2030	2025 - 2325	NA
Captain's Table II	2100 - 2145	2025 - 2325	NA
Miller Park I	1900 - 2000	NA	NA
Wooden Stairs	2200 - 2300	NA	1955 - after 0300 (2/15/92)

The Sacramento River velocity of 0.50 fps and the distance between CWTP and the Wooden Stairs dock (4,400 feet) were used to calculate the time-of-travel for the CWTP effluent plume. Later in the storm event, river water samples were collected as the CWTP effluent plume passed the Wooden Stairs location, as shown in Table 3-1. These samples were analyzed to determine the effects of the effluent discharge on river water quality.

River water samples from the Captain's Table dock, upstream of the CWTP discharge, were collected as the discharge was occurring. These samples, labeled "Captain's Table I", were analyzed to determine the background river water quality prior to CWTP effluent discharge. The Captain's Table I samples were also collected prior to the Sump 2 CSO discharge.

C. FIELD DATA RESULTS

Water temperature, dissolved oxygen (DO) concentration measurements, and time of collection for each of these samples are presented in Table 3-2. Water temperatures recorded from samples collected in the Sacramento River ranged from 9.2°C to 11.1°C. Water temperatures recorded for samples collected within the wastewater system ranged from 14.2°C to 14.6°C in Sump 2 discharge and from 10.0°C to 14.0°C in CWTP effluent. DO concentrations in Sacramento River waters ranged from 8.7 mg/l to 9.2 mg/l at the sampling sites. DO concentrations ranged from 9.0 mg/l to 10.4 mg/l in the Sump 2 discharge, and from 6.4 mg/l to 10.2 mg/l in CWTP effluent.

TABLE 3-2. FIELD DATA RESULTS FOR FEBRUARY 14, 1992 EVENT

Sample Identification	Time	Temp (°C)	DO (mg/l)
Captain's Table I	1945	10.6	8.7
Captain's Table II[1]	2110	11.1	9.2
Miller Park I[2]	2000	9.2	9.2
Wooden Stairs	1900	10.6	No Data[3]
Sump 2 Discharge	1815	14.2	10.4
	1845	14.4	9.6
	1915	14.5	9.8
	1945	14.2	9.6
	2015	14.6	9.9
	2045	14.2	9.0
CWTP Effluent	1815	10.0	10.2
	1845	11.0	6.4
	1915	14.0	7.0
	1945	13.0	7.0
	2015	13.0	7.0
	2045	13.0	6.5
	2115	12.0	7.2
	2145	12.0	6.6
	2215	12.0	7.1
	2245	12.0	6.4
	2315	12.0	6.4
	2345	12.0	6.6

1 Samples collected during the passing of the Sump 2 discharge plume.
2 "Background" river site, upstream of all other discharges, for this event.
3 The dissolved oxygen meter malfunctioned during this period.

D. EVENT-SPECIFIC QUALITY ASSURANCE ANALYSIS RESULTS

As described in Section 2, a series of in-house laboratory and project-specific quality assurance procedures were applied in the analysis of the CWCS samples. Laboratory quality assurance procedures, designed to assess laboratory operations and to establish method-specific control limits, included the analysis of method blanks, matrix duplicates, single control samples, and duplicate control samples. Matrix spike and matrix spike duplicate analyses of field samples were conducted to assess potential sample matrix effects.

The quality assurance results indicate that the laboratory testing procedures produced reliable results for the various analytes within the water quality samples. Most percent recoveries and relative percent differences were within the accepted control limits. However, several specific complications resulting from excessive relative percent differences and laboratory contaminants influenced the interpretation of the results.

The organochlorine pesticide and PCB analyses were conducted outside of the two-week holding time, due to a laboratory log-in error. Sample extraction was performed 32 days following delivery to the laboratory from the field. During the extended holding time, the samples may have metabolized, breaking down certain constituents into their sub-compounds. The organochlorine pesticides and PCB list for an EPA Method 608 analysis screen has some compounds that are metabolites of more complex analytes on the list. DDT is one such analyte where the laboratory also searched for DDD and DDE. This would provide information on the possible presence of the parent molecule. In all cases, no metabolites were detected. The chromatographs of each sample were carefully studied by the laboratory for any possible detections below the reporting limits. Although the matrix examined caused chromatography interferences, laboratory staff found no confirmed detections for any of the "target" analytes (i.e. those from the *Inland Surface Waters Plan* aquatic life objectives). Quality control samples with analyte spikes reported acceptable recoveries, giving the laboratory confidence in their ability to detect pesticides and PCBs in this matrix.

As mentioned, matrix interferences were observed in the analysis of organochlorine pesticides and PCBs for Sump 2 and CWTP. For the Sump 2 and CWTP sample analysis, reporting limits for all 19 pesticides and the 7 PCB constituents were raised to compensate for the interferences. Following the initial analysis, the results did not match the calibration curve for these analytes. The sample was then diluted to obtain more accurate results requiring that the reporting limits be raised. With a Method 608 screen analysis , it is not possible to exactly determine the cause of matrix interferences. Possible sources of interference are presence of pthalitic esters, sulfur or plasticisers. With regard to the latter, even items such as plastic cups in stormwater entering the sewers can cause a problem in the analysis. Because of this potential, the field and laboratory procedures required that sample water for organochlorine pesticide/PCB analysis be handled only in glass.

The duplicate control sample for the oil and grease analysis did not meet the relative percent difference (RPD) quality control limit (adopted from the Environmental Protection Agency's *Solid Waste Analytical Methods Manual 846, Third Edition*). The RPD is a measure of precision, determined by comparing the two percent recoveries tests. For the oil and grease analysis, the calculated RPD was 39 percent which exceeded the quality control goal of 18 percent.

The reporting limits for total and dissolved mercury analysis were raised from 0.00020 mg/l to 0.00040 mg/l to reflect limitations in sample volume. The affected samples were Captain's Table I (dissolved mercury), Sump 2 (total and dissolved mercury), and CWTP (total and dissolved mercury). Because the initial analytical results did not meet the in-house laboratory QA/QC guidelines, the laboratory re-analyzed the samples. In doing so, a significant portion of the sample volume was consumed. The laboratory split the sample volume into smaller fractions in order to conserve the remaining sample water. Therefore, the reporting limits were raised to reflect this volume reduction in analyzed sample.

The chromium analysis results may be influenced by laboratory contamination in sample preparation. Total chromium was found in one sample blank at a concentration of 0.00051 mg/l, using a reporting limit of 0.00050 mg/l.

Recoverable chromium was found in one sample blank at a concentration of 0.00080 mg/l, using a reporting limit of 0.00050 mg/l. Quality control procedures adopted from the Environmental Protection Agency's *Solid Waste Analytical Methods Manual 846, Third Edition*, states that sample blank contamination must be less than two times the Minimum Detection Limit (MDL). The chromium concentrations in the sample blanks were within these guidelines.

SECTION 4
COMPARATIVE STANDARDS

This section describes the derivation of numerical objectives for pollutant concentrations used to provide a basis for comparison to the laboratory results for the storm event. The impact on the receiving water, i.e. the Sacramento River, is estimated through the use of these water quality objectives.

A. INLAND SURFACE WATERS PLAN

The *California Inland Surface Waters Plan - Water Quality Control Plan for Inland Surface Waters of California - 91-12 WQ* (Water Resources Control Board, April 1991), provides a objective concentrations to which the laboratory analytical results can be compared. (This document is herein referred to as the *Inland Surface Waters Plan*.) Table 1 in this document presents the "Water Quality Objectives for Protection of Freshwater Aquatic Life", including 4-day average, daily average, 1-hour average, and instantaneous maximum pollutant concentration objectives.

Of the four durations (4-day, daily, 1-hour, instantaneous) for which water quality objectives are identified in Table 1 of the *Inland Surface Waters Plan*, an appropriate duration must be selected for comparison to the storm/CSO event results in this report. The duration of effluent discharges from the CWTP during this event was 9.5 hours, and Sump 2 discharged for 3 hours. These discharges exceeded 1 hour but were far shorter than 4 days. It would therefore be most appropriate to compare the event-specific results to a comparative standard for a duration between 1 hour and 4 days, consistent with the duration of the CWTP and Sump 2 discharges. Ideally, comparable standards could be made available that account for the variability of storm and discharge event durations. However, no such published standards exist. Therefore, the storm event results were compared to <u>both</u> the 4-day and the 1-hour numerical objectives. Specific results for the storm event sampled, and a comparison to calculated numerical objective concentrations, are presented in Section 5.

B. DERIVATION OF NUMERICAL OBJECTIVES

B.1. Receiving Water Hardness

The *Inland Surface Waters Plan* (specifically, Table 1) identifies objective concentrations for 21 constituents. Numerical objectives for the 9 trace metals listed are based on the hardness (as $CaCO_3$) found in the receiving water during the event. These objectives will therefore differ between water bodies. The hardness of a given water body, like other constituents, also varies over time.

A baseline hardness for the river at Miller Park dock, which is upstream of the Sump 2 outfall, was chosen to represent the Sacramento River during this storm event. Analytical results of river water collected from the Miller Park location showed a hardness concentration of 99.6 mg/l. This value, in conjunction with the river water's pH of 7.1, was used to derive the water quality objective concentrations.

B.2. Pollutant Concentration Objectives

The objectives listed in the *Inland Surface Waters Plan* were converted to pollutant concentrations to which the river sampling results were compared. These are listed in Table 4-1, including the objectives for specific organics. Equations for calculating concentrations can be found on Page 4 of the *Inland Surface Waters Plan* document.

Although the objective concentrations are compared to the Sacramento River sample results, discharges to the river cannot further degrade water quality when the river is at the objective concentration. If, for instance, the river concentration for a recoverable metal reaches the objective concentration, then the concentration of that metal in the discharge must be at or below the objective.

TABLE 4-1. WATER QUALITY OBJECTIVES FOR PROTECTION OF FRESHWATER AQUATIC LIFE FEBRUARY 14, 1992 SAMPLING EVENT

Values used in computations:
Hardness, in mg/l 99.6 (Sacramento River at Miller Park)
pH 7.1

Constituent	Units	Concentration Objectives			
		4-Day Average	Daily Average	1-Hour Average	Instantaneous Maximum
arsenic	ug/l	190.0	—	360.0	—
cadmium	ug/l	1.1	—	3.9	—
chlordane	ng/l	—	4.3		—
chromium(VI)	ug/l	11.0	—	16.0	—
copper	ug/l	11.8	—	17.7	—
DDT	ng/l	—	1.0	—	—
dieldrin	ng/l	—	1.9	—	—
endosulfan	ng/l	—	56.0	—	220.0
endrin	ng/l	—	2.3	—	180.0
heptachlor	ng/l	—	3.8	—	—
hexachlorocyclo-	ng/l	—	80.0	—	—
hexanegamma		—	—	—	—
lead	ug/l	3.2	—	81.2	—
mercury	ug/l	—	—	2.4	—
nickel	ug/l	157.1	—	1413.4	—
PCBs	ng/l	—	14.0	—	—
pentachlorophenol	ng/l	6.3	—	10.0	—
selenium	ug/l	5.0	—	20.0	—
silver	ug/l	—	—	—	4.0
toxaphene	ng/l	0.2	—	730.0	—
tributyltin	ng/l	20.0	40.0	—	60.0
zinc	ug/l	105.6	—	116.6	—

NOTES:

mg/l = milligram(s) per liter; ug/l = micrograms per liter; ng/l = nanograms per liter

Objectives for cadmium, chromium, copper, lead, nickel, silver and zinc are based on total recoverable metal concentration.

Dischargers may, at their option, meet the total chromium limitation.

The 4-day average for tributylin is the six month median.

Source: California Inland Surface Waters Plan, Water Resources Control Board, April 1991.

SECTION 5
WATER QUALITY SAMPLE RESULTS

This section summarizes the laboratory results for the storm event of February 14, 1992, and presents a comparison of water quality for the Sacramento River, Sump 2 overflow, and CWTP effluent discharges, to the *Inland Surface Waters Plan* objective concentrations. Results for chronic toxicity tests are presented in Section 6.

A. LABORATORY RESULTS

The detailed laboratory analysis data for this event are presented in Appendices A and B, separate from this report. Summaries of the laboratory results for detectable analytes are presented according to sampling location in Tables 5-1 through 5-6. Although samples were analyzed for all constituents listed in Table 2-2, only the detected constituents are included in the results tables. Tables 5-1 through 5-6 present the analysis results from: Captain's Table I (on the river upstream of the CWTP effluent discharge), Captain's Table II (on the river in the plume of Sump 2 CSO effluent discharge), Miller Park I (on the river upstream of Sump 2 CSO discharge), Wooden Stairs (on the river in the plume of CWTP effluent discharge), Sump 2, and CWTP, respectively. All tables are located at the end of this section.

A.1. General Inorganics

The analyses of general inorganics constituents for Sacramento River samples (Captain's Table I, Captain's Table II, Miller Park I, and Wooden Stairs) resulted in concentration higher than the reporting limit for all analytes except nitrite. Comparison between the Miller Park I sample, collected upstream of the Sump 2 discharge, and the Captain's Table II sample, collected in the discharge plume of Sump 2, indicates comparable analytical results. This suggests that Sump 2 effluent discharge had an insignificant effect on the Sacramento River inorganic water quality.

The results from the Captain's Table I sample, collected upstream of the effluent discharge from CWTP, and Captain's Table II sample, collected after the effluent discharge from Sump 2, are comparable. This comparison, combined with the comparison of the Miller Park I and Captain's Table II samples, strongly suggests that Sump 2 CSO, over the 3-hour discharge period, had an insignificant effect on the Sacramento River's inorganic water chemistry at the Captain's Table dock location.

The results from the Wooden Stairs river sample collected in the effluent discharge plume from CWTP are comparable to the upstream Captain's Table I sample. This suggests that CWTP effluent also had an insignificant overall effect on the Sacramento River's inorganic water chemistry.

The analysis of general inorganics constituents for the two discharges, Sump 2 and CWTP, resulted in concentrations higher than the reporting limit for all analytes except nitrite. Characteristics of the discharges are comparable, with the exception of total dissolved and total suspended solids, which were 43.0 mg/l and 387.0 mg/l, respectively, in Sump 2 discharge, to 226.0 mg/l and 180.0 mg/l, respectively, in CWTP effluent. The pH results indicate that CWTP effluent was slightly more acidic than Sump 2 discharge; the measured values were 6.0 and 6.4, respectively.

A.2. Bacteria

Bacteria counts from the Miller Park I river sample, upstream of the Sump 2 discharge, resulted in total coliform, fecal coliform, and fecal streptococcus at concentrations of 30,000 MPN/100 ml, 8,000 MPN/100 ml, and 13,000 MPN/100 ml, respectively. Bacteria counts from Captain's Table I river samples, upstream of the CWTP effluent discharge, showed the presence of 50,000 MPN/100 ml total coliform and 24,000 MPN/100 ml of both fecal coliform and fecal streptococcus.

Bacteria concentrations from the Captain's Table II river sample (in the Sump 2 CSO discharge plume), showed the presence of total coliform, fecal coliform, and fecal streptococcus at concentrations of 50,000, 30,000 and 5,000 MPN/100 ml, respectively. Analytical results for the Wooden Stairs sample, collected in the

CWTP's effluent plume, showed the presence of total coliform, fecal coliform, and fecal streptococcus at concentrations of 30,000, 5,000 and 8,000 MPN/100 ml, respectively.

The bacteria results of the river samples do not suggest a uniform trend that can be correlated to pre- and post-discharges of either Sump 2 or the CWTP. The counts for the river may be affected by stormwater discharges to the river.

Analytical results from the Sump 2 discharge sample showed the presence of total coliform, fecal coliform, and fecal streptococcus concentrations at 2,400,000, 500,000, and 50,000 MPN/100 ml, respectively. Analytical results from the CWTP effluent sample showed the presence of total coliform, fecal coliform, and fecal streptococcus concentrations at 90,000, 8,000, and 2,200 MPN/100 ml, respectively.

Bacteria counts in the CWTP samples were considerably lower than Sump 2 discharge samples, which reflects the effective chlorination treatment at the CWTP facility.

A.3. Organochlorine Pesticides/PCBs and Tributyltin

A.3.a. Discharge Results--

The organochlorine pesticides and PCB analytes were not analyzed within the two-week holding time due to a laboratory log-in error, as described in Section 3. Organochlorine pesticides and PCBs were <u>not</u> detected in either the samples of Sump 2 discharge or CWTP effluent. However, all of reporting limits for these samples were raised due to matrix interferences.

The *Inland Surface Waters Plan* identifies instantaneous maximum concentrations for endrin (180 ng/l) and endosulfan (220 ng/l). For the CWTP effluent and the Sump 2 CSO sample results, the comparison to objective concentrations produces mixed conclusions. Endrin and endosulfan-I (or -alpha) were reported as "ND" at a reporting limit of 120 ng/l; these results indicate concentrations below the instantaneous maximum objective. Because the limit was raised to 250 ng/l for endosulfan-II (or -beta) and endosulfan sulfate, no definitive conclusions can

be drawn on the presence of these constituents as compared to the instantaneous maximum objective concentration.

Hexachlorocyclo-hexanegamma (also lindane or gamma-BHC) results were "ND" for a reporting limit of 62 ng/l in both the Sump 2 and CWTP samples. Based on a daily average objective concentration of 80 ng/l, it is concluded that these sample concentrations were less than the objective.

Tributyltin was detected in Sump 2 discharge at 0.029 µg/l using a reporting limit of 0.001 µg/l. Tributyltin was also detected in CWTP effluent at 0.017 µg/l using a reporting limit of 0.001 µg/l. A comparison of the tributyltin concentrations detected in the Sump 2 discharge to the 4-day objective concentration listed in Table 4-1 shows that this analyte exceeded the objective. A comparison of the tributyltin concentrations detected in the CWTP effluent to the 4-day objective concentration shows that this analyte did not exceed the objective. (Tributyltin does not have a 1-hour objective concentration.)

A.3.b. Estimated River Results--

The organochlorine pesticide/PCB screen analysis was performed on only the discharge samples, rather than the river samples, in order to evaluate a "worst-case" condition. However, the reportable and detectable concentrations that would have occurred in the river samples can be approximated from the discharge samples and dilution factors calculated in Section 3. For these estimation purposes, dilution factors are rounded to 250:1 and 200:1, for CWTP and Sump 2 discharges to the river, respectively.

For instance, dieldrin was not detected in the Sump 2 CSO discharge, using a reporting limit of 0.12 µg/l. Using the dilution ratio, the reporting limit for the downstream river section would be 0.6 ng/l. At this reporting limit, the result would be "ND". Neither a 4-day average nor 1-hour average objective concentration is available for dieldrin; the daily average value is 1.9 ng/l. Therefore, for this constituent, it is estimated that the daily average objective concentration would have been less than the daily objective for the river downstream of Sump 2.

The results of this analysis are summarized in Tables 5-8 and 5-9. As indicated by the table, most of these compounds would not have been detected in the river when compared to the objective concentrations in the *Inland Surface Waters Plan.* One-hour and 4-day objective concentrations are provided for toxaphene. Results for both CWTP effluent and Sump 2 CSO samples were "ND". However, conclusions relative to Table 4-1 objectives cannot be determined for either the samples or the estimated "diluted" river concentration, since the reporting limit is higher.

A.4. Oil and Grease

Oil and grease concentrations were not detected in any of the river samples at a reporting limit of 1.0 mg/l. The oil and grease concentrations were detected at 2.2 mg/l and 1.4 mg/l, using a reporting limit of 1.0 mg/l, in Sump 2 discharge and CWTP effluent, respectively. However, as previously mentioned in Section 3, the RPD goal for these analyses were exceeded, which may influence the precision of these analytical results.

A.5. Trace Metals

As described in Section 3, chromium (total and/or recoverable, or a combination) was found in <u>all</u> of the quality assurance sample blanks for these event sample batches. Blanks are analyzed coincident with sample batches to detect possible instrument contamination. The low reporting limits required by this study, which are close to detection limits and are necessary for comparison to the *Inland Surface Waters Plan* objective concentrations, increase the possibility that low-level contamination will be detected. If contamination is found in a blank, it is not possible to determine the amount of contamination that may have occurred in the sample of interest, unless special sample preparation and followup analyses are conducted. Such special sample handling was beyond the scope of this study.

The analysis for trace metals in the Captain's Table I river sample, collected upstream of CWTP discharge, detected concentrations for seven total metal, seven recoverable metal, and six dissolved metal analytes, as shown in Table 5-1. However, the total and recoverable chromium analytical results may have been

influenced by contamination from laboratory sample preparation. Although the reporting limit for total mercury was raised due to sample volume restrictions, it was not detected at the higher limit.

Seven total metal, six recoverable, and five dissolved metal analytes were detected in the Captain's Table II sample, collected in the plume of Sump 2 discharge. The results are summarized in Table 5-2. Total and recoverable chromium contamination may have occurred from laboratory sample preparation.

The analysis for trace metals in the Miller Park I sample, collected upstream of the Sump 2 discharge, resulted in detectable concentrations for six total metal, seven recoverable metal, and six dissolved metal analytes, as shown in Table 5-3. Total and recoverable chromium analytical results may have been influenced by contamination.

In the Wooden Stairs sample, collected in the plume of CWTP effluent, detectable concentrations for six total metal, five recoverable metal, and four dissolved metal analytes were found, as shown in Table 5-4. Again, the total and recoverable chromium analytical results may be influenced by contamination from laboratory sample preparation.

The analysis for trace metals in the Sump 2 effluent sample detected concentrations for 7 of the 9 total metal analytes, 6 of the 8 recoverable metal analytes, and 6 of the 10 dissolved metal analytes as shown in Table 5-5. However, the total and recoverable chromium analytical results may be influenced by contamination from laboratory sample preparation. In addition, the reporting limit for the total and dissolved mercury analysis was adjusted due to sample volume restrictions.

The analysis for trace metals in the CWTP effluent sample detected concentrations for eight total metal, six recoverable metal, and six dissolved metal analytes, as shown in Table 5-6. However, the total and recoverable chromium analytical results may be influenced by contamination from laboratory sample preparations. In addition, the reporting limit for the total and dissolved mercury analysis was adjusted due to sample volume restrictions.

A.5.a. Sump 2 Discharge Effects--

A comparison of trace metal concentrations prior to and after Sump 2 discharge indicates there may have been an impact on the Sacramento River water quality. Of the six total metal concentrations detected in both pre-discharge and post-discharge samples, all six analyte concentrations were higher in the post-discharge (Captain's Table II) sample. Total silver was also detected at 0.00012 mg/l in the post-discharge water sample, but was not found in the pre-discharge sample.

Of the six recoverable metal concentrations detected in both river water samples for pre-discharge and post-discharge of Sump 2, five analytes were higher in the post-discharge (Captain's Table II) sample, while one analyte concentration was higher in the pre-discharge (Miller Park I) sample. However, recoverable arsenic was detected in the pre-discharge (Miller Park I) sample at a concentration of 0.0052 mg/l, but was not found in the post-discharge (Captain's Table II) sample.

Of the five dissolved metal concentrations detected in both sample pre- and post-discharge of Sump 2, four analyte concentrations were higher in pre-discharge (Miller Park I) sample, while one was higher in the post-discharge (Captain's Table II) sample. Dissolved arsenic was detected in the pre-discharge (Miller Park I) sample at a concentration of 0.0010 mg/l, but was not found above the reporting level in the post-discharge (Captain's Table II) sample.

A.5.b. CWTP Effluent Discharge Effects--

A comparison of metal concentrations prior to and after CWTP effluent discharge does not show a significant impact on the Sacramento River water quality. Of the six total metal concentrations detected in both pre-discharge and post-discharge river water sample samples, five analyte concentrations were higher in pre-discharge (Captain's Table I), while one was higher in the post-discharge (Wooden Stairs) sample. Total cadmium was detected in the pre-discharge (Captain's Table I) sample at 0.00055 mg/l, but was not detected in the post-discharge (Wooden Stairs) sample.

Of the five recoverable metal concentrations detected in both of CWTP's pre-discharge and post-discharge river water samples, all five analyte concentrations

were higher in the pre-discharge (Captain's Table I) sample. Recoverable arsenic and cadmium were also detected in the pre-discharge (Captain's Table I) sample at concentrations of 0.0071 mg/l and 0.00048 mg/l, respectively.

Of the four dissolved metal concentrations detected in both of CWTP's pre-discharge and post-discharge river water samples, three analyte concentrations were higher in the post-discharge (Wooden Stairs) sample, while one was higher in the pre-discharge (Captain's Table I). Dissolved arsenic and zinc were also detected in the pre-discharge (Captain's Table I) sample at concentrations of 0.0016 mg/l and 0.013 mg/l, respectively.

A.5.c. Comparison of Sump 2 and CWTP Effects--

A comparison of the analyzed trace metal constituents detected in both discharge locations, Sump 2 and CWTP, indicates strong similarities. The analysis for total metals detected concentrations for seven total metal analytes in the Sump 2 discharge and eight total metal analytes in the CWTP effluent samples. Of the metal concentrations detected in both discharges, all seven total metal analyte concentrations were higher in the Sump 2 sample. The analysis for recoverable metals in both samples detected concentrations for six analytes. All six recoverable analyte concentrations were higher in the Sump 2 sample.

The analytical results for dissolved metals in the Sump 2 discharge and CWTP effluent samples show detected concentrations for six analytes, respectively. Of the five dissolved metal concentrations detected in both samples, three analyte concentrations were higher in the CWTP sample and two analytes (chromium and lead) were at the same concentrations. Dissolved arsenic was detected in the Sump 2 sample at a concentration of 0.0027 mg/l, but was not found in the CWTP sample. Dissolved nickel was detected in the CWTP sample at a concentration of 0.0046 mg/l, but was not found above the reporting level in the Sump 2 sample.

A.5.d. Comparison to Water Quality Objectives--

A comparison of the total metal concentrations detected in the Sacramento River to the 1-hour and 4-day objective concentrations listed in Table 4-1 shows that several total metals exceeded their objectives. Total chromium and copper exceeded the 1-hour objective in the Captain's Table II and Wooden Stairs sample,

while total copper exceeded this objective in the Captain's Table I and the Miller Park I sample. Total copper and lead exceeded the 4-day objective in all of the river samples. Total chromium exceeded the objective in the Captain's Table II and the Wooden Stairs samples. In addition, total zinc exceeded the objective in the Captain's Table I and in the Captain's Table II sample.

A comparison of the recoverable metal concentrations detected in the Sacramento River to the 1-hour and 4-day objective concentrations shows that several recoverable metals exceeded their objectives. Recoverable chromium and copper exceeded the 1-hour and 4-day objectives in all of the river samples. Recoverable lead exceeded the 4-day objective in the Captain's Table II and the Miller Park I and the Wooden Stairs samples. Recoverable zinc also exceeded the 4-day objective in the Captain's Table I and the Captain's Table II samples.

Dissolved lead exceed the 4-day objective in the Captain's Table II and the Wooden Stairs samples. In addition, dissolved chromium exceed the 4-day objective in the Captain's Table II sample.

Four total metals, chromium, copper, lead, and zinc, exceeded both the 1-hour and the 4-day objective concentrations in the Sump 2 CSO sample.

A comparison of the recoverable metal concentrations detected in the Sump 2 CSO discharge to the 1-hour and the 4-day objective concentrations shows that four recoverable metals, chromium, copper, lead, and zinc, exceeded both objectives.

Dissolved lead exceeded the 4-day objective in the Sump 2 CSO sample.

A comparison of the total metal concentrations detected in the CWTP effluent discharge to the 1-hour and the 4-day objective concentrations shows that total copper and zinc exceeded both objectives. In addition, total chromium and lead exceeded the 4-day objective.

Recoverable copper and zinc exceeded both the 1-hour and the 4-day objective concentrations in the CWTP effluent discharge. In addition, recoverable chromium and lead exceeded the 4-day objective.

No dissolved metals exceeded the 1-hour or 4-day objective concentration in the CWTP effluent discharge.

A.6. Quality Control Results

As part of the quality assurance program, the laboratory split the river water samples collected upstream of the CWTP effluent discharge in half. (These were the "Captain's Table I" samples, collected upstream of the CWTP outfall, prior to the CSOs from Sump 2). These two sets of samples were then analyzed. One set of samples was labeled with an "SA" suffix in the analytical results and is referred to as the parent set of samples in this report. The other set of samples was labeled with a "DU" suffix in the analytical results and is referred to as the quality control sample set. This quality assurance methodology was used to verify the laboratory's ability to replicate precise and accurate analytical results.

A complete set of duplicate water samples was also collected from the Captain's Table dock, upstream of the CWTP outfall and prior to the CSO discharge from Sump 2. In the initial sampling plan, field duplicates (as opposed to sample splits), were intended to be used as the field quality control samples. However, the laboratory mistakenly discarded the field duplicates, due to confusion regarding sample labels and log-in upon delivery to the laboratory.

The sample split results proved to be a stringent measure of laboratory precision and accuracy. These split samples were analyzed "blind" by the laboratory, in random order by different laboratory personnel on different days. In addition, the laboratory analysts were not apprised of the sample location identification names during the analysis of the samples. Using this approach, the sample splits could be used as the field quality control check on the laboratory.

Table 5-7 compares and notes variances between the parent sample results and the quality control (duplicate) results. The variance was calculated by dividing the absolute value of the difference between the parent and QC sample results by the parent sample results (expressed as a percentage). Although several of the variances appear to be significant it should be noted that these variance

represent minute differences in actual analytical results. For example, total silver was detected at 0.00024 mg/l in the parent sample and 0.00014 mg/l in the quality control sample using a reporting limit of 0.0001 mg/l. The calculated variance of 41.7 percent for this constituent represents a 0.00010 mg/l difference in actual analytical results. To determine if this variance was significant, a survey of several laboratories was conducted and a general guideline was established. For sample split analytical results that are detected at less than five times the reporting limit, a variance range between 50 and 75 percent is acceptable. Using this guideline, the quality control results for the analysis compared favorably with those of their parent sample. Two notable exceptions exist; the parent sample results for fecal coliform and fecal streptococcus indicate concentrations of 24,000 MPN/100 ml for both analytes, while the quality control samples results indicate concentrations of 13,000 MPN/100 ml and 3,000 MPN/100 ml. These differences in analytical results represent significant variances.

TABLE 5-1. RESULTS FOR SACRAMENTO RIVER AT CAPTAIN'S TABLE I FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives [1] (1-Hour Average)	Objectives [1] (4-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	9.6		
Biological Oxygen Demand	mg/l	6.0	8.0		
Nitrate as N	mg/l	.05	0.36		
Total Kjeldahl Nitrogen as N	mg/l	.50	1.2		
Phosphorus, Total as P	mg/l	.05	0.59		
Phosphorus, Dissolved as P	mg/l	.05	0.31		
Total Dissolved Solids	mg/l	10.0	117		
Total Suspended Solids	mg/l	5.0	524		
pH	units	N/A	6.8		
Turbidity	NTU	40.0	364		
Hardness	mg/l	3.3	105		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0080	0.360	0.190
Cadmium	mg/l	.0004	0.00055	0.0039	0.0011
Chromium	mg/l	.0025	0.066	0.0160	0.0110
Copper	mg/l	.002	0.04520	0.0177	0.0118
Lead	mg/l	.0017	0.011	0.0812	0.0032
Silver	mg/l	.0001	0.00024	0.0040[2]	0.0040[2]
Zinc	mg/l	.0085	0.11	0.1166	0.1056
Trace Metals (Recoverable)					
Arsenic	mg/l	.005	0.0071	0.360	0.190
Cadmium	mg/l	.0004	0.00048	0.0039	0.0011
Chromium	mg/l	.0025	0.058	0.0160	0.0110
Copper	mg/l	.0025	0.046	0.0177	0.0118
Lead	mg/l	.0017	0.0088	0.0812	0.0032
Silver	mg/l	.0001	0.00017	0.0040[2]	0.0040[2]
Zinc	mg/l	.0085	0.11	0.1166	0.1056

TABLE 5-1. RESULTS FOR SACRAMENTO RIVER AT CAPTAIN'S TABLE I FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives [1] (1-Hour Average)	Objectives [1] (4-Day Average)
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0016	0.360	0.190
Chromium	mg/l	.0005	0.0010	0.0160	0.0110
Copper	mg/l	.0005	0.0057	0.0177	0.0118
Lead	mg/l	.0017	0.0018	0.0812	0.0032
Nickel	mg/l	.0025	0.0032	0.1571	1.4134
Zinc	mg/l	.0085	0.013	0.1166	0.1056
Bacteria					
Total Coliform	MPN/100ml	200	50,000		
Fecal Coliform	MPN/100ml	200	24,000		
Fecal Streptococcus	MPN/100ml	200	24,000		

1 Objective concentrations for trace metals are based on recoverable concentrations.
2 The objective concentration for silver is calculated as the instantaneous maximum.

TABLE 5-2. RESULTS FOR SACRAMENTO RIVER AT CAPTAIN'S TABLE II FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives [1] (1-Hour Average)	Objectives [1] (4-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	10.0		
Biological Oxygen Demand	mg/l	6.0	30.0		
Nitrate as N	mg/l	.05	0.35		
Total Kjeldahl Nitrogen as N	mg/l	.50	1.2		
Phosphorus, Total as P	mg/l	.05	0.54		
Phosphorus, Dissolved as P	mg/l	.05	0.22		
Total Dissolved Solids	mg/l	10.0	130		
Total Suspended Solids	mg/l	5.0	568		
pH	units	N/A	6.9		
Turbidity	NTU	40.0	403		
Hardness	mg/l	3.3	102		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0082	0.360	0.190
Cadmium	mg/l	.0004	0.00052	0.0039	0.0011
Chromium	mg/l	.002	0.048	0.0160	0.0110
Copper	mg/l	.0025	0.03550	0.0177	0.0118
Lead	mg/l	.0017	0.0083	0.0812	0.0032
Silver	mg/l	.0001	0.00012	0.0040[2]	0.0040[2]
Zinc	mg/l	.0085	0.11	0.1166	0.1056
Trace Metals (Recoverable)					
Cadmium	mg/l	.0004	0.00047	0.0039	0.0011
Chromium	mg/l	.0025	0.053	0.0160	0.0110
Copper	mg/l	.0025	0.042	0.0177	0.0118
Lead	mg/l	.0017	0.0094	0.0812	0.0032
Silver	mg/l	.0001	0.00017	0.0040[2]	0.0040[2]
Zinc	mg/l	.0085	0.11	0.1166	0.1056

TABLE 5-2. RESULTS FOR SACRAMENTO RIVER AT CAPTAIN'S TABLE II FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives [1] (1-Hour Average)	Objectives [1] (4-Day Average)
Trace Metals (Dissolved)					
Chromium	mg/l	.0005	0.0014	0.0160	0.0110
Copper	mg/l	.0005	0.0044	0.0177	0.0118
Lead	mg/l	.0017	0.0040	0.0812	0.0032
Nickel	mg/l	.0025	0.0034	0.1571	1.4134
Zinc	mg/l	.0085	0.044	0.1166	0.1056
Bacteria					
Total Coliform	MPN/100ml	200	50,000		
Fecal Coliform	MPN/100ml	200	30,000		
Fecal Streptococcus	MPN/100ml	200	5,000		

1 Objective concentrations for trace metals are based on recoverable concentrations.
2 The objective concentration for silver is calculated as the instantaneous maximum.

TABLE 5-3. RESULTS FOR SACRAMENTO RIVER AT MILLER PARK I FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (Four-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	13.9		
Nitrate as N	mg/l	.05	0.36		
Total Kjeldahl Nitrogen as N	mg/l	.50	1.2		
Phosphorus, Total as P	mg/l	.05	0.53		
Phosphorus, Dissolved as P	mg/l	.05	0.19		
Total Dissolved Solids	mg/l	10.0	120		
Total Suspended Solids	mg/l	5.0	790		
pH	units	N/A	7.1		
Turbidity	NTU	40.0	506		
Hardness	mg/l	3.3	99.6		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0080	0.360	0.190
Cadmium	mg/l	.0004	0.00046	0.0039	0.0011
Chromium	mg/l	.0025	0.040	0.0160	0.0110
Copper	mg/l	.0025	0.03060	0.0177	0.0118
Lead	mg/l	.0017	0.0063	0.0812	0.0032
Zinc	mg/l	.0085	0.10	0.1166	0.1056
Trace Metals (Recoverable)					
Arsenic	mg/l	.005	0.0052	0.360	0.190
Cadmium	mg/l	.0004	0.00044	0.0039	0.0011
Chromium	mg/l	.0025	0.046	0.0160	0.0110
Copper	mg/l	.0025	0.037	0.0177	0.0118
Lead	mg/l	.0017	0.0071	0.0812	0.0032
Silver	mg/l	.0001	0.00088	0.0040[2]	0.0040[2]
Zinc	mg/l	.0085	0.10	0.1166	0.1056

TABLE 5-3. RESULTS FOR SACRAMENTO RIVER AT MILLER PARK I FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (Four-Day Average)
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0010	0.360	0.190
Chromium	mg/l	.0005	0.0022	0.0160	0.0110
Copper	mg/l	.0005	0.0061	0.0177	0.0118
Lead	mg/l	.0017	0.0029	0.0812	0.0032
Nickel	mg/l	.0025	0.0044	0.1571	1.4134
Zinc	mg/l	.0085	0.021	0.1166	0.1056
Bacteria					
Total Coliform	MPN/100ml	200	30,000		
Fecal Coliform	MPN/100ml	200	8,000		
Fecal Streptococcus	MPN/100ml	200	13,000		

1 Objective concentrations for trace metals are based on recoverable concentrations.
2 The objective concentration standard for silver is calculated as the instantaneous maximum.

TABLE 5-4. RESULTS FOR SACRAMENTO RIVER AT WOODEN STAIRS FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	9.2		
Nitrate as N	mg/l	.05	0.34		
Total Kieldahl Nitrogen as N	mg/l	.50	1.2		
Phosphorus, Total as P	mg/l	.05	0.46		
Phosphorus, Dissolved as P	mg/l	.05	0.48		
Total Dissolved Solids	mg/l	10.0	133		
Total Suspended Solids	mg/l	5.0	450		
pH	units	N/A	7.0		
Turbidity	NTU	40.0	371		
Hardness	mg/l	3.3	89.6		
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0076	0.360	0.190
Chromium	mg/l	.005	0.083	0.0160	0.0110
Copper	mg/l	.0025	0.03120	0.0177	0.0118
Lead	mg/l	.0017	0.0067	0.0812	0.0032
Silver	mg/l	.0001	0.00017	0.0040[2]	0.0040[2]
Zinc	mg/l	.0085	0.082	0.1166	0.1056
Trace Metals (Recoverable)					
Chromium	mg/l	.0020	0.044	0.0160	0.0110
Copper	mg/l	.0025	0.034	0.0177	0.0118
Lead	mg/l	.0017	0.0066	0.0812	0.0032
Silver	mg/l	.0001	0.00014	0.0040[2]	0.0040[2]
Zinc	mg/l	.0085	0.087	0.1166	0.1056
Trace Metals (Dissolved)					
Chromium	mg/l	.0005	0.0013	0.0160	0.0110
Copper	mg/l	.0005	0.0042	0.0177	0.0118
Lead	mg/l	.0017	0.0053	0.0812	0.0032
Nickel	mg/l	.0025	0.0034	0.1571	1.4134

TABLE 5-4. RESULTS FOR SACRAMENTO RIVER AT WOODEN STAIRS FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
Bacteria					
Total Coliform	MPN/100ml	200	30,000		
Fecal Coliform	MPN/100ml	200	5,000		
Fecal Streptococcus	MPN/100ml	200	8,000		

1 Objective concentrations for trace metals are based on recoverable concentrations.
2 The objective concentration standard for silver is calculated as the instantaneous maximum.

TABLE 5-5. RESULTS FOR SUMP 2 DISCHARGE FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	13.9		
Biological Oxygen Demand	mg/l	6.0	45.0		
Nitrate as N	mg/l	.05	0.36		
Total Kjeldahl Nitrogen as N	mg/l	.50	3.8		
Phosphorus, Total as P	mg/l	.10	0.99		
Phosphorus, Dissolved as P	mg/l	.05	0.35		
Total Dissolved Solids	mg/l	10.0	43.0		
Total Suspended Solids	mg/l	5.0	387		
pH	units	N/A	6.4		
Turbidity	NTU	20.0	253		
Hardness	mg/l	3.3	34.3		
Oil and Grease	mg/l	1.0	2.2		
Organics					
Tributyltin	µg/l	.001	0.029	NA[3]	0.020
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0059	0.360	0.190
Cadmium	mg/l	.0004	0.00069	0.0039	0.0011
Chromium	mg/l	.001	0.030	0.0160	0.0110
Copper	mg/l	.0025	0.057	0.0177	0.0118
Lead	mg/l	.0034	0.11	0.0812	0.0032
Silver	mg/l	.0001	0.0074	0.0040[3]	0.0040[3]
Zinc	mg/l	.0085	0.21	0.1166	0.1056
Trace Metals (Recoverable)					
Cadmium	mg/l	.0004	0.00080	0.0039	0.0011
Chromium	mg/l	.001	0.024	0.0160	0.0110
Copper	mg/l	.005	0.054	0.0177	0.0118
Lead	mg/l	.0034	0.12	0.0812	0.0032
Silver	mg/l	.0001	0.0034	0.0040[3]	0.0040[3]
Zinc	mg/l	.0085	0.24	0.1166	0.1056

TABLE 5-5. RESULTS FOR SUMP 2 DISCHARGE FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0027	0.360	0.190
Chromium	mg/l	.0005	0.0011	0.0160	0.0110
Copper	mg/l	.0005	0.0048	0.0177	0.0118
Lead	mg/l	.0017	0.0021	0.0812	0.0032
Silver	mg/l	.0001	0.00041	0.0040[3]	0.0040[3]
Zinc	mg/l	.0085	0.034	0.1166	0.1056
Bacteria					
Total Coliform	MPN/100ml	200	2,400,000		
Fecal Coliform	MPN/100ml	200	500,000		
Fecal Streptococcus	MPN/100ml	200	500,000		

[1] Objective concentrations for trace metals are based on recoverable concentrations.

[2] A one-hour objective concentration for tributyltin does not exist.

[3] The objective concentration for silver is calculated as the instantaneous maximum.

TABLE 5-6. RESULTS FOR CWTP EFFLUENT FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
General Inorganics					
Chemical Oxygen Demand	mg/l	5.0	23.9		
Biological Oxygen Demand	mg/l	6.0	29.0		
Nitrate as N	mg/l	.05	1.1		
Total Kjeldahl Nitrogen as N	mg/l	.50	3.1		
Phosphorus, Total as P	mg/l	.10	0.82		
Phosphorus, Dissolved as P	mg/l	.05	0.58		
Total Dissolved Solids	mg/l	10.0	226		
Total Suspended Solids	mg/l	5.0	180		
pH	units	N/A	6.0		
Turbidity	NTU	20.0	168		
Hardness	mg/l	3.3	39.9		
Oil and Grease	mg/l	1.0	1.4		
Organics					
Tributyltin	µg/l	.001	0.017	NA[2]	0.020
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0045	0.360	0.190
Cadmium	mg/l	.0004	0.00048	0.0039	0.0011
Chromium	mg/l	.0005	0.015	0.0160	0.0110
Copper	mg/l	.0025	0.0273	0.0177	0.0118
Lead	mg/l	.0017	0.036	0.0812	0.0032
Mercury	mg/l	.0004	0.00042	0.0024	NA[3]
Silver	mg/l	.0001	0.0028	0.0040[4]	0.0040[4]
Zinc	mg/l	.0085	0.14	0.1166	0.1056
Trace Metals (Recoverable)					
Cadmium	mg/l	.0004	0.00043	0.0039	0.0011
Chromium	mg/l	.0005	0.015	0.0160	0.0110
Copper	mg/l	.0025	0.024	0.0177	0.0118
Lead	mg/l	.0017	0.036	0.0812	0.0032
Silver	mg/l	.0001	0.0017	0.0040[4]	0.0040[4]

TABLE 5-6. RESULTS FOR CWTP E NT FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Sample Results	Objectives[1] (1-Hour Average)	Objectives[1] (4-Day Average)
Zinc	mg/l	.0085	0.14	0.1166	0.1056
Trace Metals (Dissolved)					
Chromium	mg/l	.0005	0.0011	0.0160	0.0110
Copper	mg/l	.0005	0.0072	0.0177	0.0118
Lead	mg/l	.0017	0.0021	0.0812	0.0032
Nickel	mg/l	.0025	0.0046	0.1571	1.4134
Silver	mg/l	.0001	0.0016	0.0040	0.0040
Zinc	mg/l	.0085	0.073	0.1166	0.1056
Bacteria					
Total Coliform	MPN/100ml	200	90,000		
Fecal Coliform	MPN/100ml	200	8,000		
Fecal Streptococcus	MPN/100ml	200	2,200		

1 Objective concentrations for trace metals are based on recoverable concentrations.
2 A one-hour comparative standard for tributyltin does not exist.
3 A four-day objective concentration for mercury does not exist.
4 The comparative standard for silver is calculated as the instantaneous minimum.

TABLE 5-7. QUALITY CONTROL RESULTS FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Parent Sample Results	QC Sample Results	Variance[1]
General Inorganics					
Biological Oxygen Demand	mg/l	3.0	ND	5.0	42.0%[2]
Oil and Grease	mg/l	0.20	ND	ND	NC
Hardness	mg/l	3.3	56.1	52.7	6.1%
Trace Metals (Total)					
Arsenic	mg/l	.001	0.0080	0.0081	1.3%
Cadmium	mg/l	.0004	0.00055	0.00059	7.3%
Chromium	mg/l	.0025	0.066	0.061	7.6%
Copper	mg/l	.0020	0.04520	0.04550	0.7%
Lead	mg/l	.0017	0.011	0.011	0%
Mercury	mg/l	.0002	ND	ND	NC
Selenium	mg/l	.001	ND	ND	NC
Silver	mg/l	.0001	0.00024	0.00014	41.7%
Zinc	mg/l	.0085	0.11	0.12	9.1%
Trace Metals (Recoverable)					
Arsenic	mg/l	.005	0.0071	0.0054	23.9%
Cadmium	mg/l	.0004	0.00048	0.00052	8.3%
Chromium	mg/l	.0025	0.058	0.051	12.1%
Copper	mg/l	.0025	0.046	0.043[2]	6.5%
Lead	mg/l	.0017	0.0088	0.012	36.4%
Selenium	mg/l	.005	ND	ND	NC
Silver	mg/l	.0001	0.00017	0.00014	17.6%
Zinc	mg/l	.0085	0.11	0.11	0%

TABLE 5-7. QUALITY CONTROL RESULTS FOR FEBRUARY 14, 1992 EVENT

Constituents	Units	Reporting Limit	Parent Sample Results	QC Sample Results	Variance[1]
Trace Metals (Dissolved)					
Arsenic	mg/l	.001	0.0016	0.0017	6.3%
Cadmium	mg/l	.0004	ND	ND	NC
Chromium	mg/l	.0005	0.0010	0.0010	0%
Copper	mg/l	.0005	0.0057	0.0043	24.6%
Lead	mg/l	.0017	0.0018	ND	11.1%
Mercury	mg/l	.0002	ND	ND	NC
Nickel	mg/l	.0025	0.0032	0.0046	43.8%
Selenium	mg/l	.001	ND	ND	NC
Silver	mg/l	.0001	ND	ND	NC
Zinc	mg/l	.0085	0.013	0.012	7.7%
Bacteria					
Total Coliform	MPN/100ml	200	50,000	50,000	0%
Fecal Coliform	MPN/100ml	200	24,000	13,000	45.8%
Fecal Streptococcus	MPN/100ml	20	24,000	3,000	87.5%

1 Calculation for variance is as follows: absolute value of the difference between the parent and QC sample results, divided by the parent sample result, expressed as a percentage.

2 Recoverable copper reporting limit for the QC sample is 0.0020 mg/l.

3 Lead variance calculated using a value of 0.0016 mg/l for the QC sample.

TABLE 5-8. ORGANOCHLORINE PESTICIDE/PCB SUMMARY AND COMPARISON[4] FOR SUMP 2 CSO SAMPLE					
Constituent	Reporting Limit and "ND" Concentration[1] (ng/l)[3]	Objective Concentrations (ng/l)[3]		Exceedence Summary for Objective Concentrations[2]	
		Daily Average	Instantaneous Maximum	Daily Average	Instantaneous Maximum
chlordane	6.0	4.3	-----	U	-----
DDT[5]	1.3	1.0	-----	U	-----
dieldrin	0.6	1.9	-----	N	-----
endosulfan	1.3[6]	56.0	220.0	N	N
endrin	0.6	2.3	180.0	N	N
heptachlor	0.6	3.8	-----	N	-----
hexachlorocyclo-hexanegamma	0.31	80.0	-----	N	-----
PCBs	12.5[7]	14.0	-----	N	-----

1 For Sacramento River downstream of Sump 2 outfall, assumes 200:1 dilution of Sump 2 CSO sample reporting limit, for which all results were "ND".

2 Key: N = ND value is less than the objective concentration.
U = Unknown as to whether objective concentration would be exceeded.

3 ng/l = nanograms per liter or parts per trillion.

4 See Table 4-1 analytes. See text for discussion of tributyltin results.

5 Results identical for DDD; DDE estimate is 0.6, resulting in an "N" exceedence.

6 Estimate is for endosulfan II and sulfate; endosulfan I estimate is 0.5 ng/l.

7 PCBs consist of 7 compounds.

TABLE 5-9. ORGANOCHLORINE PESTICIDE/PCB SUMMARY AND COMPARISON[4] FOR CWTP EFFLUENT SAMPLE					
Constituent	Reporting Limit and "ND" Concentration[1] (ng/l)[3]	Objective Concentrations (ng/l)[3]		Exceedence Summary for Objective Concentrations[2]	
		Daily Average	Instantaneous Maximum	Daily Average	Instantaneous Maximum
chlordane	4.8	4.3	-----	U	-----
DDT[5]	1.0	1.0	-----	U	-----
dieldrin	0.5	1.9	-----	N	-----
endosulfan	1.0[6]	56.0	220.0	N	N
endrin	0.5	2.3	180.0	N	N
heptachlor	0.5	3.8	-----	N	-----
hexachlorocyclo-hexanegamma	0.25	80.0	-----	N	-----
PCBs	10.0[7]	14.0	-----	U	-----

1 For Sacramento River downstream of CWTP outfall, assumes 250:1 dilution of CWTP effluent sample reporting limit, for which all results were "ND".

2 Key: N = ND value is less than the objective concentration.
U = Unknown as to whether objective concentration would be exceeded.

3 ng/l = nanograms per liter or parts per trillion.

4 See Table 4-1 analytes. See text for discussion of tributyltin results.

5 Results identical for DDD; DDE estimate is 0.5 ng/l.

6 Estimate is for endosulfan II and sulfate; endosulfan I estimate is 0.5 ng/l.

7 PCBs consist of 7 compounds.

SECTION 6
TOXICITY PROGRAM

This section summarizes the toxicity sampling and analysis program, sample test results, and quality assurance/quality control results for the storm event of February 14, 1992.

A. BIOTOXICITY TESTING PROGRAM OVERVIEW

Provision 5 of the City's NPDES Permit requires that a biotoxicity study be completed for CWCS discharges, using 100-percent solutions of CWTP effluent and CSO discharges and the EPA's three-species test for chronic toxicity. If chronic toxicity is found in the 100-percent samples collected during the 1991/1992 wet weather season, then the City would be required to perform a dilution series analysis for the corresponding sampling locations during the 1992/1993 wet season.

During this storm event, chronic toxicity analyses were performed on 100-percent solutions of Sump 2 CSO and CWTP effluent discharges. Pioneer Reservoir did not overflow during this event, so sample material was not collected for this discharge.

The City voluntarily accelerated the toxicity testing program on CWTP effluent. During this CSO event, the City performed a chronic toxicity dilution series of CWTP effluent. Dilutions were prepared for the following effluent percentages in accordance with EPA protocols: 100, 50, 25, 12.5, 6.2. Dilutions of 3.2 and 1.6 percent were added to the series, based on the December 7, 1991 sampling event toxicity test results and the anticipated need for lower dilutions. These dilutions of the CWTP effluent were analyzed to define the No Observable Effect Concentration (NOEC) and the Lowest Observable Effect Concentration (LOEC) points for each of the biological test species. The dilution series was created by proportioning CWTP effluent and Sacramento River water (collected at Captain's Table dock during the storm event, and prior to upstream discharges). In order to determine baseline chronic toxicity for the CWTP dilution series, 100-percent river water was also analyzed for toxicity.

The following tests were performed on the 100-percent discharge (Sump 2 and CWTP) and Sacramento river samples, and on the CWTP effluent dilutions:

1. A 7-day, chronic fathead minnow (*Pimephales promelas*), static renewal, larval survival and growth test.
2. A 3-brood, 7-day, chronic cladoceran (*Ceriodaphnia dubia*), static renewal, survival and reproduction test.
3. A 4-day, chronic, algal (*Selenastrum capricornutum*), static, growth test.

Procedures for determining biotoxicity are defined in EPA 600/4-89/001 (*Short-term Methods for Estimating the Chronic Toxicity of Effluent and Receiving Water to Freshwater Organisms*) and EPA 440/4-85/032 (*Technical Support Document for Water Quality Based Toxic Control*). The EPA's three-species freshwater chronic toxicity tests were performed as specified in these protocols and NPDES permit. Method numbers are summarized in Table 6-1.

TABLE 6-1. CHRONIC TOXICITY TEST PARAMETERS AND METHOD NUMBERS

Test Specie	Type of Test	EPA Method
Fathead Minnow	Survival/Growth	1000
Ceriodaphnia (water flea)	Survival/Reproduction	1002
Selenastrum (algae)	Growth	1003

B. SAMPLE COLLECTION AND HANDLING

Toxicity sample collection for the Sump 2 and CWTP effluent discharges was similar to the inorganic constituent sample collection. Discrete samples were composited in 30-minute periods, such that each period throughout the discharge would be represented by an equal volume. The samples were refrigerated during transport to Aquatic Testing Laboratories in Ventura. All tests were initiated within the required holding time limits.

C. ANALYTICAL RESULTS

Aquatic Testing Laboratories analyzed the biotoxicity of the effluent in the dilution percentages previously described. Data resulting from the laboratory test results were entered into a data base for statistical analysis. EPA's TOXSTAT computer program was used to conduct statistical tests and to estimate the NOEC and LOEC endpoints. The three species freshwater chronic toxicity test results are summarized in the following subsections and tables.

C.1. Sacramento River Sample Results

The chronic toxicity of Sacramento River water was tested to provide baseline data for the CWTP dilution series tests. Test results were compared to the laboratory control for determining whether chronic toxicity was present in the river during this storm event at the Captain's Table dock location that could influence the results of the dilution series tests.

Fathead minnow survival and growth, *Ceriodaphnia* survival and reproduction, and *Selenastrum* growth rates were not affected in a 100-percent solution of Sacramento River water collected from the Captain's Table dock, as shown in Tables 6-2 through 6-7. It was therefore concluded that the river sample was not chronically toxic to any of the critical life stages for the three species. The river water was then used as the control for analyzing the CWTP dilution series results.

TABLE 6-2. CHRONIC TOXICITY OF SACRAMENTO RIVER WATER ON FATHEAD MINNOWS

Sample	Percent Survival	Larval Mean Dry Weight
Control-Lab	100.0%	0.36 mg
Sacramento River	100.0%	0.40 mg

Control-Lab is a laboratory control water (20% DMW).

Sacramento River is water collected from the Captain's Table dock.

TABLE 6-3. SUMMARY OF SACRAMENTO RIVER CHRONIC TOXICITY ENDPOINTS FOR FATHEAD MINNOWS		
Endpoint	Survival	Growth
NOEC	NS	NS
NS is not significantly less than control.		

TABLE 6-4. CHRONIC TOXICITY OF SACRAMENTO RIVER WATER ON CERIODAPHNIA		
Sample	Percent Survival	Mean Number of Young per Female
Control-Lab	100.0%	16.7
Sacramento River	90.0%	27.8
Control Lab is laboratory control water (20% DMW) Sacramento River is water collected from the Captain's Table dock.		

TABLE 6-5. SUMMARY OF SACRAMENTO RIVER WATER CHRONIC TOXICITY ENDPOINTS FOR CERIODAPHNIA		
Endpoint	Survival	Reproduction
NOEC	NS	NS
NS is not significantly less than control.		

TABLE 6-6. CHRONIC TOXICITY OF SACRAMENTO RIVER WATER ON SELENASTRUM	
Sample	Mean Fluorescence
Control-Lab	240.8
Sacramento River	311.5
Relative fluorescence values are in-vivo relative content measurements taken with a fluorometer equipped with a chlorophyll filter. Fluorescence value of 210 is approximately 206,000 cells/ml based on actual cell counts of the control test solution. Control-Lab is laboratory control (20% DMW). Sacramento River is water collected from the Captain's Table dock.	

TABLE 6-7. SUMMARY OF SACRAMENTO RIVER WATER CHRONIC TOXICITY FOR SELENASTRUM	
NOEC = NS	LOEC = NA
NS is not significantly less than control. NA is not applicable.	

C.2. Sump 2 CSO Discharge Sample Results

Laboratory water was used as the control to assess the chronic toxicity for the Sump 2 CSO discharge sample. The results are summarized in Tables 6-8 through 6-13. Fathead minnow growth, *Ceriodaphnia* reproduction, and *Selenastrum* growth rates were not affected in a 100-percent solution of Sump 2 discharge . Fathead minnow survival and *Ceriodaphnia* survival were both affected in a 100-percent discharge solution. Therefore, the 100-percent Sump 2 discharge collected during this event was chronically toxic to fathead minnow survival and *Ceriodaphnia* survival.

TABLE 6-8. CHRONIC TOXICITY OF SUMP 2 DISCHARGE ON FATHEAD MINNOWS		
Sample	Percent Survival	Larval Mean Dry Weight
Control-Lab	100.0%	0.36 mg
Sump 2 Discharge	77.5%*	0.36 mg
Control-Lab is a laboratory control water (20% DMW). Sump 2 Discharge is a 100% effluent solution. * Significantly lower than control.		

TABLE 6-9. SUMMARY OF SUMP 2 DISCHARGE CHRONIC TOXICITY ENDPOINTS FOR FATHEAD MINNOWS		
Endpoint	Survival	Growth
NOEC	SL	NS
SL is significantly less than control. NS is not significantly less than control.		

TABLE 6-10. CHRONIC TOXICITY OF SUMP 2 DISCHARGE ON CERIODAPHNIA		
Sample	Percent Survival	Mean Number of Young per Female
Control-Lab	100.0%	16.7
Sump 2 Discharge	0.0%*[1]	20.9

Control-Lab is laboratory control water (20% DMW)

Sump 2 Discharge is a 100% CSO discharge solution.

* Significantly less than control.

[1] For the first several days of the analysis, the survival and reproductive rates were not significantly less than control. However, during the last two days of the analysis, the adult population died. This sequence of events supports the endpoint value of 0.0% survival while maintaining a reproductive rate.

TABLE 6-11. SUMMARY OF SUMP 2 DISCHARGE CHRONIC TOXICITY ENDPOINTS FOR CERIODAPHNIA		
Endpoint	Survival	Reproduction
NOEC	SL	NS

SL is significantly less than control.

NS is significantly less than control.

TABLE 6-12. CHRONIC TOXICITY OF SUMP 2 DISCHARGE ON SELENASTRUM	
Sample	Mean Fluorescence
Control-Lab	240.8
Sump 2 Discharge	581.0

Relative fluorescence values are in-vivo relative content measurements taken with a fluorometer equipped with a chlorophyll filter.

Fluorescence value of 210 is approximately 206,000 cells/ml based on actual cell counts of the control test solution.

Control-Lab is laboratory control (20% DMW).

Sump 2 Discharge is a 100% CSO discharge solution.

TABLE 6-13. SUMMARY OF SUMP 2 DISCHARGE CHRONIC TOXICITY FOR SELENASTRUM	
NOEC = NS	LOEC = NA
NS is not significantly less than control. NA is not applicable.	

The initial analysis for Sump 2 used the laboratory water as the control. However, further inspection and analysis of the data indicates that the results would be identical using the river water (Captain's Table) as the control for statistical analyses, except for one test. The *Ceriodaphnia* growth results for the Sump 2 sample are significantly different than that of the river control water.

C.3. CWTP Effluent Sample Results

Tables 6-14 and 6-15 show that fathead minnow survival and growth were not significantly affected in a 100-percent effluent solution, therefore, a survival NOEC at 100-percent effluent was determined.

Ceriodaphnia survival and reproduction were affected in a 100-percent effluent solution, therefore, an NOEC at 50-percent effluent was determined for both life stages of specie, as shown in Table 6-16 and 6-17. The corresponding LOEC was at 100-percent effluent.

Selenastrum growth rates were affected in a 6.2-percent solution of CWTP effluent as shown in Table 6-18. Since the 6.2-percent solution was the smallest dilution prepared for this specie, an NOEC could not be determined as shown in Table 6-19. The LOEC was for the 6.2-percent effluent sample.

TABLE 6-14. CHRONIC TOXICITY OF CWTP EFFLUENT ON FATHEAD MINNOWS		
Sample Concentration	Percent Survival	Larval Mean Dry Weight
Control	100.0%	0.32 mg
1.6% Effluent	100.0%	0.34 mg
3.2% Effluent	100.0%	0.37 mg
6.2% Effluent	100.0%	0.31 mg
12.5% Effluent	100.0%	0.38 mg
25.0% Effluent	100.0%	0.33 mg
50.0% Effluent	100.0%	0.36 mg
100.0% Effluent	100.0%	0.32 mg
Control is the dilution water control collected from the river at the Captain's Table dock.		

TABLE 6-15. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY ENDPOINTS FOR FATHEAD MINNOWS		
Endpoints	Survival	Growth
NOEC	100.0%	100.0%
LOEC	NA	NA
NA is not applicable.		

TABLE 6-16. CHRONIC TOXICITY OF CWTP EFFLUENT ON CERIODAPHNIA		
Sample	Percent Survival	Mean Number of Young per Female
Control	80.0%	22.7
1.6% Effluent	100.0%	28.3
3.2% Effluent	100.0%	26.1
6.2% Effluent	100.0%	29.0
12.5% Effluent	100.0%	25.9
25.0% Effluent	100.0%	26.1
50.0% Effluent	100.0%	18.3
100.0% Effluent	0.0%*	0*

Control is the dilution water control collected from the river at the Captain's Table dock.

* Significantly less than control.

TABLE 6-17. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY ENDPOINTS FOR CERIODAPHNIA		
Endpoint	Survival	Reproduction
NOEC	50.0%	50.0%
LOEC	100.0%	100.0%

TABLE 6-18. CHRONIC TOXICITY OF CWTP EFFLUENT ON SELENASTRUM	
Effluent Concentration	Mean Fluorescence
Control	453.0
6.2% Effluent	339.8*
12.5% Effluent	320.2*
25.0% Effluent	327.8*
50.0% Effluent	260.2*
100.0% Effluent	208.2*

Relative fluorescence values are in-vivo relative content measurements taken with a fluorometer equipped with a chlorophyll filter.

Fluorescence value of 240 is approximately 210,000 cells/ml based on actual cell counts of the control test solution.

Control is dilution water collected from the river at the Captain's Table dock.

* Significantly less than control.

TABLE 6-19. SUMMARY OF CWTP EFFLUENT CHRONIC TOXICITY FOR SELENASTRUM	
NOEC = < 6.2%	LOEC = 6.2%

D. QUALITY ASSURANCE/QUALITY CONTROL

The three test species were tested with reference toxicants to determine whether the cultured species exhibited normal toxicity. Reference toxicant data were included in Appendix B submitted as a separate document. Fathead minnows were tested with a reference toxicant of sodium dodecyl sulfate (SDS). The SDS NOEC for fathead minnow survival and growth was 2.5 mg/l. *Ceriodaphnia* were tested with a reference toxicant of sodium chloride (NaCl). The NaCl NOEC for *Ceriodaphnia* was 1,000 mg/l for both survival and reproduction. *Selenastrum* were tested with a reference toxicant of copper sulfate. The chronic toxicity NOEC for *Selenastrum* was 0.010 mg/l. The reference toxicant analyses indicated that the three species exhibited a "normal" toxic response to their respective toxicants.

The control solution used for the biotoxicity analysis was Sacramento River water collected from Captain's Table dock during this CSO event. None of the species used in the analysis were adversely affected by conditions in the control sample waters.

SECTION 7
SUMMARY

This section provides a summary of the analytical program for the CSO event which occurred on February 14, 1992. During this event, CWTP discharged effluent and Sump 2 overflowed to the Sacramento River.

A. OBJECTIVES MET BY THE CSO EVENT

Based on the elements of the 1991/1992 CWCS Sampling Program, the following objectives were met by this CSO event:

1) The chronic toxicity of Sacramento River water, the Sump 2 discharge, and CWTP effluent was determined for event samples on three test species (fathead minnow, *Ceriodaphnia* and *Selenastrum*). These test samples were undiluted, i.e. 100 percent river water or discharge samples. Toxicity tests for dilutions of the CWTP effluent were conducted for the following percentages of effluent: 100, 50, 25, 12.5, 6.2, 3.2, and 1.2. River water was used for preparing the dilution series test samples. The objective was to determine the significant impact, from a statistical standpoint, of sample waters on each of the three species on critical life stages. The NOEC and LOEC represent this statistical significance.

2) The effect of pollutants (inorganics, bacteria and metals) discharged to the Sacramento River from Sump 2 and CWTP were measured by collecting and analyzing river water samples collected in the respective discharge plumes. The "effect" or impact was determined by comparing upstream, background river sample constituent concentrations to those downstream of the discharges, and by comparing analytical results for the river to the *Inland Surface Waters Plan*. Concentrations of pollutants in the Sump 2 CSO and CWTP effluent discharges were measured by collecting and analyzing samples of these matrices.

3) The presence of organochlorine pesticides and PCBs in Sump 2 and CWTP discharges was measured by collecting and analyzing samples during this

CSO event. A special analysis was performed for tributyltin.

B. SUMMARY OF RESULTS

B.1. Chronic Toxicity

EPA's three-species freshwater chronic toxicity tests were used as specified in the NPDES permit. EPA's TOXSTAT computer program was used to perform the statistical computations on the laboratory results.

B.1.a. Sacramento River Sample--

An analysis of biotoxicity of the Sacramento River was conducted to determine a baseline for the control waters used in the CWTP effluent dilution series. Fathead minnow survival and growth, *Ceriodaphnia* survival and reproduction, and *Selenastrum* growth rates were not affected in a 100-percent solution of Sacramento River water collected from the Captain's Table dock. Therefore, chronic toxicity was not present. The river water therefore could serve as the control for evaluating CWTP dilution series results.

B.1.b. Sump 2 Sample--

Fathead minnow growth, *Ceriodaphnia* reproduction and *Selenastrum* growth rates were not affected in a 100-percent solution of Sump 2 discharge. However, fathead minnow and *Ceriodaphnia* survival were both affected in a 100-percent discharge solution, therefore, survival was determined to be significantly less than the control for these species.

B.1.c. CWTP Effluent--

Toxicity tests were conducted for specified dilutions of the CWTP effluent. Fathead minnow survival and growth were not affected in a 100-percent CWTP effluent solution, therefore, the NOEC for survival and growth was 100-percent for the effluent. *Ceriodaphnia* survival and reproduction were significantly affected in a 100-percent effluent solution. Therefore, the NOEC for both survival and growth was determined to be the 50-percent effluent solution, and the corresponding LOEC was the 100-percent effluent solution. *Selenastrum* growth rates were significantly affected in a 6.2-percent solution of CWTP effluent.

Since the 6.2-percent solution was the smallest dilution conducted, a definitive NOEC could not be determined. The LOEC was the 6.2-percent effluent solution.

The "safe concentration" for these particular samples, species and critical life stages is the concentration between the LOEC and NOEC. This safe concentration is biologically, rather than only statistically, significant. Assuming *Selenastrum* to be the most sensitive specie, an appropriate river dilution to ensure normal propagation could not be estimated, since an NOEC could not be determined. During this event, the river provided approximately 250:1 dilution for CWTP effluent downstream of the outfall.

B.2. General Inorganics

The Sump 2 CSO and CWTP effluent discharges had an insignificant overall effect on the Sacramento River's inorganic water quality. Comparison between the Miller Park I sample, collected upstream of the Sump 2 discharge, and the Captain's Table II sample, collected in the effluent discharge plume of Sump 2, indicates comparable analytical results. The results from the Wooden Stairs sample collected in the effluent discharge plume from CWTP are comparable to the upstream Captain's Table I sample. Comparing the results from the Captain's Table I sample, collected upstream of the effluent discharge from CWTP, and Captain's Table II sample, collected after the effluent discharge from Sump 2, also indicates comparable results.

The analysis of general inorganics constituents for the two discharge locations, Sump 2 and CWTP, indicates comparable results, with the exception of total dissolved solids and total suspended solids.

B.3. Bacteria

The bacteria results of the river samples do not suggest a uniform trend, that can be correlated by pre- and post-discharge values, of impacts due to either Sump 2 or the CWTP effluent.

Bacteria counts in the CWTP samples were considerably lower than Sump 2 discharge

samples, which reflects the effective chlorination treatment at the CWTP facility.

B.4. Organochlorine Pesticides/PCBs and Tributyltin

The organochlorine pesticide and PCB analytes were not analyzed within the two-week holding time due to a laboratory log-in error. Organochlorine pesticides and PCBs were not detected in either the samples of Sump 2 discharge or samples of CWTP effluent, using standard EPA method analyses. However, all of reporting limits were raised due to matrix interferences.

Tributyltin was detected in Sump 2 CSO and CWTP effluent discharges at 0.029 µg/l and 0.017 µg/l, respectively, using a reporting limit of 0.001 µg/l. A comparison of the tributyltin concentrations detected in the Sump 2 discharge to the 4-day objective concentration shows that this analyte exceeded the objective. A comparison of the tributyltin concentrations detected in the CWTP effluent to the 4-day objective concentration shows that this analyte did not exceed the objective concentration. (Tributyltin does not have a 1-hour objective concentration.)

B.5. Oil and Grease

Oil and grease concentrations were not detected in any of the river samples, but were detected at 2.2 mg/l and 1.4 mg/l, in Sump 2 discharge and CWTP effluent, respectively.

B.6. Trace Metals

A comparison of the total metal concentrations detected in the Sacramento River to the 1-hour and 4-day objective concentrations listed in Table 4-1 shows that several total metals exceeded their objectives. Total chromium and copper exceeded the 1-hour objective in the Captain's Table II and Wooden Stairs sample, while total copper exceeded this objective in the Captain's Table I and the Miller Park I sample. Total copper and lead exceeded the 4-day objective in all of the river samples. Total chromium exceeded the objective in the Captain's

Table II and the Wooden Stairs samples. In addition, total zinc exceeded the objective in the Captain's Table I and in the Captain's Table II sample.

A comparison of the recoverable metal concentrations detected in the Sacramento River to the 1-hour and 4-day objective concentrations shows that several recoverable metals exceeded their objectives. Recoverable chromium and copper exceeded the 1-hour and 4-day objectives in all of the river samples. Recoverable lead exceeded the 4-day objective in the Captain's Table II and the Miller Park I and the Wooden Stairs samples. Recoverable zinc also exceeded the 4-day objective in the Captain's Table I and the Captain's Table II samples.

Dissolved lead exceeded the 4-day objective in the Captain's Table II and the Wooden Stairs samples. In addition, dissolved chromium exceeded the 4-day objective in the Captain's Table II sample.

Four total metals, chromium, copper, lead, and zinc, exceeded both objectives, the 1-hour and the 4-day objective concentrations in the Sump 2 CSO sample.

A comparison of the recoverable metal concentrations detected in the Sump 2 CSO discharge to the 1-hour and the 4-day objective concentrations shows that four recoverable metals, chromium, copper, lead, and zinc, exceeded both objectives.

Dissolved lead exceeded the 4-day objective in the Sump 2 CSO sample.

A comparison of the total metal concentrations detected in the CWTP effluent discharge to the 1-hour and the 4-day objective concentrations shows that total copper and zinc exceeded both objectives. In addition, total chromium and lead exceeded the 4-day objective.

Recoverable copper and zinc exceeded both the 1-hour and the 4-day objective concentrations in the CWTP effluent discharge. In addition, recoverable chromium and lead exceeded the 4-day objective.

No dissolved metals exceeded the 1-hour or 4-day objective concentration in the CWTP effluent discharge.

APPENDIX E

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM
IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 4

FLOODWATER SAMPLING LOCATION RECOMMENDATIONS

Judy A. Zavadil
Paula M. Arsenault
HDR Engineering, Inc.

August 13, 1992

TECHNICAL MEMORANDUM - FINAL

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN

PHASE 3, TASK 4

PREPARED BY: Judy Zavadil, HDR Engineering, Inc.
Paula Arsenault, HDR Engineering, Inc.

DATE: November 15, 1991 (Revised December 17, 1991)

SUBJECT: Floodwater Sampling Location Recommendations

A. INTRODUCTION

The report entitled *Effluent and Receiving Water Quality Assessment Study for 1990/1991 and Sampling Program for 1991/1992* (dated November, 1991) describes the sampling program that will be implemented for the upcoming wet season, including sampling of local street floodwater. The purpose of the floodwater sampling is to estimate the source of bacterial contamination and, if possible, to identify locations having consistent sewage back-ups onto streets (i.e. outflows) during storm events.

City personnel will sample floodwater at six locations. These samples will be grabs and will be analyzed only for bacterial count. This procedure is identical to the program that was implemented during the 1990/1991 wet season, except the six locations are intended to be "permanent" sampling sites for the year. The objective of this memorandum is to identify potential sampling sites. From these sites the City will select the final six locations.

B. SAMPLING LOCATIONS

B.1. Selection Process

Several potential sampling locations were identified for water quality sampling and analysis. Sites were chosen considering known low-elevation areas and known locations of street flooding, using maps developed by the City to identify trouble call locations. The following maps supplied by the City were consulted

to determine locations with consistent flooding problems, according to Rain Patrol records:

1) Rain Patrol Trouble Calls - January 12, 1990 Storm
2) Rain Patrol Trouble Calls - February 17 and 18, 1986 Storm
3) Reports of Property Damage Due to Flooding (for the two Storms listed above)

The first two maps on the list were most useful in aiding the selection of potential sampling locations, since they represent actual field conditions during a storm event. Trouble calls that occur in the same general location for several storms serve as an indication that a flooding or outflow problem exists, which may consequently represent a suitable sampling location. Property damage reports may also suggest a system problem exists, if reports have been filed consistently for a particular location. It is important to note, however, that this information is highly dependent on an individual's willingness and ability to supply information to the City.

The actual occurrence of flooding for a given storm event is dependent on several factors: the intensity of rain over time during the event, the distribution of rain (storm cells) over the service area, operation of Sumps 1A and 2, and sewer system hydraulic conditions given the other factors. The general pattern of these elements from storm to storm will determine whether flooding and/or outflows occur consistently at a particular location or only intermittently (i.e., highly storm-dependent). A comparison of the City's two trouble call maps reviewed by HDR confirms the variability of these elements.

Thirteen potential sites were identified initially, and assigned a priority of "1" or "2". The six priority 1 sites should be included in the program if possible, while priority 2 sites can be used as substitutes if the others are not suitable from a flooding or sample collection standpoint. Each of these sites should be reviewed and then visited in the field by City personnel to ensure that sampling can be conducted safely during a rain storm.

The factors used in judging whether a site should be assigned a priority 1 or 2 included:

1) The relative potential of a public health problem, i.e. a sanitary sewer outflow versus street flooding or ponding;
2) Accessibility for sampling;
3) Potential consistency of flooding.

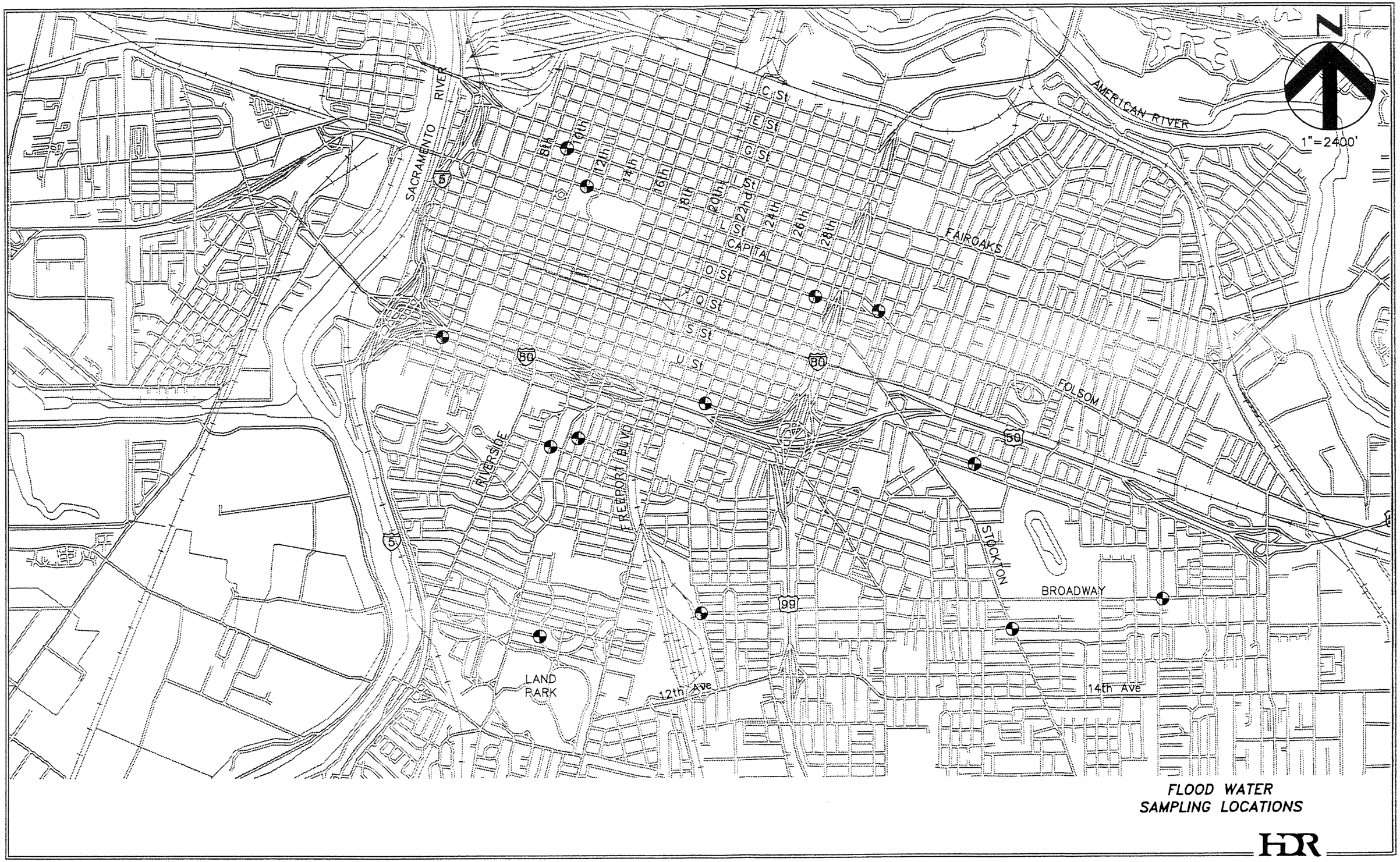
It is important to note that the manhole numbers provided to identify the potential sampling locations are for reference purposes; samples should actually be taken from the street intersection where these manholes are located.

B.2. Recommendations

Table 1 lists the locations from which the six sampling locations may be chosen, and all 13 are illustrated on Figure 1. Detailed sewer book maps with potential sites identified are provided in the Appendix.

TABLE 1. POTENTIAL FLOODWATER SAMPLING LOCATIONS				
ID No.	Location/Reason for Selection	Sewer Book Map	Manhole No.	Priority
FS-1	28th St. and N St. (low-lying area near Paragary's and known problem area)	14c	67	1
FS-2	9th St. and J St. (flooding problems indicated by Rain Patrol map 2 and general flooding in the area indicated by SWMM)	12b	221	1
FS-3	11th St. and L St. (flooding problems indicated by maps 1 and 2 and general flooding in the area indicated by SWMM; property damage reported)	13d	34	1
FS-4	42nd St. and U St. (north of UCD Medical Center; extensive flooding indicated by maps 1 and 2 and SWMM)	21c	37	1
FS-5	17 St. and 12th Ave. (north of William Land Park; known flooding location and indicated by maps 1 and 2; property damage reported in the surrounding area; low-lying area)	33d	55	1
FS-6	58th St. and Broadway (possible sanitary backup problems indicated by map 2; flooding indicated on map 1; property damage reports in the area)	19e	16	1
FS-7	23rd St. and W St. (flooding indicated by maps 1 and 2; property damage reported in the area surrounding intersection)	22d	70	2
FS-8	5th St. and Broadway (flooding indicated by maps 1 and 2; flooding indicated by the model from a major interceptor)	24c	24	2

TABLE 1. POTENTIAL FLOODWATER SAMPLING LOCATIONS				
ID No.	Location/Reason for Selection	Sewer Book Map	Manhole No.	Priority
FS-9	32nd St. and N St. (flooding indicated by maps 1, 2 and 3 and SWMM; flooding potentially occurs due to backups from a major interceptor)	15d	20	2
FS-10	14th St. and 2nd Ave. (outlet location for representative collection system basin; flooding indicated by maps; property damage reports in the area)	26d	37	2
FS-11	Marty Way and Larkin Way (known flooding indicated by maps 1 and 2; possible backups from major interceptor; property damage reported in the area)	26a	75	2
FS-12	24th St. and 6th Ave. (Curtis Park area; flooding indicated by maps 1 and 2)	32a	36	2
FS-13	Stockton Blvd. and 8th Ave. (flooding indicated by map 1; extensive flooding shown by map 2; property damage reported in the area)	31b	68	2



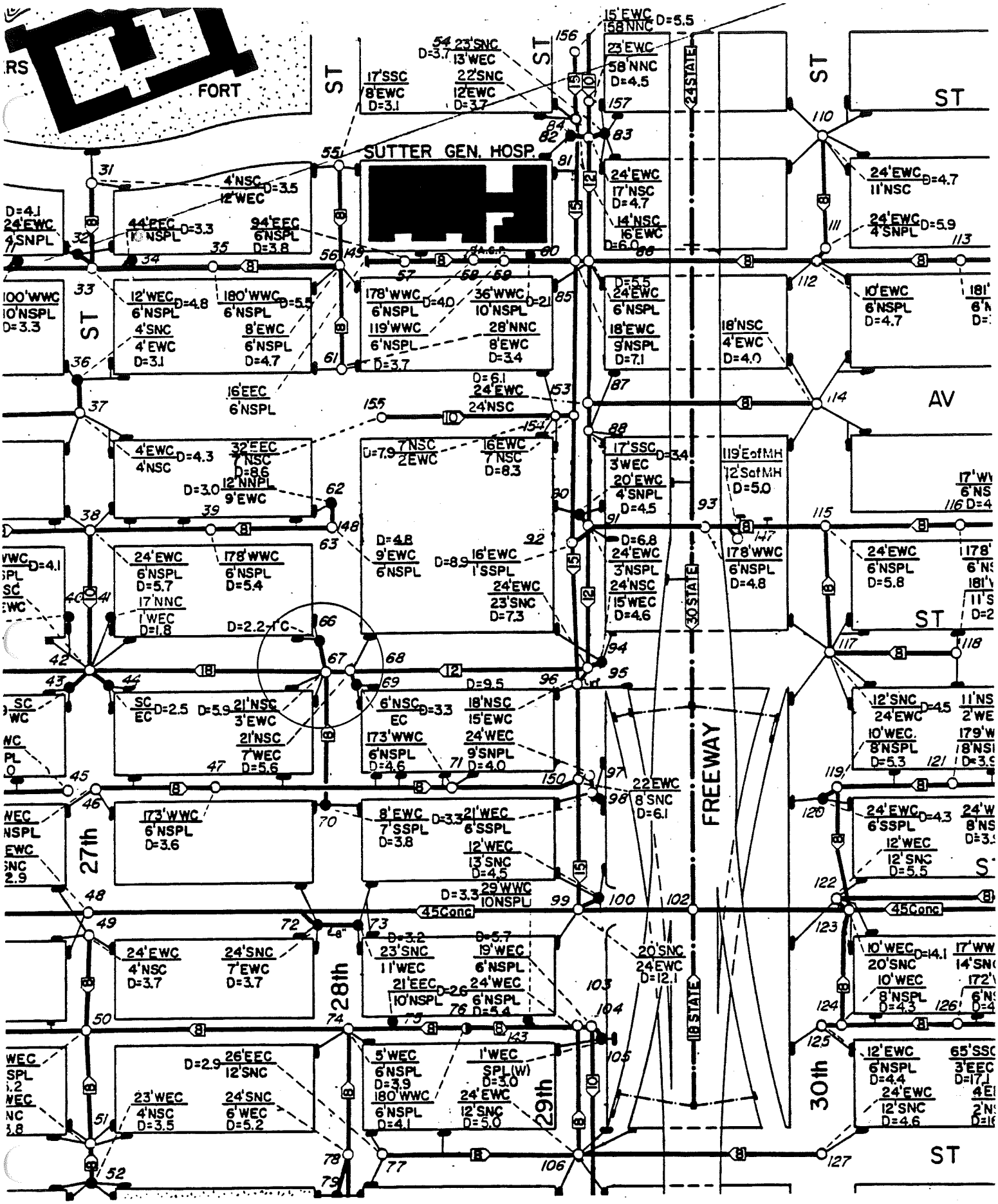
FLOOD WATER
SAMPLING LOCATIONS

HDR

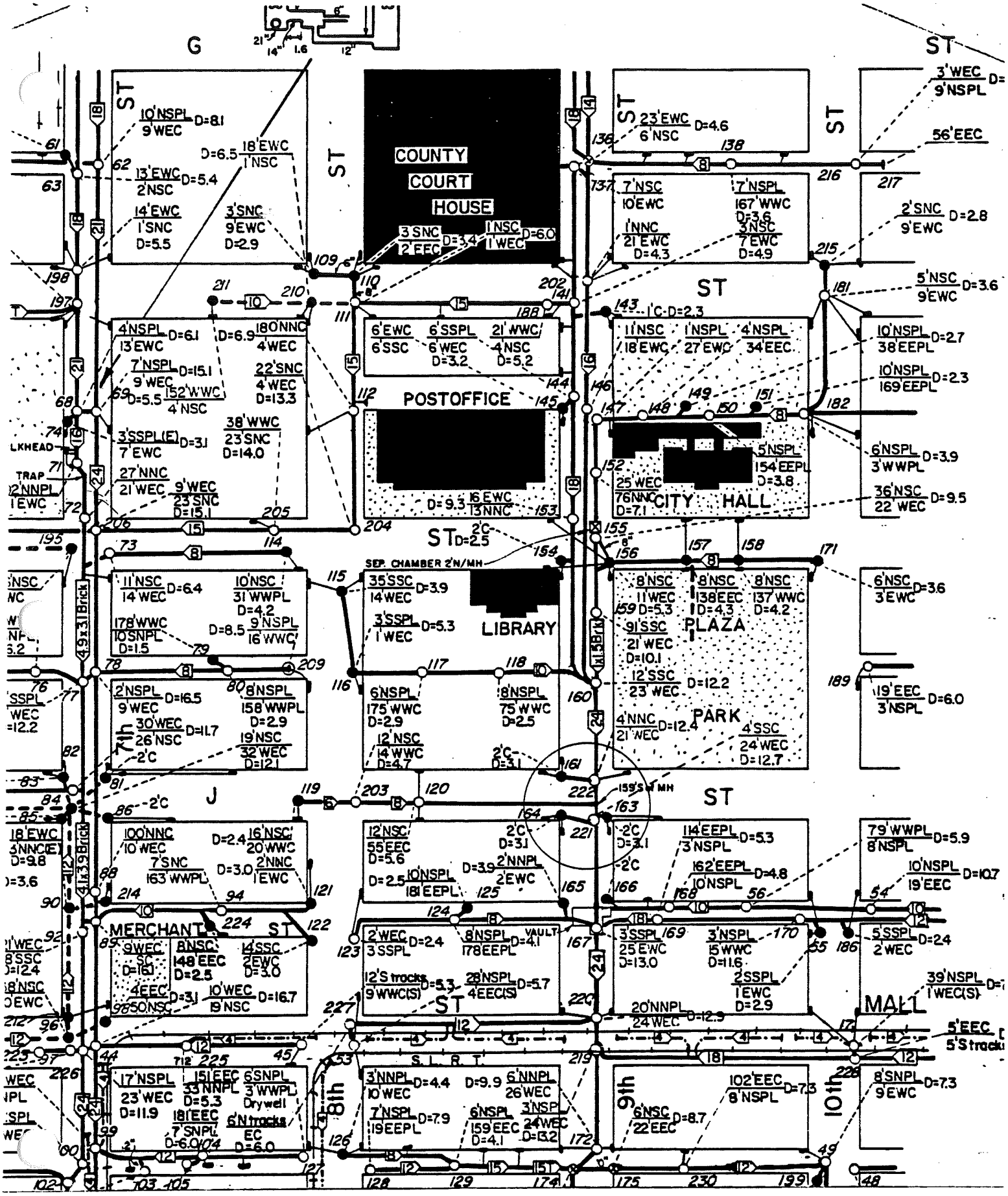
Figure 1

APPENDIX

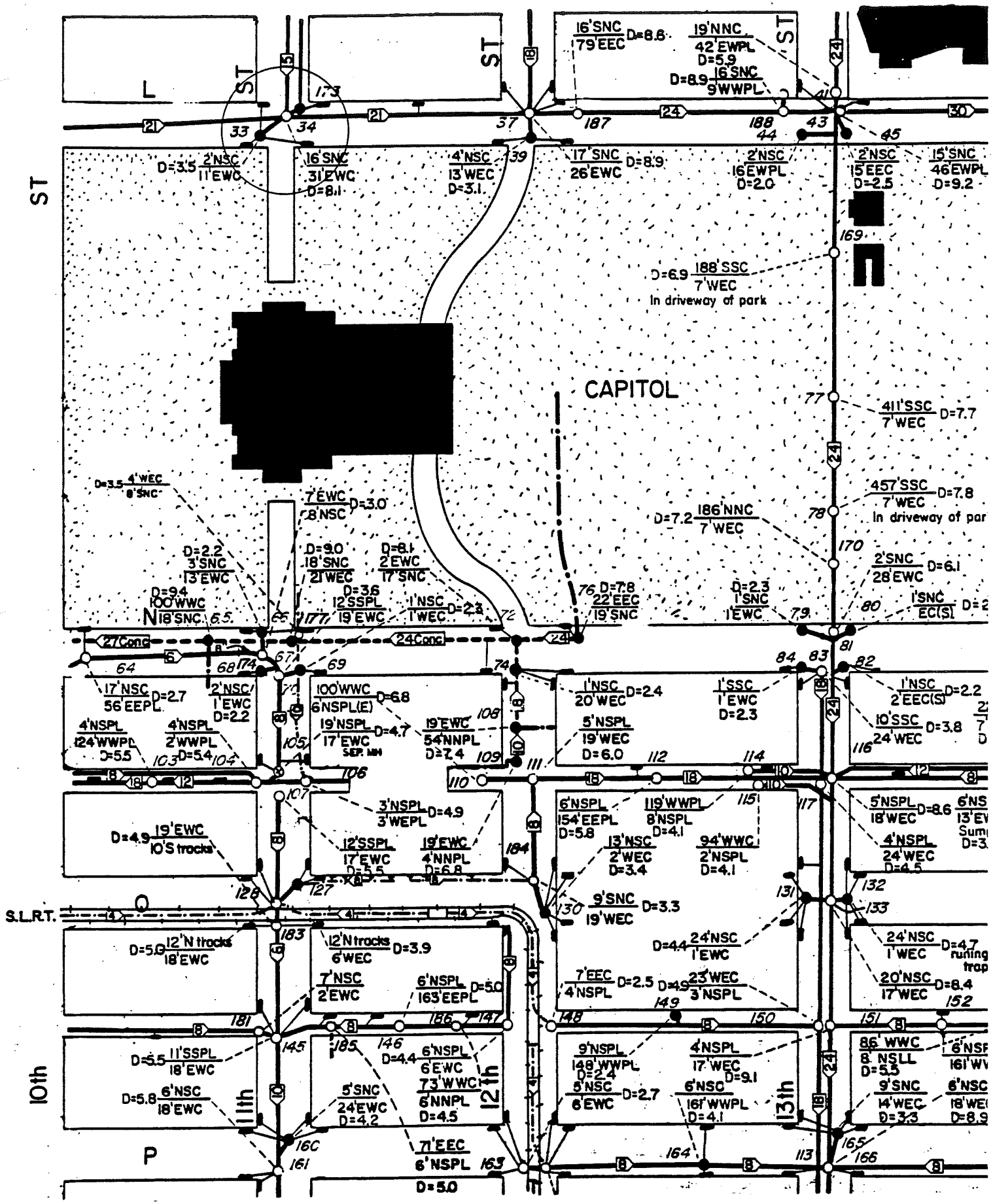
Potential Floodwater Sampling Location Maps



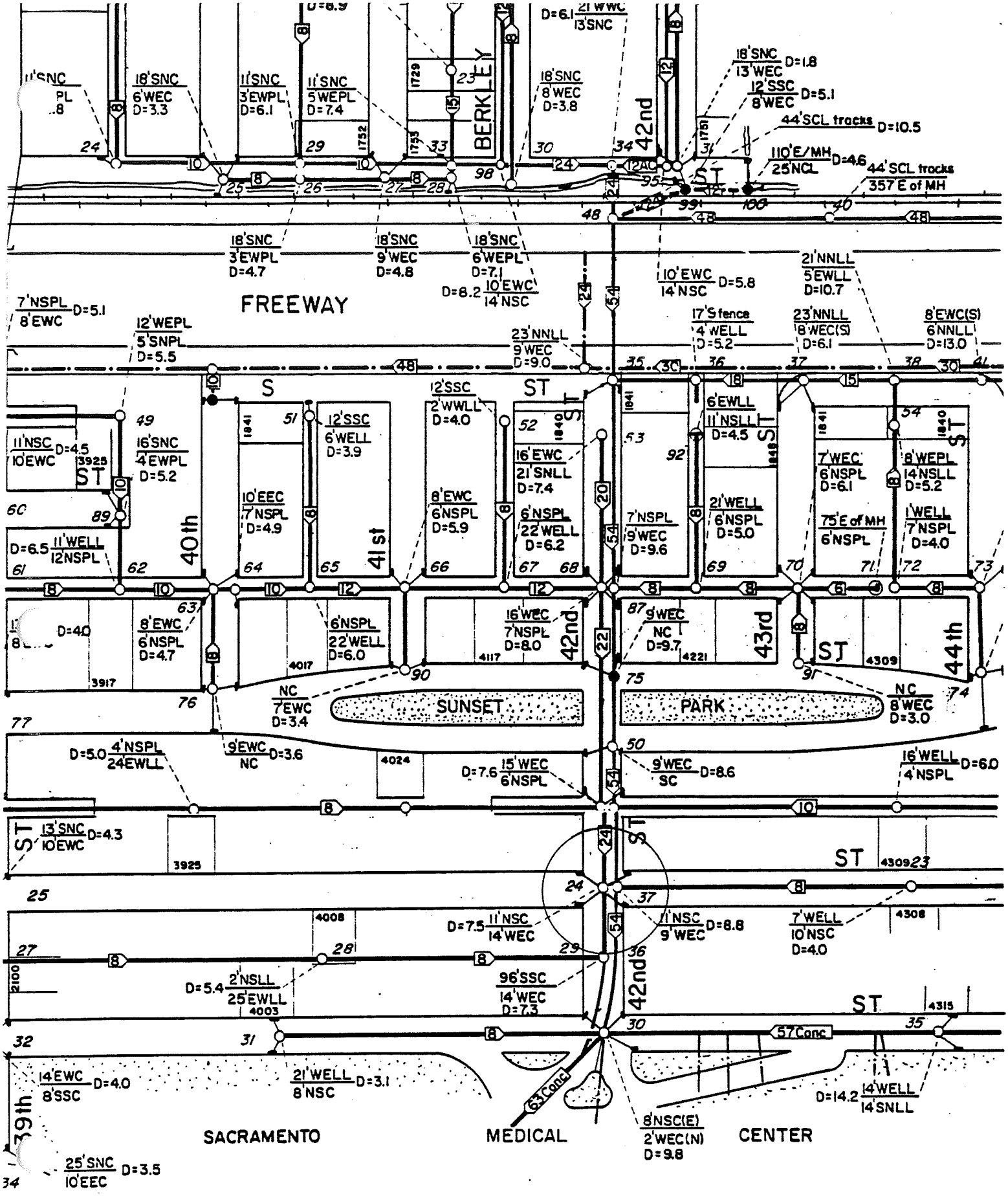
Site FS-1 Manhole 67/14c, 28th & N Streets



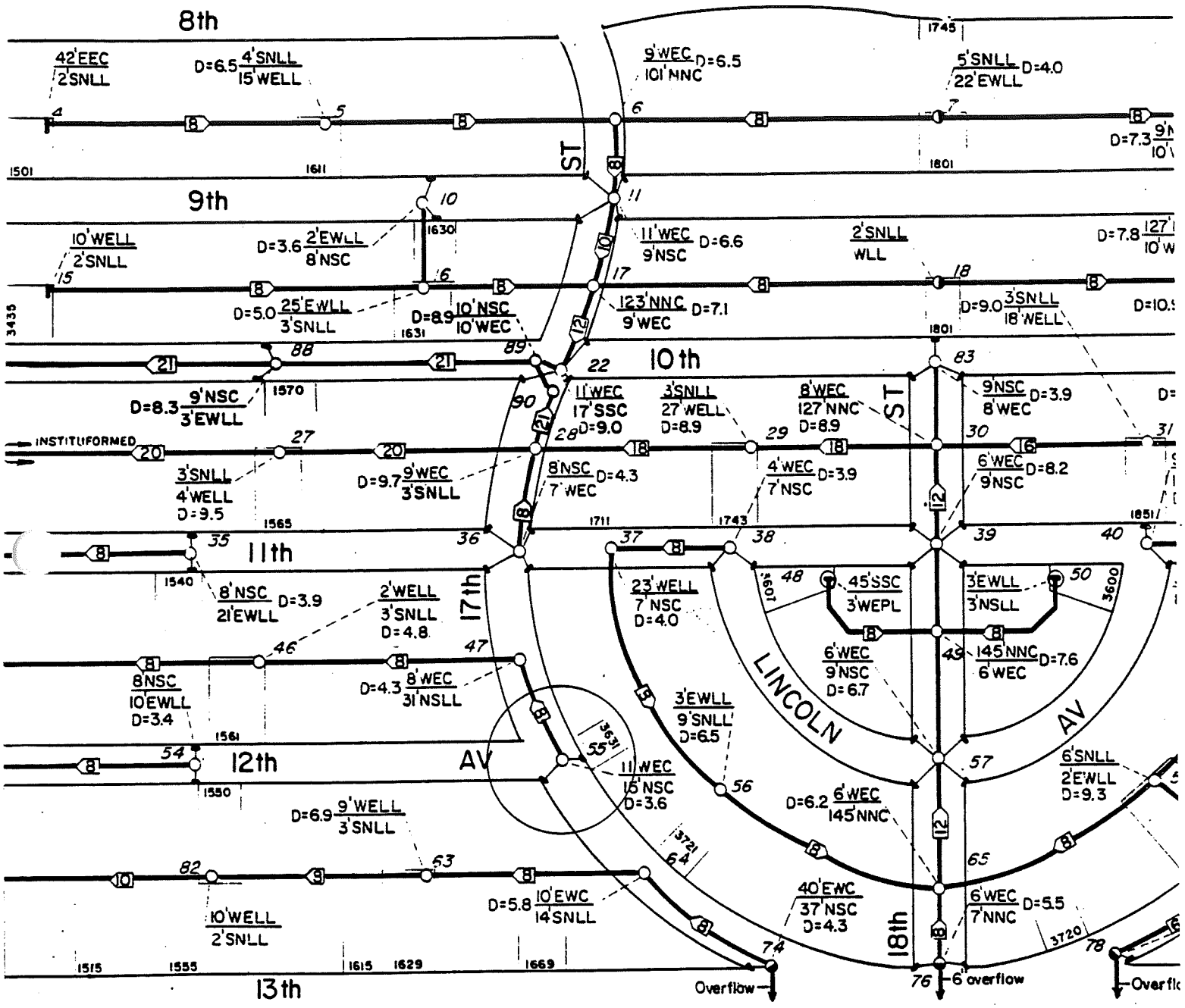
Site FS-2 Manhole 221/12b, 9th & J Streets



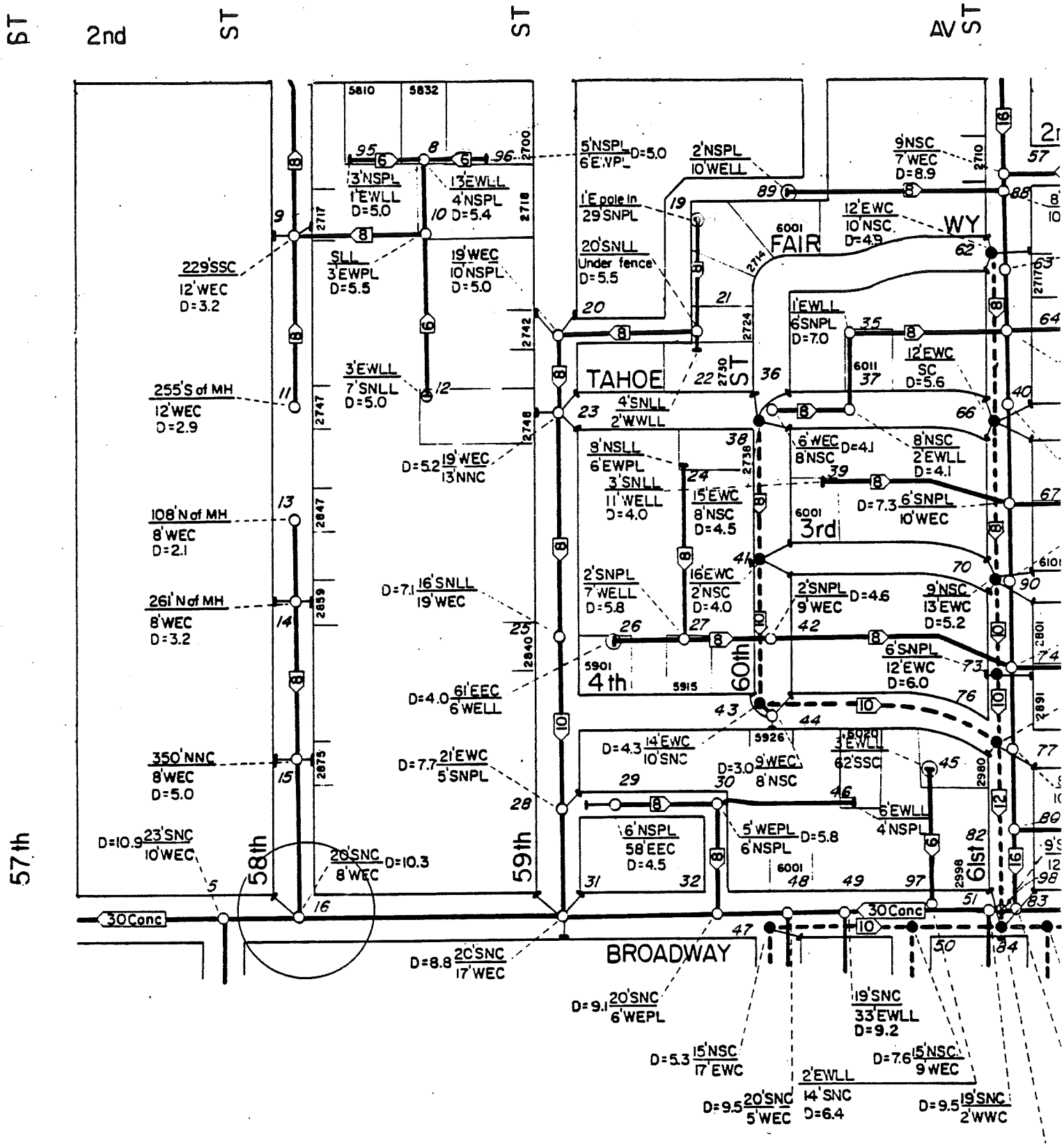
Site FS-3 Manhole 34/13d, 11th & L Streets



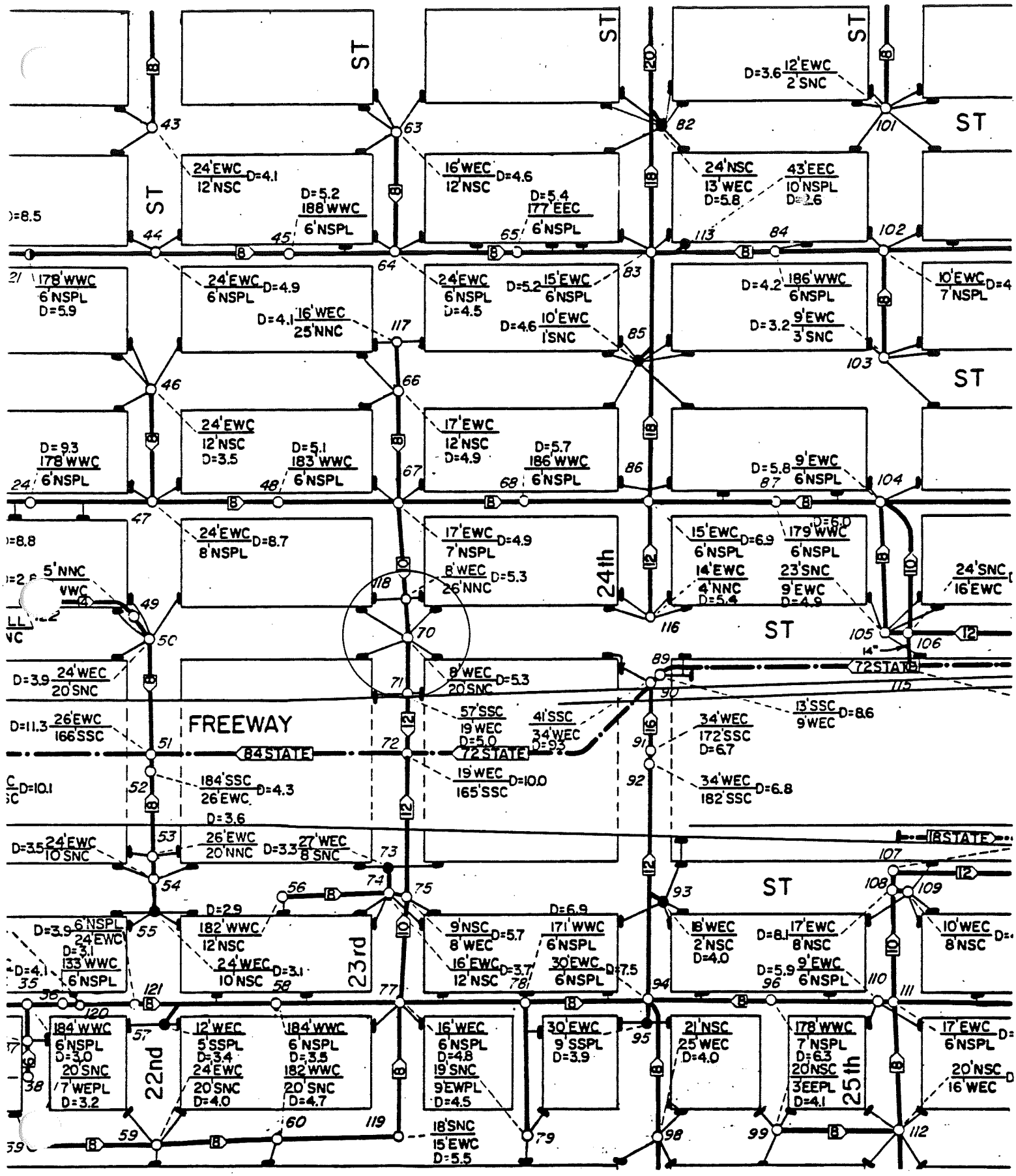
Site FS-4 Manhole 37/21c, 42nd & U Streets



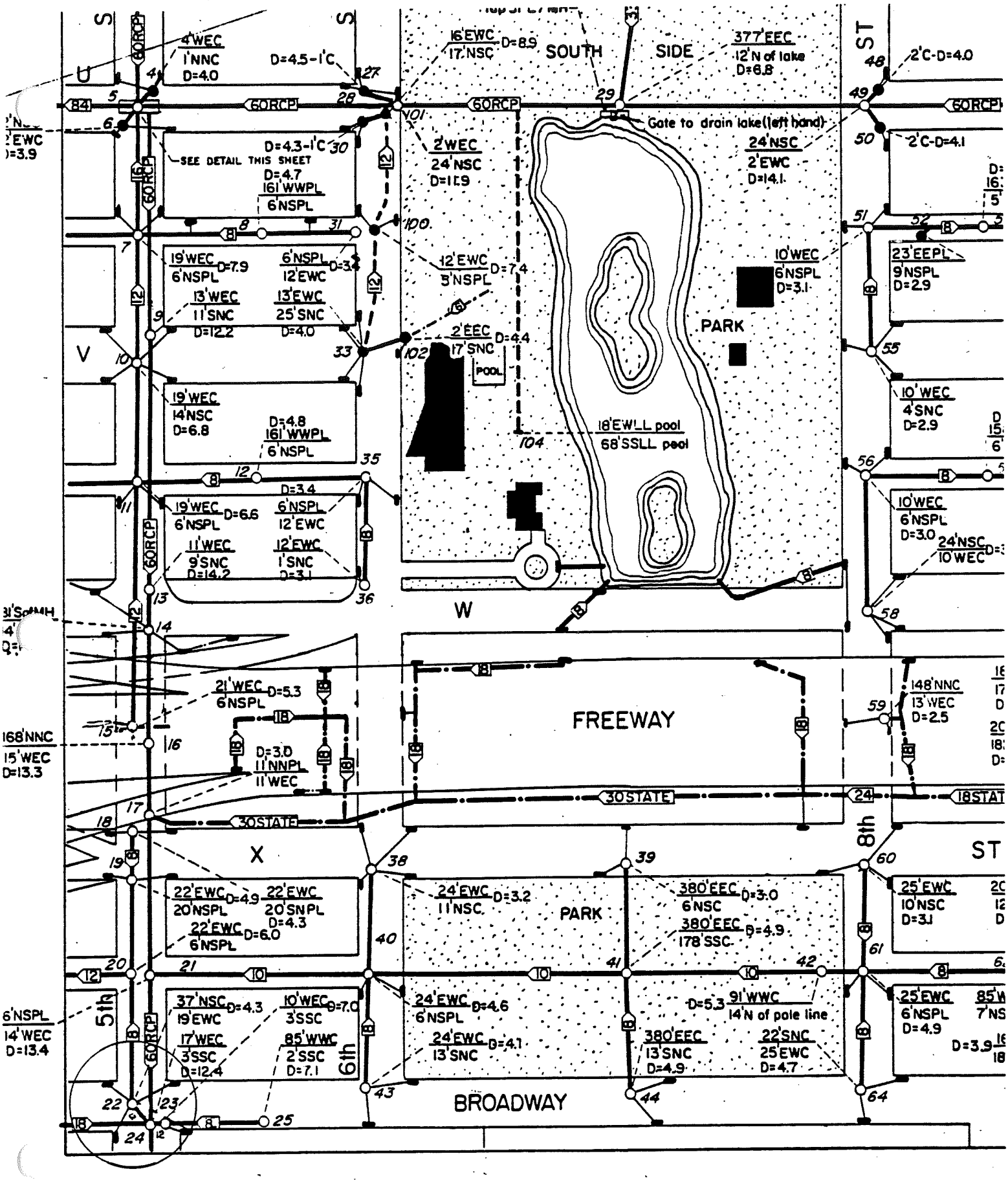
Site FS-5 Manhole 55/33d, 17th Street & 12th Avenue



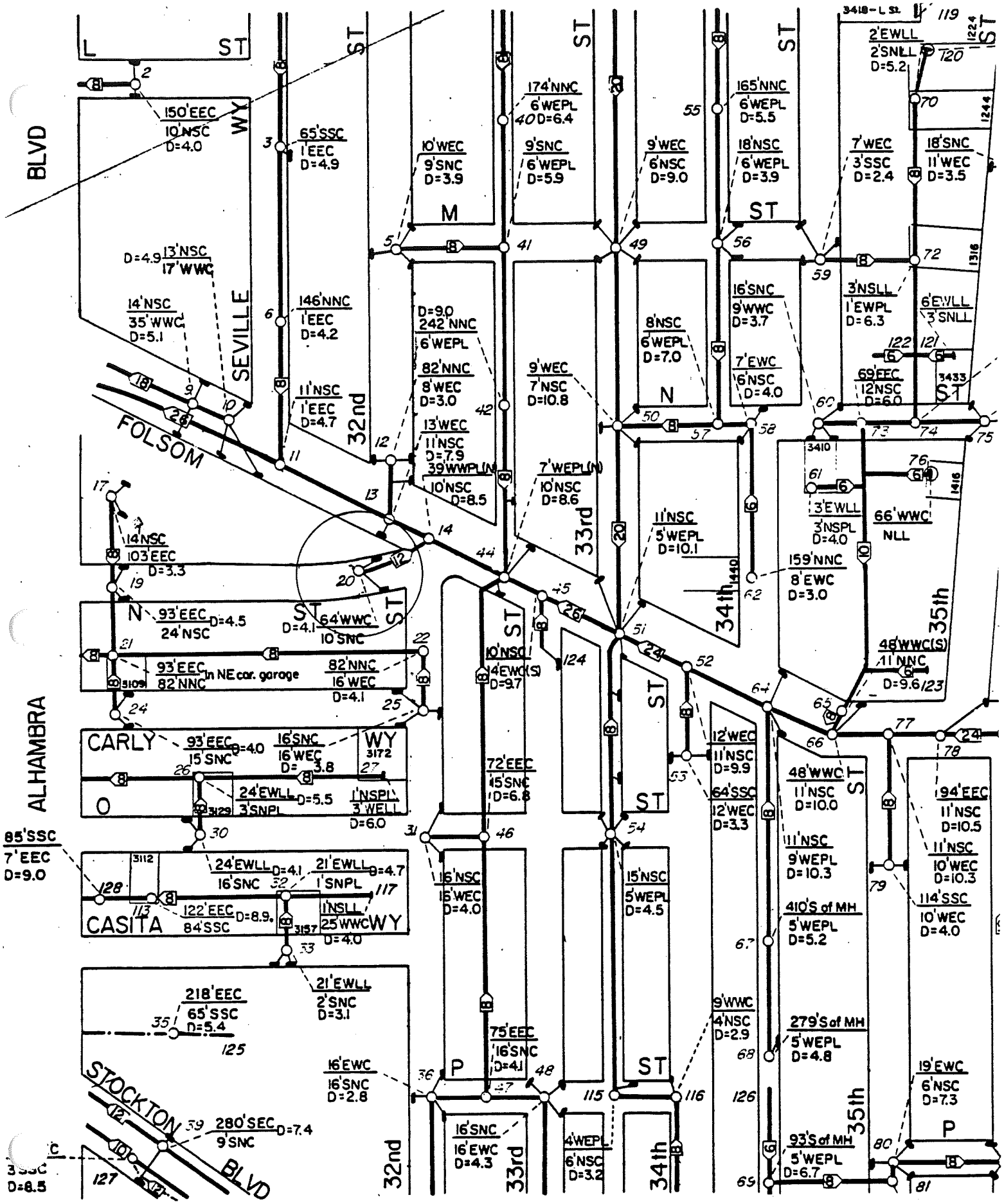
Site FS-6 Manhole 16/19e, 58th Street & Broadway



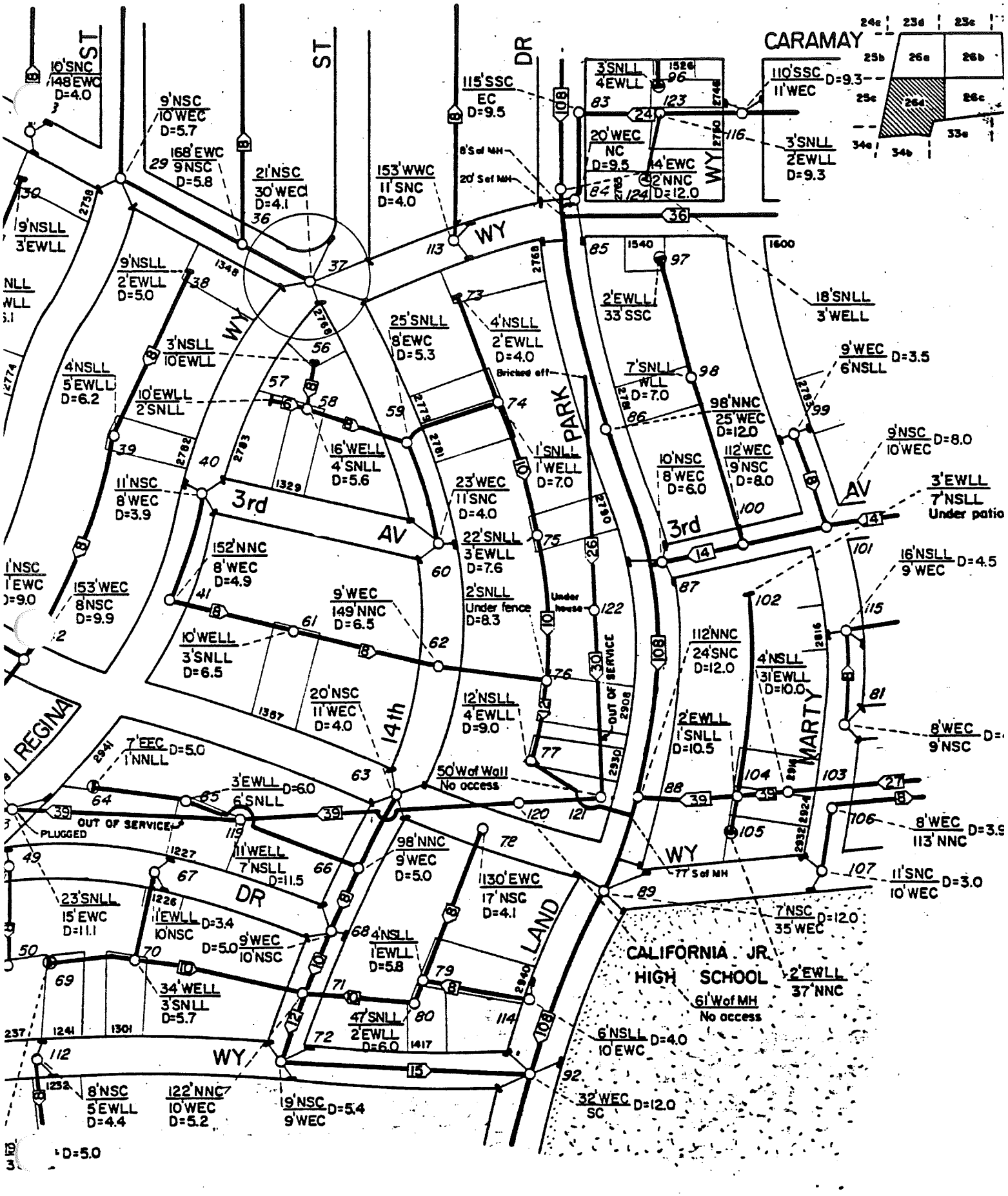
Site FS-7 Manhole 70/22d, 23rd & W Streets



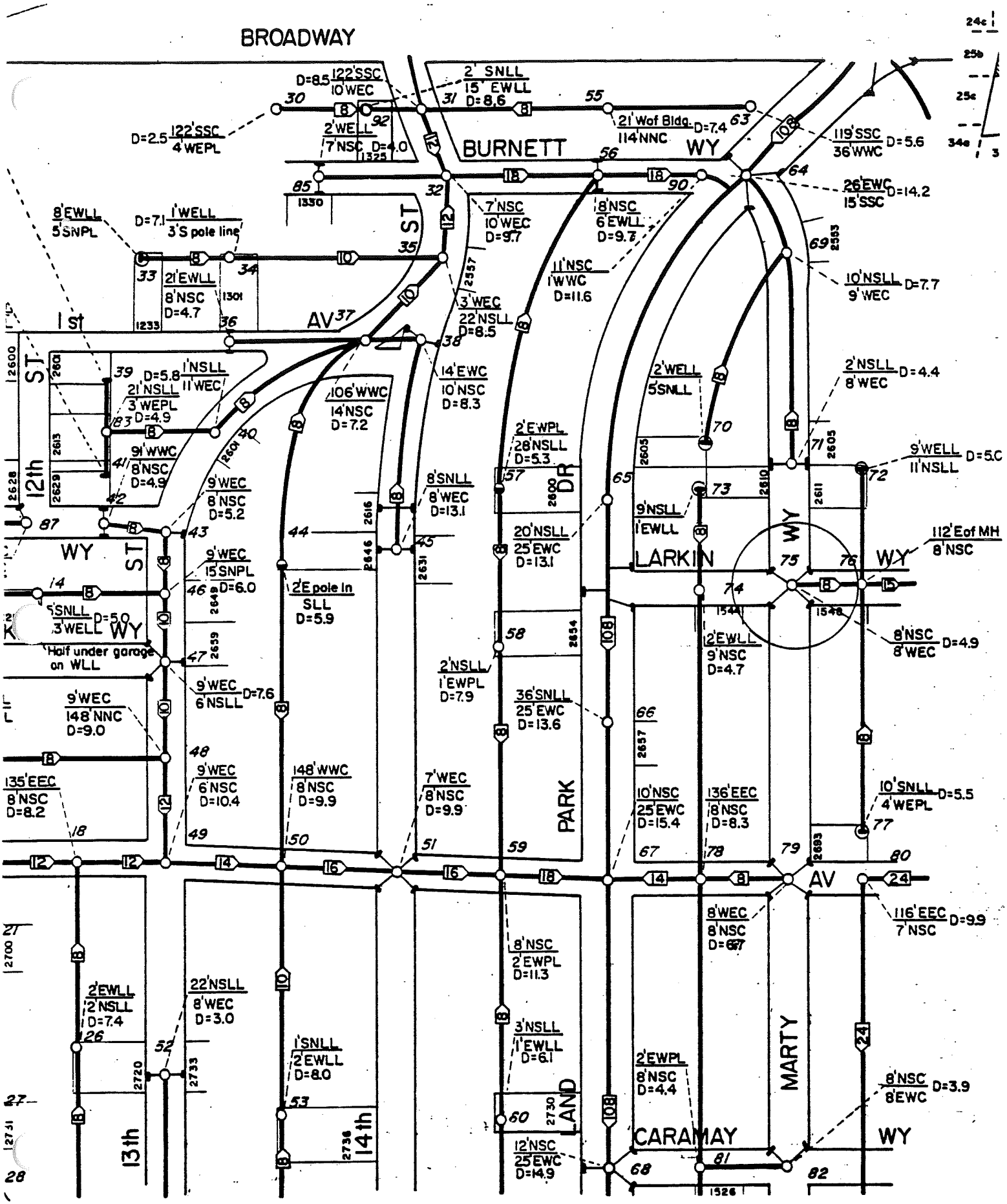
Site FS-8 Manhole 24/24c, 5th Street & Broadway



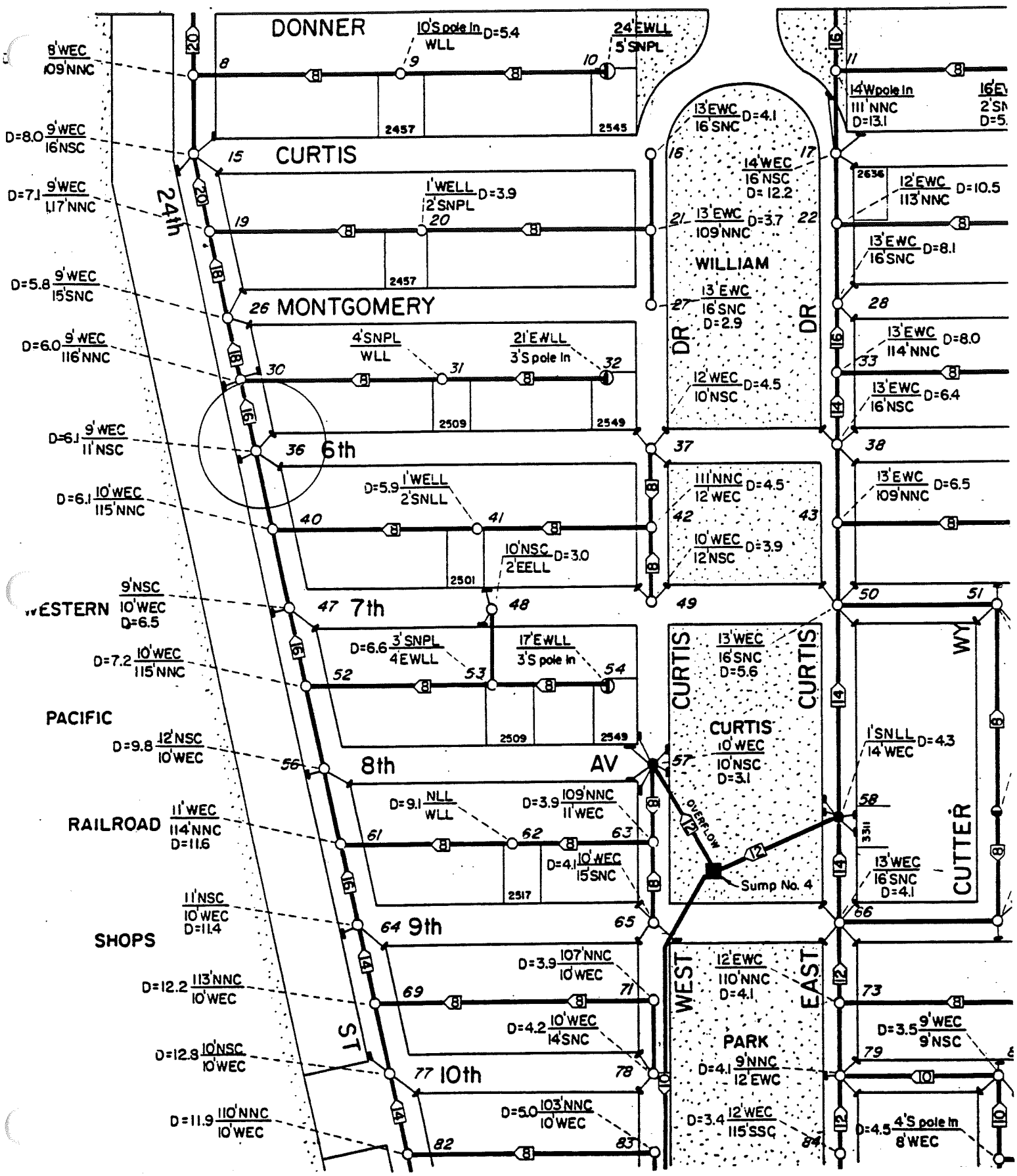
Site FS-9 Manhole 20/15d, 32nd & N Streets



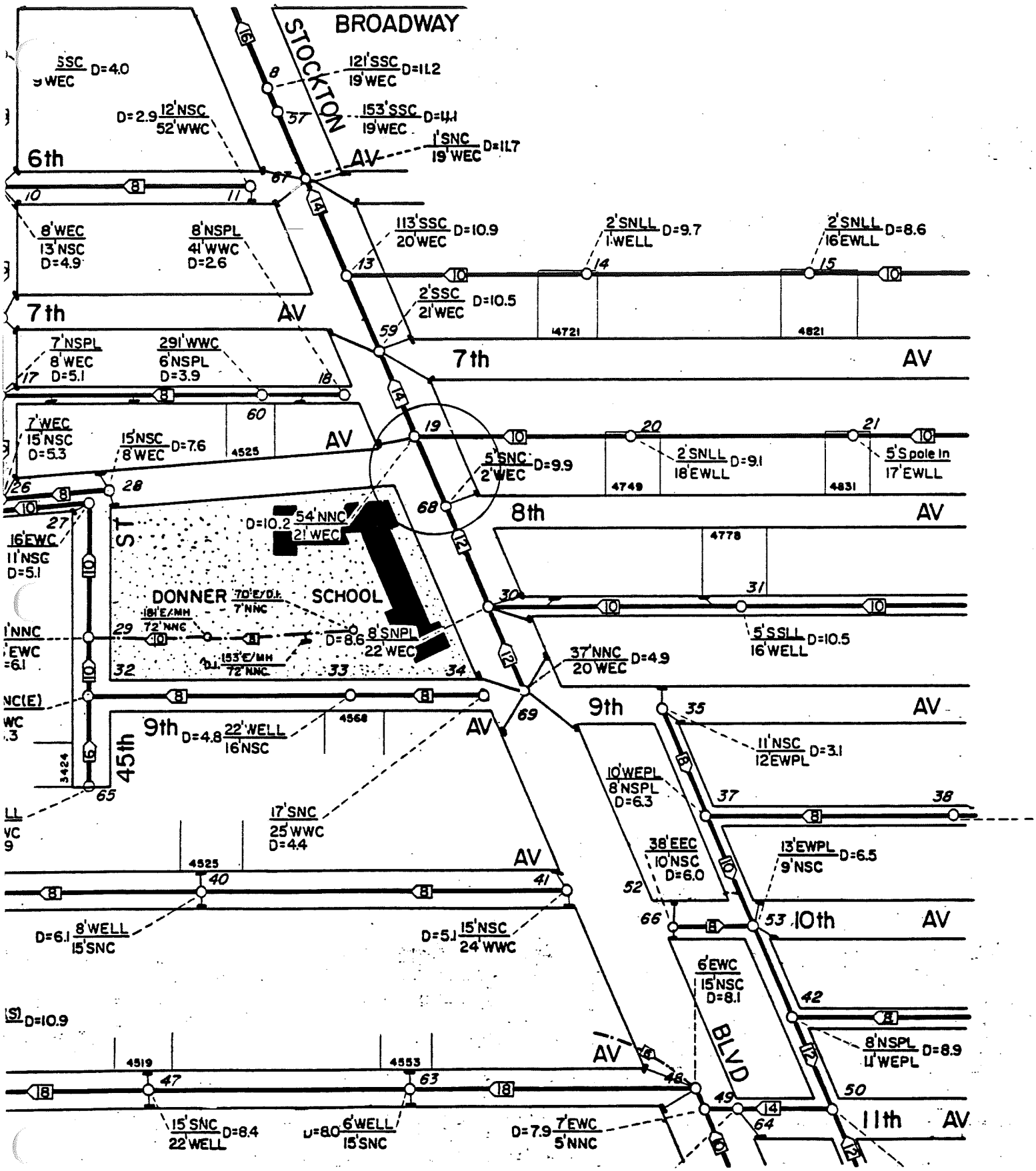
Site FS-10 Manhole 37/26d, 14th Street & 2nd Avenue



Site FS-11 Manhole 75/ 26a, Marty Way & Larkin Way



Site FS-12 Manhole 36/32a, 24th Street & 6th Avenue



Site FS-13 Manhole 68/31b, Stockton Blvd. & 8th Avenue

APPENDIX F

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 2

FLOW MONITORING EQUIPMENT EVALUATION

Monte D. Scharbow
HDR Engineering, Inc.

August 13, 1992

TECHNICAL MEMORANDUM

CITY OF SACRAMENTO COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

PHASE 3, TASK 2

PREPARED BY: Monte Scharbow, HDR Engineering, Inc.

DATE: November 18, 1991

SUBJECT: Flow Monitoring Equipment Evaluation

A. INTRODUCTION

During the 1991/1992 wet season, the City of Sacramento will be collecting flow data for the combined sewer system. Temporary and permanent flow monitoring instruments will be installed throughout the collection system. The schedule for collecting flow data with the temporary instruments is dependent on the City's ability to calibrate and install the instruments. Uses for the flow data and objectives of the City's flow monitoring and data collection program include the following:

- Sacramento SWMM calibration, and current and future refinements in hydrologic and hydraulic analysis of the combined sewer system.
- Potential future modification of combined sewer system operation and control.
- Verification of surcharge and/or flooding conditions.
- Assessment of the present interim NPDES plan of operations for the combined system.

The City presently has obtained, on loan from the County of Sacramento, several portable Montedoro-Whitney flow meters, for use on a temporary basis. The City is intending to install six of these meters in the combined sewer system for flow measurement and data acquisition. The objective is to use wet weather flow data to calibrate the model during the Phase 3 analysis.

Because of a patent infringement lawsuit, the Montedoro-Whitney company is no longer in business. Consequently, replacement parts and repair and calibration services are unavailable. The City is attempting to prepare the meters for service with its own staff and assistance from UC Davis, but it is unknown at this time if this attempt will be successful. The current project schedule dictates that the data collection "cutoff" date for use specifically in the Phase 3 work be in early January.

The permanent flow monitoring installations will be experimental with regard to equipment type, to determine if reasonable and reliable flow measurements can be obtained.

This memorandum presents: the criteria developed for selecting the permanent equipment, a brief review of the equipment considered, selection of the recommended equipment, and a brief description of a typical flow monitoring installation. An evaluation of flow metering equipment for installation in four permanent flow monitoring installations in the City's combined sewer system is presented, followed by recommendations for purchasing equipment.

B. SELECTION CRITERIA

The combined sewer system, where the flow monitoring instruments will be installed, consists of manholes which generally have a number of stormwater drain inlets and sanitary sewer inlets entering each manhole at various heights above the manhole invert. Therefore, a typical manhole has inflows dropping to the manhole invert. This can result in considerable turbulence in the manholes, making flow measurement extremely difficult. In addition, during storm conditions, the manholes may surcharge, potentially to the manhole rim. System outflows can also occur if the hydraulic grade line for the system exceeds the manhole rim elevation. With these conditions in mind, the following criteria were developed for the evaluation and selection of the flow monitoring equipment:

- Measure storm flows from 1/2 pipe flow to complete manhole surcharge.
- Measure water level and velocity in open channel conditions and pipe velocity and manhole level in surcharge conditions.

- Suitable for installation within existing pipes, sizes 15 inches diameter to 60 inches diameter.
- Suitable software data reduction capability, allowing for water depth recording and data analysis from partially filled to surcharged pipe conditions.
- In-manhole equipment materials compatible with sewage liquids and gases.
- In-manhole equipment to occupy minimal space for continued manhole access for maintenance.
- Minimal maintenance requirements.
- In-manhole equipment capable of submergence without damage.
- Minimum one-week data storage.
- Minimum data collection capability of 10 minutes.
- Level (feet), velocity (feet per second), and flow (million gallons per day) local readout capability.
- Level range: 0 to 18 feet.
- Velocity range: 0 to 15 fps.
- Modem telemetry capability.
- Totalizer capability.
- Acceptable minimum flow accuracy.
- Software that is easy to use and has useful outputs.
- Velocity measured by magnetic or acoustic sensors. (Velocities determined by empirical formulas, i.e. Mannings equation, are not acceptable.)
- DC power operation and battery backup when AC power fails.

C. EQUIPMENT REVIEW

The selection criteria for measuring flow in a pipe under open channel as well as surcharge conditions greatly limits the number of vendors in the equipment selection process. There are numerous flow measurement instruments which are suitable for measuring open channel flow in pipes, and numerous instruments which are suitable for measuring flow in a pressurized pipe, but most of these cannot measure both types of flows. An extensive review of manufacturers' literature and HDR's experience revealed only four manufacturers with equipment meeting this criteria. The four manufacturers are:

- Marsh-McBirney. Inc.
- Badger Meter, Inc.
- Accusonic
- ADS Services, Inc.

The Marsh-McBirney flow meter uses electromagnetic velocity and differential pressure depth sensors. The other three use ultrasonic velocity and ultrasonic depth sensors. Other depth sensor systems, such as bubbler pressure, may be available.

Of these four alternative flow meters, ADS Services was found to be the only vendor providing full service, including equipment supply, installation, maintenance and data retrieval. A full service program was determined by City staff to be too costly, therefore, ADS was not considered further.

The Marsh-McBirney Model 266 flow meter system includes a velocity and level transducer which is mounted at the invert of the pipe. A meter panel contains the controls, data logging and telemetry electronics of the system. Additional components include flow measurement software and personal computers for data transfer.

The Badger Meter Series 5000 flow meter includes a pair of transit-time ultrasonic velocity sensors (mounted on the pipe wall about a quarter of a pipe diameter up from the pipe invert), an acoustic or bubbler level sensor for mounting in the manhole, and a meter panel similar to the Marsh-McBirney flow meter described above. Likewise, software and personal computers are provided for data reduction.

The Accusonic Model 7410 flow meter system includes transit-time ultrasonic velocity sensors mounted on the pipe wall, and a level sensor. This system uses acoustic or bubbler level sensors mounted in the manhole for measuring depth. The electronics, software and personal computer equipment are similar to those for the other systems.

D. EQUIPMENT EVALUATION AND SELECTION

The four alternative flow meters were evaluated using the selection criteria defined above, and other relevant factors which could affect their applicability. Table 1 shows a comparison of the alternative flow meters with respect to the selection criteria.

TABLE 1. FLOW METER COMPARISON

Criteria	Manufacturer		
	Marsh-McBirney	Badger Meter	Accusonic
Level, velocity & flow: 1/2 pipe thru surcharge	Yes	Yes	Yes
Pipe installation	8" to 138"	12" to 120"	Greater than 30"
Software/data management	Yes	Yes	Yes
Suitable materials	Yes	Yes	Yes
Maintenance	Moderate to High	Low	Low
Survive surcharge	Yes	Yes	Yes
Local flow display	Yes	Yes	Yes
Has necessary Level/velocity ranges	Yes	Yes	Yes
Flow accuracy, +/- (advertised)	3 percent	4 percent	3 percent
Modem telemetry capability	Yes	Yes	Yes

As indicated by Table 1, the Marsh-McBirney and Badger flow meters satisfy all of the primary criteria. The Accusonic flow meter is not available for pipes less than about 30 inches in diameter, but was considered for possible use in larger pipes.

The Marsh-McBirney, Badger Meter, and Accusonic flow meters are evaluated in more detail in the following paragraphs, based on the above criteria and equipment cost.

D.1. Marsh-McBirney Evaluation

The Marsh-McBirney 266 flow meter system satisfies all of the criteria. The slight disadvantage apparent from Table 1 involves maintenance. The velocity and depth sensors are designed for mounting on the invert of the pipe. This location is susceptible to silt and other material accumulation which can interfere with the electromagnetic signal and cause erroneous readings. Grease and oils can also build up on the sensor and cause erroneous readings. Therefore, periodic cleaning of the sensors is required to ensure accurate measurements. It may be possible to rotate the sensor to approximately a 4 or 8 o'clock position so that it is not quite as susceptible to material buildup, and therefore less maintenance intensive. A software modification would be required to accomplish this, and it is unknown how much loss of accuracy would result.

Potential interference for this type of flow meter includes substances or materials in the flow regime which may affect the conductivity of the liquid, causing unreliable measurements or fluctuations in velocity readings. Turbulence can also adversely affect the velocity and level measurements.

Discussions with experienced HDR personnel and other municipal users of this instrument revealed concerns about the accuracy of the velocity measurements, and about the significant effort necessary to field calibrate the instruments. In addition, considerable maintenance (up to two cleanings per day) is required to sustain reliability. The users reported that a reasonable expectation of flow accuracy is plus or minus 20 to 30 percent.

Most of the reported experience was strictly with sanitary sewer flow measurement, not combined sewer wet-weather flow measurement. The dry weather, low-flow conditions in a sewer would be more likely to result in measurement interferences. The transducer is located at the pipe invert where it is more susceptible to material accumulations and flow measurement inaccuracies when velocities are low. At higher velocities measured during wet weather conditions in combined sewers, accumulations on the transducers are less likely. Therefore, it is expected that the extremes of the reported flow inaccuracies would not be experienced under wet weather, high flow conditions.

The estimated cost of the Marsh-McBirney 266 flow meter system is $8,000, not including personal computers for data transfer or start-up supervision. The cost of a suitable personal computer is about $3,000. One day of start-up supervision would cost about $1,500.

D.2. Badger Meter Evaluation

The Badger Meter 5000 instrument satisfies all of the criteria and may have a slight advantage with respect to maintenance. As discussed above, velocity sensors mounted on the pipe invert are susceptible to accumulation of material which may interfere with data transmission, and cause erroneous readings. However, the Badger Meter sensors are mounted approximately a quarter of the pipe diameter up from the pipe invert, which places them above the main area of material collection in the pipe. In addition, the Badger Meter's ultrasonic sensors are less susceptible to grease and oil interference than a magnetic type sensor. Therefore, it is expected that maintenance would be less than that for the Marsh-McBirney instrument.

The Badger Meter standard acoustic depth sensor would not be suitable for this application because inlet flows at differing elevations into the manhole could interfere with the sensor signal. However, a bubbler pressure or acoustic-uplooking level detector system could be used for depth measurements.

A potential interference with transit-time acoustic flow meters such as this is entrained air. Air bubbles in the flow stream can adversely affect the acoustic signal, resulting in erroneous readings. Because of the turbulence caused by multiple inlets at varying heights in manholes, entrained air may be a problem. However, with the sensors installed in the inlet to a manhole and the pipe lengths between manholes expected to be fairly long, remaining air entrainment at the point of measurement should be minimal.

Discussions with users of this equipment indicate that it is fairly maintenance free, and that the flow accuracy is within 2 to 3 percent of the advertised accuracy. One concern with the instrument is that flow below the level of the sensors is determined by depth and Mannings equation. This results in

discontinuities during the transition to flow values which are determined by velocity measurements as the sensors become submerged. Because the purpose of this equipment is to measure storm flows when pipes in the system are normally more than half full, this transition concern is insignificant.

The equipment cost for the Badger Meter 5000 is estimated to be $14,000 to $16,000, including one day of start-up supervision, but not including personal computers for data reduction.

D.3. Accusonic Evaluation

The Accusonic Model 7410 flow meter is the most accurate of the three instruments evaluated. The reason for the greater accuracy is the multiple transducer/flow path configuration for velocity measurement. Integration of velocities measured at two or three levels in the pipe gives a higher accuracy of flow measurement than the other instruments which base the velocity on only one flow path. It is this multiple transducer configuration that limits the size pipe for which this instrument is suitable. For pipes smaller than about 30 inches in diameter, accurate transducer placement is difficult and makes installation prohibitive.

As with the Badger Meter instrument, arrangement of the transducers at points in the pipe other than the invert results in much less material accumulation and, therefore, less interference from that source. Maintenance of these transducers would be less than the Marsh-McBirney instrument. Like the Badger Meter instrument, the Accusonic instrument is a transit-time acoustic meter. Therefore, the same potential interferences due to entrained air would exist. A bubbler or acoustic level detector system would be used for depth measurement.

The equipment cost for the Accusonic 7410 meter is estimated to be $28,000, not including installation and start-up supervision or personal computers for data reduction. Installation and start-up supervision is estimated at $2,000 to $4,000 per site.

E. RECOMMENDATION

Based on the above evaluation, all three instruments appear to be suited for this application. However, the Accusonic device is limited to pipes over 30 inches in diameter. The Badger Meter is nearly twice the cost of the Marsh-McBirney Meter per instrument, but is much more accurate and requires less maintenance. The Accusonic instrument is the most accurate but is at least three times the cost of the Marsh-McBirney meter. The higher accuracy of the Accusonic meter is not necessary for this application. Therefore, the higher cost does not justify use of the Accusonic meter.

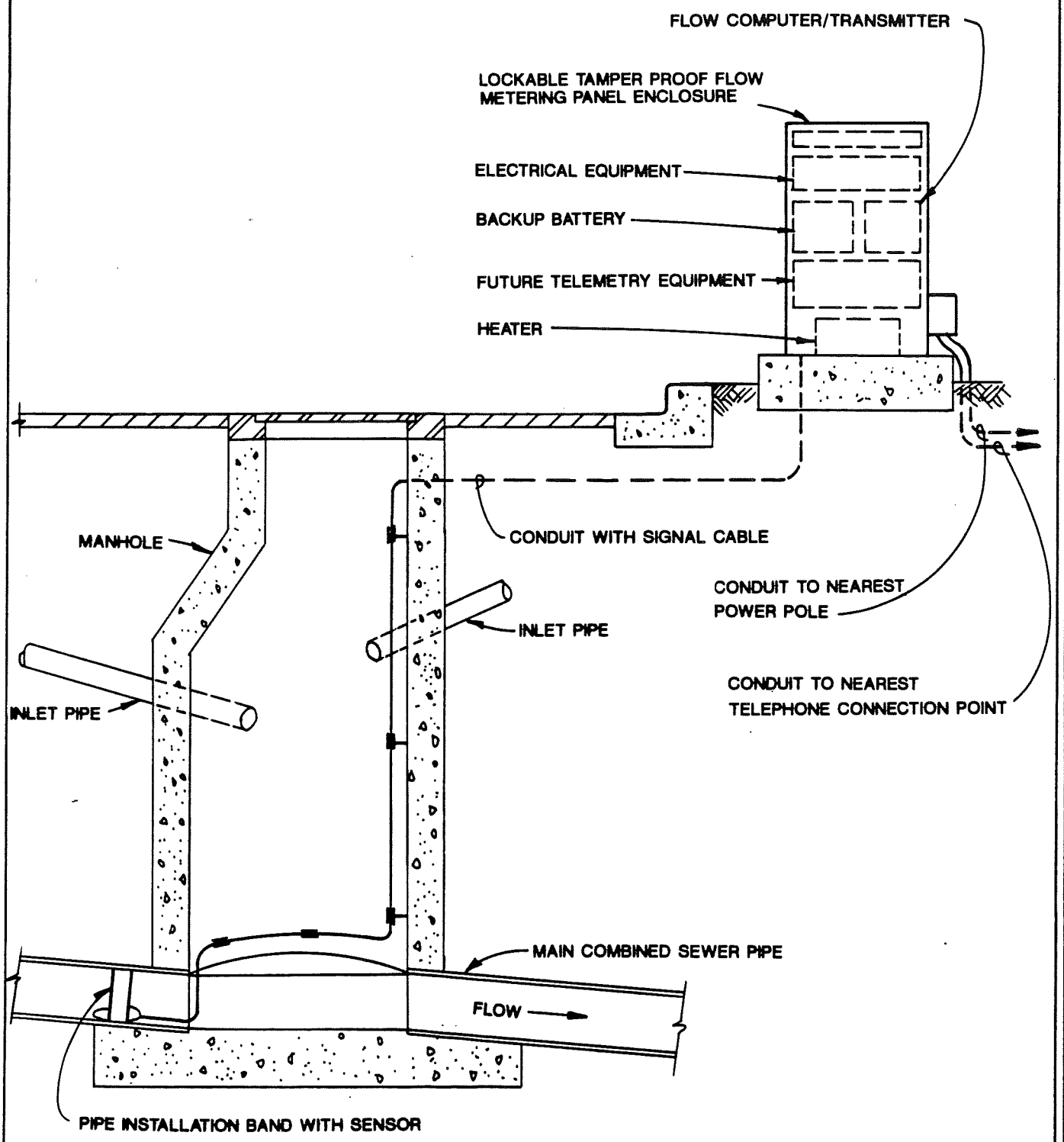
Because these flow monitoring stations are intended as a trial for obtaining storm flow measurements, it is recommended that two stations using the Marsh-McBirney 266 and two stations using the Badger Meter 5000 be installed at this time. The two instruments can then be properly compared and evaluated based on their performance in the City of Sacramento Combined Sewer System. After a period of operation, the more cost-effective flow measurement system can be selected for an expanded flow monitoring program.

In addition, if the City is unable to get the temporary Montedoro-Whitney flow meters ready for operation, four portable flow meters should be obtained to perform the flow measurements intended for those instruments. The Marsh-McBirney Flo-Tote appears to be the only suitable portable unit available. The cost for these units is about $7,000 each, not including software or computers.

F. TYPICAL FLOW MONITORING INSTALLATION

A typical permanent flow monitoring installation would include the sensors, which are located in a manhole and pipe inlet, and a flow metering panel mounted on a concrete pad near the manhole, but off the street. The panel should be lockable and tamper-proof, and would contain the flow meter electronics, including transmitter and data storage, telemetry equipment (future), a space heater, a light fixture, a backup battery power supply and other necessary electrical equipment. A typical installation using the Marsh-McBirney instrument is illustrated on Figure 1. A typical installation for the Badger Meter instrument

would be similar, however, additional equipment for level measurement would be required. The estimated construction cost for a typical installation is $70,000 to $90,000, depending on the instrumentation used and distances from the manhole to a power source and a telephone connection point.



TYPICAL FLOW MONITORING INSTALLATION FOR MARSH-MCBIRNEY INSTRUMENT

HDR

Figure 1

APPENDIX G

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 2

FLOW MONITORING LOCATION RECOMMENDATIONS

Judy A. Zavadil
Paula M. Arsenault
HDR Engineering, Inc.

August 13, 1992

TECHNICAL MEMORANDUM - DRAFT
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN
PHASE 3, TASK 2

PREPARED BY: Judy Zavadil, HDR Engineering, Inc.
Paula M. Arsenault, HDR Engineering, Inc.
DATE: November 19, 1991
SUBJECT: Flow Monitoring Location Recommendations

A. INTRODUCTION

During the 1991/1992 wet season, the City will be collecting flow and water level data at selected combined sewer manhole locations. There are two key purposes for collecting combined system flow data: 1) calibration of the Sacramento SWMM; and 2) assistance in assessing the effectiveness of the City's *Plan of Operations*. Task 2 of the Phase 3 work effort involves providing overall assistance in: selecting velocity/depth monitoring instruments and other equipment, preparing bid documents (for permanently installed meters), installing the equipment, and analyzing the data collected. A technical memorandum was prepared, entitled *Flow Monitoring Equipment Evaluation* (draft version dated September 30, 1991), which evaluates a variety of instruments and related equipment currently available from vendors, and recommends particular instruments for temporary (portable) and permanent installations.

The purpose of this memorandum is to present recommended locations for both the temporary and permanent installations. These monitoring locations are intended to meet the model calibration objective of this task. A maximum of three storm events will be selected for use in model calibration. The actual number of events selected will depend on weather conditions over the next few months.

Prior to final selection of suitable monitoring locations, City staff review and field inspection will be necessary. Based on the review and field inspection, it is likely that some of the potential installation locations will be eliminated from further consideration. For this reason, over 20 prioritized installation locations are recommended for review and field inspection before the final

locations are selected, even though the Phase 3 Task 2 scope of work calls for the identification of only 10 to 12 potential sites. HDR staff will accompany City personnel in field inspection of up to 12 of the 22 locations.

Four to six temporary locations will ultimately be selected for installation of portable instruments. During the mid- to latter part of the wet season, four locations will be selected for installing permanent monitoring instruments. These permanent sites may be identical to or different from the temporary locations, depending on the results of the field inspection.

B. USE OF MONITORING DATA

As stated above, the primary objective in collecting flow data is for calibrating hydrologic input parameters to the model. Both the portable and permanent instruments will be capable of collecting water depth and velocity data. Under gravity flow conditions (i.e. prior to system surcharging), water depth readings are converted to cross-sectional area (of flow) and multiplied by velocity to determine flow rate. Flow rates can then be compared to flow rates generated either by RUNOFF or EXTRAN block models of the Sacramento SWMM. RUNOFF block results can be used where the basin(s) being monitored are relatively small and system storage effects are minimal. Under gravity flow conditions, EXTRAN conduit hydrographs (flow rate versus time) are used for calibration purposes if routing and storage effects need to be considered.

Under surcharge conditions, flows cannot be accurately calculated using the measured velocity, because the system is operating under pressurized conditions. Wastewater is stored or "backed up" in the system as the sewers overload with stormwater runoff. In this case, measured water depths and total hydrograph volumes of combined sewage passing through a conduit would be used for calibration of SWMM runoff parameters. If flooding takes place during a storm, the data cannot be used to calibrate runoff parameters, and could only be used as a check on the overall accuracy of the model for simulating flooding at a particular location.

Another use of the data is to verify estimates of wet weather sanitary flow rates (including infiltration and inflow or I/I) being discharged to the combined

- Select well-defined (relative to land use) representative basins to determine whether runoff parameters are reasonable.
- Select basins where outflows are suspected to determine whether reverse flows and outflows are occurring.

D. RECOMMENDED MONITORING LOCATIONS

Table 1 lists the recommended potential locations for installing flow monitoring equipment. Each of these sites will need to be reviewed by City staff, and some of them will need to be inspected. A determination will then be made as to whether each site is either suitable for installing a temporary portable instrument or a permanent instrument, or is unsuitable for either type. Based on the review and field screening, the final sites will be selected.

Table 1 provides both the City manhole number and SWMM node number (or closest node for each location), along with a street location description. In addition, the probe location (i.e. which pipe the instrument is to be installed in) is specified with respect to the manhole. Figure 1 provides an illustration of the mounting direction the instrument. Comments are also provided noting the basis for selection or use of the data.

Figure 2 is an overall location map showing all of the recommended potential monitoring sites. Appendix A contains detailed location maps prepared from the City's 1"=200' sewer maps. The ID numbers on Table 1 correspond to the circled numbers on the appended maps. Selected manholes are indicated by a shaded circle surrounding the manhole.

The six sites with numerical rankings have been identified primarily for temporary flow monitoring. These sites are recommended because of the immediate need for obtaining flow data related to model calibration, for well-defined basins with minimal upstream regulation, and for estimating sanitary flow and I/I contributions from separated areas to the east. A "T" identifies locations suitable for temporary portable instrument installation, and a "P" denotes those directed towards permanent long-term monitoring.

system from adjacent separated areas. During the 1988 study conducted by Robert E. Young, storm-induced I/I was estimated to be substantial. Ten percent of the runoff for a given basin was estimated to contribute to the separate sanitary system. This estimate needs to be verified. It is possible that storm-induced infiltration in these areas may not be accurately modelled using a hydrographic method. However, no data have been collected to warrant a change in the current approach.

Lastly, water level data could be used to identify areas of flooding. Flooding would be indicated by the instruments as measurements of water depth as high as, or higher than, the manhole rim elevation.

C. SITE SELECTION CONSIDERATIONS

In order to identify potential flow monitoring locations in the system, criteria were established based on model calibration/verification needs. The key use of the data will be to calibrate RUNOFF block inputs, which describe the hydrologic characteristics for each basin in the combined sewer system. In addition, the data can be used to identify potential problems in the EXTRAN block inputs, such as routing and instability problems. The following list includes both the criteria and considerations used in selecting the potential monitoring locations.

- Select basins where land use has been well-defined.
- Select basins representing different types of areas in the combined system, relative to land use and collection system construction, such as residential density.
- Select locations where City flood maps indicate flooding and the SWMM model does not indicate flooding exists.
- Verify that flooding indicated by SWMM is present.
- Choose locations on major laterals and mains upstream of connection points to major interceptors, to check the operation of EXTRAN and calibrate runoff from major basins in the service area.
- Select locations at the downstream end of a single basin or multiple basins to calibrate separate sanitary basin contributions, including rainfall-induced I/I.

TABLE 1. RECOMMENDED FLOW MONITORING LOCATIONS

ID No.	Manhole No.	Probe Direction	Pipe Diameter (inches)	Site Priority	SWMM Node	Location	Comments
1	20/24b	N	24	6	near 311	Between R and S Sts. on 5th St.	Sanitary flow contribution from west end of the combined system (separated downtown and old town area).
2	100/8c	E	22	T	960	41st and C Sts.	Sanitary flow and I/I contributions from east side of combined area (several basins).
3	171/15c	E	12	2	near 941	37th and P Sts.	Representative collection system basin (portion of 9411); hydrologic parameter calibration through EXTRAN or RUNOFF.
4	51/26a	W	16	1	near 166	14th St. and 2nd Ave.	Representative collection system basin (1661); hydrologic parameter calibration through EXTRAN or RUNOFF.
5	84/26d or 83/26d	N or E	24	T	near 509	Land Park Dr. and Markham Wy.	Dense residential basins in the south-central portion of the combined area (basins 1651, 1641, 5092); calibration of hydrology through EXTRAN or RUNOFF.

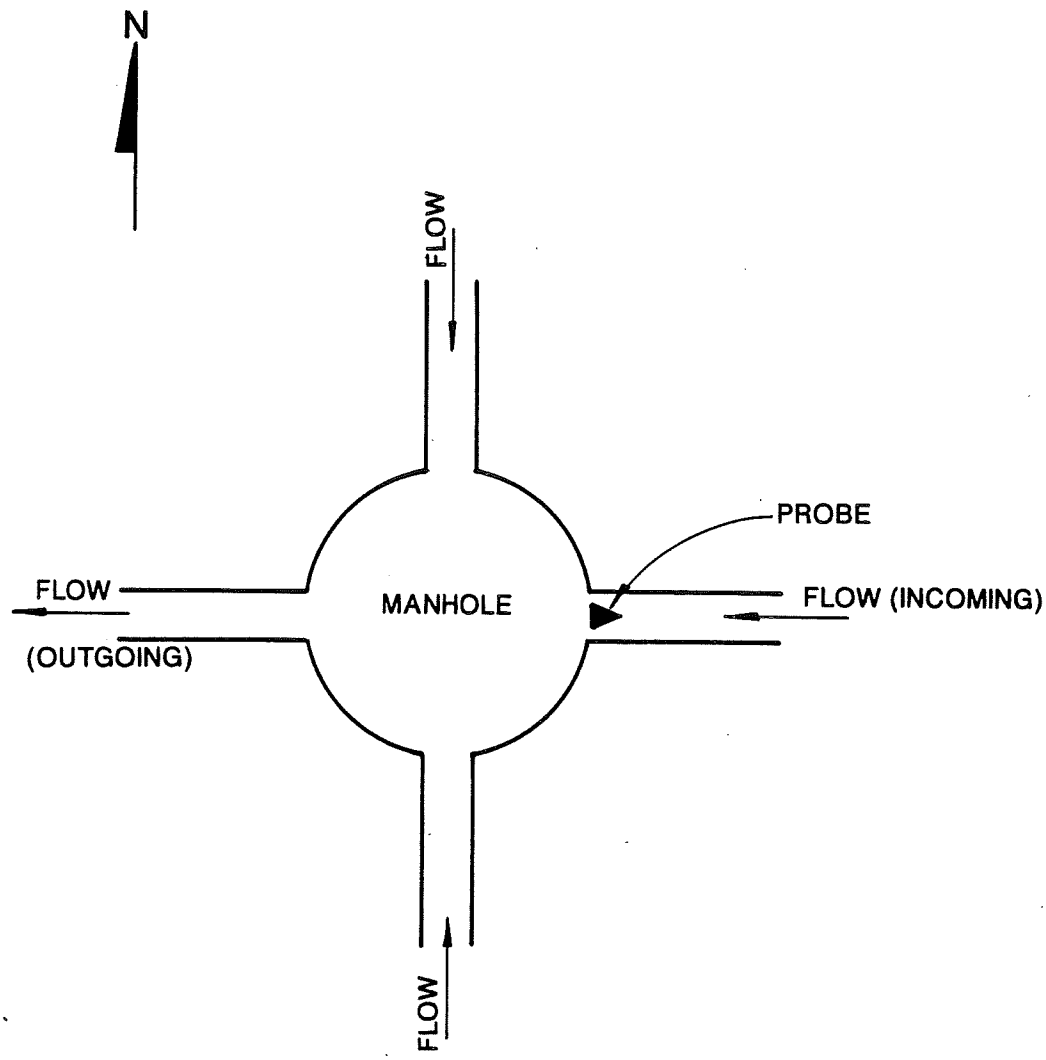
TABLE 1. RECOMMENDED FLOW MONITORING LOCATIONS							
ID No.	Manhole No.	Probe Direction	Pipe Diameter (inches)	Site Priority	SWMM Node	Location	Comments
6	58/32a	S	14	T	near 191	Between 7th and 9th Sts. on E. Curtis	Individual basin (1991) located in the southeast portion of the combined area; runoff parameter calibration; also verify occurrence of flooding.
7	43/32d	S	14	T	near 783	Between 11th and 12th Sts. on Martin Luther King Blvd.	Individual basin (7831) located in the southeast portion of the combined area; runoff parameter calibration.
8	48/21b	N	24	4	762	42nd and R Sts.	42nd St. area monitoring; area has been well-defined relative to land use and hydrologic parameters; known to have flooding problems; City has defined land use and subareas in detail.
9	110/9d	N	14	5	near 568 & 569	Between E and F Sts. on 25th St.	Representative collection system basin (5681); land use well-defined through detailed evaluation of collection system hydraulics.

TABLE 1. RECOMMENDED FLOW MONITORING LOCATIONS

ID No.	Manhole No.	Probe Direction	Pipe Diameter (inches)	Site Priority	SWMM Node	Location	Comments
10	78/26c	N	22	T	162	Between 4th Ave. and Vallejo Dr. on Freeport Blvd.	Overall calibration of a urbanized basins in the south portion of the combined area through EXTRAN; verify routing and surcharge levels.
11	71/14d	N	18	T	599	Between M and N Sts. on 22nd St.	Downtown area; primary purpose is EXTRAN calibration of flooding and surcharging levels; also runoff calibration for downtown land use; collects relatively well-defined set of downtown basins (5961, 5981, 5991).
12	71/13b	N	18	T/P	350	Between I and J Sts. on 17th St.	Downtown area calibration through EXTRAN; verification of flooding and surcharging levels.
13	32/33d	W	16	T/P	179	Between 10th and 11th Aves. on 19th St.	Verification of flooding plus runoff parameter calibration; small basin in the south quadrant of the combined system; City maps indicate flooding but model does not indicate flooding occurs.

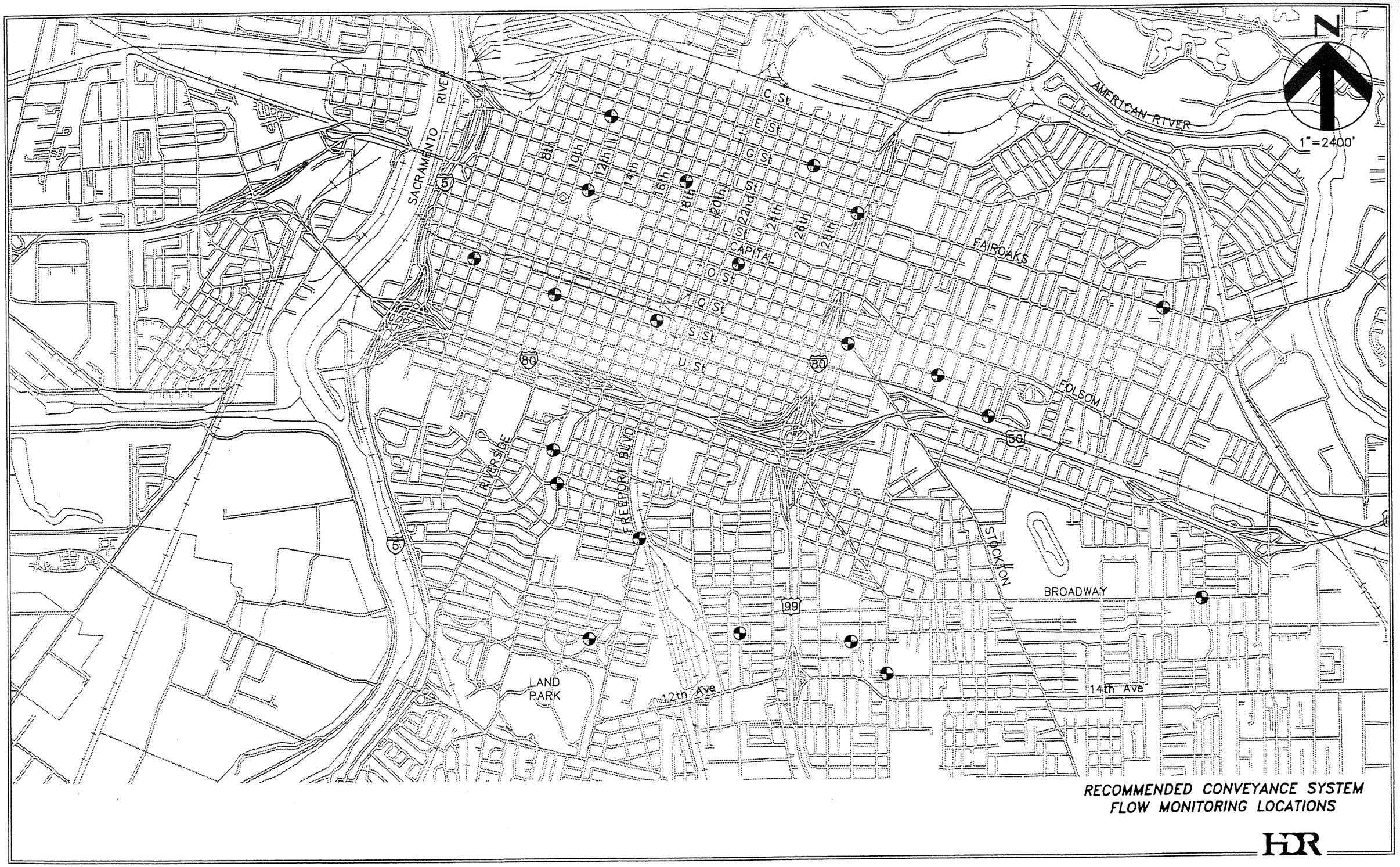
TABLE 1. RECOMMENDED FLOW MONITORING LOCATIONS							
ID No.	Manhole No.	Probe Direction	Pipe Diameter (inches)	Site Priority	SWMM Node	Location	Comments
14	68/14b	N	18	T	588	Between G and H Sts. on 29th St.	Verification of flooding and surcharging levels through EXTRAN; calibration of runoff volumes; node collects relatively well-defined set of basins (5861, 5871, 5881); City maps indicate flooding occurs in vicinity of this node.
15	119/22b	N	66	P	918	Alhambra Blvd. at Q St.	Major interceptor; calibration through the EXTRAN model.
16	40/32b	E	84	P	near 721 & 722	35th St. and 9th Ave.	Major interceptor collects southeast quadrant of the combined system; calibration through the EXTRAN model.
17	81/23b	N	102	P	518	Between R and S Sts. on 18th St.	Major interceptor; calibration through the EXTRAN model.
18	39/23a	E	54	P	near 341	11th and S Sts.	Major interceptor; calibration through the EXTRAN model.

TABLE 1. RECOMMENDED FLOW MONITORING LOCATIONS							
ID No.	Manhole No.	Probe Direction	Pipe Diameter (inches)	Site Priority	SWMM Node	Location	Comments
19	39/7c	E	18	3	near 934	H St. between 52nd and 53rd Sts.	Sanitary and I/I flow contributions from the northeast portion of the combined area (basins 9341, 9342).
20	51/19e	E	30	P	near 797	61st St. and Broadway	Sanitary and I/I flow contributions from the southeast combined area plus the Calif. Youth Author. (basins 7971, 7972, 7973, 7974).
21	37/10d	W	21	T/P	362	11th and G Sts.	Calibration through EXTRAN of the downtown area; check if laterals/mains are surcharged in downtown area near locations where flooding indicated by model.
22	34/13d	W	21	T/P	322	11th and L Sts.	Calibration through EXTRAN of the downtown area; check if laterals/mains are surcharged in downtown area near locations where flooding indicated by model



THIS ILLUSTRATES AN EXAMPLE OF DEPTH/VELOCITY PROBE MOUNTING. THE PROBE HEAD IS MOUNTED "E" (EAST), i.e. POINTED EAST SUCH THAT FLOW IS COMING TOWARDS PROBE IN POSITIVE DIRECTION. THIS INSTRUMENT WOULD MEASURE TOTAL FLOW FOR UPSTREAM BASIN(S).

Figure 1. INSTRUMENT INSTALLATION SCHEMATIC EXAMPLE



RECOMMENDED CONVEYANCE SYSTEM
FLOW MONITORING LOCATIONS

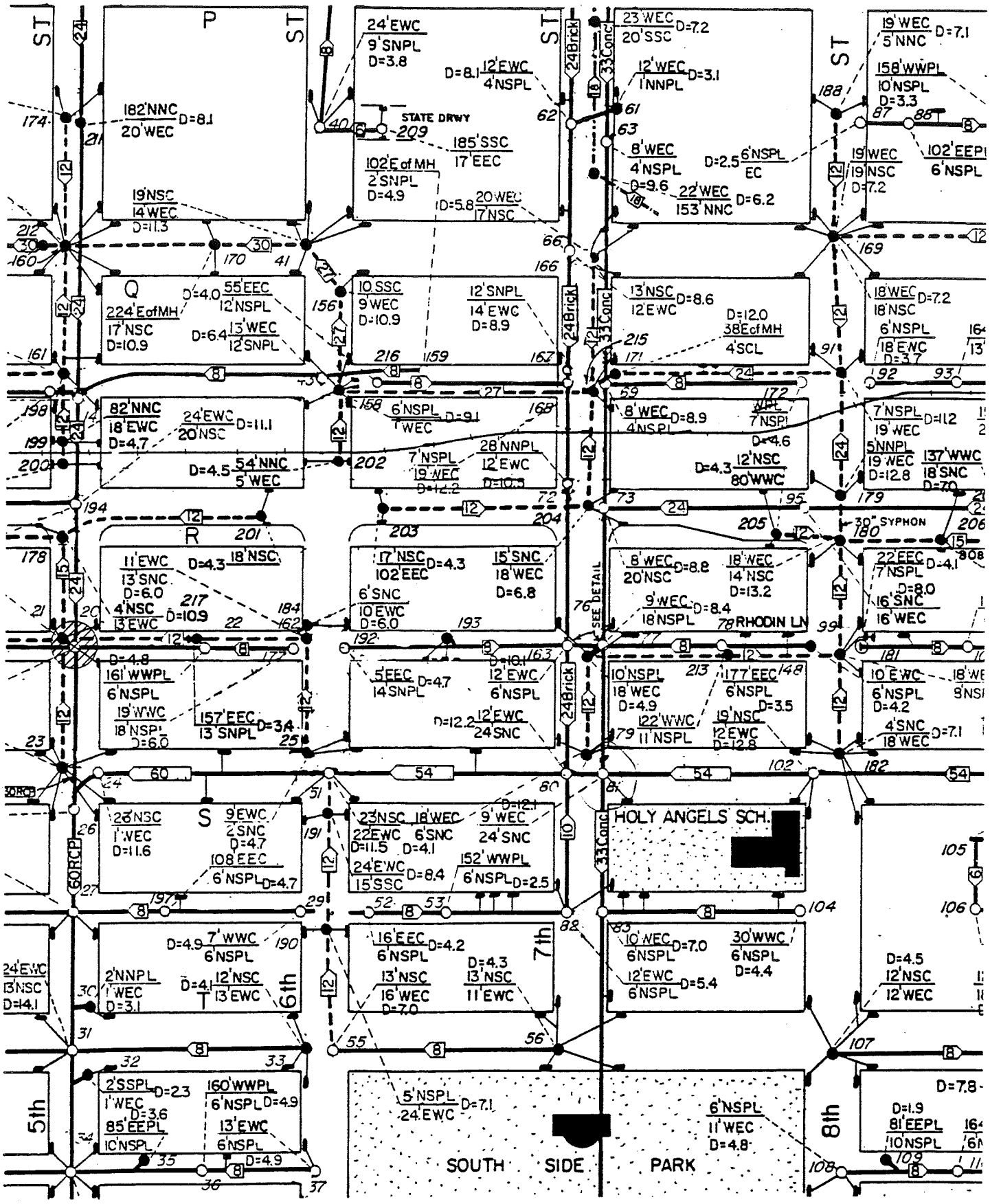
HDR

Figure 2

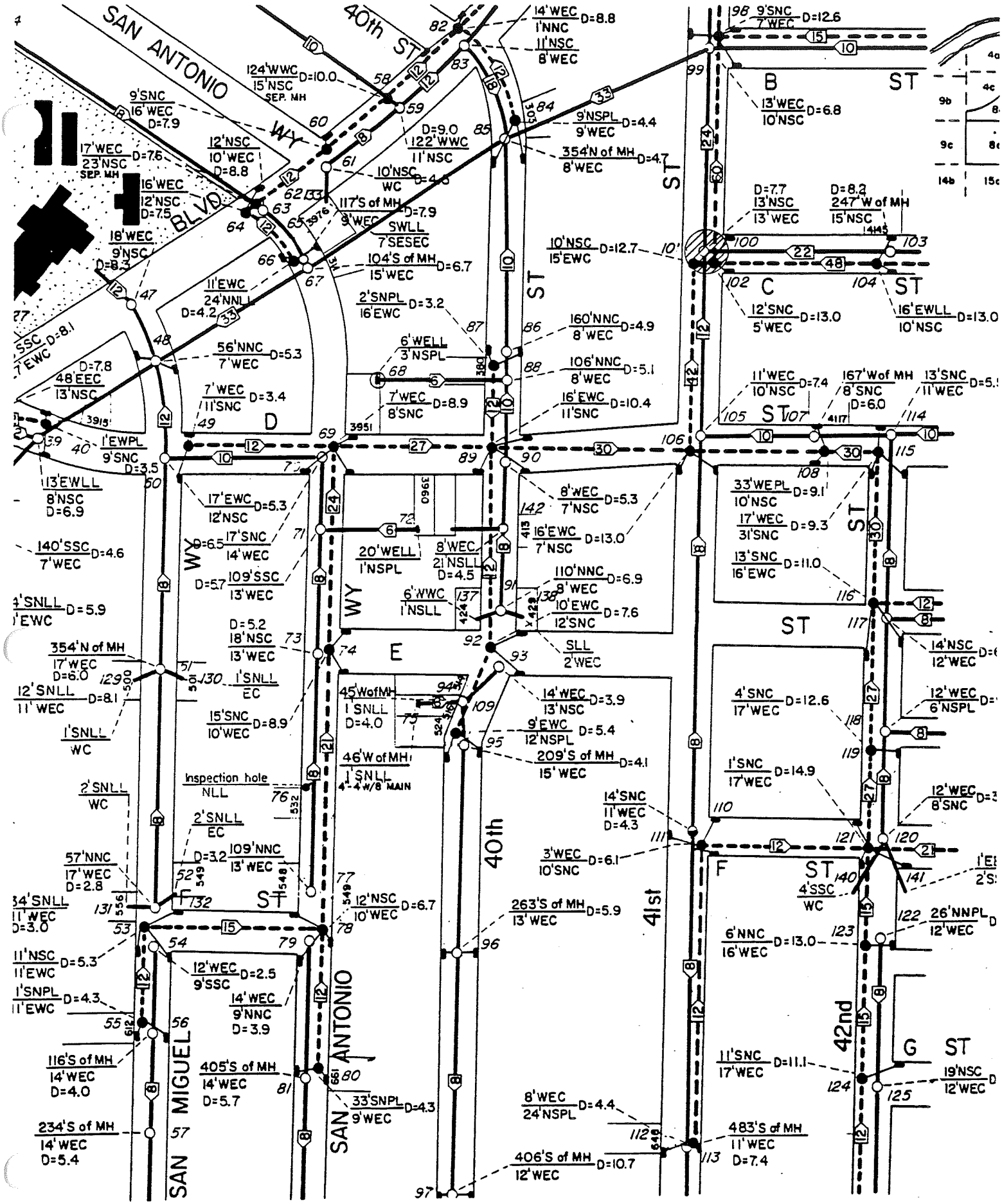
0740-001-141-19
T2-FIG02.DWG - XREF=P:\BASEMAPS\VSEG\SE & SW
07-13-92 - KAH

Appendix A

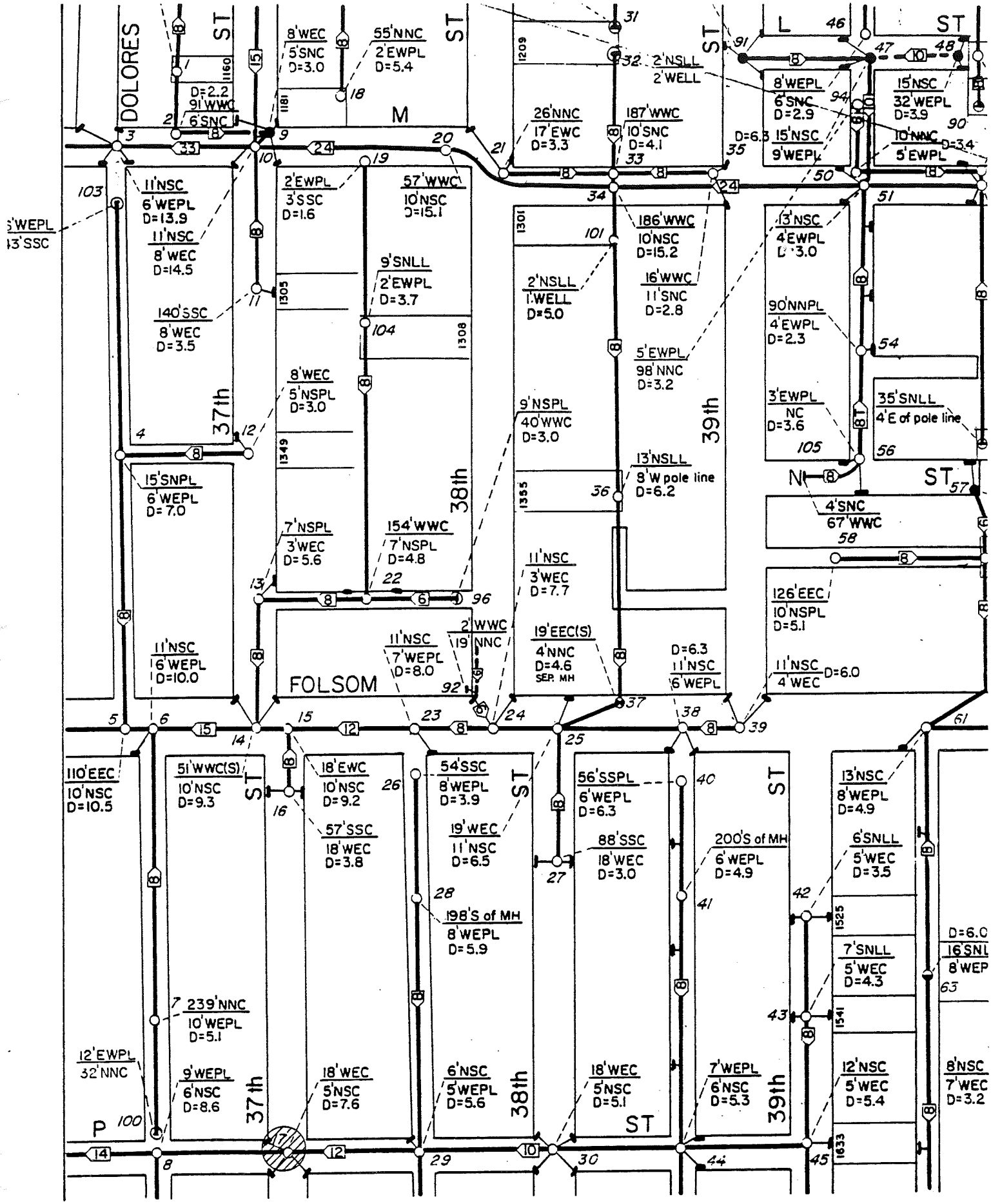
Detailed Maps for Recommended Potential Monitoring Locations



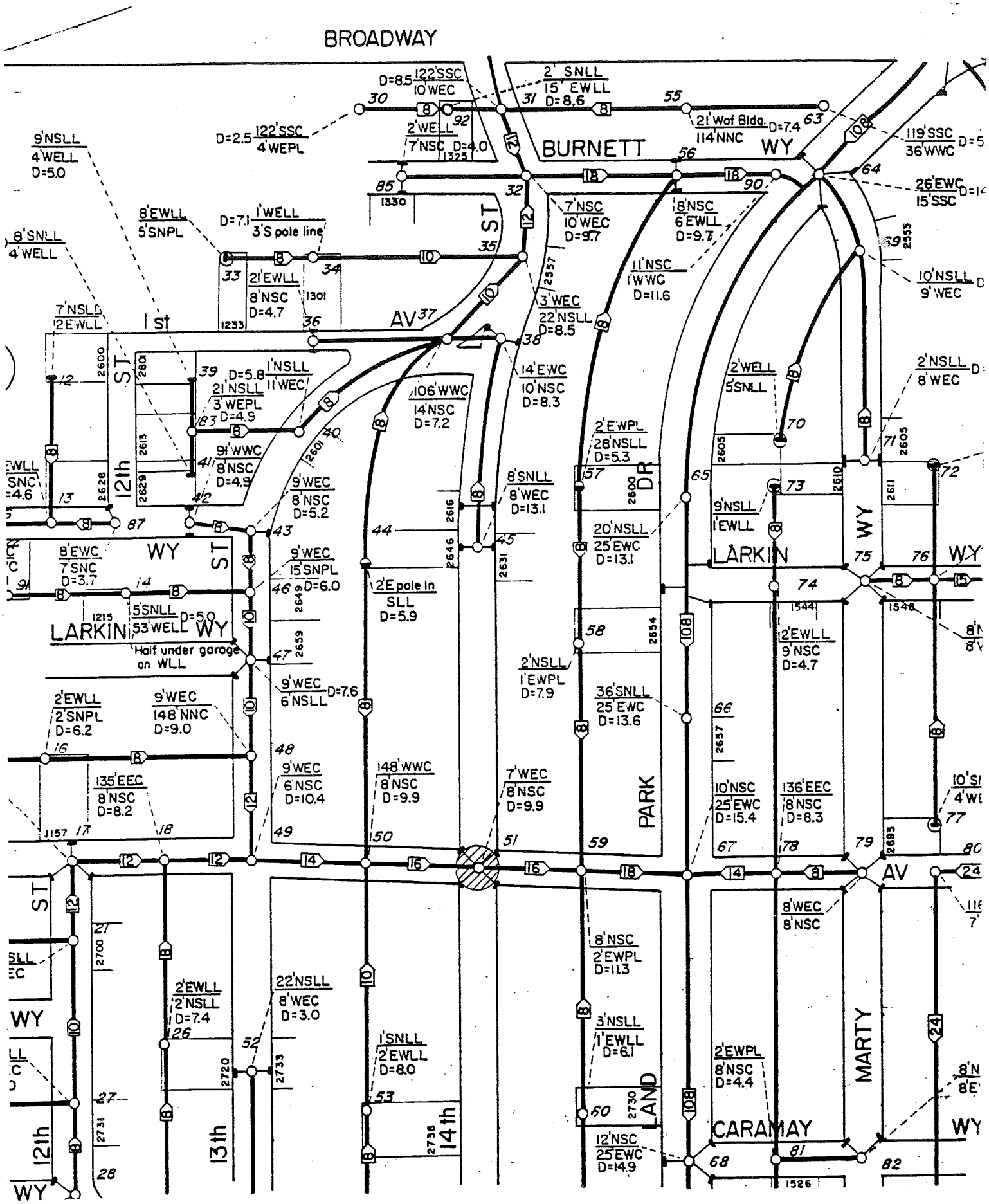
① *Manhole 20/24b, 5th Street Between R & S Streets*



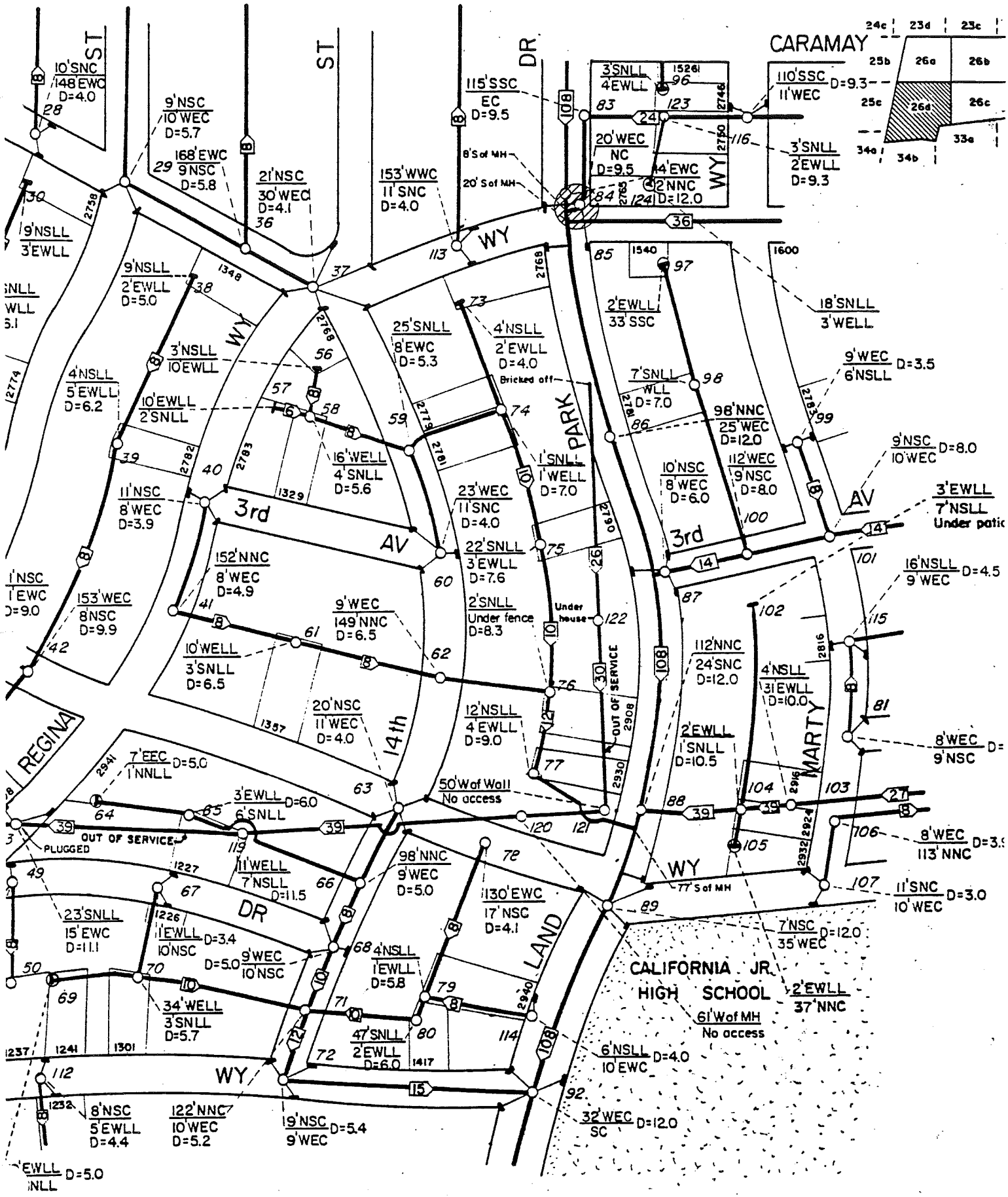
② **Manhole 100/8c, 41st & C Streets**



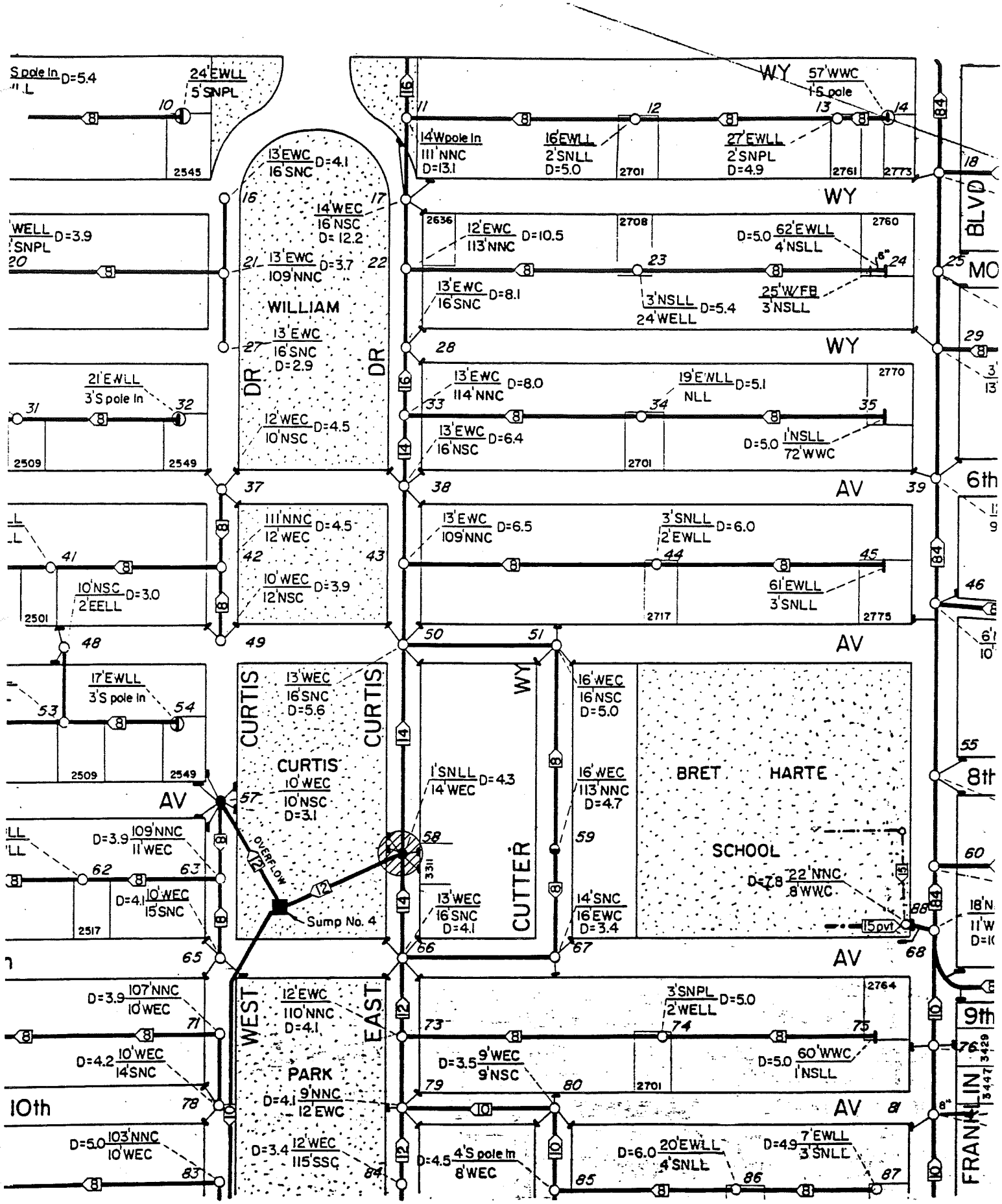
③ ***Manhole 17/15c, 37th & P Streets***



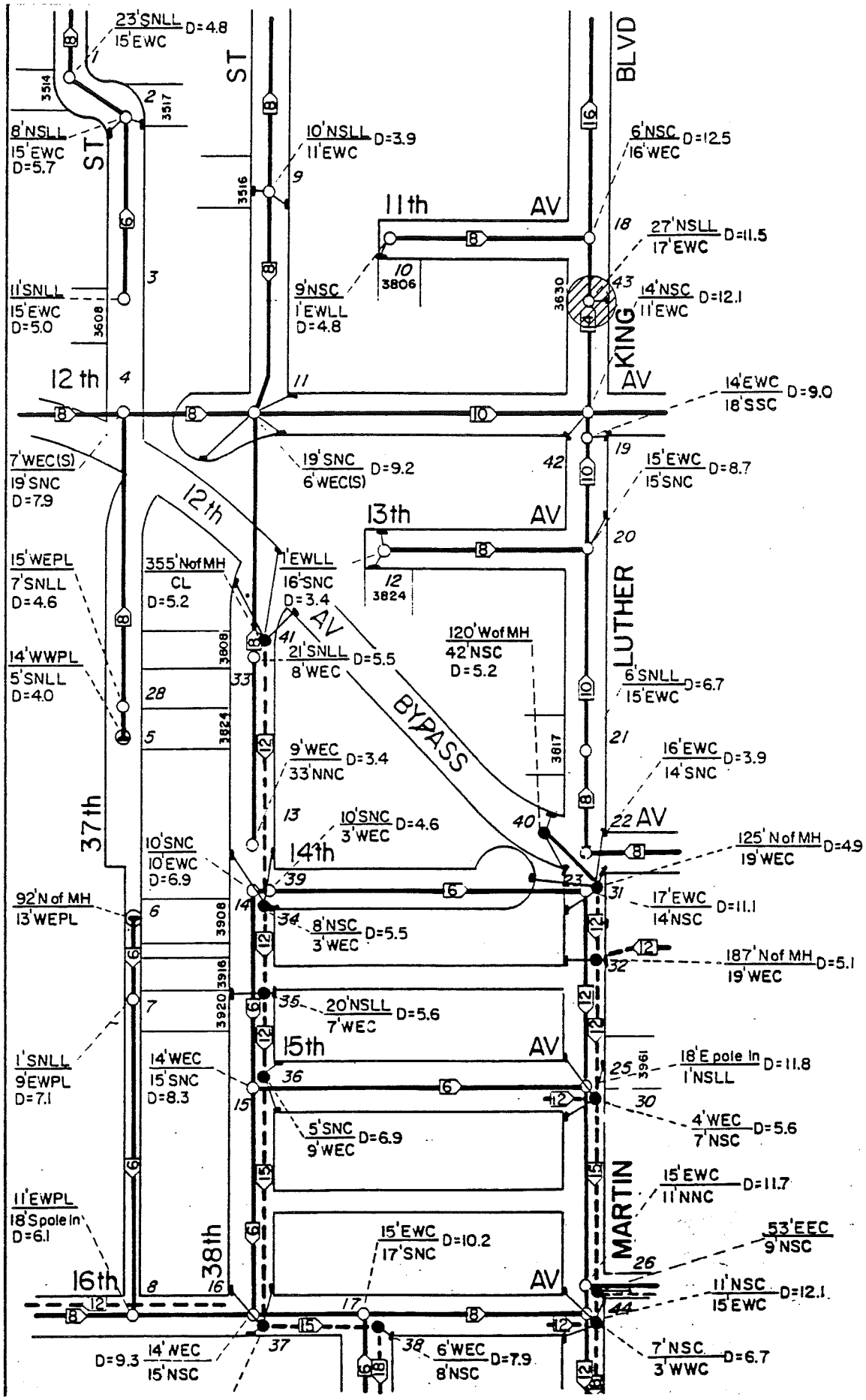
④ ***Manhole 51/26a, 14th Street & 2nd Avenue***



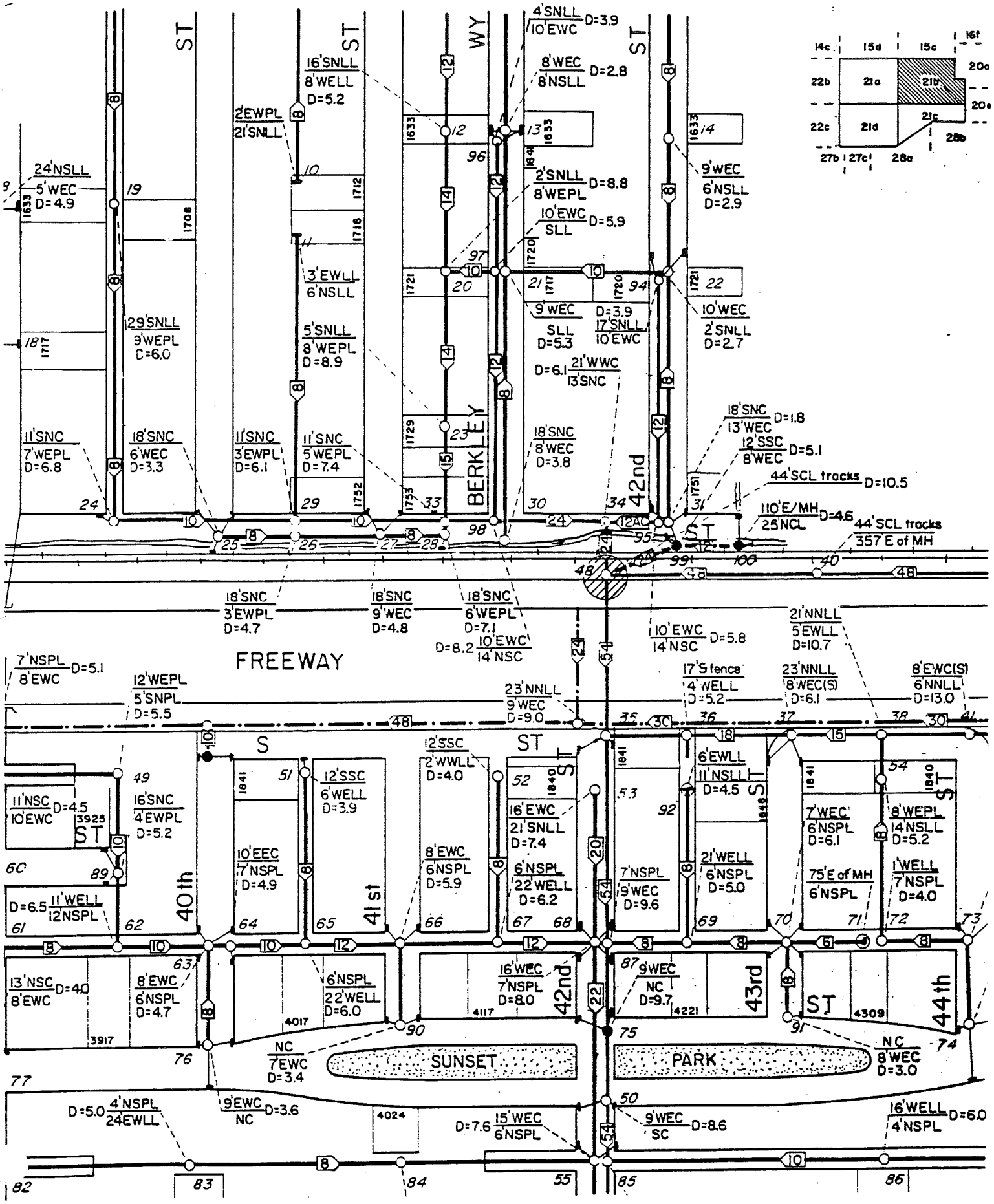
(5) ***Manhole 84/26d, Land Park Drive & Markham Way***



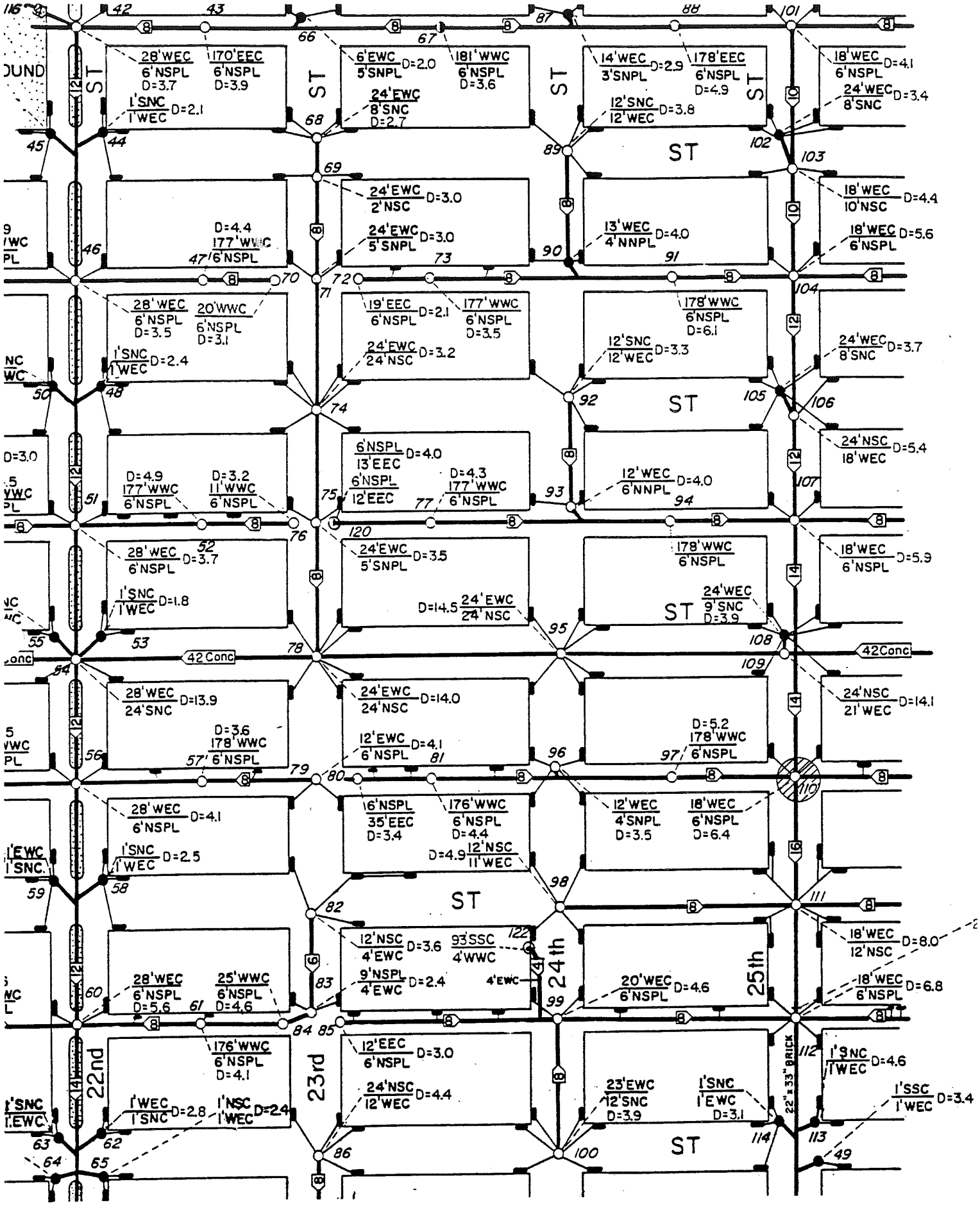
6 *Manhole 58/32a, East Curtis Drive Between 7th & 9th Avenues*



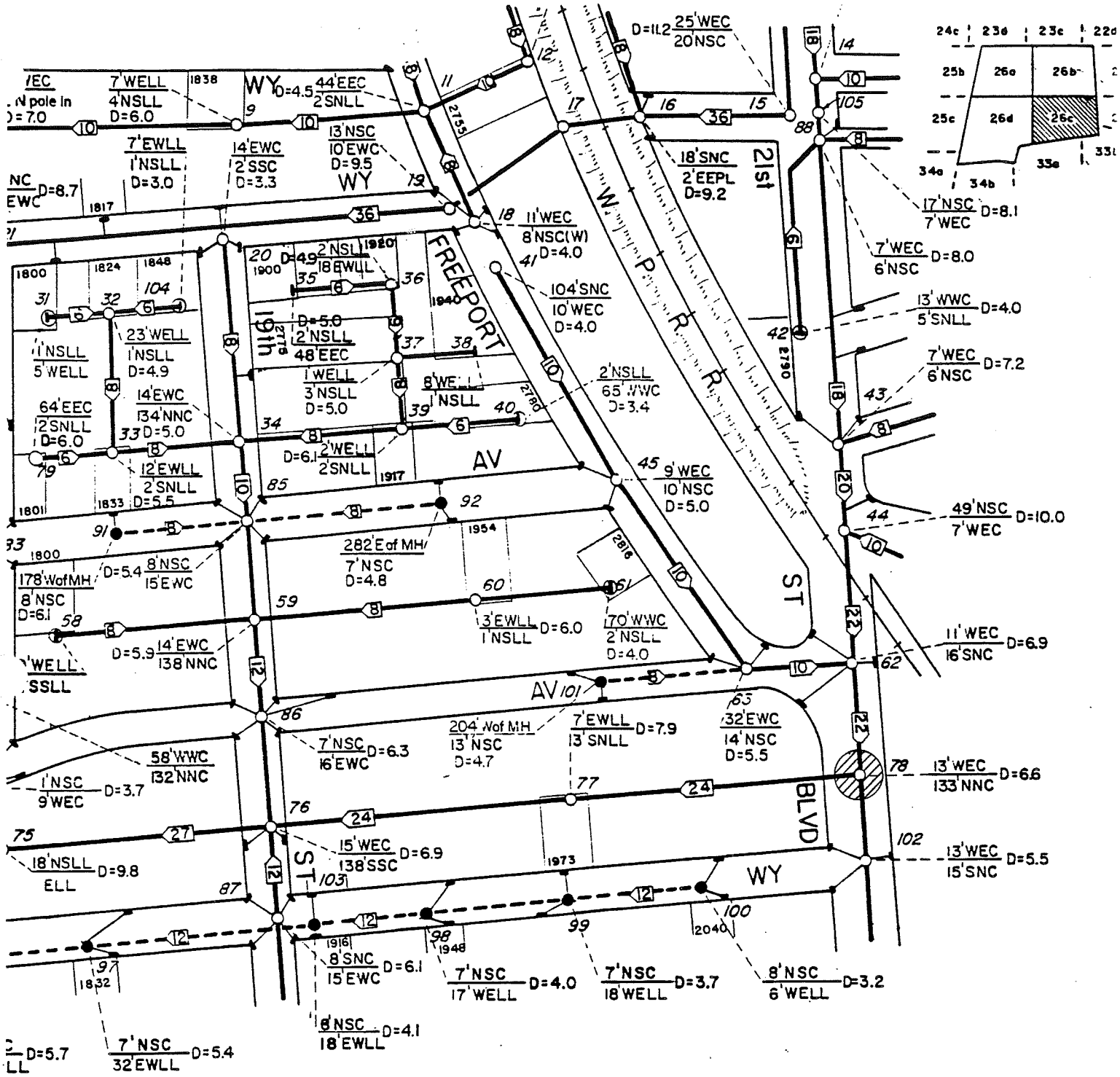
(7) ***Manhole 43/32d, Martin Luther King Blvd. Between 11th & 12th Streets***



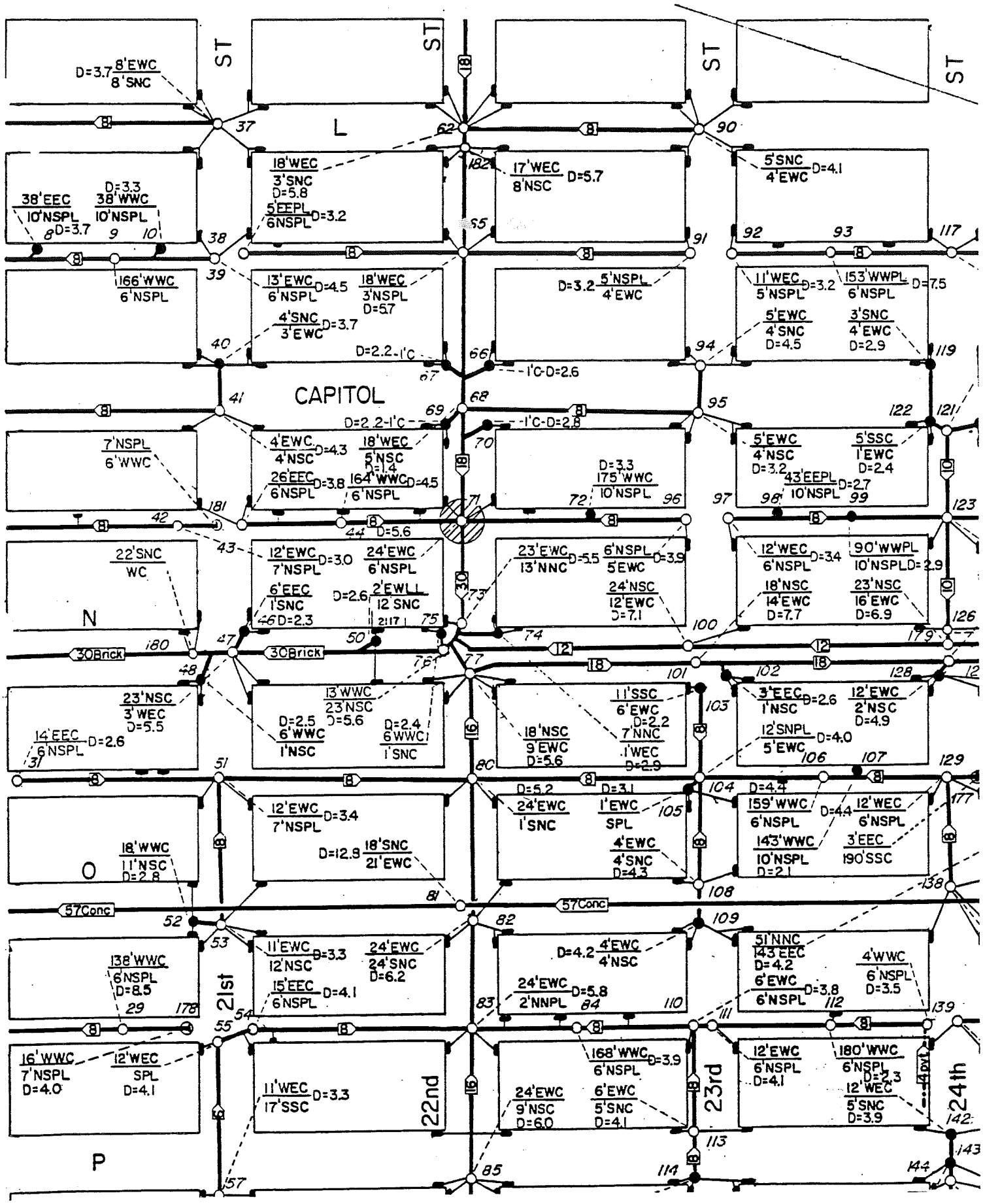
(8) ***Manhole 48/21b, 42nd & R Streets***



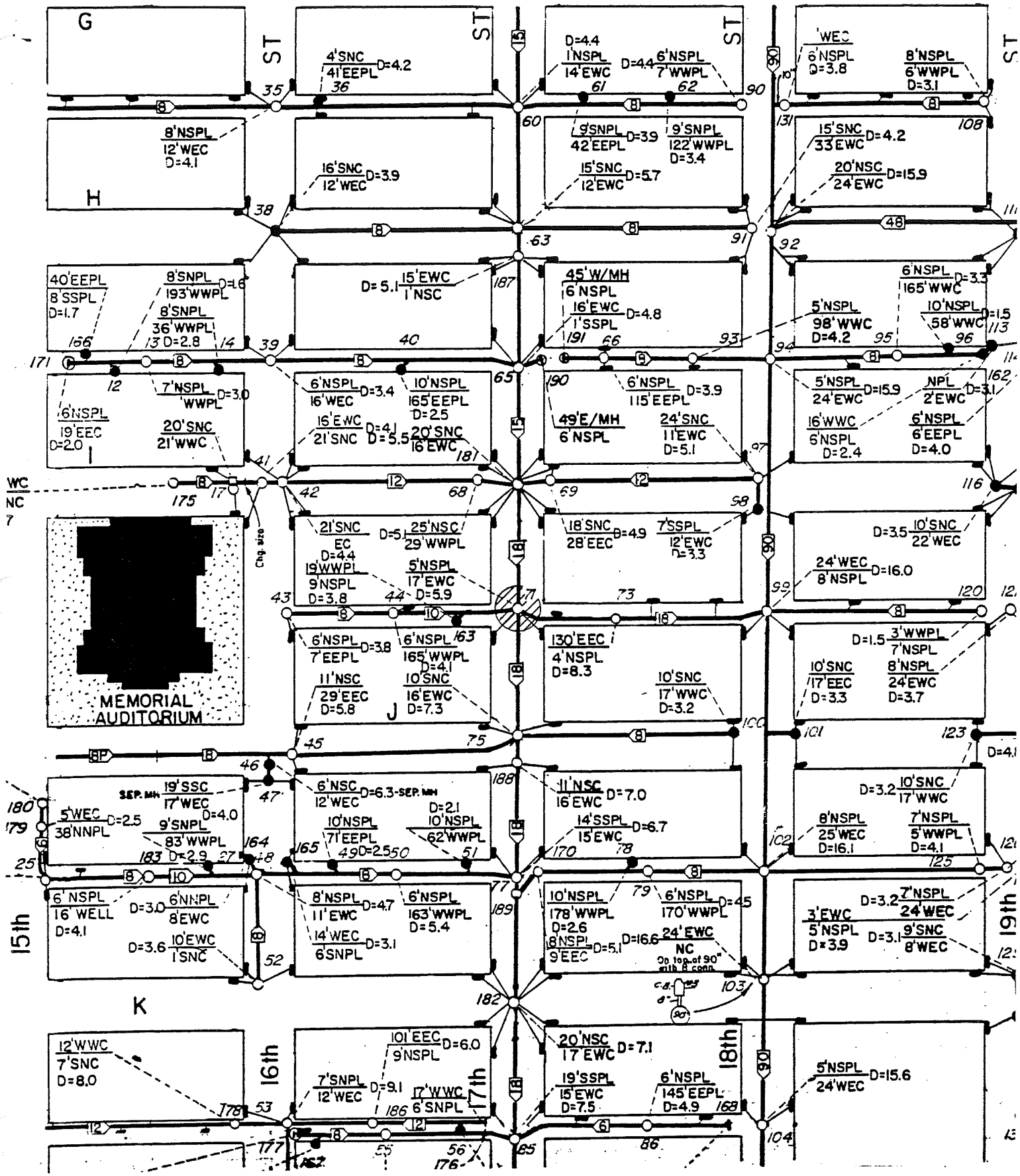
(9) Manhole 110/9d, 25th Street Between E & F Streets



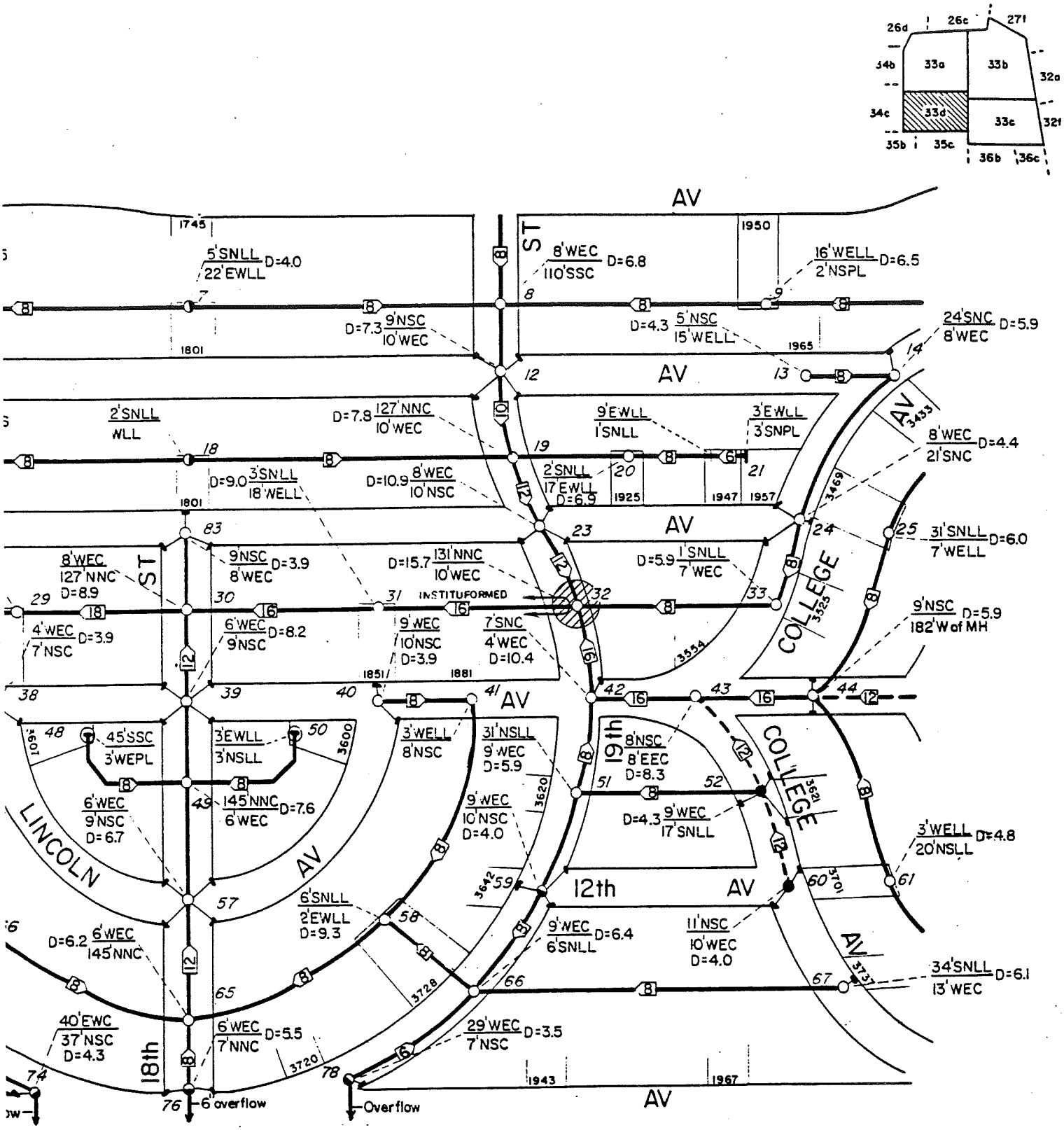
(10) ***Manhole 78/26c, 21st Street Between Vallejo & 4th Streets***



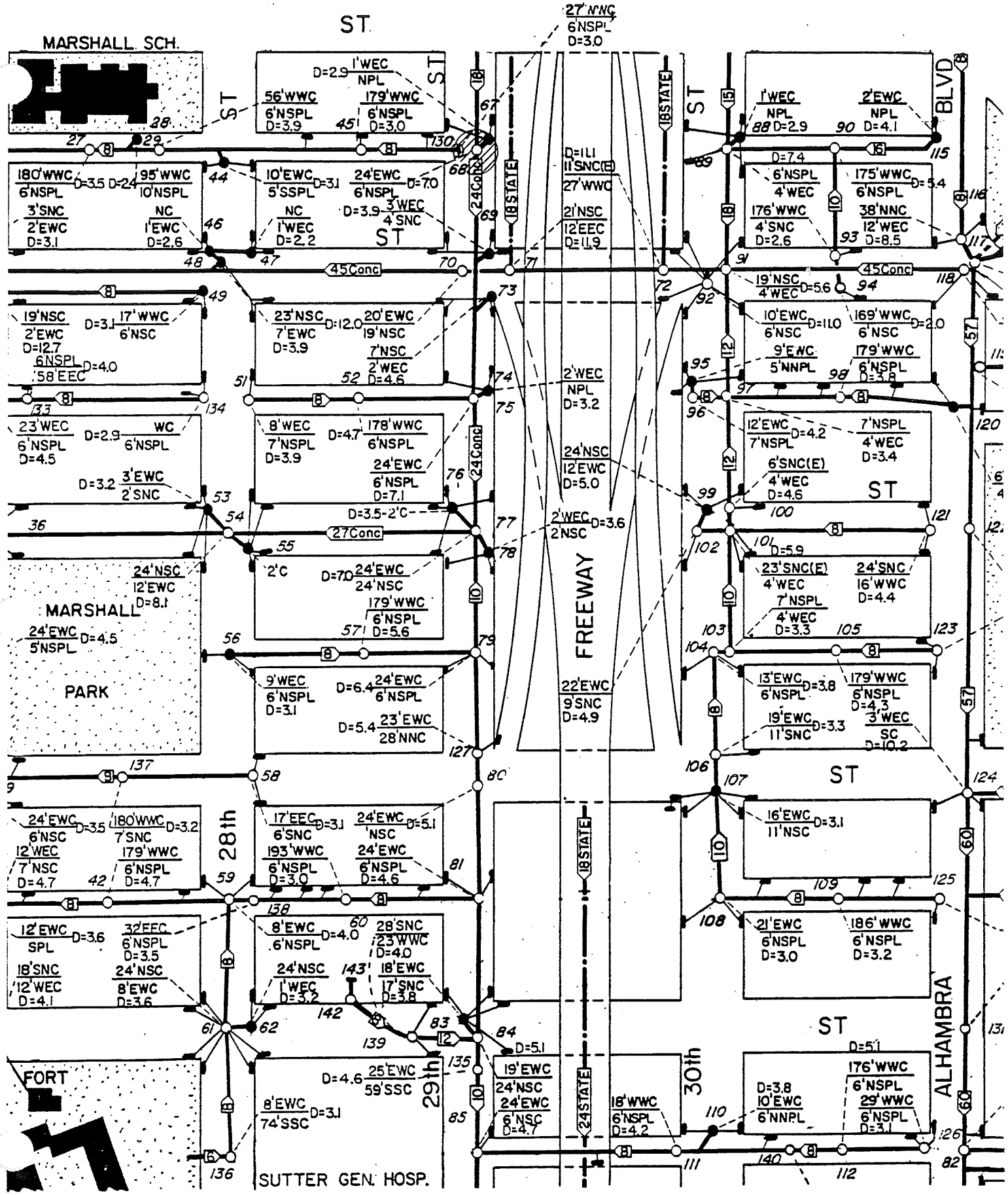
(11) ***Manhole 71/14d, 22nd Street Between M & N Streets***



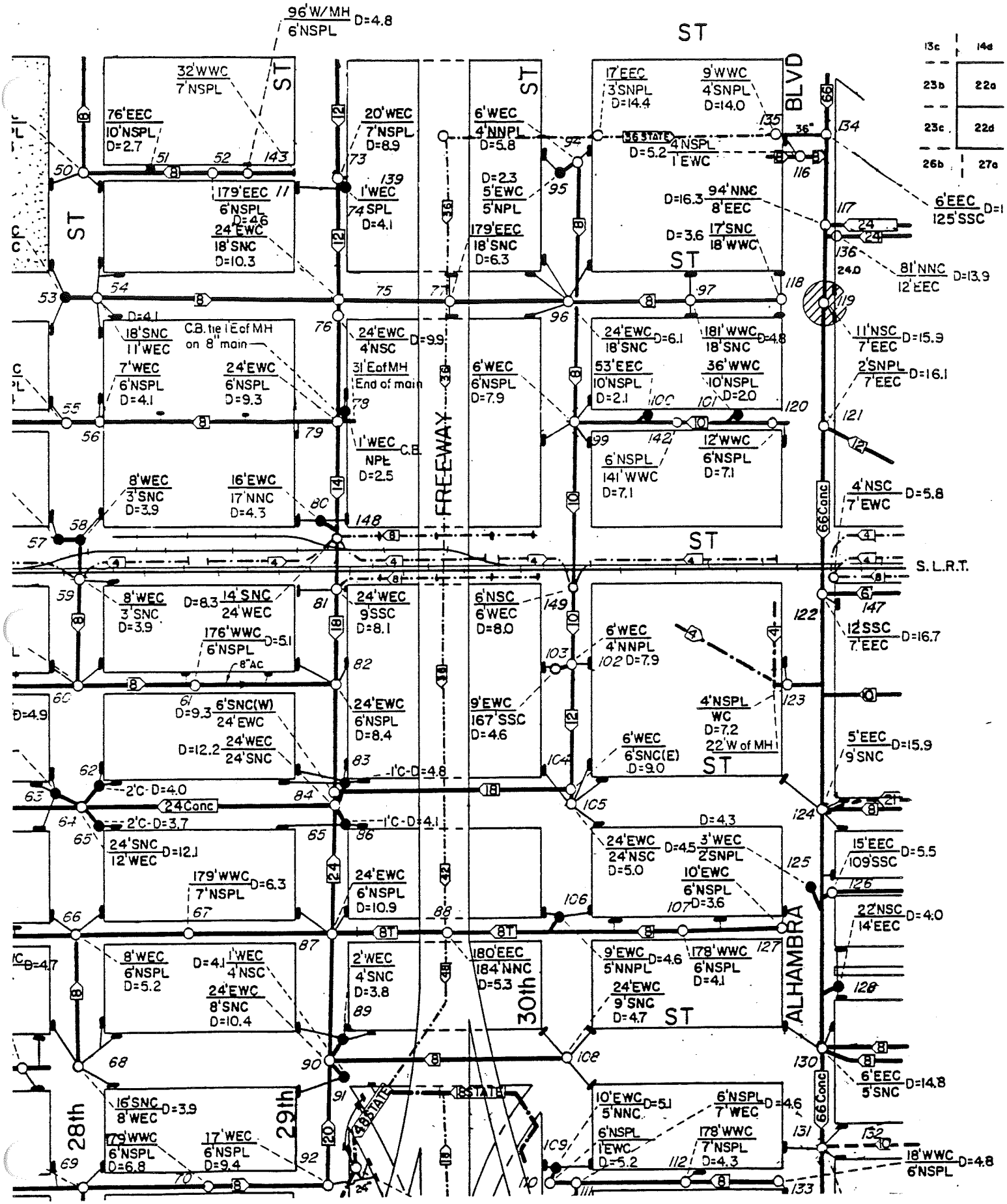
(12) *Manhole 71/13b, 17th Street Between I & J Streets*



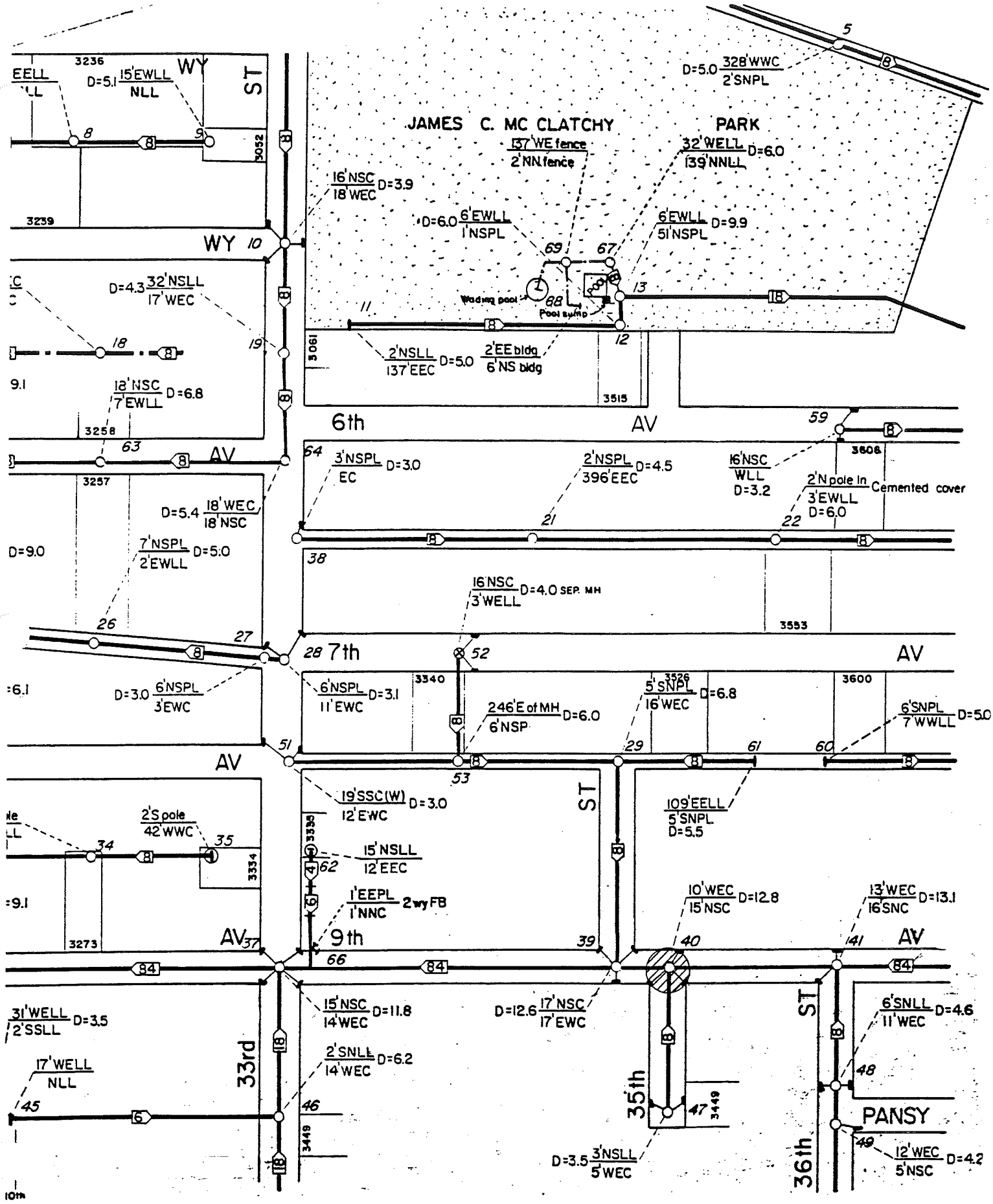
(13) ***Manhole 32/33d, 19th Street Between 10th & 11th Streets***



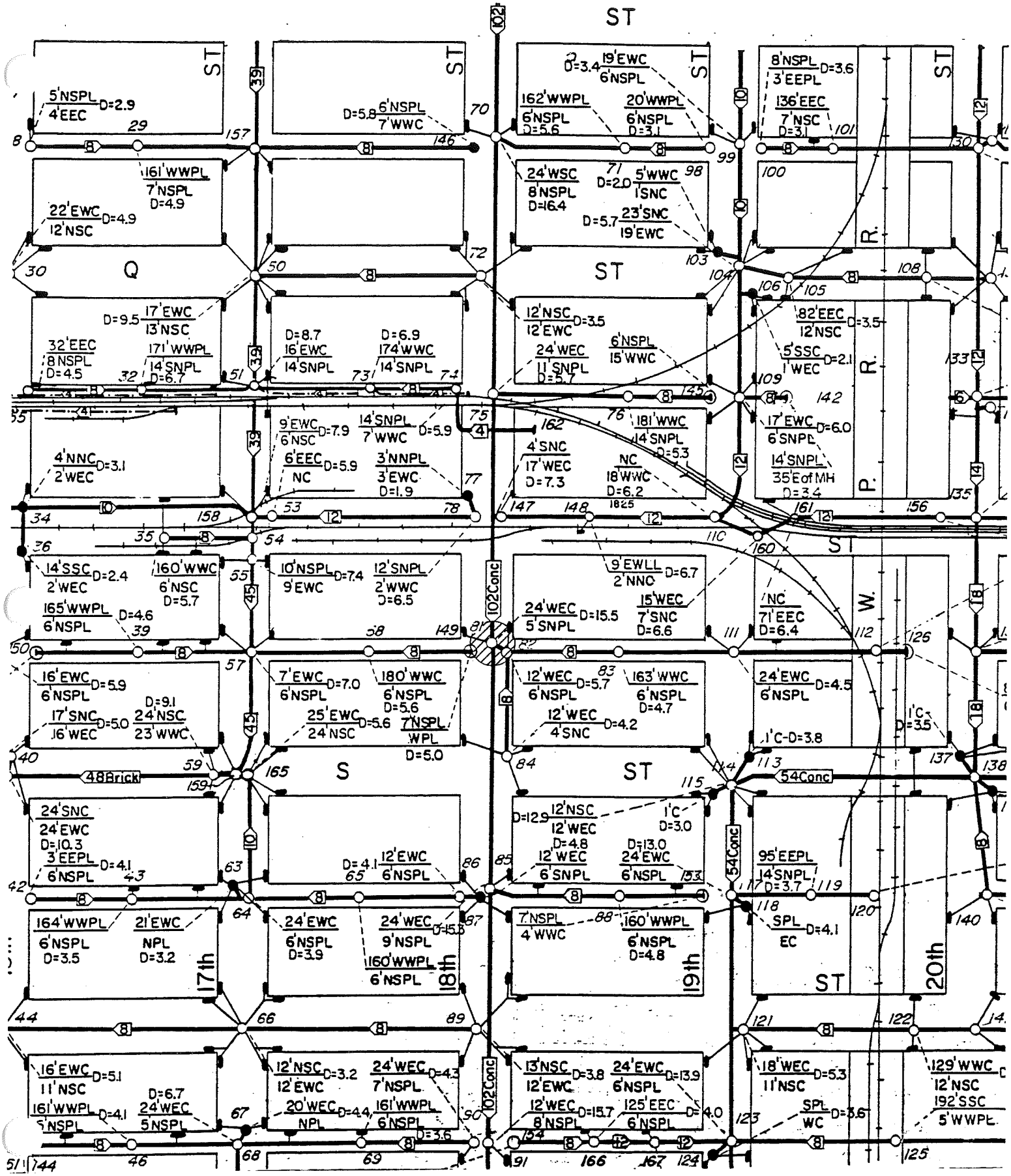
(14) **Manhole 68/14b, 29th Street Between G & H Streets**



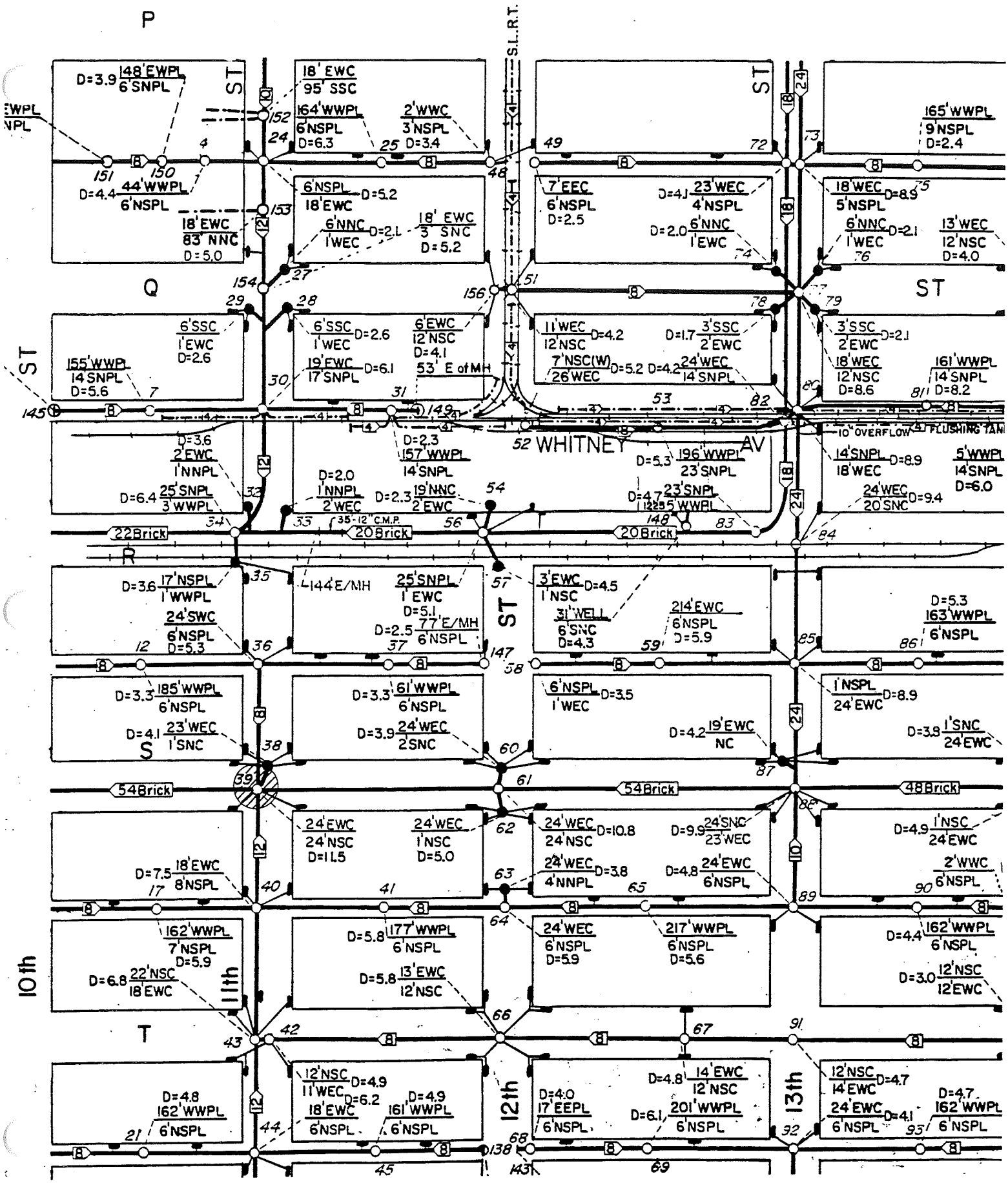
(15) **Manhole 119/22b, Alhambra & Stockton Blvds.**



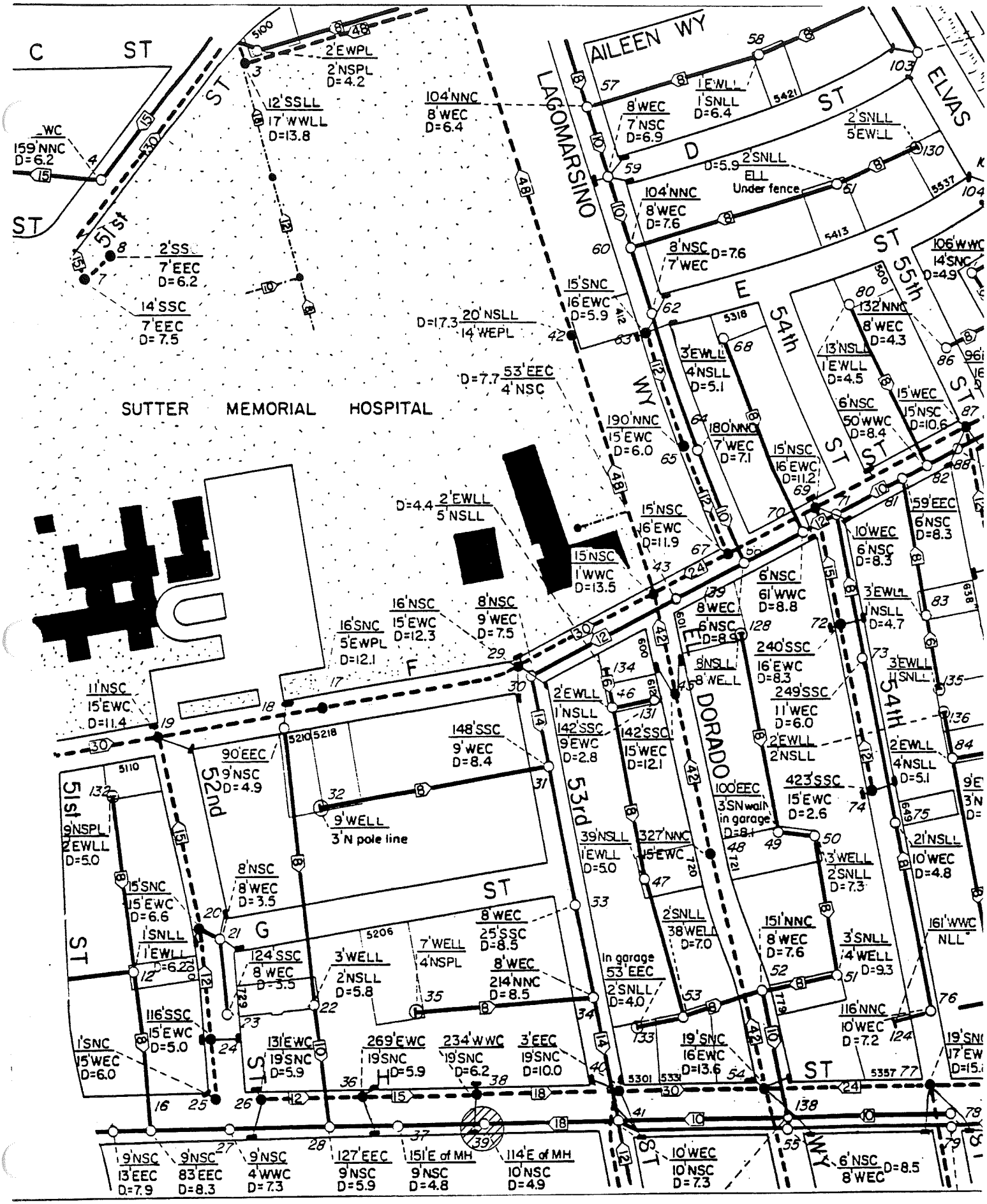
(16) ***Manhole 40/32b, 35th Street & 9th Avenue***



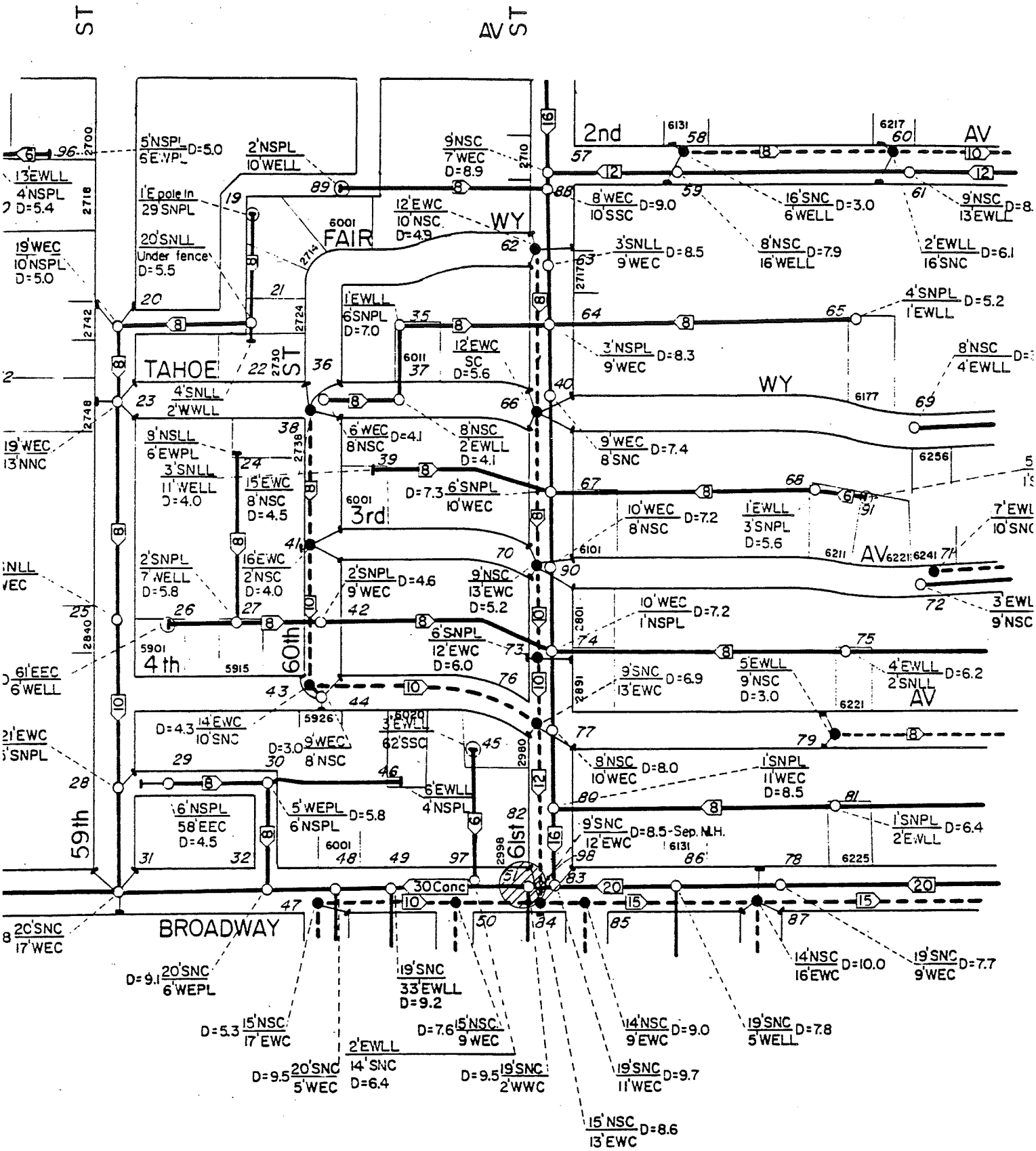
(17) **Manhole 81/23b, 18th Street Between R & S Streets**



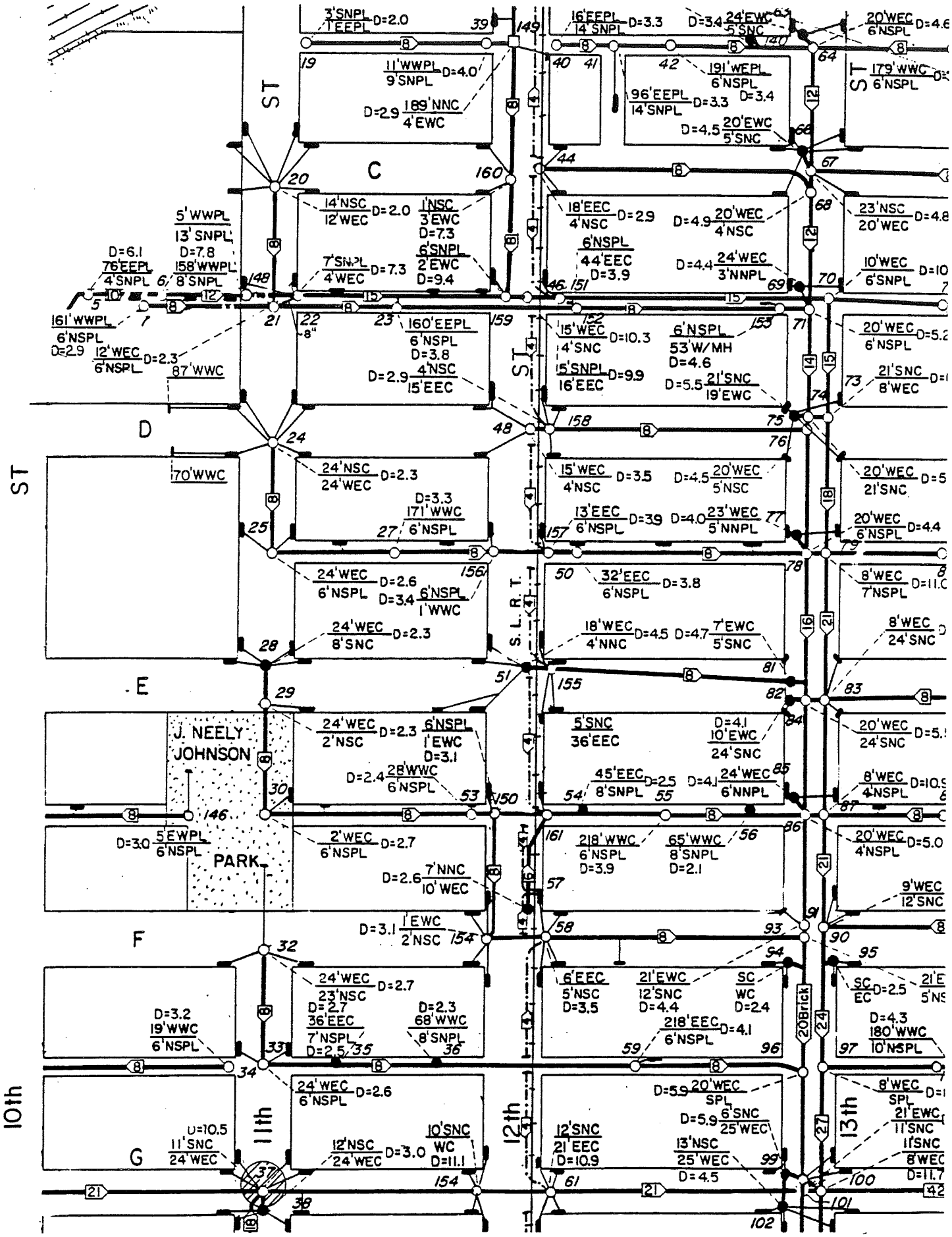
(18) ***Manhole 39/23a, 11th & S Streets***



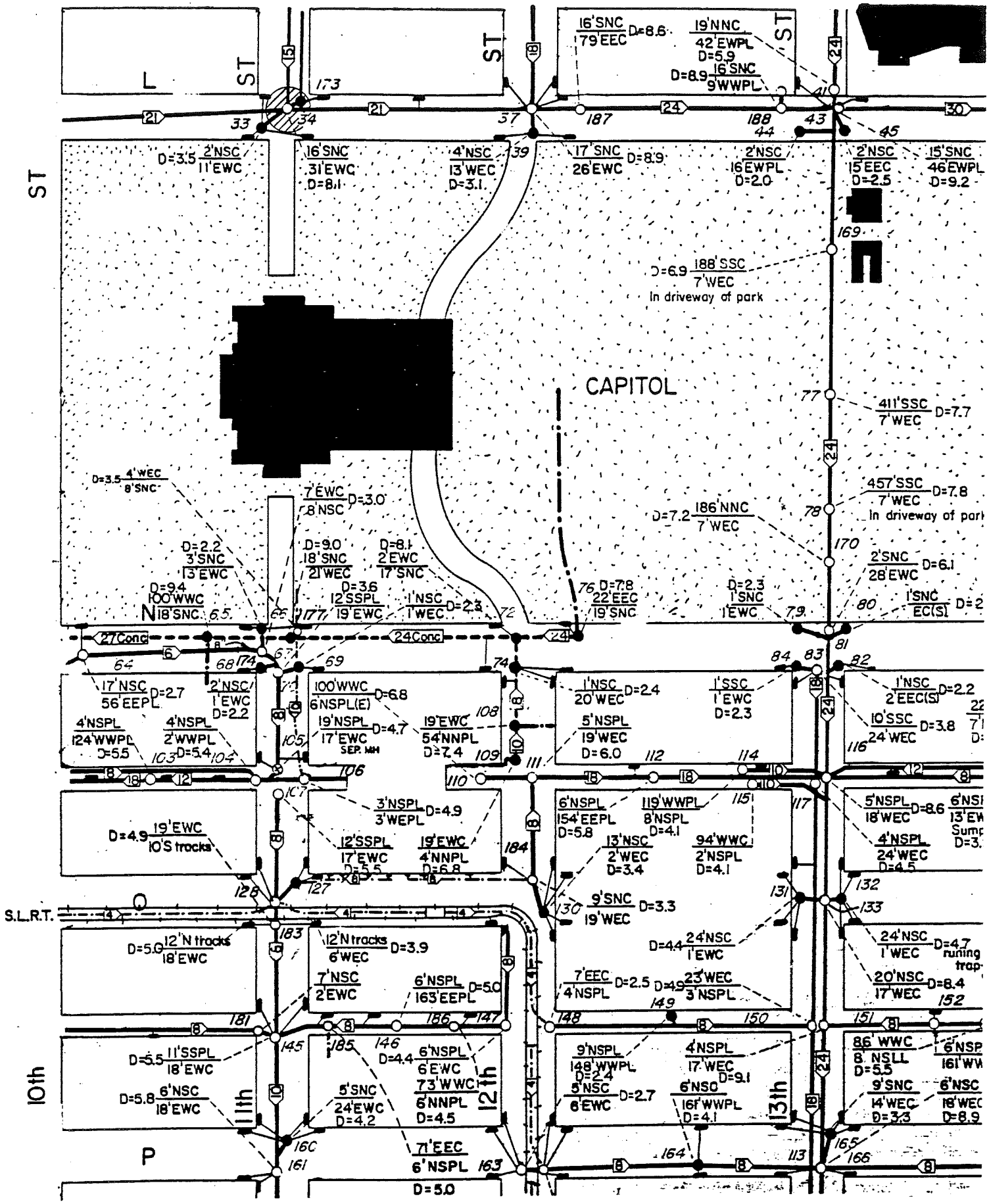
(19) ***Manhole 39/7c, 53rd & H Streets***



(20) Manhole 51/19e, 61st Street & Broadway



(21) **Manhole 37/10d, 11th & G Streets**



(22) *Manhole 34/13d, 11th & L Streets*

APPENDIX H

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM
IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 3

DESIGN STORMS, RAINFALL ANALYSIS AND FLOOD VOLUME MAPS FOR THE PHASE 3 ANALYSIS

Judy A. Zavadil
Laurie Carrette
Paula M. Arsenault
HDR Engineering, Inc.

Jack Humphrey
Hydmet, Inc.

August 13, 1992

TECHNICAL MEMORANDUM
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN
PHASE 3, TASK 3

PREPARED BY: Judy Zavadil, Laurie Carrette and Paula Arsenault, HDR Engineering, Inc.
Jack Humphrey, HYDMET, Inc.

DATE: July 1, 1992

SUBJECT: Design Storms, Rainfall Analysis and Flood Volume Maps for the Phase 3 Analysis

A. INTRODUCTION

Design storms are a hypothetical distribution of rainfall depths or intensities over a time increment for a given storm duration and frequency. Synthetic design storms were used in the Phase 3 analysis to determine the performance of proposed improvement plans measured in terms of flood and CSO control. Design storms were input to the SSWMM91 RUNOFF block to create runoff hydrographs. Alternative improvement plan facilities such as conveyance improvements, storage facilities and treatment facilities were then sized to handle the flows produced by the design storms.

Two sets of design storms were used to analyze alternative improvement plans. The first involves design storms for flood control project sizing. Design storms for flood control are six hours in duration with return periods of 2, 5, 10 and 25 years. The second involves design storms for sizing CSO control facilities, such as new storage facilities and treatment plants. Storm frequencies of 1/year and 5/year were used for CSO control project sizing. Each set of alternative plans is designed to meet a combination of CSO and flood control design storm criteria; an example is the 5-year flood control and 1/year CSO control combination.

This memorandum describes the approach used in developing the design storms for Phase 3 of the Combined Sewer Improvement and Implementation Plan Study. Design storms used for various types of improvement projects involved in the Phase 3 analysis are summarized in Table 1.

TABLE 1. DESIGN STORM CRITERIA FOR ALTERNATIVE IMPROVEMENT PLAN PROJECTS						
	CSO Control		Flood Control			
Alternative	5/Yr	1/Yr	2-Yr	5-Yr	10-Yr	25-Yr
Tunnels	X	X	X	X	X	X
Storage	X	X	X	X	X	
Treatment	X	X	X	X	X	
Separation - New Storm Drain					X	

The flood volume analysis and mapping for the existing system as a consequence of the 2-, 5-, 10-, and 25-year design storms is also included as part of this memorandum. These maps identify the flooding volumes imposed on the existing system by the four flood control design storms, and represent the conditions for which improvements plans were designed. Existing system SSWMM91 network maps, including peak flooding volumes for the four flood control design storms, are appended to this memorandum.

B. PREVIOUS STUDIES

The analysis presented herein draws extensively upon precipitation statistical analyses recently completed for the City and County of Sacramento. (This work will be referred herein as the "County Hydrology Study".) A portion of this study involved developing rainfall depth-duration-frequency data and a procedure for constructing design storms specifically for the County area, including the City. The results of the rainfall analyses are presented in a technical memorandum entitled *Task 1.4: Analysis of Precipitation Data* (dated November 12, 1990), which was provided to the City's Flood Control and Sewer Division and the County's Department of Water Resources. Other work associated with the County Hydrology Study, including the development of correction factors for adjusting for elevation and spacial conditions, were prepared and submitted to the City and County. The County's Task 1.4 TM is appended to this memorandum for reference purposes. (Graphical analysis plots are included only for the National Weather Service downtown Sacramento gage (No. 190), due to the numerous plots contained

in the original TM.) It should also be noted that the elevation correction factors presented in the Task 1.4 TM have been modified since the document was submitted. Elevation correction factors, however, were not used in developing design storms for the City combined sewer system.

C. FLOOD CONTROL DESIGN EVENTS

C.1. Frequency Selection

In the Phase 2 study, the 10-year design event was used for flood control analysis. Because of the large capital investment necessary to control flooding as a consequence of the 10-year event, the City proposed to evaluate more frequent design events, in order to determine a cost-effective and affordable level of flood control for the combined sewer system. The 2- and 5-year events were proposed to provide cost and benefit comparisons. The 10-year event was also used again to evaluate alternatives in the Phase 3 study.

In addition, the Regional Board requested that alternatives be evaluated for an event that would produce flows approximately 30 percent greater than the 10-year event. Since peak flows are a result of peak intensities, the 25-year event was selected. This event contains peak intensities that are approximately 27 percent greater than the 10-year event peak intensities.

C.2. Design Event Definitions

Design storms are a distribution of rainfall depths or intensities over a time increment for a given storm duration and frequency. The following three elements define a design storm: precipitation depth; duration; and frequency. Definitions for these elements follow.

- Precipitation depth is the amount of precipitation occurring during a specified storm duration. The depths of rainfall are statistical depths obtained by analyzing historical precipitation data to find the maximum rainfall depth for each duration for a particular frequency. Precipitation depth is usually expressed in inches.

- Duration is the specified length of a storm event. The duration of a design event for sizing conveyance facilities (pipelines, pumping stations) should be long enough to capture the entire runoff response from the most hydraulically remote point located in the watershed. The duration chosen for the Phase 2 alternative analysis for flood control was 6 hours; use of this duration has continued for the Phase 3 analysis. This 6-hour duration would provide adequate time for the entire watershed to contribute to direct runoff, and for flows from the farthest point in the system to reach Sump 2.

- The potential for a given rainfall event to occur is specified according to the precipitation depth and duration. This is expressed as return period (for events less frequent than 1/year) or frequency (for events more frequent than 1/year), in years. The return period or frequency assigns a probabilistic definition to a precipitation event.

C.3. Depth-Duration-Frequency Analysis

Given a long history of maximum rainfall depths for varying durations, a statistical interpretation (log-normal) can be made to determine estimates of maximum rainfall intensities or depths as a function of storm duration and of return frequency. As previously mentioned, a detailed precipitation analysis to derive Depth-Duration-Frequency, D-D-F, values from 88 years of precipitation records for the NWS Sacramento Gage No. 190, located in downtown Sacramento, was completed as part of the County study. These storms were based on annual maximum frequency analysis for 2, 5, 10, 15, 30 minute, 1, 2, 3, 6, 12 and 24 hour, 2, 3, 5, and 10 day durations for recurrence intervals of 2, 5, 10, 25, 50, 100, 200 and 500 years. The resulting depth-duration-frequency values are given in Table 2.

Further discussion of the methodology involved in developing depth-duration-frequency values can be found in the appended technical memorandum. This methodology is consistent with that used by the California Department of Water Resources.

TABLE 2. DEPTH-DURATION-FREQUENCY DATA FOR NWS DOWNTOWN SACRAMENTO GAGE NO. 190 (inches)								
Duration	2yr	5yr	10yr	25yr	50yr	100yr	200yr	500yr
2 min	0.07	0.12	0.16	0.21	0.25	0.29	0.33	0.39
5 min	0.13	0.20	0.25	0.32	0.38	0.44	0.49	0.58
10 min	0.19	0.29	0.36	0.46	0.54	0.62	0.70	0.82
15 min	0.23	0.35	0.43	0.55	0.64	0.73	0.82	0.96
30 min	0.32	0.47	0.57	0.72	0.83	0.94	1.04	1.22
1 hr	0.45	0.64	0.77	0.94	1.07	1.21	1.33	1.53
2 hr	0.64	0.88	1.04	1.26	1.42	1.59	1.76	2.00
3 hr	0.77	1.04	1.23	1.47	1.66	1.85	2.03	2.31
6 hr	1.06	1.40	1.65	1.95	2.22	2.50	2.75	3.10
12 hr	1.43	1.91	2.25	2.67	3.00	3.30	3.60	4.00
1 day	1.90	2.50	2.98	3.46	3.85	4.25	4.60	5.20
2 day	2.51	3.40	3.95	4.65	5.15	5.70	6.20	7.00
3 day	3.00	4.07	4.65	5.50	6.20	6.80	7.50	8.40
5 day	3.61	4.91	5.76	6.85	7.63	8.42	9.20	10.29
10 day	4.73	6.44	7.54	8.96	9.97	11.01	11.95	13.45

C.4. Area Adjustments

Point precipitation gage statistics are only representative of areas of a few hundred acres. The distance for significant correlation between point gage measurements is characteristically a few miles for short duration precipitation (less than one hour) and up to a few hundred miles for long duration precipitation. Multiplying factors were developed for converting point gage data to area data. These factors are given in Table 3 and Table 4. The values in Table 3 are used to adjust the short duration depths (less than an hour), and Table 4 is used for the longer duration depths (e.g. 2- and 3- hour depths).

TABLE 3. AREA MULTIPLIERS FOR PRECIPITATION CLOUDBURST STORM (1-HR AND LESS DURATIONS)								
Area (sq mi)	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr	500-yr
0.4	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0.6	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0.8	0.94	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.0	0.92	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2.0	0.85	0.96	1.00	1.00	1.00	1.00	1.00	1.00
3.0	0.82	0.92	0.98	1.00	1.00	1.00	1.00	1.00
4.0	0.79	0.89	0.95	1.00	1.00	1.00	1.00	1.00
5.0	0.77	0.87	0.93	1.00	1.00	1.00	1.00	1.00
6.0	0.76	0.85	0.91	0.98	1.00	1.00	1.00	1.00
7.0	0.74	0.84	0.90	0.96	1.00	1.00	1.00	1.00
8.0	0.73	0.83	0.88	0.95	0.99	1.00	1.00	1.00
9.0	0.72	0.82	0.87	0.93	0.97	1.00	1.00	1.00
10.0	0.71	0.81	0.86	0.92	0.96	0.99	1.00	1.00
12.0	0.70	0.79	0.84	0.91	0.94	0.97	1.00	1.00
14.0	0.69	0.78	0.83	0.89	0.93	0.96	0.99	1.00
16.0	0.68	0.77	0.82	0.88	0.91	0.94	0.98	1.00
18.0	0.67	0.76	0.81	0.87	0.90	0.93	0.97	1.00
20.0	0.66	0.75	0.80	0.86	0.89	0.92	0.96	1.00
25.0	0.65	0.73	0.78	0.84	0.87	0.90	0.93	0.98
30.0	0.63	0.72	0.76	0.82	0.85	0.88	0.91	0.96
35.0	0.62	0.70	0.75	0.80	0.84	0.87	0.90	0.94
40.0	0.61	0.69	0.74	0.79	0.83	0.85	0.89	0.93
45.0	0.61	0.68	0.73	0.78	0.82	0.84	0.87	0.91
50.0	0.60	0.68	0.72	0.77	0.81	0.83	0.86	0.90
60.0	0.59	0.66	0.71	0.76	0.79	0.82	0.85	0.89
70.0	0.58	0.65	0.70	0.75	0.78	0.80	0.83	0.87
80.0	0.57	0.64	0.69	0.73	0.77	0.79	0.82	0.86
90.0	0.56	0.63	0.68	0.73	0.76	0.78	0.81	0.85
100.0	0.55	0.63	0.67	0.72	0.75	0.77	0.80	0.84
150.0	0.53	0.60	0.64	0.69	0.71	0.74	0.77	0.80
200.0	0.51	0.58	0.62	0.66	0.69	0.71	0.74	0.78

TABLE 4. AREA MULTIPLIERS FOR PRECIPITATION GENERAL RAINSTORM						
Area (sq mi)	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
0.4	1.00	1.00	1.00	1.00	1.00	1.00
0.6	1.00	1.00	1.00	1.00	1.00	1.00
0.8	1.00	1.00	1.00	1.00	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00
2.0	0.98	0.99	0.99	0.99	1.00	1.00
3.0	0.98	0.98	0.99	0.99	0.99	1.00
4.0	0.97	0.98	0.98	0.99	0.99	0.99
5.0	0.97	0.97	0.98	0.99	0.99	0.99
6.0	0.96	0.97	0.98	0.98	0.99	0.99
7.0	0.96	0.97	0.98	0.98	0.99	0.99
8.0	0.96	0.96	0.97	0.98	0.99	0.99
9.0	0.95	0.96	0.97	0.98	0.98	0.99
10.0	0.95	0.96	0.97	0.98	0.98	0.99
12.0	0.94	0.95	0.97	0.97	0.98	0.99
14.0	0.93	0.95	0.96	0.97	0.98	0.98
16.0	0.92	0.94	0.96	0.97	0.98	0.98
18.0	0.91	0.93	0.95	0.96	0.98	0.97
20.0	0.90	0.93	0.95	0.96	0.97	0.97
25.0	0.87	0.91	0.93	0.95	0.96	0.96
30.0	0.85	0.90	0.92	0.94	0.96	0.96
35.0	0.84	0.88	0.91	0.93	0.95	0.96
40.0	0.82	0.87	0.90	0.93	0.94	0.95
45.0	0.81	0.86	0.89	0.92	0.94	0.95
50.0	0.80	0.85	0.89	0.91	0.93	0.95
60.0	0.78	0.84	0.87	0.91	0.93	0.95
70.0	0.77	0.83	0.86	0.90	0.92	0.94
80.0	0.75	0.82	0.86	0.89	0.91	0.94
90.0	0.74	0.81	0.85	0.89	0.91	0.94
100.0	0.73	0.80	0.84	0.88	0.91	0.93
150.0	0.70	0.77	0.82	0.86	0.89	0.93
200.0	0.67	0.75	0.80	0.85	0.88	0.92

C.5. Precipitation Temporal Distribution

Time distribution of rainfall within storms is important in estimating design events. Distributions vary with storm type (orographic, convective), intensity and duration. There is no typical distribution that is applicable to all situations.

A symmetrical temporal distribution was chosen to represent the precipitation pattern in Sacramento. A 5-minute time increment for the precipitation was chosen, because it is the most detailed increment available, giving the storm the most detailed definition.

The storm pattern is formed by arranging the 5-minute incremental precipitation depths such that the maximum 5-minute depth is contained within the maximum 10-minute depth, the maximum 10-minute depth is contained within the maximum 15-minute depth, and so forth. The design storm pattern consists of 5-minute incremental precipitation depths nested within the storm duration in an alternating pattern.

The maximum 5-minute depth is placed at the center of the event and the second highest value following the center. The incremental 5-minute depths are created by subtracting the maximum 5-minute depth from the maximum 10-minute depth, and the 10-minute depth is subtracted from the 15-minute depth, and so forth, until an incremental depth for each time period is calculated. This storm construction method is identical to that used by the Bureau of Reclamation.

C.6. Results

The resulting 2, 5, 10 and 25 year hyetographs, presented in Table 5, were used in the analysis of flood and outflow control alternatives. Figures 1 and 2 are a graphic of the hyetographs. The flood control design storms were transmitted to the City as part of the SSWMM91 RUNOFF block input files for the existing combined sewer system.

TABLE 5. FLOOD CONTROL DESIGN STORMS FOR 2-, 5-, 10-, AND 25-YEAR RECURRENCE INTERVALS				
Time	2-Yr Depths (inches)	5-Yr Depths (inches)	10-Yr Depths (inches)	25-Yr Depths (inches)
00:00	0.000	0.000	0.000	0.000
00:05	0.008	0.010	0.012	0.013
00:10	0.008	0.010	0.012	0.013
00:15	0.008	0.010	0.012	0.013
00:20	0.008	0.010	0.012	0.013
00:25	0.008	0.010	0.012	0.013
00:30	0.008	0.010	0.012	0.013
00:35	0.008	0.010	0.012	0.013
00:40	0.008	0.010	0.012	0.013
00:45	0.008	0.010	0.012	0.013
00:50	0.008	0.010	0.012	0.013
00:55	0.008	0.010	0.012	0.013
01:00	0.008	0.010	0.012	0.013
01:05	0.008	0.010	0.012	0.013
01:10	0.008	0.010	0.012	0.013
01:15	0.008	0.010	0.012	0.013
01:20	0.008	0.010	0.012	0.013
01:25	0.008	0.010	0.012	0.013
01:30	0.008	0.010	0.012	0.013
01:35	0.011	0.014	0.016	0.018
01:40	0.011	0.014	0.016	0.018
01:45	0.011	0.014	0.016	0.018
01:50	0.011	0.014	0.016	0.018
01:55	0.011	0.014	0.016	0.018
02:00	0.011	0.014	0.016	0.018
02:05	0.015	0.023	0.028	0.029
02:10	0.015	0.023	0.028	0.029

TABLE 5. FLOOD CONTROL DESIGN STORMS FOR 2-, 5-, 10-, AND 25-YEAR RECURRENCE INTERVALS				
Time	2-Yr Depths (inches)	5-Yr Depths (inches)	10-Yr Depths (inches)	25-Yr Depths (inches)
02:15	0.015	0.023	0.028	0.029
02:20	0.015	0.023	0.028	0.029
02:25	0.015	0.023	0.028	0.029
02:30	0.015	0.023	0.028	0.029
02:35	0.015	0.023	0.028	0.033
02:40	0.015	0.023	0.028	0.033
02:45	0.015	0.023	0.028	0.033
02:50	0.021	0.032	0.040	0.052
02:55	0.028	0.048	0.060	0.082
03:00	0.092	0.160	0.213	0.292
03:05	0.042	0.072	0.094	0.128
03:10	0.021	0.032	0.040	0.052
03:15	0.021	0.032	0.040	0.052
03:20	0.015	0.023	0.028	0.033
03:25	0.015	0.023	0.028	0.033
03:30	0.015	0.023	0.028	0.033
03:35	0.015	0.023	0.028	0.029
03:40	0.015	0.023	0.028	0.029
03:45	0.015	0.023	0.028	0.029
03:50	0.015	0.023	0.028	0.029
03:55	0.015	0.023	0.028	0.029
04:00	0.015	0.023	0.028	0.029
04:05	0.011	0.014	0.016	0.018
04:10	0.011	0.014	0.016	0.018
04:15	0.011	0.014	0.016	0.018
04:20	0.011	0.014	0.016	0.018
04:25	0.011	0.014	0.016	0.018

TABLE 5. FLOOD CONTROL DESIGN STORMS FOR 2-, 5-, 10-, AND 25-YEAR RECURRENCE INTERVALS				
Time	2-Yr Depths (inches)	5-Yr Depths (inches)	10-Yr Depths (inches)	25-Yr Depths (inches)
04:30	0.011	0.014	0.016	0.018
04:35	0.008	0.010	0.012	0.013
04:40	0.008	0.010	0.012	0.013
04:45	0.008	0.010	0.012	0.013
04:50	0.008	0.010	0.012	0.013
04:55	0.008	0.010	0.012	0.013
05:00	0.008	0.010	0.012	0.013
05:05	0.008	0.010	0.012	0.013
05:10	0.008	0.010	0.012	0.013
05:15	0.008	0.010	0.012	0.013
05:20	0.008	0.010	0.012	0.013
05:25	0.008	0.010	0.012	0.013
05:30	0.008	0.010	0.012	0.013
05:35	0.008	0.010	0.012	0.013
05:40	0.008	0.010	0.012	0.013
05:45	0.008	0.010	0.012	0.013
05:50	0.008	0.010	0.012	0.013
05:55	0.008	0.010	0.012	0.013
06:00	0.008	0.010	0.012	0.013

D. CSO CONTROL DESIGN STORMS

In addition to flood control design storms, alternative improvement plans were evaluated for the control of design events representing frequencies of 1/year and 5/year storms.

The design criteria for controlling CSOs (untreated discharges) from the system is typically based on frequency of occurrence (e.g. control CSOs to 5 events per

year). The use of design storms assumes that precipitation of a particular recurrence will produce a sewer system response of the same recurrence. However, rainfall frequency is rarely coincident with the CSO frequency. For instance, runoff resulting from a 2-year design event does not necessarily result in a CSO every other year. The relationship between rainfall frequency and CSO frequency is highly system-specific, depending on both the runoff and storage/routing characteristics of the system.

In order to select a design storm that correlates with a CSO frequency, long-term records of historical CSOs, historical precipitation, and a consistent record of operation practices are required. In the case of the Sacramento system, however, operation of the CWCS has been relatively inconsistent with regard to the transferring of wastewater from Sump 1A and Sump 2 to the CWTP and/or Pioneer Reservoir. In addition, historical records of CSOs are not comprehensive enough to derive such a relationship.

With the lack of information available to adequately determine a design storm that relates to a particular CSO frequency, the system should be analyzed for the long-term record. A continuous simulation model can be used to predict the future frequency of CSOs from a sewer system using historical long-term rainfall records. It was determined by the City staff that this approach would be a refinement to the current study, and would be completed during future evaluations.

The Phase 3 study retained the design storm approach for CSO analysis with the City's assumption that the frequency of the design storm correlates with the frequency of CSOs. The use of the design storm is valuable in that it provides a relative measure by which alternatives can be compared without requiring large amounts of time and computing resources.

D.1. Frequency Selection

The use of the 1/year design storm is based on a Phase 2 technical memorandum *Survey of Cities Developing or Implementing CSO Control Plans*, dated July 1, 1991. This study reported that a number of states impose a limitation of one CSO

event per year, or control of CSOs for the 1 year storm event. One of the objectives of the Phase 3 study was to identify the costs of providing a similar level of CSO control for Sacramento.

The objective of the 5/year design storm analysis was intended to assess the CSO upgrade costs, if any, to provide a CSO control protection level similar to the existing CWCS. Based on 4 years of recent historical data the existing CWCS spills approximately 4 to 5 times per year to the Sacramento River. The underlying assumption made by the City was that the design storm frequency for the system would correspond to the CSO frequency for the existing system.

D.2. Statistical Analysis

Design storms for smaller (i.e. more frequent) 1/year and 5/year recurrence intervals cannot be derived using a annual maximum analysis. Development of design storms for these recurrence intervals required a partial duration series analysis.

The partial duration series is defined by the occurrence or frequency per year of independent events. A 1/year event defined by a partial duration series is not the same as a 1-year recurrence interval defined by a annual maximum series. A 1-year recurrence interval cannot be determined using maximum annual frequency analysis assuming a log-normal or other distribution. Probability theory has derived a relationship between partial duration series analysis and annual maximum series. This relationship is shown in Table 6. The two series, or analysis methods, become very close at recurrence intervals exceeding 10 years.

Independent storms were defined as separated by 3 consecutive hours with 0.01 inches precipitation or less per hour. (Hourly rainfall of 0.01 inches is considered trace rainfall and was therefore excluded.) This definition is appropriate, since nearly all precipitation events in Sacramento are caused by mesoscale or synoptic scale storm systems which cause well-defined precipitation periods. Convective or showery precipitation represents only a small proportion of events in Sacramento.

TABLE 6. CORRESPONDING RECURRENCE INTERVALS (YEARS) FOR ANALYTICAL METHODS	
Partial Duration Series	Annual Maximum Series
1.0	1.58
1.45	2.00
2.0	2.54
5.0	5.52
10.0	10.5

Mesoscale is a meteorologic term which implies time scales of tens of minutes and distance scales of tens to hundreds of miles. Examples of Mesoscale systems are frontal precipitation bands. Synoptic scale describes atmospheric phenomena with time scales of hours to days and distance scales of thousands of miles. Convective or showery precipitation, caused by instable air masses, consists of intermittent precipitation over a few miles and lasting less than an hour.

The 1936-1990 Sacramento hourly record was used to determine depth-frequency statistics for 1-hr, 2-hr, 3-hr, 6-hr, 12-hr and 24-hr durations, within storms defined by an interstorm period. (Long-term hourly precipitation data for 1903-1935 is not available from the NOAA.) A frequency curve for total storm depth for all durations was also developed in order to determine the appropriate total volume for typical 1/year and 5/year storms. For each duration, an annual maximum series and a partial duration series were plotted. These frequency distributions (depth versus frequency or return period) were plotted on log-log or semi-log graph paper and fit with a smooth curve which was generally close to a straight line. The plotting positions for the 2-, 5- and 10-year recurrence intervals were adjusted for consistency with the partial duration series according to Table 6. The resulting plots are presented as Figures 4 through 10.

Depth values for 1/year and 5/year frequency were then re-plotted versus duration in minutes on log-log graph paper. Values for 2-, 5- and 10-year recurrence intervals, taken from the County Hydrology Study, were also plotted for comparison as a check on the partial duration analysis. It was evident that the slope of the curves fit to both sets of values were similar, and the same slope

could be used for extrapolating the 1/year and 5/year relationships to durations shorter than one hour. Using this approach, shorter duration depths (e.g. 10-minute and 30-minute) were derived for 1/year and 5/year frequencies.

A comparison was also made between the 1903-1990 statistics and the 1936-1990 statistics. There were no significant differences in depth-duration values for the 2-year recurrence interval, therefore, no corrections were used for the differences in length of record. The comparison focused on the 2-year statistics since this frequency is closest to the frequencies of interest: 1/year and 5/year.

D.3. Storm Duration

A frequency analysis was also performed to determine the duration (in hours) of independent storm events versus recurrence interval, which is presented in Figure 11. The objective was to determine an appropriate, realistic storm length for the 1/year and 5/year storm events, based on the long-term record rather than an arbitrary selection. The length of the storm determines the volume of the storm. Volume is an important consideration in the development of the CSO control design storms, because of the effect of volume on the sizing of storage facilities to contain the runoff. The partial duration series analysis found that a 1/year event is typically 21 hours in duration and a 5/year event is typically 12 hours in duration.

D.4. Area and Other Adjustments

Design storms for the 1/year and 5/year frequencies were not adjusted for area factors. Area multipliers are not available for return periods less than (storms more frequent than) the 2-year return. In any case, the effect of area multipliers would be minimal (values of 0.98 to 0.99) considering the area of the combined sewer system and the duration of the CSO storms.

D.5. Design Storm Construction and Results

Depths for durations from 10 minutes to 24 hours, for the 1/year and 5/year

frequencies were extracted from Figure 12. The resulting values are listed in Tables 7 and 8. The design storms were then constructed using 10-minute increments, using the methods described in Subsection C.5. The resulting design storms are presented in Table 9 and in Figure 3.

TABLE 7. ONE-PER-YEAR DESIGN STORM (21 HOURS) (10-MINUTE TIME STEPS)

Time Increment	Depth (inches)	Number of Values	Intensity (in/hr)
10-min	.18	1	1.08
30-min	.30	2	.36
1-hr	.42	3	.24
2-hr	.59	6	.17
3-hr	.71	6	.12
6-hr	.99	18	.093
12-hr	1.37	36	.063
24-hr	1.90	54	.059

TABLE 8. FIVE-PER-YEAR DESIGN STORM (12 HOURS) (10-MINUTE TIME STEPS)

Time Increment	Depth (inches)	Number of Values	Intensity (in/hr)
10-min	.11	1	.66
30-min	.18	2	.21
1-hr	.26	3	.16
2-hr	.35	6	.09
3-hr	.43	6	.08
6-hr	.59	18	.053
12-hr	.82	36	.038

TABLE 9. CSO CONTROL DESIGN STORMS FOR 1/YR AND 5/YR FREQUENCIES		
Time	1/Yr Depth (inches)	5/Yr Depth (inches)
00:00	0.000	0.000
00:10	0.010	0.006
00:20	0.010	0.006
00:30	0.010	0.006
00:40	0.010	0.006
00:50	0.010	0.006
01:00	0.010	0.006
01:10	0.010	0.006
01:20	0.010	0.006
01:30	0.010	0.006
01:40	0.010	0.006
01:50	0.010	0.006
02:00	0.010	0.006
02:10	0.010	0.006
02:20	0.010	0.006
02:30	0.010	0.006
02:40	0.010	0.006
02:50	0.010	0.006
03:00	0.010	0.006
03:10	0.010	0.009
03:20	0.010	0.009
03:30	0.010	0.009
03:40	0.010	0.009
03:50	0.010	0.009
04:00	0.010	0.009
04:10	0.010	0.009
04:20	0.010	0.009
04:30	0.010	0.009

TABLE 9. CSO CONTROL DESIGN STORMS FOR 1/YR AND 5/YR FREQUENCIES		
Time	1/Yr Depth (inches)	5/Yr Depth (inches)
04:40	0.011	0.013
04:50	0.011	0.013
05:00	0.011	0.013
05:10	0.011	0.015
05:20	0.011	0.015
05:30	0.011	0.015
05:40	0.011	0.027
05:50	0.011	0.035
06:00	0.011	0.110
06:10	0.011	0.035
06:20	0.011	0.027
06:30	0.011	0.027
06:40	0.011	0.015
06:50	0.011	0.015
07.00	0.011	0.015
07:10	0.011	0.013
07:20	0.011	0.013
07:30	0.011	0.013
07:40	0.016	0.009
07:50	0.016	0.009
08.00	0.016	0.009
08:10	0.016	0.009
08:20	0.016	0.009
08:30	0.016	0.009
08:40	0.016	0.009
08:50	0.016	0.009
09.00	0.016	0.009
09:10	0.020	0.006

TABLE 9. CSO CONTROL DESIGN STORMS FOR 1/YR AND 5/YR FREQUENCIES		
Time	1/Yr Depth (inches)	5/Yr Depth (inches)
09:20	0.020	0.006
09:30	0.020	0.006
09:40	0.028	0.006
09:50	0.028	0.006
10:00	0.028	0.006
10:10	0.040	0.006
10:20	0.060	0.006
10:30	0.180	0.006
10:40	0.060	0.006
10:50	0.040	0.006
11:00	0.040	0.006
11:10	0.028	0.006
11:20	0.028	0.006
11:30	0.028	0.006
11:40	0.020	0.006
11:50	0.020	0.006
12:00	0.020	0.006
12:10	0.016	0.000
12:20	0.016	
12:30	0.016	
12:40	0.016	
12:50	0.016	
13:00	0.016	
13:10	0.016	
13:20	0.016	
13:30	0.016	
13:40	0.011	
13:50	0.011	

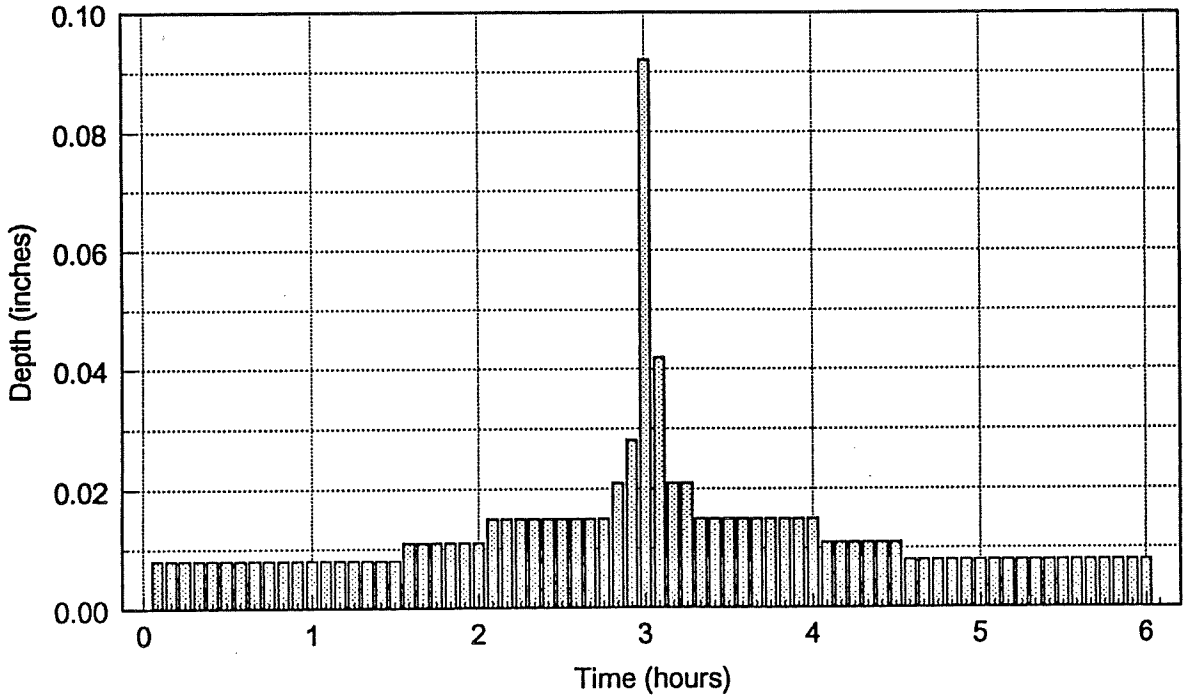
TABLE 9. CSO CONTROL DESIGN STORMS FOR 1/YR AND 5/YR FREQUENCIES		
Time	1/Yr Depth (inches)	5/Yr Depth (inches)
14:00	0.011	
14:10	0.011	
14:20	0.011	
14:30	0.011	
14:40	0.011	
14:50	0.011	
15:00	0.011	
15:10	0.011	
15:20	0.011	
15:30	0.011	
15:40	0.011	
15:50	0.011	
16:00	0.011	
16:10	0.011	
16:20	0.011	
16:30	0.011	
16:40	0.010	
16:50	0.010	
17:00	0.010	
17:10	0.010	
17:20	0.010	
17:30	0.010	
17:40	0.010	
17:50	0.010	
18:00	0.010	
18:10	0.010	
18:20	0.010	
18:30	0.010	

TABLE 9. CSO CONTROL DESIGN STORMS FOR 1/YR AND 5/YR FREQUENCIES		
Time	1/Yr Depth (inches)	5/Yr Depth (inches)
18:40	0.010	
18:50	0.010	
19:00	0.010	
19:10	0.010	
19:20	0.010	
19:30	0.010	
19:40	0.010	
19:50	0.010	
20:00	0.010	
20:10	0.010	
20:20	0.010	
20:30	0.010	
20:40	0.010	
20:50	0.010	
21:00	0.010	

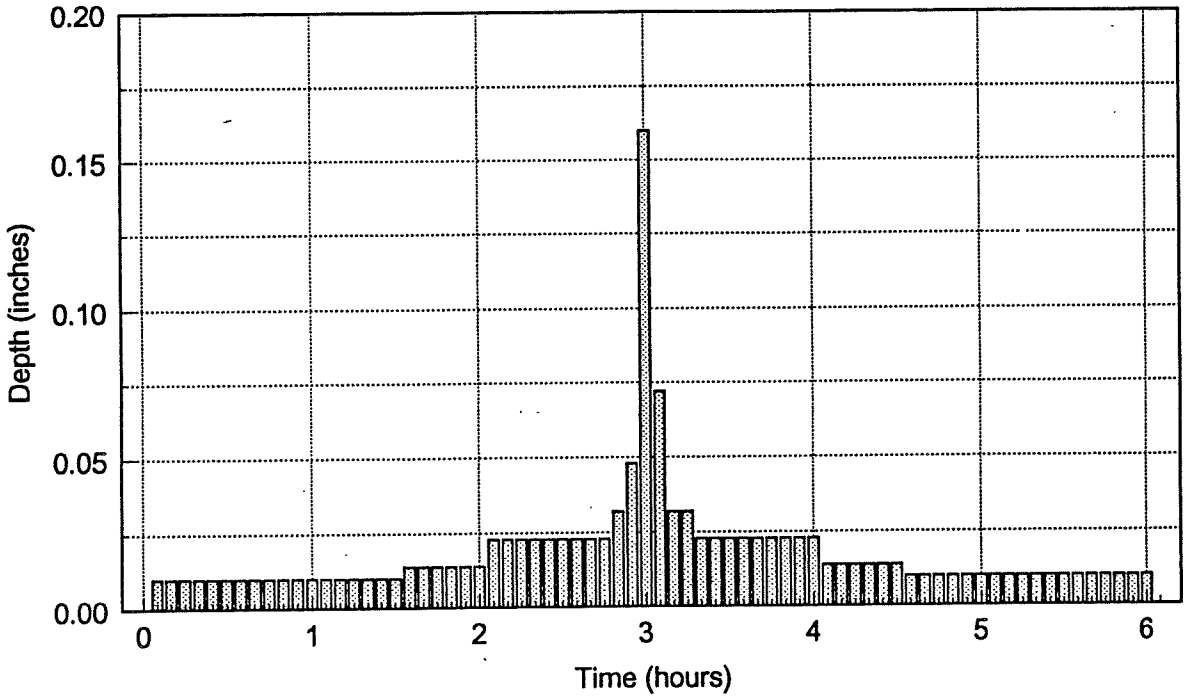
E. FLOOD VOLUMES

The four flood control design storms were imposed on the existing combined sewer system. Peak flood volumes are presented on the four 1"=500′ maps appended to this TM. Table 10 summarizes the design storm flood volumes for the existing CWCS. These values are output from the SSWMM91.

TABLE 10. DESIGN STORM FLOOD VOLUMES FOR EXISTING SYSTEM (CUBIC FEET)				
	Flood Control Design Storm			
Peak Volume	2-Year	5-Year	10-Year	25-Year
Sum of Individual Junction Peak Flood Volume	128,334	10,397,210	27,068,090	46,577,240
System-Wide Peak Flood Volume	70,076	4,478,369	9,862,243	15,893,110



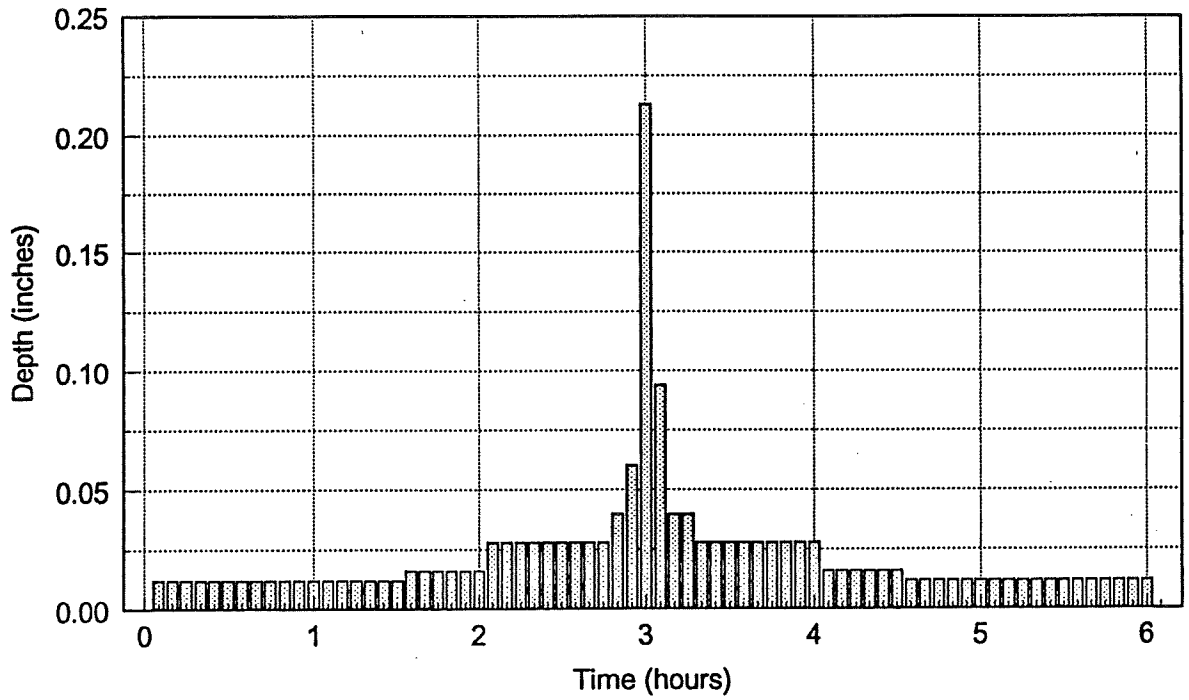
HYETOGRAPH FOR THE 2-YEAR EVENT



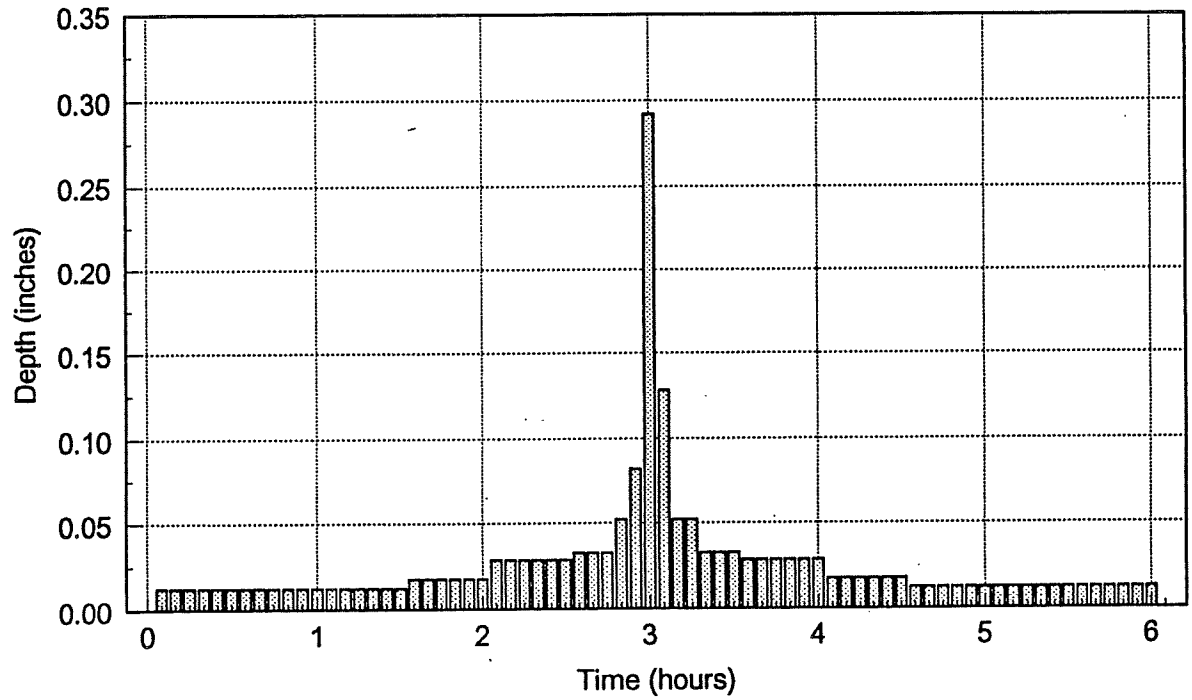
HYETOGRAPH FOR THE 5-YEAR EVENT

HDR

Figure 1



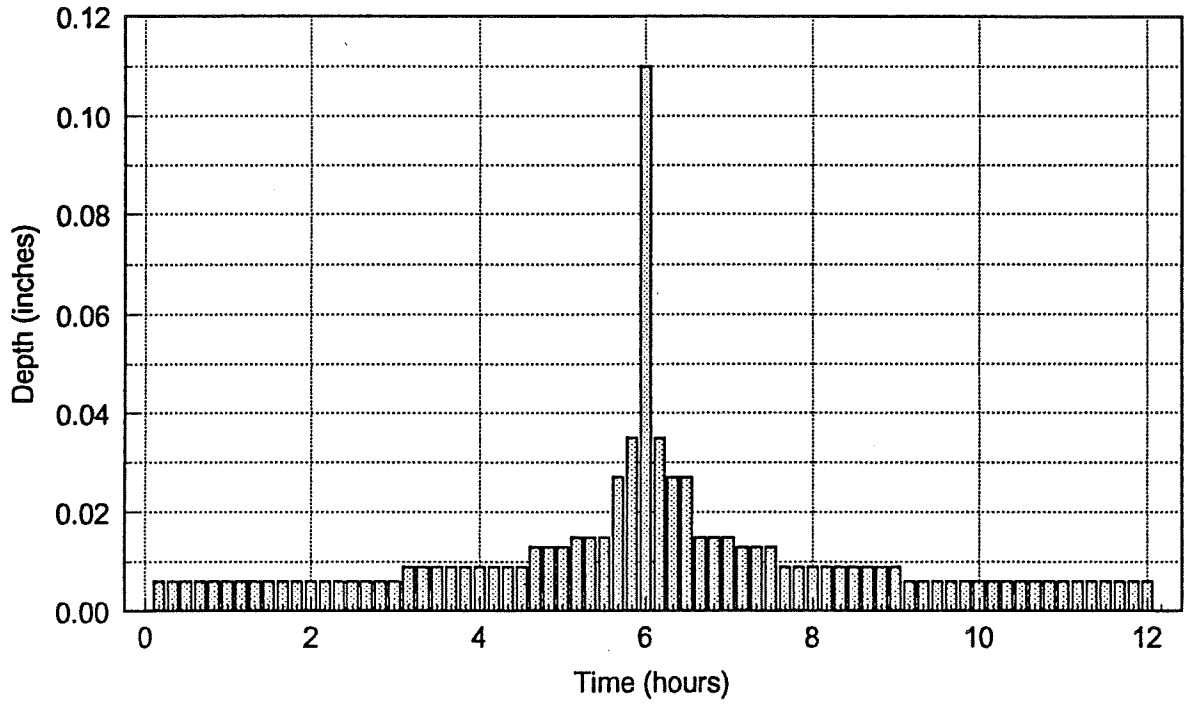
HYETOGRAPH FOR 10-YEAR EVENT



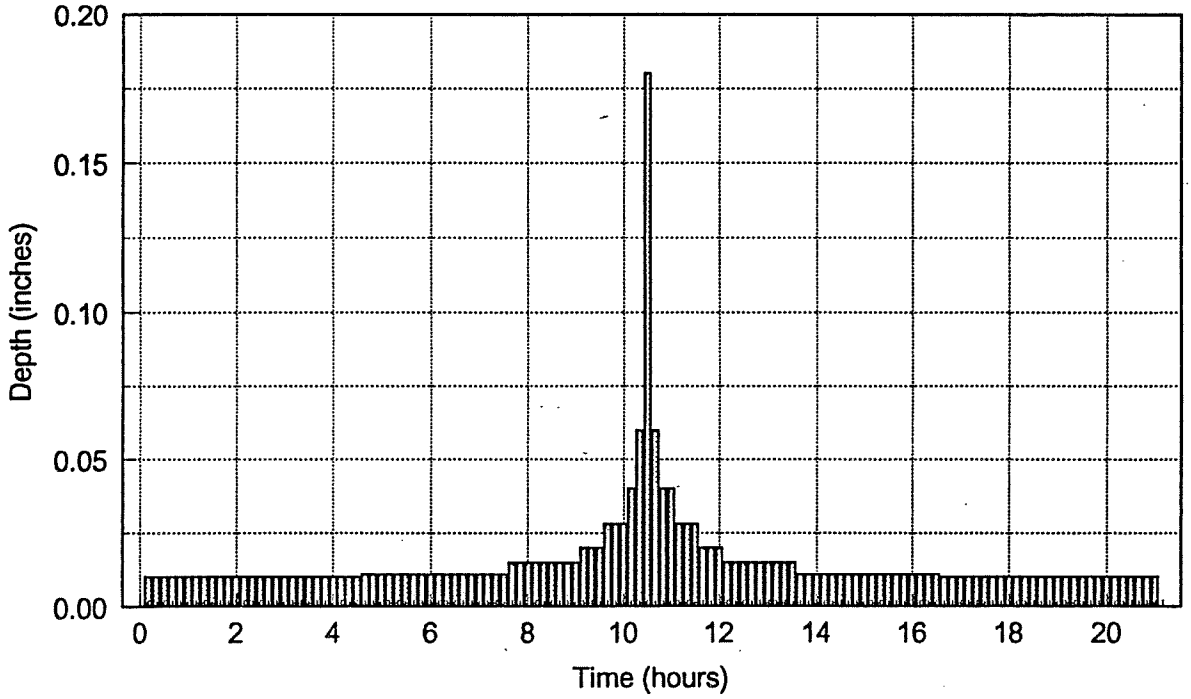
HYETOGRAPH FOR THE 25-YEAR EVENT

HDR

Figure 2



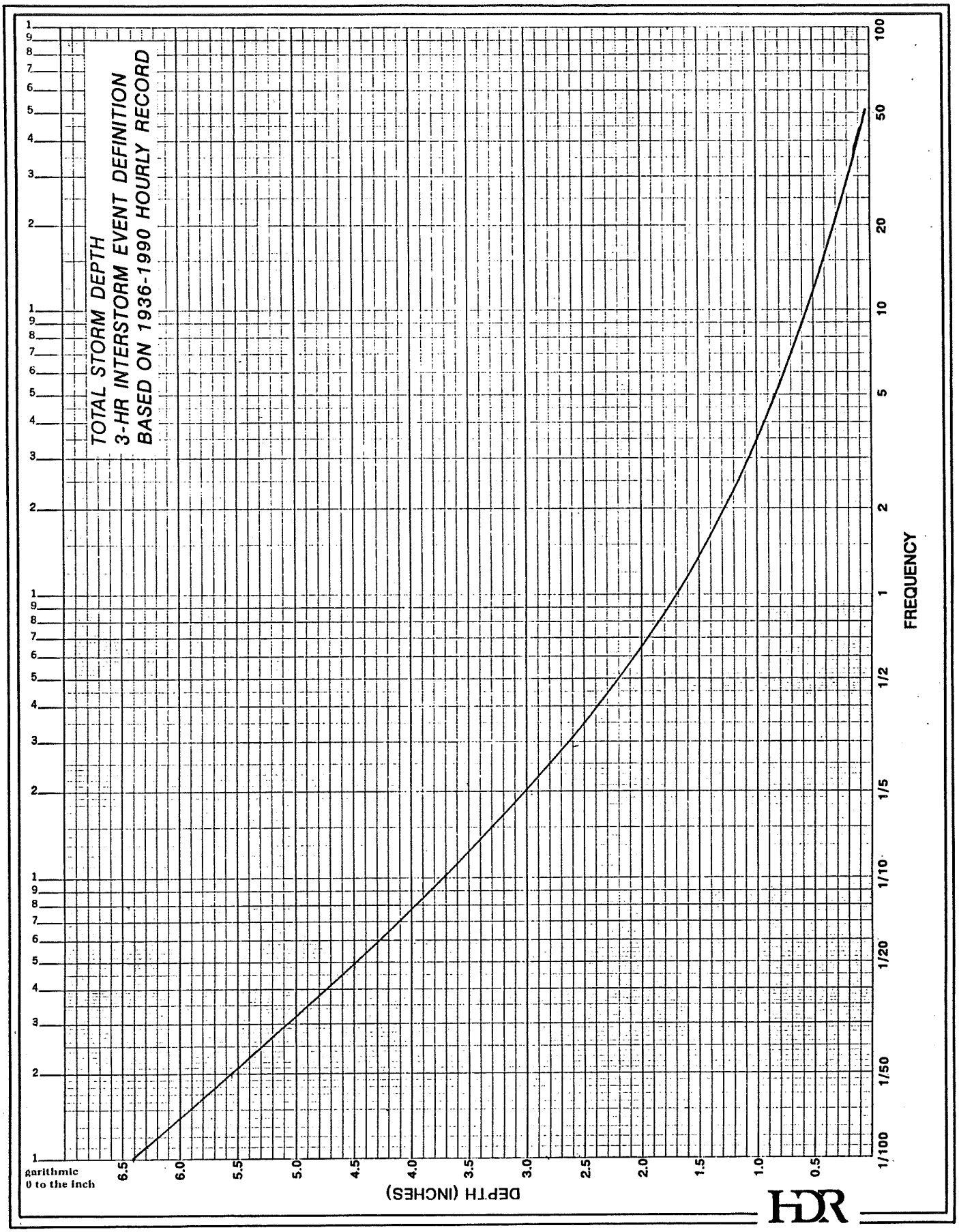
HYETOGRAPH FOR THE 5/YEAR CSO EVENT



HYETOGRAPH FOR THE 1/YEAR CSO EVENT

HDR

Figure 3


TOTAL STORM DEPTH
3-HR INTERSTORM EVENT DEFINITION
BASED ON 1936-1990 HOURLY RECORD
DEPTH (INCHES)
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
FREQUENCY
100
50
20
10
5
2
1
1/2
1/5
1/10
1/20
1/50
1/100
garithmic
0 to the inch

HDR

Figure 4

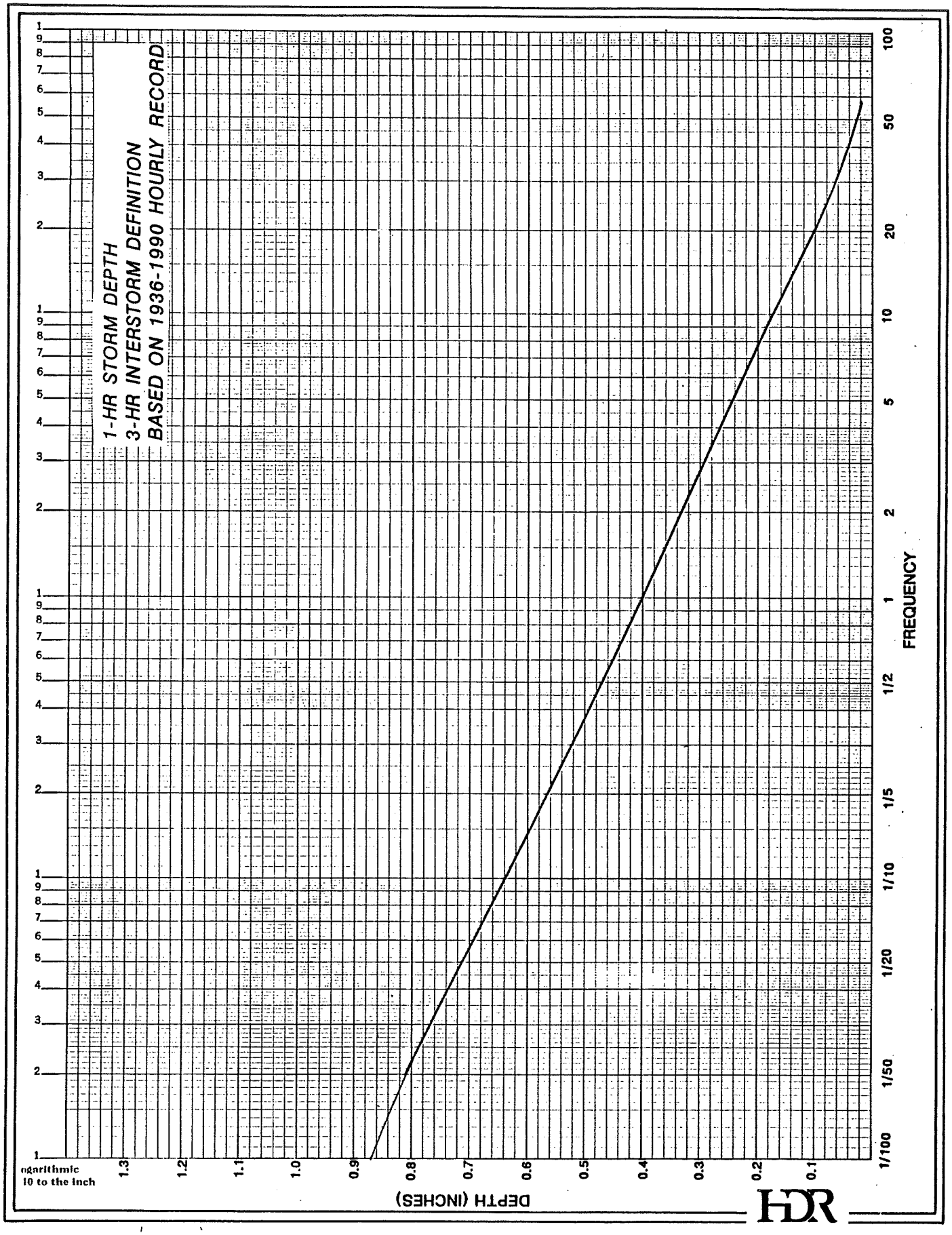
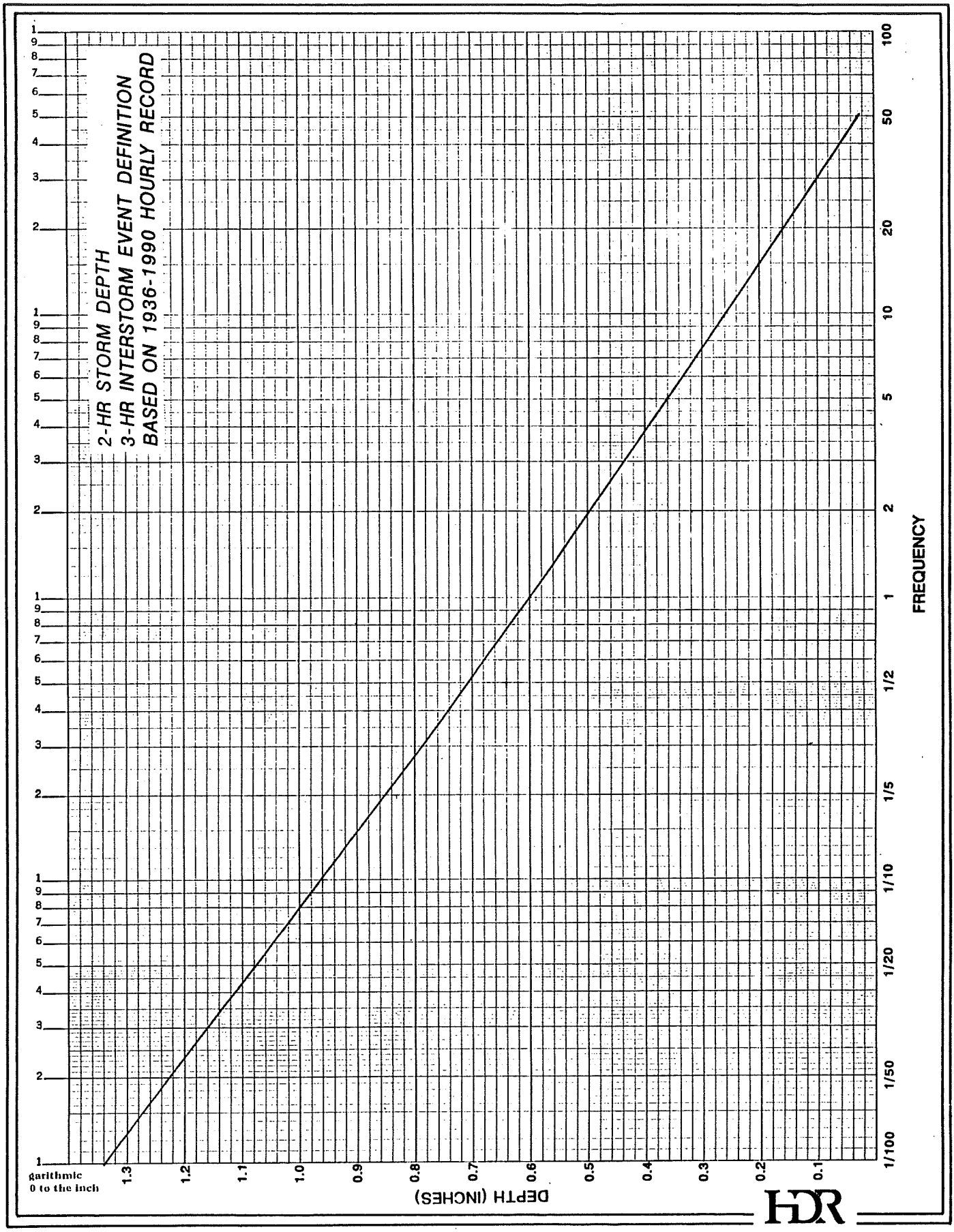
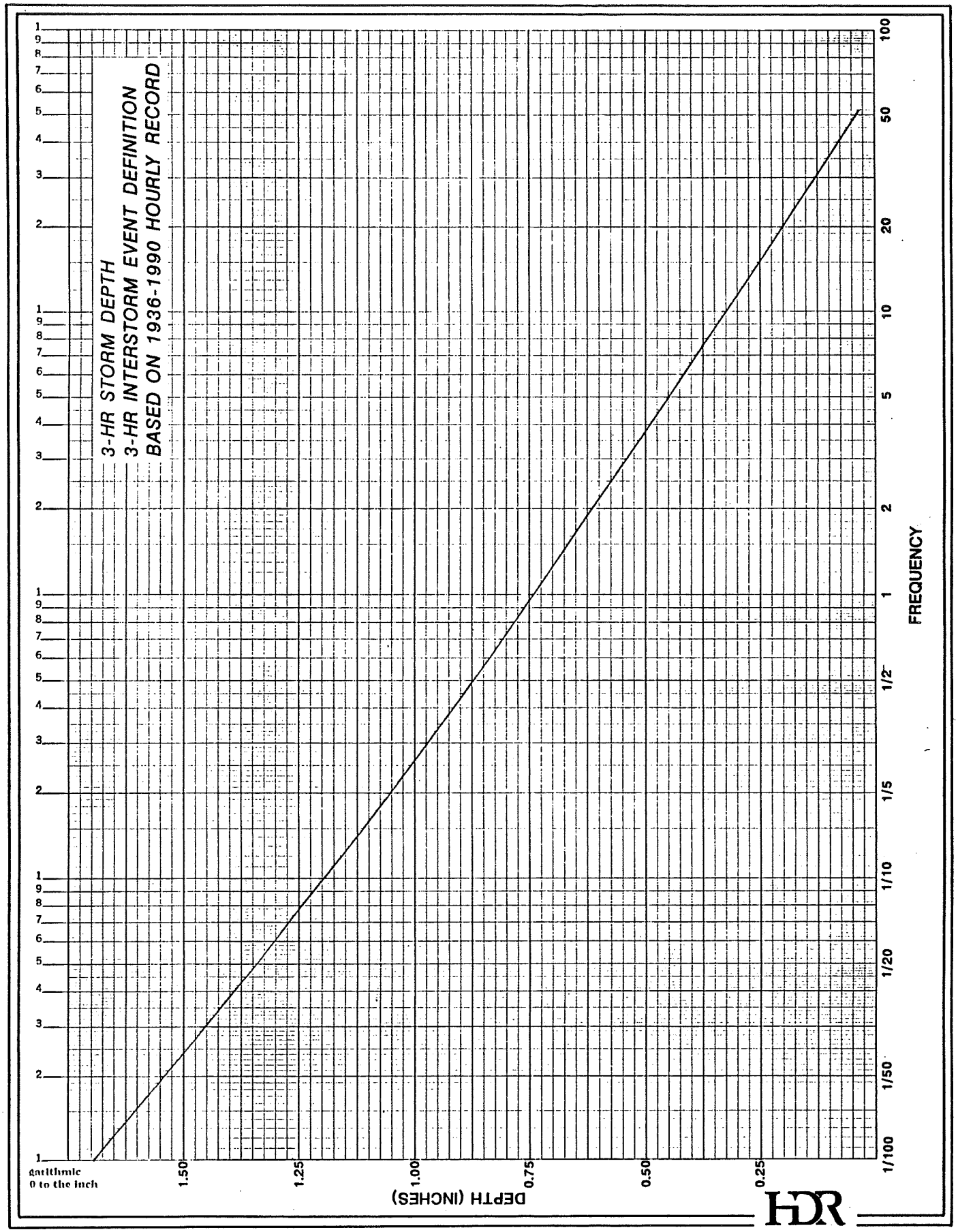

1-HR STORM DEPTH
3-HR INTERSTORM DEFINITION
BASED ON 1936-1990 HOURLY RECORD
DEPTH (INCHES)
FREQUENCY
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
100
50
20
10
5
2
1
1/2
1/5
1/10
1/20
1/50
1/100
ogarithmic
10 to the inch
HDR

Figure 5


2-HR STORM DEPTH
3-HR INTERSTORM EVENT DEFINITION
BASED ON 1936-1990 HOURLY RECORD
DEPTH (INCHES)
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
FREQUENCY
100
50
20
10
5
2
1
1/2
1/5
1/10
1/20
1/50
1/100
garithmic
0 to the inch

HDR

Figure 6


3-HR STORM DEPTH
3-HR INTERSTORM EVENT DEFINITION
BASED ON 1936-1990 HOURLY RECORD
DEPTH (INCHES)
1.50
1.25
1.00
0.75
0.50
0.25
FREQUENCY
100
50
20
10
5
2
1
1/2
1/5
1/10
1/20
1/50
1/100
garithmic
0 to the inch

HDR

Figure 7

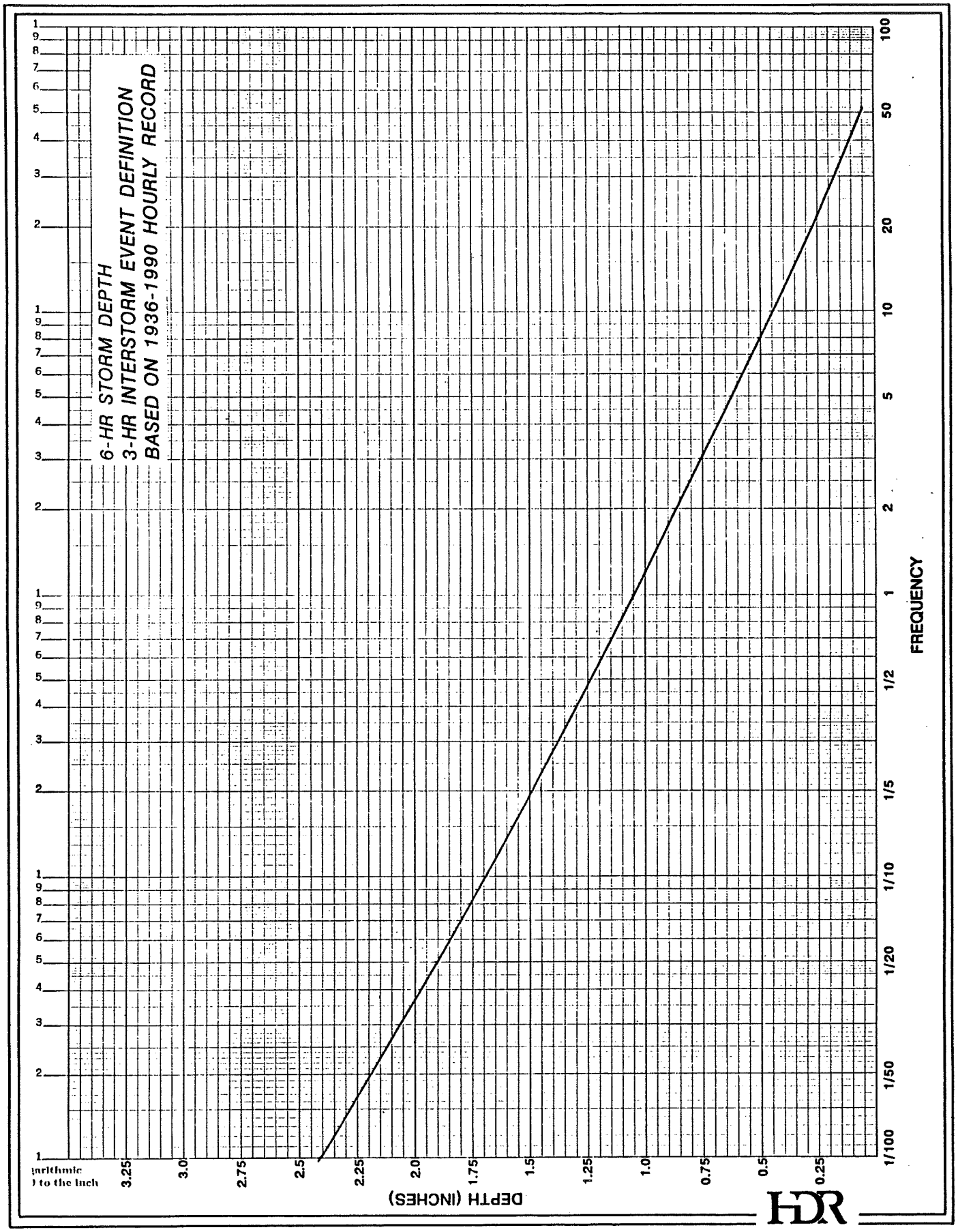

6-HR STORM DEPTH
3-HR INTERSTORM EVENT DEFINITION
BASED ON 1936-1990 HOURLY RECORD
DEPTH (INCHES)
3.25
3.0
2.75
2.5
2.25
2.0
1.75
1.5
1.25
1.0
0.75
0.5
0.25
FREQUENCY
100
50
20
10
5
2
1
1/2
1/5
1/10
1/20
1/50
1/100
arithmic
) to the inch
HDR

Figure 8

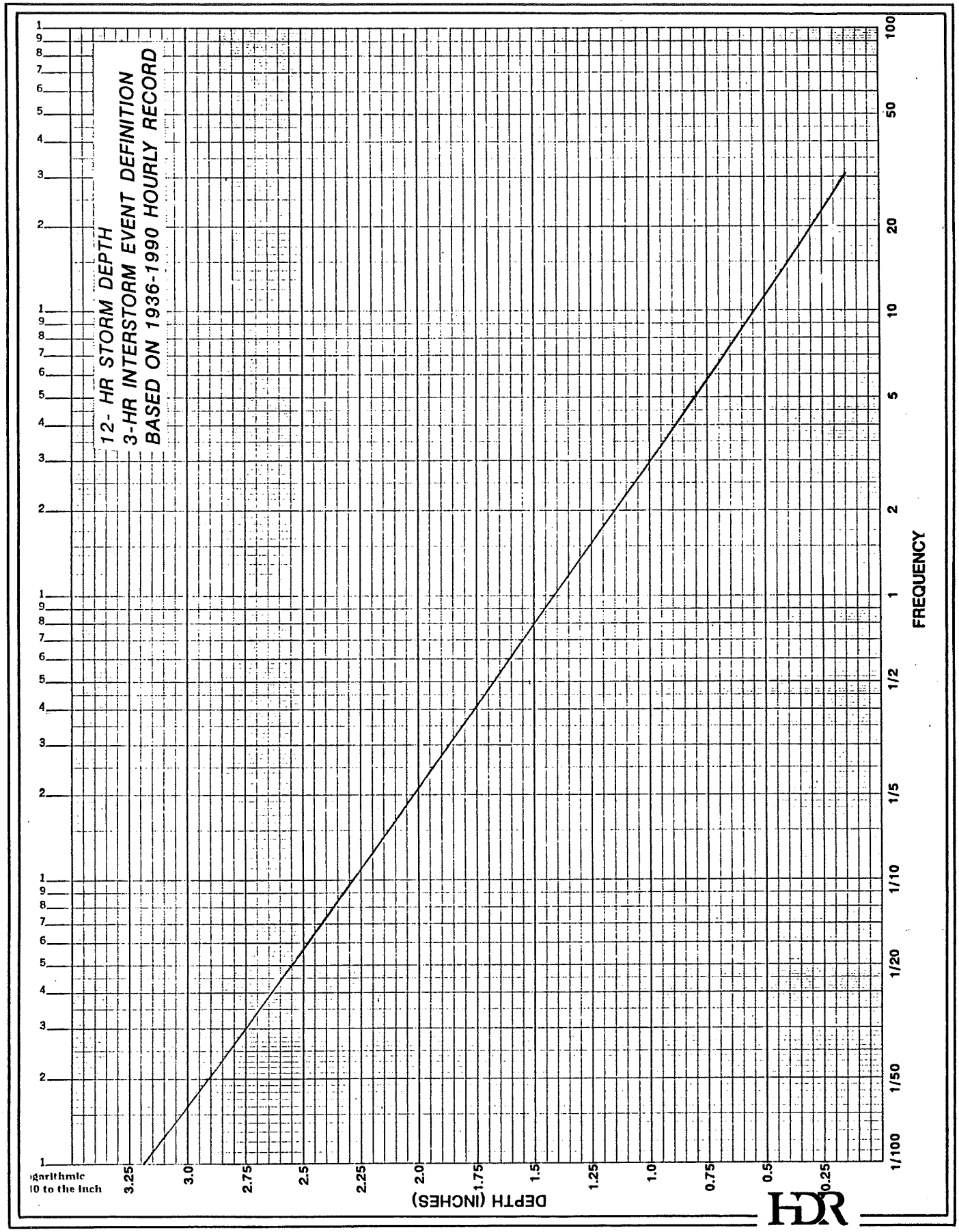
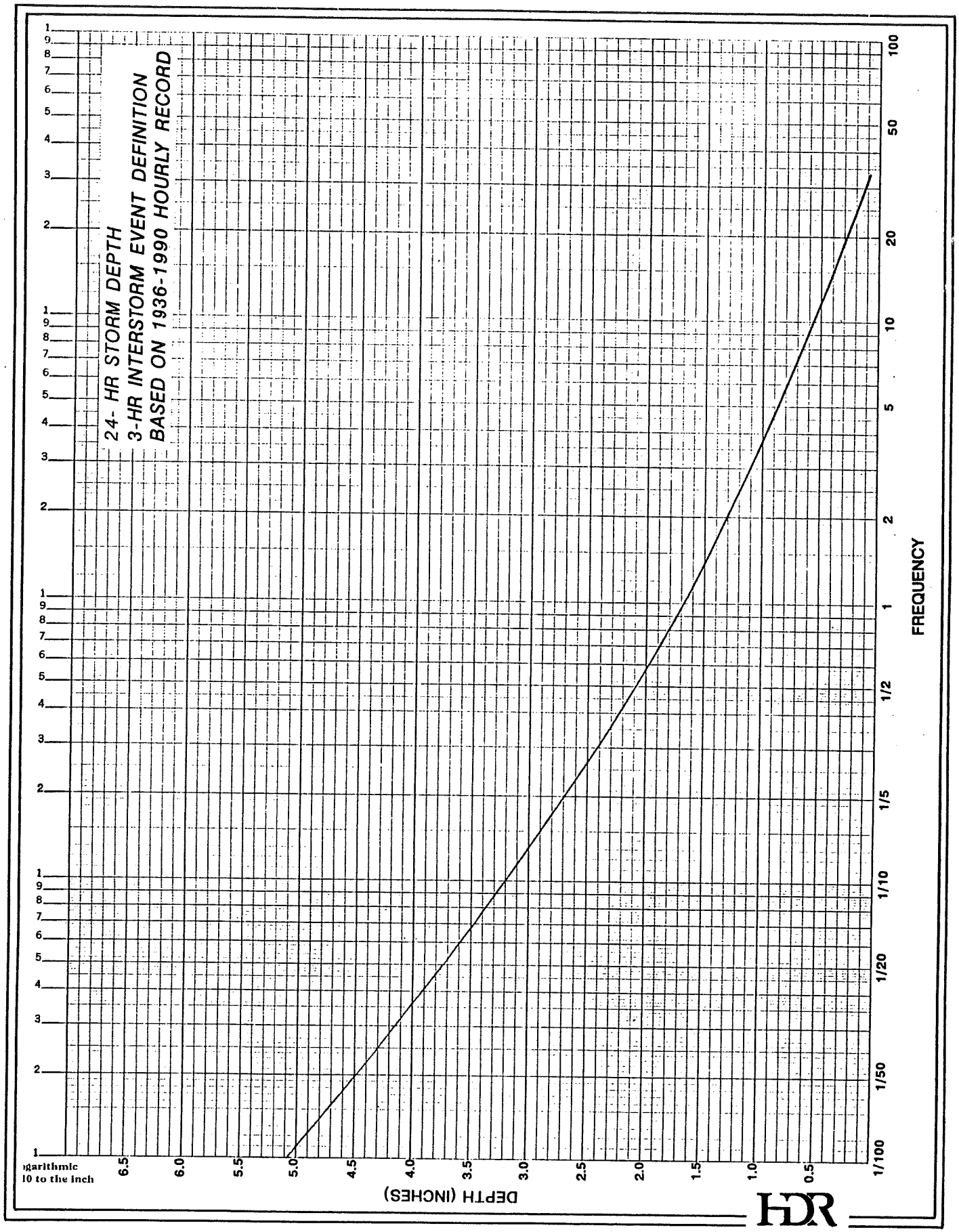

12- HR STORM DEPTH
3-HR INTERSTORM EVENT DEFINITION
BASED ON 1936-1990 HOURLY RECORD
FREQUENCY
100
50
20
10
5
2
1
1/2
1/5
1/10
1/20
1/50
1/100
DEPTH (INCHES)
3.25
3.0
2.75
2.5
2.25
2.0
1.75
1.5
1.25
1.0
0.75
0.5
0.25
garithmic
10 to the inch
HDR

Figure 9



HDR

Figure 10

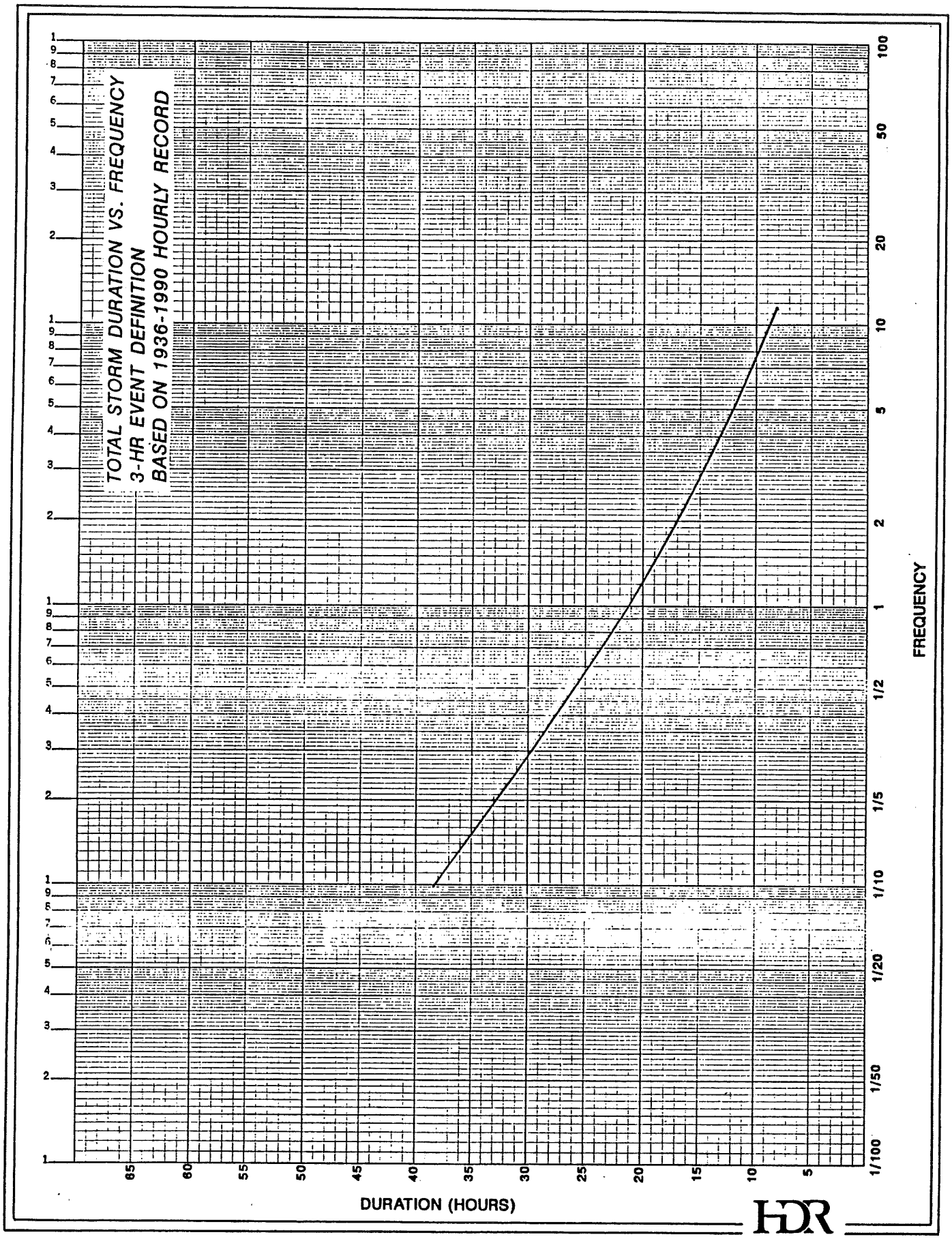
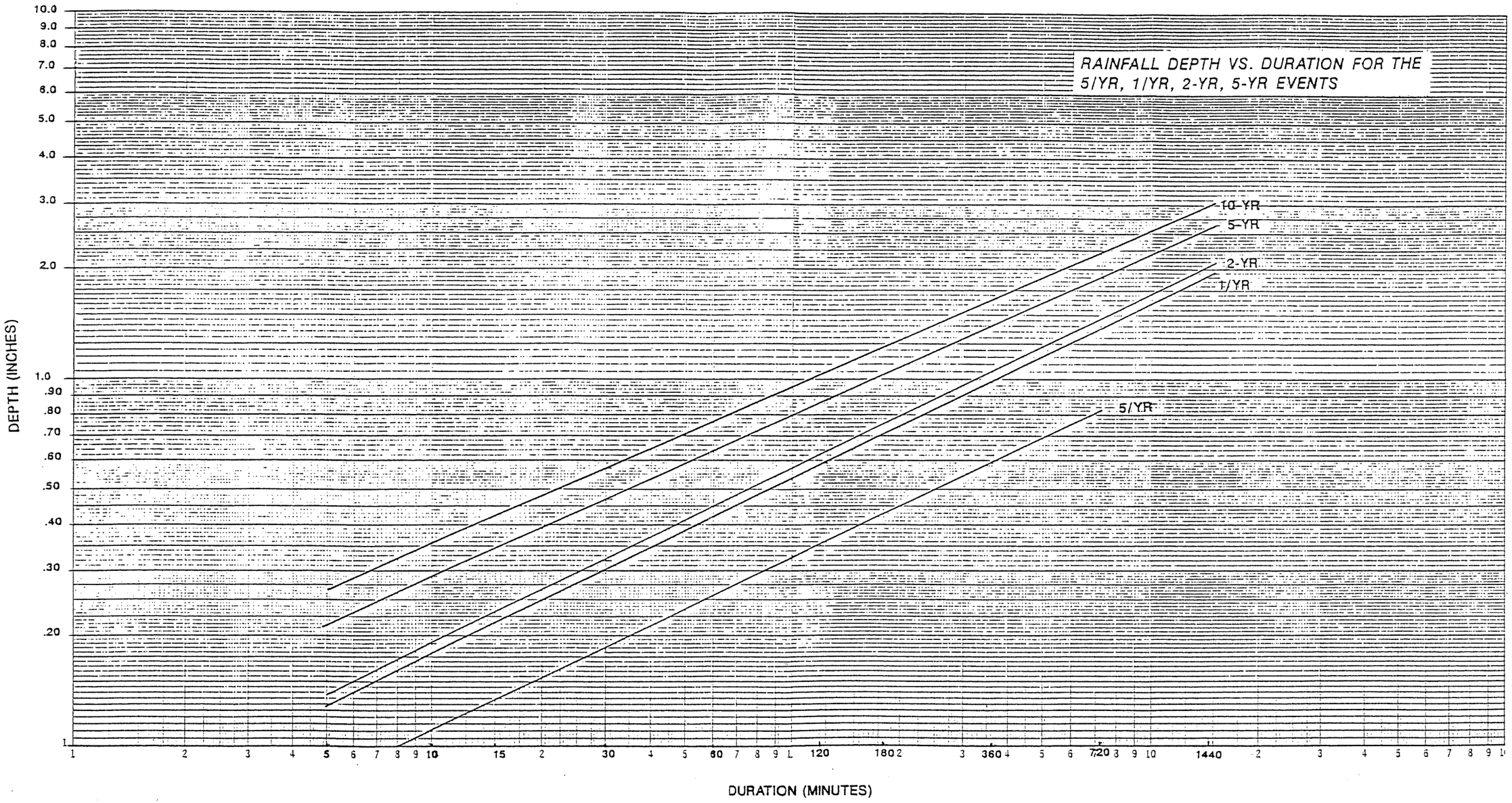

TOTAL STORM DURATION VS. FREQUENCY
3-HR EVENT DEFINITION
BASED ON 1936-1990 HOURLY RECORD
DURATION (HOURS)
FREQUENCY
65
60
55
50
45
40
35
30
25
20
15
10
5
100
50
20
10
5
2
1
1/2
1/5
1/10
1/20
1/50
1/100
HDR

Figure 11



HDR

Figure 12

APPENDIX A

TASK 1.4: ANALYSIS OF PRECIPITATION DATA

SACRAMENTO COUNTY HYDROLOGY STUDY

TECHNICAL MEMORANDUM
SACRAMENTO CITY/COUNTY DRAINAGE MANUAL
TASKS 1.4

Prepared By: Paula M. Arsenault, HDR Engineering
Jack Humphrey, HYDMET

Date: November 12, 1990

Subject: Analysis of Precipitation Data

INTRODUCTION

The objective of the precipitation analysis is to develop depth-duration-frequency (D-D-F) curves for the Sacramento County area. These curves serve as the basis from which design storms are developed, representing the "nested" or inclusive maximum intensities (or depths) for durations from 5 minutes to 24 hours. In addition to developing the actual curves, the analysis addresses the question of whether the County should be represented by a single or multiple set of depth-duration-frequency curves. That is, should design storm statistics be developed for regions in the County, to represent, for instance, elevation differences across the County. In completing this analysis, it was considered essential that the design storms analysis produce realistic results, i.e., historically and meteorologically possible events.

BASIS OF INFORMATION

Data Collection

The County of Sacramento and the National Weather Service (NWS) maintain numerous precipitation gages throughout the County. As described in Technical Memorandum 1.2, many of the County's currently-operating gages have long-term records, while several others were installed relatively recently as part of the County's ALERT system. These latter gages were generally installed since 1986 and therefore have less than four years of records. The National Weather Service maintains three gages in the area: at Metro Airport, at Executive Airport, and in downtown Sacramento.

Analysis of historical precipitation records has been performed by the Department of Water Resources (DWR) and published in Bulletin 195. The most recent published update of this document was August 1986.

For the purposes of this analysis, the County staff provided HDR with raw data and statistical analyses (D-D-F curves) which were updated by the DWR's consultant in February 1990. For reference purposes, a copy of this document is presented as Appendix B. In this case, "raw data" consists of the annual maximum depths for durations from 5 minutes to 24 hours, for the Sacramento County gages and NWS gages located within the County.

Annual maximums are available from the updated Bulletin 195 publication through the year 1989. On January 12, 1990, the area experienced an intense, cloudburst type of event. Based on the downtown NWS gage, this event had a 100-year return for a 2-hour duration. This event was assumed to represent the annual maximum depths for all durations for the 1990 calendar year.

Using the February 1990 updated DWR annual maximum precipitation data and statistics, a comparison was made between the approach used and results of the DWR analysis method and our independent evaluation. As previously described, an objective of the current study has been to validate the design storm statistics presented in the DWR publications.

Gages Selected for Analysis

Gages operating under the NWS and County ALERT systems were selected for analysis based on the length of record. Most gages within the ALERT system were eliminated from further evaluation because only 3 or 4 years of data are available. Ideally, at least 20 years of record should be available for consideration in the statistical analysis. In many cases, the period of record for a particular gage varies based on the selected duration. For instance, gage No. 270 (Elk Grove) has 27 years of record for the 30-minute duration, but only 10 years of record for the 5-minute duration. This phenomena appears to be related to the data reduction that has historically been performed on gage strip charts. In many cases, short duration data were missing for earlier years of a record, while recent years were

mostly complete.

Based on this review, nine long-term gages were selected for analysis. These are listed in Table 1.

STATISTICAL ANALYSIS

The objective of the analysis was to plot the annual maximum values for each duration on a log-normal distribution, and then fit an ordinary regression to the data. This was performed for each gage and duration. A comparison was then made between the NWS Downtown gage (No. 190) and each of the gages in the County. Since the NWS Downtown data has an extensive and complete record (88 years), it was initially estimated that this gage could reasonably represent design storm conditions for the County-wide area. The comparison would then indicate if significant regional differences in the analysis results are significant enough to warrant development of design storms for distinct areas within the County. The detailed, graphical results of the statistical analyses are presented in Appendix A.

Log-Probability Regression

Data (annual maximum depths) from the February 1990 DWR publication were entered onto spreadsheets. The annual maximum values were ranked and the probability of occurrence calculated. A log-normal plot was created by using a transformation function. Regressions were then computed, first using all of the data points and then adjusting the curves to yield a best fit for values exceeding the 2-year return. The depths with these higher return periods (lower frequency) are most important in determining extrapolated values, i.e., to the 100-year and 500-year return periods.

Table 1. Gages Selected for Statistical Analysis

Gage Name	Number	Period of Record (years)[1]
NWS Sacramento Downtown	190	88
Cresta Park	267	31
Elk Grove	270	28
NWS Metro Airport	274	28
Navion	275	22
Orangevale	276	30
Rancho Cordova	277	26
Rio Linda	278	20
NWS Sacramento Executive Airport	4507	50

[1] Including the January 1990 event. Shorter durations were generally missing periods of record, except for NWS Sacramento Downtown.

The curves for the NWS Downtown gage were checked for consistency and accuracy. Regression curves were adjusted slightly so that the relationships for each duration were parallel rather than convergent. The curves showing the poorest fit were for the 1-, 2- and 3-hour durations. Initial curve fits resulted in converging lines with many of the 2-hour values exceeding the 3-hour values. The curves were adjusted slightly to produce consistency in the curves among the longer and shorter duration.

These inconsistencies are a function of the approach used in reducing of the raw gage data. Ideally, annual maximums are all determined using the smallest increment of time measured by a gage, e.g. 1-minute of 5 minutes. Often, data reduction for the 1-hour duration and greater is performed using clock hour rather than a 60-minute duration. This produces inconsistencies between the 30-minute statistics and the longer durations.

Despite the incongruencies, the NWS Downtown gage displayed a good consistent fit to the data and between durations. Table 2 summarizes the results for the NWS Downtown gage.

Comparison to Regional Gages

The shorter duration statistics (less than 1 hour duration for all other gages generally compared poorly to the NWS Downtown gage. This was expected primarily due to the lack of long-term records for the short duration data at most stations. Curve fits for the longer durations were generally closer. In addition, most of the regional gages showed a lack of consistency in fitting the long-normal distribution and between durations.

Table 2. Depth-Duration-Frequency Data for NWS Downtown Sacramento Gage No. 190

Duration	Frequency						
	2-yr.	5-yr.	10-yr.	25-yr.	50-yr.	100-yr.	500-yr.
5 min.	0.13	0.20	0.25	0.32	0.38	0.44	0.58
10 min.	0.19	0.29	0.36	0.46	0.54	0.62	0.82
15 min.	0.23	0.35	0.43	0.55	0.64	0.73	0.96
30 min.	0.32	0.47	0.58	0.73	0.83	0.94	1.22
1 hr.	0.45	0.64	0.77	0.94	1.06	1.19	1.50
2 hrs.	0.633	0.78	0.97	1.23	1.43	1.63	2.15
3 hrs.	0.71	1.00	1.20	1.45	1.64	1.83	2.30
6 hrs.	1.05	1.43	1.68	1.99	2.22	2.45	3.00
12 hrs.	1.43	1.91	2.21	2.60	2.87	3.15	3.80
24 hrs.	2.00	2.60	2.98	3.46	3.79	4.13	4.90
2-day	2.51	3.53	4.20	5.09	5.73	6.40	7.98
3-day	2.91	4.07	4.84	5.84	6.56	7.31	9.10
5-day	3.61	4.91	5.76	6.85	7.63	8.42	10.29
10-day	4.73	6.44	7.54	8.96	9.97	11.01	13.45

Table 3. Comparison to Sacramento Gage
Percent Deviation for 100-Year Return

Duration	Cresta Park	Elk Grove F.H.	Metro Airport	Navion	Orange-vale	Rancho Cordova	Rio Linda	Executive Airport
5-min	0.0%	-19.5%	-18.4%	-18.4%	-4.5%	-6.4%	-22.7%	24.3%
10-min	20.8%	-21.8%	-10.9%	-10.2%	3.9%	6.8%	-24.8%	5.6%
15-min	8.8%	-5.9%	-8.2%	5.0%	3.1%	5.1%	0.0%	26.0%
30-min	26.7%	0.0%	0.0%	3.9%	3.3%	7.5%	0.0%	23.9%
1-hr	11.0%	0.0%	21.9%	4.5%	0.0%	9.1%	0.0%	0.0%
2-hr	6.4%	-21.2%	6.4%	19.8%	-15.6%	4.1%	-22.3%	-18.8%
3-hr	2.7%	-22.9%	9.6%	15.6%	9.1%	0.0%	0.0%	0.0%
6-hr	9.4%	-9.0%	0.0%	17.5%	-11.4%	-5.0%	0.0%	0.0%
12-hr	16.9%	0.0%	-2.8%	28.8%	3.9%	0.0%	3.1%	0.0%
24-hr	12.0%	0.0%	-9.8%	9.0%	0.9%	0.0%	-1.8%	17.0%

Table 4. Comparison to Sacramento Gage
Percent Deviation for 10-Year Return

Gage	Cresta Park	Elk Grove F.H.	Metro Airport	Navion	Orange-vale	Rancho Cordova	Rio Linda	Executive Airport
5-min	0.0%	-26.3%	-20.5%	-22.6%	-9.5%	-7.7%	-26.1%	27.6%
10-min	23.2%	-21.5%	-13.2%	-12.2%	6.8%	13.0%	-25.5%	14.4%
15-min	18.0%	-5.2%	-14.1%	6.7%	9.3%	12.4%	0.0%	28.0%
30-min	34.1%	0.0%	0.0%	11.6%	10.0%	13.4%	0.0%	25.8%
1-hr	13.0%	0.0%	10.1%	9.3%	0.0%	8.4%	0.0%	0.0%
2-hr	17.6%	-3.5%	14.5%	30.7%	3.7%	12.3%	-7.2%	-3.1%
3-hr	5.0%	-11.0%	9.3%	21.2%	3.0%	0.0%	0.0%	0.0%
6-hr	6.7%	-7.1%	0.0%	16.9%	-1.1%	-4.9%	0.0%	0.0%
12-hr	11.0%	0.0%	-4.9%	22.7%	7.1%	0.0%	3.9%	0.0%
24-hr	8.8%	0.0%	-12.2%	10.5%	2.7%	0.0%	-5.1%	4.5%

The primary reasons for such inconsistencies is related to the length of record available at the County gages. The NWS Downtown gage has a record long enough to easily extrapolate the 100-year and 500-year depths. Statistical differences which are due to regional trends are difficult to detect from the shorter period of record. Based on the graphical results displayed in Appendix A, it was concluded that design storm statistics (D-D-F curves) could be most reliably generated from the NWS Downtown gage. Further, regional differences within the County can be accounted for by adjusting these curves, based on general climatic and topographic factors.

DESIGN STORM MULTIPLIERS

Average annual precipitation and topography, along with the previously described statistical analysis, were used to develop design storm adjustment factors.

Average Annual Isohyetal Map

An updated isohyetal map was developed for examining trends within the County, which is attached. An initial map was generated using primarily the following studies:

Morrison Creek Hydrology, Corps of Engineers, March 1980
Dry Creek Hydrology, Corps of Engineers, July 1984

The isohyetes were updated using data from the 9 selected gages plus gage No. 269 (Eagles Nest), which is located in the southern part of the County.

Statistical Comparison

A comparison was made between the NWS Downtown gage and the other 8 gages at the 10-year and 100-year return periods. The objective was to determine if any regional trends could be developed from the graphical statistical analysis. Tables 3 and 4 illustrate the results. Only the Navion and Elk Grove gages were consistent in their results and corresponded with the trends indicated by the isohyetal map.

Topographic Analysis

In order to determine the effect of topography, an evaluation was made comparing elevation to rainfall depth. Essentially, the analysis used functions developed for the Placer County Stormwater Management Manual was used, along with topographic maps. In Placer County, a linear function is being used to correct baseline precipitation statistics for elevation differences across the County. In general, short duration storm depths decrease with increasing elevation, while longer duration depths increase with increasing elevation. Although the elevation differences are relatively minor across Sacramento County, elevation becomes an important factor in the northeast corner of the County, where elevation climbs. This is particularly true for durations of 12-hours and longer.

In order to account for regional and elevation differences across the County which effect design storm statistics, a dual approach is proposed. The approach combines the use of a zone map for lower elevations and shorter durations with a correction function for higher elevations and longer durations. The correction function is of the format:

Precipitation (inches) = m x Elevation (feet) + b
Where: m is the correction factor shown in Table 5
b is taken from Table 2

This function would only be used for areas exceeding 100 feet in elevation and durations of 6 hours and longer. The zone map (attached) would be used for durations less than 6 hours in areas exceeding 100 feet elevation and for durations of 1 hour and longer for areas less than 100 feet elevation. No correction would be applied to low-elevation short duration statistics. Table 6 summarizes the methods.

Table 5. Elevation Correction Factors

Duration	2-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR	1000-YEAR
1-hour	0.000016	0.000032	0.000029	0.000047	0.000063	0.000071
2-hour	0.000057	0.000095	0.000111	0.000128	0.000160	0.000189
3-hour	0.000101	0.000144	0.000175	0.000199	0.000225	0.000285
6-hour	0.000170	0.000273	0.000337	0.000365	0.000484	0.000555
12-hour	0.000321	0.000513	0.000656	0.000834	0.000883	0.001064
1-day	0.000564	0.000935	0.001183	0.001337	0.001523	0.001909
2-day	0.000981	0.001647	0.001989	0.002310	0.002614	0.003247
3-day	0.001474	0.002550	0.002797	0.003248	0.003785	0.004585
5-day	0.002020	0.003384	0.003720	0.004269	0.004722	0.005950
10-day	0.002567	0.004236	0.004722	0.005238	0.005871	0.007362

Table 6. Design Storm Correction Methods

Elevation	Storm Duration		
	<1 Hour	1 Hour - 6 Hour	6 Hour - 10 Day
<100′	None	Zone Map	Zone Map
>100′	Zone Map	Function	Function

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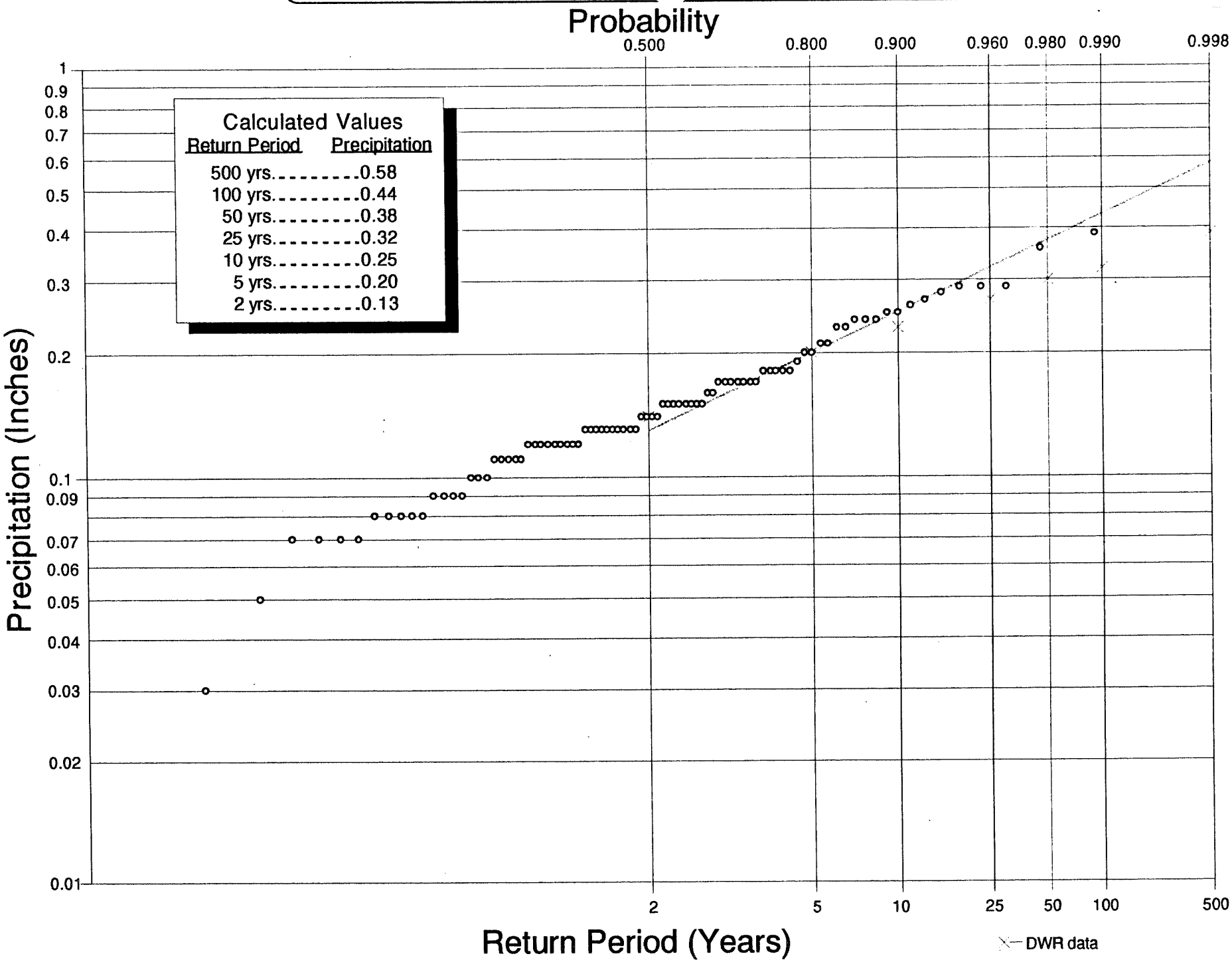

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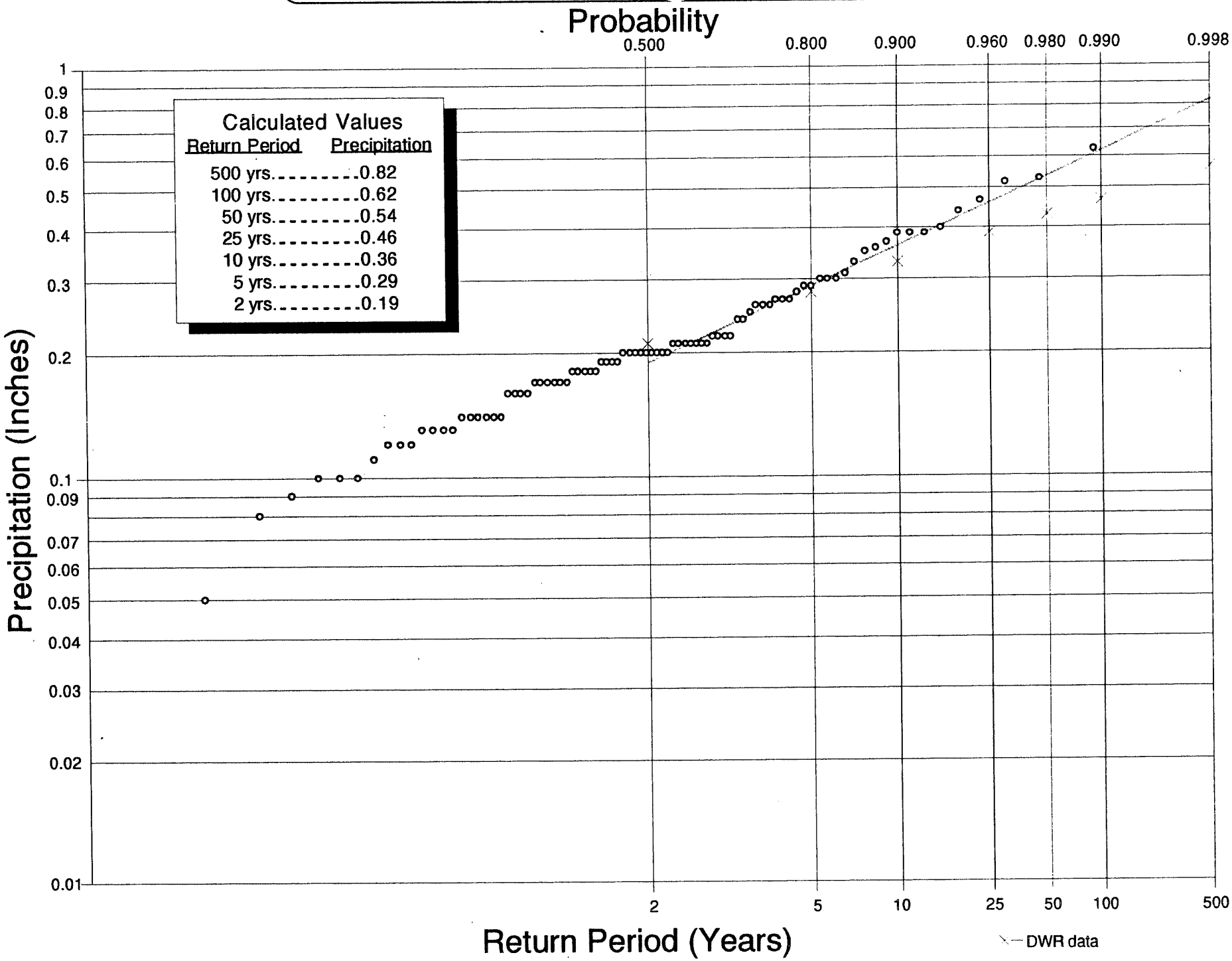

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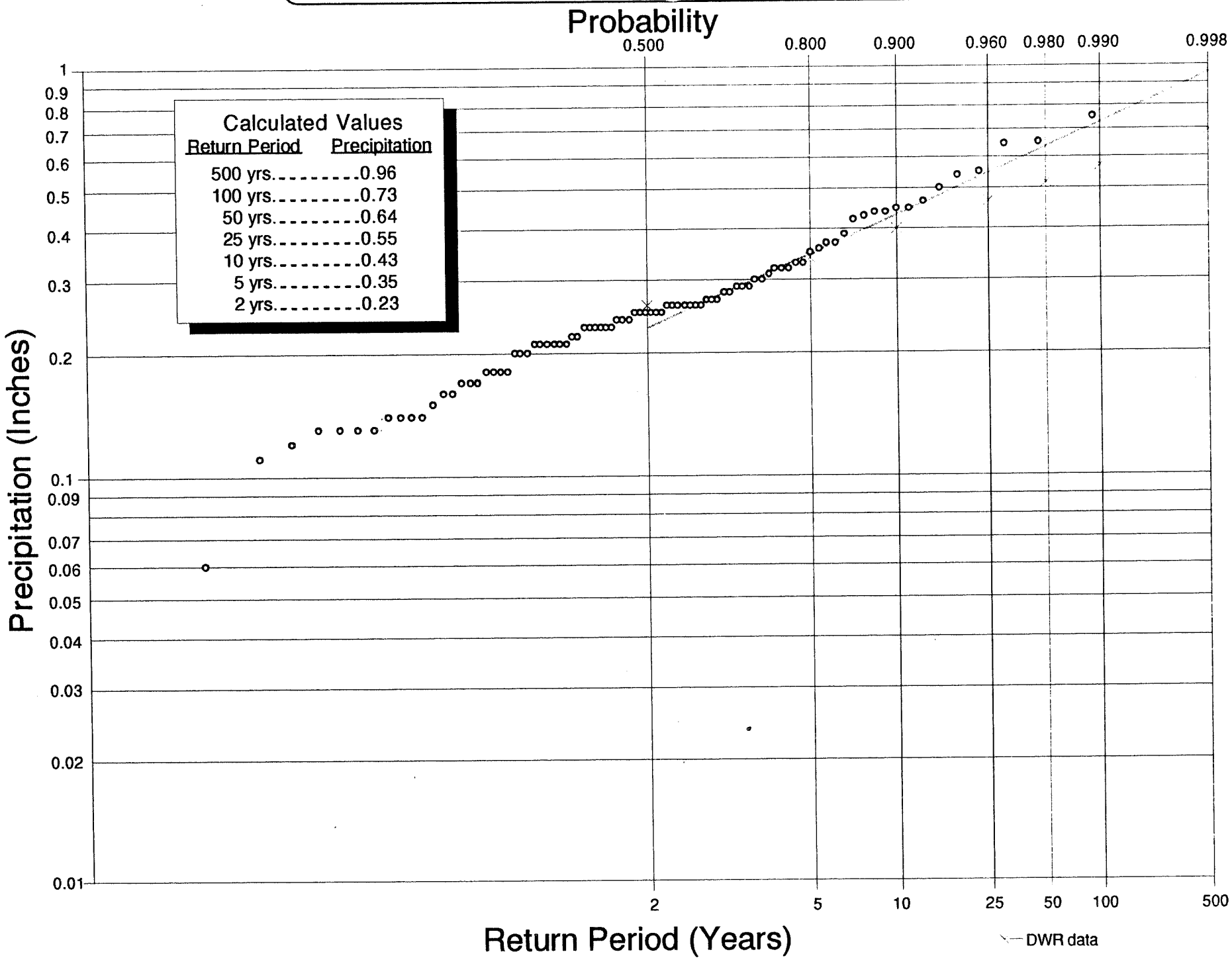

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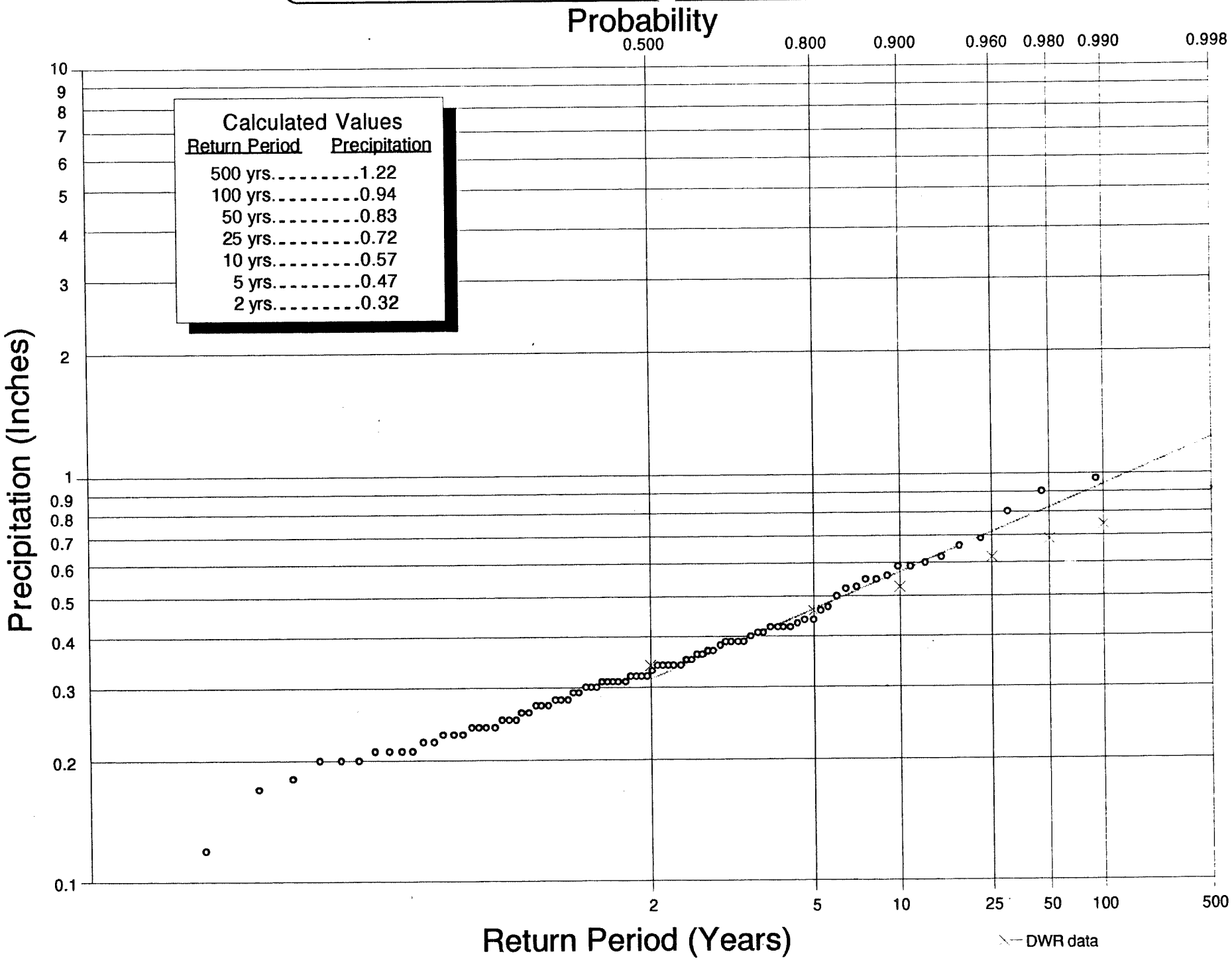

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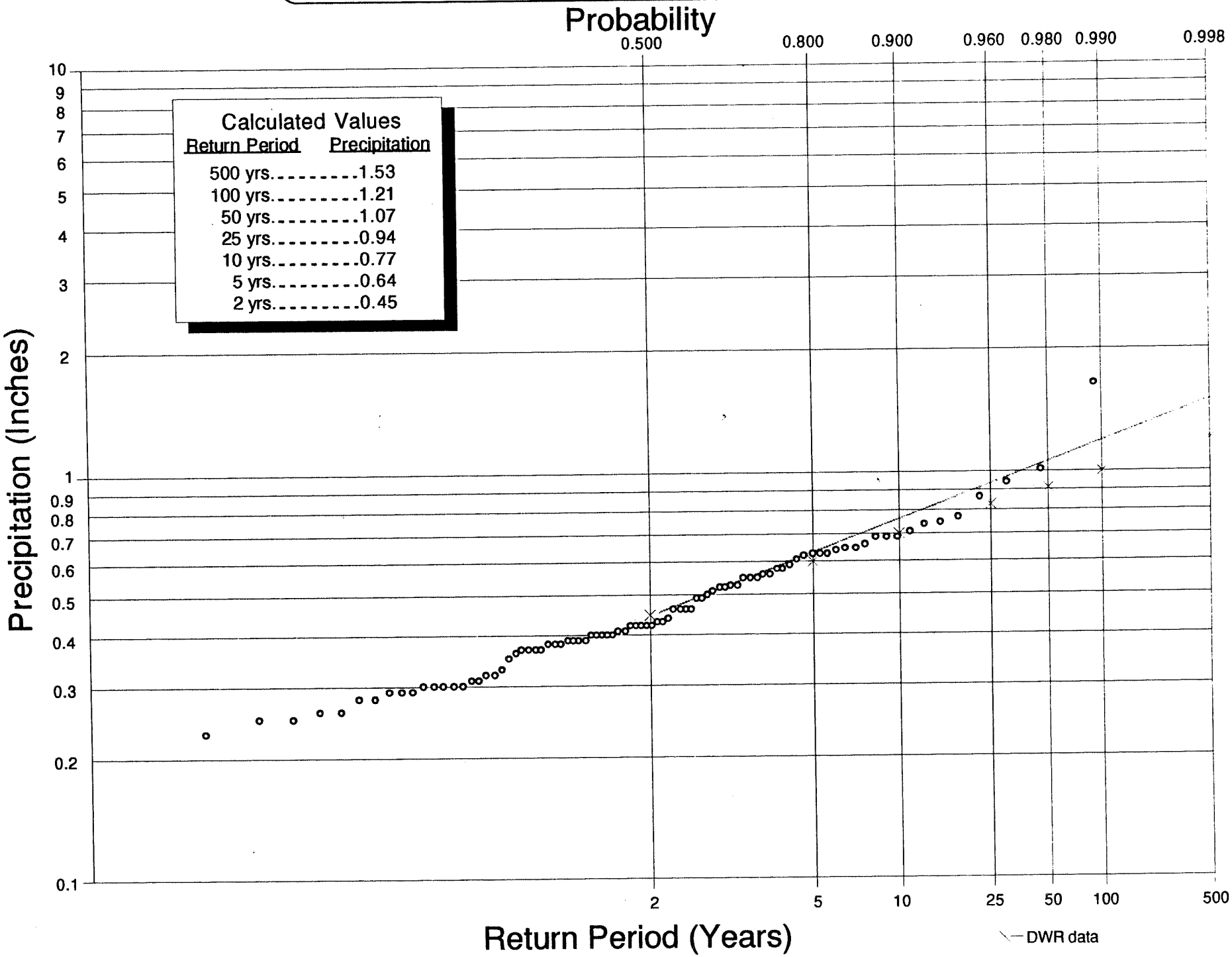

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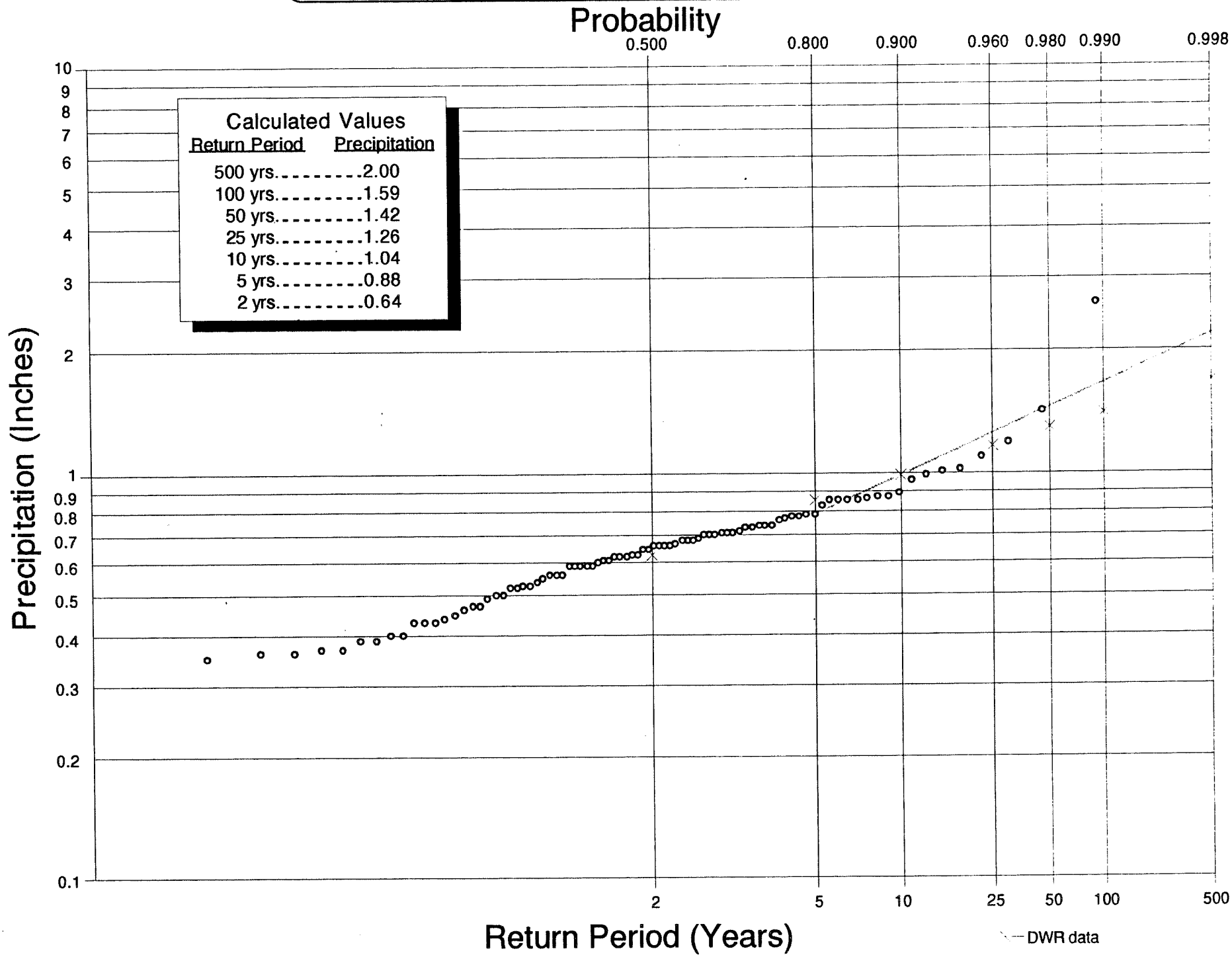

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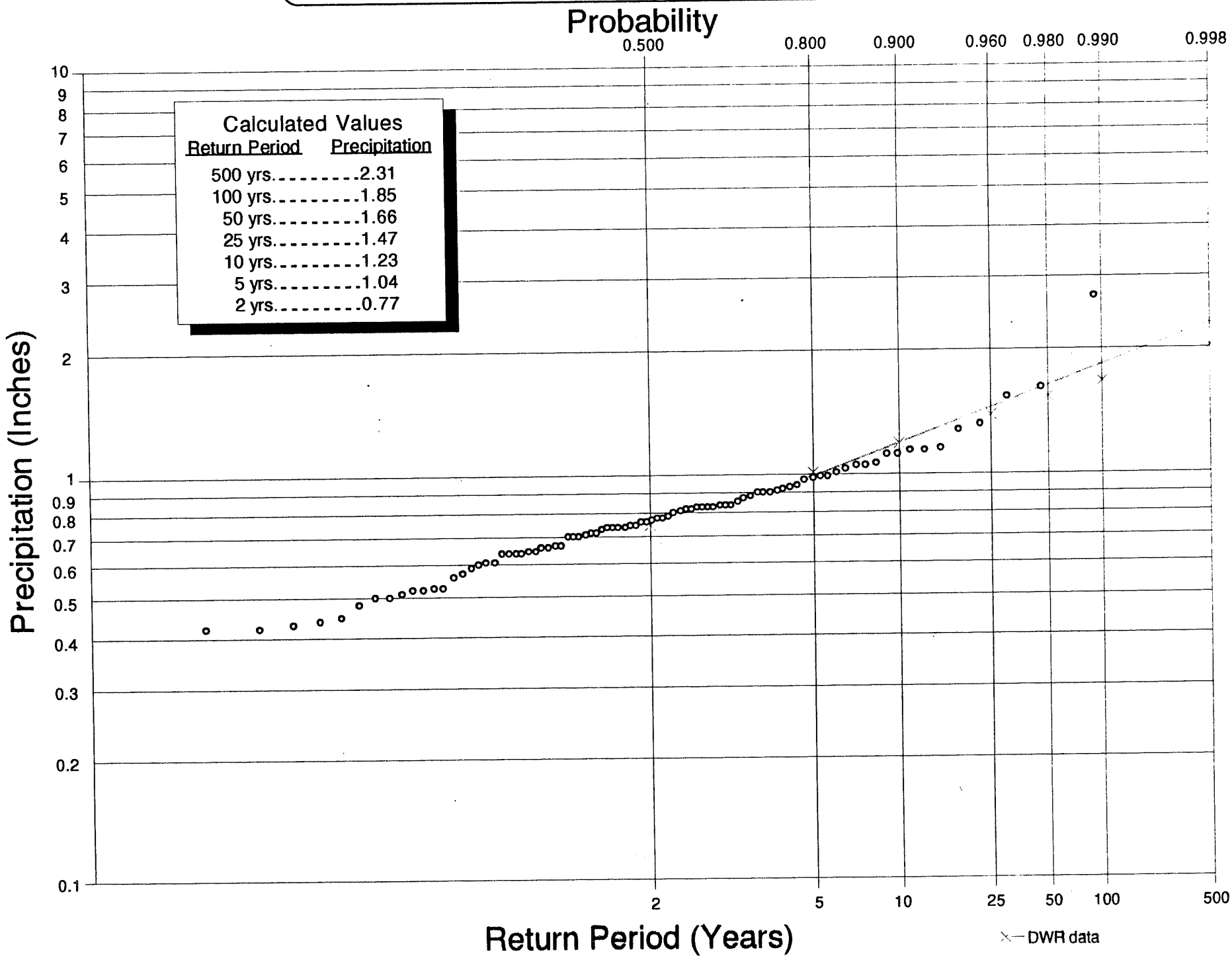

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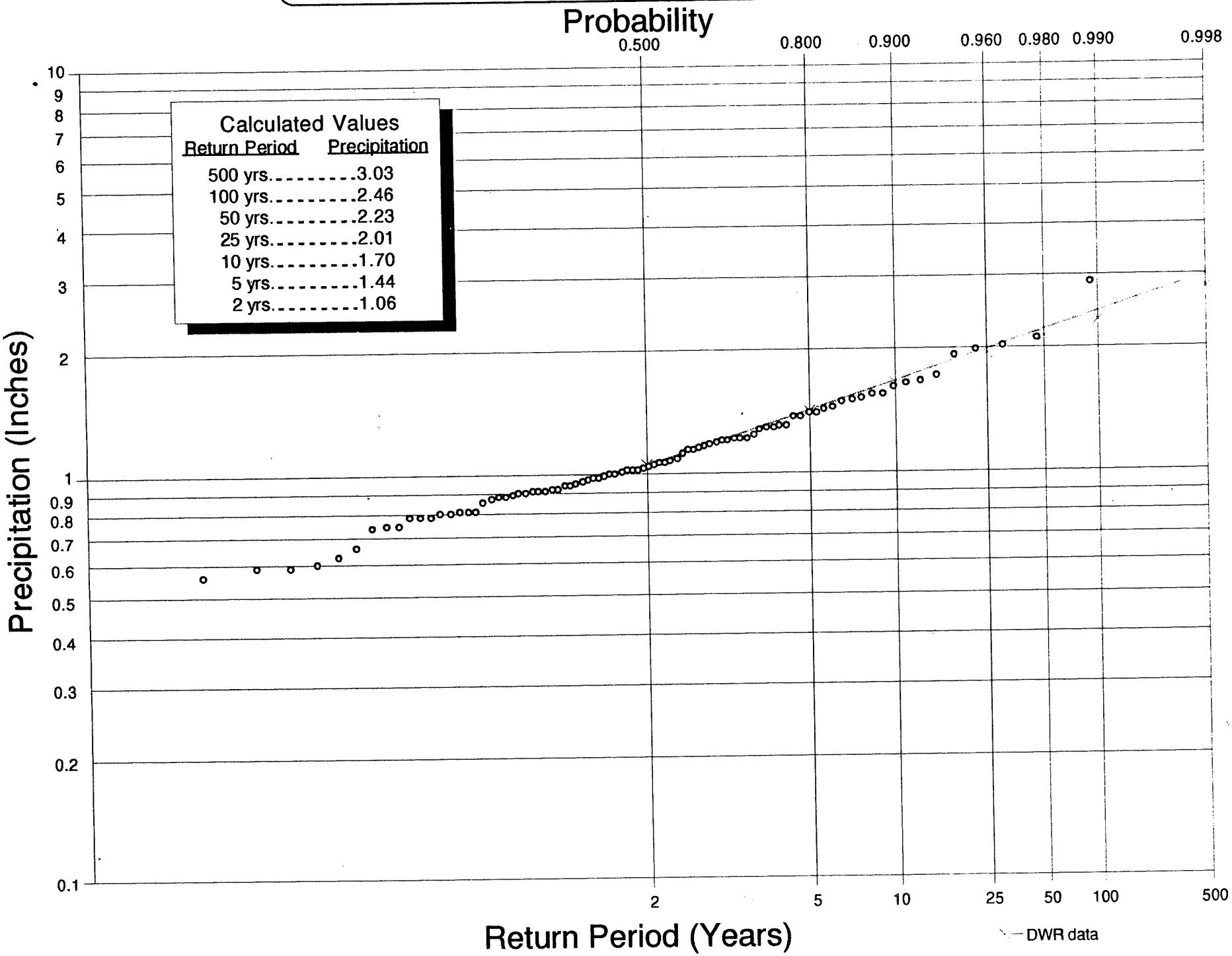

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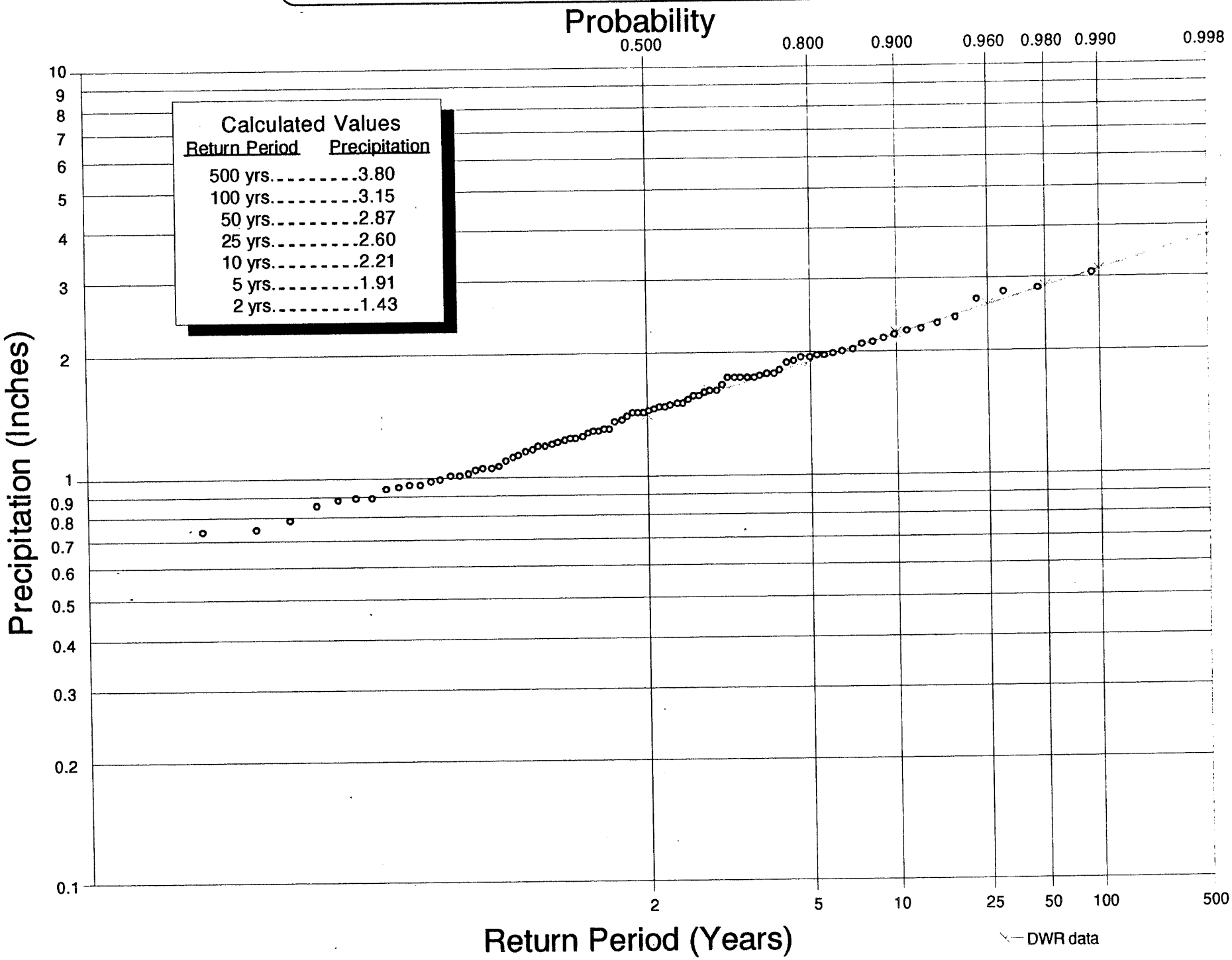

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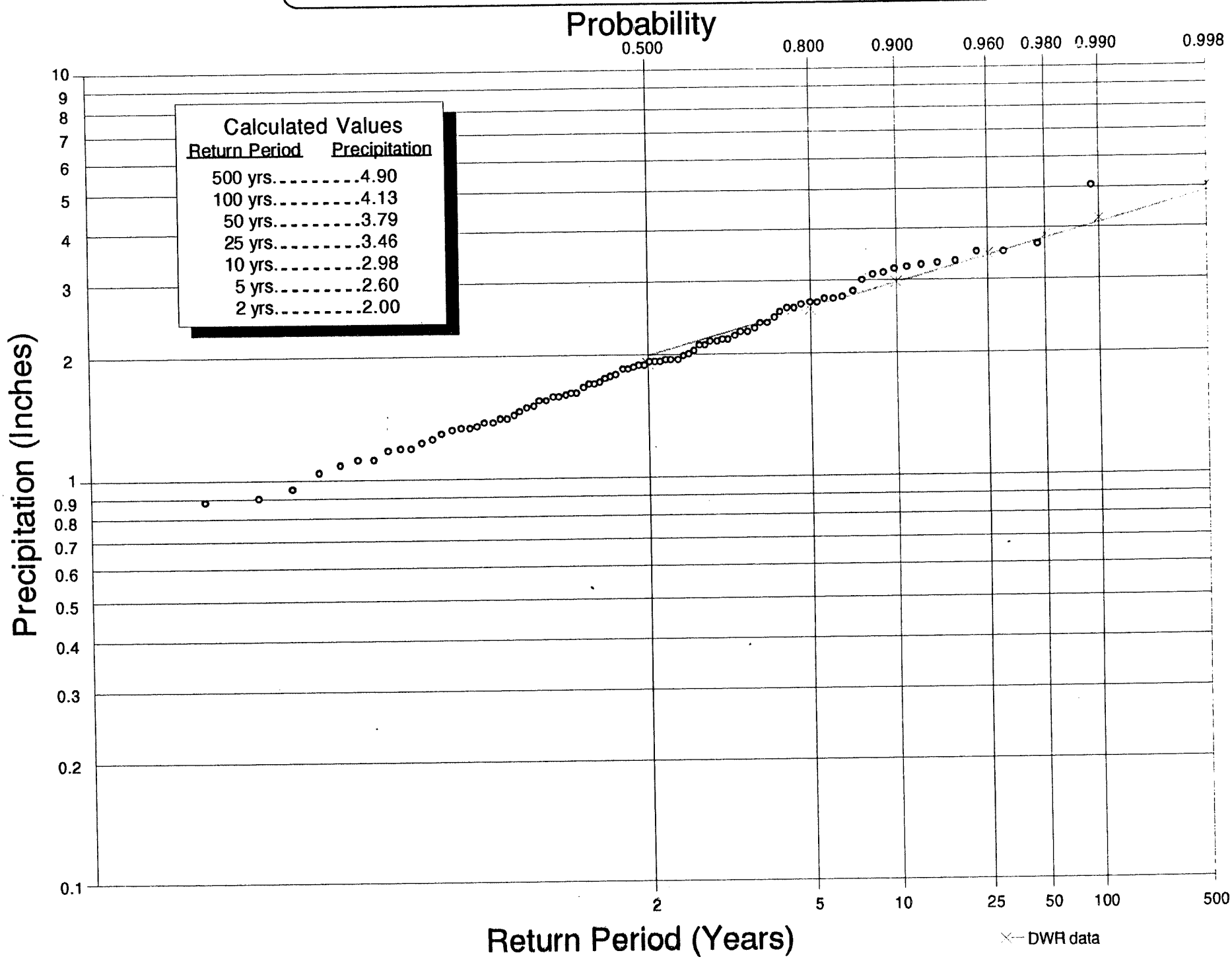

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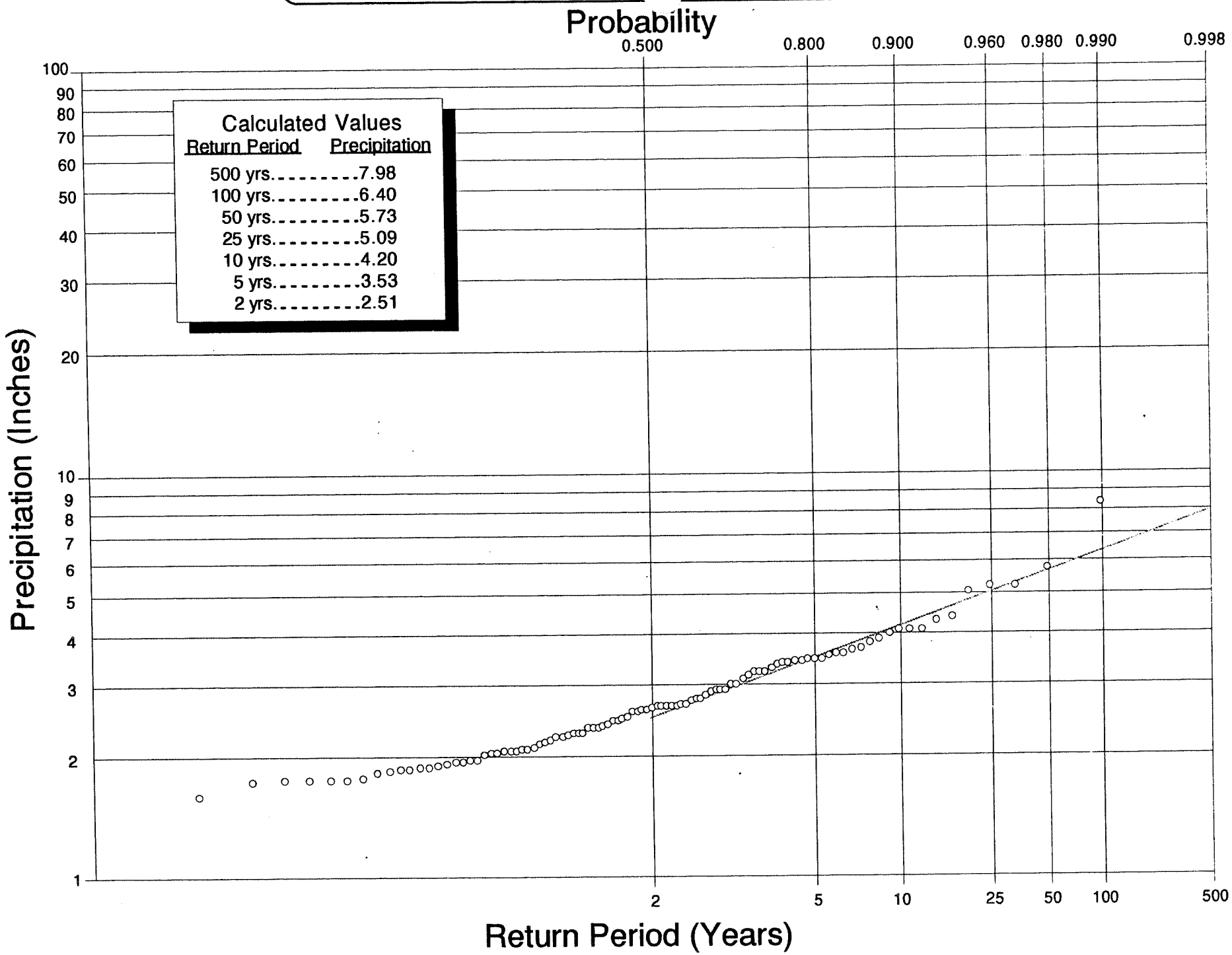

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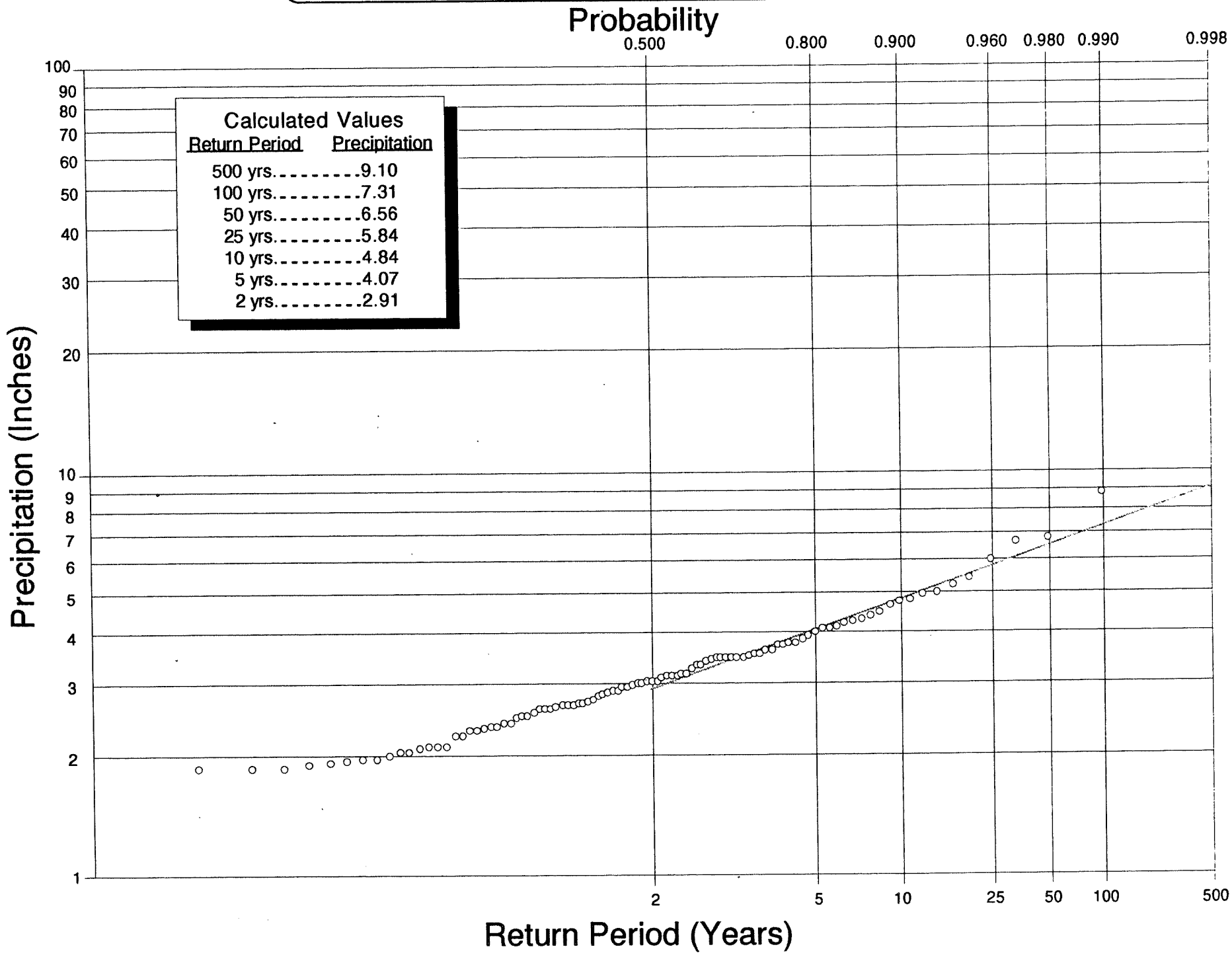

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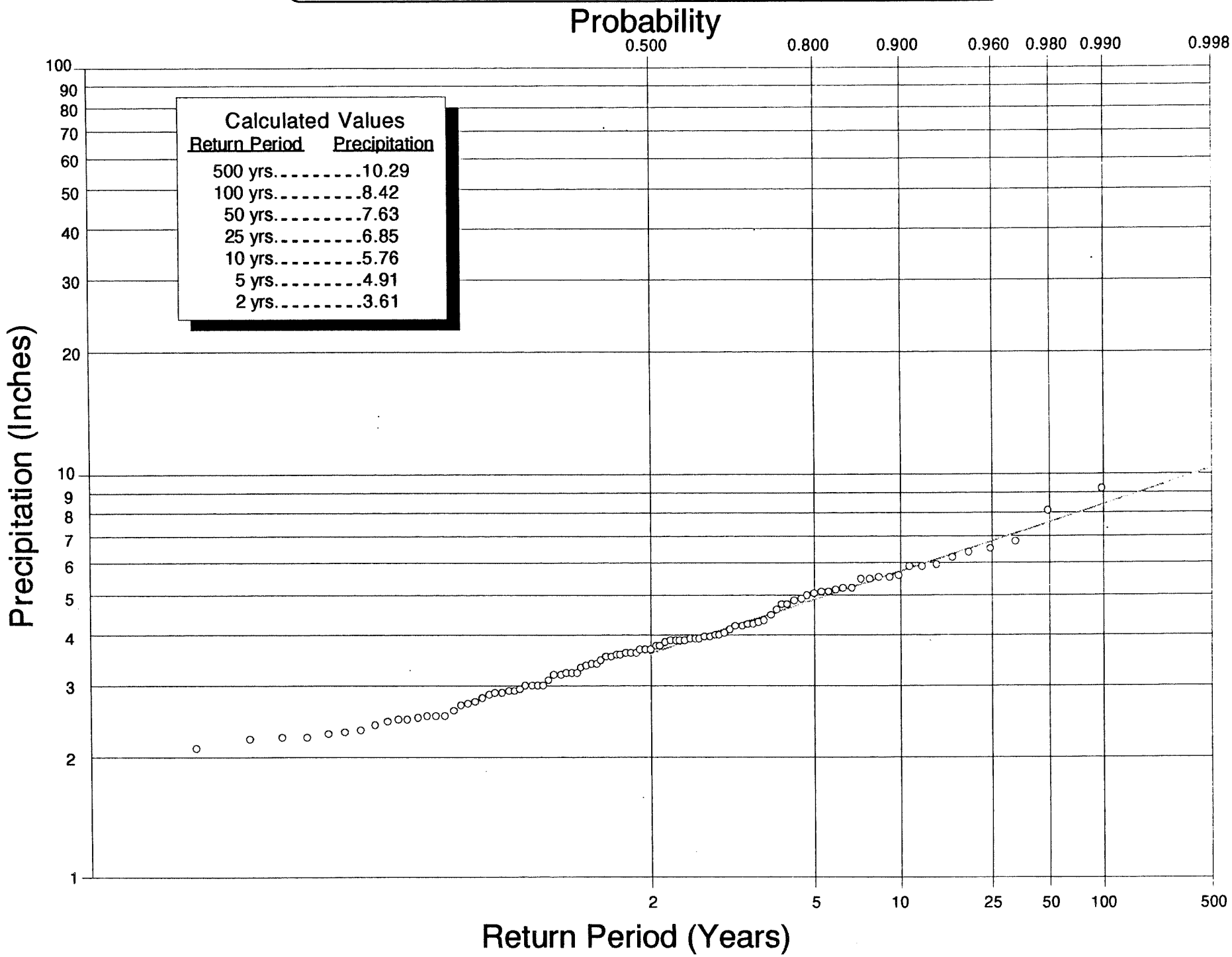

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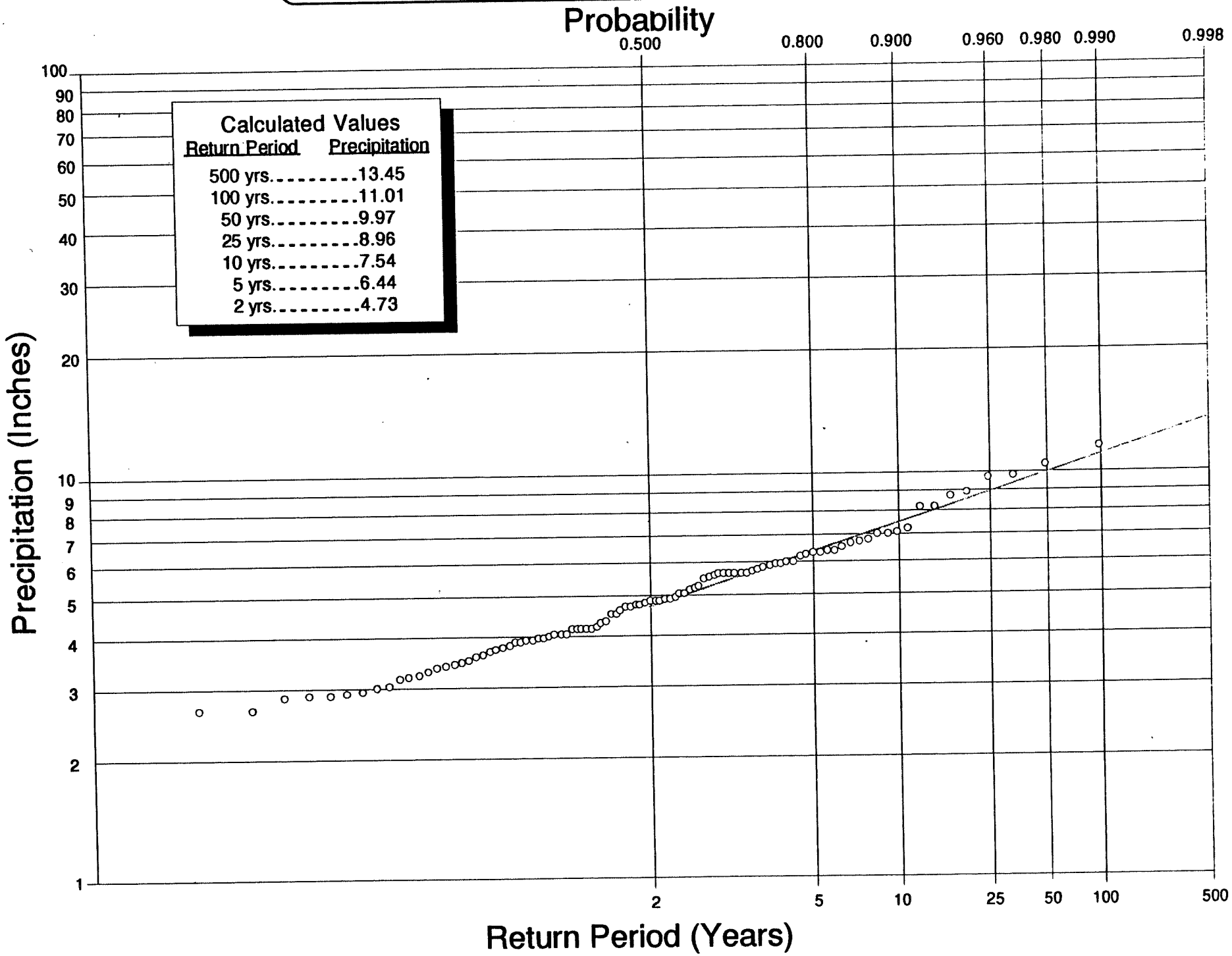

APPENDIX I

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM
IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 3

APPROACH TO LONG DURATION STORM DEVELOPMENT AND USE FOR THE PHASE 3 ANALYSIS

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TECHNICAL MEMORANDUM
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN
PHASE 3, TASK 3

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SUBJECT: Approach to Long-Duration Design Storm Development and Use for the Phase 3 Analysis

A. INTRODUCTION

This memorandum serves as a supplement to the technical memorandum (TM) entitled *Design Storms, Rainfall Analysis and Flood Volume Maps for the Phase 3 Analysis*. The latter document described the development of synthetic design storms used to determine the performance of the Phase 3 improvement plans in terms of flood and CSO control. Synthetic design storms are a hypothetical distribution of rainfall depths or intensities over a time increment for a given storm duration and frequency.

For the Phase 3 study, it was intended that two sets of design storms be used to analyze alternative improvement plans. The first involved design storms for flood/outflow control project sizing. Design storms for flood control are six hours in duration with return periods of 2, 5, 10 and 25 years. The second involved design storms for sizing CSO control facilities, such as new storage facilities and treatment plants. Design storms with frequencies of 1/year and 5/year were used for CSO control project sizing.

The use of design storms in designing conveyance and storage facilities is widespread. Design storms are valuable in that they provide a relative measure by which several alternatives can be compared. The use of design storms, however, does require some assumptions that may not apply in all cases.

One of these assumptions is that a given frequency of precipitation will produce runoff of the same frequency. This assumption may not always be true due to varying conditions in the drainage basin. For example, the same precipitation may

be applied to two basins and the timing and volume of the runoff may differ as a result of the initial moisture in the soil from previous events. For example, the soil will allow much more precipitation to infiltrate, and produce less runoff in a fall storm with little previous rainfall, than in a spring storm with recent rainfall.

Another critical assumption in the use of design storms is that a storm event of a particular frequency will produce a conveyance system response, such as combined sewer overflow (CSO), of the same frequency. This assumption may be invalid in the case of a conveyance system that has a significant amount of storage. The relationship between rainfall frequency and CSO frequency is highly system-specific, depending on both the runoff and storage/routing characteristics of the system.

In order to select a design storm that correlates with a CSO frequency, long-term records of historical CSOs, historical precipitation, and a consistent record of operation practices are required. In the case of the Sacramento system, however, operation of the CWCS has been relatively inconsistent with regard to the transferring of wastewater from Sump 1A and Sump 2 to the CWTP and/or Pioneer Reservoir. In addition, historical records of CSOs are not comprehensive enough to derive such a relationship.

The purpose of this memorandum is to describe the efforts to use alternative methods to the design storm approach to determine the frequency of flooding and CSOs in the Phase 3 analysis. These methods include continuous simulation and the use of long duration historical storms.

B. CONTINUOUS SIMULATION

The above-mentioned problems associated with use of synthetic design storms in analyzing a system as hydraulically complex as the CWCS can be solved through the use of continuous simulation. Continuous simulation involves modeling a system using a long-term rainfall record (usually hourly) or a selected representative period (e.g. 10 years) of that record. Statistics, such a CSO or flooding frequency, can then be derived directly from the simulation under actual,

historical rainfall conditions, rather than extrapolated from the use of single design events.

B.1. Overview of the Continuous Simulation SSWMM91

A continuous simulation version of SSWMM91 was developed for the Phase 3 analysis. The SSWMM91 model was modified to operate on a data base of hourly records rather than a single design event. The data base, obtained for the NWS downtown Sacramento gage, consists of 55 years of total hourly rainfall depth measured to a hundredth of an inch. The model scans and analyzes selected blocks of rainfall based on the definition of a rainfall "event". All rainfall events are then simulated in the system and summarized output is created from which statistical analysis can be performed.

A rainfall event in the continuous simulation model is defined by an interstorm period and a cutoff depth. The interstorm period is an integer number of hours for which there is 0.01 inch of rainfall or less per hour. This depth defines the separation of events in the rainfall record. The choice of the interstorm period is important in defining storm events that are critical to the system being analyzed. A key purpose of the continuous SWMM model is to evaluate the operation and design storm sizing for storage projects. The occurrence of sequential events with minimal interstorm periods impacts the volume and pumping requirements for the facility. If two storm events are closely spaced in time, a storage facility may not have enough time to empty, in order to accommodate the entire volume of the two events. To account for such a situation, a longer interstorm period representing the empty rate of the facility should be specified by the user. Otherwise, the second of the two events would not be accurately represented by the model because the model assumes empty storage facilities at the beginning of each event. Selecting a longer interstorm period will cause the events spaced closely in time to be identified by the model as one larger event with multiple peaks. The model will then operate on the larger event and storage effects will be considered between peaks.

The storm cutoff volume is used as a "switch", in decimal inches of rainfall, below which the model ignores the event. This volume should be set to a depth

such that all storms producing flooding and/or CSOs will be identified and processed by the model. Setting this volume to a low value will cause the model to identify and process events that would not produce flooding or CSOs. Processing these events may be a substantial waste of computer time and disk memory.

A more detailed description of the continuous simulation SSWMM91 is given in the Phase 3, Task 2 TM entitled *Continuous Simulation Version of the Sacramento SWMM*.

B.2. Continuous Simulation of the Existing CWCS

Continuous simulation was used in the Phase 3 analysis in order to determine the frequency of CSOs from the existing CWCS. The CWCS model was set up according to the original plan of operations. Refer to the Phase 3, Task 6 TM entitled *Improvement Plans*, Section 3 for further description of system operations modeling. The entire 55 years of record was modeled with a 36-hour interstorm period. CSO volume versus frequency plots for Sump 2 and Pioneer Reservoir were generated from the analysis and are shown in Figures 1 and 2. The analysis indicated that Sump 2 and Pioneer Reservoir overflow an average of approximately 3 and 13 times per year, respectively. These values are represented by the lowest volume points plotted on the curve. Statistical output files used to generate the frequency curves are attached in Appendix A.

The CSO volumes resulting from the two CSO control design storms (1/year and 5/year recurrences) were also compared to the continuous simulation results. The 5/year synthetic design storm does not cause CSOs at either location. The existence of available storage capacity in the system results in the fact that a rainfall event of a given frequency (e.g. 5/yr) will rarely cause a CSO event of the same frequency. For example, a less frequent (greater return period) precipitation event is required to cause a 5/year CSO event. The CSO volume produced by the 1/year synthetic design event compared favorably to the average annual CSO volume from the continuous simulation results. This may be attributable to the longer duration (21 hours) of the 1/year design event. The "mismatch" between the design storm frequency and CSO frequency indicated a need for further verification of CSO statistics.

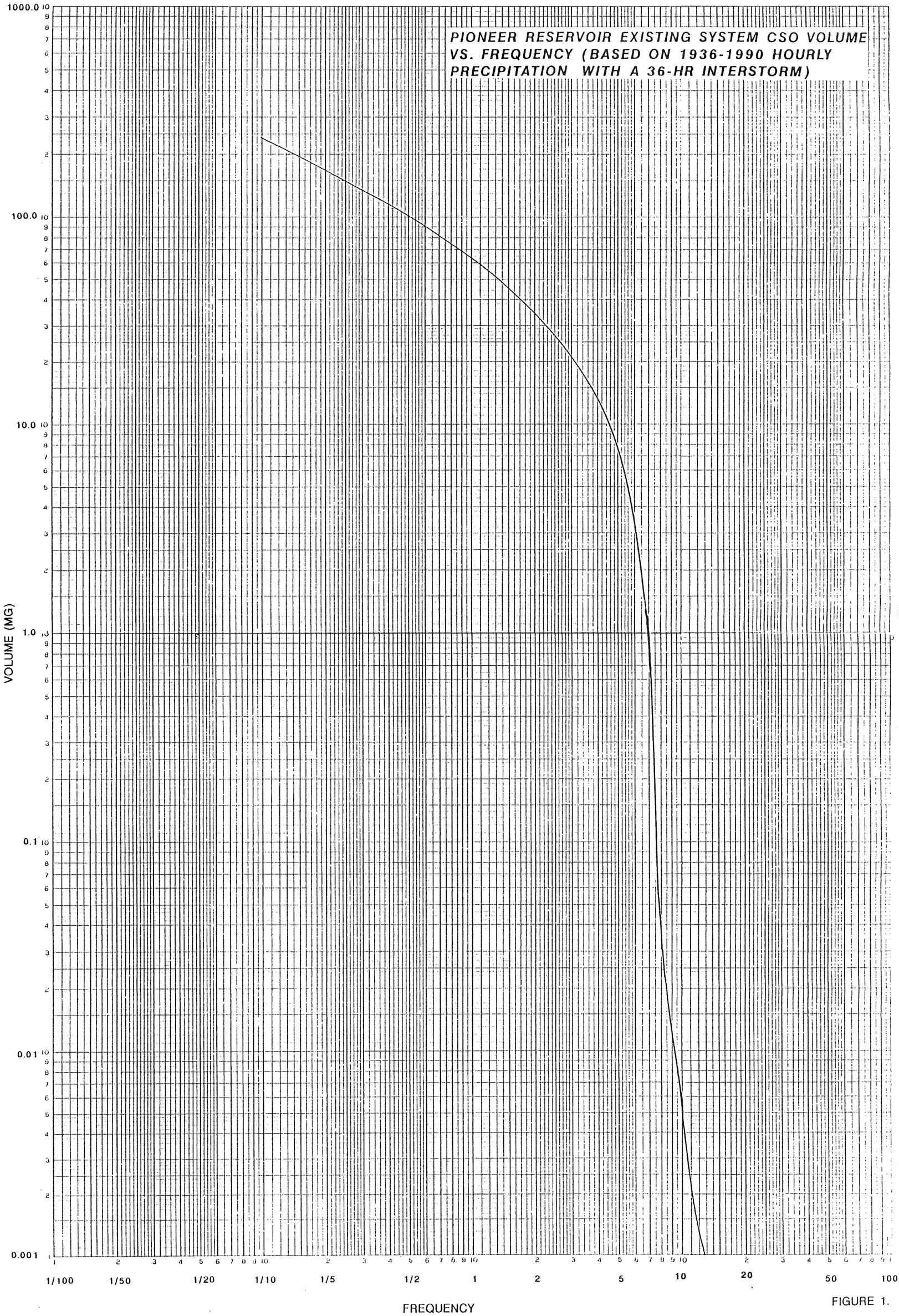

PIONEER RESERVOIR EXISTING SYSTEM CSO VOLUME VS. FREQUENCY (BASED ON 1936-1990 HOURLY PRECIPITATION WITH A 36-HR INTERSTORM)
VOLUME (MG)
1000.0
100.0
10.0
1.0
0.1
0.01
0.001
1/100
1/50
1/20
1/10
1/5
1/2
1
2
5
10
20
50
100
FREQUENCY

FIGURE 1.

NATIONAL
12-081

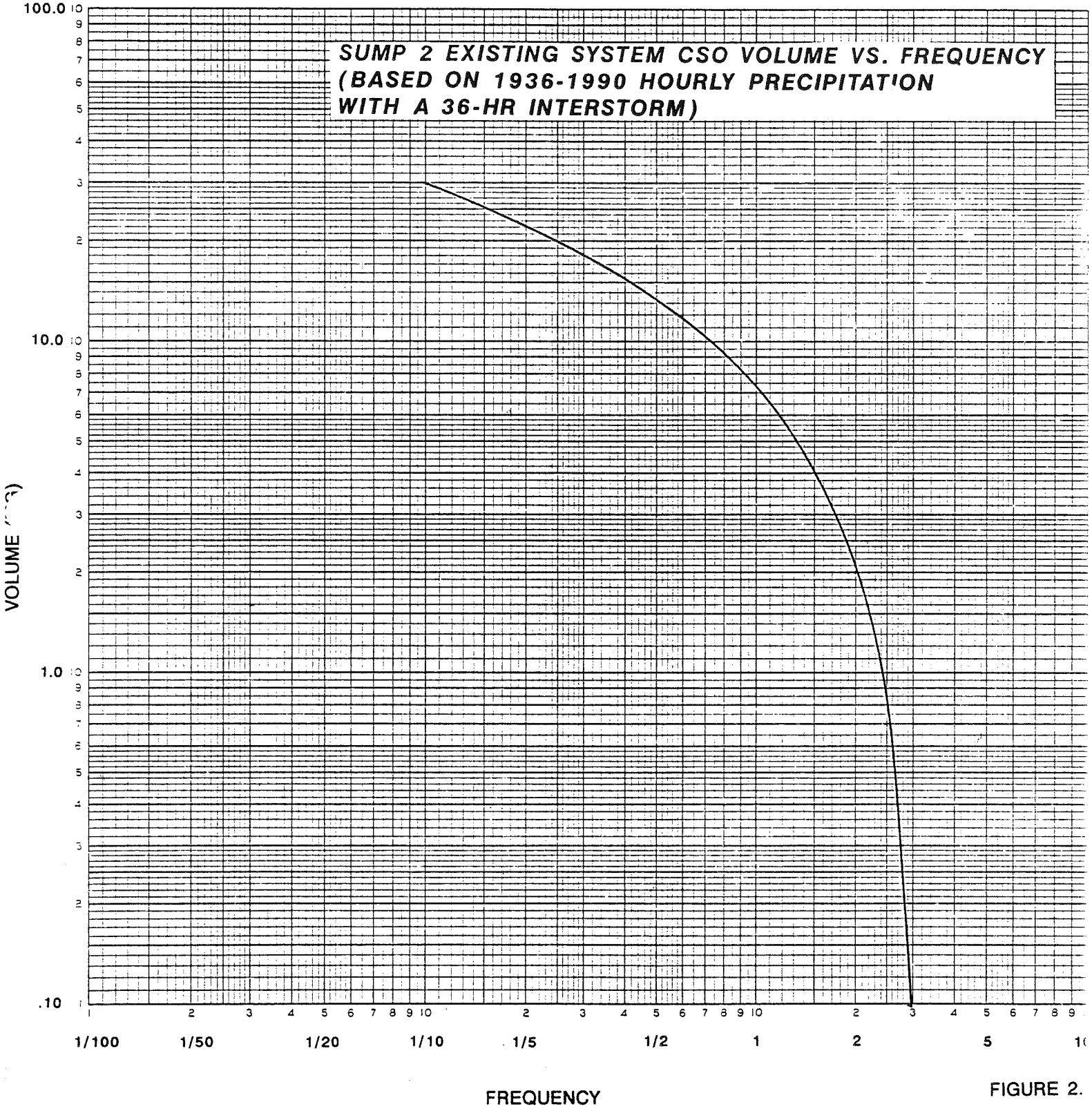


FIGURE 2.

Full Logarithmic, 3 × 3 Cycles

B.3. Application of Continuous Simulation to Least Cost Plans

The objective of using continuous simulation for the least cost alternative plans would be to determine the need, if any, for increasing conveyance or storage/pump facility sizes in order to accommodate long-duration, multiple peak storm events that would actually occur according to the long-term record. Multiple-peak, long-duration storms having significant volumes could cause a flood control facility to be overwhelmed because the facilities may not be able to empty adequately between rainfall peaks. If a facility does not have sufficient time to empty between peaks or "waves" in the hyetograph, storage capacity may not be available to handle the "wave" of rainfall that follows.

Continuous simulation could be used to verify the flood and CSO frequency of the alternative improvement plans. The tunnel alternatives were the least-cost alternative for the 5-, 10- and 25-year flood control design storms. In these alternatives, tunnels perform two functions. First, tunnels convey excess design flows from locations throughout the CWCS to the river or Sump 1A and Sump 2. Second, the tunnels serve as storage for the CSO design storm volume. Continuous simulation, with respect to the tunnel alternatives, would determine whether flooding would occur due to insufficient conveyance in the tunnel once every 5, 10 or 25 years. Continuous simulation would also determine if the tunnel volume was sufficient to store flows to reduce CSOs to 5/year or 1/year.

The continuous simulation approach provides the most accurate representation of the frequency-based statistics for a system, such as the frequency of CSOs or flooding. It would also provide the best information on which to prepare a benefit/cost analysis. However, because of the schedule constraints for the Phase 3 study, a compromise approach was proposed to account for the varying conditions posed by long-duration storms without the time and resources required for continuous simulation.

The Phase 3 study retained the design storm approach for CSO analysis with the City's assumption that the frequency of the design storm correlates adequately with the frequency of CSOs for the purposes of the study. However, a long duration storm approach was derived to verify the adequacy of the least-cost

alternatives with respect to flood control.

C. LONG DURATION STORM APPROACH

The long duration storm analysis consisted of choosing historical storms of long durations, having frequencies similar to the synthetic design storm frequencies. Selecting a rainfall event from the historical rainfall record appropriate for analyzing a storage facility requires the use of the emptying time for that particular facility. In order for storage projects within the City's combined sewer system to empty to the Sacramento Regional Wastewater Treatment Plant (SRWTP) for secondary treatment, a period of reduced or zero rain is required. Therefore, these long-duration events are "defined" within the long-term rainfall record by interstorm periods. This definition is contrary to the manner in which synthetic design events are defined, which is according to rainfall duration.

C.1. Phase 3 Scope of Work

The Phase 3 Scope of Work (SOW) outlined an approach for developing long-duration design events, which follows. Additional explanation is in parentheses.

1) Perform runs using HDR's PROVE (Precipitation Runoff Overflow Volume Evaluator) model, with the long-term hourly rainfall record for the NWS Sacramento downtown gage. Runs using interstorm periods of 24 and 48 hours will be used in the model.

2) Develop frequency curves of total storm depth versus return period/frequency for each of the interstorm periods, using the model output. (This is a partial duration series analysis for total storm depth.)

3) Select storage facility "design storm" depths from the frequency curves for the return periods of 2, 5, 10 and 25 years with the interstorms of 24 and 48 hours.

4) Review the long-term hourly record to select the actual events that most closely match the depth on the frequency curves. It is likely that the "match" will not be exact (because the events are actual, selected from the historical record, rather than synthesized).

5) Set up the RUNOFF block input files for the 8 "design" storms selected. Hourly rainfall will be subdivided into 10-minute time steps (of equal intensity).

The evaluation using long-duration storms would be performed only on the least-cost improvement plans for each of the four levels of flood control.

C.2. Modifications to the Phase 3 Approach

Three modifications were made to the Phase 3 Scope of Work to adapt the analysis to the tunnel alternatives which were the least-cost plans resulting from the design storm analysis: 1) four storms were chosen from the historical record for each depth relating to a design frequency; 2) the interstorm period was set at 72 hours rather than 24 and 48 hours; and 3) only the 5- and 10-year design storm frequencies were considered.

C.2.a. Multiple Historical Storms--

There were several reasons for using multiple historical storms rather than a single event, as described in the Phase 3 SOW:

1) First, assigning a frequency to long-duration rainfall events to correspond to the flood control facilities for a particular alternative improvement plan is problematic. Defining long events, which can be several hundred hours in duration, according to total depth is not necessarily appropriate. However, accepted statistical methods are not available for classifying historical events with multiple peaks and breaks in rainfall.

2) Second, selecting only one event from the historical record, based on frequency according to total event rainfall depth, is difficult, since

these storms vary dramatically in the number of peaks, peak intensity and duration. It would be difficult to determine, by inspection, which one of these would be most appropriate for system analysis.

3) Third, the response of the system to a particular rainfall event is difficult to determine in selecting the most appropriate event. The interstorm period used to classify rainfall "events" (or more appropriately, segments of the historical record), should correspond to the emptying time for a particular improvement plan configuration. This would include the requirements for both existing (i.e. Pioneer Reservoir) and new facilities (e.g. the 5-year tunnel system), including routing effects within the piping systems. In addition, the amount of combined sewage volume retained in these facilities is subject to operational considerations for controlling flooding and CSOs.

For these reasons, it was determined that a multiple-storm approach was necessary for evaluating alternative improvement plans that include storage facilities. Rather than selecting one event for each of the return periods (e.g. 5-year) of interest, multiple historical events were to be used for each of the analyses. Because of the long duration of each of these events, hourly rainfall would be used in the SWMM model.

C.2.b. Interstorm Period Selection--

The minimum volume that must be retained by existing and new storage facilities within the CWCS for secondary treatment is that produced by the CSO control design event. Approximately 3 days is required to empty the 1/year CSO design event from Pioneer Reservoir and a new tunnel or storage facility, following the end of rainfall. This emptying period was estimated based on a trial model run using the EXTRAN setup for the 10-year tunnel alternative and the 1/year CSO control design storm. The volumes remaining in Pioneer Reservoir and the tunnel system at the end of the model run were assumed to empty at a rate of 28 mgd. From this volume and rate, a total emptying time was computed. Therefore, 3 days (72 hours) was used as an interstorm period, for the purposes of this analysis.

D. HISTORICAL STORM SELECTION FOR 3-DAY INTERSTORM PERIOD

D.1. Partial Duration Series Analysis

The partial duration series is defined by the occurrence or frequency per year of independent events. Independent storms within the historical, hourly rainfall record were defined as separated by 72 consecutive hours with 0.01 inches or zero precipitation. (Hourly rainfall of 0.01 inches is considered trace rainfall and was therefore excluded from the analysis.) The 1936-1990 NWS Sacramento hourly record was used to develop a frequency curve for total storm depth for all durations of rainfall. This curve was then used to determine the appropriate total volume for storms having return periods of 2, 5, 10 and 25 years. Annual maximum and partial duration series curves were plotted on semi-log paper and fit with a single smooth curve. Plotting positions for the 2-, 5- and 10-year recurrence were adjusted for consistency with the partial duration series. Figure 3 is the frequency curve resulting from the partial duration series analysis.

D.2. Selection of Rainfall Events

Using the frequency curve (total depth versus frequency) developed for the 72-hour interstorm period event definition, the total depth for the 5- and 10- year return periods are 6.85 and 7.93 inches respectively. The detailed printout for the PROVE model run, along with the hourly rainfall record, was reviewed to select events with these approximate total depths. Characteristics of the events identified in the historical record as having a volume close (+/- 10 percent) to the 5 and 10 year volumes are summarized in Tables 1 and 2.

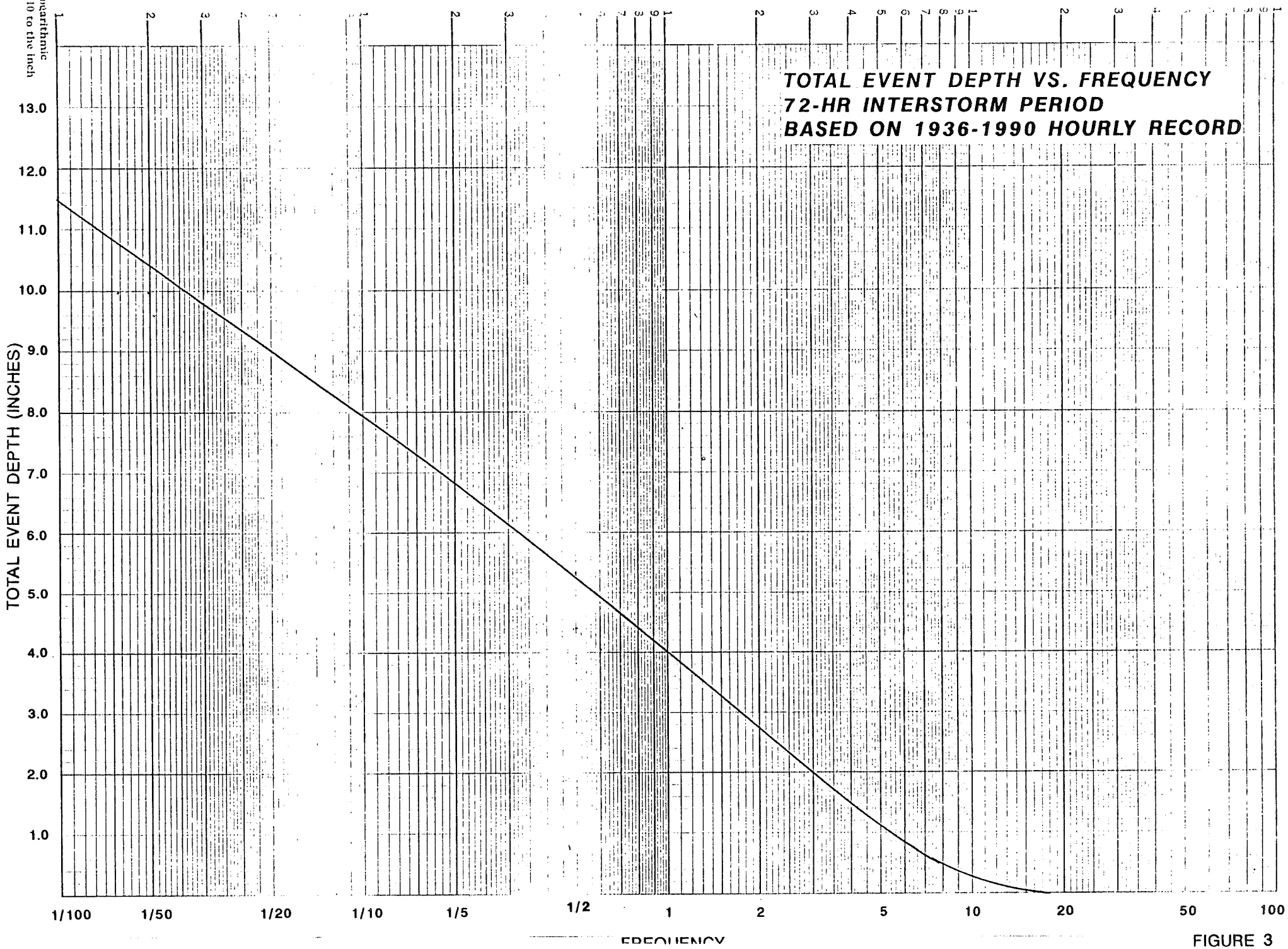

TOTAL EVENT DEPTH VS. FREQUENCY
72-HR INTERSTORM PERIOD
BASED ON 1936-1990 HOURLY RECORD
Semi-Logarithmic
Cycles x 10 to the inch
TOTAL EVENT DEPTH (INCHES)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
1/100
1/50
1/20
1/10
1/5
1/2
1
2
5
10
20
50
100
FREQUENCY

FIGURE 3

TABLE 1. 5-YEAR LONG-DURATION RAINFALL EVENTS FOR 72-HOUR INTERSTORM PERIOD			
Event No.	Beginning Time (yr/mo/da/hr)	Duration (hours)	Total Depth (inches)
1	43/01/20/02	249	6.84
2	62/10/10/12	84	6.77
3	70/11/24/17	332	6.75
4	80/02/13/22	205	6.90

TABLE 2. 10-YEAR LONG-DURATION RAINFALL EVENTS FOR 72-HOUR INTERSTORM PERIOD			
Event No.	Beginning Time (yr/mo/da/hr)	Duration (hours)	Total Depth (inches)
1	36/02/10/21	337	7.67
2	38/01/28/09	414	8.34
3	62/02/06/05	313	7.29
4	67/01/20/06	260	7.76

These storms are presented in both tabular and graphic form in Appendix B.

D.3. Long Duration Storm Results for 5- and 10-Year Tunnel Alternatives

Hourly rain from the long-term record was input into RUNOFF. The continuous simulation version of SSWMM91 was used because of the extensive length of the storms.

The EXTRAN setup for the 5- and 10-year tunnel alternatives includes a weir (in reality, a pumping station) that allows overflows to the Sacramento River from the tunnel system. CSOs are allowed during the 5- and 10-year storm events, provided that the combined sewage volume for the 1/year storm is stored for treatment at the SRWTP. Consequently, the long duration design storm can only

verify the ability of the alternative to control flooding to the design frequency intended. The CSO criteria therefore cannot be verified.

When subjected to the 10-year long duration events, the 10-year tunnel did not experience flooding. This verifies the adherence of the 10-year tunnel to frequency of the design criteria. Runs for the 5-year tunnel alternative however, were not successful. The SSWMM91 program was unable to complete the calculations for all four storms. A discussion of this problem is provided below.

E. CONTINUOUS SSWMM91 LIMITATIONS

The current continuous SSWMM91 model is limited in its ability to analyze alternatives that include weirs. With the current model, any surcharging of a consolidation conduit or tunnel to which a weir empties will cause the program to yield unstable results or terminate the program run. Current model weirs are setup for a particular synthetic design storm, and storms in excess of the design storm with respect to volumes and peaks cannot be handled by the model.

The operation of a system can be changed, in reality, to compensate for varying flows. Changes in operation are limited in the SSMM91 model. Finally, continuous simulation and long duration analysis require great amounts of time and computing power. A compromise for this difficulty would be a careful selection of interstorm period and cutoff depth in order to reduce the evaluation of events which have little or no adverse effect on the system. Another compromise would be to select a representative period of rain to use in the evaluation, instead of the entire period of record. This approach is valid as long as the representative period chosen is at least 5 times the frequency of the desired analysis.

APPENDIX A
CSOSTAT RUNS FOR EXISTING SYSTEM
ENTIRE PERIOD OF RECORD

<<< SUMMARY DATA FOR CONDUIT 1213 >>>

NUMBER OF CSO VOLUMES IN EACH CLASS

CLASS (MG)

YEAR	0.002	0.00	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.04	0.05	0.06	0.08	0.10	NO. EVENTS
36	5	0	0	1	1	0	0	0	2	0	1	1	0	0	0	8	19
37	1	0	0	0	0	1	0	0	0	0	1	1	0	0	1	9	14
38	3	2	0	0	0	0	2	0	0	0	0	0	2	0	0	11	20
39	1	0	0	0	1	0	0	0	0	0	2	1	1	0	0	5	11
40	3	1	0	1	1	1	1	1	1	0	4	3	1	1	0	22	41
41	2	1	0	1	0	1	0	2	1	1	3	0	0	0	0	16	28
42	1	0	0	0	0	0	0	0	0	1	1	2	0	2	0	10	17
43	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	6	7
44	3	0	0	0	1	1	0	1	0	0	4	0	0	0	0	9	19
45	1	1	0	0	0	1	1	1	2	2	0	0	1	0	0	12	22
46	4	0	0	0	0	0	0	0	0	1	0	0	0	0	0	3	8
47	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	4	5
48	2	1	0	0	0	2	0	0	0	1	0	0	1	0	0	6	13
49	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	2	3
50	1	0	0	0	0	1	2	0	1	0	0	0	0	0	0	12	17
51	3	0	0	1	1	0	0	0	0	0	1	1	0	0	0	10	17
52	3	1	1	1	0	0	2	2	1	1	0	1	2	0	0	16	31
53	0	0	0	0	0	0	0	0	0	0	1	0	1	0	0	4	6
	5	0	1	0	1	0	0	2	0	0	2	0	0	0	0	12	23
	0	0	2	0	0	0	0	0	1	0	0	0	0	0	0	4	7
56	1	1	0	0	1	0	1	0	0	1	0	0	0	0	0	10	15
57	1	0	0	0	2	0	0	1	0	1	1	0	0	0	0	6	12
58	3	1	0	1	0	1	1	0	1	1	2	1	0	0	0	15	27
59	2	0	0	0	1	0	0	0	1	0	0	1	0	0	1	5	11
60	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	2	3
61	1	0	1	2	0	0	1	0	0	0	2	0	0	0	0	10	17
62	0	0	0	0	0	1	0	0	0	0	0	0	1	1	0	6	9
63	1	1	2	0	0	1	0	0	0	0	0	1	0	0	0	6	12
64	2	0	0	0	1	1	0	0	0	1	1	0	0	0	0	6	12
65	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	3
66	2	0	1	0	0	0	0	1	0	1	1	0	0	0	0	5	11
67	0	0	0	0	0	0	0	0	0	0	2	1	0	0	0	6	9
68	0	0	0	0	0	0	0	0	1	0	1	1	0	0	0	6	9
69	1	0	1	0	0	0	1	0	1	1	1	1	0	0	0	9	16
70	0	0	0	1	0	0	1	0	0	0	3	0	0	0	0	8	13
71	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	6
72	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
73	5	0	0	0	1	0	0	0	0	0	3	0	0	0	0	11	20
74	1	0	0	0	0	0	0	1	0	0	1	0	0	0	0	6	9
75	0	0	0	0	1	0	0	0	0	1	1	0	0	0	0	6	9
76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
77	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	4	5
78	6	1	2	0	0	0	0	0	0	0	0	0	0	0	0	8	17
79	1	1	0	0	0	1	0	2	0	0	3	0	0	0	0	13	21
80	4	0	1	0	2	0	2	1	2	0	3	0	1	0	0	11	27
81	2	0	2	0	0	0	0	2	1	0	2	1	0	0	0	12	22
	0	1	0	0	1	0	1	1	0	0	2	1	0	0	0	12	19
	1	1	1	1	1	1	1	0	1	0	0	1	1	0	0	16	26
84	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	3
85	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	5	6
86	2	0	0	0	0	0	1	0	0	2	4	1	0	0	0	11	21
87	2	0	1	0	0	0	0	0	0	0	1	0	0	0	0	5	9

88	1	0	0	0	0	0	1	0	2	0	0	0	0	0	0	3	7
89	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	6	8
90	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	4	6
TO	80.	14.	16.	11.	19.	14.	19.	19.	20.	16.	57.	22.	12.	4.	2.	425.	750
AVE NO. EVENTS	1.45	0.25	0.29	0.20	0.35	0.25	0.35	0.35	0.36	0.29	1.04	0.40	0.22	0.07	0.04	7.73	13.64
CUMULATIVE NO. EVENTS	13.64	12.18	11.93	11.64	11.44	11.09	10.84	10.49	10.15	9.78	9.49	8.45	8.05	7.84	7.76	7.73	
EXCEEDENCE CURVE: PERCENT	100.0	89.3	87.5	85.3	83.9	81.3	79.5	76.9	74.4	71.7	69.6	62.0	59.1	57.5	56.9	56.7	
AVE VOL (MG)	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.04	0.06	0.07	35.35	

YEAR	ANNUAL CSO VOLUME (MG)
36	324.31
37	547.84
38	270.52
39	199.45
40	842.20
41	313.39
42	252.93
43	374.40
44	226.51
45	373.98
46	45.76
	111.91
	228.91
49	24.45
50	362.90
51	183.38
52	550.28
53	105.32
54	331.97
55	511.08
56	168.19
57	153.47
58	608.52
59	163.36
60	41.60
61	118.29
62	688.77
63	241.61
64	298.83
65	58.45
66	122.69
67	248.10
68	40.92
69	400.58
70	359.37
71	41.81
72	19.45
73	502.58
74	97.16
	114.37
	0.00
77	65.32
78	352.36
79	233.69
80	329.02

81	185.92
82	832.38
83	619.90
84	85.52
	187.83
	668.80
87	211.13
88	101.32
89	197.41
90	290.33
TOTAL CSO VOLUME	15030.55
AVE ANNUAL VOLUME	273.28

MONTHLY CSO VOLUMES (MG)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
36	14.1	280.6	0.0	18.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.7
37	0.0	200.4	163.4	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	183.2
38	94.5	154.5	21.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	38.9	0.0	46.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	114.1
40	31.3	403.3	90.3	0.0	6.0	0.0	0.0	0.0	0.0	0.0	0.0	311.4
41	54.1	27.5	126.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	100.7
42	65.5	14.8	116.1	28.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.6
43	344.8	0.0	9.2	20.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	175.5	18.0	14.3	0.0	0.0	0.0	0.0	0.0	0.0	6.5	12.1	0.0
	225.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	39.3	0.0	109.7
46	0.0	0.0	2.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	43.3	0.0
47	0.0	72.9	37.3	0.0	0.0	0.0	0.0	0.0	0.0	1.7	0.0	0.0
48	0.0	0.0	15.9	0.0	94.9	0.0	0.0	0.0	0.0	56.8	0.0	61.3
49	1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	23.0	0.0
50	21.8	54.7	15.2	0.0	0.0	0.0	0.0	0.0	0.0	55.2	190.5	25.5
51	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.1	178.2	0.0
52	242.1	0.0	73.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	63.9	170.4
53	7.2	0.0	0.0	82.1	0.0	0.0	0.0	14.5	0.0	0.0	1.6	0.0
54	83.1	95.0	3.7	16.5	0.0	0.0	0.0	0.0	0.0	0.0	48.0	85.7
55	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	511.1
56	102.5	18.7	0.0	0.0	3.3	0.0	0.0	0.0	27.2	16.4	0.0	0.0
57	20.0	31.9	0.0	0.0	0.0	0.0	0.0	0.0	10.4	0.0	0.0	91.1
58	75.6	275.9	256.8	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59	53.2	43.1	0.0	0.0	0.0	0.0	0.0	0.0	67.1	0.0	0.0	0.0
60	0.0	19.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	22.3	0.0
61	27.4	0.0	25.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	65.8	0.0
62	0.0	204.3	10.0	0.0	0.0	0.0	0.0	0.0	0.0	474.4	0.0	0.0
63	102.5	0.0	44.6	49.3	0.0	0.0	0.0	0.0	0.0	40.5	4.8	0.0
64	184.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	26.7	10.7	77.1
65	47.1	0.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	6.3	0.0
66	44.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	78.4	0.0
67	226.1	0.0	22.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
68	24.3	0.0	13.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0
69	228.0	30.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	142.4
70	76.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	282.7	0.0
71	0.0	0.0	37.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.3
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	19.4	0.0
	167.1	225.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	110.2	0.0
74	0.0	8.2	0.0	0.0	0.0	7.1	0.0	0.0	0.0	0.0	0.0	81.8
75	3.7	31.1	76.2	0.0	0.0	0.0	0.0	0.0	0.0	3.4	0.0	0.0
76	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
77	11.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	23.8	30.4

78	299.9	0.0	26.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	25.7	0.0
79	100.1	51.9	15.6	0.0	0.0	0.0	0.0	0.0	0.0	39.9	0.0	26.0
80	99.5	229.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
81	98.0	0.0	25.0	0.0	0.0	0.0	0.0	0.0	0.0	35.7	0.0	27.2
	239.4	105.6	139.1	89.9	0.0	0.0	0.0	0.0	0.0	41.2	131.2	85.9
	115.2	0.0	147.0	72.1	0.0	0.0	0.0	0.0	0.0	0.0	105.3	180.2
84	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.0	57.5	0.0
85	0.0	61.5	3.7	0.0	0.0	0.0	0.0	0.0	0.0	1.9	96.8	24.1
86	80.5	529.8	58.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
87	0.0	79.1	71.6	0.0	0.0	0.0	0.0	0.0	0.0	24.3	36.1	0.0
88	64.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	37.2	0.0
89	0.0	0.0	46.4	0.0	0.0	0.0	0.0	0.0	109.1	42.0	0.0	0.0
90	179.2	99.4	0.0	0.0	11.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
VERAGE MONTHLY VOLUMES	74.0	61.2	31.9	7.0	2.2	0.1	0.0	0.3	3.9	17.1	30.5	45.1

MONTHLY NUMBER OF EVENTS

36	1	14	0	2	0	0	0	0	0	0	0	2
37	0	3	4	1	0	0	0	0	0	0	2	4
38	10	9	1	0	0	0	0	0	0	0	0	0
39	2	0	6	0	0	0	0	0	0	0	0	3
40	5	10	6	0	5	0	0	0	0	0	0	15
41	2	2	5	0	1	0	0	0	0	0	0	18
42	2	2	3	9	0	0	0	0	0	0	0	1
43	3	0	3	1	0	0	0	0	0	0	0	0
44	5	5	1	0	0	0	0	0	0	1	7	0
45	10	0	0	0	0	0	0	0	0	4	0	8
46	0	0	2	0	0	0	0	0	0	0	6	0
47	0	1	3	0	0	0	0	0	0	1	0	0
48	0	0	3	0	1	0	0	0	0	2	0	7
	1	0	0	0	0	0	0	0	0	0	2	0
	3	5	2	0	0	0	0	0	0	1	3	3
51	0	0	0	0	0	0	0	0	0	2	15	0
52	16	0	3	0	0	0	0	0	0	0	5	7
53	2	0	0	1	0	0	0	1	0	0	2	0
54	3	4	2	1	0	0	0	0	0	0	7	6
55	0	0	0	0	0	0	0	0	0	0	0	7
56	10	2	0	0	1	0	0	0	1	1	0	0
57	4	1	0	0	0	0	0	0	3	0	0	4
58	4	15	7	0	1	0	0	0	0	0	0	0
59	6	3	0	0	0	0	0	0	2	0	0	0
60	0	1	0	0	0	0	0	0	0	0	2	0
61	6	0	1	0	0	0	0	0	0	0	10	0
62	0	6	1	0	0	0	0	0	0	2	0	0
63	4	0	3	2	0	0	0	0	0	1	2	0
64	2	0	0	0	0	0	0	0	0	5	2	3
65	1	0	0	1	0	0	0	0	0	0	1	0
66	2	0	0	0	0	0	0	0	0	0	9	0
67	6	0	3	0	0	0	0	0	0	0	0	0
68	4	0	3	0	0	0	0	0	0	0	2	0
69	6	2	0	0	0	0	0	0	0	0	0	8
70	2	0	0	0	0	0	0	0	0	0	11	0
71	0	0	1	0	0	0	0	0	0	0	0	5
72	0	0	0	0	0	0	0	0	0	0	2	0
73	9	6	0	0	0	0	0	0	0	0	5	0
74	0	2	0	0	0	1	0	0	0	0	0	6
75	4	1	2	0	0	0	0	0	0	2	0	0
76	0	0	0	0	0	0	0	0	0	0	0	0
	1	0	0	0	0	0	0	0	0	0	1	3
	9	0	3	0	0	0	0	0	0	0	5	0
79	3	7	4	0	0	0	0	0	0	1	0	6
80	6	21	0	0	0	0	0	0	0	0	0	0
81	13	0	4	0	0	0	0	0	0	3	0	2
82	1	3	3	4	0	0	0	0	0	1	3	4

83	9	0	4	5	0	0	0	0	0	0	6	2
84	0	0	0	0	0	0	0	0	0	1	2	0
85	0	1	1	0	0	0	0	0	0	2	1	1
86	10	4	7	0	0	0	0	0	0	0	0	0
	0	2	2	0	0	0	0	0	0	4	1	0
	5	0	0	0	0	0	0	0	0	0	2	0
89	0	0	2	0	0	0	0	0	3	3	0	0
90	3	1	0	0	2	0	0	0	0	0	0	0
VERAGE MONTHLY NO. EVENTS	3.5	2.4	1.7	0.5	0.2	0.0	0.0	0.0	0.2	0.7	2.1	2.3

ANNUAL MAXIMUM CSO EVENT

YEAR	MG	RANKED YEAR	RANKED MG	MG
36	132.1	62	466.3	466.3
37	153.3	86	382.4	382.4
38	81.1	43	269.8	269.8
39	109.5	82	239.4	239.4
40	232.0	55	234.8	234.8
41	100.9	40	232.0	232.0
42	71.1	45	210.2	210.2
43	269.8	67	206.3	206.3
44	174.4	83	179.2	179.2
45	210.2	44	174.4	174.4
46	32.0	52	167.6	167.6
47	72.9	90	167.0	167.0
48	94.9	58	165.9	165.9
49	23.0	70	164.3	164.3
	128.8	73	157.9	157.9
	87.4	64	153.6	153.6
52	167.6	37	153.3	153.3
53	82.1	36	132.1	132.1
54	83.1	50	128.8	128.8
55	234.8	78	119.9	119.9
56	65.4	39	109.5	109.5
57	54.8	89	105.5	105.5
58	165.9	63	102.5	102.5
59	67.0	41	100.9	100.9
60	22.3	85	96.8	96.8
61	36.6	48	94.9	94.9
62	466.3	80	91.9	91.9
63	102.5	69	90.3	90.3
64	153.6	51	87.4	87.4
65	47.1	54	83.1	83.1
66	45.3	53	82.1	82.1
67	206.3	38	81.1	81.1
68	16.4	47	72.9	72.9
69	90.3	87	71.5	71.5
70	164.3	42	71.1	71.1
71	37.5	59	67.0	67.0
72	18.2	56	65.4	65.4
73	157.9	79	65.3	65.3
74	57.0	74	57.0	57.0
75	52.2	57	54.8	54.8
76	0.0	75	52.2	52.2
77	23.8	65	47.1	47.1
	119.9	81	45.9	45.9
	65.3	66	45.3	45.3
80	91.9	88	43.0	43.0
81	45.9	71	37.5	37.5
82	239.4	61	36.6	36.6
83	179.2	84	34.9	34.9

ANNUAL NUMBER OF CSO EVENTS

YEAR	NO.	RANKED YEAR	RANKED NO.
36	19	40	41
37	14	52	31
38	20	41	28
39	11	58	27
40	41	80	27
41	28	83	26
42	17	54	23
43	7	45	22
44	19	81	22
45	22	79	21
46	8	86	21
47	5	38	20
48	13	73	20
49	3	36	19
50	17	44	19
51	17	82	19
52	31	42	17
53	6	50	17
54	23	51	17
55	7	61	17
56	15	78	17
57	12	69	16
58	27	56	15
59	11	37	14
60	3	48	13
61	17	70	13
62	9	57	12
63	12	63	12
64	12	64	12
65	3	39	11
66	11	59	11
67	9	66	11
68	9	62	9
69	16	67	9
70	13	68	9
71	6	74	9
72	2	75	9
73	20	87	9
74	9	46	8
75	9	89	8
76	0	43	7
77	5	55	7
78	17	88	7
79	21	53	6
80	27	71	6
81	22	85	6
82	19	90	6
83	26	47	5

ANNUAL VOLUME OF CSO EVENTS

RANKED YEAR	RANKED MG	PROBABILITY YEAR	PROBABILITY MG	MG	(FOR PLOTTING)
36	324.3	40	842.2	842.2	98.18
37	547.8	82	832.4	832.4	96.36
38	270.5	62	688.8	688.8	94.55
39	199.4	86	668.8	668.8	92.73
40	842.2	83	619.9	619.9	90.91
41	313.4	58	608.5	608.5	89.09
42	252.9	52	550.3	550.3	87.27
43	374.4	37	547.8	547.8	85.45
44	226.5	55	511.1	511.1	83.64
45	374.0	73	502.6	502.6	81.82
46	45.8	69	400.6	400.6	80.00
47	111.9	43	374.4	374.4	78.18
48	228.9	45	374.0	374.0	76.36
49	24.4	50	362.9	362.9	74.55
50	362.9	70	359.4	359.4	72.73
51	183.4	78	352.4	352.4	70.91
52	550.3	54	332.0	332.0	69.09
53	105.3	80	329.0	329.0	67.27
54	332.0	36	324.3	324.3	65.45
55	511.1	41	313.4	313.4	63.64
56	168.2	64	298.8	298.8	61.82
57	153.5	90	290.3	290.3	60.00
58	608.5	38	270.5	270.5	58.18
59	163.4	42	252.9	252.9	56.36
60	41.6	67	248.1	248.1	54.55
61	118.3	63	241.6	241.6	52.73
62	688.8	79	233.7	233.7	50.91
63	241.6	48	228.9	228.9	49.09
64	298.8	44	226.5	226.5	47.27
65	58.5	87	211.1	211.1	45.45
66	122.7	39	199.4	199.4	43.64
67	248.1	89	197.4	197.4	41.82
68	40.9	85	187.8	187.8	40.00
69	400.6	81	185.9	185.9	38.18
70	359.4	51	183.4	183.4	36.36
71	41.8	56	168.2	168.2	34.55
72	19.4	59	163.4	163.4	32.73
73	502.6	57	153.5	153.5	30.91
74	97.2	66	122.7	122.7	29.09
75	114.4	61	118.3	118.3	27.27
76	0.0	75	114.4	114.4	25.45
77	65.3	47	111.9	111.9	23.64
78	352.4	53	105.3	105.3	21.82
79	233.7	88	101.3	101.3	20.00
80	329.0	74	97.2	97.2	18.18
81	185.9	84	85.5	85.5	16.36
82	832.4	77	65.3	65.3	14.55
83	619.9	65	58.5	58.5	12.73

84	34.9	46	32.0	32.0	84	3	77	5	84	85.5	46	45.8	45.8	10.91
85	96.8	77	23.8	23.8	85	6	49	3	85	187.8	71	41.8	41.8	9.09
86	382.4	49	23.0	23.0	86	21	60	3	86	668.8	60	41.6	41.6	7.27
87	71.5	60	22.3	22.3	87	9	65	3	87	211.1	68	40.9	40.9	5.45
	43.0	72	18.2	18.2	88	7	84	3	88	101.3	49	24.4	24.4	3.64
	105.5	68	16.4	16.4	89	8	72	2	89	197.4	72	19.4	19.4	1.82
90	167.0	68	0.0	0.0	90	6	72	0	90	290.3	72	0.0	0.0	0.00

<<< SUMMARY DATA FOR CONDUIT 1213 >>>

NUMBER OF CSO VOLUMES IN EACH CLASS

CLASS (MG)

YEAR	0.100	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.20	1.40	1.60	1.80	2.00	2.20	NO. EVENTS
36	11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	8	19
37	5	0	0	0	0	0	0	1	0	0	1	0	1	0	0	6	14
38	9	1	0	0	0	0	0	1	0	0	0	1	0	0	0	8	20
39	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	11
40	19	0	1	1	0	0	0	0	0	0	1	0	0	0	0	19	41
41	12	0	0	0	1	1	0	0	0	1	0	0	0	0	0	13	28
42	7	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	17
43	1	0	0	0	0	1	0	0	0	0	1	0	0	0	0	4	7
44	10	0	0	0	0	0	0	0	0	0	1	0	0	0	0	8	19
45	10	0	0	0	0	0	0	1	0	0	0	0	1	0	0	10	22
46	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	8
47	1	0	0	0	0	0	0	0	0	0	0	0	0	1	0	3	5
48	7	0	0	0	0	0	0	0	0	1	0	0	0	0	0	5	13
49	1	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	3
50	5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	11	17
51	7	0	0	0	0	0	0	0	0	0	1	2	0	0	1	6	17
52	15	0	1	0	0	0	0	0	0	1	0	0	0	0	0	14	31
53	2	0	0	0	0	0	0	0	0	0	0	0	1	0	0	3	6
	11	0	0	0	0	0	0	0	0	0	0	0	0	1	0	11	23
	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	7
56	5	0	0	0	0	0	1	0	0	0	0	0	1	0	0	8	15
57	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	12
58	12	1	1	1	0	1	0	0	0	0	0	0	0	0	0	11	27
59	6	0	0	0	0	0	0	0	1	0	0	0	0	0	0	4	11
60	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	3
61	7	0	0	0	0	0	0	0	0	0	1	0	1	0	1	7	17
62	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	9
63	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	12
64	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	12
65	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	3
66	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	11
67	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	9
68	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	9
69	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	9	16
70	5	0	0	0	0	0	0	0	1	0	0	0	0	0	0	7	13
71	3	0	0	0	0	0	0	0	0	0	0	1	0	0	0	2	6
72	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1	2
73	9	0	0	0	0	0	0	0	0	0	0	0	1	0	0	10	20
74	3	0	0	0	0	0	0	0	0	0	0	0	0	1	0	5	9
75	3	0	0	0	0	0	0	0	0	0	0	0	1	0	0	5	9
76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
77	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	5
78	9	0	0	0	0	0	0	0	0	1	0	0	0	0	0	7	17
79	8	0	0	1	1	0	1	0	0	0	0	0	0	0	0	10	21
80	16	1	0	0	0	0	0	0	0	0	0	0	0	0	0	10	27
81	10	0	1	1	0	1	0	0	0	0	0	0	0	0	0	9	22
	7	0	0	0	0	0	1	0	0	0	0	0	0	0	0	11	19
	10	0	0	0	1	0	0	1	1	0	1	0	0	0	0	12	26
84	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	3
85	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	6
86	10	0	0	0	0	0	0	0	0	0	1	0	0	0	0	10	21
87	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	9

88	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	7
89	2	0	0	0	1	0	0	0	0	0	0	0	0	0	0	5	8
90	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	6
	325.	3.	4.	4.	4.	4.	3.	4.	3.	4.	8.	5.	8.	3.	5.	363.	750
AVE NO. EVENTS	5.91	0.05	0.07	0.07	0.07	0.07	0.05	0.07	0.05	0.07	0.15	0.09	0.15	0.05	0.09	6.60	13.64
CUMULATIVE NO. EVENTS	13.64	7.73	7.67	7.60	7.53	7.45	7.38	7.33	7.25	7.20	7.13	6.98	6.89	6.75	6.69	6.60	
EXCEEDENCE CURVE: PERCENT	100.0	56.7	56.3	55.7	55.2	54.7	54.1	53.7	53.2	52.8	52.3	51.2	50.5	49.5	49.1	48.4	
AVE VOL (MG)	0.01	0.16	0.26	0.35	0.43	0.55	0.66	0.76	0.82	0.96	1.09	1.27	1.52	1.62	1.86	41.23	

YEAR	ANNUAL CSO VOLUME (MG)
36	324.31
37	547.84
38	270.52
39	199.45
40	842.20
41	313.39
42	252.93
43	374.40
44	226.51
45	373.98
46	45.76
	111.91
	228.91
49	24.45
50	362.90
51	183.38
52	550.28
53	105.32
54	331.97
55	511.08
56	168.19
57	153.47
58	608.52
59	163.36
60	41.60
61	118.29
62	688.77
63	241.61
64	298.83
65	58.45
66	122.69
67	248.10
68	40.92
69	400.58
70	359.37
71	41.81
72	19.45
73	502.58
74	97.16
	114.37
	0.00
77	65.32
78	352.36
79	233.69
80	329.02

81	185.92
82	832.38
83	619.90
84	85.52
	187.83
	668.80
87	211.13
88	101.32
89	197.41
90	290.33

TOTAL CSO VOLUME 15030.55

AVE ANNUAL VOLUME 273.28

MONTHLY CSO VOLUMES (MG)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
36	14.1	280.6	0.0	18.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.7
37	0.0	200.4	163.4	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	183.2
38	94.5	154.5	21.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	38.9	0.0	46.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	114.1
40	31.3	403.3	90.3	0.0	6.0	0.0	0.0	0.0	0.0	0.0	0.0	311.4
41	54.1	27.5	126.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	100.7
42	65.5	14.8	116.1	28.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.6
43	344.8	0.0	9.2	20.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	175.5	18.0	14.3	0.0	0.0	0.0	0.0	0.0	0.0	6.5	12.1	0.0
	225.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	39.3	0.0	109.7
46	0.0	0.0	2.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	43.3	0.0
47	0.0	72.9	37.3	0.0	0.0	0.0	0.0	0.0	0.0	1.7	0.0	0.0
48	0.0	0.0	15.9	0.0	94.9	0.0	0.0	0.0	0.0	56.8	0.0	61.3
49	1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	23.0	0.0
50	21.8	54.7	15.2	0.0	0.0	0.0	0.0	0.0	0.0	55.2	190.5	25.5
51	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.1	178.2	0.0
52	242.1	0.0	73.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	63.9	170.4
53	7.2	0.0	0.0	82.1	0.0	0.0	0.0	14.5	0.0	0.0	1.6	0.0
54	83.1	95.0	3.7	16.5	0.0	0.0	0.0	0.0	0.0	0.0	48.0	85.7
55	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	511.1
56	102.5	18.7	0.0	0.0	3.3	0.0	0.0	0.0	27.2	16.4	0.0	0.0
57	20.0	31.9	0.0	0.0	0.0	0.0	0.0	0.0	10.4	0.0	0.0	91.1
58	75.6	275.9	256.8	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59	53.2	43.1	0.0	0.0	0.0	0.0	0.0	0.0	67.1	0.0	0.0	0.0
60	0.0	19.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	22.3	0.0
61	27.4	0.0	25.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	65.8	0.0
62	0.0	204.3	10.0	0.0	0.0	0.0	0.0	0.0	0.0	474.4	0.0	0.0
63	102.5	0.0	44.6	49.3	0.0	0.0	0.0	0.0	0.0	40.5	4.8	0.0
64	184.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	26.7	10.7	77.1
65	47.1	0.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	6.3	0.0
66	44.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	78.4	0.0
67	226.1	0.0	22.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
68	24.3	0.0	13.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0
69	228.0	30.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	142.4
70	76.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	282.7	0.0
71	0.0	0.0	37.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.3
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	19.4	0.0
	167.1	225.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	110.2	0.0
74	0.0	8.2	0.0	0.0	0.0	7.1	0.0	0.0	0.0	0.0	0.0	81.8
75	3.7	31.1	76.2	0.0	0.0	0.0	0.0	0.0	0.0	3.4	0.0	0.0
76	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
77	11.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	23.8	30.4

78	299.9	0.0	26.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	25.7	0.0
79	100.1	51.9	15.6	0.0	0.0	0.0	0.0	0.0	0.0	39.9	0.0	26.0
80	99.5	229.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
81	98.0	0.0	25.0	0.0	0.0	0.0	0.0	0.0	0.0	35.7	0.0	27.2
	239.4	105.6	139.1	89.9	0.0	0.0	0.0	0.0	0.0	41.2	131.2	85.9
	115.2	0.0	147.0	72.1	0.0	0.0	0.0	0.0	0.0	0.0	105.3	180.2
84	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.0	57.5	0.0
85	0.0	61.5	3.7	0.0	0.0	0.0	0.0	0.0	0.0	1.9	96.8	24.1
86	80.5	529.8	58.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
87	0.0	79.1	71.6	0.0	0.0	0.0	0.0	0.0	0.0	24.3	36.1	0.0
88	64.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	37.2	0.0
89	0.0	0.0	46.4	0.0	0.0	0.0	0.0	0.0	109.1	42.0	0.0	0.0
90	179.2	99.4	0.0	0.0	11.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
.VERAGE MONTHLY VOLUMES	74.0	61.2	31.9	7.0	2.2	0.1	0.0	0.3	3.9	17.1	30.5	45.1

MONTHLY NUMBER OF EVENTS

36	1	14	0	2	0	0	0	0	0	0	0	2
37	0	3	4	1	0	0	0	0	0	0	2	4
38	10	9	1	0	0	0	0	0	0	0	0	0
39	2	0	6	0	0	0	0	0	0	0	0	3
40	5	10	6	0	5	0	0	0	0	0	0	15
41	2	2	5	0	1	0	0	0	0	0	0	18
42	2	2	3	9	0	0	0	0	0	0	0	1
43	3	0	3	1	0	0	0	0	0	0	0	0
44	5	5	1	0	0	0	0	0	0	1	7	0
45	10	0	0	0	0	0	0	0	0	4	0	8
46	0	0	2	0	0	0	0	0	0	0	6	0
47	0	1	3	0	0	0	0	0	0	1	0	0
48	0	0	3	0	1	0	0	0	0	2	0	7
	1	0	0	0	0	0	0	0	0	0	2	0
	3	5	2	0	0	0	0	0	0	1	3	3
51	0	0	0	0	0	0	0	0	0	2	15	0
52	16	0	3	0	0	0	0	0	0	0	5	7
53	2	0	0	1	0	0	0	1	0	0	2	0
54	3	4	2	1	0	0	0	0	0	0	7	6
55	0	0	0	0	0	0	0	0	0	0	0	7
56	10	2	0	0	1	0	0	0	1	1	0	0
57	4	1	0	0	0	0	0	0	3	0	0	4
58	4	15	7	0	1	0	0	0	0	0	0	0
59	6	3	0	0	0	0	0	0	2	0	0	0
60	0	1	0	0	0	0	0	0	0	0	2	0
61	6	0	1	0	0	0	0	0	0	0	10	0
62	0	6	1	0	0	0	0	0	0	2	0	0
63	4	0	3	2	0	0	0	0	0	1	2	0
64	2	0	0	0	0	0	0	0	0	5	2	3
65	1	0	0	1	0	0	0	0	0	0	1	0
66	2	0	0	0	0	0	0	0	0	0	9	0
67	6	0	3	0	0	0	0	0	0	0	0	0
68	4	0	3	0	0	0	0	0	0	0	2	0
69	6	2	0	0	0	0	0	0	0	0	0	8
70	2	0	0	0	0	0	0	0	0	0	11	0
71	0	0	1	0	0	0	0	0	0	0	0	5
72	0	0	0	0	0	0	0	0	0	0	2	0
73	9	6	0	0	0	0	0	0	0	0	5	0
74	0	2	0	0	0	1	0	0	0	0	0	6
75	4	1	2	0	0	0	0	0	0	2	0	0
76	0	0	0	0	0	0	0	0	0	0	0	0
	1	0	0	0	0	0	0	0	0	0	1	3
	9	0	3	0	0	0	0	0	0	0	5	0
79	3	7	4	0	0	0	0	0	0	1	0	6
80	6	21	0	0	0	0	0	0	0	0	0	0
81	13	0	4	0	0	0	0	0	0	3	0	2
82	1	3	3	4	0	0	0	0	0	1	3	4

83	9	0	4	5	0	0	0	0	0	0	6	2
84	0	0	0	0	0	0	0	0	0	1	2	0
85	0	1	1	0	0	0	0	0	0	2	1	1
86	10	4	7	0	0	0	0	0	0	0	0	0
	0	2	2	0	0	0	0	0	0	4	1	0
	5	0	0	0	0	0	0	0	0	0	2	0
89	0	0	2	0	0	0	0	0	3	3	0	0
90	3	1	0	0	2	0	0	0	0	0	0	0
VERAGE MONTHLY NO. EVENTS	3.5	2.4	1.7	0.5	0.2	0.0	0.0	0.0	0.2	0.7	2.1	2.3

ANNUAL MAXIMUM CSO EVENT

YEAR	MG	RANKED YEAR	RANKED MG	MG
36	132.1	62	466.3	466.3
37	153.3	86	382.4	382.4
38	81.1	43	269.8	269.8
39	109.5	82	239.4	239.4
40	232.0	55	234.8	234.8
41	100.9	40	232.0	232.0
42	71.1	45	210.2	210.2
43	269.8	67	206.3	206.3
44	174.4	83	179.2	179.2
45	210.2	44	174.4	174.4
46	32.0	52	167.6	167.6
47	72.9	90	167.0	167.0
48	94.9	58	165.9	165.9
49	23.0	70	164.3	164.3
	128.8	73	157.9	157.9
	87.4	64	153.6	153.6
52	167.6	37	153.3	153.3
53	82.1	36	132.1	132.1
54	83.1	50	128.8	128.8
55	234.8	78	119.9	119.9
56	65.4	39	109.5	109.5
57	54.8	89	105.5	105.5
58	165.9	63	102.5	102.5
59	67.0	41	100.9	100.9
60	22.3	85	96.8	96.8
61	36.6	48	94.9	94.9
62	466.3	80	91.9	91.9
63	102.5	69	90.3	90.3
64	153.6	51	87.4	87.4
65	47.1	54	83.1	83.1
66	45.3	53	82.1	82.1
67	206.3	38	81.1	81.1
68	16.4	47	72.9	72.9
69	90.3	87	71.5	71.5
70	164.3	42	71.1	71.1
71	37.5	59	67.0	67.0
72	18.2	56	65.4	65.4
73	157.9	79	65.3	65.3
74	57.0	74	57.0	57.0
75	52.2	57	54.8	54.8
76	0.0	75	52.2	52.2
77	23.8	65	47.1	47.1
	119.9	81	45.9	45.9
	65.3	66	45.3	45.3
80	91.9	88	43.0	43.0
81	45.9	71	37.5	37.5
82	239.4	61	36.6	36.6
83	179.2	84	34.9	34.9

ANNUAL NUMBER OF CSO EVENTS

YEAR	NO.	RANKED YEAR	RANKED NO.
36	19	40	41
37	14	52	31
38	20	41	28
39	11	58	27
40	41	80	27
41	28	83	26
42	17	54	23
43	7	45	22
44	19	81	22
45	22	79	21
46	8	86	21
47	5	38	20
48	13	73	20
49	3	36	19
50	17	44	19
51	17	82	19
52	31	42	17
53	6	50	17
54	23	51	17
55	7	61	17
56	15	78	17
57	12	69	16
58	27	56	15
59	11	37	14
60	3	48	13
61	17	70	13
62	9	57	12
63	12	63	12
64	12	64	12
65	3	39	11
66	11	59	11
67	9	66	11
68	9	62	9
69	16	67	9
70	13	68	9
71	6	74	9
72	2	75	9
73	20	87	9
74	9	46	8
75	9	89	8
76	0	43	7
77	5	55	7
78	17	88	7
79	21	53	6
80	27	71	6
81	22	85	6
82	19	90	6
83	26	47	5

ANNUAL VOLUME OF CSO EVENTS

YEAR	MG	RANKED YEAR	RANKED MG	MG	PROBABILITY (FOR PLOTTING)
36	324.3	40	842.2	842.2	98.18
37	547.8	82	832.4	832.4	96.36
38	270.5	62	688.8	688.8	94.55
39	199.4	86	668.8	668.8	92.73
40	842.2	83	619.9	619.9	90.91
41	313.4	58	608.5	608.5	89.09
42	252.9	52	550.3	550.3	87.27
43	374.4	37	547.8	547.8	85.45
44	226.5	55	511.1	511.1	83.64
45	374.0	73	502.6	502.6	81.82
46	45.8	69	400.6	400.6	80.00
47	111.9	43	374.4	374.4	78.18
48	228.9	45	374.0	374.0	76.36
49	24.4	50	362.9	362.9	74.55
50	362.9	70	359.4	359.4	72.73
51	183.4	78	352.4	352.4	70.91
52	550.3	54	332.0	332.0	69.09
53	105.3	80	329.0	329.0	67.27
54	332.0	36	324.3	324.3	65.45
55	511.1	41	313.4	313.4	63.64
56	168.2	64	298.8	298.8	61.82
57	153.5	90	290.3	290.3	60.00
58	608.5	38	270.5	270.5	58.18
59	163.4	42	252.9	252.9	56.36
60	41.6	67	248.1	248.1	54.55
61	118.3	63	241.6	241.6	52.73
62	688.8	79	233.7	233.7	50.91
63	241.6	48	228.9	228.9	49.09
64	298.8	44	226.5	226.5	47.27
65	58.5	87	211.1	211.1	45.45
66	122.7	39	199.4	199.4	43.64
67	248.1	89	197.4	197.4	41.82
68	40.9	85	187.8	187.8	40.00
69	400.6	81	185.9	185.9	38.18
70	359.4	51	183.4	183.4	36.36
71	41.8	56	168.2	168.2	34.55
72	19.4	59	163.4	163.4	32.73
73	502.6	57	153.5	153.5	30.91
74	97.2	66	122.7	122.7	29.09
75	114.4	61	118.3	118.3	27.27
76	0.0	75	114.4	114.4	25.45
77	65.3	47	111.9	111.9	23.64
78	352.4	53	105.3	105.3	21.82
79	233.7	88	101.3	101.3	20.00
80	329.0	74	97.2	97.2	18.18
81	185.9	84	85.5	85.5	16.36
82	832.4	77	65.3	65.3	14.55
83	619.9	65	58.5	58.5	12.73

84	34.9	46	32.0	32.0	84	3	77	5	84	85.5	46	45.8	45.8	10.91
85	96.8	77	23.8	23.8	85	6	49	3	85	187.8	71	41.8	41.8	9.09
86	382.4	49	23.0	23.0	86	21	60	3	86	668.8	60	41.6	41.6	7.27
87	71.5	60	22.3	22.3	87	9	65	3	87	211.1	68	40.9	40.9	5.45
	43.0	72	18.2	18.2	88	7	84	3	88	101.3	49	24.4	24.4	3.64
	105.5	68	16.4	16.4	89	8	72	2	89	197.4	72	19.4	19.4	1.82
90	167.0	68	0.0	0.0	90	6	72	0	90	290.3	72	0.0	0.0	0.00

<<< SUMMARY DATA FOR CONDUIT 1213 >>>

NUMBER OF CSO VOLUMES IN EACH CLASS

CLASS (MG)

YEAR	1.800	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00	12.00	14.00	16.00	18.00	20.00	22.00	NO. EVENTS
36	11	0	0	0	0	0	0	1	0	0	1	0	2	0	1	3	19
37	8	0	0	0	0	0	0	1	0	0	0	0	0	0	0	5	14
38	12	0	0	0	0	0	0	0	1	0	0	1	0	0	0	6	20
39	6	0	0	0	1	0	0	0	0	0	0	0	0	0	0	4	11
40	22	0	0	1	2	2	0	0	1	0	0	1	1	0	1	10	41
41	15	0	2	0	3	1	0	0	0	1	0	0	0	0	0	6	28
42	7	1	1	0	0	0	1	1	0	0	1	0	1	0	0	4	17
43	3	0	0	0	0	0	0	0	1	0	0	0	0	0	0	3	7
44	11	0	1	1	1	0	2	0	0	0	0	1	1	0	0	1	19
45	12	0	1	1	0	1	0	0	0	1	0	0	0	1	1	4	22
46	5	0	1	0	0	0	0	0	0	0	1	0	0	0	0	1	8
47	2	0	0	0	0	0	0	0	0	0	0	0	1	0	0	2	5
48	8	0	0	0	0	0	0	0	0	0	0	0	1	0	0	4	13
49	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3
50	5	1	0	1	0	0	0	1	1	1	0	0	2	0	0	5	17
51	10	1	0	0	0	1	1	0	0	0	0	0	0	0	0	4	17
52	17	0	2	0	0	0	1	0	1	1	1	0	0	0	0	8	31
53	3	0	0	0	0	0	0	1	0	0	0	0	1	0	0	1	6
[illegible]	12	0	3	1	1	0	0	0	0	0	0	0	0	1	0	5	23
[illegible]	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	7
56	7	0	0	1	0	1	0	0	0	0	0	0	2	1	1	2	15
57	6	0	1	0	0	0	0	0	0	0	1	0	0	1	0	3	12
58	16	0	1	2	0	0	0	1	0	0	0	0	0	0	0	7	27
59	7	0	0	0	0	0	0	0	0	0	0	0	1	0	0	3	11
60	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	3
61	9	1	1	0	0	1	0	0	0	0	0	2	0	0	1	2	17
62	3	0	0	0	0	0	0	0	1	0	2	0	0	0	0	3	9
63	6	0	0	0	2	0	0	0	0	0	0	0	0	0	0	4	12
64	6	0	0	0	1	0	0	0	0	0	1	0	0	0	0	4	12
65	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	1	3
66	6	0	0	1	1	0	0	0	0	0	0	0	0	0	0	3	11
67	3	0	1	1	0	0	0	0	0	0	2	1	0	0	0	1	9
68	3	0	1	3	0	0	0	0	0	1	0	0	0	1	0	0	9
69	7	0	1	0	0	1	0	0	0	0	0	0	0	0	0	7	16
70	6	0	0	0	0	0	0	0	1	0	0	0	0	0	0	6	13
71	4	0	0	1	0	0	0	0	0	0	0	0	0	0	0	1	6
72	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	2
73	10	0	0	1	0	0	1	1	0	0	0	0	0	0	0	7	20
74	4	0	0	1	1	0	0	1	0	0	0	0	0	0	0	2	9
75	4	0	1	1	0	0	0	0	0	0	0	0	0	0	0	3	9
76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
77	1	0	0	0	0	0	0	0	1	0	1	0	0	0	0	2	5
78	10	0	0	1	0	0	1	0	0	0	0	0	0	0	1	4	17
79	11	0	1	0	0	0	1	1	0	0	0	1	1	1	0	4	21
80	17	0	0	1	0	0	0	0	0	1	0	0	0	2	1	5	27
81	13	0	1	1	0	0	1	0	0	0	0	0	0	0	1	5	22
[illegible]	8	0	1	0	0	0	0	0	0	0	0	0	0	0	1	9	19
[illegible]	14	0	0	1	1	0	0	0	0	0	1	0	1	0	0	8	26
84	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	3
85	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	3	6
86	11	0	0	0	2	0	0	0	0	0	1	0	0	0	1	6	21
87	4	0	0	0	0	0	0	0	0	0	1	0	0	0	0	4	9

88	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	7
89	3	0	0	1	0	0	0	0	0	0	0	0	0	0	1	3	8
90	2	0	0	0	0	0	0	0	0	0	1	1	0	0	0	2	6
T	382.	5.	21.	23.	16.	9.	10.	9.	8.	6.	15.	8.	15.	8.	13.	202.	750
AVE NO. EVENTS	6.95	0.09	0.38	0.42	0.29	0.16	0.18	0.16	0.15	0.11	0.27	0.15	0.27	0.15	0.24	3.67	13.64
CUMULATIVE NO. EVENTS	13.64	6.69	6.60	6.22	5.80	5.51	5.35	5.16	5.00	4.85	4.75	4.47	4.33	4.05	3.91	3.67	
EXCEEDENCE CURVE: PERCENT	100.0	49.1	48.4	45.6	42.5	40.4	39.2	37.9	36.7	35.6	34.8	32.8	31.7	29.7	28.7	26.9	
AVE VOL (MG)	0.14	1.86	2.50	3.56	4.48	5.29	6.47	7.40	8.53	9.69	10.90	12.91	14.70	16.63	19.01	67.26	

YEAR	ANNUAL CSO VOLUME (MG)
36	324.31
37	547.84
38	270.52
39	199.45
40	842.20
41	313.39
42	252.93
43	374.40
44	226.51
45	373.98
46	45.76
	111.91
	228.91
49	24.45
50	362.90
51	183.38
52	550.28
53	105.32
54	331.97
55	511.08
56	168.19
57	153.47
58	608.52
59	163.36
60	41.60
61	118.29
62	688.77
63	241.61
64	298.83
65	58.45
66	122.69
67	248.10
68	40.92
69	400.58
70	359.37
71	41.81
72	19.45
73	502.58
74	97.16
	114.37
	0.00
77	65.32
78	352.36
79	233.69
80	329.02

81	185.92
82	832.38
83	619.90
84	85.52
	187.83
	668.80
87	211.13
88	101.32
89	197.41
90	290.33

TOTAL CSO VOLUME 15030.55

AVE ANNUAL VOLUME 273.28

MONTHLY CSO VOLUMES (MG)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
36	14.1	280.6	0.0	18.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.7
37	0.0	200.4	163.4	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	183.2
38	94.5	154.5	21.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	38.9	0.0	46.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	114.1
40	31.3	403.3	90.3	0.0	6.0	0.0	0.0	0.0	0.0	0.0	0.0	311.4
41	54.1	27.5	126.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	100.7
42	65.5	14.8	116.1	28.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.6
43	344.8	0.0	9.2	20.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	175.5	18.0	14.3	0.0	0.0	0.0	0.0	0.0	0.0	6.5	12.1	0.0
	225.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	39.3	0.0	109.7
46	0.0	0.0	2.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	43.3	0.0
47	0.0	72.9	37.3	0.0	0.0	0.0	0.0	0.0	0.0	1.7	0.0	0.0
48	0.0	0.0	15.9	0.0	94.9	0.0	0.0	0.0	0.0	56.8	0.0	61.3
49	1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	23.0	0.0
50	21.8	54.7	15.2	0.0	0.0	0.0	0.0	0.0	0.0	55.2	190.5	25.5
51	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.1	178.2	0.0
52	242.1	0.0	73.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	63.9	170.4
53	7.2	0.0	0.0	82.1	0.0	0.0	0.0	14.5	0.0	0.0	1.6	0.0
54	83.1	95.0	3.7	16.5	0.0	0.0	0.0	0.0	0.0	0.0	48.0	85.7
55	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	511.1
56	102.5	18.7	0.0	0.0	3.3	0.0	0.0	0.0	27.2	16.4	0.0	0.0
57	20.0	31.9	0.0	0.0	0.0	0.0	0.0	0.0	10.4	0.0	0.0	91.1
58	75.6	275.9	256.8	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59	53.2	43.1	0.0	0.0	0.0	0.0	0.0	0.0	67.1	0.0	0.0	0.0
60	0.0	19.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	22.3	0.0
61	27.4	0.0	25.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	65.8	0.0
62	0.0	204.3	10.0	0.0	0.0	0.0	0.0	0.0	0.0	474.4	0.0	0.0
63	102.5	0.0	44.6	49.3	0.0	0.0	0.0	0.0	0.0	40.5	4.8	0.0
64	184.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	26.7	10.7	77.1
65	47.1	0.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	6.3	0.0
66	44.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	78.4	0.0
67	226.1	0.0	22.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
68	24.3	0.0	13.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0
69	228.0	30.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	142.4
70	76.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	282.7	0.0
71	0.0	0.0	37.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.3
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	19.4	0.0
	167.1	225.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	110.2	0.0
74	0.0	8.2	0.0	0.0	0.0	7.1	0.0	0.0	0.0	0.0	0.0	81.8
75	3.7	31.1	76.2	0.0	0.0	0.0	0.0	0.0	0.0	3.4	0.0	0.0
76	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
77	11.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	23.8	30.4

78	299.9	0.0	26.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	25.7	0.0
79	100.1	51.9	15.6	0.0	0.0	0.0	0.0	0.0	0.0	39.9	0.0	26.0
80	99.5	229.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
81	98.0	0.0	25.0	0.0	0.0	0.0	0.0	0.0	0.0	35.7	0.0	27.2
	239.4	105.6	139.1	89.9	0.0	0.0	0.0	0.0	0.0	41.2	131.2	85.9
	115.2	0.0	147.0	72.1	0.0	0.0	0.0	0.0	0.0	0.0	105.3	180.2
84	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.0	57.5	0.0
85	0.0	61.5	3.7	0.0	0.0	0.0	0.0	0.0	0.0	1.9	96.8	24.1
86	80.5	529.8	58.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
87	0.0	79.1	71.6	0.0	0.0	0.0	0.0	0.0	0.0	24.3	36.1	0.0
88	64.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	37.2	0.0
89	0.0	0.0	46.4	0.0	0.0	0.0	0.0	0.0	109.1	42.0	0.0	0.0
90	179.2	99.4	0.0	0.0	11.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
VERAGE MONTHLY VOLUMES	74.0	61.2	31.9	7.0	2.2	0.1	0.0	0.3	3.9	17.1	30.5	45.1

MONTHLY NUMBER OF EVENTS

36	1	14	0	2	0	0	0	0	0	0	0	2
37	0	3	4	1	0	0	0	0	0	0	2	4
38	10	9	1	0	0	0	0	0	0	0	0	0
39	2	0	6	0	0	0	0	0	0	0	0	3
40	5	10	6	0	5	0	0	0	0	0	0	15
41	2	2	5	0	1	0	0	0	0	0	0	18
42	2	2	3	9	0	0	0	0	0	0	0	1
43	3	0	3	1	0	0	0	0	0	0	0	0
44	5	5	1	0	0	0	0	0	0	1	7	0
45	10	0	0	0	0	0	0	0	0	4	0	8
46	0	0	2	0	0	0	0	0	0	0	6	0
47	0	1	3	0	0	0	0	0	0	1	0	0
48	0	0	3	0	1	0	0	0	0	2	0	7
	1	0	0	0	0	0	0	0	0	0	2	0
	3	5	2	0	0	0	0	0	0	1	3	3
51	0	0	0	0	0	0	0	0	0	2	15	0
52	16	0	3	0	0	0	0	0	0	0	5	7
53	2	0	0	1	0	0	0	1	0	0	2	0
54	3	4	2	1	0	0	0	0	0	0	7	6
55	0	0	0	0	0	0	0	0	0	0	0	7
56	10	2	0	0	1	0	0	0	1	1	0	0
57	4	1	0	0	0	0	0	0	3	0	0	4
58	4	15	7	0	1	0	0	0	0	0	0	0
59	6	3	0	0	0	0	0	0	2	0	0	0
60	0	1	0	0	0	0	0	0	0	0	2	0
61	6	0	1	0	0	0	0	0	0	0	10	0
62	0	6	1	0	0	0	0	0	0	2	0	0
63	4	0	3	2	0	0	0	0	0	1	2	0
64	2	0	0	0	0	0	0	0	0	5	2	3
65	1	0	0	1	0	0	0	0	0	0	1	0
66	2	0	0	0	0	0	0	0	0	0	9	0
67	6	0	3	0	0	0	0	0	0	0	0	0
68	4	0	3	0	0	0	0	0	0	0	2	0
69	6	2	0	0	0	0	0	0	0	0	0	8
70	2	0	0	0	0	0	0	0	0	0	11	0
71	0	0	1	0	0	0	0	0	0	0	0	5
72	0	0	0	0	0	0	0	0	0	0	2	0
73	9	6	0	0	0	0	0	0	0	0	5	0
74	0	2	0	0	0	1	0	0	0	0	0	6
75	4	1	2	0	0	0	0	0	0	2	0	0
76	0	0	0	0	0	0	0	0	0	0	0	0
	1	0	0	0	0	0	0	0	0	0	1	3
	9	0	3	0	0	0	0	0	0	0	5	0
79	3	7	4	0	0	0	0	0	0	1	0	6
80	6	21	0	0	0	0	0	0	0	0	0	0
81	13	0	4	0	0	0	0	0	0	3	0	2
82	1	3	3	4	0	0	0	0	0	1	3	4

83	9	0	4	5	0	0	0	0	0	0	6	2
84	0	0	0	0	0	0	0	0	0	1	2	0
85	0	1	1	0	0	0	0	0	0	2	1	1
86	10	4	7	0	0	0	0	0	0	0	0	0
[illegible]	0	2	2	0	0	0	0	0	0	4	1	0
[illegible]	5	0	0	0	0	0	0	0	0	0	2	0
89	0	0	2	0	0	0	0	0	3	3	0	0
90	3	1	0	0	2	0	0	0	0	0	0	0
AVERAGE MONTHLY NO. EVENTS	3.5	2.4	1.7	0.5	0.2	0.0	0.0	0.0	0.2	0.7	2.1	2.3

ANNUAL MAXIMUM CSO EVENT

YEAR	MG	RANKED YEAR	RANKED MG	MG
36	132.1	62	466.3	466.3
37	153.3	86	382.4	382.4
38	81.1	43	269.8	269.8
39	109.5	82	239.4	239.4
40	232.0	55	234.8	234.8
41	100.9	40	232.0	232.0
42	71.1	45	210.2	210.2
43	269.8	67	206.3	206.3
44	174.4	83	179.2	179.2
45	210.2	44	174.4	174.4
46	32.0	52	167.6	167.6
47	72.9	90	167.0	167.0
48	94.9	58	165.9	165.9
49	23.0	70	164.3	164.3
[illegible]	128.8	73	157.9	157.9
[illegible]	87.4	64	153.6	153.6
52	167.6	37	153.3	153.3
53	82.1	36	132.1	132.1
54	83.1	50	128.8	128.8
55	234.8	78	119.9	119.9
56	65.4	39	109.5	109.5
57	54.8	89	105.5	105.5
58	165.9	63	102.5	102.5
59	67.0	41	100.9	100.9
60	22.3	85	96.8	96.8
61	36.6	48	94.9	94.9
62	466.3	80	91.9	91.9
63	102.5	69	90.3	90.3
64	153.6	51	87.4	87.4
65	47.1	54	83.1	83.1
66	45.3	53	82.1	82.1
67	206.3	38	81.1	81.1
68	16.4	47	72.9	72.9
69	90.3	87	71.5	71.5
70	164.3	42	71.1	71.1
71	37.5	59	67.0	67.0
72	18.2	56	65.4	65.4
73	157.9	79	65.3	65.3
74	57.0	74	57.0	57.0
75	52.2	57	54.8	54.8
76	0.0	75	52.2	52.2
77	23.8	65	47.1	47.1
[illegible]	119.9	81	45.9	45.9
[illegible]	65.3	66	45.3	45.3
80	91.9	88	43.0	43.0
81	45.9	71	37.5	37.5
82	239.4	61	36.6	36.6
83	179.2	84	34.9	34.9

ANNUAL NUMBER OF CSO EVENTS

YEAR	NO.	RANKED YEAR	RANKED NO.
36	19	40	41
37	14	52	31
38	20	41	28
39	11	58	27
40	41	80	27
41	28	83	26
42	17	54	23
43	7	45	22
44	19	81	22
45	22	79	21
46	8	86	21
47	5	38	20
48	13	73	20
49	3	36	19
50	17	44	19
51	17	82	19
52	31	42	17
53	6	50	17
54	23	51	17
55	7	61	17
56	15	78	17
57	12	69	16
58	27	56	15
59	11	37	14
60	3	48	13
61	17	70	13
62	9	57	12
63	12	63	12
64	12	64	12
65	3	39	11
66	11	59	11
67	9	66	11
68	9	62	9
69	16	67	9
70	13	68	9
71	6	74	9
72	2	75	9
73	20	87	9
74	9	46	8
75	9	89	8
76	0	43	7
77	5	55	7
78	17	88	7
79	21	53	6
80	27	71	6
81	22	85	6
82	19	90	6
83	26	47	5

ANNUAL VOLUME OF CSO EVENTS

RANKED YEAR	RANKED MG	PROBABILITY YEAR	PROBABILITY MG	MG	(FOR PLOTTING)
36	324.3	40	842.2	842.2	98.18
37	547.8	82	832.4	832.4	96.36
38	270.5	62	688.8	688.8	94.55
39	199.4	86	668.8	668.8	92.73
40	842.2	83	619.9	619.9	90.91
41	313.4	58	608.5	608.5	89.09
42	252.9	52	550.3	550.3	87.27
43	374.4	37	547.8	547.8	85.45
44	226.5	55	511.1	511.1	83.64
45	374.0	73	502.6	502.6	81.82
46	45.8	69	400.6	400.6	80.00
47	111.9	43	374.4	374.4	78.18
48	228.9	45	374.0	374.0	76.36
49	24.4	50	362.9	362.9	74.55
50	362.9	70	359.4	359.4	72.73
51	183.4	78	352.4	352.4	70.91
52	550.3	54	332.0	332.0	69.09
53	105.3	80	329.0	329.0	67.27
54	332.0	36	324.3	324.3	65.45
55	511.1	41	313.4	313.4	63.64
56	168.2	64	298.8	298.8	61.82
57	153.5	90	290.3	290.3	60.00
58	608.5	38	270.5	270.5	58.18
59	163.4	42	252.9	252.9	56.36
60	41.6	67	248.1	248.1	54.55
61	118.3	63	241.6	241.6	52.73
62	688.8	79	233.7	233.7	50.91
63	241.6	48	228.9	228.9	49.09
64	298.8	44	226.5	226.5	47.27
65	58.5	87	211.1	211.1	45.45
66	122.7	39	199.4	199.4	43.64
67	248.1	89	197.4	197.4	41.82
68	40.9	85	187.8	187.8	40.00
69	400.6	81	185.9	185.9	38.18
70	359.4	51	183.4	183.4	36.36
71	41.8	56	168.2	168.2	34.55
72	19.4	59	163.4	163.4	32.73
73	502.6	57	153.5	153.5	30.91
74	97.2	66	122.7	122.7	29.09
75	114.4	61	118.3	118.3	27.27
76	0.0	75	114.4	114.4	25.45
77	65.3	47	111.9	111.9	23.64
78	352.4	53	105.3	105.3	21.82
79	233.7	88	101.3	101.3	20.00
80	329.0	74	97.2	97.2	18.18
81	185.9	84	85.5	85.5	16.36
82	832.4	77	65.3	65.3	14.55
83	619.9	65	58.5	58.5	12.73

84	34.9	46	32.0	32.0
85	96.8	77	23.8	23.8
86	382.4	49	23.0	23.0
87	71.5	60	22.3	22.3
	43.0	72	18.2	18.2
	105.5	68	16.4	16.4
90	167.0	68	0.0	0.0

84	3	77	5
85	6	49	3
86	21	60	3
87	9	65	3
88	7	84	3
89	8	72	2
90	6	72	0

84	85.5	46	45.8	45.8	10.91
85	187.8	71	41.8	41.8	9.09
86	668.8	60	41.6	41.6	7.27
87	211.1	68	40.9	40.9	5.45
88	101.3	49	24.4	24.4	3.64
89	197.4	72	19.4	19.4	1.82
90	290.3	72	0.0	0.0	0.00

<<< SUMMARY DATA FOR CONDUIT 1213 >>>

NUMBER OF CSO VOLUMES IN EACH CLASS

CLASS (MG)

YEAR	*****	22.00	24.00	26.00	28.00	30.00	32.00	34.00	36.00	38.00	40.00	44.00	48.00	52.00	56.00	60.00	NO. EVENTS
36	16	0	1	0	0	0	0	0	0	0	0	0	0	0	0	2	19
37	9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	14
38	14	1	1	1	0	0	0	0	1	0	0	0	0	0	0	2	20
39	7	1	0	1	0	0	0	0	0	0	1	0	0	0	0	1	11
40	31	1	1	1	0	0	0	0	1	0	0	0	0	0	0	6	41
41	22	1	0	0	1	1	0	0	0	0	0	0	1	0	1	1	28
42	13	0	0	0	0	1	0	0	0	0	0	0	1	0	0	2	17
43	4	1	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7
44	18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	19
45	18	0	0	0	0	0	1	0	1	1	0	0	0	0	0	1	22
46	7	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	8
47	3	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1	5
48	9	1	0	0	0	0	0	0	0	0	1	0	0	0	0	2	13
49	2	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	3
50	12	0	1	0	0	0	0	0	0	1	0	0	0	0	1	2	17
51	13	0	2	0	0	0	1	0	0	0	0	0	0	0	0	1	17
52	23	2	0	0	0	0	0	0	0	0	0	3	0	0	0	3	31
53	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6
	18	0	0	0	1	0	0	0	0	0	0	1	0	0	0	3	23
	3	0	0	0	0	0	0	0	0	0	0	1	0	0	0	3	7
56	13	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	15
57	9	0	0	0	0	0	1	0	0	1	0	0	0	0	1	0	12
58	20	0	0	1	0	0	0	0	0	0	0	0	1	0	1	4	27
59	8	0	0	0	0	0	0	0	0	0	1	1	0	0	0	1	11
60	2	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	3
61	15	0	0	1	0	0	0	0	0	1	0	0	0	0	0	0	17
62	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	9
63	8	0	0	0	0	0	0	0	0	0	1	1	0	1	0	1	12
64	8	1	0	0	0	0	1	0	0	0	0	0	0	0	0	2	12
65	2	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	3
66	8	0	0	1	0	0	0	0	0	0	0	0	2	0	0	0	11
67	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9
68	9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	9
69	9	1	0	0	1	0	0	0	0	0	0	1	0	0	0	4	16
70	7	0	0	0	2	1	0	0	1	0	0	0	0	0	0	2	13
71	5	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	6
72	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
73	13	0	0	0	0	1	0	0	0	0	0	0	1	0	1	4	20
74	7	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1	9
75	6	0	0	1	0	0	1	0	0	0	0	0	0	0	1	0	9
76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
77	3	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	5
78	13	0	0	0	1	0	0	0	0	0	0	0	0	0	0	3	17
79	17	0	0	0	0	0	1	0	1	0	1	0	0	0	0	1	21
80	22	0	0	0	0	1	1	0	0	0	0	0	0	1	0	2	27
81	17	0	2	0	1	0	0	0	1	0	0	0	1	0	0	0	22
	10	0	0	0	0	0	0	1	0	0	0	1	0	1	1	5	19
	18	0	0	0	0	1	0	0	1	1	0	0	1	1	0	3	26
84	0	0	1	0	0	1	0	0	1	0	0	0	0	0	0	0	3
85	3	0	0	1	0	0	0	0	0	0	0	0	0	0	0	2	6
86	15	2	0	1	0	0	0	1	0	0	0	0	0	0	0	2	21
87	5	0	0	1	0	0	0	0	0	1	0	0	0	0	0	2	9

88	4	1	0	0	0	0	0	0	0	1	0	1	0	0	0	0	7
89	5	0	0	0	1	0	0	0	0	0	0	1	0	0	0	1	8
90	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6
TOT	548.	14.	14.	10.	9.	7.	8.	2.	8.	8.	5.	11.	9.	4.	7.	86.	750
AVE NO. EVENTS	9.96	0.25	0.25	0.18	0.16	0.13	0.15	0.04	0.15	0.15	0.09	0.20	0.16	0.07	0.13	1.56	13.64
CUMULATIVE NO. EVENTS	13.64	3.67	3.42	3.16	2.98	2.82	2.69	2.55	2.51	2.36	2.22	2.13	1.93	1.76	1.69	1.56	
EXCEEDENCE CURVE: PERCENT	100.0	26.9	25.1	23.2	21.9	20.7	19.7	18.7	18.4	17.3	16.3	15.6	14.1	12.9	12.4	11.5	
AVE VOL (MG)	2.63	21.02	22.84	24.79	27.07	28.57	31.36	33.38	34.75	36.72	39.36	42.11	45.98	50.55	54.17	113.22	

YEAR	ANNUAL CSO VOLUME (MG)
36	324.31
37	547.84
38	270.52
39	199.45
40	842.20
41	313.39
42	252.93
43	374.40
44	226.51
45	373.98
46	45.76
	111.91
	228.91
49	24.45
50	362.90
51	183.38
52	550.28
53	105.32
54	331.97
55	511.08
56	168.19
57	153.47
58	608.52
59	163.36
60	41.60
61	118.29
62	688.77
63	241.61
64	298.83
65	58.45
66	122.69
67	248.10
68	40.92
69	400.58
70	359.37
71	41.81
72	19.45
73	502.58
74	97.16
	114.37
	0.00
77	65.32
78	352.36
79	233.69
80	329.02

81	185.92
82	832.38
83	619.90
84	85.52
	187.83
	668.80
87	211.13
88	101.32
89	197.41
90	290.33
TOTAL CSO VOLUME	15030.55
AVE ANNUAL VOLUME	273.28

MONTHLY CSO VOLUMES (MG)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
36	14.1	280.6	0.0	18.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.7
37	0.0	200.4	163.4	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	183.2
38	94.5	154.5	21.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	38.9	0.0	46.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	114.1
40	31.3	403.3	90.3	0.0	6.0	0.0	0.0	0.0	0.0	0.0	0.0	311.4
41	54.1	27.5	126.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	100.7
42	65.5	14.8	116.1	28.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.6
43	344.8	0.0	9.2	20.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	175.5	18.0	14.3	0.0	0.0	0.0	0.0	0.0	0.0	6.5	12.1	0.0
	225.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	39.3	0.0	109.7
46	0.0	0.0	2.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	43.3	0.0
47	0.0	72.9	37.3	0.0	0.0	0.0	0.0	0.0	0.0	1.7	0.0	0.0
48	0.0	0.0	15.9	0.0	94.9	0.0	0.0	0.0	0.0	56.8	0.0	61.3
49	1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	23.0	0.0
50	21.8	54.7	15.2	0.0	0.0	0.0	0.0	0.0	0.0	55.2	190.5	25.5
51	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.1	178.2	0.0
52	242.1	0.0	73.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	63.9	170.4
53	7.2	0.0	0.0	82.1	0.0	0.0	0.0	14.5	0.0	0.0	1.6	0.0
54	83.1	95.0	3.7	16.5	0.0	0.0	0.0	0.0	0.0	0.0	48.0	85.7
55	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	511.1
56	102.5	18.7	0.0	0.0	3.3	0.0	0.0	0.0	27.2	16.4	0.0	0.0
57	20.0	31.9	0.0	0.0	0.0	0.0	0.0	0.0	10.4	0.0	0.0	91.1
58	75.6	275.9	256.8	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59	53.2	43.1	0.0	0.0	0.0	0.0	0.0	0.0	67.1	0.0	0.0	0.0
60	0.0	19.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	22.3	0.0
61	27.4	0.0	25.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	65.8	0.0
62	0.0	204.3	10.0	0.0	0.0	0.0	0.0	0.0	0.0	474.4	0.0	0.0
63	102.5	0.0	44.6	49.3	0.0	0.0	0.0	0.0	0.0	40.5	4.8	0.0
64	184.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	26.7	10.7	77.1
65	47.1	0.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	6.3	0.0
66	44.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	78.4	0.0
67	226.1	0.0	22.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
68	24.3	0.0	13.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0
69	228.0	30.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	142.4
70	76.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	282.7	0.0
71	0.0	0.0	37.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.3
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	19.4	0.0
	167.1	225.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	110.2	0.0
74	0.0	8.2	0.0	0.0	0.0	7.1	0.0	0.0	0.0	0.0	0.0	81.8
75	3.7	31.1	76.2	0.0	0.0	0.0	0.0	0.0	0.0	3.4	0.0	0.0
76	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
77	11.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	23.8	30.4

78	299.9	0.0	26.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	25.7	0.0
79	100.1	51.9	15.6	0.0	0.0	0.0	0.0	0.0	0.0	39.9	0.0	26.0
80	99.5	229.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
81	98.0	0.0	25.0	0.0	0.0	0.0	0.0	0.0	0.0	35.7	0.0	27.2
	239.4	105.6	139.1	89.9	0.0	0.0	0.0	0.0	0.0	41.2	131.2	85.9
	115.2	0.0	147.0	72.1	0.0	0.0	0.0	0.0	0.0	0.0	105.3	180.2
84	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.0	57.5	0.0
85	0.0	61.5	3.7	0.0	0.0	0.0	0.0	0.0	0.0	1.9	96.8	24.1
86	80.5	529.8	58.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
87	0.0	79.1	71.6	0.0	0.0	0.0	0.0	0.0	0.0	24.3	36.1	0.0
88	64.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	37.2	0.0
89	0.0	0.0	46.4	0.0	0.0	0.0	0.0	0.0	109.1	42.0	0.0	0.0
90	179.2	99.4	0.0	0.0	11.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
VERAGE MONTHLY VOLUMES	74.0	61.2	31.9	7.0	2.2	0.1	0.0	0.3	3.9	17.1	30.5	45.1

MONTHLY NUMBER OF EVENTS

36	1	14	0	2	0	0	0	0	0	0	0	2
37	0	3	4	1	0	0	0	0	0	0	2	4
38	10	9	1	0	0	0	0	0	0	0	0	0
39	2	0	6	0	0	0	0	0	0	0	0	3
40	5	10	6	0	5	0	0	0	0	0	0	15
41	2	2	5	0	1	0	0	0	0	0	0	18
42	2	2	3	9	0	0	0	0	0	0	0	1
43	3	0	3	1	0	0	0	0	0	0	0	0
44	5	5	1	0	0	0	0	0	0	1	7	0
45	10	0	0	0	0	0	0	0	0	4	0	8
46	0	0	2	0	0	0	0	0	0	0	6	0
47	0	1	3	0	0	0	0	0	0	1	0	0
48	0	0	3	0	1	0	0	0	0	2	0	7
	1	0	0	0	0	0	0	0	0	0	2	0
	3	5	2	0	0	0	0	0	0	1	3	3
51	0	0	0	0	0	0	0	0	0	2	15	0
52	16	0	3	0	0	0	0	0	0	0	5	7
53	2	0	0	1	0	0	0	1	0	0	2	0
54	3	4	2	1	0	0	0	0	0	0	7	6
55	0	0	0	0	0	0	0	0	0	0	0	7
56	10	2	0	0	1	0	0	0	1	1	0	0
57	4	1	0	0	0	0	0	0	3	0	0	4
58	4	15	7	0	1	0	0	0	0	0	0	0
59	6	3	0	0	0	0	0	0	2	0	0	0
60	0	1	0	0	0	0	0	0	0	0	2	0
61	6	0	1	0	0	0	0	0	0	0	10	0
62	0	6	1	0	0	0	0	0	0	2	0	0
63	4	0	3	2	0	0	0	0	0	1	2	0
64	2	0	0	0	0	0	0	0	0	5	2	3
65	1	0	0	1	0	0	0	0	0	0	1	0
66	2	0	0	0	0	0	0	0	0	0	9	0
67	6	0	3	0	0	0	0	0	0	0	0	0
68	4	0	3	0	0	0	0	0	0	0	2	0
69	6	2	0	0	0	0	0	0	0	0	0	8
70	2	0	0	0	0	0	0	0	0	0	11	0
71	0	0	1	0	0	0	0	0	0	0	0	5
72	0	0	0	0	0	0	0	0	0	0	2	0
73	9	6	0	0	0	0	0	0	0	0	5	0
74	0	2	0	0	0	1	0	0	0	0	0	6
75	4	1	2	0	0	0	0	0	0	2	0	0
76	0	0	0	0	0	0	0	0	0	0	0	0
	1	0	0	0	0	0	0	0	0	0	1	3
	9	0	3	0	0	0	0	0	0	0	5	0
79	3	7	4	0	0	0	0	0	0	1	0	6
80	6	21	0	0	0	0	0	0	0	0	0	0
81	13	0	4	0	0	0	0	0	0	3	0	2
82	1	3	3	4	0	0	0	0	0	1	3	4

83	9	0	4	5	0	0	0	0	0	0	6	2
84	0	0	0	0	0	0	0	0	0	1	2	0
85	0	1	1	0	0	0	0	0	0	2	1	1
86	10	4	7	0	0	0	0	0	0	0	0	0
	0	2	2	0	0	0	0	0	0	4	1	0
	5	0	0	0	0	0	0	0	0	0	2	0
89	0	0	2	0	0	0	0	0	3	3	0	0
90	3	1	0	0	2	0	0	0	0	0	0	0
VERAGE MONTHLY NO. EVENTS	3.5	2.4	1.7	0.5	0.2	0.0	0.0	0.0	0.2	0.7	2.1	2.3

ANNUAL MAXIMUM CSO EVENT

YEAR	MG	RANKED YEAR	RANKED MG	MG
36	132.1	62	466.3	466.3
37	153.3	86	382.4	382.4
38	81.1	43	269.8	269.8
39	109.5	82	239.4	239.4
40	232.0	55	234.8	234.8
41	100.9	40	232.0	232.0
42	71.1	45	210.2	210.2
43	269.8	67	206.3	206.3
44	174.4	83	179.2	179.2
45	210.2	44	174.4	174.4
46	32.0	52	167.6	167.6
47	72.9	90	167.0	167.0
48	94.9	58	165.9	165.9
49	23.0	70	164.3	164.3
	128.8	73	157.9	157.9
	87.4	64	153.6	153.6
52	167.6	37	153.3	153.3
53	82.1	36	132.1	132.1
54	83.1	50	128.8	128.8
55	234.8	78	119.9	119.9
56	65.4	39	109.5	109.5
57	54.8	89	105.5	105.5
58	165.9	63	102.5	102.5
59	67.0	41	100.9	100.9
60	22.3	85	96.8	96.8
61	36.6	48	94.9	94.9
62	466.3	80	91.9	91.9
63	102.5	69	90.3	90.3
64	153.6	51	87.4	87.4
65	47.1	54	83.1	83.1
66	45.3	53	82.1	82.1
67	206.3	38	81.1	81.1
68	16.4	47	72.9	72.9
69	90.3	87	71.5	71.5
70	164.3	42	71.1	71.1
71	37.5	59	67.0	67.0
72	18.2	56	65.4	65.4
73	157.9	79	65.3	65.3
74	57.0	74	57.0	57.0
75	52.2	57	54.8	54.8
76	0.0	75	52.2	52.2
77	23.8	65	47.1	47.1
	119.9	81	45.9	45.9
	65.3	66	45.3	45.3
80	91.9	88	43.0	43.0
81	45.9	71	37.5	37.5
82	239.4	61	36.6	36.6
83	179.2	84	34.9	34.9

ANNUAL NUMBER OF CSO EVENTS

YEAR	NO.	RANKED YEAR	RANKED NO.
36	19	40	41
37	14	52	31
38	20	41	28
39	11	58	27
40	41	80	27
41	28	83	26
42	17	54	23
43	7	45	22
44	19	81	22
45	22	79	21
46	8	86	21
47	5	38	20
48	13	73	20
49	3	36	19
50	17	44	19
51	17	82	19
52	31	42	17
53	6	50	17
54	23	51	17
55	7	61	17
56	15	78	17
57	12	69	16
58	27	56	15
59	11	37	14
60	3	48	13
61	17	70	13
62	9	57	12
63	12	63	12
64	12	64	12
65	3	39	11
66	11	59	11
67	9	66	11
68	9	62	9
69	16	67	9
70	13	68	9
71	6	74	9
72	2	75	9
73	20	87	9
74	9	46	8
75	9	89	8
76	0	43	7
77	5	55	7
78	17	88	7
79	21	53	6
80	27	71	6
81	22	85	6
82	19	90	6
83	26	47	5

ANNUAL VOLUME OF CSO EVENTS

YEAR	MG	RANKED YEAR	RANKED MG	PROBABILITY MG	(FOR PLOTTING)
36	324.3	40	842.2	842.2	98.18
37	547.8	82	832.4	832.4	96.36
38	270.5	62	688.8	688.8	94.55
39	199.4	86	668.8	668.8	92.73
40	842.2	83	619.9	619.9	90.91
41	313.4	58	608.5	608.5	89.09
42	252.9	52	550.3	550.3	87.27
43	374.4	37	547.8	547.8	85.45
44	226.5	55	511.1	511.1	83.64
45	374.0	73	502.6	502.6	81.82
46	45.8	69	400.6	400.6	80.00
47	111.9	43	374.4	374.4	78.18
48	228.9	45	374.0	374.0	76.36
49	24.4	50	362.9	362.9	74.55
50	362.9	70	359.4	359.4	72.73
51	183.4	78	352.4	352.4	70.91
52	550.3	54	332.0	332.0	69.09
53	105.3	80	329.0	329.0	67.27
54	332.0	36	324.3	324.3	65.45
55	511.1	41	313.4	313.4	63.64
56	168.2	64	298.8	298.8	61.82
57	153.5	90	290.3	290.3	60.00
58	608.5	38	270.5	270.5	58.18
59	163.4	42	252.9	252.9	56.36
60	41.6	67	248.1	248.1	54.55
61	118.3	63	241.6	241.6	52.73
62	688.8	79	233.7	233.7	50.91
63	241.6	48	228.9	228.9	49.09
64	298.8	44	226.5	226.5	47.27
65	58.5	87	211.1	211.1	45.45
66	122.7	39	199.4	199.4	43.64
67	248.1	89	197.4	197.4	41.82
68	40.9	85	187.8	187.8	40.00
69	400.6	81	185.9	185.9	38.18
70	359.4	51	183.4	183.4	36.36
71	41.8	56	168.2	168.2	34.55
72	19.4	59	163.4	163.4	32.73
73	502.6	57	153.5	153.5	30.91
74	97.2	66	122.7	122.7	29.09
75	114.4	61	118.3	118.3	27.27
76	0.0	75	114.4	114.4	25.45
77	65.3	47	111.9	111.9	23.64
78	352.4	53	105.3	105.3	21.82
79	233.7	88	101.3	101.3	20.00
80	329.0	74	97.2	97.2	18.18
81	185.9	84	85.5	85.5	16.36
82	832.4	77	65.3	65.3	14.55
83	619.9	65	58.5	58.5	12.73

84	34.9	46	32.0	32.0	84	3	77	5	84	85.5	46	45.8	45.8	10.91
85	96.8	77	23.8	23.8	85	6	49	3	85	187.8	71	41.8	41.8	9.09
86	382.4	49	23.0	23.0	86	21	60	3	86	668.8	60	41.6	41.6	7.27
87	71.5	60	22.3	22.3	87	9	65	3	87	211.1	68	40.9	40.9	5.45
	43.0	72	18.2	18.2	88	7	84	3	88	101.3	49	24.4	24.4	3.64
	105.5	68	16.4	16.4	89	8	72	2	89	197.4	72	19.4	19.4	1.82
90	167.0	68	0.0	0.0	90	6	72	0	90	290.3	72	0.0	0.0	0.00

<<< SUMMARY DATA FOR CONDUIT 1225 >>>

NUMBER OF CSO VOLUMES IN EACH CLASS

CLASS (MG)

YEAR	0.100	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.10	1.20	1.30	1.40	1.50	1.60	NO. EVENTS
36	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	2	3
37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	5
38	0	0	0	0	1	0	0	0	0	0	1	0	0	0	0	2	4
39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
40	0	0	1	0	0	0	0	0	0	1	0	0	0	0	0	5	7
41	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	5	6
42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
45	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	4
46	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
47	1	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	3
48	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	2	3
49	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1
50	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	4	5
51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	3
52	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	5	6
53	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2
	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	3	4
	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	3	4
56	1	1	0	0	0	0	0	0	0	1	0	0	0	0	0	1	4
57	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	1	3
58	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	4	6
59	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	4
60	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
61	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
62	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
63	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	3	4
64	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1	3
65	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
66	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1
67	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
68	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
69	0	0	1	0	0	0	0	0	0	0	0	0	0	0	1	3	5
70	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	3	4
71	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
72	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
73	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	6
74	1	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	2
75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
77	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
78	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	4
79	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	3	5
80	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	4
81	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
	0	0	1	0	0	0	0	0	1	0	0	0	0	0	0	5	7
	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	7	8
84	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1	2
85	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2
86	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	2	4
87	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	3	4

88	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
89	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	3	4
90	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
(	13.	3.	7.	1.	1.	3.	2.	1.	4.	6.	2.	2.	0.	1.	2.	120.	168
.VE NO. VENTS	0.24	0.05	0.13	0.02	0.02	0.05	0.04	0.02	0.07	0.11	0.04	0.04	0.00	0.02	0.04	2.18	3.05
:UMULATIVE IO. EVENTS	3.05	2.82	2.76	2.64	2.62	2.60	2.55	2.51	2.49	2.42	2.31	2.27	2.24	2.24	2.22	2.18	
:XCEEDENCE CURVE: 'ERCENT	100.0	92.3	90.5	86.3	85.7	85.1	83.3	82.1	81.5	79.2	75.6	74.4	73.2	73.2	72.6	71.4	
\VE VOL (MG)	0.03	0.15	0.24	0.33	0.49	0.57	0.61	0.74	0.86	0.95	1.05	1.17	0.00	1.40	1.42	10.83	

YEAR	ANNUAL CSO VOLUME (MG)
36	27.15
37	77.48
38	24.47
39	14.12
40	62.27
41	38.09
42	15.37
43	52.01
44	3.64
45	25.32
4[illegible]	0.17
[illegible]	9.50
4[illegible]	25.23
49	0.60
50	41.01
51	21.50
52	43.83
53	12.47
54	17.91
55	37.67
56	3.64
57	10.34
58	56.19
59	17.65
60	0.00
61	0.02
62	119.94
63	12.17
64	31.14
65	6.90
66	0.92
67	24.16
68	0.00
69	26.09
70	20.99
71	5.10
72	0.00
73	62.99
7[illegible]	1.24
[illegible]	8.51
7[illegible]	0.00
77	0.00
78	28.20
79	10.04
80	14.91

81	4.49
82	59.20
83	57.11
84	3.85
	7.01
	94.61
87	7.65
88	9.12
89	29.75
90	40.15
TOTAL CSO VOLUME	1323.92
AVE ANNUAL VOLUME	24.07

MONTHLY CSO VOLUMES (MG)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
36	0.0	27.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	0.0	32.8	23.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	20.9
38	8.5	14.9	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	4.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.3
40	0.0	43.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	18.4
41	4.7	6.7	19.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	7.4
42	0.0	0.0	12.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.1
43	52.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	3.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	18.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.3	0.0	4.1
46	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0
47	0.0	8.1	1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
48	0.0	0.0	0.0	0.0	16.2	0.0	0.0	0.0	0.0	8.4	0.0	0.6
49	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.0
50	0.0	2.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.4	28.2	0.6
51	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	21.5	0.0
52	11.6	0.0	5.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.4	20.6
53	0.0	0.0	0.0	12.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
54	7.8	7.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5	0.9
55	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	37.7
56	1.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	2.5	0.0	0.0	0.0
57	0.0	0.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.6
58	7.8	6.7	41.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59	0.0	8.3	0.0	0.0	0.0	0.0	0.0	0.0	9.3	0.0	0.0	0.0
60	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
61	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
62	0.0	17.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	102.5	0.0	0.0
63	0.2	0.0	3.9	2.1	0.0	0.0	0.0	0.0	0.0	6.0	0.0	0.0
64	31.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
65	6.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
66	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.9	0.0
67	24.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
68	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
69	14.5	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	11.3
70	6.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.2	0.0
71	0.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	14.4	38.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.1	0.0
74	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2
75	0.0	0.0	8.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
76	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
77	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

78	26.0	0.0	2.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
79	3.7	0.0	2.8	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0	0.0
80	3.1	11.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
81	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.5	0.0	0.0
[illegible]	29.3	4.0	9.6	0.3	0.0	0.0	0.0	0.0	0.0	0.8	15.2	0.0
8[illegible]	9.1	0.0	23.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.8	9.7
84	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	3.3	0.0
85	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.8	0.0
86	0.7	93.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
87	0.0	1.6	0.9	0.0	0.0	0.0	0.0	0.0	0.0	2.0	3.1	0.0
88	4.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.5	0.0
89	0.0	0.0	1.1	0.0	0.0	0.0	0.0	0.0	20.1	8.6	0.0	0.0
90	23.6	16.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
\VERAGE MONTHLY VOLUMES	5.8	6.3	3.0	0.3	0.3	0.0	0.0	0.0	0.6	2.7	2.4	2.8

MONTHLY NUMBER OF EVENTS

36	0	3	0	0	0	0	0	0	0	0	0	0
37	0	2	1	0	0	0	0	0	0	0	0	2
38	2	1	1	0	0	0	0	0	0	0	0	0
39	1	0	0	0	0	0	0	0	0	0	0	1
40	0	4	0	0	0	0	0	0	0	0	0	3
41	1	2	1	0	0	0	0	0	0	0	0	2
42	0	0	1	0	0	0	0	0	0	0	0	1
43	2	0	0	0	0	0	0	0	0	0	0	0
44	1	0	0	0	0	0	0	0	0	0	0	0
45	1	0	0	0	0	0	0	0	0	1	0	2
46	0	0	0	0	0	0	0	0	0	0	1	0
47	0	1	2	0	0	0	0	0	0	0	0	0
48	0	0	0	0	1	0	0	0	0	1	0	1
[illegible]	0	0	0	0	0	0	0	0	0	0	1	0
5[illegible]	0	1	0	0	0	0	0	0	0	1	2	1
51	0	0	0	0	0	0	0	0	0	0	3	0
52	2	0	1	0	0	0	0	0	0	0	2	1
53	0	0	0	1	0	0	0	1	0	0	0	0
54	1	1	0	0	0	0	0	0	0	0	1	1
55	0	0	0	0	0	0	0	0	0	0	0	4
56	2	1	0	0	0	0	0	0	1	0	0	0
57	0	1	0	0	0	0	0	0	0	0	0	2
58	1	3	2	0	0	0	0	0	0	0	0	0
59	1	2	0	0	0	0	0	0	1	0	0	0
60	0	1	0	0	0	0	0	0	0	0	0	0
61	0	0	1	0	0	0	0	0	0	0	1	0
62	0	1	0	0	0	0	0	0	0	1	0	0
63	1	0	1	1	0	0	0	0	0	1	0	0
64	2	0	0	0	0	0	0	0	0	0	0	1
65	1	0	0	0	0	0	0	0	0	0	0	0
66	0	0	0	0	0	0	0	0	0	0	1	0
67	1	0	0	0	0	0	0	0	0	0	0	0
68	0	0	0	0	0	0	0	0	0	0	0	0
69	2	1	0	0	0	0	0	0	0	0	0	2
70	1	0	0	0	0	0	0	0	0	0	3	0
71	0	0	1	0	0	0	0	0	0	0	0	0
72	0	0	0	0	0	0	0	0	0	0	0	0
73	2	2	0	0	0	0	0	0	0	0	2	0
74	0	0	0	0	0	0	0	0	0	0	0	2
75	0	0	1	0	0	0	0	0	0	0	0	0
76	0	0	0	0	0	0	0	0	0	0	0	0
[illegible]	0	0	0	0	0	0	0	0	0	0	0	0
[illegible]	3	0	1	0	0	0	0	0	0	0	0	0
79	2	0	2	0	0	0	0	0	0	1	0	0
80	1	3	0	0	0	0	0	0	0	0	0	0
81	0	0	0	0	0	0	0	0	0	1	0	0
82	1	1	1	1	0	0	0	0	0	1	2	0

83	3	0	1	0	0	0	0	0	0	0	3	1
84	0	0	0	0	0	0	0	0	0	1	1	0
85	0	1	0	0	0	0	0	0	0	0	1	0
86	2	2	0	0	0	0	0	0	0	0	0	0
	0	1	1	0	0	0	0	0	0	1	1	0
	1	0	0	0	0	0	0	0	0	0	1	0
89	0	0	1	0	0	0	0	0	1	2	0	0
90	1	1	0	0	0	0	0	0	0	0	0	0
VERAGE MONTHLY NO. EVENTS	0.7	0.7	0.4	0.1	0.0	0.0	0.0	0.0	0.1	0.2	0.5	0.5

ANNUAL MAXIMUM CSO EVENT

YEAR	MG	RANKED YEAR	RANKED MG	MG
36	13.7	62	102.5	102.5
37	25.6	86	64.9	64.9
38	14.9	43	42.1	42.1
39	9.3	64	30.9	30.9
40	27.5	73	30.2	30.2
41	19.2	82	29.3	29.3
42	12.2	40	27.5	27.5
43	42.1	58	27.2	27.2
44	3.6	37	25.6	25.6
45	18.9	67	24.2	24.2
46	0.2	90	23.6	23.6
47	8.1	83	23.5	23.5
48	16.2	52	20.6	20.6
49	0.6	55	20.5	20.5
	17.8	89	20.1	20.1
	16.2	41	19.2	19.2
52	20.6	45	18.9	18.9
53	12.5	50	17.8	17.8
54	7.8	78	16.6	16.6
55	20.5	48	16.2	16.2
56	2.5	51	16.2	16.2
57	8.2	38	14.9	14.9
58	27.2	36	13.7	13.7
59	9.3	69	13.1	13.1
60	0.0	53	12.5	12.5
61	0.0	42	12.2	12.2
62	102.5	70	11.3	11.3
63	6.0	59	9.3	9.3
64	30.9	39	9.3	9.3
65	6.9	75	8.5	8.5
66	0.9	57	8.2	8.2
67	24.2	47	8.1	8.1
68	0.0	54	7.8	7.8
69	13.1	65	6.9	6.9
70	11.3	85	6.8	6.8
71	5.1	80	6.0	6.0
72	0.0	63	6.0	6.0
73	30.2	71	5.1	5.1
74	1.2	88	4.6	4.6
75	8.5	81	4.5	4.5
76	0.0	44	3.6	3.6
77	0.0	79	3.6	3.6
	16.6	84	3.3	3.3
	3.6	87	3.1	3.1
80	6.0	56	2.5	2.5
81	4.5	74	1.2	1.2
82	29.3	66	0.9	0.9
83	23.5	49	0.6	0.6

ANNUAL NUMBER OF CSO EVENTS

YEAR	NO.	RANKED YEAR	RANKED NO.
36	3	83	8
37	5	40	7
38	4	82	7
39	2	41	6
40	7	52	6
41	6	58	6
42	2	73	6
43	2	37	5
44	1	50	5
45	4	69	5
46	1	79	5
47	3	38	4
48	3	45	4
49	1	54	4
50	5	55	4
51	3	56	4
52	6	59	4
53	2	63	4
54	4	70	4
55	4	78	4
56	4	80	4
57	3	86	4
58	6	87	4
59	4	89	4
60	1	36	3
61	2	47	3
62	2	48	3
63	4	51	3
64	3	57	3
65	1	64	3
66	1	39	2
67	1	42	2
68	0	43	2
69	5	53	2
70	4	61	2
71	1	62	2
72	0	74	2
73	6	84	2
74	2	85	2
75	1	88	2
76	0	90	2
77	0	44	1
78	4	46	1
79	5	49	1
80	4	60	1
81	1	65	1
82	7	66	1
83	8	67	1

ANNUAL VOLUME OF CSO EVENTS

YEAR	MG	RANKED YEAR	RANKED MG	PROBABILITY MG	(FOR PLOTTING)
36	27.2	62	119.9	119.9	98.18
37	77.5	86	94.6	94.6	96.36
38	24.5	37	77.5	77.5	94.55
39	14.1	73	63.0	63.0	92.73
40	62.3	40	62.3	62.3	90.91
41	38.1	82	59.2	59.2	89.09
42	15.4	83	57.1	57.1	87.27
43	52.0	58	56.2	56.2	85.45
44	3.6	43	52.0	52.0	83.64
45	25.3	52	43.8	43.8	81.82
46	0.2	50	41.0	41.0	80.00
47	9.5	90	40.2	40.2	78.18
48	25.2	41	38.1	38.1	76.36
49	0.6	55	37.7	37.7	74.55
50	41.0	64	31.1	31.1	72.73
51	21.5	89	29.8	29.8	70.91
52	43.8	78	28.2	28.2	69.09
53	12.5	36	27.2	27.2	67.27
54	17.9	69	26.1	26.1	65.45
55	37.7	45	25.3	25.3	63.64
56	3.6	48	25.2	25.2	61.82
57	10.3	38	24.5	24.5	60.00
58	56.2	67	24.2	24.2	58.18
59	17.6	51	21.5	21.5	56.36
60	0.0	70	21.0	21.0	54.55
61	0.0	54	17.9	17.9	52.73
62	119.9	59	17.6	17.6	50.91
63	12.2	42	15.4	15.4	49.09
64	31.1	80	14.9	14.9	47.27
65	6.9	39	14.1	14.1	45.45
66	0.9	53	12.5	12.5	43.64
67	24.2	63	12.2	12.2	41.82
68	0.0	57	10.3	10.3	40.00
69	26.1	79	10.0	10.0	38.18
70	21.0	47	9.5	9.5	36.36
71	5.1	88	9.1	9.1	34.55
72	0.0	75	8.5	8.5	32.73
73	63.0	87	7.6	7.6	30.91
74	1.2	85	7.0	7.0	29.09
75	8.5	65	6.9	6.9	27.27
76	0.0	71	5.1	5.1	25.45
77	0.0	81	4.5	4.5	23.64
78	28.2	84	3.8	3.8	21.82
79	10.0	44	3.6	3.6	20.00
80	14.9	56	3.6	3.6	18.18
81	4.5	74	1.2	1.2	16.36
82	59.2	66	0.9	0.9	14.55
83	57.1	49	0.6	0.6	12.73

84	3.3	46	0.2	0.2	84	2	71	1	84	3.8	46	0.2	0.2	10.91
85	6.8	61	0.0	0.0	85	2	75	1	85	7.0	61	0.0	0.0	9.09
86	64.9	60	0.0	0.0	86	4	81	1	86	94.6	60	0.0	0.0	7.27
87	3.1	60	0.0	0.0	87	4	81	0	87	7.6	60	0.0	0.0	5.45
[illegible]	4.6	60	0.0	0.0	88	2	81	0	88	9.1	60	0.0	0.0	3.64
[illegible]	20.1	60	0.0	0.0	89	4	81	0	89	29.8	60	0.0	0.0	1.82
90	23.6	60	0.0	0.0	90	2	81	0	90	40.2	60	0.0	0.0	0.00

<<< SUMMARY DATA FOR CONDUIT 1225 >>>

NUMBER OF CSO VOLUMES IN EACH CLASS

CLASS (MG)

YEAR	1.400	1.50	1.60	1.80	2.00	2.20	2.40	2.80	3.00	3.20	3.40	4.60	3.80	4.00	4.20	4.40	NO. EVENTS
36	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	3
37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	5
38	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	4
39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
40	2	0	0	0	1	0	0	0	0	0	0	0	0	0	0	4	7
41	1	0	0	0	0	0	0	1	0	0	0	1	0	0	0	3	6
42	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	2
43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
44	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1
45	1	0	0	0	0	0	1	0	0	0	0	1	0	0	0	1	4
46	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
47	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3
48	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	3
49	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
50	1	0	0	0	0	0	0	1	0	0	0	0	0	0	0	3	5
51	0	0	0	1	0	0	0	0	0	0	0	1	0	0	0	1	3
52	1	0	0	0	0	0	0	0	0	0	0	1	0	0	0	4	6
53	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2
[illegible]	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	2	4
55	1	0	0	0	0	0	0	0	0	1	0	0	0	0	0	2	4
56	3	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	4
57	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3
58	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	6
59	1	0	0	0	0	0	0	0	0	0	0	2	0	0	0	1	4
60	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
61	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
62	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
63	1	0	0	0	0	1	0	0	0	0	0	1	0	0	0	1	4
64	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3
65	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
66	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
67	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
68	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
69	1	1	0	0	0	1	0	0	0	0	0	0	0	0	0	2	5
70	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	2	4
71	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
72	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
73	0	0	1	0	0	0	0	0	0	1	0	0	0	0	0	4	6
74	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
77	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
78	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	2	4
79	2	0	0	0	0	0	0	1	0	0	0	2	0	0	0	0	5
80	0	0	0	0	0	0	0	1	0	2	0	0	0	0	0	1	4
81	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1
[illegible]	2	0	0	0	0	0	0	0	0	0	0	1	0	0	0	4	7
83	1	0	0	0	0	0	0	1	0	0	0	2	0	0	0	4	8
84	1	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	2
85	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2
86	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	4
87	1	0	0	1	0	1	0	0	0	1	0	0	0	0	0	0	4

88	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1	2
89	1	0	0	0	0	0	0	0	0	0	1	0	0	0	0	2	4
90	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
TC	46.	2.	2.	3.	1.	5.	1.	6.	0.	6.	2.	15.	0.	0.	0.	79.	168
AVE NO. EVENTS	0.84	0.04	0.04	0.05	0.02	0.09	0.02	0.11	0.00	0.11	0.04	0.27	0.00	0.00	0.00	1.44	3.05
CUMULATIVE NO. EVENTS	3.05	2.22	2.18	2.15	2.09	2.07	1.98	1.96	1.85	1.85	1.75	1.71	1.44	1.44	1.44	1.44	
EXCEEDENCE CURVE: PERCENT	100.0	72.6	71.4	70.2	68.5	67.9	64.9	64.3	60.7	60.7	57.1	56.0	47.0	47.0	47.0	47.0	
AVE VOL (MG)	0.48	1.42	1.52	1.69	1.88	2.11	2.34	2.62	0.00	3.10	3.34	3.98	0.00	0.00	0.00	14.88	

YEAR	ANNUAL CSO VOLUME (MG)
36	27.15
37	77.48
38	24.47
39	14.12
40	62.27
41	38.09
42	15.37
43	52.01
44	3.64
45	25.32
46	0.17
	9.50
	25.23
49	0.60
50	41.01
51	21.50
52	43.83
53	12.47
54	17.91
55	37.67
56	3.64
57	10.34
58	56.19
59	17.65
60	0.00
61	0.02
62	119.94
63	12.17
64	31.14
65	6.90
66	0.92
67	24.16
68	0.00
69	26.09
70	20.99
71	5.10
72	0.00
73	62.99
74	1.24
	8.51
	0.00
77	0.00
78	28.20
79	10.04
80	14.91

81	4.49
82	59.20
83	57.11
84	3.85
[illegible]	7.01
86	94.61
87	7.65
88	9.12
89	29.75
90	40.15
TOTAL CSO VOLUME	1323.92
AVE ANNUAL VOLUME	24.07

MONTHLY CSO VOLUMES (MG)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
36	0.0	27.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	0.0	32.8	23.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	20.9
38	8.5	14.9	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	4.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.3
40	0.0	43.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	18.4
41	4.7	6.7	19.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	7.4
42	0.0	0.0	12.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.1
43	52.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
[illegible]	3.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
45	18.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.3	0.0	4.1
46	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0
47	0.0	8.1	1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
48	0.0	0.0	0.0	0.0	16.2	0.0	0.0	0.0	0.0	8.4	0.0	0.6
49	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.0
50	0.0	2.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.4	28.2	0.6
51	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	21.5	0.0
52	11.6	0.0	5.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.4	20.6
53	0.0	0.0	0.0	12.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
54	7.8	7.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5	0.9
55	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	37.7
56	1.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	2.5	0.0	0.0	0.0
57	0.0	0.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.6
58	7.8	6.7	41.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59	0.0	8.3	0.0	0.0	0.0	0.0	0.0	0.0	9.3	0.0	0.0	0.0
60	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
61	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
62	0.0	17.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	102.5	0.0	0.0
63	0.2	0.0	3.9	2.1	0.0	0.0	0.0	0.0	0.0	6.0	0.0	0.0
64	31.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
65	6.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
66	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.9	0.0
67	24.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
68	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
69	14.5	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	11.3
70	6.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.2	0.0
71	0.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
[illegible]	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
73	14.4	38.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.1	0.0
74	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2
75	0.0	0.0	8.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
76	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
77	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

78	26.0	0.0	2.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
79	3.7	0.0	2.8	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0	0.0
80	3.1	11.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
81	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.5	0.0	0.0
	29.3	4.0	9.6	0.3	0.0	0.0	0.0	0.0	0.0	0.8	15.2	0.0
	9.1	0.0	23.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.8	9.7
84	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	3.3	0.0
85	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.8	0.0
86	0.7	93.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
87	0.0	1.6	0.9	0.0	0.0	0.0	0.0	0.0	0.0	2.0	3.1	0.0
88	4.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.5	0.0
89	0.0	0.0	1.1	0.0	0.0	0.0	0.0	0.0	20.1	8.6	0.0	0.0
90	23.6	16.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVERAGE MONTHLY VOLUMES	5.8	6.3	3.0	0.3	0.3	0.0	0.0	0.0	0.6	2.7	2.4	2.8

MONTHLY NUMBER OF EVENTS

36	0	3	0	0	0	0	0	0	0	0	0	0
37	0	2	1	0	0	0	0	0	0	0	0	2
38	2	1	1	0	0	0	0	0	0	0	0	0
39	1	0	0	0	0	0	0	0	0	0	0	1
40	0	4	0	0	0	0	0	0	0	0	0	3
41	1	2	1	0	0	0	0	0	0	0	0	2
42	0	0	1	0	0	0	0	0	0	0	0	1
43	2	0	0	0	0	0	0	0	0	0	0	0
44	1	0	0	0	0	0	0	0	0	0	0	0
45	1	0	0	0	0	0	0	0	0	1	0	2
46	0	0	0	0	0	0	0	0	0	0	1	0
47	0	1	2	0	0	0	0	0	0	0	0	0
48	0	0	0	0	1	0	0	0	0	1	0	1
	0	0	0	0	0	0	0	0	0	0	1	0
	0	1	0	0	0	0	0	0	0	1	2	1
51	0	0	0	0	0	0	0	0	0	0	3	0
52	2	0	1	0	0	0	0	0	0	0	2	1
53	0	0	0	1	0	0	0	1	0	0	0	0
54	1	1	0	0	0	0	0	0	0	0	1	1
55	0	0	0	0	0	0	0	0	0	0	0	4
56	2	1	0	0	0	0	0	0	1	0	0	0
57	0	1	0	0	0	0	0	0	0	0	0	2
58	1	3	2	0	0	0	0	0	0	0	0	0
59	1	2	0	0	0	0	0	0	1	0	0	0
60	0	1	0	0	0	0	0	0	0	0	0	0
61	0	0	1	0	0	0	0	0	0	0	1	0
62	0	1	0	0	0	0	0	0	0	1	0	0
63	1	0	1	1	0	0	0	0	0	1	0	0
64	2	0	0	0	0	0	0	0	0	0	0	1
65	1	0	0	0	0	0	0	0	0	0	0	0
66	0	0	0	0	0	0	0	0	0	0	1	0
67	1	0	0	0	0	0	0	0	0	0	0	0
68	0	0	0	0	0	0	0	0	0	0	0	0
69	2	1	0	0	0	0	0	0	0	0	0	2
70	1	0	0	0	0	0	0	0	0	0	3	0
71	0	0	1	0	0	0	0	0	0	0	0	0
72	0	0	0	0	0	0	0	0	0	0	0	0
73	2	2	0	0	0	0	0	0	0	0	2	0
74	0	0	0	0	0	0	0	0	0	0	0	2
75	0	0	1	0	0	0	0	0	0	0	0	0
76	0	0	0	0	0	0	0	0	0	0	0	0
	0	0	0	0	0	0	0	0	0	0	0	0
	3	0	1	0	0	0	0	0	0	0	0	0
79	2	0	2	0	0	0	0	0	0	1	0	0
80	1	3	0	0	0	0	0	0	0	0	0	0
81	0	0	0	0	0	0	0	0	0	1	0	0
82	1	1	1	1	0	0	0	0	0	1	2	0

83	3	0	1	0	0	0	0	0	0	0	3	1
84	0	0	0	0	0	0	0	0	0	1	1	0
85	0	1	0	0	0	0	0	0	0	0	1	0
86	2	2	0	0	0	0	0	0	0	0	0	0
[illegible]	0	1	1	0	0	0	0	0	0	1	1	0
[illegible]	1	0	0	0	0	0	0	0	0	0	1	0
89	0	0	1	0	0	0	0	0	1	2	0	0
90	1	1	0	0	0	0	0	0	0	0	0	0
VERAGE MONTHLY NO. EVENTS	0.7	0.7	0.4	0.1	0.0	0.0	0.0	0.0	0.1	0.2	0.5	0.5

ANNUAL MAXIMUM CSO EVENT

YEAR	MG	RANKED YEAR	RANKED MG	MG
36	13.7	62	102.5	102.5
37	25.6	86	64.9	64.9
38	14.9	43	42.1	42.1
39	9.3	64	30.9	30.9
40	27.5	73	30.2	30.2
41	19.2	82	29.3	29.3
42	12.2	40	27.5	27.5
43	42.1	58	27.2	27.2
44	3.6	37	25.6	25.6
45	18.9	67	24.2	24.2
46	0.2	90	23.6	23.6
47	8.1	83	23.5	23.5
48	16.2	52	20.6	20.6
49	0.6	55	20.5	20.5
[illegible]	17.8	89	20.1	20.1
[illegible]	16.2	41	19.2	19.2
52	20.6	45	18.9	18.9
53	12.5	50	17.8	17.8
54	7.8	78	16.6	16.6
55	20.5	48	16.2	16.2
56	2.5	51	16.2	16.2
57	8.2	38	14.9	14.9
58	27.2	36	13.7	13.7
59	9.3	69	13.1	13.1
60	0.0	53	12.5	12.5
61	0.0	42	12.2	12.2
62	102.5	70	11.3	11.3
63	6.0	59	9.3	9.3
64	30.9	39	9.3	9.3
65	6.9	75	8.5	8.5
66	0.9	57	8.2	8.2
67	24.2	47	8.1	8.1
68	0.0	54	7.8	7.8
69	13.1	65	6.9	6.9
70	11.3	85	6.8	6.8
71	5.1	80	6.0	6.0
72	0.0	63	6.0	6.0
73	30.2	71	5.1	5.1
74	1.2	88	4.6	4.6
75	8.5	81	4.5	4.5
76	0.0	44	3.6	3.6
77	0.0	79	3.6	3.6
[illegible]	16.6	84	3.3	3.3
[illegible]	3.6	87	3.1	3.1
80	6.0	56	2.5	2.5
81	4.5	74	1.2	1.2
82	29.3	66	0.9	0.9
83	23.5	49	0.6	0.6

ANNUAL NUMBER OF CSO EVENTS

YEAR	NO.	RANKED YEAR	RANKED NO.
36	3	83	8
37	5	40	7
38	4	82	7
39	2	41	6
40	7	52	6
41	6	58	6
42	2	73	6
43	2	37	5
44	1	50	5
45	4	69	5
46	1	79	5
47	3	38	4
48	3	45	4
49	1	54	4
50	5	55	4
51	3	56	4
52	6	59	4
53	2	63	4
54	4	70	4
55	4	78	4
56	4	80	4
57	3	86	4
58	6	87	4
59	4	89	4
60	1	36	3
61	2	47	3
62	2	48	3
63	4	51	3
64	3	57	3
65	1	64	3
66	1	39	2
67	1	42	2
68	0	43	2
69	5	53	2
70	4	61	2
71	1	62	2
72	0	74	2
73	6	84	2
74	2	85	2
75	1	88	2
76	0	90	2
77	0	44	1
78	4	46	1
79	5	49	1
80	4	60	1
81	1	65	1
82	7	66	1
83	8	67	1

ANNUAL VOLUME OF CSO EVENTS

RANKED YEAR	RANKED MG	PROBABILITY YEAR	PROBABILITY MG	MG	(FOR PLOTTING)
36	27.2	62	119.9	119.9	98.18
37	77.5	86	94.6	94.6	96.36
38	24.5	37	77.5	77.5	94.55
39	14.1	73	63.0	63.0	92.73
40	62.3	40	62.3	62.3	90.91
41	38.1	82	59.2	59.2	89.09
42	15.4	83	57.1	57.1	87.27
43	52.0	58	56.2	56.2	85.45
44	3.6	43	52.0	52.0	83.64
45	25.3	52	43.8	43.8	81.82
46	0.2	50	41.0	41.0	80.00
47	9.5	90	40.2	40.2	78.18
48	25.2	41	38.1	38.1	76.36
49	0.6	55	37.7	37.7	74.55
50	41.0	64	31.1	31.1	72.73
51	21.5	89	29.8	29.8	70.91
52	43.8	78	28.2	28.2	69.09
53	12.5	36	27.2	27.2	67.27
54	17.9	69	26.1	26.1	65.45
55	37.7	45	25.3	25.3	63.64
56	3.6	48	25.2	25.2	61.82
57	10.3	38	24.5	24.5	60.00
58	56.2	67	24.2	24.2	58.18
59	17.6	51	21.5	21.5	56.36
60	0.0	70	21.0	21.0	54.55
61	0.0	54	17.9	17.9	52.73
62	119.9	59	17.6	17.6	50.91
63	12.2	42	15.4	15.4	49.09
64	31.1	80	14.9	14.9	47.27
65	6.9	39	14.1	14.1	45.45
66	0.9	53	12.5	12.5	43.64
67	24.2	63	12.2	12.2	41.82
68	0.0	57	10.3	10.3	40.00
69	26.1	79	10.0	10.0	38.18
70	21.0	47	9.5	9.5	36.36
71	5.1	88	9.1	9.1	34.55
72	0.0	75	8.5	8.5	32.73
73	63.0	87	7.6	7.6	30.91
74	1.2	85	7.0	7.0	29.09
75	8.5	65	6.9	6.9	27.27
76	0.0	71	5.1	5.1	25.45
77	0.0	81	4.5	4.5	23.64
78	28.2	84	3.8	3.8	21.82
79	10.0	44	3.6	3.6	20.00
80	14.9	56	3.6	3.6	18.18
81	4.5	74	1.2	1.2	16.36
82	59.2	66	0.9	0.9	14.55
83	57.1	49	0.6	0.6	12.73

84	3.3	46	0.2	0.2	84	2	71	1	84	3.8	46	0.2	0.2	10.91
85	6.8	61	0.0	0.0	85	2	75	1	85	7.0	61	0.0	0.0	9.09
86	64.9	60	0.0	0.0	86	4	81	1	86	94.6	60	0.0	0.0	7.27
87	3.1	60	0.0	0.0	87	4	81	0	87	7.6	60	0.0	0.0	5.45
[illegible]	4.6	60	0.0	0.0	88	2	81	0	88	9.1	60	0.0	0.0	3.64
[illegible]	20.1	60	0.0	0.0	89	4	81	0	89	29.8	60	0.0	0.0	1.82
90	23.6	60	0.0	0.0	90	2	81	0	90	40.2	60	0.0	0.0	0.00

<<< SUMMARY DATA FOR CONDUIT 1225 >>>

NUMBER OF CSO VOLUMES IN EACH CLASS

CLASS (MG)

YEAR	4.200	4.40	4.80	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	NO. EVENTS
36	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	3
37	0	0	0	0	0	0	0	0	1	0	0	0	0	1	0	3	5
38	2	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	4
39	0	0	0	1	0	0	0	0	0	0	0	0	1	0	0	0	2
40	3	0	0	0	0	0	1	0	0	0	0	0	0	0	1	2	7
41	3	0	1	0	0	0	0	0	1	0	0	0	0	0	0	1	6
42	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2
43	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	2
44	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
45	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4
46	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
47	2	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	3
48	1	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	3
49	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
50	2	0	0	0	0	0	0	0	0	0	0	0	1	0	1	1	5
51	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3
52	1	1	0	0	1	1	0	0	1	0	0	0	0	0	0	1	6
53	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2
[illegible]	2	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	4
[illegible]	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	4
56	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4
57	2	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	3
58	2	0	0	1	0	0	0	0	0	1	0	0	0	0	0	2	6
59	2	1	0	0	0	0	0	0	0	0	0	0	1	0	0	0	4
60	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
61	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
62	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
63	3	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	4
64	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3
65	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	1
66	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
67	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
68	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
69	3	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	5
70	2	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1	4
71	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	1
72	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
73	2	0	0	0	0	0	0	1	0	0	1	0	0	0	0	2	6
74	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
75	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1
76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
77	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
78	2	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	4
79	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5
80	3	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	4
81	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1
[illegible]	3	0	0	0	0	1	0	0	0	0	0	0	1	1	0	1	7
[illegible]	4	0	0	0	1	0	0	0	0	0	1	0	0	1	0	1	8
84	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
85	1	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	2
86	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	4
87	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4

88	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	2
89	2	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	4
90	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2
C	85.	2.	4.	2.	4.	4.	1.	4.	4.	3.	6.	1.	5.	4.	2.	37.	168
VE NO. EVENTS	1.55	0.04	0.07	0.04	0.07	0.07	0.02	0.07	0.07	0.05	0.11	0.02	0.09	0.07	0.04	0.67	3.05
UMULATIVE NO. EVENTS	3.05	1.51	1.47	1.40	1.36	1.29	1.22	1.20	1.13	1.05	1.00	0.89	0.87	0.78	0.71	0.67	
XCEEDENCE CURVE: PERCENT	100.0	49.4	48.2	45.8	44.6	42.3	39.9	39.3	36.9	34.5	32.7	29.2	28.6	25.6	23.2	22.0	
VE VOL (MG)	1.54	4.31	4.58	4.84	5.20	5.83	6.13	6.89	7.19	7.78	8.25	8.51	9.33	9.71	10.38	23.31	

YEAR	ANNUAL CSO VOLUME (MG)
36	27.15
37	77.48
38	24.47
39	14.12
40	62.27
41	38.09
42	15.37
43	52.01
44	3.64
45	25.32
46	0.17
	9.50
	25.23
49	0.60
50	41.01
51	21.50
52	43.83
53	12.47
54	17.91
55	37.67
56	3.64
57	10.34
58	56.19
59	17.65
60	0.00
61	0.02
62	119.94
63	12.17
64	31.14
65	6.90
66	0.92
67	24.16
68	0.00
69	26.09
70	20.99
71	5.10
72	0.00
73	62.99
74	1.24
	8.51
	0.00
77	0.00
78	28.20
79	10.04
80	14.91

81	4.49
82	59.20
83	57.11
84	3.85
[illegible]	7.01
[illegible]	94.61
87	7.65
88	9.12
89	29.75
90	40.15
TOTAL CSO VOLUME	1323.92
AVE ANNUAL VOLUME	24.07

MONTHLY CSO VOLUMES (MG)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
36	0.0	27.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	0.0	32.8	23.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	20.9
38	8.5	14.9	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	4.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.3
40	0.0	43.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	18.4
41	4.7	6.7	19.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	7.4
42	0.0	0.0	12.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.1
43	52.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
[illegible]	3.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
[illegible]	18.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.3	0.0	4.1
46	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0
47	0.0	8.1	1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
48	0.0	0.0	0.0	0.0	16.2	0.0	0.0	0.0	0.0	8.4	0.0	0.6
49	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.0
50	0.0	2.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.4	28.2	0.6
51	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	21.5	0.0
52	11.6	0.0	5.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.4	20.6
53	0.0	0.0	0.0	12.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
54	7.8	7.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5	0.9
55	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	37.7
56	1.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	2.5	0.0	0.0	0.0
57	0.0	0.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.6
58	7.8	6.7	41.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59	0.0	8.3	0.0	0.0	0.0	0.0	0.0	0.0	9.3	0.0	0.0	0.0
60	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
61	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
62	0.0	17.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	102.5	0.0	0.0
63	0.2	0.0	3.9	2.1	0.0	0.0	0.0	0.0	0.0	6.0	0.0	0.0
64	31.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
65	6.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
66	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.9	0.0
67	24.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
68	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
69	14.5	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	11.3
70	6.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.2	0.0
71	0.0	0.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
[illegible]	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
[illegible]	14.4	38.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.1	0.0
74	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2
75	0.0	0.0	8.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
76	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
77	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

78	26.0	0.0	2.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
79	3.7	0.0	2.8	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0	0.0
80	3.1	11.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
81	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.5	0.0	0.0
	29.3	4.0	9.6	0.3	0.0	0.0	0.0	0.0	0.0	0.8	15.2	0.0
	9.1	0.0	23.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.8	9.7
84	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	3.3	0.0
85	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.8	0.0
86	0.7	93.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
87	0.0	1.6	0.9	0.0	0.0	0.0	0.0	0.0	0.0	2.0	3.1	0.0
88	4.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.5	0.0
89	0.0	0.0	1.1	0.0	0.0	0.0	0.0	0.0	20.1	8.6	0.0	0.0
90	23.6	16.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AVERAGE MONTHLY VOLUMES	5.8	6.3	3.0	0.3	0.3	0.0	0.0	0.0	0.6	2.7	2.4	2.8

MONTHLY NUMBER OF EVENTS

36	0	3	0	0	0	0	0	0	0	0	0	0
37	0	2	1	0	0	0	0	0	0	0	0	2
38	2	1	1	0	0	0	0	0	0	0	0	0
39	1	0	0	0	0	0	0	0	0	0	0	1
40	0	4	0	0	0	0	0	0	0	0	0	3
41	1	2	1	0	0	0	0	0	0	0	0	2
42	0	0	1	0	0	0	0	0	0	0	0	1
43	2	0	0	0	0	0	0	0	0	0	0	0
44	1	0	0	0	0	0	0	0	0	0	0	0
45	1	0	0	0	0	0	0	0	0	1	0	2
46	0	0	0	0	0	0	0	0	0	0	1	0
47	0	1	2	0	0	0	0	0	0	0	0	0
48	0	0	0	0	1	0	0	0	0	1	0	1
	0	0	0	0	0	0	0	0	0	0	1	0
	0	1	0	0	0	0	0	0	0	1	2	1
51	0	0	0	0	0	0	0	0	0	0	3	0
52	2	0	1	0	0	0	0	0	0	0	2	1
53	0	0	0	1	0	0	0	1	0	0	0	0
54	1	1	0	0	0	0	0	0	0	0	1	1
55	0	0	0	0	0	0	0	0	0	0	0	4
56	2	1	0	0	0	0	0	0	1	0	0	0
57	0	1	0	0	0	0	0	0	0	0	0	2
58	1	3	2	0	0	0	0	0	0	0	0	0
59	1	2	0	0	0	0	0	0	1	0	0	0
60	0	1	0	0	0	0	0	0	0	0	0	0
61	0	0	1	0	0	0	0	0	0	0	1	0
62	0	1	0	0	0	0	0	0	0	1	0	0
63	1	0	1	1	0	0	0	0	0	1	0	0
64	2	0	0	0	0	0	0	0	0	0	0	1
65	1	0	0	0	0	0	0	0	0	0	0	0
66	0	0	0	0	0	0	0	0	0	0	1	0
67	1	0	0	0	0	0	0	0	0	0	0	0
68	0	0	0	0	0	0	0	0	0	0	0	0
69	2	1	0	0	0	0	0	0	0	0	0	2
70	1	0	0	0	0	0	0	0	0	0	3	0
71	0	0	1	0	0	0	0	0	0	0	0	0
72	0	0	0	0	0	0	0	0	0	0	0	0
73	2	2	0	0	0	0	0	0	0	0	2	0
74	0	0	0	0	0	0	0	0	0	0	0	2
75	0	0	1	0	0	0	0	0	0	0	0	0
76	0	0	0	0	0	0	0	0	0	0	0	0
	0	0	0	0	0	0	0	0	0	0	0	0
	3	0	1	0	0	0	0	0	0	0	0	0
79	2	0	2	0	0	0	0	0	0	1	0	0
80	1	3	0	0	0	0	0	0	0	0	0	0
81	0	0	0	0	0	0	0	0	0	1	0	0
82	1	1	1	1	0	0	0	0	0	1	2	0

83	3	0	1	0	0	0	0	0	0	0	3	1
84	0	0	0	0	0	0	0	0	0	1	1	0
85	0	1	0	0	0	0	0	0	0	0	1	0
86	2	2	0	0	0	0	0	0	0	0	0	0
	0	1	1	0	0	0	0	0	0	1	1	0
	1	0	0	0	0	0	0	0	0	0	1	0
89	0	0	1	0	0	0	0	0	1	2	0	0
90	1	1	0	0	0	0	0	0	0	0	0	0
VERAGE MONTHLY NO. EVENTS	0.7	0.7	0.4	0.1	0.0	0.0	0.0	0.0	0.1	0.2	0.5	0.5

ANNUAL MAXIMUM CSO EVENT

YEAR	MG	RANKED YEAR	RANKED MG	MG
36	13.7	62	102.5	102.5
37	25.6	86	64.9	64.9
38	14.9	43	42.1	42.1
39	9.3	64	30.9	30.9
40	27.5	73	30.2	30.2
41	19.2	82	29.3	29.3
42	12.2	40	27.5	27.5
43	42.1	58	27.2	27.2
44	3.6	37	25.6	25.6
45	18.9	67	24.2	24.2
46	0.2	90	23.6	23.6
47	8.1	83	23.5	23.5
48	16.2	52	20.6	20.6
49	0.6	55	20.5	20.5
	17.8	89	20.1	20.1
	16.2	41	19.2	19.2
52	20.6	45	18.9	18.9
53	12.5	50	17.8	17.8
54	7.8	78	16.6	16.6
55	20.5	48	16.2	16.2
56	2.5	51	16.2	16.2
57	8.2	38	14.9	14.9
58	27.2	36	13.7	13.7
59	9.3	69	13.1	13.1
60	0.0	53	12.5	12.5
61	0.0	42	12.2	12.2
62	102.5	70	11.3	11.3
63	6.0	59	9.3	9.3
64	30.9	39	9.3	9.3
65	6.9	75	8.5	8.5
66	0.9	57	8.2	8.2
67	24.2	47	8.1	8.1
68	0.0	54	7.8	7.8
69	13.1	65	6.9	6.9
70	11.3	85	6.8	6.8
71	5.1	80	6.0	6.0
72	0.0	63	6.0	6.0
73	30.2	71	5.1	5.1
74	1.2	88	4.6	4.6
75	8.5	81	4.5	4.5
76	0.0	44	3.6	3.6
77	0.0	79	3.6	3.6
	16.6	84	3.3	3.3
	3.6	87	3.1	3.1
80	6.0	56	2.5	2.5
81	4.5	74	1.2	1.2
82	29.3	66	0.9	0.9
83	23.5	49	0.6	0.6

ANNUAL NUMBER OF CSO EVENTS

YEAR	NO.	RANKED YEAR	RANKED NO.
36	3	83	8
37	5	40	7
38	4	82	7
39	2	41	6
40	7	52	6
41	6	58	6
42	2	73	6
43	2	37	5
44	1	50	5
45	4	69	5
46	1	79	5
47	3	38	4
48	3	45	4
49	1	54	4
50	5	55	4
51	3	56	4
52	6	59	4
53	2	63	4
54	4	70	4
55	4	78	4
56	4	80	4
57	3	86	4
58	6	87	4
59	4	89	4
60	1	36	3
61	2	47	3
62	2	48	3
63	4	51	3
64	3	57	3
65	1	64	3
66	1	39	2
67	1	42	2
68	0	43	2
69	5	53	2
70	4	61	2
71	1	62	2
72	0	74	2
73	6	84	2
74	2	85	2
75	1	88	2
76	0	90	2
77	0	44	1
78	4	46	1
79	5	49	1
80	4	60	1
81	1	65	1
82	7	66	1
83	8	67	1

ANNUAL VOLUME OF CSO EVENTS

YEAR	MG	RANKED YEAR	RANKED MG	PROBABILITY MG	(FOR PLOTTING)
36	27.2	62	119.9	119.9	98.18
37	77.5	86	94.6	94.6	96.36
38	24.5	37	77.5	77.5	94.55
39	14.1	73	63.0	63.0	92.73
40	62.3	40	62.3	62.3	90.91
41	38.1	82	59.2	59.2	89.09
42	15.4	83	57.1	57.1	87.27
43	52.0	58	56.2	56.2	85.45
44	3.6	43	52.0	52.0	83.64
45	25.3	52	43.8	43.8	81.82
46	0.2	50	41.0	41.0	80.00
47	9.5	90	40.2	40.2	78.18
48	25.2	41	38.1	38.1	76.36
49	0.6	55	37.7	37.7	74.55
50	41.0	64	31.1	31.1	72.73
51	21.5	89	29.8	29.8	70.91
52	43.8	78	28.2	28.2	69.09
53	12.5	36	27.2	27.2	67.27
54	17.9	69	26.1	26.1	65.45
55	37.7	45	25.3	25.3	63.64
56	3.6	48	25.2	25.2	61.82
57	10.3	38	24.5	24.5	60.00
58	56.2	67	24.2	24.2	58.18
59	17.6	51	21.5	21.5	56.36
60	0.0	70	21.0	21.0	54.55
61	0.0	54	17.9	17.9	52.73
62	119.9	59	17.6	17.6	50.91
63	12.2	42	15.4	15.4	49.09
64	31.1	80	14.9	14.9	47.27
65	6.9	39	14.1	14.1	45.45
66	0.9	53	12.5	12.5	43.64
67	24.2	63	12.2	12.2	41.82
68	0.0	57	10.3	10.3	40.00
69	26.1	79	10.0	10.0	38.18
70	21.0	47	9.5	9.5	36.36
71	5.1	88	9.1	9.1	34.55
72	0.0	75	8.5	8.5	32.73
73	63.0	87	7.6	7.6	30.91
74	1.2	85	7.0	7.0	29.09
75	8.5	65	6.9	6.9	27.27
76	0.0	71	5.1	5.1	25.45
77	0.0	81	4.5	4.5	23.64
78	28.2	84	3.8	3.8	21.82
79	10.0	44	3.6	3.6	20.00
80	14.9	56	3.6	3.6	18.18
81	4.5	74	1.2	1.2	16.36
82	59.2	66	0.9	0.9	14.55
83	57.1	49	0.6	0.6	12.73

84	3.3	46	0.2	0.2	84	2	71	1	84	3.8	46	0.2	0.2	10.91
85	6.8	61	0.0	0.0	85	2	75	1	85	7.0	61	0.0	0.0	9.09
86	64.9	60	0.0	0.0	86	4	81	1	86	94.6	60	0.0	0.0	7.27
87	3.1	60	0.0	0.0	87	4	81	0	87	7.6	60	0.0	0.0	5.45
	4.6	60	0.0	0.0	88	2	81	0	88	9.1	60	0.0	0.0	3.64
	20.1	60	0.0	0.0	89	4	81	0	89	29.8	60	0.0	0.0	1.82
90	23.6	60	0.0	0.0	90	2	81	0	90	40.2	60	0.0	0.0	0.00

APPENDIX B
HYETOGRAPHS AND TABLES OF LONG-DURATION STORMS

5 YEAR, LONG STORM DURATION EVENT				
HOUR	JAN 20 1943	OCT 10 1962	NOV 24 1970	FEB 13 1980
0	0	0	0	0
1	0.01	0.04	0.01	0.02
2	0.03	0.06	0.02	0
3	0.02	0.06	0.02	0
4	0.06	0	0.02	0
5	0.07	0	0.02	0
6	0.04	0	0.1	0.01
7	0.04	0	0.08	0.01
8	0.07	0	0	0.08
9	0.17	0	0.01	0.09
10	0.14	0	0.01	0.03
11	0.02	0	0	0
12	0.01	0	0.01	0.11
13	0.03	0	0.01	0.13
14	0.03	0	0	0.01
15	0.03	0	0	0.03
16	0.04	0	0.01	0.03
17	0.06	0	0	0.01
18	0.01	0	0.01	0.04
19	0.01	0	0	0.05
20	0.01	0.01	0	0.02
21	0	0.01	0.03	0
22	0.04	0.04	0.04	0
23	0.15	0.04	0	0
24	0.06	0.02	0	0.01
25	0.1	0	0	0
26	0.17	0	0	0
27	0.14	0	0	0.01
28	0.11	0.01	0	0.12
29	0.12	0.02	0	0.09
30	0.09	0	0	0.06
31	0.13	0	0	0
32	0.15	0.18	0	0.01
33	0.04	0.2	0	0.01
34	0.17	0.11	0.41	0.01
35	0.45	0.04	0.02	0.15
36	0.55	0.11	0	0.15
37	0.53	0.1	0	0.1
38	0.36	0.07	0	0.08
39	0.03	0.03	0	0
40	0	0.06	0	0
41	0	0.02	0	0.17
42	0	0.04	0	0.11
43	0	0.01	0	0.01
44	0	0.01	0	0
45	0	0.03	0	0
46	0	0.02	0	0
47	0	0.02	0	0
48	0	0	0	0

5 YEAR, LONG STORM DURATION EVENT				
HOUR	JAN 20 1943	OCT 10 1962	NOV 24 1970	FEB 13 1980
49	0	0	0	0
50	0.02	0.02	0	0.02
51	0	0.17	0	0.03
52	0	0.01	0	0
53	0	0.04	0	0
54	0	0.03	0	0
55	0	0.04	0	0
56	0	0.57	0	0
57	0	0.27	0	0
58	0	0.21	0	0
59	0	0.17	0	0.03
60	0	0.16	0	0.07
61	0	0.17	0	0.05
62	0	0.11	0	0.04
63	0.02	0.11	0	0.12
64	0.02	0.17	0	0.06
65	0.21	0.18	0	0.05
66	0.12	0.22	0	0
67	0.19	0.18	0	0
68	0.38	0.14	0	0.09
69	0.08	0.29	0	0.01
70	0.11	0.19	0.01	0.13
71	0.01	0.2	0	0.14
72	0	0.16	0.01	0.02
73	0	0.16	0.02	0.13
74	0	0.16	0	0.02
75	0.02	0.3	0.01	0
76	0	0.55	0.01	0.06
77	0	0.21	0	0.02
78	0	0.15	0	0.01
79	0	0.03	0.01	0.01
80	0	0.05	0	0
81	0	0.01	0.02	0
82	0.01	0.01	0.05	0
83	0.02	0.02	0.23	0
84	0	0.03	0.3	0
85	0	0	0.19	0.02
86	0		0.09	0.09
87	0	6.85	0.08	0.1
88	0		0.02	0.05
89	0		0.01	0.06
90	0		0	0.08
91	0		0	0
92	0		0	0.26
93	0		0	0.21
94	0		0.03	0.17
95	0		0.04	0.08
96	0		0.04	0.05
97	0		0.21	0

5 YEAR, LONG STORM DURATION EVENT				
HOUR	JAN 20 1943	OCT 10 1962	NOV 24 1970	FEB 13 1980
98	0		0.06	0
99	0		0.12	0
100	0		0.07	0
101	0		0.01	0
102	0		0.08	0
103	0		0.15	0.01
104	0		0.17	0
105	0		0.25	0
106	0		0.12	0
107	0		0.01	0.05
108	0		0.03	0.01
109	0		0.15	0.17
110	0		0.14	0.09
111	0		0.16	0.01
112	0		0.23	0.01
113	0		0.2	0
114	0		0.12	0
115	0		0.01	0
116	0		0.04	0.05
117	0		0.01	0
118	0		0	0
119	0		0	0
120	0		0	0
121	0		0.01	0.02
122	0		0.01	0.03
123	0		0	0.04
124	0		0	0.23
125	0		0	0.17
126	0		0	0.02
127	0.03		0	0.08
128	0.05		0	0.11
129	0		0	0.05
130	0.04		0	0.4
131	0.02		0	0.1
132	0		0	0.11
133	0		0	0
134	0		0	0.01
135	0		0	0.01
136	0		0.03	0
137	0		0.02	0
138	0		0.02	0
139	0.01		0	0
140	0		0	0
141	0		0.01	0
142	0		0	0
143	0		0.02	0
144	0		0.09	0
145	0		0.27	0
146	0		0.22	0

5 YEAR, LONG STORM DURATION EVENT				
HOUR	JAN 20 1943	OCT 10 1962	NOV 24 1970	FEB 13 1980
147	0		0	0
148	0.01		0.06	0
149	0		0	0
150	0.04		0	0
151	0		0	0
152	0		0	0
153	0		0	0
154	0.01		0	0.01
155	0		0	0.01
156	0		0	0.02
157	0		0	0
158	0		0	0
159	0		0	0
160	0		0	0
161	0		0	0
162	0		0	0
163	0		0	0.04
164	0		0	0.08
165	0		0	0.04
166	0.02		0	0.08
167	0		0	0.07
168	0.01		0	0.2
169	0		0.02	0.02
170	0		0	0
171	0		0	0.06
172	0		0	0.08
173	0		0.33	0.03
174	0		0.06	0.03
175	0		0.01	0.02
176	0		0.02	0.04
177	0		0.03	0.24
178	0		0.02	0.08
179	0		0.09	0
180	0		0.25	0
181	0		0.03	0
182	0		0	0
183	0		0	0
184	0		0	0
185	0		0	0
186	0		0	0
187	0		0.01	0
188	0		0	0
189	0		0	0
190	0		0	0
191	0		0	0
192	0		0	0
193	0		0	0
194	0		0	0
195	0		0	0

5 YEAR, LONG STORM DURATION EVENT				
HOUR	JAN 20	OCT 10	NOV 24	FEB 13
	1943	1962	1970	1980
196	0		0	0
197	0		0	0
198	0		0	0
199	0		0	0
200	0		0	0
201	0		0	0.05
202	0		0	0.06
203	0		0	0.01
204	0		0	0
205	0		0	0.03
206	0		0	0
207	0		0	
208	0		0	7.12
209	0		0	
210	0		0	
211	0		0	
212	0		0	
213	0		0.01	
214	0		0	
215	0		0	
216	0		0	
217	0		0	
218	0.01		0	
219	0.07		0.03	
220	0.07		0.04	
221	0.01		0.02	
222	0.02		0.02	
223	0.03		0.05	
224	0.04		0.07	
225	0		0.08	
226	0.01		0.06	
227	0		0.05	
228	0.02		0.01	
229	0.02		0.01	
230	0.23		0	
231	0.02		0.11	
232	0.08		0.15	
233	0.03		0.06	
234	0.02		0.02	
235	0.01		0.03	
236	0.03		0.03	
237	0.07		0.02	
238	0.01		0.01	
239	0.01		0.02	
240	0		0.01	
241	0.03		0.02	
242	0.06		0	
243	0.05		0	
244	0.22		0	

5 YEAR, LONG STORM DURATION EVENT				
HOUR	JAN 20 1943	OCT 10 1962	NOV 24 1970	FEB 13 1980
245	0.08		0	
246	0.01		0	
247	0		0	
248	0		0	
249	0.02		0	
250	0		0	
251			0	
252	7.02		0	
253			0	
254			0	
255			0	
256			0.01	
257			0	
258			0	
259			0	
260			0	
261			0	
262			0	
263			0	
264			0	
265			0	
266			0	
267			0	
268			0	
269			0	
270			0	
271			0	
272			0	
273			0	
274			0	
275			0	
276			0	
277			0	
278			0	
279			0	
280			0.01	
281			0	
282			0	
283			0	
284			0	
285			0	
286			0	
287			0	
288			0	
289			0	
290			0	
291			0	
292			0	
293			0	

5 YEAR, LONG STORM DURATION EVENT				
HOUR	JAN 20 1943	OCT 10 1962	NOV 24 1970	FEB 13 1980
294			0	
295			0	
296			0.01	
297			0.06	
298			0.04	
299			0	
300			0	
301			0	
302			0	
303			0	
304			0	
305			0	
306			0	
307			0	
308			0	
309			0	
310			0	
311			0	
312			0	
313			0.01	
314			0	
315			0	
316			0	
317			0	
318			0	
319			0	
320			0	
321			0	
322			0	
323			0	
324			0	
325			0	
326			0	
327			0	
328			0	
329			0	
330			0	
331			0.01	
332			0	
333			0.02	
334			0.01	
335			0	

TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936	1938	1940	1962	1967
	FEB10	JAN28	DEC16	FEB6	JAN20
0	0.00	0.00	0.00	0.00	0.00
1	0.02	0.03	0.01	0.05	0.01
2	0.15	0.00	0.02	0.09	0.00
3	0.11	0.02	0.00	0.11	0.05
4	0.11	0.01	0.02	0.03	0.06
5	0.11	0.02	0.02	0.08	0.06
6	0.22	0.04	0.03	0.00	0.09
7	0.08	0.01	0.02	0.00	0.03
8	0.21	0.00	0.04	0.00	0.02
9	0.26	0.00	0.02	0.00	0.03
10	0.05	0.00	0.04	0.00	0.04
11	0.06	0.00	0.05	0.00	0.04
12	0.09	0.00	0.00	0.00	0.04
13	0.06	0.00	0.01	0.00	0.05
14	0.13	0.00	0.00	0.00	0.06
15	0.24	0.00	0.00	0.00	0.11
16	0.38	0.00	0.00	0.00	0.05
17	0.05	0.00	0.00	0.00	0.02
18	0.01	0.00	0.00	0.00	0.08
19	0.02	0.03	0.00	0.00	0.12
20	0.00	0.01	0.00	0.00	0.13
21	0.00	0.00	0.01	0.00	0.13
22	0.00	0.00	0.09	0.07	0.09
23	0.03	0.00	0.12	0.09	0.06
24	0.27	0.00	0.14	0.08	0.00
25	0.00	0.00	0.04	0.11	0.03
26	0.01	0.00	0.00	0.11	0.15
27	0.04	0.00	0.03	0.06	0.13
28	0.02	0.00	0.00	0.03	0.06
29	0.00	0.00	0.01	0.00	0.07
30	0.03	0.00	0.00	0.00	0.08
31	0.02	0.00	0.00	0.00	0.16
32	0.01	0.00	0.00	0.00	0.23
33	0.03	0.00	0.02	0.00	0.15
34	0.01	0.00	0.02	0.00	0.21
35	0.19	0.00	0.00	0.00	0.14
36	0.63	0.00	0.00	0.00	0.06
37	0.05	0.00	0.00	0.00	0.01
38	0.00	0.00	0.00	0.00	0.02
39	0.00	0.00	0.01	0.00	0.08
40	0.10	0.00	0.02	0.00	0.26
41	0.00	0.00	0.00	0.00	0.59
42	0.00	0.00	0.00	0.00	0.18
43	0.05	0.00	0.00	0.00	0.10
44	0.00	0.00	0.00	0.00	0.01
45	0.00	0.00	0.00	0.00	0.00
46	0.00	0.00	0.00	0.00	0
47	0.00	0.00	0.01	0.00	0
48	0.07	0.00	0.03	0.00	0

TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936 FEB10	1938 JAN28	1940 DEC16	1962 FEB6	1967 JAN20
49	0.10	0.00	0.04	0.00	0
50	0.15	0.00	0.00	0.00	0
51	0.10	0.00	0.02	0.00	0
52	0.01	0.00	0.06	0.00	0
53	0.01	0.00	0.01	0.00	0
54	0.00	0.00	0.12	0.00	0
55	0.01	0.00	0.05	0.00	0
56	0.00	0.00	0.14	0.00	0
57	0.01	0.00	0.07	0.00	0
58	0.00	0.00	0.01	0.00	0
59	0.00	0.00	0.28	0.00	0
60	0.00	0.00	0.10	0.00	0
61	0.00	0.00	0.00	0.00	0
62	0.00	0.00	0.00	0.00	0
63	0.00	0.00	0.00	0.00	0
64	0.00	0.03	0.00	0.00	0
65	0.00	0.04	0.00	0.00	0
66	0.00	0.04	0.00	0.00	0
67	0.00	0.01	0.00	0.03	0
68	0.03	0.02	0.00	0.07	0
69	0.01	0.03	0.00	0.18	0
70	0.00	0.03	0.00	0.14	0.00
71	0.03	0.09	0.00	0.10	0.00
72	0.05	0.06	0.00	0.13	0.00
73	0.06	0.08	0.00	0.14	0.00
74	0.05	0.11	0.00	0.10	0.00
75	0.00	0.07	0.00	0.02	0.00
76	0.01	0.11	0.00	0.02	0.00
77	0.01	0.09	0.00	0.00	0.00
78	0.01	0.17	0.00	0.00	0.00
79	0.02	0.07	0.00	0.00	0.00
80	0.00	0.05	0.00	0.01	0.00
81	0.00	0.04	0.00	0.00	0.00
82	0.00	0.09	0.00	0.00	0.00
83	0.00	0.03	0.00	0.13	0.00
84	0.00	0.00	0.00	0.19	0.00
85	0.00	0.03	0.00	0.53	0.00
86	0.00	0.09	0.00	0.30	0.00
87	0.00	0.07	0.00	0.03	0.00
88	0.00	0.00	0.00	0.00	0.00
89	0.00	0.00	0.00	0.00	0.00
90	0.00	0.00	0.00	0.09	0.00
91	0.00	0.00	0.00	0.05	0.00
92	0.00	0.06	0.00	0.03	0.03
93	0.02	0.01	0.00	0.07	0.11
94	0.00	0.01	0.00	0.09	0.08
95	0.03	0.02	0.00	0.06	0.13
96	0.04	0.01	0.00	0.05	0.11
97	0.05	0.00	0.00	0.01	0.13

TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936 FEB10	1938 JAN28	1940 DEC16	1962 FEB6	1967 JAN20
98	0.04	0.01	0.00	0.11	0.04
99	0.01	0.00	0.00	0.08	0.02
100	0.00	0.00	0.00	0.00	0.09
101	0.00	0.00	0.00	0.00	0.01
102	0.00	0.00	0.00	0.02	0.16
103	0.00	0.04	0.00	0.05	0.04
104	0.00	0.00	0.00	0.14	0.00
105	0.00	0.00	0.00	0.02	0.00
106	0.00	0.00	0.00	0.05	0.13
107	0.00	0.00	0.00	0.07	0.14
108	0.00	0.00	0.00	0.05	0.00
109	0.00	0.00	0.00	0.16	0.15
110	0.00	0.00	0.00	0.08	0.05
111	0.00	0.00	0.00	0.02	0.00
112	0.00	0.05	0.00	0.02	0.01
113	0.00	0.01	0.01	0.00	0.00
114	0.00	0.07	0.00	0.01	0.00
115	0.00	0.01	0.00	0.00	0.00
116	0.00	0.02	0.02	0.00	0.00
117	0.00	0.00	0.06	0.00	0.00
118	0.00	0.00	0.03	0.00	0
119	0.03	0.00	0.08	0.01	0
120	0.04	0.00	0.03	0.00	0
121	0.01	0.00	0.03	0.00	0
122	0.00	0.00	0.00	0.00	0
123	0.01	0.00	0.07	0.00	0
124	0.09	0.00	0.19	0.01	0
125	0.08	0.00	0.12	0.00	0
126	0.00	0.01	0.20	0.00	0
127	0.00	0.06	0.06	0.00	0
128	0.00	0.09	0.01	0.00	0
129	0.00	0.07	0.16	0.00	0
130	0.00	0.54	0.12	0.00	0
131	0.00	0.15	0.02	0.00	0
132	0.00	0.07	0.10	0.00	0
133	0.00	0.06	0.16	0.00	0
134	0.00	0.01	0.23	0.00	0
135	0.00	0.07	0.32	0.00	0
136	0.02	0.01	0.15	0.00	0
137	0.00	0.00	0.00	0.01	0
138	0.00	0.00	0.08	0.01	0
139	0.02	0.16	0.03	0.00	0
140	0.00	0.07	0.16	0.00	0
141	0.00	0.02	0.03	0.00	0
142	0.02	0.01	0.08	0.00	0.00
143	0.00	0.00	0.03	0.00	0.00
144	0.00	0.00	0.04	0.00	0.00
145	0.00	0.00	0.03	0.00	0.00
146	0.02	0.00	0.00	0.00	0.00

TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936	1938	1940	1962	1967
	FEB10	JAN28	DEC16	FEB6	JAN20
147	0.00	0.00	0.00	0.00	0.00
148	0.00	0.00	0.00	0.00	0.00
149	0.00	0.02	0.00	0.00	0.01
150	0.00	0.00	0.02	0.00	0.01
151	0.00	0.00	0.02	0.00	0.03
152	0.00	0.00	0.00	0.00	0.05
153	0.00	0.00	0.00	0.00	0.03
154	0.00	0.00	0.00	0.02	0.04
155	0.00	0.00	0.01	0.03	0.02
156	0.00	0.00	0.00	0.05	0.00
157	0.00	0.00	0.00	0.08	0.00
158	0.00	0.00	0.00	0.04	0.00
159	0.00	0.00	0.00	0.02	0.00
160	0.00	0.01	0.00	0.00	0.00
161	0.00	0.08	0.00	0.00	0.00
162	0.00	0.12	0.00	0.00	0.00
163	0.01	0.33	0.06	0.00	0.00
164	0.00	0.01	0.15	0.03	0.02
165	0.00	0.06	0.19	0.03	0.00
166	0.00	0.00	0.11	0.03	0
167	0.00	0.00	0.21	0.02	0
168	0.00	0.00	0.30	0.03	0
169	0.08	0.00	0.01	0.02	0
170	0.14	0.00	0.04	0.09	0
171	0.21	0.00	0.01	0.07	0
172	0.24	0.00	0.00	0.01	0
173	0.00	0.00	0.00	0.01	0
174	0.00	0.01	0.01	0.01	0
175	0.00	0.03	0.01	0.05	0
176	0.01	0.00	0.03	0.04	0
177	0.01	0.00	0.28	0.09	0
178	0.00	0.00	0.02	0.01	0
179	0.00	0.00	0.00	0.00	0
180	0.00	0.00	0.00	0.00	0
181	0.00	0.00	0.06	0.03	0
182	0.00	0.00	0.00	0.00	0
183	0.00	0.00	0.00	0.00	0
184	0.00	0.00	0.00	0.00	0
185	0.00	0.00	0.00	0.00	0
186	0.00	0.00	0.00	0.00	0
187	0.00	0.01	0.00	0.00	0
188	0.00	0.00	0.00	0.00	0
189	0.00	0.00	0.00	0.00	0
190	0.00	0.00	0.00	0.00	0.01
191	0.00	0.05	0.00	0.00	0.00
192	0.00	0.03	0.00	0.00	0.00
193	0.00	0.06	0.00	0.00	0.05
194	0.00	0.01	0.00	0.00	0.00
195	0.00	0.02	0.00	0.00	0.00

TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936 FEB10	1938 JAN28	1940 DEC16	1962 FEB6	1967 JAN20
196	0.00	0.02	0.25	0.00	0.03
197	0.00	0.00	0.10	0.00	0.13
198	0.01	0.01	0.10	0.02	0.08
199	0.01	0.01	0.01	0.07	0.09
200	0.00	0.03	0.00	0.15	0.06
201	0.02	0.07	0.04	0.12	0.15
202	0.05	0.02	0.00	0.04	0.10
203	0.05	0.00	0.00	0.01	0.03
204	0.02	0.00	0.00	0.00	0.00
205	0.00	0.01	0.00	0.00	0.01
206	0.00	0.01	0.00	0.00	0.00
207	0.00	0.02	0.08	0.11	0.00
208	0.00	0.02	0.00	0.35	0.00
209	0.00	0	0.00	0.04	0.00
210	0.00	0	0.00	0.04	0.00
211	0.00	0	0.00	0.15	0.00
212	0.00	0	0.00	0.15	0.00
213	0.00	0	0.00	0.15	0.00
214	0.00	0	0.00	0.00	0.00
215	0.00	0	0.00	0.00	0.12
216	0.00	0	0.00	0.00	0.15
217	0.00	0	0.00	0.00	0.10
218	0.00	0	0.10	0.04	0.12
219	0.01	0	0.08	0.20	0.02
220	0.00	0	0.16	0.06	0.00
221	0.01	0	0.13	0.06	0.00
222	0.01	0	0.05	0.02	0.00
223	0.00	0	0.06	0.00	0.01
224	0.01	0	0.21	0.01	0.01
225	0.01	0	0.19	0.00	0.01
226	0.00	0	0.17	0.00	0.00
227	0.00	0	0.05	0.00	0.00
228	0.00	0	0.04	0.01	0.00
229	0.00	0	0.01	0.00	0.00
230	0.00	0	0.00	0.00	0.00
231	0.00	0	0.03	0.00	0.00
232	0.00	0	0.01	0.00	0.01
233	0.00	0.00	0.01	0.00	0.03
234	0.00	0.00	0.07	0.00	0.00
235	0.00	0.00	0.05	0.03	0.00
236	0.01	0.00	0.00	0.01	0.00
237	0.04	0.00	0.00	0.00	0.00
238	0.04	0.00	0.00	0.00	0.00
239	0.03	0.00	0.00	0.00	0.00
240	0.10	0.00	0.00	0.02	0.00
241	0.10	0.00	0.00	0.07	0.00
242	0.07	0.00	0.00	0.00	0.02
243	0.04	0.00	0.00	0.00	0.10
244	0.13	0.00	0.00	0.00	0.14

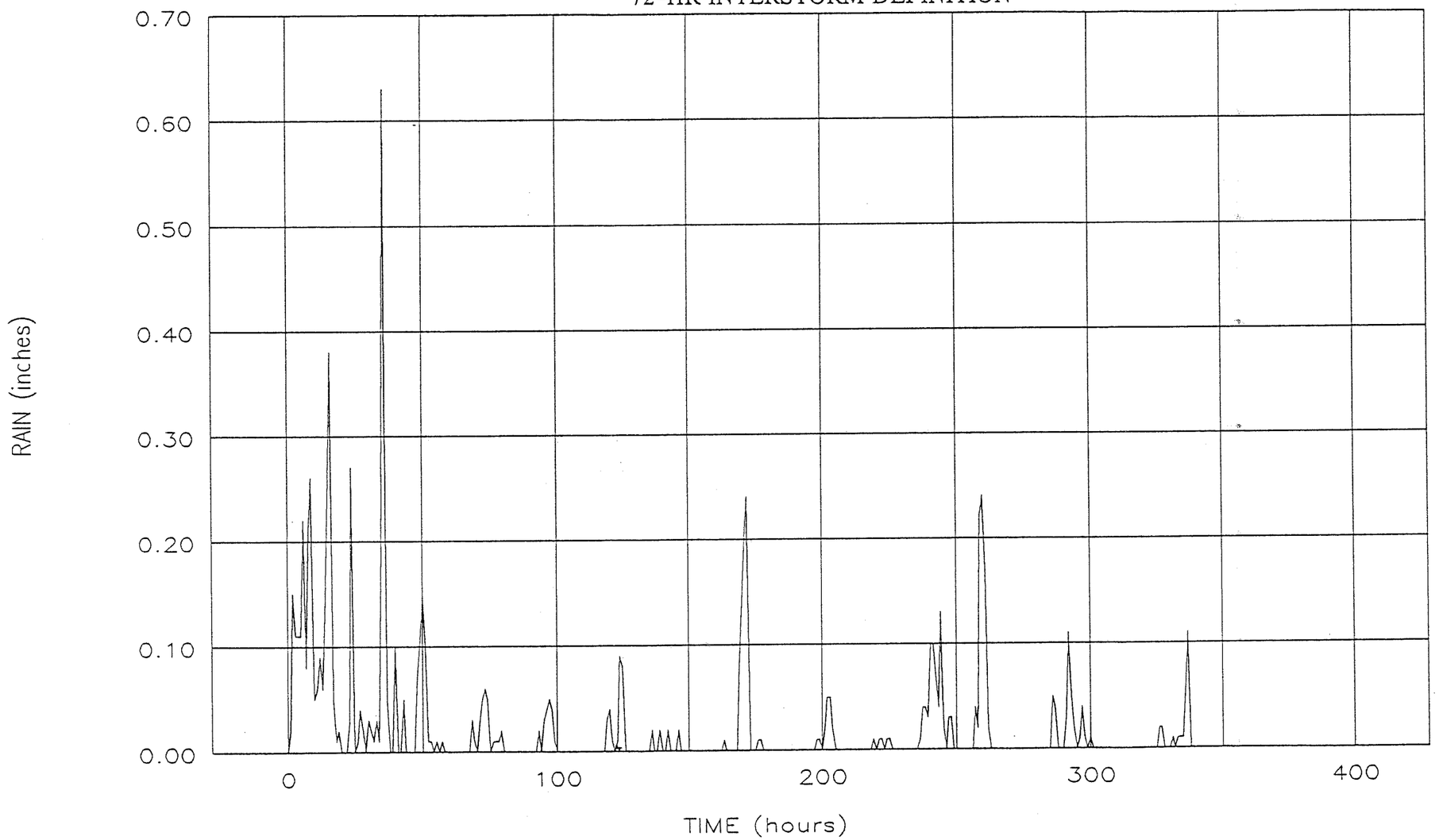
TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936	1938	1940	1962	1967
	FEB10	JAN28	DEC16	FEB6	JAN20
245	0.02	0.00	0.00	0.00	0.09
246	0.00	0.00	0.00	0.00	0.10
247	0.03	0.00	0.00	0.00	0.07
248	0.03	0.00	0.00	0.00	0.05
249	0.00	0.00	0.00	0.00	0.04
250	0.00	0.00	0.00	0.00	0.07
251	0.00	0.00	0.06	0.00	0.01
252	0.00	0.00	0.13	0.00	0.00
253	0.00	0.00	0.15	0.00	0.00
254	0.00	0.01	0.15	0.00	0.00
255	0.00	0.02	0.04	0.00	0.00
256	0.00	0.01	0.00	0.00	0.00
257	0.04	0.00	0.03	0.00	0.00
258	0.02	0.03	0.08	0.00	0.00
259	0.22	0.03	0.03	0.00	0.00
260	0.24	0.02	0.01	0.00	0.02
261	0.17	0.03	0.01	0.00	0.10
262	0.02	0.04	0.01	0.00	0.04
263	0.00	0.02	0.06	0.00	0.01
264	0.00	0.03	0.11	0.00	0.01
265	0.00	0.02	0.04	0.00	0.00
266	0.00	0.02	0.00	0.00	
267	0.00	0.03	0.01	0.00	
268	0.00	0.00	0.01	0.00	
269	0.00	0.01	0.00	0.00	
270	0.00	0.00	0.04	0.00	
271	0.00	0.00	0.00	0.00	
272	0.00	0.00	0.01	0.00	
273	0.00	0.00	0.01	0.00	
274	0.00	0.00	0.01	0.00	
275	0.00	0.02	0.00	0.00	
276	0.00	0.05	0.00	0.00	
277	0.00	0.02	0.00	0.00	
278	0.00	0.01	0.00	0.00	
279	0.00	0.00	0.01	0.00	
280	0.00	0.00	0.01	0.00	
281	0.00	0.00	0.00	0.00	
282	0.00	0.00	0.01	0.00	
283	0.00	0.03	0.00	0.00	
284	0.00	0.03	0.00	0.00	
285	0.00	0.05	0.00	0.00	
286	0.05	0.06	0.03	0.00	
287	0.04	0.08	0.04	0.00	
288	0.00	0.09	0.01	0.00	
289	0.00	0.00	0.00	0.00	
290	0.00	0.00	0.00	0.00	
291	0.03	0.00	0.03	0.00	
292	0.11	0.04	0.01	0.02	
293	0.05	0.00	0.01	0.09	

TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936	1938	1940	1962	1967
	FEB10	JAN28	DEC16	FEB6	JAN20
294	0.02	0.03	0.00	0.05	
295	0.00	0.00	0.01	0.01	
296	0.01	0.02	0.00	0.01	
297	0.04	0.00	0.01	0.00	
298	0.01	0.01	0.00	0.00	
299	0.00	0.00	0.00	0.00	
300	0.01	0.00	0.00	0.02	
301	0.00	0.00	0.00	0.00	
302	0.00	0.00	0.00	0.01	
303	0.00	0.01	0.00	0.00	
304	0.00	0.01	0.00	0.03	
305	0.00	0.01	0.00	0.00	
306	0.00	0.06	0.00	0.00	
307	0.00	0.02	0.00	0.01	
308	0.00	0.02	0.00	0.00	
309	0.00	0.00	0.00	0.00	
310	0.00	0.00	0.00	0.03	
311	0.00	0.00	0.00	0.03	
312	0.00	0.00	0.00	0.00	
313	0.00	0.00	0.00	0.03	
314	0.00	0.04	0.00	0.00	
315	0.00	0.03	0.07		
316	0.00	0.14	0.03	7.48	
317	0.00	0.19	0.00		
318	0.00	0.06	0.00		
319	0.00	0.21	0.00		
320	0.00	0.01	0.01		
321	0.00	0.00	0.00		
322	0.00	0.00	0.01		
323	0.00	0.00	0.00		
324	0.00	0.06	0.00		
325	0.00	0.11	0.00		
326	0.02	0.12	0.00		
327	0.02	0.03	0.00		
328	0.00	0.08	0.00		
329	0.00	0.05	0.00		
330	0.00	0.08	0.00		
331	0.01	0.02	0.00		
332	0.00	0.01	0.00		
333	0.01	0.00	0.05		
334	0.01	0.00	0.07		
335	0.01	0.01	0.04		
336	0.06	0.05	0.00		
337	0.11	0.06			
338	0.00	0.13			
339		0.13			
340		0.14			
341		0.03			
342		0.03			

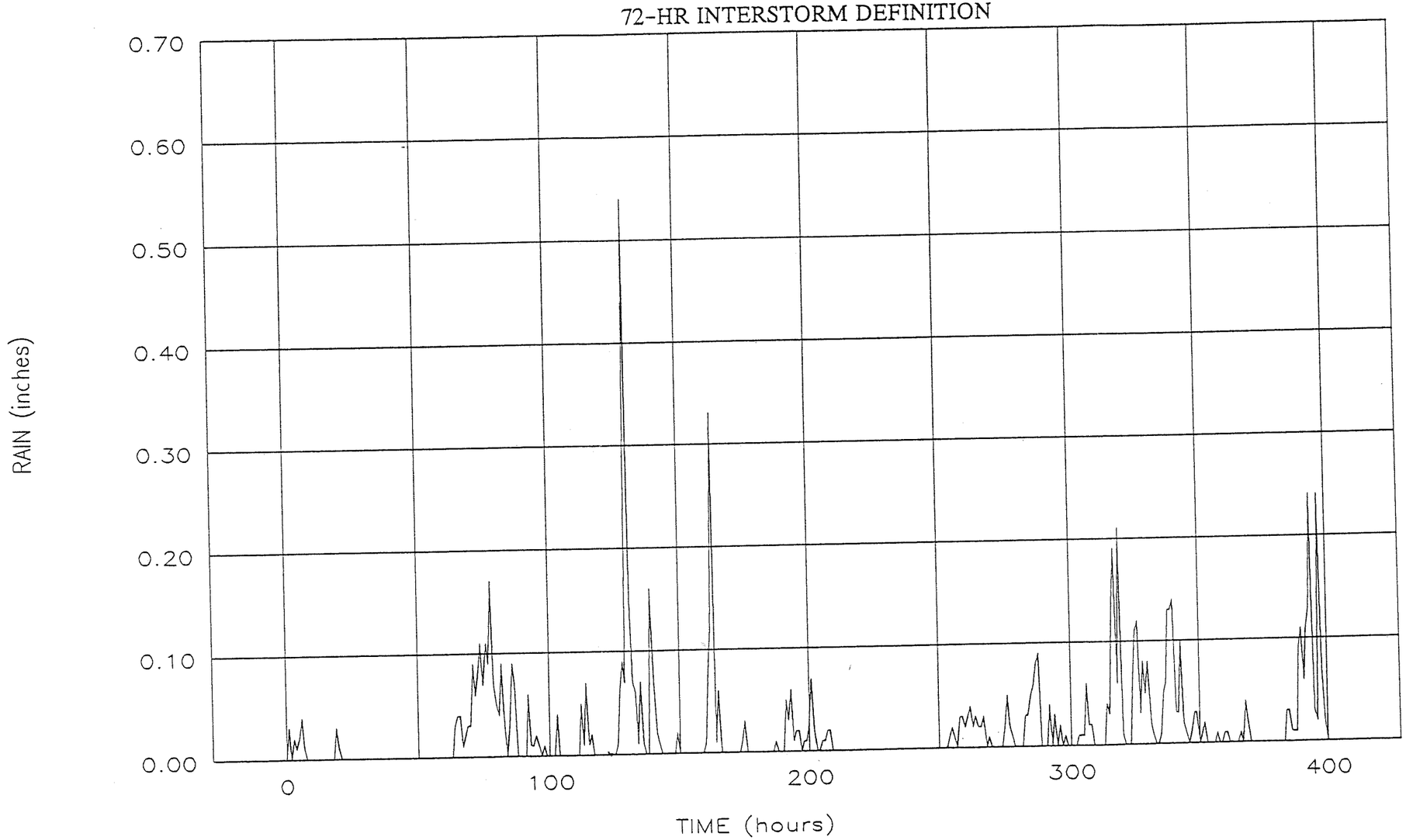
TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936 FEB10	1938 JAN28	1940 DEC16	1962 FEB6	1967 JAN20
343		0.10			
344		0.02			
345		0.01			
346		0.00			
347		0.01			
348		0.03			
349		0.03			
350		0.00			
351		0.01			
352		0.02			
353		0.00			
354		0.00			
355		0.00			
356		0.00			
357		0.01			
358		0.00			
359		0.00			
360		0.01			
361		0.01			
362		0.00			
363		0.00			
364		0.00			
365		0.00			
366		0.01			
367		0.00			
368		0.04			
369		0.02			
370		0.00			
371		0.00			
372		0.00			
373		0.00			
374		0.00			
375		0.00			
376		0.00			
377		0.00			
378		0.00			
379		0.00			
380		0.00			
381		0.00			
382		0.00			
383		0.00			
384		0.03			
385		0.03			
386		0.01			
387		0.01			
388		0.01			
389		0.09			
390		0.11			
391		0.06			

TEN-YEAR, LONG DURATION STORM EVENTS					
HOUR	1936 FEB10	1938 JAN28	1940 DEC16	1962 FEB6	1967 JAN20
392		0.11			
393		0.13			
394		0.24			
395		0.03			
396		0.02			
397		0.24			
398		0.07			
399		0.02			
400		0.00			
401		0.00			
402		0.00			
403		0.01			
404		0.01			
405		0.00			
406		0.00			
407		0.00			
408		0.00			
409		0.00			
410		0.00			
411		0.00			
412		0.00			
413		0.01			
414		0.02			
415		0.00			

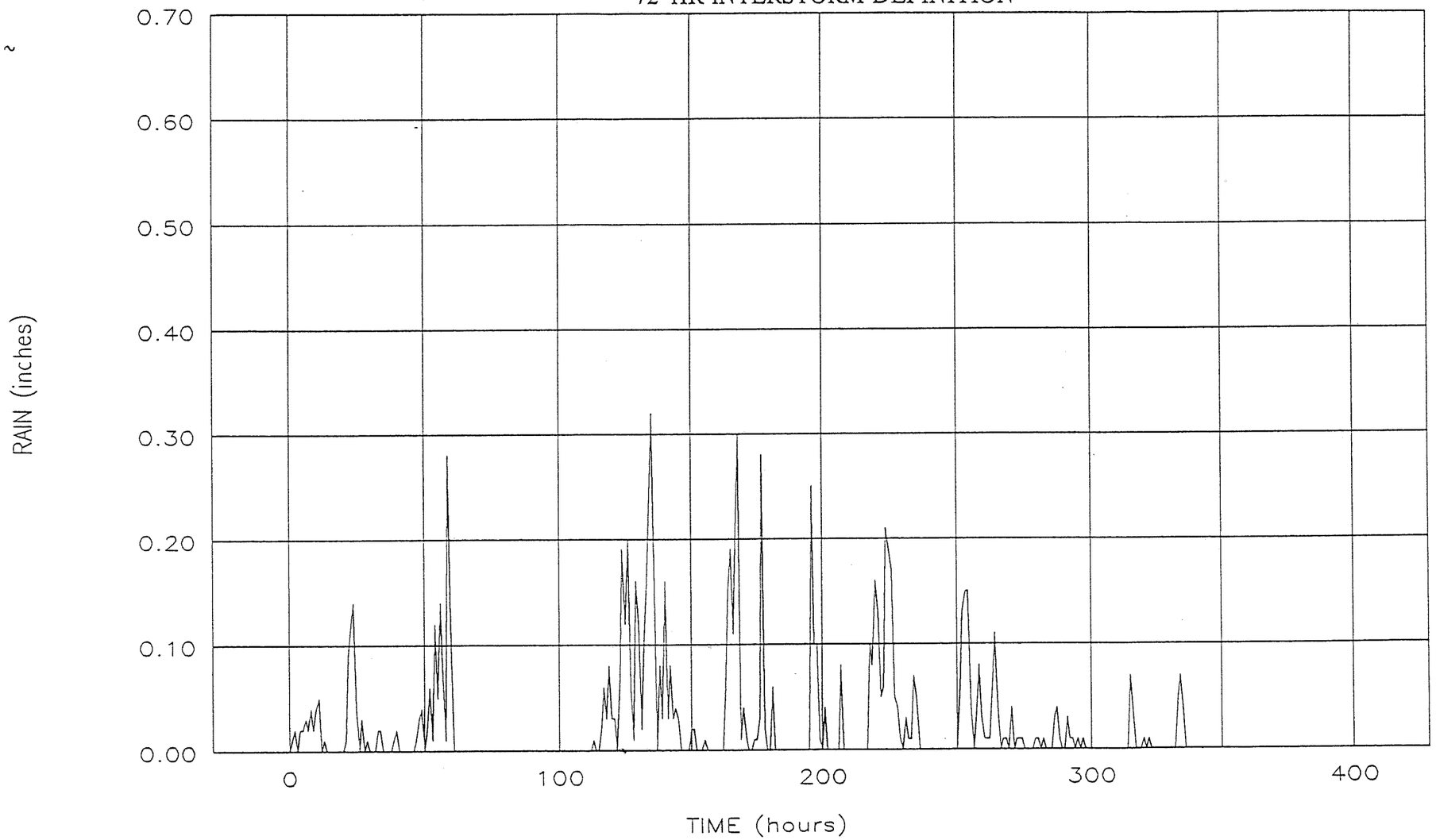
HISTORICAL EVENT FEBUARY 10, 1936
337-HR DURATION, 7.67 INCHES TOTAL DEPTH
72-HR INTERSTORM DEFINITION



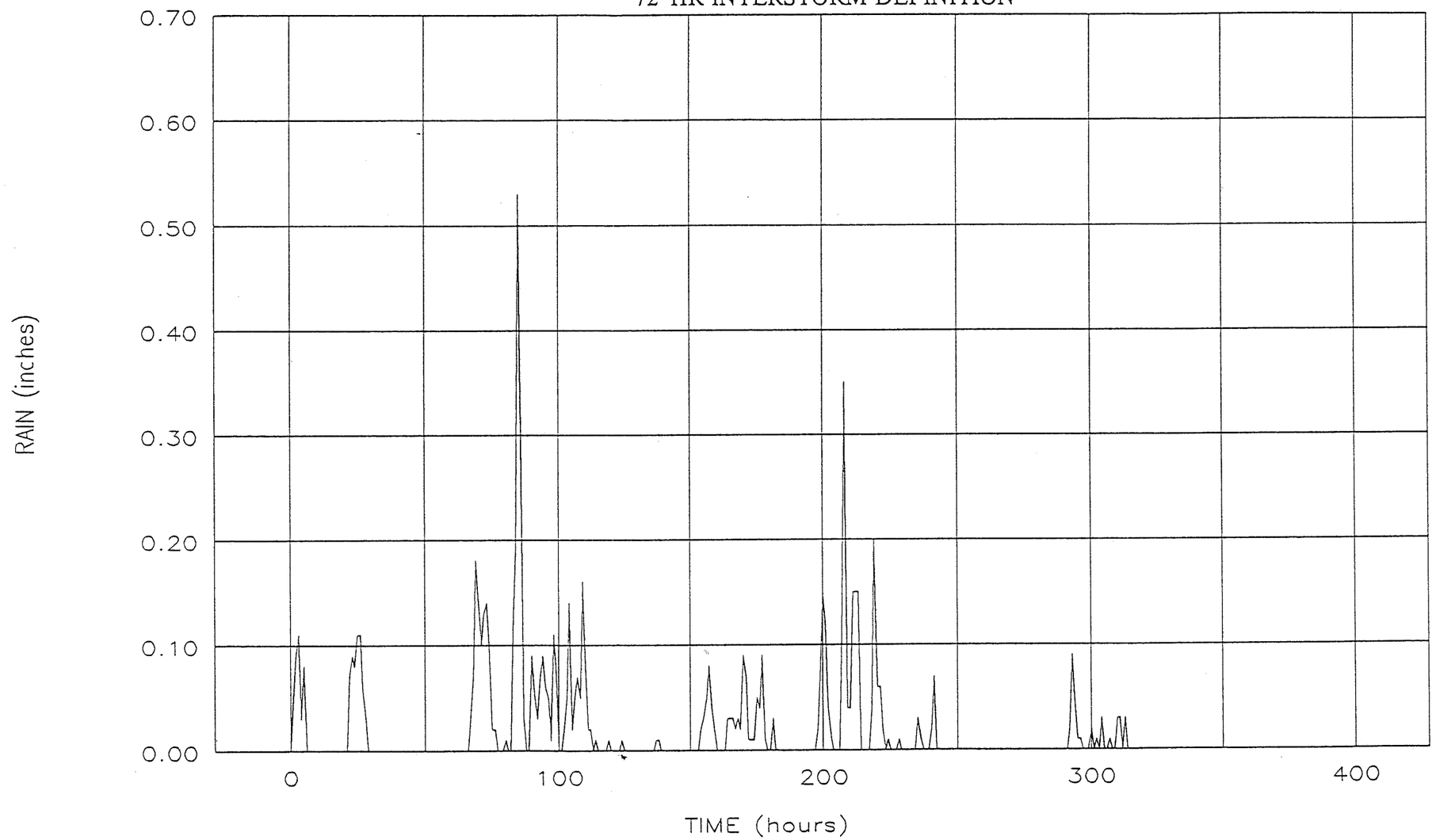
HISTORICAL EVENT JANUARY 20, 1938
414-HR DURATION, 8.34 INCHES TOTAL DEPTH.
72-HR INTERSTORM DEFINITION



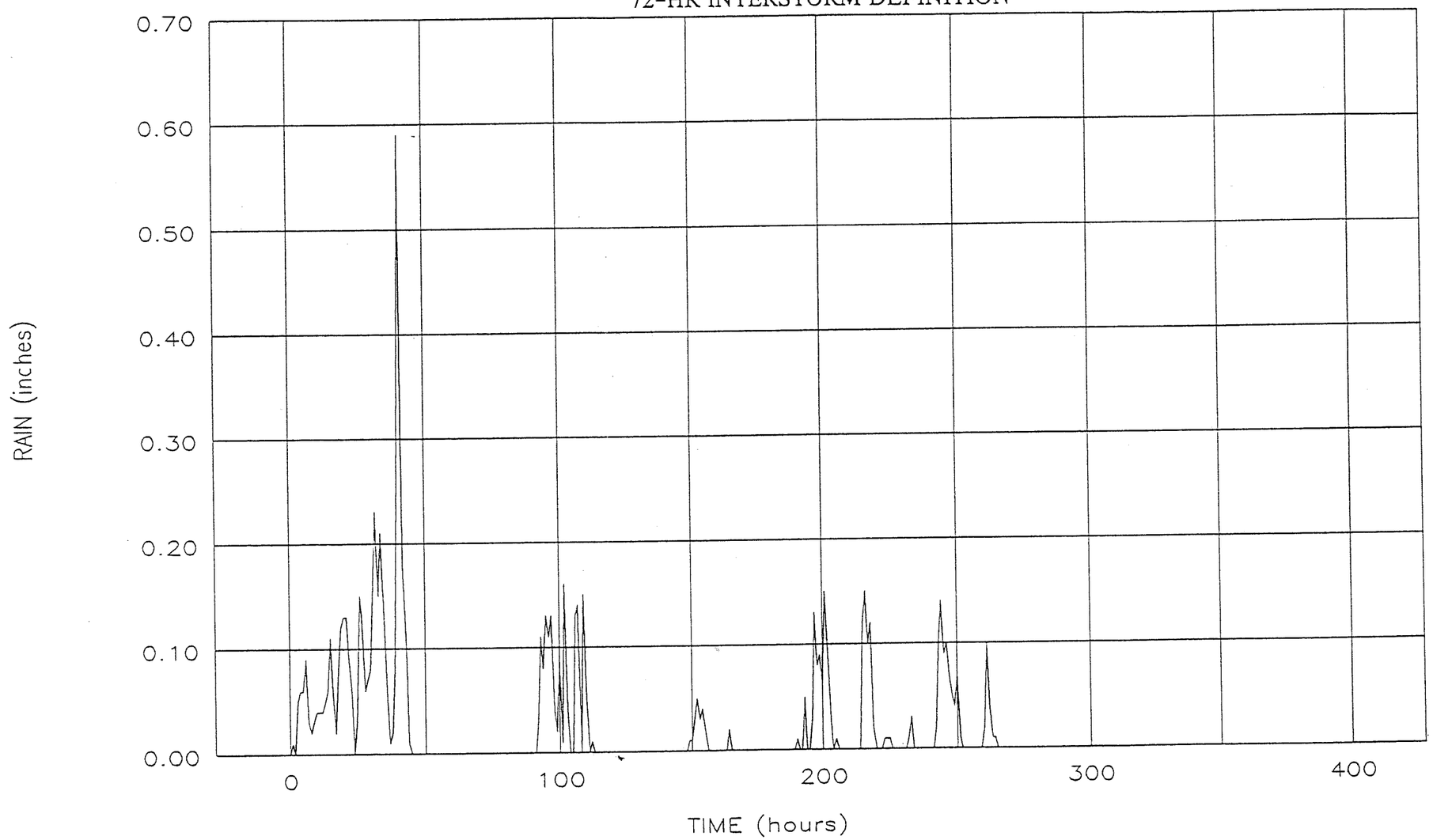
HISTORICAL EVENT DECEMBER 16, 1940
335-HR DURATION, 9.01 INCHES TOTAL DEPTH.
72-HR INTERSTORM DEFINITION



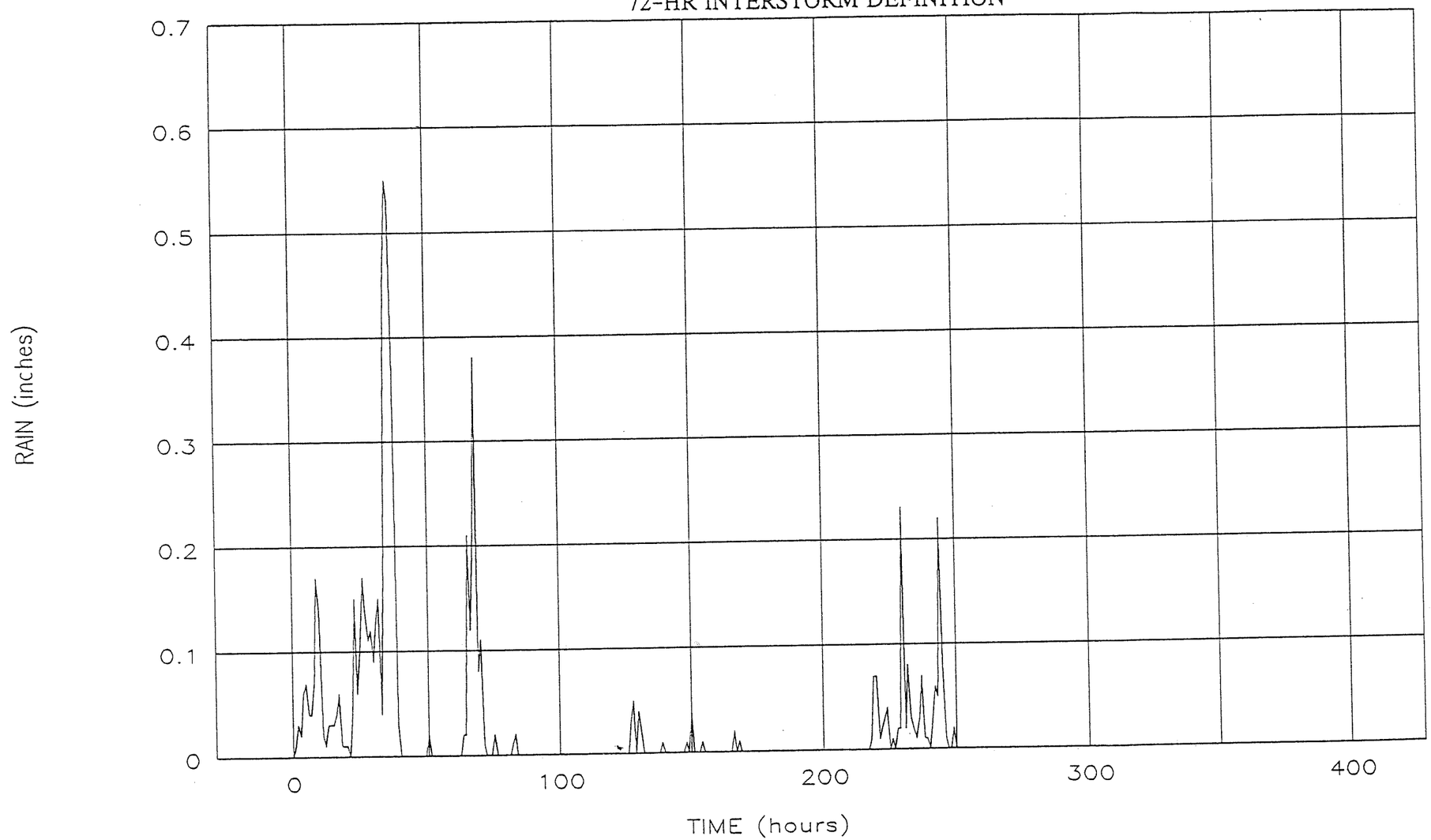
HISTORICAL EVENT FEBUARY 6, 1962
313-HR DURATION, 7.29 INCHES TOTAL DEPTH
72-HR INTERSTORM DEFINITION



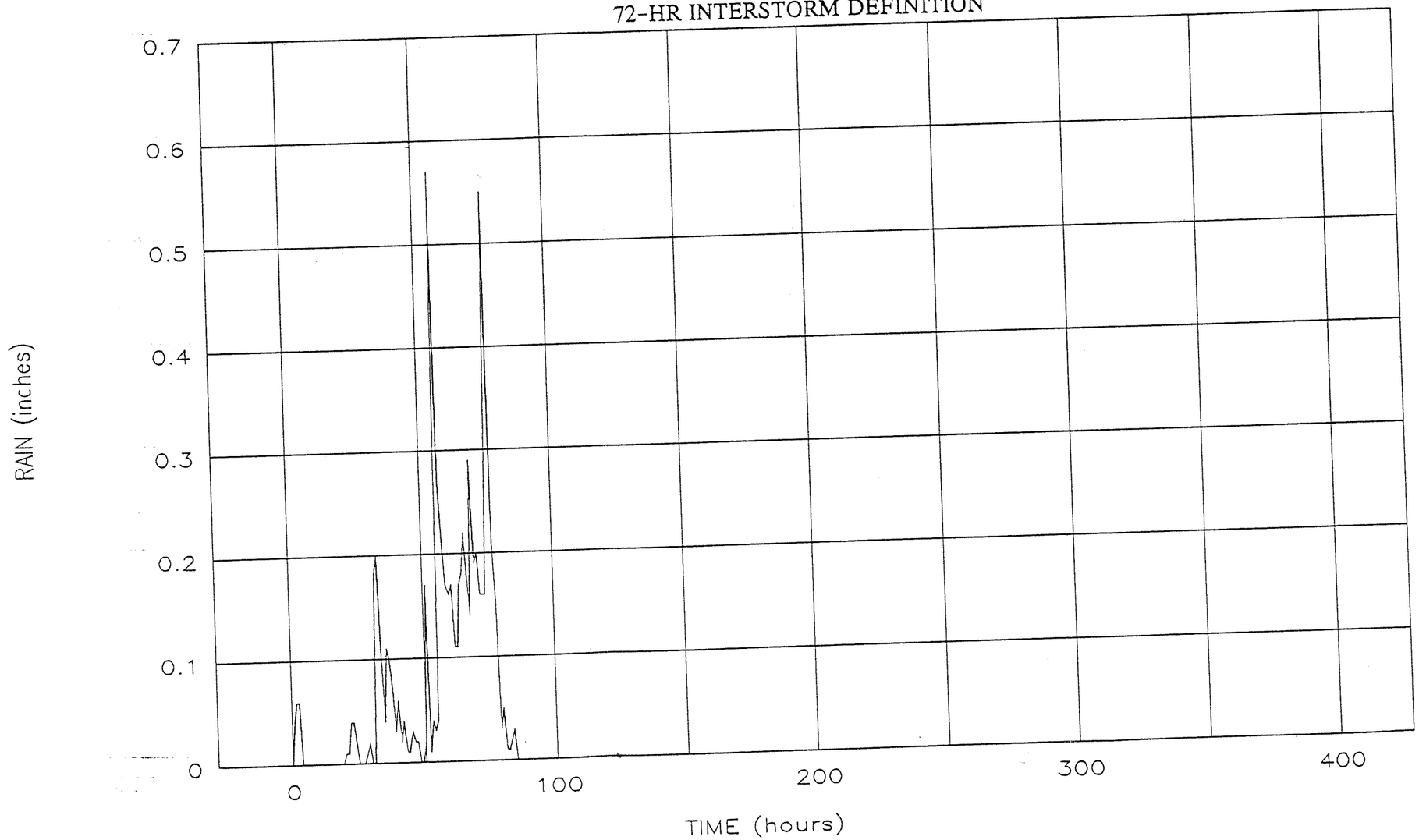
HISTORCIAL EVENT JANUARY 20, 1967
260-HR DURATION, 7.76 INCHES TOTAL DEPTH.
72-HR INTERSTORM DEFINITION



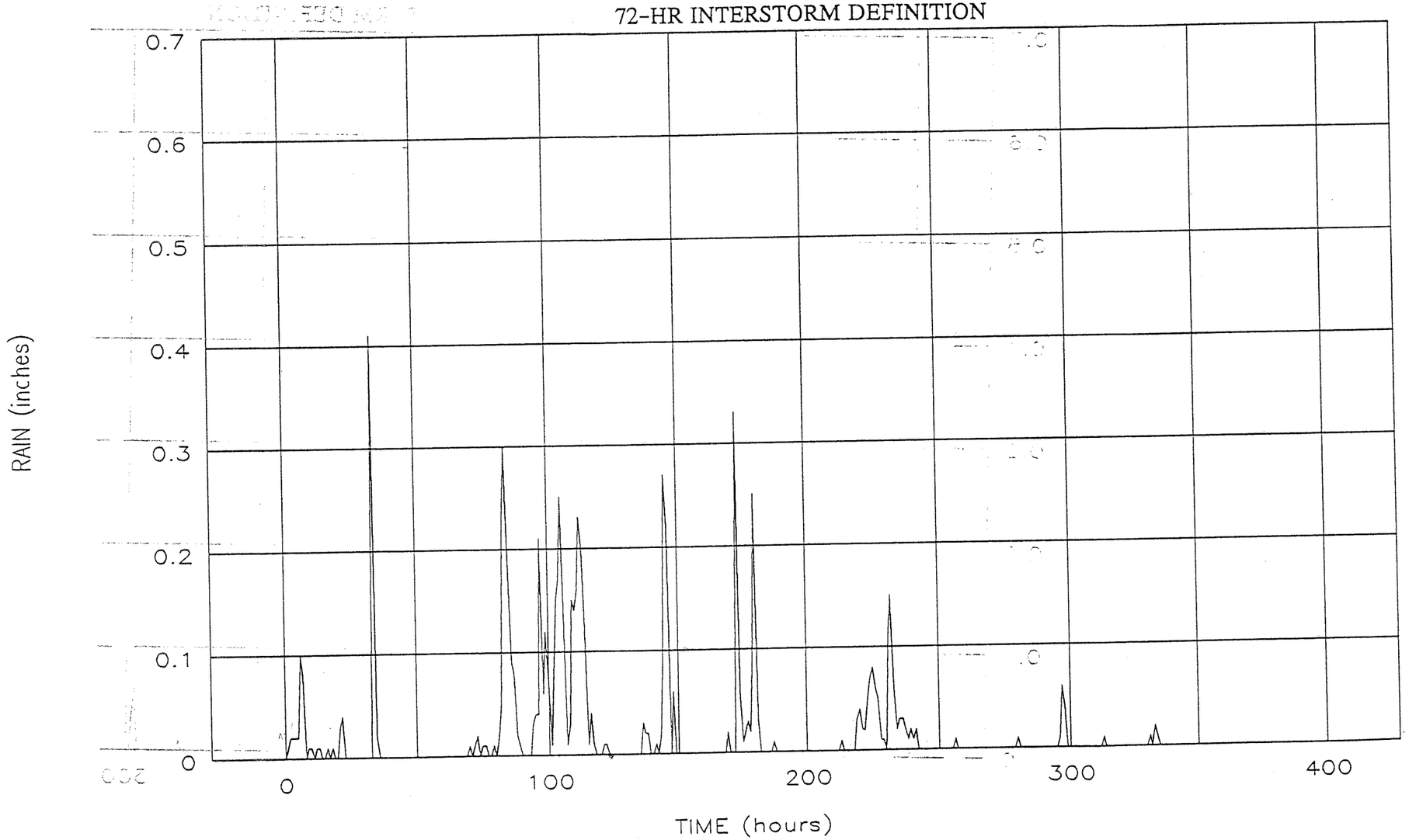
HISTORICAL EVENT JANUARY 20, 1943
249-HR DURATION, 6.84 INCHES TOTAL DEPTH
72-HR INTERSTORM DEFINITION



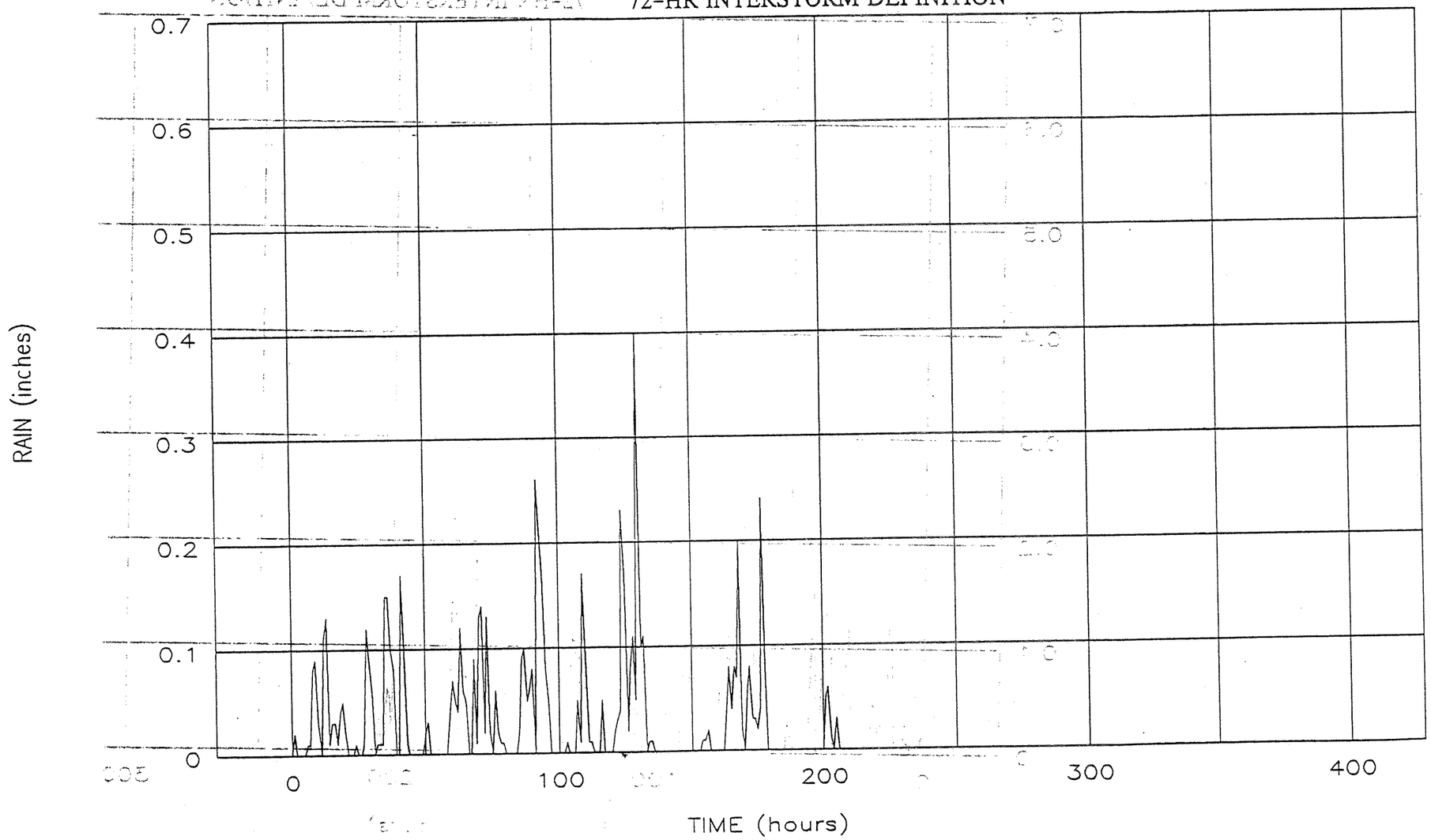
HISTORICAL EVENT OCTOBER 10, 1962
84-HR DURATION, 6.77 INCHES TOTAL DEPTH
72-HR INTERSTORM DEFINITION



HISTORICAL EVENT NOVEMBER 24, 1970
332-HR DURATION, 6.75 INCHES TOTAL DEPTH
72-HR INTERSTORM DEFINITION



HISTORICAL EVENT FEBUARY 13, 1980
205-HR DURATION, 6.90 INCHES TOTAL DEPTH
72-HR INTERSTORM DEFINITION



APPENDIX J

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM
IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 5

COMBINED SEWER COLLECTION SYSTEM HYDRAULIC EVALUATION, IMPROVEMENTS AND COSTS FOR THE 2-, 5-, 10-, AND 25-YEAR STORMS

Laurie Carrette
Judy A. Zavadil
Greg Jones
Paula M. Arsenault
HDR Engineering, Inc.

August 13, 1992

TECHNICAL MEMORANDUM
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN
PHASE 3, TASK 5

PREPARED BY: Laurie Carrette, Judy Zavadil, Greg Jones and Paula Arsenault, HDR Engineering, Inc.

DATE: July 1, 1992

SUBJECT: Combined Sewer Collection System Hydraulic Evaluation, Improvements and Costs for the 2-, 5-, 10- and 25-Year Storms

A. INTRODUCTION

Improvements to the combined wastewater collection system, were evaluated by two approaches based on pipe diameter. Improvements to the laterals, mains and interceptors, defined as pipes greater than 16 inches in diameter, were determined using the Sacramento Swmm model. It was not possible to evaluate small-diameter pipes, which represent a substantial portion of the overall drainage piping, using the Sacramento SWMM. Therefore, improvements to the "local" collection system, pipes less than 16 inches in diameter were analyzed separately according to the methodology presented in this memorandum.

A hydraulic and cost estimating analysis was completed for three representative collection system basins during Phase 2. The basins were selected based on their representation of hydraulic capacity, pipe layout, type and age of pipes, and physical condition within the combined sewer system. The Rational Method was used to estimate conveyance needs and pipe system improvements for a design storm event. Cost estimates were prepared for modifications such as drainage inlets, service lateral connections and pipe replacements in each representative basin. The results were then converted to per-acre cost estimates which were extrapolated to the entire combined sewer collection system.

Using the above approach, a system-wide collection system cost was generated as part of the alternative improvement plan cost estimates, as presented in the *Phase 2 Detailed Technical Report*. Appendix F to the report, a technical

memorandum entitled *Hydraulic Evaluation and 10-Year Storm Improvements for the Combined Sewer Collection System* (July 1, 1991), provides a detailed description of the analysis.

"Local" collection system costs are a significant percentage of the total improvement plan costs. In order to refine and verify the system-wide collection system improvement cost estimates prepared during the Phase 2 study, the Phase 3 Scope of Work included selecting and analyzing three additional representative collection system basins and completing a re-evaluation of the original three Phase 2 basins. The addition of three basins was intended to enhance the accuracy of cost estimates for collection system improvements.

The approach used in Phase 3 for analyzing the collection piping hydraulics is identical to that used during Phase 2 study. Differences from the Phase 2 study include: 1) calculation of service lateral connection costs based on a City estimate of the billing accounts in the service area; 2) verification of the pipe replacement costs using an additional method based on total lineal feet of pipe in the combined sewer area; 3) modification of the allocation of representative basin results to the entire collection system.

This technical memorandum describes the hydraulic evaluation of the collection system piping in the six representative basins, and the application of the results to the entire combined sewer collection system. An assessment of drainage inlet and service lateral replacement needs is also explained. Cost estimates for improvements are presented. Unit costs for pipes, drainage inlets and service lateral replacements are presented in a separate Phase 3 technical memorandum entitled *Unit Cost Refinements*.

The results of this analysis are included in cost estimates for alternative improvement plans wherein the sewers remain combined (e.g. tunnel, treatment, and storage system alternatives), and for the sewer separation alternative involving the construction of a new storm drain system.

B. REPRESENTATIVE BASINS

A basin number identifier has been assigned to each basin for reference purposes. These identifiers are specific to the collection system evaluation work, and are not related to the SWMM basin numbering system. The three representative combined sewer basins analyzed during Phase 2 are identified in Table 1 as Basin Nos. 1, 2, and 3. The three "additional" Phase 3 basins are Nos. 4 through 6.

TABLE 1. REPRESENTATIVE BASINS USED IN PHASE 2 AND 3 ANALYSES

Representative Basin ID No.[1]	Location Description	SWMM Basin Nos.	Sewer Book Pages
1	E and 25th Sts.	5681	9a, 9b, 9c, 9d
2	P and 38th Sts.	9411	15c, 21b
3	14th St. and 2nd Ave.	1661, 5101	26a, 26d
4	25th St. and 4th Ave.	1561	27a, 27b, 27f
5	Harkness Ave. and Larkin Way	1651	26b,23c, 22d
6	56th St. and Broadway	7951	30a, 30f

1 Not intended to be basin "type", as used in Phase 2 study.

As previously mentioned, the six selected basins are intended to represent the collection system as a whole, since the combined sewer system contains far too many pipes to reasonably include in the SWMM model. In addition to reviewing the representative basins with regard to pipe configuration, age, and condition, the amount or length of sewers receiving this detailed analysis can be compared to the total length of collection system piping.

The length of collection system piping analyzed in the six representative basins represents approximately 2 to 4 percent of the collection system piping in the City's combined sewer system, in the 8- to approximately 15-inch diameter range

(including non-standard 16-inch pipes). This estimate is based on the City's *Report on Pipe Diameter and Length - Sewer and Drainage Collections Systems* (dated March 1977). This report is the most up-to-date inventory of the total length of pipeline within in the City's sanitary, storm and combined sewer systems, according to diameter. Table 2 summarizes length comparison for the Phase 3 analysis.

TABLE 2. SYSTEM-WIDE VERSUS REPRESENTATIVE BASIN PIPE LENGTHS

Pipe Diameter (inches)	Total Combined System Lengths (feet)	Total Combined System Lengths w/o Back Yard Easements (feet)	Representative Basin Lengths (feet)	Percent Represented by Basins (%)
8	762,100	666,075	20,350	3.1
10	120,630	118,030	4,480	3.8
12	89,190	89,190	3,170	3.6
14 + 15	50,475	50,475	1,110	2.2
16	26,770	26,770	580	2.2

There are numerous back yard easement sewers that convey <u>only</u> sanitary flow. Because these pipes carry only sanitary flow prior to connection to the combined system, no capacity improvements for stormwater conveyance would be necessary. Including this feature of the City's collection system layout in the representative basins was an important consideration in selecting representative basins for both the Phase 2 and Phase 3 studies.

Three additional basins, Basin Nos. 4, 5 and 6, primarily residential in land use, were selected for improving the accuracy of the overall collection system hydraulic and cost analyses, and are identified in Table 1. Based on a review of the 1"= 500' SWMM basin maps, it was determined that the three additional areas should be selected to better represent the wide variety of basins in the southern half of the combined sewer service area. Basins in this area vary widely in terms

of collection system layout, and are less consistent than basins in the downtown portion of the combined system. The downtown area, which is relatively consistent in terms of pipe layout, is well represented by Basin 1. Therefore, the additional basins were selected to enhance the hydraulic and cost estimates for the variety of collection system layouts in other areas, such as those with numerous backyard easements.

Figures 2 through 13, located at the end of this memorandum, illustrate the basin boundaries on the City *Sewer Book* maps (1"=200′), and illustrate the system schematics (pipes, drainage inlets, dimensions, etc.) used as a basis for the analysis.

C. ASSUMPTIONS AND COMPUTATION METHOD FOR HYDRAULIC ANALYSIS

C.1. Assumptions

The following is a list of assumptions along with the computational process used in the sewer hydraulic analysis.

1. The City's 1"=200′ maps were used to develop drainage boundaries for points of interest in the system. Drainage divides were drawn to reflect lot drainage to the street from back yards. Field reconnaissance and survey spot checks were made to verify basin boundaries.

2. Sanitary flows were neglected in the analysis.

3. All areas were assumed to have approximately the same amount of imperviousness (which is represented in the C coefficient). This is a valid assumption, because the results from the six representative basins are being applied to the combined sewer collection system on a system-wide basis. The overall imperviousness of the combined sewer system is approximately 50 percent.

4. For the purpose of analyzing piping improvements, drainage inlets were assumed to provide adequate capacity for peak runoff to enter the system.

The need for drainage inlet improvements was then analyzed separately. Carryover discharges (i.e. routing of gutter flow from one inlet to the next one downstream) were not calculated because detailed topography maps were not available. In addition, it was assumed that for the 2-, 5- and 10-year storms, the depth of gutter flow will generally not exceed the height of the curbs.

5. Overland slope was taken to be uniformly equal to 1.5 percent. Overland flow is defined as flow over "short grass", directly into a storm sewer inlet or paved street. These assumptions and the nomograph in Appendix C were used to estimate overland flow velocity and corresponding travel times.

6. Manhole lid elevations were used to compute the slope in gutters when surveyed gutter flow lines were not available. The slope of the gutter and the nomograph for paved overland flow were used to determine the flow velocity and therefore travel time for the surface conveyance system. Where survey data were not available, assumptions for manhole rim invert elevations were estimated.

7. Pipe slopes were calculated from invert elevations and the 1"=200′ sewer maps, from which pipe lengths were determined. Velocities were computed using Manning's equation for a circular pipe with an assumed n-value of 0.013. From this, time of travel within a given pipe segment was estimated.

8. Full-flow pipe conditions were used when calculating the flow velocity for time of concentration. Concrete pipe was used for all calculations.

9. For a given analysis point, the total time of travel (overland plus gutter plus pipe routing) allowed for the selection of the appropriate rainfall intensity. The rainfall intensity was then used in the Rational Formula (along with a C-value and the drainage area) to determine a flow rate for a given point in the system.

10. Using the calculated flow and pipe slope, a pipe diameter was selected.

11. Required pipe diameters resulting from the hydraulic analysis sizes were rounded to the next largest standard manufactured size when the calculations resulted in a non-standard diameter. For example, a calculated requirement for a 14-inch pipe was rounded to a standard 15-inch pipe diameter.

C.2. Methodology

The Rational Method for analyzing peak flow is widely used by design engineers. It is an empirical, simplistic equation, usually used in the design of minor storm sewer systems draining less than 200 acres. It is based on an intensity-runoff relationship as follows:

$$Q = CiA$$

Where:
- Q = runoff, in cfs
- C = runoff coefficient
- i = rainfall intensity, in inches/hour
- A = drainage area, in acres

The Rational Method assumes: 1) a uniform rainfall intensity over the entire watershed during the entire storm duration; 2) the maximum runoff rate occurs when the rainfall lasts as long or longer than the time of concentration; and 3) the time of concentration is the time required for the runoff from the most hydraulically remote part of the watershed to reach the design point or point of analysis.

Although the equation has only three variables, the determination of these variables is based on judgment and experience. The three variables are defined in the following sections.

C.2.a. Runoff Coefficient--

The runoff coefficient, C, incorporates hydrological characteristics, soil types,

antecedent conditions, etc. Judgment and recommended values are used to determine a representative, composite runoff coefficient for a drainage basin. Average runoff coefficients are provided in ASCE Manual No. 37.

C.2.b. Rainfall Intensity--

The rainfall intensity, i, is obtained from intensity-duration-frequency (I-D-F) curves based on statistical analysis of rainfall data for various frequency storm events. Figure 1 presents the I-D-F curve used in this analysis, and was derived from the depth-duration-frequency data developed in a previously completed study for the City and County of Sacramento.

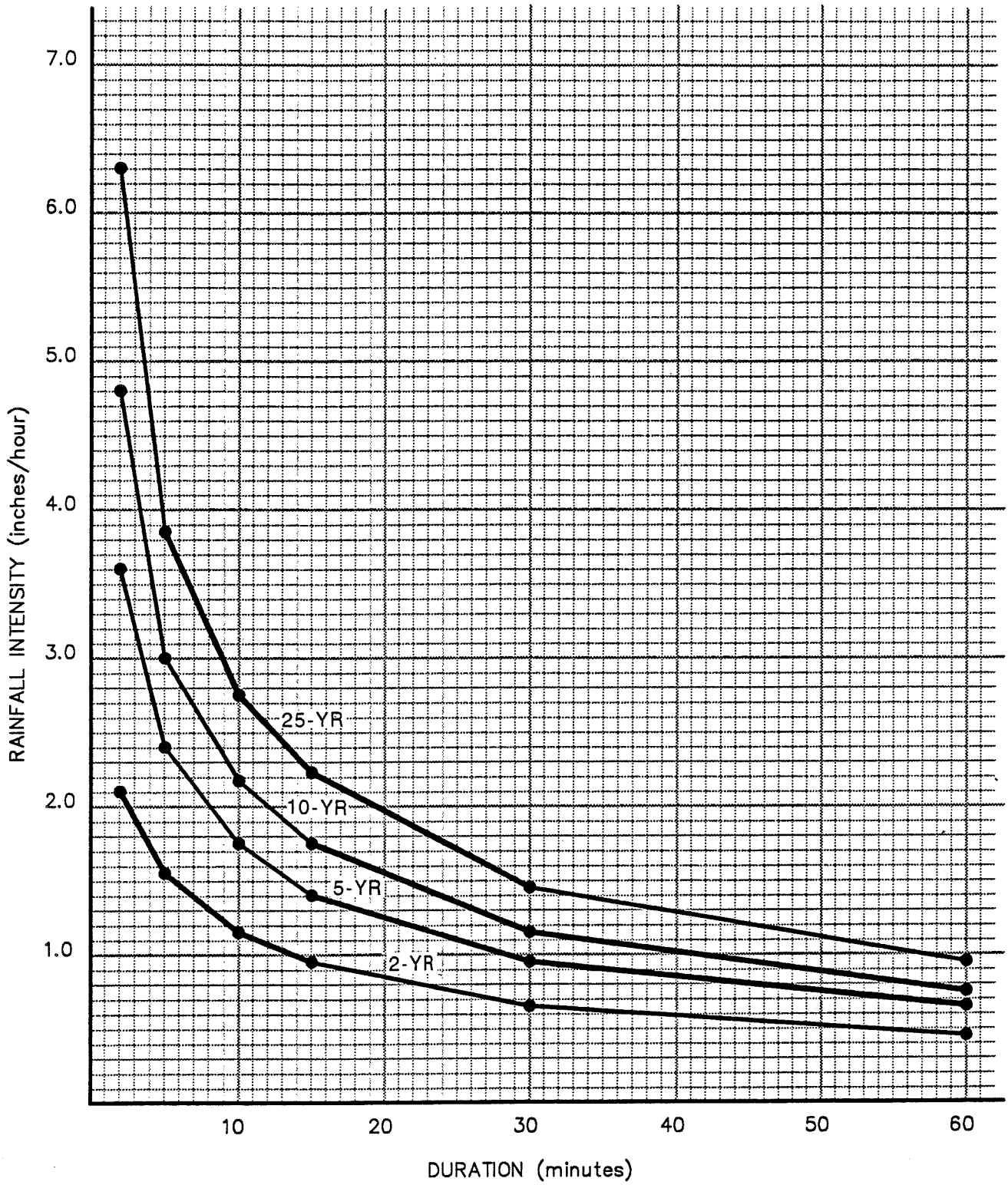
The intensity is determined for a storm duration equal to the time of concentration. The time of concentration is the sum of inlet time and channel flow time and is equal to the length of reach divided by average channel velocity. A nomograph for the SCS Upland Method (see Appendix C) provides average velocities for overland flow and considers the basin landscaping, grassed waterways, paved areas and small upland gullies.

C.2.c. Drainage Area--

The drainage area is the contributing area that flows to a design point which produces the maximum peak flow. Manholes were assigned a letter code for identification purposes, so each segment analyzed could be identified according to the manhole codes.

C.3. Calculations

The Rational Method was used to determine the peak runoff at each design point for the 2-, 5-, 10- and 25-year events. A runoff coefficient, C, for a single family residential area was assumed. The rainfall intensity, i, was abstracted from Figure 1. Design points were chosen at manholes where pipe sizes changed, or pipe systems intersected. The time of concentration was calculated based on a combination of overland, street and pipe flow. The largest time of concentration governed when sizing a pipe system with multiple subbasins. Manning's equation was then used to select the proper pipe size based on the resulting flow calculations.



INTENSITY–DURATION–FREQUENCY CURVES
NWS SACRAMENTO, CA

(source: Draft Sacramento City/County Drainage Manual, July 1991)

HDR

Figure 1

D. PIPE IMPROVEMENTS AND COST ESTIMATES

This section describes the application of representative basin analysis results to the combined sewer system as a whole in order to estimate pipe replacement costs. The pipe replacement costs in Phase 3 were generated using two methods. In the first approach, a per-acre cost was developed and applied to the combined sewer system area. In the second approach, the hydraulic analysis of the six representative basins was instead used to determine upsizing requirements. The upsizing requirements of the representative basins were then applied to the total pipe length in the collection system. For example, the percentage of 8-inch pipe that must be upsized to 12-inch in the representative basins was applied to the total length of 8-inch pipes in the entire collection system.

Tables 3, 4, 5 and 6 summarize the hydraulic analysis results for the 2-, 5-, 10- and 25-year design storm events. On these tables, existing 14-inch and 15-inch pipes were grouped together. Although 16-inch pipes are not a current, standard diameter, these were included in the analysis.

D.1. Analysis of Pipe Replacement Based on Pipe Lengths

An evaluation of pipe improvements for each design storm and representative basin was prepared. The results for the six representative basins were used to draw conclusions about the amount, in linear feet and percent, of existing collection system piping that would need to be upsized to larger diameters. Tables 3 through 6 summarize the analysis, with detailed calculations presented in Appendix C.

For example, 26.8 percent (relative to length) of the existing 8-inch pipe in the six representative basins (excluding back yard easement piping) needed to be upsized to 12 inches to convey the 2-year design storm event. It was therefore assumed that 26.8 percent of the 8-inch pipes system-wide would require upsizing to 12 inches, in order to convey the 2-year storm. Percentages derived in this manner were then applied to the entire combined sewer collection system pipe lengths given in Table 1.

TABLE 3. SUMMARY OF PIPE REPLA(ENTS AND COSTS FOR THE 2-YEAR STORM

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASINS 1 TO 6
2-YEAR RATIONAL METHOD ANALYSIS

	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
	8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING 8	3,090	5,970	5,455	4,400	1,435	0	0	0	0	0	0	0	0	20,350
EXISTING 10	0	0	925	1,250	1,985	110	210	0	0	0	0	0	0	4,480
EXISTING 12	0	0	0	130	1,045	825	830	340	0	0	0	0	0	3,170
EXISTING 14	0	0	0	0	45	0	465	410	190	0	0	0	0	1,110
EXISTING 16	0	0	0	0	0	0	110	0	0	470	0	0	0	580
TOTALS (LF)	3,090	5,970	6,380	5,780	4,510	935	1,615	750	190	470	0	0	0	29,690

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASINS 1 TO 6
2-YEAR RATIONAL METHOD ANALYSIS

	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL
	8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING 8	15.2%	29.3%	26.8%	21.6%	7.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 10	0.0%	0.0%	20.6%	27.9%	44.3%	2.5%	4.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 12	0.0%	0.0%	0.0%	4.1%	33.0%	26.0%	26.2%	10.7%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 14	0.0%	0.0%	0.0%	0.0%	4.1%	0.0%	41.9%	36.9%	17.1%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	19.0%	0.0%	0.0%	81.0%	0.0%	0.0%	0.0%	100.0%
TOTALS (%)	10.4%	20.1%	21.5%	19.5%	15.2%	3.1%	5.4%	2.5%	0.6%	1.6%	0.0%	0.0%	0.0%	100.0%

CITY-WIDE COLLECTION SYSTEM PIPE REPLACEMENT COSTS
2-YEAR RATIONAL METHOD ANALYSIS

	TOTAL LENGTH (LF)	PIPE REPLACEMENT DIAMETERS (INCHES)												
		8	10	12	15	18	21	24	27	30	33	36	42	48
EXISTING 8	666,075	101,243	195,160	178,508	143,872	47,291	0	0	0	0	0	0	0	0
EXISTING 10	118,030	0	0	24,314	32,930	52,287	2,951	5,547	0	0	0	0	0	0
EXISTING 12	89,190	0	0	0	3,657	29,433	23,189	23,368	9,543	0	0	0	0	0
EXISTING 14	50,475	0	0	0	0	2,069	0	21,149	18,625	8,631	0	0	0	0
EXISTING 16	26,770	0	0	0	0	0	0	5,086	0	0	21,684	0	0	0
TOTALS (LF)	950,540	101,243	195,160	202,822	180,459	131,081	26,140	55,151	28,169	8,631	21,684	0	0	0
UNIT COST ($)		$0.00	$79.85	$94.77	$111.35	$129.35	$147.05	$209.41	$237.49	$269.03	$302.12	$341.67	$416.47	$500.82
TOTAL COST ($ * 10^6)		$0.00	$15.58	$19.22	$20.09	$16.96	$3.84	$11.55	$6.69	$2.32	$6.55	$0.00	$0.00	$0.00
										PIPE REPLACEMENT TOTAL COST ($ * 10^6)				$102.81

TABLE 4. SUMMARY OF PIPE REPL .MENTS AND COSTS FOR THE 5-YEAR STORM

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASINS 1 TO 6
5-YEAR RATIONAL METHOD ANALYSIS

	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
	8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING 8	260	4,130	5,465	5,465	3,715	1,315	0	0	0	0	0	0	0	20,350
EXISTING 10	0	0	550	695	1,305	1,610	110	210	0	0	0	0	0	4,480
EXISTING 12	0	0	0	0	130	1,045	825	215	840	115	0	0	0	3,170
EXISTING 14	0	0	0	0	0	45	0	465	410	0	190	0	0	1,110
EXISTING 16	0	0	0	0	0	0	0	110	0	0	0	470	0	580
TOTALS (LF)	260	4,130	6,015	6,160	5,150	4,015	935	1,000	1,250	115	190	470	0	29,690

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASINS 1 TO 6
5-YEAR RATIONAL METHOD ANALYSIS

	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL
	8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING 8	1.3%	20.3%	26.9%	26.9%	18.3%	6.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 10	0.0%	0.0%	12.3%	15.5%	29.1%	35.9%	2.5%	4.7%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 12	0.0%	0.0%	0.0%	0.0%	4.1%	33.0%	26.0%	6.8%	26.5%	3.6%	0.0%	0.0%	0.0%	100.0%
EXISTING 14	0.0%	0.0%	0.0%	0.0%	0.0%	4.1%	0.0%	41.9%	36.9%	0.0%	17.1%	0.0%	0.0%	100.0%
EXISTING 16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	19.0%	0.0%	0.0%	0.0%	81.0%	0.0%	100.0%
TOTALS (%)	0.9%	13.9%	20.3%	20.7%	17.3%	13.5%	3.1%	3.4%	4.2%	0.4%	0.6%	1.6%	0.0%	100.0%

CITY-WIDE COLLECTION SYSTEM PIPE REPLACEMENT COSTS
5-YEAR RATIONAL METHOD ANALYSIS

	TOTAL LENGTH (LF)	PIPE REPLACEMENT DIAMETERS (INCHES)												
		8	10	12	15	18	21	24	27	30	33	36	42	48
EXISTING 8	666,075	8,659	135,213	179,174	179,174	121,892	43,295	0	0	0	0	0	0	0
EXISTING 10	118,030	0	0	14,518	18,295	34,347	42,373	2,951	5,547	0	0	0	0	0
EXISTING 12	89,190	0	0	0	0	3,657	29,433	23,189	6,065	23,635	3,211	0	0	0
EXISTING 14	50,475	0	0	0	0	0	2,069	0	21,149	18,625	0	8,631	0	0
EXISTING 16	26,770	0	0	0	0	0	0	0	5,086	0	0	0	21,684	0
TOTALS (LF)	950,540	8,659	135,213	193,692	197,469	159,895	117,170	26,140	37,848	42,261	3,211	8,631	21,684	0
UNIT COST ($)		$0.00	$79.85	$94.77	$111.35	$129.35	$147.05	$209.41	$237.49	$269.03	$302.12	$341.67	$416.47	$500.82
TOTAL COST ($ * 10^6)		$0.00	$10.80	$18.36	$21.99	$20.68	$17.23	$5.47	$8.99	$11.37	$0.97	$2.95	$9.03	$0.00
										PIPE REPLACEMENT TOTAL COST ($ * 10^6)				$127.83

7140001.169

TABLE 5. SUMMARY OF PIPE REPI EMENTS AND COSTS FOR THE 10-YEAR STORM

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASINS 1 TO 6
10-YEAR RATIONAL METHOD ANALYSIS

	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
	8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING 8	260	2,510	4,160	7,495	3,195	1,415	1,315	0	0	0	0	0	0	20,350
EXISTING 10	0	0	0	815	615	1,935	795	110	210	0	0	0	0	4,480
EXISTING 12	0	0	0	0	0	795	675	530	215	615	340	0	0	3,170
EXISTING 14	0	0	0	0	0	0	45	0	465	410	0	190	0	1,110
EXISTING 16	0	0	0	0	0	0	0	0	110	0	0	470	0	580
TOTALS (LF)	260	2,510	4,160	8,310	3,810	4,145	2,830	640	1,000	1,025	340	660	0	29,690

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASINS 1 TO 6
10-YEAR RATIONAL METHOD ANALYSIS

	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL
	8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING 8	1.3%	12.3%	20.4%	36.8%	15.7%	7.0%	6.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 10	0.0%	0.0%	0.0%	18.2%	13.7%	43.2%	17.7%	2.5%	4.7%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 12	0.0%	0.0%	0.0%	0.0%	0.0%	25.1%	21.3%	16.7%	6.8%	19.4%	10.7%	0.0%	0.0%	100.0%
EXISTING 14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	4.1%	0.0%	41.9%	36.9%	0.0%	17.1%	0.0%	100.0%
EXISTING 16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	19.0%	0.0%	0.0%	81.0%	0.0%	100.0%
TOTALS (%)	0.9%	8.5%	14.0%	28.0%	12.8%	14.0%	9.5%	2.2%	3.4%	3.5%	1.1%	2.2%	0.0%	100.0%

CITY-WIDE COLLECTION SYSTEM PIPE REPLACEMENT COSTS
10-YEAR RATIONAL METHOD ANALYSIS

	TOTAL LENGTH (LF)	PIPE REPLACEMENT DIAMETERS (INCHES)												
		8	10	12	15	18	21	24	27	30	33	36	42	48
EXISTING 8	666,075	8,659	81,927	135,879	245,116	104,574	46,625	43,295	0	0	0	0	0	0
EXISTING 10	118,030	0	0	0	21,481	16,170	50,989	20,891	2,951	5,547	0	0	0	0
EXISTING 12	89,190	0	0	0	0	0	22,387	18,997	14,895	6,065	17,303	9,543	0	0
EXISTING 14	50,475	0	0	0	0	0	0	2,069	0	21,149	18,625	0	8,631	0
EXISTING 16	26,770	0	0	0	0	0	0	0	0	5,086	0	0	21,684	0
TOTALS (LF)	950,540	8,659	81,927	135,879	266,597	120,744	120,001	85,253	17,845	37,848	35,928	9,543	30,315	0
UNIT COST ($)		$0.00	$79.85	$94.77	$111.35	$129.35	$147.05	$209.41	$237.49	$269.03	$302.12	$341.67	$416.47	$500.82
TOTAL COST ($ * 10^6)		$0.00	$6.54	$12.88	$29.69	$15.62	$17.65	$17.85	$4.24	$10.18	$10.85	$3.26	$12.63	$0.00
										PIPE REPLACEMENT TOTAL COST ($ * 10^6)				$141.38

TABLE 6. SUMMARY OF PIPE REPL .MENTS AND COSTS FOR THE 25-YEAR STORM

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASINS 1 TO 6
25-YEAR RATIONAL METHOD ANALYSIS

	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
	8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING 8	85	175	4,090	6,795	4,295	3,475	235	1,200	0	0	0	0	0	20,350
EXISTING 10	0	0	0	550	505	1,120	1,365	620	110	210	0	0	0	4,480
EXISTING 12	0	0	0	0	0	130	785	555	530	215	840	115	0	3,170
EXISTING 14	0	0	0	0	0	0	0	45	155	310	410	190	0	1,110
EXISTING 16	0	0	0	0	0	0	0	0	110	0	0	0	470	580
TOTALS (LF)	85	175	4,090	7,345	4,800	4,725	2,385	2,420	905	735	1,250	305	470	29,690

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASINS 1 TO 6
25-YEAR RATIONAL METHOD ANALYSIS

	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL
	8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING 8	0.4%	0.9%	20.1%	33.4%	21.1%	17.1%	1.2%	5.9%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING 10	0.0%	0.0%	0.0%	12.3%	11.3%	25.0%	30.5%	13.8%	2.5%	4.7%	0.0%	0.0%	0.0%	100.0%
EXISTING 12	0.0%	0.0%	0.0%	0.0%	0.0%	4.1%	24.8%	17.5%	16.7%	6.8%	26.5%	3.6%	0.0%	100.0%
EXISTING 14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	4.1%	14.0%	27.9%	36.9%	17.1%	0.0%	100.0%
EXISTING 16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	19.0%	0.0%	0.0%	0.0%	81.0%	100.0%
TOTALS (%)	0.3%	0.6%	13.8%	24.7%	16.2%	15.9%	8.0%	8.2%	3.0%	2.5%	4.2%	1.0%	1.6%	100.0%

CITY-WIDE COLLECTION SYSTEM PIPE REPLACEMENT COSTS
25-YEAR RATIONAL METHOD ANALYSIS

	TOTAL LENGTH (LF)	PIPE REPLACEMENT DIAMETERS (INCHES)												
		8	10	12	15	18	21	24	27	30	33	36	42	48
EXISTING 8	666,075	2,664	5,995	133,881	222,469	140,542	113,899	7,993	39,298	0	0	0	0	0
EXISTING 10	118,030	0	0	0	14,518	13,337	29,508	35,999	16,288	2,951	5,547	0	0	0
EXISTING 12	89,190	0	0	0	0	0	3,657	22,119	15,608	14,895	6,065	23,635	3,211	0
EXISTING 14	50,475	0	0	0	0	0	0	0	2,069	7,067	14,083	18,625	8,631	0
EXISTING 16	26,770	0	0	0	0	0	0	0	0	5,086	0	0	0	21,684
TOTALS (LF)	950,540	2,664	5,995	133,881	236,987	153,879	147,063	66,111	73,264	29,998	25,695	42,261	11,842	21,684
UNIT COST ($)		$0.00	$79.85	$94.77	$111.35	$129.35	$147.05	$209.41	$237.49	$269.03	$302.12	$341.67	$416.47	$500.82
TOTAL COST ($ * 10^6)		$0.00	$0.48	$12.69	$26.39	$19.90	$21.63	$13.84	$17.40	$8.07	$7.76	$14.44	$4.93	$10.86
										PIPE REPLACEMENT TOTAL COST ($ * 10^6)				$158.39

The length of sanitary-only pipes which are routed through back yard easements, were not included as part of the total lengths. Most of the back yard easement pipes convey only sanitary flow, and are therefore not subject to the evaluation of flood control improvements. HDR completed an inventory of these sewers using the 1"=500′ sewer system maps; the results are included in Appendix C.

All of the pipes within the representative basins will not require replacement. Where a length is shown in Tables 3 through 6 as being the same diameter for "existing" and "replacement", neither upsizing nor replacement is required. For instance, as shown on Table 3, 101,243 feet of existing 8-inch pipe is given a replacement diameter of 8 inches. This means that these pipes need not be replaced to convey the 2-year design storm, and will therefore do contribute to the collection system improvement cost.

D.2. Cost Estimates Based on Pipe Lengths

Using the replacement percentages derived from the representative basins and the total system inventory, computations for system-wide replacements were prepared. For instance, using the previous examples for replacement pipe length computations, the 26.8 percent value for upsizing from 8 inches to 12 inches for a 2-year storm was applied against a total existing system 8-inch pipe length of 666,075 feet, to estimate a need for 178,508 feet of 12-inch combined sewer pipe replacement. This length was added to the length of existing collection system pipe that would need to be upsized from 10 inches to 12 inches (for a 2-year storm), to derive a total of 202,822 feet of new 12-inch sewer. Multiplying the footage by a unit cost of $94.77 per foot, a cost estimate was computed for this component of the 2-year storm improvements. A summary of total pipe replacement cost estimates is provided in Table 7. Unit replacement costs used for this analysis were determined in the Phase 3 technical memorandum *Unit Cost Refinements*.

TABLE 7. SUMMARY OF PIPE IMPROVEMENT COST ESTIMATES BASED ON PIPE LENGTH	
Design Storm	Cost Estimate ($ millions)
2-Year	$ 102.81
5-Year	$ 127.83
10-Year	$ 141.38
25-Year	$ 158.39

D.3. Analysis of Pipe Replacement Based on Area

The details of this approach were developed and explained during Phase 2 of this project. A summary of the basic approach includes the following steps: 1) costs for all pipe replacements are calculated for each basin; 2) the individual basin costs are divided by the area of each representative basin; 3) an average per-acre cost for all six representative basins was developed and applied to the total area of the combined sewer system.

During Phase 2, a per-acre unit cost for pipe improvements was developed for each of the three representative basins. Following the calculation of individual per-acre costs, the remaining basins in the combined sewer system were reviewed and classified to determine which of the three representative basins would most closely represent the improvement needs of each basin. This was accomplished using the City's 1"=500' sewer maps. All of the basins were assigned a basin type. The total acreage of each basin type was then totalled and the corresponding per-acre costs were applied to develop an overall system cost. In this Phase 3 analysis however, allocation of costs based on basin type was not performed by HDR. Instead, an average per-acre cost for all six representative basins was developed and applied to the total area of the combined sewer system.

In addition however, the basin "type" methodology of the Phase 2 study was applied to the six selected basins of the Phase 3 study in an analysis by John Tomko, Consulting Engr., for the City. This analysis is attached as Appendix G.

Areas excluded from the analysis included separated areas, cemeteries, railroad yards, parks and other open areas. Appendix F presents the analysis to determine the sewered acreage. The acreage used in cost estimates for the total collection system improvement is 5835 acres.

D.4. Cost Estimates Based on Area

Pipe improvement costs developed using the area method for the six representative basins are calculated in detail in Appendix D. These costs are summarized in Table 8, together with the average per-acre value for each design storm. Table 9 shows the pipe replacement improvement costs for the entire combined sewer service area, using the unit values derived in Table 8.

A comparison of the total costs for the two pipe replacement methods shows that the area-method values are 12 to 18 percent lower than the pipe-length method values. The average of the total costs derived from the three methods will be used for the collection cost estimates of the Phase 3 improvement plans. Table 10 summarizes the calculated total costs for each method and the average value.

TABLE 8. REPRESENTATIVE BASIN PIPE IMPROVEMENT COSTS BASED ON AREA

Representative Basin	Drainage (acres)	2-Year Storm ($/acre)	5-Year Storm ($/acre)	10-Year Storm ($/acre)	25-Year Storm ($/acre)
1	36.7	$15,961	$18,661	$20,978	$23,077
2	34.1	$7,657	$8,724	$10,177	$10,878
3	43.7	$14,956	$18,371	$19,987	$23,036
4	33.7	$13,514	$15,447	$16,718	$18,901
5	44.0	$20,863	$24,414	$27,261	$30,062
6	28.5	$19,608	$22,605	$24,572	$28,223
Average		$15,427	$18,037	$19,949	$22,363

TABLE 9. SUMMARY OF PIPE IMPROVEMENT COST ESTIMATE BASED ON AREA	
Design Storm	Cost Estimate ($ Million)
2-Year	90.02
5-Year	105.24
10-Year	116.40
25-Year	130.49

TABLE 10. SUMMARY OF PIPE IMPROVEMENT COST ESTIMATES				
Method	2-Year	5-Year	10-Year	25-Year
Pipe Improvements Area Method (HDR)	90.02	105.24	116.40	130.49
Pipe Improvements Area Method (John Tomko)	105.00	122.25	137.29	151.55
Pipe Improvements Length Method	102.81	127.83	141.38	158.39
Average of Pipe Improvement Methods	99.28	118.44	131.69	146.81

E. DRAINAGE INLET ANALYSIS

Consistent with pipe upgrades, street drainage inlets will need to be improved in order to ensure that stormwater runoff enters the collection system for a given design storm. To estimate needed drainage inlet improvements and to prepare a cost estimate for this element of the overall collection system, the capacity of the existing drainage inlets was analyzed. The number of drainage inlets that would need to be replaced and added for a corresponding design storm within the representative basins was then determined. Improvement needs were then converted to a per-acre basis for each design storm, and applied against the total number of combined sewer service area acres involved in the improvement plan. Detailed analysis computations are presented in Appendices A and B.

E.1. Assumptions for Drainage Inlet Analysis

Due to the complexity of analyzing drainage inlet capacity and the lack of detailed topographic information, several simplifying assumptions were necessary to complete the analysis. The following assumptions were used in the analysis.

1. All drainage inlets were assumed to be located in a sump area.

2. All existing inlets are cast iron, with dimensions of 14 inches by 14 inches, having 15 openings with dimensions of 1-5/8 inches by 3 inches each.

3. The replacement inlet assumed is a City of Sacramento Type "B".

4. Stormwater runoff is distributed equally to each inlet.

5. Weir flow is computed according to the equation:

$$Q = 3.3Lh^{1.5}$$

where Q = flow, cfs
L = weir length, feet
h = height of water above weir, feet

The selected weir coefficient was based on the City's study on flooding in the 42nd Street area.

6. Orifice flow is computed according to the equation:

$$Q = 0.6A\sqrt{2gh}$$

where Q = flow, cfs
A = orifice area, square feet
h = height of water above orifice, feet
g = gravitational acceleration, 32.2 ft/sec

7. Weir length is the total edge length over which flow spills in entering the inlet, i.e. three sides of the inlet. The orifice area is the total open space (sum of the hole space in the grate) through which flow can enter the inlet.

For existing inlets: weir length = L = 3.5 feet
orifice area = A = 0.40 square feet
Assuming 50 percent clogging: L = 2.4 ft; A = 0.2 sq ft

For replacement inlets: weir length = 6.5 feet
orifice area = 3.6 square feet
Assuming 50 percent clogging: L = 5.0 ft; A = 1.8 sq ft

8. The maximum allowable water depth is 0.5 feet, which assumes 6-inch curbs.

9. Orifice flow is assumed for existing inlets; maximum flow is 0.7 cfs. Weir flow is assumed for replacement inlets; maximum flow is 5.8 cfs.

E.2. Computational Procedure

This subsection summarizes the computational procedures for the drainage inlet hydraulic analysis and improvement estimates.

It was assumed that each existing inlet collects a discrete amount of flow from a subcatchment within the representative basin, that cannot be routed to another inlet. If two inlets exist, then a minimum of two inlets will be required for replacement, if the existing inlets have inadequate capacity.

The Rational Method was used to calculate peak flow for a given pipe segment. Since the hydrologic analysis was for piping improvements, peak flow computations are cumulative from upstream to downstream, and do not represent the peak flow for each individual subcatchment. Individual subcatchment flows available from the pipe segment analysis were used in the analysis of drainage inlet capacity requirements.

For drainage inlets in the most upstream subcatchments of a basin, peak flow was available for the individual subcatchments in the basin. A flow rate per inlet could then be calculated. For drainage inlets in downstream subcatchments, only cumulative flow rates were available. Therefore, each inlet was assumed to collect an equal portion of the peak flow for the system, which allowed for the calculation of a flow rate per inlet for a given basin.

If the calculated flow rate per inlet was greater than 0.7 cfs (the maximum capacity for an existing drainage inlet), it was estimated that additional inlets will be needed. The minimum assumed improvement was the replacement of existing drainage inlets with new ones. If the required flow rate per inlet was calculated to be greater than 5.8 cfs (the maximum capacity of a new inlet), an appropriate number of additional inlets were identified to handle the excess flow.

The number of replacement and additional drainage inlets per acre was computed based on the results for the six representative basins. Each design storm was analyzed in a similar fashion.

E.3. Cost Estimates

Total costs for system-wide improvements to drainage inlets (DI) were computed by applying the drainage inlet per-acre value derived from the detailed inlet analysis in Appendix A to the entire combined sewer collection system area. Table 11 summarizes the costs for collection system drainage inlet improvements according to design storm.

TABLE 11. SUMMARY OF DRAINAGE INLET IMPROVEMENT COST ESTIMATES

Design Storm	DI Per Acre[1]	Cost Estimate ($ millions)[2]
2-Year	0.34	$ 5.16
5-Year	0.67	$ 10.16
10-Year	0.95	$ 14.41
25-Year	1.13	$ 17.14

1 Added or replaced.

2 Based on a unit cost of $2,600 per DI.

F. ANALYSIS OF SERVICE LATERAL REPLACEMENTS

F.1. Approach to Evaluation

As pipes are replaced in the combined sewer collection system for controlling flooding and outflows, it is anticipated that service laterals connected to replacement pipes will also require replacement and repair. Existing service connections will require disconnection and then replacement during the construction of collection system improvements. Because the majority of the combined sewer system is 50 to 100 years old, it is estimated that a substantial number of service pipes will be damaged.

In order to prepare costs for this component of the overall collection system improvements, several estimates were required: 1) the number of connections per acre in the combined sewer system; 2) the number of service connections to back yard easements that would be unaffected by the collection system improvements; and 3) the number of connections that would require replacement during the construction of collection system pipe replacements. As with the other elements of the collection system evaluation, cost estimates are required for the four design storms included in the analysis.

F.2. Analysis Assumptions and Computations

The following assumptions were used in the analysis of service laterals. The analysis of the number of service connections needed relies on proportions to pipe sizes and lengths for the six representative basins as described below.

1. The City's Department of Data Management reported the number of billing accounts in the combined sewer system to be 24,359. This number was taken to represent the total number of service connections.

2. It is assumed that all service laterals are connected to either 8-inch or 10-inch pipes, for the purpose of this planning level analysis. Further, it is estimated that the number of service laterals connected to pipes of either size is proportional to the total lengths of pipe in the system. If

100 percent of the connections are to either 8- or 10-inch pipes, then 86.3 percent are to 8-inch pipes and 13.7 are to 10-inch pipes.

Calculation:

No. of Service Laterals Connected to 8-inch Pipes: 24,359 X 86.3% = 21,022
No. of Service Laterals Connected to 10-inch Pipes: 24,359 X 13.7% = 3,337

3. From the previous analysis of replacement pipe lengths, and the inventory of back yard easement piping, the percentages of 8-inch and 10-inch collection system pipes that are located in backyard easements (sanitary flow only) are 12.5 and 2.2, respectively. These percentages, which are based on pipe lengths, are assumed to be directly proportional to the number of service laterals that will be unaffected by collection system piping improvements. Conversely, 87.5 percent and 97.8 percent of the existing 8-and 10-inch pipes, respectively, are not in back yard easements and may be affected by collection system improvement construction, depending on the design storm considered.

Calculation:

No. of Connections to 8-inch Pipe not in Back Yard Easements:
21,022 X 87.5% = 18,394
No. of Connections to 10-inch Pipe not in Back Yard Easements:
3,337 X 97.8% = 3,264

4. Using Tables 3 through 6, the proportions of service laterals that remain intact or are replaced can be estimated. For the service laterals connected to 8-inch street pipe:

- 2-year storm: 15.2 percent remain intact / 84.8 percent are replaced
- 5-year storm: 1.3 percent remain intact / 98.7 percent are replaced
- 10-year storm: 1.3 percent remain intact / 98.7 percent are replaced
- 25-year storm: 0.4 percent remain intact / 99.6 percent are replaced

For the service laterals connected to 10-inch street pipe, all are replaced.

Calculation:

No. of Service Laterals Connected to 8-inch Replaced Street Pipes:
2-Year Storm, 18,394 X 84.8% = 15,598
5-Year Storm, 18,394 X 98.7% = 18,155
10-Year Storm, 18,394 X 98.7% = 18,155
25-Year Storm, 18,394 X 99.6% = 18,320

No. of Service Laterals Connected to 10-inch Replaced Street Pipes: 3,264

F.3. Cost Estimates

Using this stepwise analysis process and assumptions, per-acre and total costs were prepared. Table 12 summarizes the computations for each design storm.

TABLE 12. SUMMARY CALCULATIONS AND COST ESTIMATE FOR SEWER SERVICE CONNECTION REPLACEMENT AND REPAIR

Design Storm	Total No. of Replacement Service Laterals[1]	Lateral Replacement Cost[2] ($ millions)
2-year	18,862	$ 46.21
5-year	21,419	$ 52.48
10-year	21,419	$ 52.48
25-year	21,584	$ 52.88

1 Includes both 8- and 10-inch connections.

2 Based on a unit cost of $2,450 per lateral.

G. TOTAL COLLECTION SYSTEM IMPROVEMENT COST ESTIMATES

Table 13 presents the total costs for collection system improvements, including collection system pipe upgrades, drainage inlet improvements, and service lateral repair and replacement. These costs will be used for the cost estimates developed for the alternative improvement plans developed as part of Task 6 during the Phase 3 study.

TABLE 13. SUMMARY OF SYSTEM-WIDE COLLECTION SYSTEM IMPROVEMENT COST ESTIMATES ($ Millions)

Item	2-Year	5-Year	10-Year	25-Year
Average of Pipe Improvement Methods	$99.28	$118.44	$131.69	$146.81
Drainage Inlet Improvements	$5.16	$10.16	$14.41	$17.14
Service Lateral Connections	$46.21	$52.48	$52.48	$52.88
Total Cost	$150.65	$181.08	$198.58	$216.83

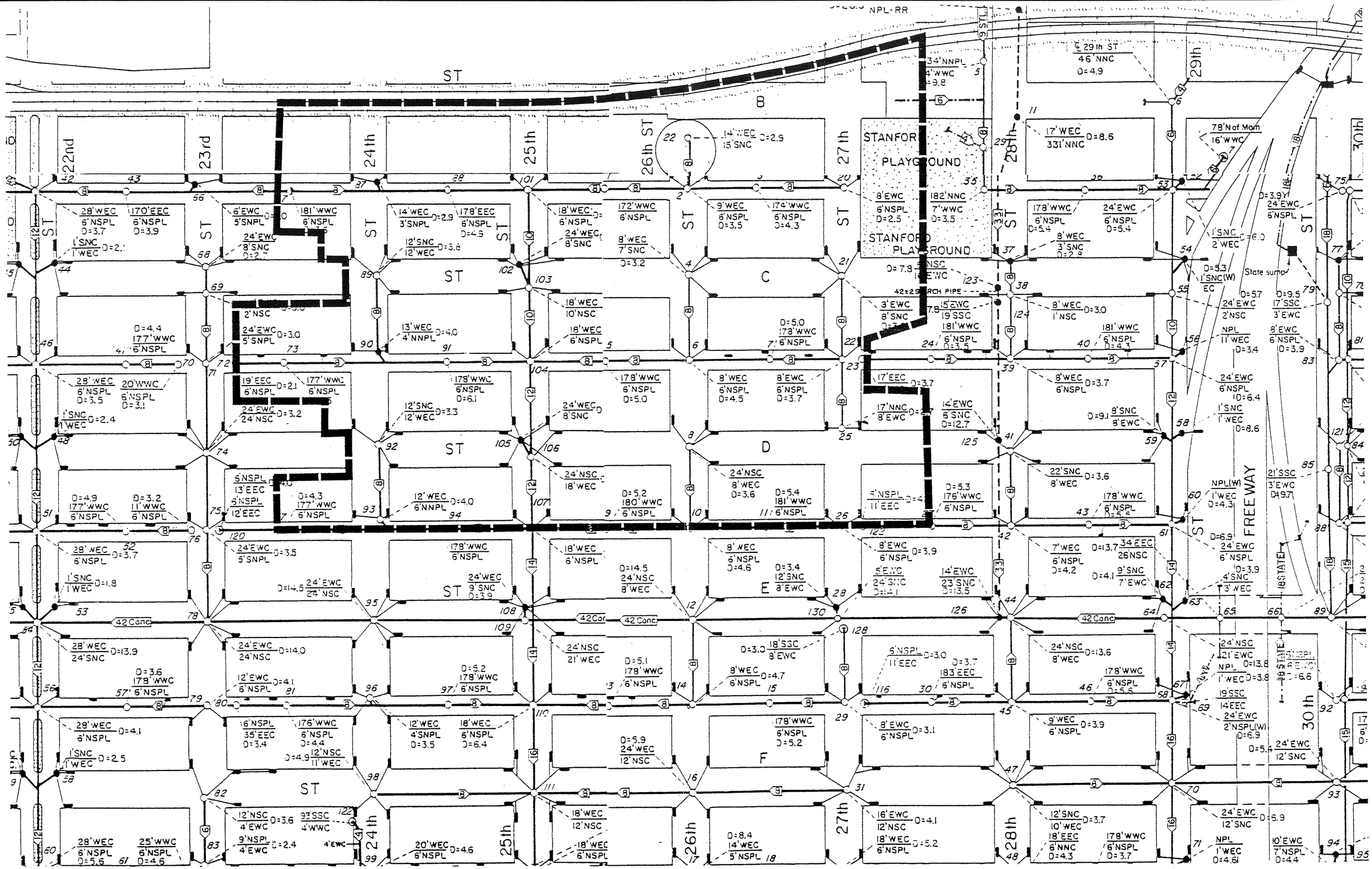
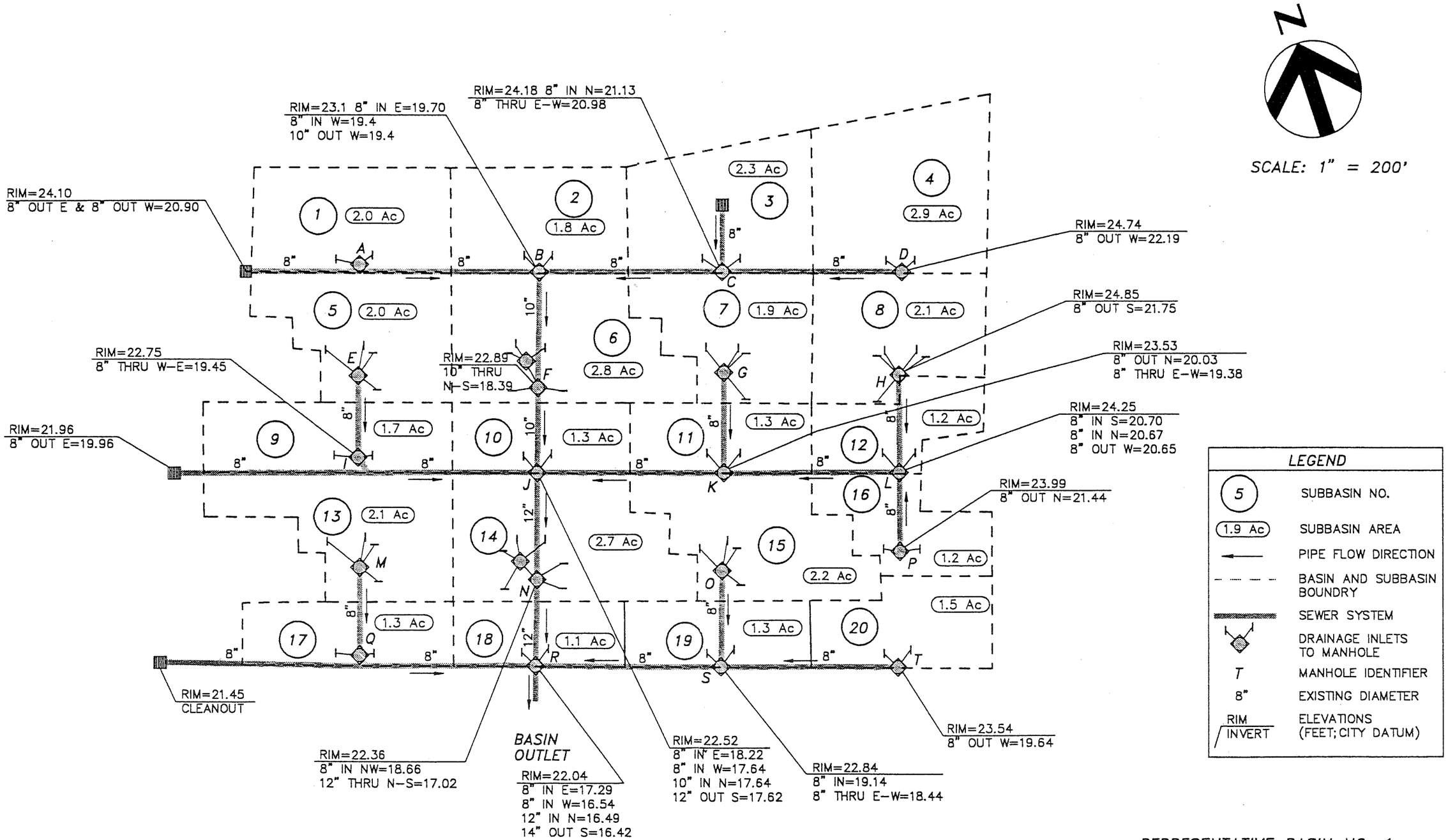


FIGURE 2. REPRESENTATIVE BASIN 1
SWMM Basin 5681 - E & 25th Streets



REPRESENTATIVE BASIN NO. 1
E ST AND 25TH SCHEMATIC

HDR

Figure 3

JOB-NUMBER: 07140-001-141-35 FILE-NAME: BASIN1.DWG REVISION-DATE: -14-92

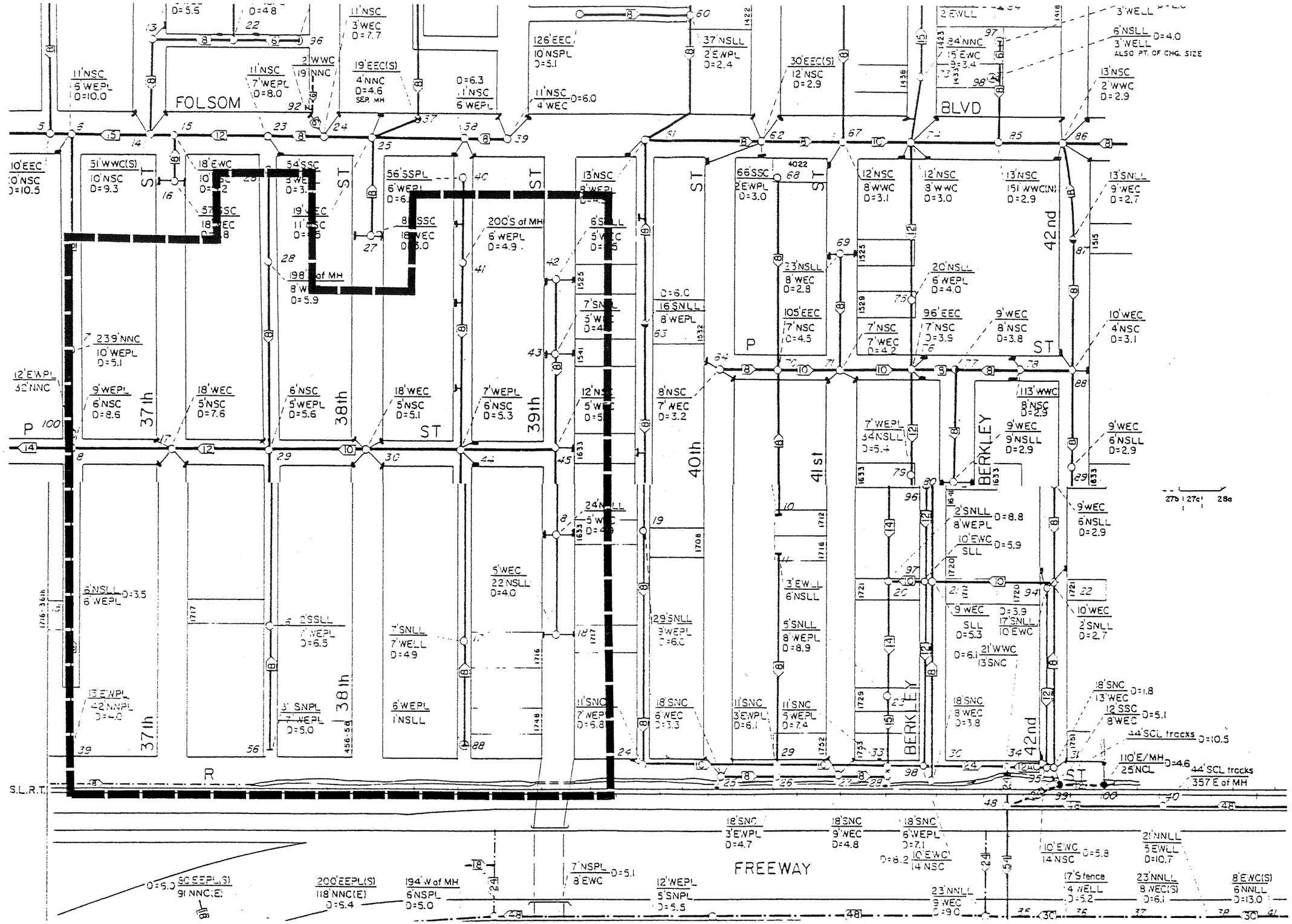
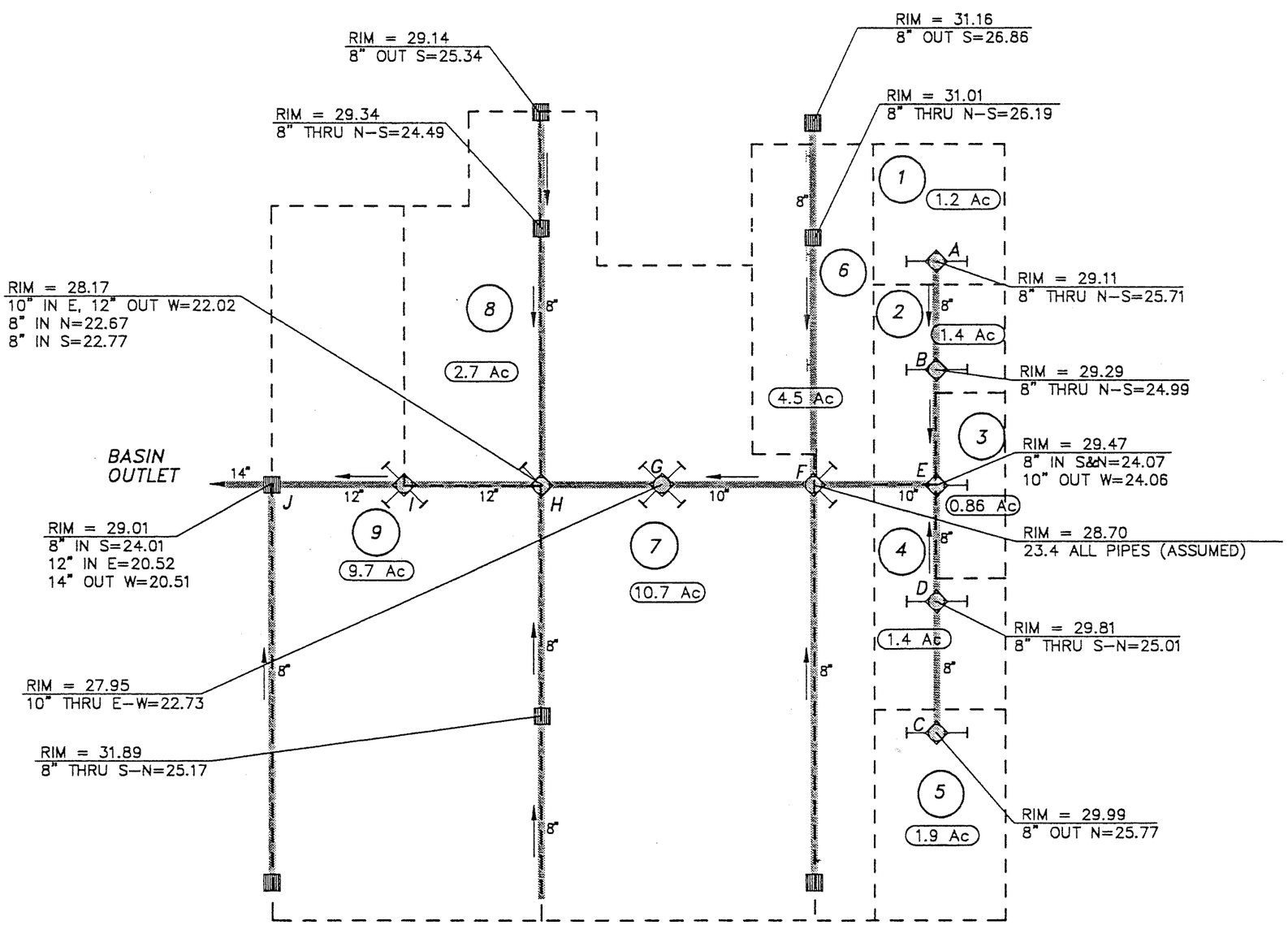


FIGURE 4. REPRESENTATIVE BASIN 2
SWMM Basin 9411 - P & 38th Streets



SCALE: 1" = 200'

LEGEND	
(5)	SUBBASIN NO.
(1.4 Ac)	SUBBASIN AREA
←	PIPE FLOW DIRECTION
— — —	BASIN AND SUBBASIN BOUNDRY
▬	SEWER SYSTEM
◇	DRAINAGE INLETS TO MANHOLE
▥	MANHOLE ONLY
T	MANHOLE IDENTIFIER
8"	EXISTING DIAMETER
RIM / INVERT	ELEVATIONS (FEET; CITY DATUM)

REPRESENTATIVE BASIN NO. 2
P AND 38TH STREETS SCHEMATIC

HDR

Figure 5

JOB-1. JER: 07140-001-141-35 FILE-NAME: BASIN2.DWG REVISIC ATE: 01-14-92

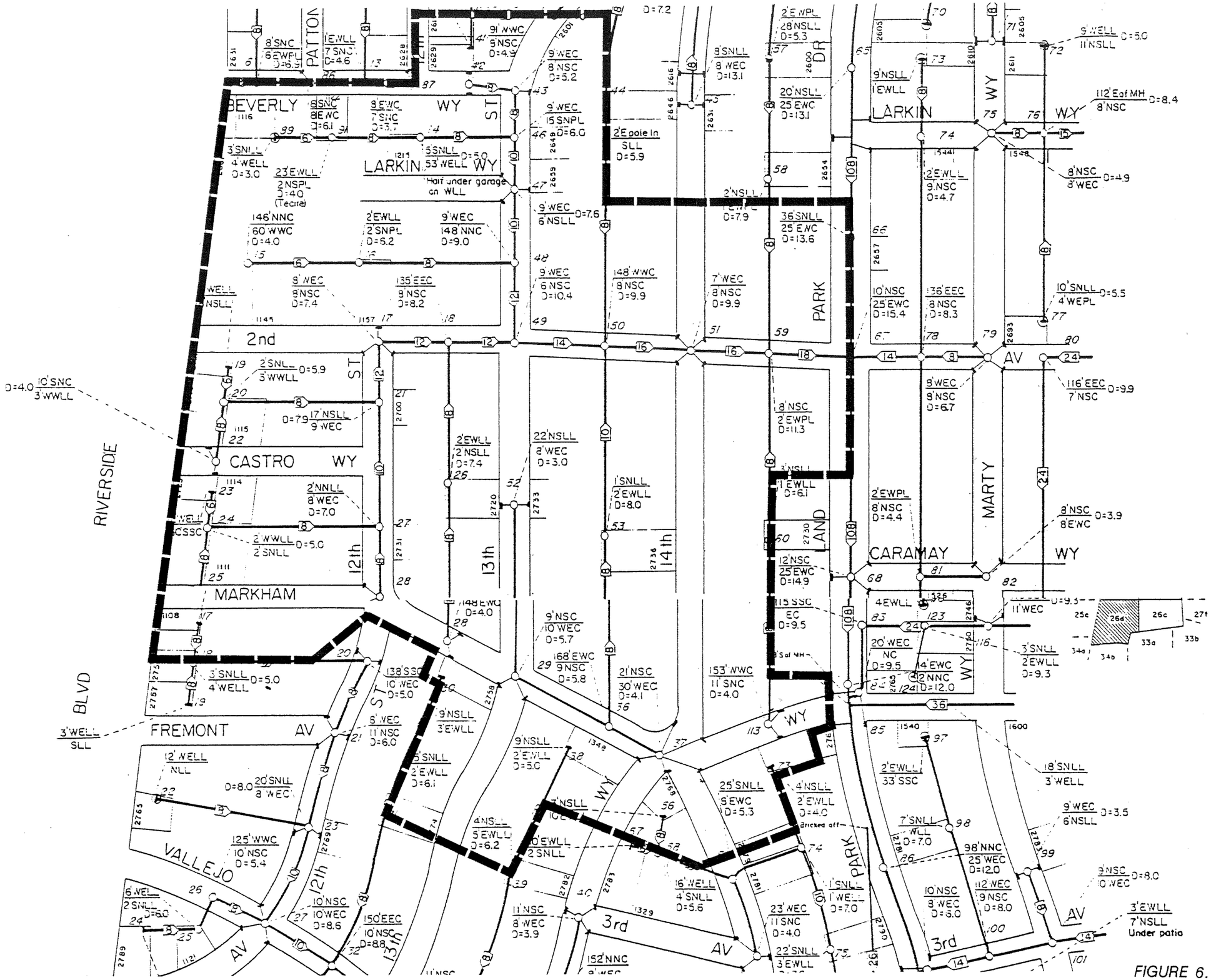
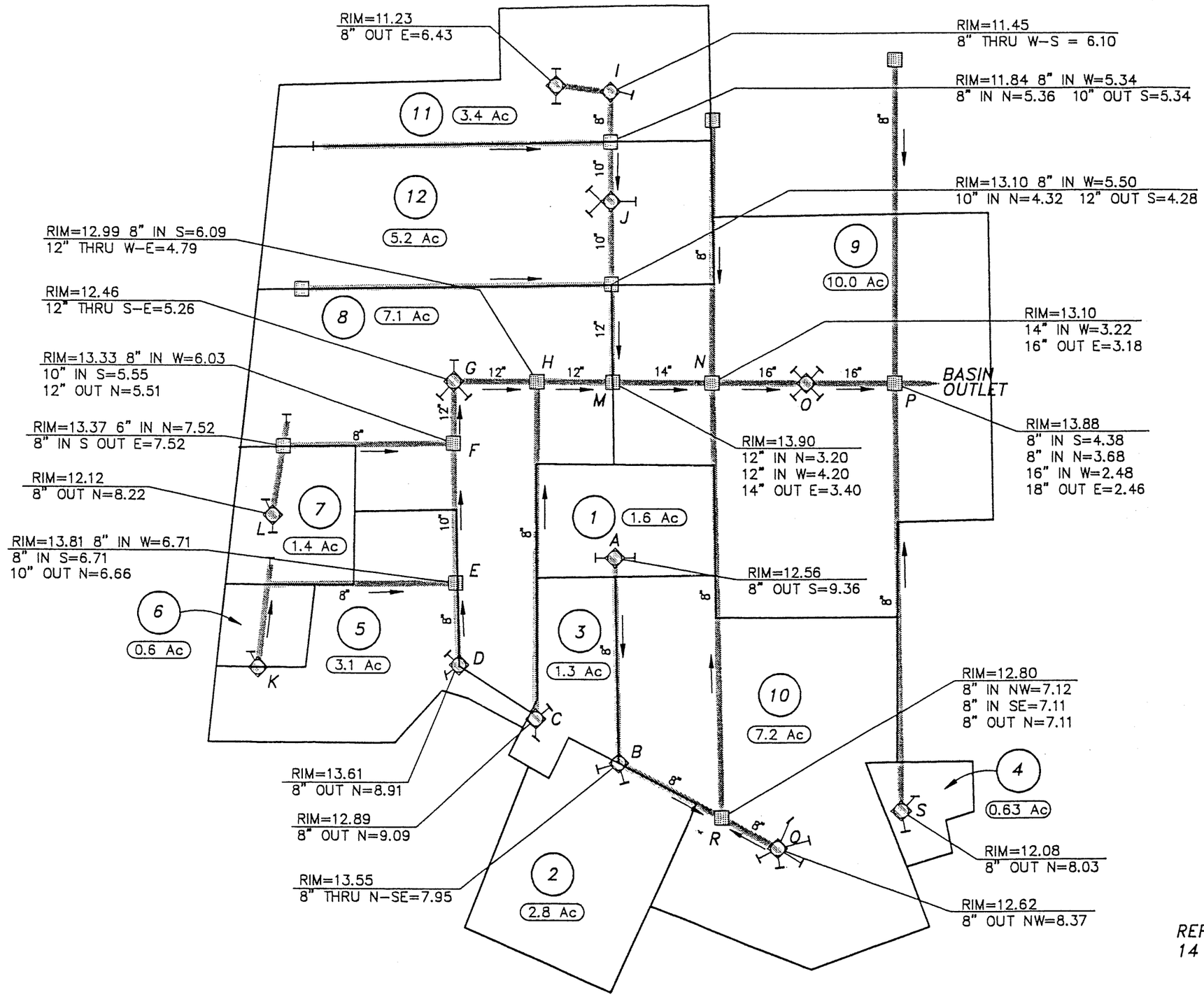


FIGURE 6. REPRESENTATIVE BASIN 3
SWMM Basin 1661, 5101 - 14th Street & 2nd Avenue



SCALE: 1" =200'

LEGEND	
5	SUBBASIN NO.
1.9 Ac	SUBBASIN AREA
←	PIPE FLOW DIRECTION
———	BASIN AND SUBBASIN BOUNDRY
▬▬	SEWER SYSTEM
	DRAINAGE INLETS TO MANHOLE
▩	MANHOLE ONLY
T	MANHOLE IDENTIFIER
8"	EXISTING DIAMETER
RIM / INVERT	ELEVATIONS (FEET; CITY DATUM)

REPRESENTATIVE BASIN NO. 3
14 ST. AND 2ND AVENUE SCHEMATIC

HDR

Figure 7

JOB: 07140-001-141-22 FILE-NAME: BASIN3.DWG REVISION-DA ,-14-92

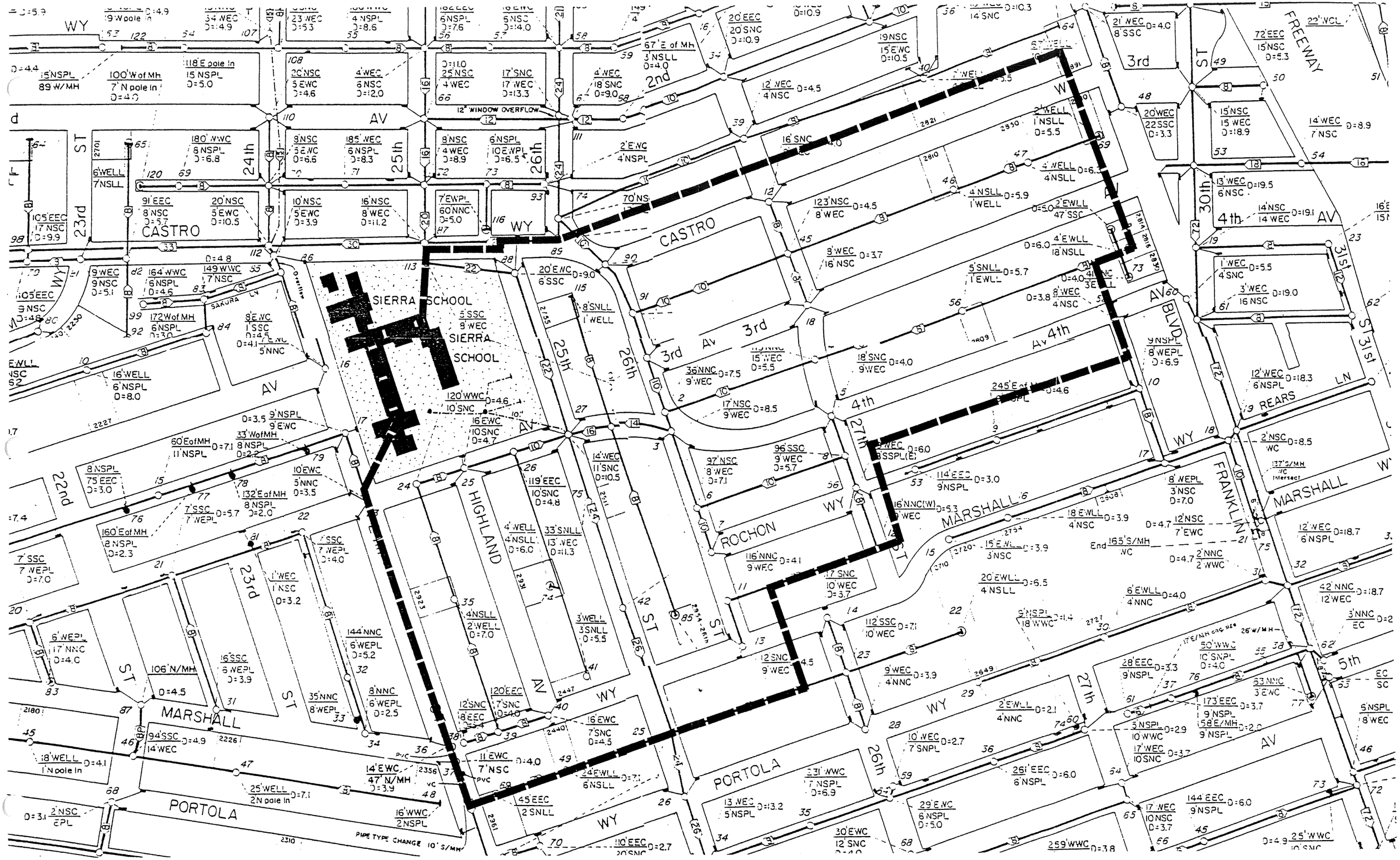
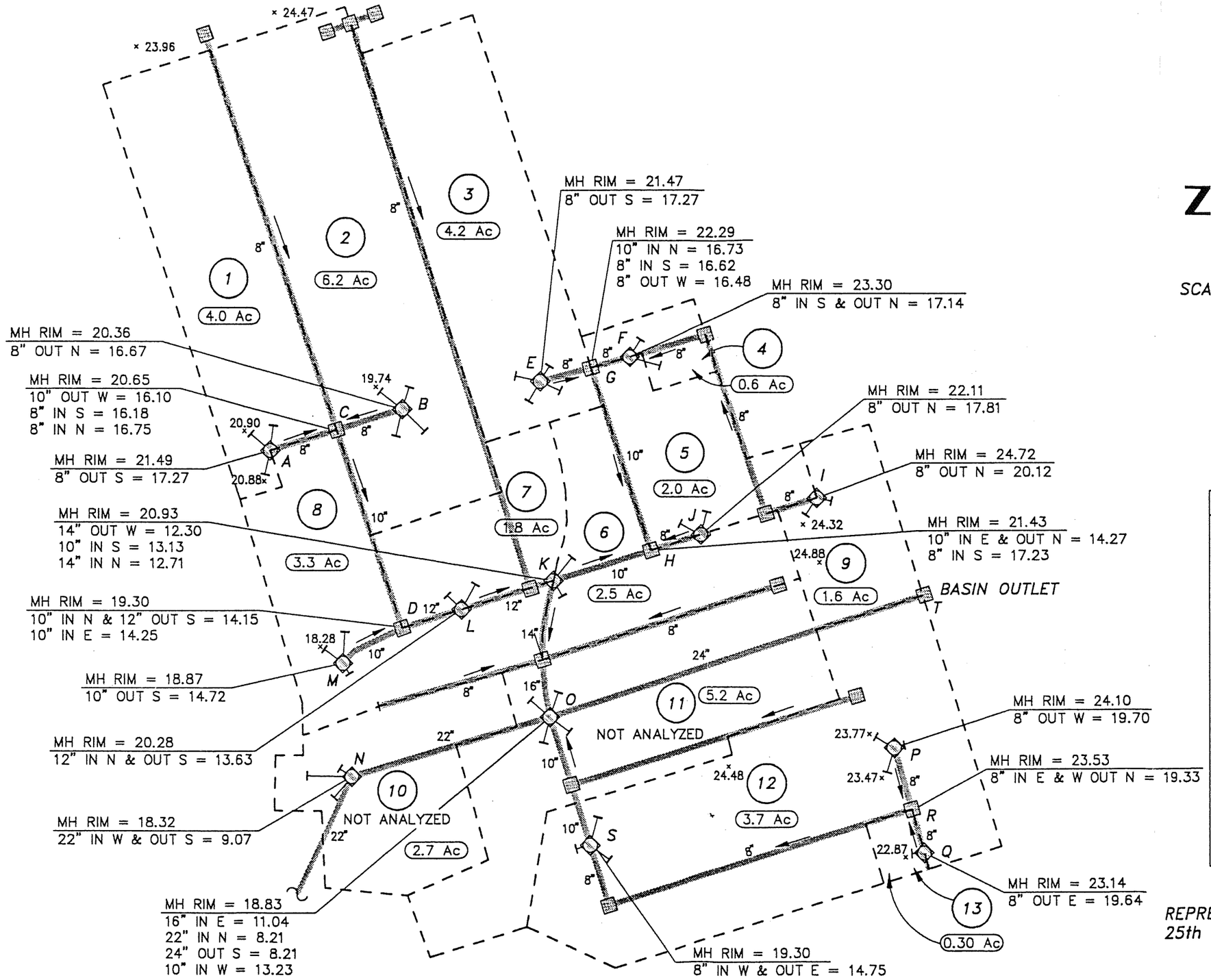


FIGURE 8. REPRESENTATIVE BASIN 4
SWMM Basin 1561 - 25th Street & 4th Avenue



N

SCALE: 1" =200'

LEGEND	
5	SUBBASIN NO.
1.1 Ac	SUBBASIN AREA
←	PIPE FLOW DIRECTION
— — —	BASIN AND SUBBASIN BOUNDRY
═══	SEWER SYSTEM
	DRAINAGE INLETS TO MANHOLE
	MANHOLE ONLY
T	MANHOLE IDENTIFIER
8"	EXISTING DIAMETER
RIM / INVERT	ELEVATIONS (FEET; CITY DATUM)
× 19.92	GUTTER ELEVATION

REPRESENTATIVE BASIN NO. 4
25th STREET & 4th AVENUE SCHEMATIC

HDR

Figure 9

JOB-ER: 07140-001-141-35 FILE-NAME: BASIN4.DWG REVISION-DA1 -14-92

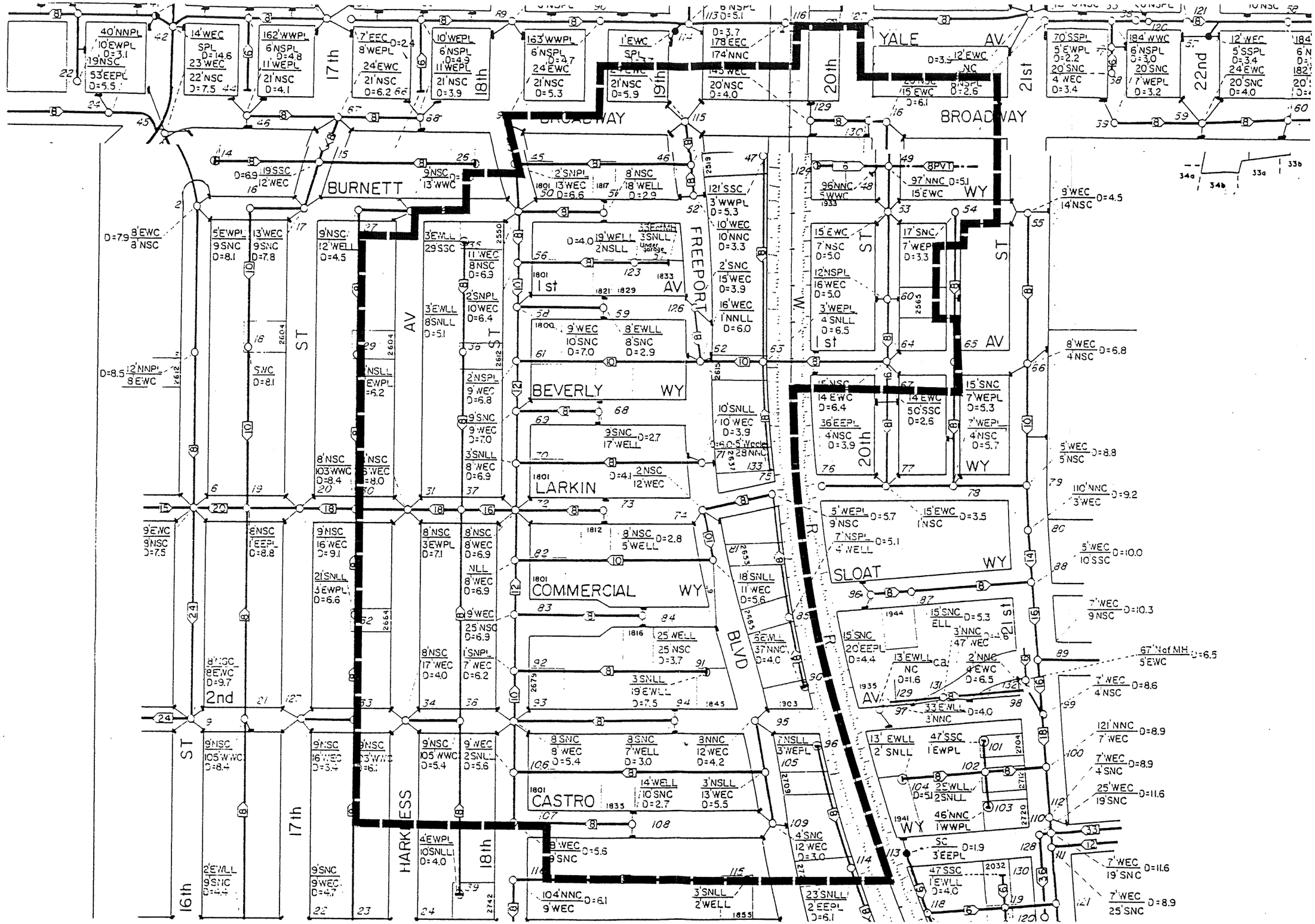


FIGURE 10. REPRESENTATIVE BASIN 5 SWMM Basin 1651 - Harkness Avenue & Larkin Way

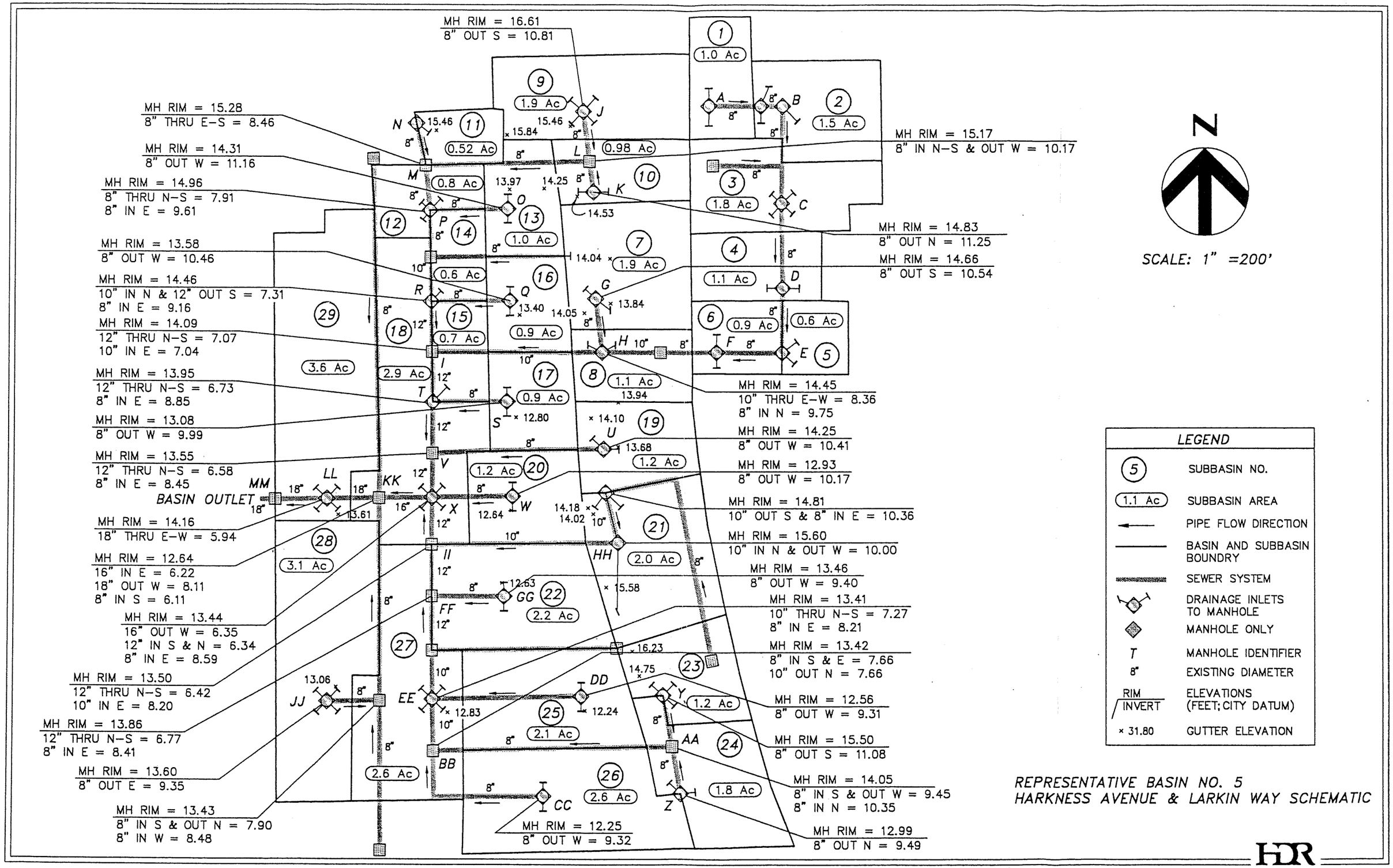


Figure 11

JOB NUMBER: 07140-001-141-35 FILE-NAME: BASIN5.DWG REVISION-1 01-13-92

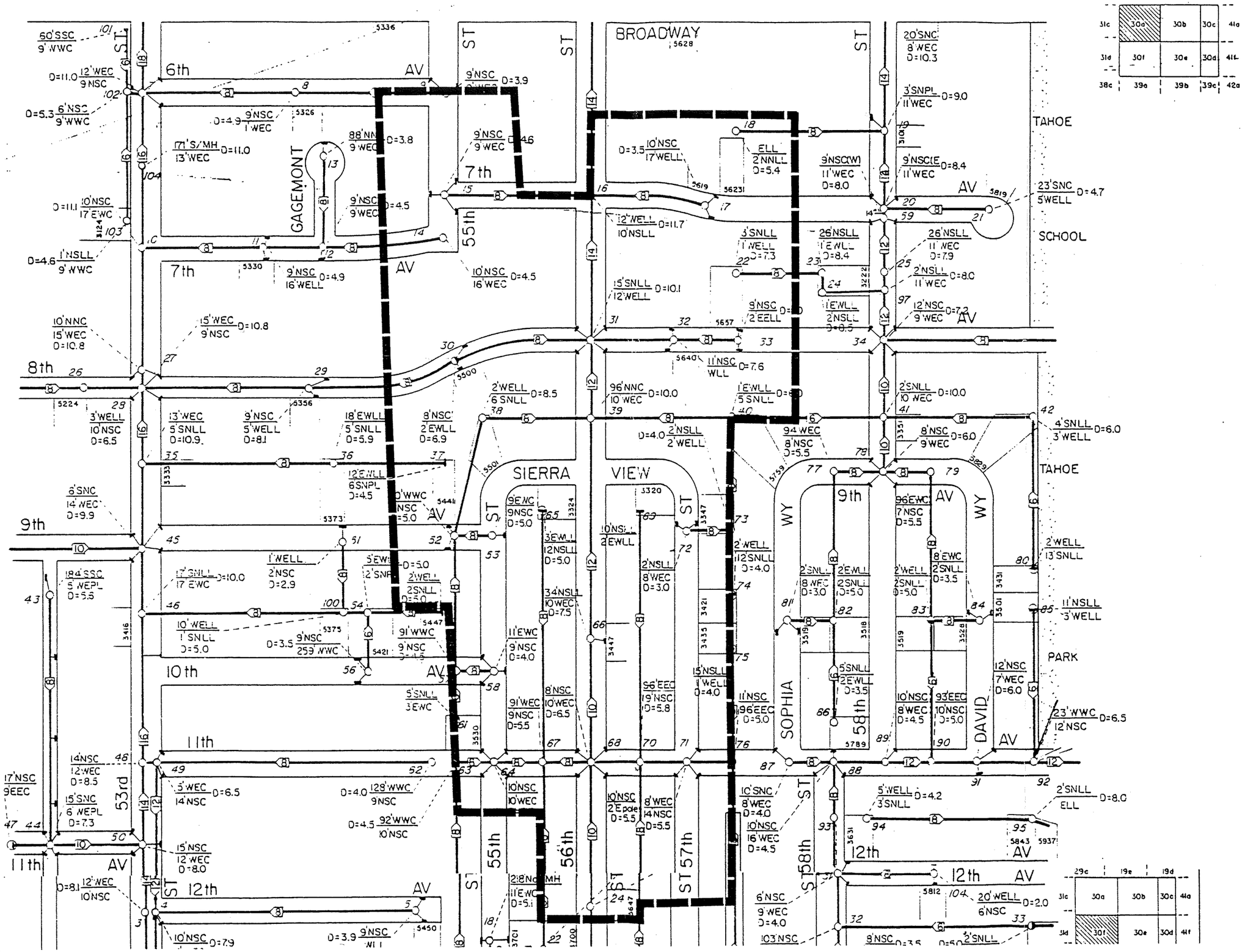
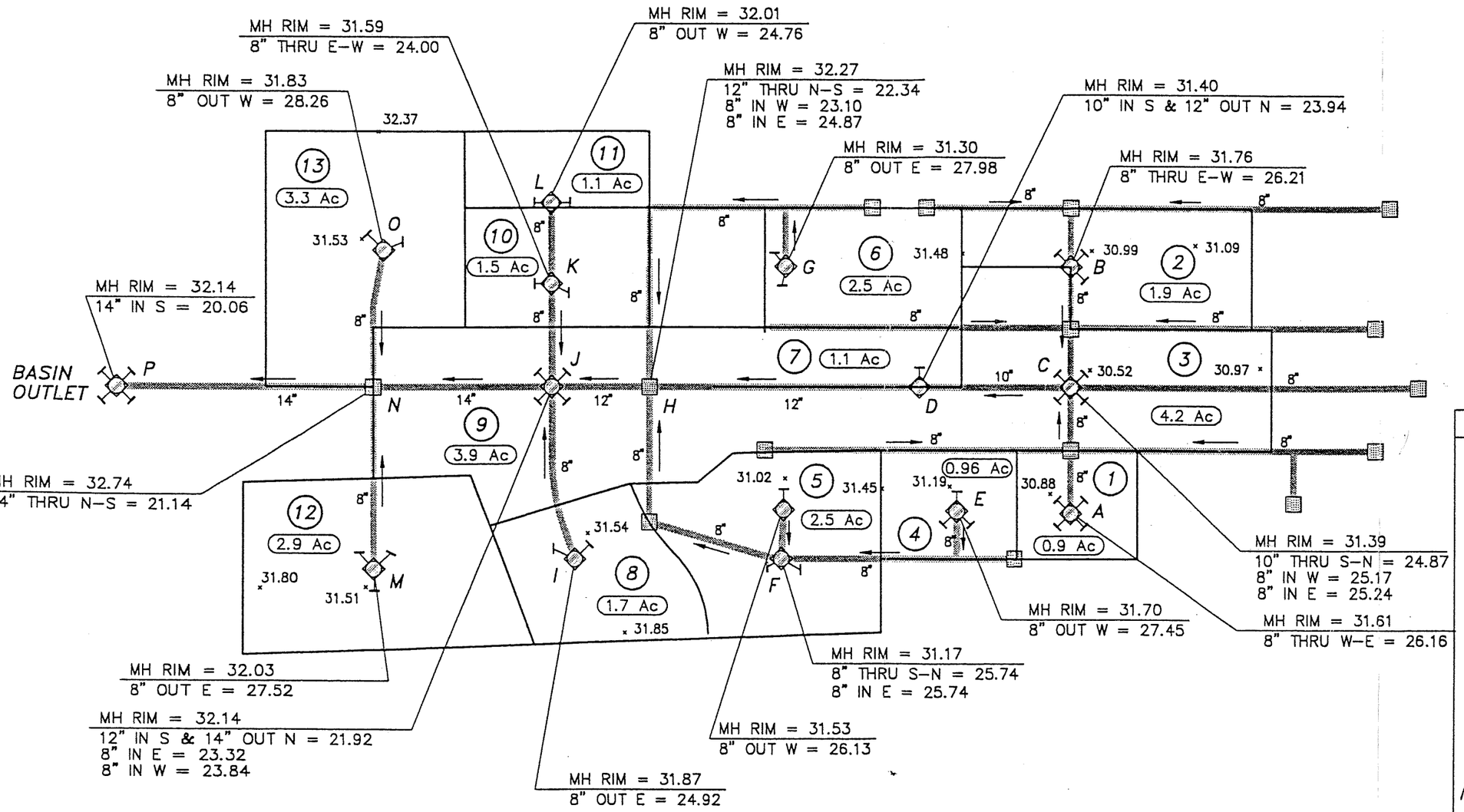


FIGURE 12. REPRESENTATIVE BASIN 6
SWMM Basin 7951 - 56th Street & Broadway



SCALE: 1" =200'

LEGEND	
5	SUBBASIN NO.
1.1 Ac	SUBBASIN AREA
←	PIPE FLOW DIRECTION
—	BASIN AND SUBBASIN BOUNDRY
▬	SEWER SYSTEM
	DRAINAGE INLETS TO MANHOLE
◆	MANHOLE ONLY
T	MANHOLE IDENTIFIER
8"	EXISTING DIAMETER
RIM / INVERT	ELEVATIONS (FEET; CITY DATUM)
× 31.80	GUTTER ELEVATION

REPRESENTATIVE BASIN NO. 6
56th ST. AND BROADWAY SCHEMATIC

HDR

Figure 13

JOB: 07140-001-141-35 FILE-NAME: BASIN6.DWG REVISION-DA 1-14-92

APPENDIX A

DRAINAGE INLET

DETAILED ANALYSIS FOR ALL BASINS

APPENDIX A

DRAINAGE INLET DETAILED ANALYSIS FOR ALL BASINS

TABLE NO.	DESCRIPTION
A-1	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 1 TO 6
A-2	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 1 TO 6
A-3	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 1 TO 6
A-4	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 1 TO 6

TABLE A-1
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASINS 1 TO 6

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

	PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
BASIN 1	A TO B	2.0	2	1.5	0.75	FLOODING	2
	D TO C	2.9	2	1.7	0.85	FLOODING	2
	C TO B		2	2.9			
	B TO F		2	4.4			
	E TO I	2.0	4	1.5	0.38	OK	
	I TO J		2	2.6			
	F TO J		5	5.5			
	H TO L	2.1	5	1.6	0.32	OK	
	P TO L	1.2	2	1.0	0.50	OK	
	L TO K		2	3.3			
	G TO K	1.9	4	1.5	0.38	OK	
	K TO J		2	5.0			
	J TO N		2	11.1			
	T TO S	1.5	2	1.1	0.55	OK	
	O TO S	2.2	4	1.4	0.35	OK	
	S TO R		2	3.0			
	M TO Q	2.1	4	1.2	0.30	OK	
	Q TO R		2	1.7			
	N TO R		6	12.2			
BASIN 2	A TO B	1.2	2	0.9	0.46	OK	
	B TO E		2	1.7			
	C TO D	1.9	2	1.1	0.55	OK	
	D TO E		2	1.8			
	E TO F		1	3.4			
	F TO G		2	5.6			
	G TO H		4	10.3			
	H TO I		1	11.2			
	I TO J		3	15.3			
BASIN 3	A TO B	1.6	2	1.1	0.57	OK	
	B TO R		3	2.7			
	Q TO R	7.2	4	4.3	1.08	FLOODING	4
	R TO N			7.0			
	K TO E	0.6	1	0.6	0.60	OK	
	D TO E	3.1	2	1.8	0.90	FLOODING	2
	E TO F			2.3			
	L TO F		2	1.2			
	F TO G			2.6			
	C TO H	1.3	2	0.8	0.40	OK	
	G TO H		3	6.1			
	H TO M			6.6			
	I TO J	3.4	3	2.2	0.73	FLOODING	3
	J TO M		3	5.2			
	M TO N			10.6			
	S TO P	0.6	2	0.6	0.30	OK	
	N TO P		4	19.9			
BASIN 4	A TO C	4.0	3	2.1	0.70	FLOODING	3
	B TO C	6.2	4	3.4	0.85	FLOODING	4
	C TO D			5.0			
	I TO F	1.6	2	1.3	0.65	OK	
	F TO G		2	1.4			
	E TO G	4.2	4	2.5	0.63	OK	
	G TO H			3.6			
	J TO H	2.0	2	1.4	0.70	OK	
	H TO K			4.5			
	M TO D	3.3	3	2.6	0.87	FLOODING	3
	D TO L			6.2			
	L TO K		2	7.0			
	K TO O		2	11.8			
	Q TO R	0.3	1	0.3	0.30	OK	
	P TO R	3.5	3	2.5	0.83	FLOODING	3
	R TO S			2.6			
	S TO O		3	4.1			

TABLE A-1
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASINS 1 TO 6

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

	PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
BASIN 5	A TO B	1.0	2	0.8	0.40	OK	
	B TO C		3	1.8			
	C TO D		4	2.8			
	D TO E		2	3.3			
	E TO F		2	3.5			
	F TO H		2	3.8			
	G TO H	1.9	2	1.2	0.60	OK	
	H TO I		2	5.2			
	J TO L	1.9	4	1.5	0.38	OK	
	K TO L	1.0	2	0.8	0.40	OK	
	L TO M			2.1			
	N TO M	0.5	1	0.4	0.40	OK	
	M TO P			2.2			
	O TO P	1.0	2	0.9	0.45	OK	
	P TO R		3	3.3			
	Q TO R	0.9	2	0.8	0.40	OK	
	R TO I		1	3.8			
	I TO T			8.0			
	S TO T	0.9	2	0.8	0.40	OK	
	T TO V		1	8.2			
	U TO V	1.2	2	0.9	0.45	OK	
	V TO X			8.5			
	W TO X	1.2	2	1.2	0.60	OK	
	Y TO AA	1.2	3	1.1	0.37	OK	
	Z TO AA	1.8	2	1.5	0.75	FLOODING	2
	AA TO BB			2.2			
	CC TO BB	2.6	2	1.8	0.90	FLOODING	2
	BB TO EE			3.3			
	DD TO EE	2.1	2	1.5	0.75	FLOODING	2
	EE TO FF		4	5.9			
	GG TO FF	2.2	2	2.1	1.05	FLOODING	2
	FF TO II			6.8			
	HH TO II	2.0	3	1.5	0.50	OK	
	II TO X			7.6			
	X TO KK		4	17.0			
	JJ TO KK	3.1	4	1.7	0.43	OK	
	KK TO LL			18.0			
	LL TO MM		4	19.4			
BASIN 6	A TO C	0.9	4	0.7	0.18	OK	
	B TO C	1.9	3	1.3	0.43	OK	
	C TO D		4	4.3			
	D TO H		1	4.6			
	E TO F	1.0	3	0.8	0.27	OK	
	F TO G		3	2.4			
	G TO H	2.5	2	1.3	0.65	OK	
	H TO J			6.4			
	L TO K	1.1	2	0.9	0.46	OK	
	K TO J		2	2.0			
	I TO J	1.7	2	1.3	0.65	OK	
	J TO N		4	9.9			
	O TO N	3.3	2	2.5	1.25	FLOODING	2
	M TO N	2.9	3	2.0	0.67	OK	
	N TO O			12.4			
	TOTALS:	105.7	243				36

0.34 DI/AC X 220.7 AC = 76 NEW DI'S

BASIN 1 - E & 25TH STREETS
BASIN 2 - P & 38TH STREETS
BASIN 3 - 14TH STREET & 2ND AVENUE
BASIN 4 - 25TH STREET & 4TH AVENUE
BASIN 5 - HARKNESS AVE. & LARKIN WAY
BASIN 6 - 56TH STREET & BROADWAY

TABLE A-2
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASINS 1 TO 6

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

	PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
BASIN 1	A TO B	2.0	2	2.3	1.15	FLOODING	2
	D TO C	2.9	2	2.6	1.30	FLOODING	2
	C TO B		2	4.3			
	B TO F		2	6.9			
	E TO I	2.0	4	2.3	0.58	OK	
	I TO J		2	4.2			
	F TO J		5	8.6			
	H TO L	2.1	5	2.5	0.50	OK	
	P TO L	1.2	2	1.6	0.80	FLOODING	2
	L TO K		2	5.0			
	G TO K	1.9	4	2.3	0.58	OK	
	K TO J		2	7.8			
	J TO N		2	17.3			
	T TO S	1.5	2	1.7	0.85	FLOODING	2
	O TO S	2.2	4	2.2	0.55	OK	
	S TO R		2	4.7			
	M TO Q	2.1	4	1.8	0.45	OK	
	Q TO R		2	2.7			
	N TO R		6	18.8			
BASIN 2	A TO B	1.2	2	1.4	0.70	OK	
	B TO E		2	2.5			
	C TO D	1.9	2	1.7	0.85	FLOODING	2
	D TO E		2	2.8			
	E TO F		1	5.5			
	F TO G		2	8.8			
	G TO H		4	16.5			
	H TO I		1	17.8			
	I TO J		3	23.7			
BASIN 3	A TO B	1.6	2	1.8	0.90	FLOODING	2
	B TO R		3	4.3			
	Q TO R	7.2	4	6.7	1.68	FLOODING	4
	R TO N			10.6			
	K TO E	0.6	1	1.0	0.97	FLOODING	1
	D TO E	3.1	2	2.7	1.35	FLOODING	2
	E TO F			3.1			
	L TO F		2	1.8			
	F TO G			4.1			
	C TO H	1.3	2	1.2	0.60	OK	
	G TO H		3	9.5			
	H TO M			10.3			
	I TO J	3.4	3	3.3	1.10	FLOODING	3
	J TO M		3	7.8			
	M TO N			16.4			
	S TO P	0.6	2	1.0	0.49	OK	
	N TO P		4	30.6			
BASIN 4	A TO C	4.0	3	3.1	1.03	FLOODING	3
	B TO C	6.2	4	5.3	1.33	FLOODING	4
	C TO D			7.8			
	I TO F	1.6	2	2.0	1.00	FLOODING	2
	F TO G		2	2.2			
	E TO G	4.2	4	3.8	0.95	FLOODING	4
	G TO H			5.6			
	J TO H	2.0	2	2.2	1.10	FLOODING	2
	H TO K			6.9			
	M TO D	3.3	3	3.9	1.30	FLOODING	3
	D TO L			9.5			
	L TO K		2	10.6			
	K TO O		2	17.9			
	Q TO R	0.3	1	0.5	0.50	OK	
	P TO R	3.5	3	3.8	1.27	FLOODING	3
	R TO S			4.0			
	S TO O		3	6.6			

TABLE A-2
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASINS 1 TO 6

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

	PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
BASIN 5	A TO B	1.0	2	1.2	0.60	OK	
	B TO C		3	2.7			
	C TO D		4	4.4			
	D TO E		2	5.1			
	E TO F		2	5.4			
	F TO H		2	5.9			
	G TO H	1.9	2	1.9	0.95	FLOODING	2
	H TO I		2	8.1			
	J TO L	1.9	4	2.2	0.55	OK	
	K TO L	1.0	2	1.2	0.60	OK	
	L TO M			3.3			
	N TO M	0.5	1	0.7	0.70	OK	
	M TO P			3.5			
	O TO P	1.0	2	1.6	0.80	FLOODING	2
	P TO R		3	5.1			
	Q TO R	0.9	2	1.3	0.65	OK	
	R TO I		1	6.1			
	I TO T			12.5			
	S TO T	0.9	2	1.4	0.70	OK	
	T TO V		1	12.9			
	U TO V	1.2	2	1.4	0.70	OK	
	V TO X			13.4			
	W TO X	1.2	2	1.9	0.95	FLOODING	2
	Y TO AA	1.2	3	1.8	0.60	OK	
	Z TO AA	1.8	2	2.3	1.15	FLOODING	2
	AA TO BB			3.4			
	CC TO BB	2.6	2	2.7	1.35	FLOODING	2
	BB TO EE			5.1			
	DD TO EE	2.1	2	2.3	1.15	FLOODING	2
	EE TO FF		4	9.1			
	GG TO FF	2.2	2	3.4	1.70	FLOODING	2
	FF TO II			10.4			
	HH TO II	2.0	3	2.3	0.77	FLOODING	3
	II TO X			11.9			
	X TO KK		4	26.3			
	JJ TO KK	3.1	4	2.7	0.68	OK	
	KK TO LL			27.9			
	LL TO MM		4	30.1			
BASIN 6	A TO C	0.9	4	1.2	0.30	OK	
	B TO C	1.9	3	1.9	0.63	OK	
	C TO D		4	6.6			
	D TO H		1	7.0			
	E TO F	1.0	3	1.2	0.40	OK	
	F TO G		3	3.7			
	G TO H	2.5	2	2.0	1.00	FLOODING	2
	H TO J			9.9			
	L TO K	1.1	2	1.5	0.75	FLOODING	2
	K TO J		2	3.1			
	I TO J	1.7	2	1.9	0.95	FLOODING	2
	J TO N		4	15.3			
	O TO N	3.3	2	3.8	1.90	FLOODING	2
	M TO N	2.9	3	3.0	1.00	FLOODING	3
	N TO O			18.9			
	TOTALS:	105.7	243				71

0.67 DI/AC X 220.7 AC = 149 NEW DI'S

BASIN 1 - E & 25TH STREETS
BASIN 2 - P & 38TH STREETS
BASIN 3 - 14TH STREET & 2ND AVENUE
BASIN 4 - 25TH STREET & 4TH AVENUE
BASIN 5 - HARKNESS AVE. & LARKIN WAY
BASIN 6 - 56TH STREET & BROADWAY

TABLE A-3
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASINS 1 TO 6

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

	PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
BASIN 1	A TO B	2.0	2	2.8	1.40	FLOODING	2
	D TO C	2.9	2	3.3	1.65	FLOODING	2
	C TO B		2	5.3			
	B TO F		2	8.5			
	E TO I	2.0	4	2.9	0.73	FLOODING	4
	I TO J		2	5.0			
	F TO J		5	10.7			
	H TO L	2.1	5	3.1	0.62	OK	
	P TO L	1.2	2	2.0	1.00	FLOODING	2
	L TO K		2	6.4			
	G TO K	1.9	4	2.8	0.70	OK	
	K TO J		2	9.9			
	J TO N		2	21.6			
	T TO S	1.5	2	2.1	1.05	FLOODING	2
	O TO S	2.2	4	2.8	0.70	OK	
	S TO R		2	5.9			
	M TO Q	2.1	4	2.2	0.55	OK	
	Q TO R		2	3.4			
	N TO R		6	23.1			
BASIN 2	A TO B	1.2	2	1.7	0.85	FLOODING	2
	B TO E		2	3.1			
	C TO D	1.9	2	2.1	1.05	FLOODING	2
	D TO E		2	3.4			
	E TO F		1	6.8			
	F TO G		2	10.8			
	G TO H		4	21.7			
	H TO I		1	23.2			
	I TO J		3	31.4			
BASIN 3	A TO B	1.6	2	2.2	1.10	FLOODING	2
	B TO R		3	5.3			
	Q TO R	7.2	4	8.4	2.10	FLOODING	4
	R TO N			13.1			
	K TO E	0.6	1	1.3	1.30	FLOODING	1
	D TO E	3.1	2	3.3	1.65	FLOODING	2
	E TO F			3.9			
	L TO F		2	2.3			
	F TO G			5.1			
	C TO H	1.3	2	1.5	0.75	FLOODING	2
	G TO H		3	11.8			
	H TO M			12.7			
	I TO J	3.4	3	4.1	1.37	FLOODING	3
	J TO M		3	9.8			
	M TO N			20.2			
	S TO P	0.6	2	1.3	0.65	OK	
	N TO P		4	38.2			
BASIN 4	A TO C	4.0	3	3.9	1.30	FLOODING	3
	B TO C	6.2	4	6.5	1.63	FLOODING	4
	C TO D			9.5			
	I TO F	1.6	2	2.4	1.20	FLOODING	2
	F TO G		2	2.7			
	E TO G	4.2	4	4.7	1.18	FLOODING	4
	G TO H			7.0			
	J TO H	2.0	2	2.7	1.35	FLOODING	2
	H TO K			8.6			
	M TO D	3.3	3	4.9	1.63	FLOODING	3
	D TO L			11.7			
	L TO K		2	13.1			
	K TO O		2	22.1			
	Q TO R	0.3	1	0.6	0.60	OK	
	P TO R	3.5	3	4.7	1.57	FLOODING	3
	R TO S			5.0			
	S TO O		3	8.2			

TABLE A-3
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASINS 1 TO 6

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

	PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
BASIN 5	A TO B	1.0	2	1.5	0.75	FLOODING	2
	B TO C		3	3.4			
	C TO D		4	5.5			
	D TO E		2	6.4			
	E TO F		2	6.8			
	F TO H		2	7.5			
	G TO H	1.9	2	2.3	1.15	FLOODING	2
	H TO I		2	10.1			
	J TO L	1.9	4	2.8	0.70	OK	
	K TO L	1.0	2	1.5	0.75	FLOODING	2
	L TO M			4.2			
	N TO M	0.5	1	0.8	0.80	FLOODING	1
	M TO P			4.4			
	O TO P	1.0	2	2.0	1.00	FLOODING	2
	P TO R		3	6.4			
	Q TO R	0.9	2	1.7	0.85	FLOODING	2
	R TO I		1	7.7			
	I TO T			15.9			
	S TO T	0.9	2	1.8	0.90	FLOODING	2
	T TO V		1	16.5			
	U TO V	1.2	2	1.8	0.90	FLOODING	2
	V TO X			17.1			
	W TO X	1.2	2	2.5	1.25	FLOODING	2
	Y TO AA	1.2	3	2.3	0.77	FLOODING	3
	Z TO AA	1.8	2	3.0	1.50	FLOODING	2
	AA TO BB			4.2			
	CC TO BB	2.6	2	3.4	1.70	FLOODING	2
	BB TO EE			6.5			
	DD TO EE	2.1	2	2.8	1.40	FLOODING	2
	EE TO FF		4	11.5			
	GG TO FF	2.2	2	4.5	2.25	FLOODING	2
	FF TO II			13.1			
	HH TO II	2.0	3	2.8	0.93	FLOODING	3
	II TO X			14.9			
	X TO KK		4	33.4			
	JJ TO KK	3.1	4	3.4	0.85	FLOODING	4
	KK TO LL			35.1			
	LL TO MM		4	37.6			
BASIN 6	A TO C	0.9	4	1.5	0.38	OK	
	B TO C	1.9	3	2.4	0.80	FLOODING	3
	C TO D		4	8.2			
	D TO H		1	8.7			
	E TO F	1.0	3	1.6	0.53	OK	
	F TO G		3	4.7			
	G TO H	2.5	2	2.5	1.25	FLOODING	2
	H TO J			12.4			
	L TO K	1.1	2	1.8	0.90	FLOODING	2
	K TO J		2	3.9			
	I TO J	1.7	2	2.4	1.20	FLOODING	2
	J TO N		4	19.1			
	O TO N	3.3	2	4.8	2.40	FLOODING	2
	M TO N	2.9	3	3.7	1.23	FLOODING	3
	N TO O			23.7			
	TOTALS:	105.7	243				100

0.95 DI/AC X 220.7 AC = 209 NEW DI'S

BASIN 1 - E & 25TH STREETS
BASIN 2 - P & 38TH STREETS
BASIN 3 - 14TH STREET & 2ND AVENUE
BASIN 4 - 25TH STREET & 4TH AVENUE
BASIN 5 - HARKNESS AVE. & LARKIN WAY
BASIN 6 - 56TH STREET & BROADWAY

TABLE A-4
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASINS 1 TO 6

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

	PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
BASIN 1	A TO B	2.0	2	3.6	1.80	FLOODING	2
	D TO C	2.9	2	4.2	2.10	FLOODING	2
	C TO B		2	6.9			
	B TO F		2	11.1			
	E TO I	2.0	4	3.7	0.93	FLOODING	4
	I TO J		2	6.4			
	F TO J		5	14.0			
	H TO L	2.1	5	4.0	0.80	FLOODING	5
	P TO L	1.2	2	2.6	1.30	FLOODING	2
	L TO K		2	8.1			
	G TO K	1.9	4	3.6	0.90	FLOODING	4
	K TO J		2	12.6			
	J TO N		2	28.2			
	T TO S	1.5	2	2.7	1.35	FLOODING	2
	O TO S	2.2	4	3.6	0.90	FLOODING	4
	S TO R		2	7.5			
	M TO Q	2.1	4	2.8	0.70	OK	
	Q TO R		2	4.4			
	N TO R		6	30.1			
BASIN 2	A TO B	1.2	2	2.2	1.10	FLOODING	2
	B TO E		2	4.0			
	C TO D	1.9	2	2.7	1.35	FLOODING	2
	D TO E		2	4.4			
	E TO F		1	8.7			
	F TO G		2	14.0			
	G TO H		4	26.2			
	H TO I		1	28.4			
	I TO J		3	37.9			
BASIN 3	A TO B	1.6	2	2.9	1.45	FLOODING	2
	B TO R		3	6.9			
	Q TO R	7.2	4	10.7	2.68	FLOODING	4
	R TO N			16.8			
	K TO E	0.6	1	1.7	1.70	FLOODING	1
	D TO E	3.1	2	4.2	2.10	FLOODING	2
	E TO F			5.0			
	L TO F		2	3.0			
	F TO G			6.6			
	C TO H	1.3	2	1.8	0.90	FLOODING	2
	G TO H		3	15.2			
	H TO M			16.3			
	I TO J	3.4	3	5.3	1.77	FLOODING	3
	J TO M		3	12.5			
	M TO N			26.2			
	S TO P	0.6	2	1.7	0.85	FLOODING	2
	N TO P		4	49.4			
BASIN 4	A TO C	4.0	3	4.9	1.63	FLOODING	3
	B TO C	6.2	4	8.3	2.08	FLOODING	4
	C TO D			12.2			
	I TO F	1.6	2	3.1	1.55	FLOODING	2
	F TO G		2	3.5			
	E TO G	4.2	4	6.0	1.50	FLOODING	4
	G TO H			8.9			
	J TO H	2.0	2	3.4	1.70	FLOODING	2
	H TO K			11.1			
	M TO D	3.3	3	6.3	2.10	FLOODING	3
	D TO L			15.1			
	L TO K		2	16.8			
	K TO O		2	28.4			
	Q TO R	0.3	1	0.7	0.70	OK	
	P TO R	3.5	3	6.1	2.03	FLOODING	3
	R TO S			6.4			
	S TO O		3	10.6			

TABLE A-4
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASINS 1 TO 6

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

	PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
BASIN 5	A TO B	1.0	2	1.9	0.95	FLOODING	2
	B TO C		3	4.4			
	C TO D		4	7.1			
	D TO E		2	8.3			
	E TO F		2	8.9			
	F TO H		2	9.7			
	G TO H	1.9	2	3.0	1.50	FLOODING	2
	H TO I		2	13.2			
	J TO L	1.9	4	3.6	0.90	FLOODING	4
	K TO L	1.0	2	1.9	0.95	FLOODING	2
	L TO M			5.3			
	N TO M	0.5	1	1.1	1.10	FLOODING	1
	M TO P			5.6			
	O TO P	1.0	2	2.7	1.35	FLOODING	2
	P TO R		3	8.4			
	Q TO R	0.9	2	2.2	1.10	FLOODING	2
	R TO I		1	10.1			
	I TO T			20.6			
	S TO T	0.9	2	2.4	1.20	FLOODING	2
	T TO V		1	21.4			
	U TO V	1.2	2	2.3	1.15	FLOODING	2
	V TO X			22.2			
	W TO X	1.2	2	3.3	1.65	FLOODING	2
	Y TO AA	1.2	3	2.9	0.97	FLOODING	3
	Z TO AA	1.8	2	3.8	1.90	FLOODING	2
	AA TO BB			5.4			
	CC TO BB	2.6	2	4.3	2.15	FLOODING	2
	BB TO EE			8.3			
	DD TO EE	2.1	2	3.6	1.80	FLOODING	2
	EE TO FF		4	14.7			
	GG TO FF	2.2	2	5.9	2.95	FLOODING	2
	FF TO II			16.8			
	HH TO II	2.0	3	3.6	1.20	FLOODING	3
	II TO X			19.1			
	X TO KK		4	43.3			
	JJ TO KK	3.1	4	4.3	1.08	FLOODING	4
	KK TO LL			45.9			
	LL TO MM		4	48.8			
BASIN 6	A TO C	0.9	4	1.9	0.48	OK	
	B TO C	1.9	3	3.1	1.03	FLOODING	3
	C TO D		4	10.6			
	D TO H		1	11.4			
	E TO F	1.0	3	2.0	0.67	OK	
	F TO G		3	6.0			
	G TO H	2.5	2	3.2	1.60	FLOODING	2
	H TO J			15.8			
	L TO K	1.1	2	2.4	1.20	FLOODING	2
	K TO J		2	5.0			
	I TO J	1.7	2	3.1	1.55	FLOODING	2
	J TO N		4	24.3			
	O TO N	3.3	2	6.1	3.05	FLOODING	2
	M TO N	2.9	3	4.8	1.60	FLOODING	3
	N TO O			30.4			
	TOTALS:	105.7	243				119

1.13 DI/AC X 220.7 AC = 249 NEW DI'S

BASIN 1 - E & 25TH STREETS
BASIN 2 - P & 38TH STREETS
BASIN 3 - 14TH STREET & 2ND AVENUE
BASIN 4 - 25TH STREET & 4TH AVENUE
BASIN 5 - HARKNESS AVE. & LARKIN WAY
BASIN 6 - 56TH STREET & BROADWAY

APPENDIX B

DRAINAGE INLET

DETAILED ANALYSIS FOR INDIVIDUAL BASINS

APPENDIX B

DRAINAGE INLET DETAILED ANALYSIS FOR INDIVIDUAL BASINS

TABLE NO.	DESCRIPTION
B-1	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 1: E AND 25TH STREETS
B-2	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 1: E AND 25TH STREETS
B-3	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 1: E AND 25TH STREETS
B-4	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 1: E AND 25TH STREETS
B-5	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 2: P AND 38TH STREETS
B-6	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 2: P AND 38TH STREETS
B-7	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 2: P AND 38TH STREETS
B-8	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 2: P AND 38TH STREETS
B-9	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE
B-10	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE
B-11	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE
B-12	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE

APPENDIX B

DRAINAGE INLET DETAILED ANALYSIS FOR INDIVIDUAL BASINS

TABLE NO.	DESCRIPTION
B-13	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE
B-14	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE
B-15	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE
B-16	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE
B-17	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY
B-18	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY
B-19	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY
B-20	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY
B-21	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY
B-22	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY
B-23	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY
B-24	DRAINAGE INLET IMPROVEMENTS REPLACEMENT ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY

TABLE B-1
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	2	2	1.5	0.75	FLOODING	2
D TO C	2.9	2	1.7	0.85	FLOODING	2
C TO B		2	2.9			
B TO F		2	4.4			
E TO I	2	4	1.5	0.38	OK	
I TO J		2	2.6			
F TO J		5	5.5			
H TO L	2.1	5	1.6	0.32	OK	
P TO L	1.2	2	1.0	0.50	OK	
L TO K		2	3.3			
G TO K	1.9	4	1.5	0.38	OK	
K TO J		2	5.0			
J TO N		2	11.1			
T TO S	1.5	2	1.1	0.55	OK	
O TO S	2.2	4	1.4	0.35	OK	
S TO R		2	3.0			
M TO Q	2.1	4	1.2	0.30	OK	
Q TO R		2	1.7			
N TO R		6	12.2			
OUTLET (R)		2	16.1			
TOTALS:	17.9	58				4

0.22 DI/AC X 36.7 AC = 9 NEW DI'S

TABLE B-2
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	2	2	2.3	1.15	FLOODING	2
D TO C	2.9	2	2.6	1.30	FLOODING	2
C TO B		2	4.3			
B TO F		2	6.9			
E TO I	2	4	2.3	0.58	OK	
I TO J		2	4.2			
F TO J		5	8.6			
H TO L	2.1	5	2.5	0.50	OK	
P TO L	1.2	2	1.6	0.80	FLOODING	2
L TO K		2	5.0			
G TO K	1.9	4	2.3	0.58	OK	
K TO J		2	7.8			
J TO N		2	17.3			
T TO S	1.5	2	1.7	0.85	FLOODING	2
O TO S	2.2	4	2.2	0.55	OK	
S TO R		2	4.7			
M TO Q	2.1	4	1.8	0.45	OK	
Q TO R		2	2.7			
N TO R		6	18.8			
OUTLET (R)		2	25.0			
TOTALS:	17.9	58				8

0.45 DI/AC X 36.7 AC = 17 NEW DI'S

TABLE B-3
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	2	2	2.8	1.40	FLOODING	2
D TO C	2.9	2	3.3	1.65	FLOODING	2
C TO B		2	5.3			
B TO F		2	8.5			
E TO I	2	4	2.9	0.73	FLOODING	4
I TO J		2	5.0			
F TO J		5	10.7			
H TO L	2.1	5	3.1	0.62	OK	
P TO L	1.2	2	2.0	1.00	FLOODING	2
L TO K		2	6.4			
G TO K	1.9	4	2.8	0.70	OK	
K TO J		2	9.9			
J TO N		2	21.6			
T TO S	1.5	2	2.1	1.05	FLOODING	2
O TO S	2.2	4	2.8	0.70	OK	
S TO R		2	5.9			
M TO Q	2.1	4	2.2	0.55	OK	
Q TO R		2	3.4			
N TO R		6	23.1			
OUTLET (R)		2	30.7			
TOTALS:	17.9	58				12

0.67 DI/AC X 36.7 AC = 25 NEW DI'S

TABLE B-4
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	2	2	3.6	1.80	FLOODING	2
D TO C	2.9	2	4.2	2.10	FLOODING	2
C TO B		2	6.9			
B TO F		2	11.1			
E TO I	2	4	3.7	0.93	FLOODING	4
I TO J		2	6.4			
F TO J		5	14.0			
H TO L	2.1	5	4.0	0.80	FLOODING	5
P TO L	1.2	2	2.6	1.30	FLOODING	2
L TO K		2	8.1			
G TO K	1.9	4	3.6	0.90	FLOODING	4
K TO J		2	12.6			
J TO N		2	28.2			
T TO S	1.5	2	2.7	1.35	FLOODING	2
O TO S	2.2	4	3.6	0.90	FLOODING	4
S TO R		2	7.5			
M TO Q	2.1	4	2.8	0.70	OK	
Q TO R		2	4.4			
N TO R		6	30.1			
OUTLET		2	40.0			
TOTALS:	17.9	58				25

1.40 DI/AC X 36.7 AC = 52 NEW DI'S

TABLE B-5
DRAINAGE INLET IMPROVEMENT REPLACMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.2	2	0.9	0.46	OK	
B TO E		2	1.7			
C TO D	1.9	2	1.1	0.55	OK	
D TO E		2	1.8			
E TO F		1	3.4			
F TO G		2	5.6			
G TO H		4	10.3			
H TO I		1	11.2			
I TO J		3	15.3			
TOTALS:	3.1	19				0

0 DI/AC X 34.1 AC = 0 NEW DI'S

TABLE B-6
DRAINAGE INLET IMPROVEMENT REPLACMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.2	2	1.4	0.70	OK	
B TO E		2	2.5			
C TO D	1.9	2	1.7	0.85	FLOODING	2
D TO E		2	2.8			
E TO F		1	5.5			
F TO G		2	8.8			
G TO H		4	16.5			
H TO I		1	17.8			
I TO J		3	23.7			
TOTALS:	3.1	19				2

0.65 DI/AC X 34.1 AC = 23 NEW DI'S

TABLE B-7
DRAINAGE INLET IMPROVEMENT REPLACMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.2	2	1.7	0.85	FLOODING	2
B TO E		2	3.1			
C TO D	1.9	2	2.1	1.05	FLOODING	2
D TO E		2	3.4			
E TO F		1	6.8			
F TO G		2	10.8			
G TO H		4	21.7			
H TO I		1	23.2			
I TO J		3	31.4			
TOTALS:	3.1	19				4

1.29 DI/AC X 34.1 AC = 45 NEW DI'S

TABLE B-8
DRAINAGE INLET IMPROVEMENT REPLACMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.2	2	2.2	1.10	FLOODING	2
B TO E		2	4.0			
C TO D	1.9	2	2.7	1.35	FLOODING	2
D TO E		2	4.4			
E TO F		1	8.7			
F TO G		2	14.0			
G TO H		4	26.2			
H TO I		1	28.4			
I TO J		3	37.9			
TOTALS:	3.1	19				4

1.29 DI/AC X 34.1 AC = 45 NEW DI'S

TABLE B-9
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.6	2	1.1	0.57	OK	
B TO R		3	2.7			
Q TO R	7.2	4	4.3	1.08	FLOODING	4
R TO N			7.0			
K TO E	0.6	1	0.6	0.60	OK	
D TO E	3.1	2	1.8	0.90	FLOODING	2
E TO F			2.3			
L TO F		2	1.2			
F TO G			2.6			
C TO H	1.3	2	0.8	0.40	OK	
G TO H		3	6.1			
H TO M			6.6			
I TO J	3.4	3	2.2	0.73	FLOODING	3
J TO M		3	5.2			
M TO N			10.6			
S TO P	0.63	2	0.6	0.30	OK	
N TO P		4	19.9			
TOTALS:	17.83	31				9

0.50 DI/AC X 43.7 AC = 23 NEW DI'S

TABLE B-10
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.6	2	1.8	0.90	FLOODING	2
B TO R		3	4.3			
Q TO R	7.2	4	6.7	1.68	FLOODING	4
R TO N			10.6			
K TO E	0.6	1	1.0	0.97	FLOODING	1
D TO E	3.1	2	2.7	1.35	FLOODING	2
E TO F			3.1			
L TO F		2	1.8			
F TO G			4.1			
C TO H	1.3	2	1.2	0.60	OK	
G TO H		3	9.5			
H TO M			10.3			
I TO J	3.4	3	3.3	1.10	FLOODING	3
J TO M		3	7.8			
M TO N			16.4			
S TO P	0.63	2	1.0	0.49	OK	
N TO P		4	30.6			
TOTALS:	17.83	31				12

0.67 DI/AC X 43.7 AC = 30 NEW DI'S

TABLE B-11
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.6	2	2.2	1.10	FLOODING	2
B TO R		3	5.3			
Q TO R	7.2	4	8.4	2.10	FLOODING	4
R TO N			13.1			
K TO E	0.6	1	1.3	1.30	FLOODING	1
D TO E	3.1	2	3.3	1.65	FLOODING	2
E TO F			3.9			
L TO F		2	2.3			
F TO G			5.1			
C TO H	1.3	2	1.5	0.75	FLOODING	2
G TO H		3	11.8			
H TO M			12.7			
I TO J	3.4	3	4.1	1.37	FLOODING	3
J TO M		3	9.8			
M TO N			20.2			
S TO P	0.63	2	1.3	0.65	OK	
N TO P		4	38.2			
TOTALS:	17.83	31				14

0.79 DI/AC X 43.7 AC = 35 NEW DI'S

TABLE B-12
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.6	2	2.9	1.45	FLOODING	2
B TO R		3	6.9			
Q TO R	7.2	4	10.7	2.68	FLOODING	4
R TO N			16.8			
K TO E	0.6	1	1.7	1.70	FLOODING	1
D TO E	3.1	2	4.2	2.10	FLOODING	2
E TO F			5.0			
L TO F		2	3.0			
F TO G			6.6			
C TO H	1.3	2	1.8	0.90	FLOODING	2
G TO H		3	15.2			
H TO M			16.3			
I TO J	3.4	3	5.3	1.77	FLOODING	3
J TO M		3	12.5			
M TO N			26.2			
S TO P	0.63	2	1.7	0.85	FLOODING	2
N TO P		4	49.4			
TOTALS:	17.83	31				16

0.90 DI/AC X 43.7 AC = 40 NEW DI'S

TABLE B-13
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO C	4.0	3	2.1	0.70	FLOODING	3
B TO C	6.2	4	3.4	0.85	FLOODING	4
C TO D			5.0			
I TO F	1.6	2	1.3	0.65	OK	
F TO G		2	1.4			
E TO G	4.2	4	2.5	0.63	OK	
G TO H			3.6			
J TO H	2.0	2	1.4	0.70	OK	
H TO K			4.5			
M TO D	3.3	3	2.6	0.87	FLOODING	3
D TO L			6.2			
L TO K		2	7.0			
K TO O		2	11.8			
Q TO R	0.3	1	0.3	0.30	OK	
P TO R	3.5	3	2.5	0.83	FLOODING	3
R TO S			2.6			
S TO O		3	4.1			
TOTALS:	25.1	31				13

0.52 DI/AC X 33.7 AC = 18 NEW DI'S

TABLE B-14
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO C	4	3	3.1	1.03	FLOODING	3
B TO C	6.2	4	5.3	1.33	FLOODING	4
C TO D			7.8			
I TO F	1.6	2	2.0	1.00	FLOODING	2
F TO G		2	2.2			
E TO G	4.2	4	3.8	0.95	FLOODING	4
G TO H			5.6			
J TO H	2	2	2.2	1.10	FLOODING	2
H TO K			6.9			
M TO D	3.3	3	3.9	1.30	FLOODING	3
D TO L			9.5			
L TO K		2	10.6			
K TO O		2	17.9			
Q TO R	0.3	1	0.5	0.50	OK	
P TO R	3.5	3	3.8	1.27	FLOODING	3
R TO S			4.0			
S TO O		3	6.6			
TOTALS:	25.1	31				21

0.84 DI/AC X 33.7 AC = 29 NEW DI'S

TABLE B-15
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO C	4	3	3.9	1.30	FLOODING	3
B TO C	6.2	4	6.5	1.63	FLOODING	4
C TO D			9.5			
I TO F	1.6	2	2.4	1.20	FLOODING	2
F TO G		2	2.7			
E TO G	4.2	4	4.7	1.18	FLOODING	4
G TO H			7.0			
J TO H	2	2	2.7	1.35	FLOODING	2
H TO K			8.6			
M TO D	3.3	3	4.9	1.63	FLOODING	3
D TO L			11.7			
L TO K		2	13.1			
K TO O		2	22.1			
Q TO R	0.3	1	0.6	0.60	OK	
P TO R	3.5	3	4.7	1.57	FLOODING	3
R TO S			5.0			
S TO O		3	8.2			
TOTALS:	25.1	31				21

0.84 DI/AC X 33.7 AC = 29 NEW DI'S

TABLE B-16
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO C	4	3	4.9	1.63	FLOODING	3
B TO C	6.2	4	8.3	2.08	FLOODING	4
C TO D			12.2			
I TO F	1.6	2	3.1	1.55	FLOODING	2
F TO G		2	3.5			
E TO G	4.2	4	6.0	1.50	FLOODING	4
G TO H			8.9			
J TO H	2	2	3.4	1.70	FLOODING	2
H TO K			11.1			
M TO D	3.3	3	6.3	2.10	FLOODING	3
D TO L			15.1			
L TO K		2	16.8			
K TO O		2	28.4			
Q TO R	0.3	1	0.7	0.70	OK	
P TO R	3.5	3	6.1	2.03	FLOODING	3
R TO S			6.4			
S TO O		3	10.6			
TOTALS:	25.1	31				21

0.84 DI/AC X 33.7 AC = 29 NEW DI'S

TABLE B-17
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.0	2	0.8	0.40	OK	
B TO C		3	1.8			
C TO D		4	2.8			
D TO E		2	3.3			
E TO F		2	3.5			
F TO H		2	3.8			
G TO H	1.9	2	1.2	0.60	OK	
H TO I		2	5.2			
J TO L	1.9	4	1.5	0.38	OK	
K TO L	1.0	2	0.8	0.40	OK	
L TO M			2.1			
N TO M	0.5	1	0.4	0.40	OK	
M TO P			2.2			
O TO P	1.0	2	0.9	0.45	OK	
P TO R		3	3.3			
Q TO R	0.9	2	0.8	0.40	OK	
R TO I		1	3.8			
I TO T			8.0			
S TO T	0.9	2	0.8	0.40	OK	
T TO V		1	8.2			
U TO V	1.2	2	0.9	0.45	OK	
V TO X			8.5			
W TO X	1.2	2	1.2	0.60	OK	
Y TO AA	1.2	3	1.1	0.37	OK	
Z TO AA	1.8	2	1.5	0.75	FLOODING	2
AA TO BB			2.2			
CC TO BB	2.6	2	1.8	0.90	FLOODING	2
BB TO EE			3.3			
DD TO EE	2.1	2	1.5	0.75	FLOODING	2
EE TO FF		4	5.9			
GG TO FF	2.2	2	2.1	1.05	FLOODING	2
FF TO II			6.8			
HH TO II	2.0	3	1.5	0.50	OK	

TABLE B-18
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.0	2	1.2	0.60	OK	
B TO C		3	2.7			
C TO D		4	4.4			
D TO E		2	5.1			
E TO F		2	5.4			
F TO H		2	5.9			
G TO H	1.9	2	1.9	0.95	FLOODING	2
H TO I		2	8.1			
J TO L	1.9	4	2.2	0.55	OK	
K TO L	1.0	2	1.2	0.60	OK	
L TO M			3.3			
N TO M	0.5	1	0.7	0.70	OK	
M TO P			3.5			
O TO P	1.0	2	1.6	0.80	FLOODING	2
P TO R		3	5.1			
Q TO R	0.9	2	1.3	0.65	OK	
R TO I		1	6.1			
I TO T			12.5			
S TO T	0.9	2	1.4	0.70	OK	
T TO V		1	12.9			
U TO V	1.2	2	1.4	0.70	OK	
V TO X			13.4			
W TO X	1.2	2	1.9	0.95	FLOODING	2
Y TO AA	1.2	3	1.8	0.60	OK	
Z TO AA	1.8	2	2.3	1.15	FLOODING	2
AA TO BB			3.4			
CC TO BB	2.6	2	2.7	1.35	FLOODING	2
BB TO EE			5.1			
DD TO EE	2.1	2	2.3	1.15	FLOODING	2
EE TO FF		4	9.1			
GG TO FF	2.2	2	3.4	1.70	FLOODING	2
FF TO II			10.4			
HH TO II	2.0	3	2.3	0.77	FLOODING	3

TABLE B-17
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
II TO X			7.6			
X TO KK		4	17.0			
JJ TO KK	3.1	4	1.7	0.43	OK	
KK TO LL			18.0			
LL TO MM		4	19.4			
TOTALS:	26.5	71				8

0.30 DI/AC X 44 AC = 14 NEW DI'S

TABLE B-18
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
II TO X			11.9			
X TO KK		4	26.3			
JJ TO KK	3.1	4	2.7	0.68	OK	
KK TO LL			27.9			
LL TO MM		4	30.1			
TOTALS:	26.5	71				17

0.64 DI/AC X 44 AC = 29 NEW DI'S

TABLE B-19
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.0	2	1.5	0.75	FLOODING	2
B TO C		3	3.4			
C TO D		4	5.5			
D TO E		2	6.4			
E TO F		2	6.8			
F TO H		2	7.5			
G TO H	1.9	2	2.3	1.15	FLOODING	2
H TO I		2	10.1			
J TO L	1.9	4	2.8	0.70	OK	
K TO L	1.0	2	1.5	0.75	FLOODING	2
L TO M			4.2			
N TO M	0.5	1	0.8	0.80	FLOODING	1
M TO P			4.4			
O TO P	1.0	2	2.0	1.00	FLOODING	2
P TO R		3	6.4			
Q TO R	0.9	2	1.7	0.85	FLOODING	2
R TO I		1	7.7			
I TO T			15.9			
S TO T	0.9	2	1.8	0.90	FLOODING	2
T TO V		1	16.5			
U TO V	1.2	2	1.8	0.90	FLOODING	2
V TO X			17.1			
W TO X	1.2	2	2.5	1.25	FLOODING	2
Y TO AA	1.2	3	2.3	0.77	FLOODING	3
Z TO AA	1.8	2	3.0	1.50	FLOODING	2
AA TO BB			4.2			
CC TO BB	2.6	2	3.4	1.70	FLOODING	2
BB TO EE			6.5			
DD TO EE	2.1	2	2.8	1.40	FLOODING	2
EE TO FF		4	11.5			
GG TO FF	2.2	2	4.5	2.25	FLOODING	2
FF TO II			13.1			
HH TO II	2.0	3	2.8	0.93	FLOODING	3

TABLE B-20
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO B	1.0	2	1.9	0.95	FLOODING	2
B TO C		3	4.4			
C TO D		4	7.1			
D TO E		2	8.3			
E TO F		2	8.9			
F TO H		2	9.7			
G TO H	1.9	2	3.0	1.50	FLOODING	2
H TO I		2	13.2			
J TO L	1.9	4	3.6	0.90	FLOODING	4
K TO L	1.0	2	1.9	0.95	FLOODING	2
L TO M			5.3			
N TO M	0.5	1	1.1	1.10	FLOODING	1
M TO P			5.6			
O TO P	1.0	2	2.7	1.35	FLOODING	2
P TO R		3	8.4			
Q TO R	0.9	2	2.2	1.10	FLOODING	2
R TO I		1	10.1			
I TO T			20.6			
S TO T	0.9	2	2.4	1.20	FLOODING	2
T TO V		1	21.4			
U TO V	1.2	2	2.3	1.15	FLOODING	2
V TO X			22.2			
W TO X	1.2	2	3.3	1.65	FLOODING	2
Y TO AA	1.2	3	2.9	0.97	FLOODING	3
Z TO AA	1.8	2	3.8	1.90	FLOODING	2
AA TO BB			5.4			
CC TO BB	2.6	2	4.3	2.15	FLOODING	2
BB TO EE			8.3			
DD TO EE	2.1	2	3.6	1.80	FLOODING	2
EE TO FF		4	14.7			
GG TO FF	2.2	2	5.9	2.95	FLOODING	2
FF TO II			16.8			
HH TO II	2.0	3	3.6	1.20	FLOODING	3

TABLE B-19
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
II TO X			14.9			
X TO KK		4	33.4			
JJ TO KK	3.1	4	3.4	0.85	FLOODING	4
KK TO LL			35.1			
LL TO MM		4	37.6			
TOTALS:	26.5	71				35

1.32 DI/AC X 44 AC = 59 NEW DI'S

TABLE B-20
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
II TO X			19.1			
X TO KK		4	43.3			
JJ TO KK	3.1	4	4.3	1.08	FLOODING	4
KK TO LL			45.9			
LL TO MM		4	48.8			
TOTALS:	26.5	71				39

1.47 DI/AC X 44 AC = 65 NEW DI'S

TABLE B-21
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO C	0.9	4	0.7	0.18	OK	
B TO C	1.9	3	1.3	0.43	OK	
C TO D		4	4.3			
D TO H		1	4.6			
E TO F	0.96	3	0.8	0.27	OK	
F TO G		3	2.4			
G TO H	2.5	2	1.3	0.65	OK	
H TO J			6.4			
L TO K	1.1	2	0.9	0.46	OK	
K TO J		2	2.0			
I TO J	1.7	2	1.3	0.65	OK	
J TO N		4	9.9			
O TO N	3.3	2	2.5	1.25	FLOODING	2
M TO N	2.9	3	2.0	0.67	OK	
N TO O			12.4			
TOTALS:	15.26	35				2

0.13 DI/AC X 28.5 AC = 4 NEW DI'S

TABLE B-22
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO C	0.9	4	1.2	0.30	OK	
B TO C	1.9	3	1.9	0.63	OK	
C TO D		4	6.6			
D TO H		1	7.0			
E TO F	0.96	3	1.2	0.40	OK	
F TO G		3	3.7			
G TO H	2.5	2	2.0	1.00	FLOODING	2
H TO J			9.9			
L TO K	1.1	2	1.5	0.75	FLOODING	2
K TO J		2	3.1			
I TO J	1.7	2	1.9	0.95	FLOODING	2
J TO N		4	15.3			
O TO N	3.3	2	3.8	1.90	FLOODING	2
M TO N	2.9	3	3.0	1.00	FLOODING	3
N TO O			18.9			
TOTALS:	15.26	35				11

0.72 DI/AC X 28.5 AC = 21 NEW DI'S

TABLE B-23
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO C	0.9	4	1.5	0.38	OK	
B TO C	1.9	3	2.4	0.80	FLOODING	3
C TO D		4	8.2			
D TO H		1	8.7			
E TO F	0.96	3	1.6	0.53	OK	
F TO G		3	4.7			
G TO H	2.5	2	2.5	1.25	FLOODING	2
H TO J			12.4			
L TO K	1.1	2	1.8	0.90	FLOODING	2
K TO J		2	3.9			
I TO J	1.7	2	2.4	1.20	FLOODING	2
J TO N		4	19.1			
O TO N	3.3	2	4.8	2.40	FLOODING	2
M TO N	2.9	3	3.7	1.23	FLOODING	3
N TO O			23.7			
TOTALS:	15.26	35				14

0.92 DI/AC X 28.5 AC = 27 NEW DI'S

TABLE B-24
DRAINAGE INLET IMPROVEMENT REPLACEMENT ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

EXISTING INLET - MAX. Q: 0.7 CFS
REPLACEMENT INLET - MAX. Q: 5.8 CFS

PIPE SECTION	DRAINAGE AREA (AC)	NO. EXIST. DI'S	PEAK FLOW (CFS)	FLOW/DI (CFS)	FLOODING OR OK	NO. NEW DI'S
A TO C	0.9	4	1.9	0.48	OK	
B TO C	1.9	3	3.1	1.03	FLOODING	3
C TO D		4	10.6			
D TO H		1	11.4			
E TO F	0.96	3	2.0	0.67	OK	
F TO G		3	6.0			
G TO H	2.5	2	3.2	1.60	FLOODING	2
H TO J			15.8			
L TO K	1.1	2	2.4	1.20	FLOODING	2
K TO J		2	5.0			
I TO J	1.7	2	3.1	1.55	FLOODING	2
J TO N		4	24.3			
O TO N	3.3	2	6.1	3.05	FLOODING	2
M TO N	2.9	3	4.8	1.60	FLOODING	3
N TO O			30.4			
TOTALS:	15.26	35				14

0.92 DI/AC X 28.5 AC = 27 NEW DI'S

APPENDIX C

DETAILED ANALYSIS OF PIPE REPLACEMENT LENGTHS AND INVENTORY OF BACKYARD EASEMENTS

APPENDIX C

DETAILED ANALYSIS OF PIPE REPLACEMENT LENGTHS AND INVENTORY OF BACKYARD EASEMENTS

TABLE NO.	DESCRIPTION
C-1	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 2-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 1: E AND 25TH STREETS
C-2	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 1: E AND 25TH STREETS 2-YEAR RATIONAL METHOD ANALYSIS
C-3	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 1: E AND 25TH STREETS 2-YEAR RATIONAL METHOD ANALYSIS
C-4	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 5-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 1: E AND 25TH STREETS
C-5	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 1: E AND 25TH STREETS 5-YEAR RATIONAL METHOD ANALYSIS
C-6	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 1: E AND 25TH STREETS 5-YEAR RATIONAL METHOD ANALYSIS
C-7	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 10-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 1: E AND 25TH STREETS
C-8	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 1: E AND 25TH STREETS 10-YEAR RATIONAL METHOD ANALYSIS
C-9	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 1: E AND 25TH STREETS 10-YEAR RATIONAL METHOD ANALYSIS
C-10	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 25-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 1: E AND 25TH STREETS
C-11	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 1: E AND 25TH STREETS 25-YEAR RATIONAL METHOD ANALYSIS

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TABLE NO.	DESCRIPTION
C-12	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 1: E AND 25TH STREETS 25-YEAR RATIONAL METHOD ANALYSIS
C-13	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 2-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 2: P AND 38TH STREETS
C-14	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 2: P AND 38TH STREETS 2-YEAR RATIONAL METHOD ANALYSIS
C-15	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 2: P AND 38TH STREETS 2-YEAR RATIONAL METHOD ANALYSIS
C-16	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 5-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 2: P AND 38TH STREETS
C-17	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 2: P AND 38TH STREETS 5-YEAR RATIONAL METHOD ANALYSIS
C-18	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 2: P AND 38TH STREETS 5-YEAR RATIONAL METHOD ANALYSIS
C-19	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 10-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 2: P AND 38TH STREETS
C-20	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 2: P AND 38TH STREETS 10-YEAR RATIONAL METHOD ANALYSIS
C-21	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 2: P AND 38TH STREETS 10-YEAR RATIONAL METHOD ANALYSIS
C-22	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 25-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 2: P AND 38TH STREETS

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DETAILED ANALYSIS OF PIPE REPLACEMENT LENGTHS AND INVENTORY OF BACKYARD EASEMENTS

TABLE NO.	DESCRIPTION
C-23	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 2: P AND 38TH STREETS 25-YEAR RATIONAL METHOD ANALYSIS
C-24	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 2: P AND 38TH STREETS 25-YEAR RATIONAL METHOD ANALYSIS
C-25	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 2-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE
C-26	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE 2-YEAR RATIONAL METHOD ANALYSIS
C-27	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE 2-YEAR RATIONAL METHOD ANALYSIS
C-28	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 5-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE
C-29	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE 5-YEAR RATIONAL METHOD ANALYSIS
C-30	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE 5-YEAR RATIONAL METHOD ANALYSIS
C-31	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 10-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE
C-32	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE 10-YEAR RATIONAL METHOD ANALYSIS
C-33	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE 10-YEAR RATIONAL METHOD ANALYSIS

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DETAILED ANALYSIS OF PIPE REPLACEMENT LENGTHS AND INVENTORY OF BACKYARD EASEMENTS

TABLE NO.	DESCRIPTION
C-34	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 25-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE
C-35	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE 25-YEAR RATIONAL METHOD ANALYSIS
C-36	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 3: 14TH STREET AND 2ND AVENUE 25-YEAR RATIONAL METHOD ANALYSIS
C-37	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 2-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE
C-38	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE 2-YEAR RATIONAL METHOD ANALYSIS
C-39	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE 2-YEAR RATIONAL METHOD ANALYSIS
C-40	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 5-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE
C-41	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE 5-YEAR RATIONAL METHOD ANALYSIS
C-42	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE 5-YEAR RATIONAL METHOD ANALYSIS
C-43	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 10-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE
C-44	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE 10-YEAR RATIONAL METHOD ANALYSIS

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TABLE NO.	DESCRIPTION
C-45	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE 10-YEAR RATIONAL METHOD ANALYSIS
C-46	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 25-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE
C-47	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE 25-YEAR RATIONAL METHOD ANALYSIS
C-48	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 4: 25TH STREET AND 4TH AVENUE 25-YEAR RATIONAL METHOD ANALYSIS
C-49	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 2-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY
C-50	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY 2-YEAR RATIONAL METHOD ANALYSIS
C-51	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY 2-YEAR RATIONAL METHOD ANALYSIS
C-52	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 5-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY
C-53	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY 5-YEAR RATIONAL METHOD ANALYSIS
C-54	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY 5-YEAR RATIONAL METHOD ANALYSIS
C-55	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 10-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY

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DETAILED ANALYSIS OF PIPE REPLACEMENT LENGTHS AND INVENTORY OF BACKYARD EASEMENTS

TABLE NO.	DESCRIPTION
C-56	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY 10-YEAR RATIONAL METHOD ANALYSIS
C-57	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY 10-YEAR RATIONAL METHOD ANALYSIS
C-58	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 25-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY
C-59	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY 25-YEAR RATIONAL METHOD ANALYSIS
C-60	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 5: HARKNESS AVENUE AND LARKIN WAY 25-YEAR RATIONAL METHOD ANALYSIS
C-61	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 2-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY
C-62	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY 2-YEAR RATIONAL METHOD ANALYSIS
C-63	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY 2-YEAR RATIONAL METHOD ANALYSIS
C-64	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 5-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY
C-65	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY 5-YEAR RATIONAL METHOD ANALYSIS
C-66	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY 5-YEAR RATIONAL METHOD ANALYSIS

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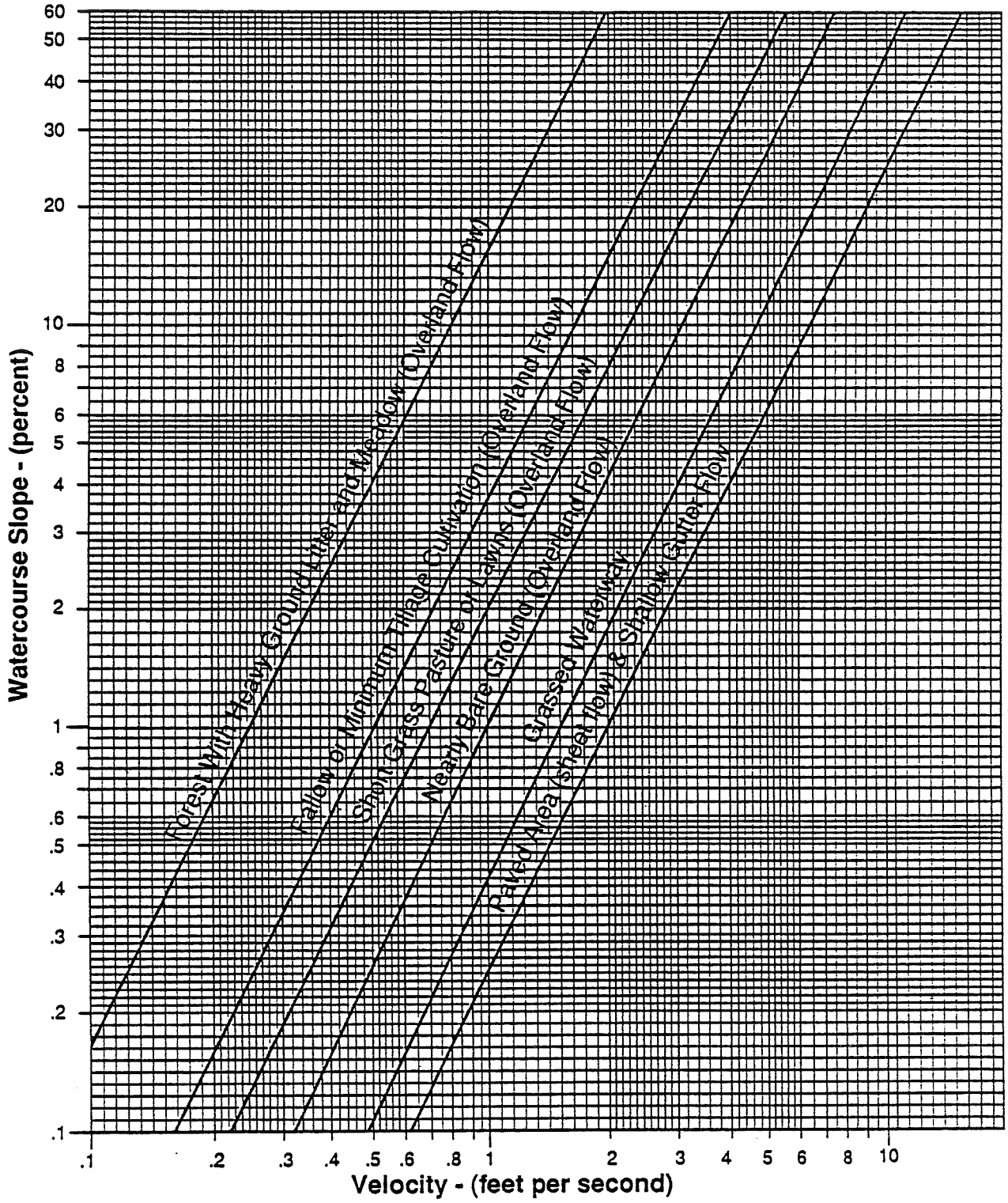
DETAILED ANALYSIS OF PIPE REPLACEMENT LENGTHS AND INVENTORY OF BACKYARD EASEMENTS

TABLE NO.	DESCRIPTION
C-67	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 10-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY
C-68	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY 10-YEAR RATIONAL METHOD ANALYSIS
C-69	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY 10-YEAR RATIONAL METHOD ANALYSIS
C-70	COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS 25-YEAR RATIONAL METHOD ANALYSIS EXISTING 8 INCH TO REPLACEMENT DIAMETER REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY
C-71	SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY 25-YEAR RATIONAL METHOD ANALYSIS
C-72	PERCENTAGE OF REPLACEMENT PIPE LENGTHS REPRESENTATIVE BASINS 6: 56TH STREET AND BROADWAY 25-YEAR RATIONAL METHOD ANALYSIS
C-73	INVENTORY OF BACKYARD EASEMENTS



(Source: Soil Conservation Service, National Engineering Handbook Section 4, Hydrology, 1985)

HDR Engineering, Inc.

Travel Time Velocities for Estimating Time of Concentration

Date: June 1989

Figure: 2-4

TABLE C-1

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
2-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	360	1.5	8	12			360											
D TO C	370	1.7	8	12			370											
C TO B	395	2.9	8	15				395										
B TO F	230	4.4	10	18														
E TO I	205	1.5	8	12			205											
I TO J	375	2.6	8	15				375										
F TO J	175	5.5	10	18														
H TO L	200	1.6	8	10		200												
P TO L	160	1.0	8	10		160												
L TO K	360	3.3	8	15				360										
G TO K	200	1.5	8	10		200												
K TO J	390	5.0	8	18					390									
J TO N	230	11.1	12	24														
T TO S	360	1.1	8	10		360												
O TO S	190	1.4	8	12			190											
S TO R	395	3.0	8	15				395										
M TO Q	190	1.2	8	10		190												
Q TO R	360	1.7	8	12			360											
N TO R	170	12.2	12	24														
TOTAL:	5,315	8 INCH REPLACEMENT			0	1,110	1,485	1,525	390	0	0	0	0	0	0	0	0	4,510 LF

TABLE C-2

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 1: E AND 25TH STREETS
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	1,110	1,485	1,525	390	0	0	0	0	0	0	0	0	4,510
EXISTING	10	0	0	0	0	405	0	0	0	0	0	0	0	0	405
EXISTING	12	0	0	0	0	0	0	400	0	0	0	0	0	0	400
TOTALS:		0	1,110	1,485	1,525	795	0	400	0	0	0	0	0	0	5,315

TABLE C-3

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 1: E AND 25TH STREETS
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	24.6%	32.9%	33.8%	8.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		0.0%	20.9%	27.9%	28.7%	15.0%	0.0%	7.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-4

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
5-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	360	2.3	8	15				360										
D TO C	370	2.6	8	15				370										
C TO B	395	4.3	8	18					395									
B TO F	230	6.9	10	18														
E TO I	205	2.3	8	12			205											
I TO J	375	4.2	8	18					375									
F TO J	175	8.6	10	21														
H TO L	200	2.5	8	12			200											
P TO L	160	1.6	8	12			160											
L TO K	360	5.0	8	18					360									
G TO K	200	2.3	8	12			200											
K TO J	390	7.8	8	21						390								
J TO N	230	17.3	12	30														
T TO S	360	1.7	8	12			360											
O TO S	190	2.2	8	15				190										
S TO R	395	4.7	8	18					395									
M TO Q	190	1.8	8	12			190											
Q TO R	360	2.7	8	15				360										
N TO R	170	18.8	12	30														
TOTAL:	5,315	8 INCH REPLACEMENT			0	0	1,315	1,280	1,525	390	0	0	0	0	0	0	0	4,510 LF

TABLE C-5

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 1: E AND 25TH STREETS
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	0	1,315	1,280	1,525	390	0	0	0	0	0	0	0	4,510
EXISTING	10	0	0	0	0	230	175	0	0	0	0	0	0	0	405
EXISTING	12	0	0	0	0	0	0	0	0	400	0	0	0	0	400
TOTALS:		0	0	1,315	1,280	1,755	565	0	0	400	0	0	0	0	5,315

TABLE C-6

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 1: E AND 25TH STREETS
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	0.0%	29.2%	28.4%	33.8%	8.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	56.8%	43.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		0.0%	0.0%	24.7%	24.1%	33.0%	10.6%	0.0%	0.0%	7.5%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-7

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
10-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	360	2.8	8	15				360										
D TO C	370	3.3	8	15				370										
C TO B	395	5.3	8	18					395									
B TO F	230	8.5	10	21														
E TO I	205	2.9	8	15				205										
I TO J	375	5.0	8	18					375									
F TO J	175	10.7	10	24														
H TO L	200	3.1	8	15				200										
P TO L	160	2.0	8	12			160											
L TO K	360	6.4	8	21						360								
G TO K	200	2.8	8	15				200										
K TO J	390	9.9	8	24							390							
J TO N	230	21.6	12	33														
T TO S	360	2.1	8	15				360										
O TO S	190	2.8	8	15				190										
S TO R	395	5.9	8	21						395								
M TO Q	190	2.2	8	15				190										
Q TO R	360	3.4	8	15				360										
N TO R	170	23.1	12	33														
TOTAL:	5,315	8 INCH REPLACEMENT			0	0	160	2,435	770	755	390	0	0	0	0	0	0	4,510 LF

TABLE C-8

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 1: E AND 25TH STREETS
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	0	160	2,435	770	755	390	0	0	0	0	0	0	4,510
EXISTING	10	0	0	0	0	0	230	175	0	0	0	0	0	0	405
EXISTING	12	0	0	0	0	0	0	0	0	0	400	0	0	0	400
TOTALS:		0	0	160	2,435	770	985	565	0	0	400	0	0	0	5,315

TABLE C-9

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 1: E AND 25TH STREETS
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	0.0%	3.5%	54.0%	17.1%	16.7%	8.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	0.0%	56.8%	43.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		0.0%	0.0%	3.0%	45.8%	14.5%	18.5%	10.6%	0.0%	0.0%	7.5%	0.0%	0.0%	0.0%	100.0%

TABLE C-10

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
25-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	360	3.6	8	18					360									
D TO C	370	4.2	8	18					370									
C TO B	395	6.9	8	21						395								
B TO F	230	11.1	10	24														
E TO I	205	3.7	8	15				205										
I TO J	375	6.4	8	21						375								
F TO J	175	14.0	10	24														
H TO L	200	4.0	8	15				200										
P TO L	160	2.6	8	15				160										
L TO K	360	8.1	8	21						360								
G TO K	200	3.6	8	15				200										
K TO J	390	12.6	8	27								390						
J TO N	230	28.2	12	36														
T TO S	360	2.7	8	15				360										
O TO S	190	3.6	8	15				190										
S TO R	395	7.5	8	21						395								
M TO Q	190	2.8	8	15				190										
Q TO R	360	4.4	8	18					360									
N TO R	170	30.1	12	36														
TOTAL:	5,315	8 INCH REPLACEMENT			0	0	0	1,505	1,090	1,525	0	390	0	0	0	0	0	4,510 LF

TABLE C-11

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 1: E AND 25TH STREETS
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	0	0	1,505	1,090	1,525	0	390	0	0	0	0	0	4,510
EXISTING	10	0	0	0	0	0	0	405	0	0	0	0	0	0	405
EXISTING	12	0	0	0	0	0	0	0	0	0	0	400	0	0	400
TOTALS:		0	0	0	1,505	1,090	1,525	405	390	0	0	400	0	0	5,315

TABLE C-12

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 1: E AND 25TH STREETS
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	0.0%	0.0%	33.4%	24.2%	33.8%	0.0%	8.6%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	100.0%
TOTALS:		0.0%	0.0%	0.0%	28.3%	20.5%	28.7%	7.6%	7.3%	0.0%	0.0%	7.5%	0.0%	0.0%	100.0%

TABLE C-13

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
2-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	160	0.9	8	10		160												
B TO E	200	1.7	8	12			200											
C TO D	220	1.1	8	10		220												
D TO E	185	1.8	8	12			185											
E TO F	210	3.4	10	15														
F TO G	220	5.6	10	18														
G TO H	210	10.3	10	24														
H TO I	215	11.2	12	24														
I TO J	225	15.3	12	27														
TOTAL:	1,845	8 INCH REPLACEMENT			0	380	385	0	0	0	0	0	0	0	0	0	0	765 LF

TABLE C-14

SUMMARY TABLE OF REPLACEMENT PIPES
REPRESENTATIVE BASIN 2: P AND 38TH STREETS
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	380	385	0	0	0	0	0	0	0	0	0	0	765
EXISTING	10	0	0	0	210	220	0	210	0	0	0	0	0	0	640
EXISTING	12	0	0	0	0	0	0	215	225	0	0	0	0	0	440
TOTALS:		0	380	385	210	220	0	425	225	0	0	0	0	0	1,845

TABLE C-15

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 2: P AND 38TH STREETS
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	49.7%	50.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	32.8%	34.4%	0.0%	32.8%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	48.9%	51.1%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		0.0%	20.6%	20.9%	11.4%	11.9%	0.0%	23.0%	12.2%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-16

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
5-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	160	1.4	8	10		160												
B TO E	200	2.5	8	15				200										
C TO D	220	1.7	8	12			220											
D TO E	185	2.8	8	15				185										
E TO F	210	5.5	10	18														
F TO G	220	8.8	10	21														
G TO H	210	16.5	10	27														
H TO I	215	17.8	12	27														
I TO J	225	23.7	12	30														
TOTAL:	1,845	8 INCH REPLACEMENT			0	160	220	385	0	0	0	0	0	0	0	0	0	765 LF

TABLE C-17

SUMMARY TABLE OF REPLACEMENT PIPES
REPRESENTATIVE BASIN 2: P AND 38TH STREETS
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	160	220	385	0	0	0	0	0	0	0	0	0	765
EXISTING	10	0	0	0	0	210	220	0	210	0	0	0	0	0	640
EXISTING	12	0	0	0	0	0	0	0	215	225	0	0	0	0	440
TOTALS:		0	160	220	385	210	220	0	425	225	0	0	0	0	1,845

TABLE C-18

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 2: P AND 38TH STREETS
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	20.9%	28.8%	50.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	32.8%	34.4%	0.0%	32.8%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	48.9%	51.1%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		0.0%	8.7%	11.9%	20.9%	11.4%	11.9%	0.0%	23.0%	12.2%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-19

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
10-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	160	1.7	8	12			160											
B TO E	200	3.1	8	15				200										
C TO D	220	2.1	8	12			220											
D TO E	185	3.4	8	15				185										
E TO F	210	6.8	10	21														
F TO G	220	10.8	10	24														
G TO H	210	21.7	10	30														
H TO I	215	23.2	12	30														
I TO J	225	31.4	12	36														
TOTAL:	1,845	8 INCH REPLACEMENT			0	0	380	385	0	0	0	0	0	0	0	0	0	765 LF

TABLE C-20

SUMMARY TABLE OF REPLACEMENT PIPES
REPRESENTATIVE BASIN 2: P AND 38TH STREETS
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	0	380	385	0	0	0	0	0	0	0	0	0	765
EXISTING	10	0	0	0	0	0	210	220	0	210	0	0	0	0	640
EXISTING	12	0	0	0	0	0	0	0	0	215	0	225	0	0	440
TOTALS:		0	0	380	385	0	210	220	0	425	0	225	0	0	1,845

TABLE C-21

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 2: P AND 38TH STREETS
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	0.0%	49.7%	50.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	0.0%	32.8%	34.4%	0.0%	32.8%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	48.9%	0.0%	51.1%	0.0%	0.0%	100.0%
TOTALS:		0.0%	0.0%	20.6%	20.9%	0.0%	11.4%	11.9%	0.0%	23.0%	0.0%	12.2%	0.0%	0.0%	100.0%

TABLE C-22

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
25-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	160	2.2	8	12			160											
B TO E	200	4.0	8	15				200										
C TO D	220	2.7	8	15				220										
D TO E	185	4.4	8	15				185										
E TO F	210	8.7	10	21														
F TO G	220	14.0	10	27														
G TO H	210	26.2	10	33														
H TO I	215	28.4	12	33														
I TO J	225	37.9	12	36														
TOTAL:	1,845	8 INCH REPLACEMENT			0	0	160	605	0	0	0	0	0	0	0	0	0	765 LF

TABLE C-23

SUMMARY TABLE OF REPLACEMENT PIPES
REPRESENTATIVE BASIN 2: P AND 38TH STREETS
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	0	160	605	0	0	0	0	0	0	0	0	0	765
EXISTING	10	0	0	0	0	0	210	0	220	0	210	0	0	0	640
EXISTING	12	0	0	0	0	0	0	0	0	0	215	225	0	0	440
TOTALS:		0	0	160	605	0	210	0	220	0	425	225	0	0	1,845

TABLE C-24

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 2: P AND 38TH STREETS
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	0.0%	20.9%	79.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	0.0%	32.8%	0.0%	34.4%	0.0%	32.8%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	48.9%	51.1%	0.0%	0.0%	100.0%
TOTALS:		0.0%	0.0%	8.7%	32.8%	0.0%	11.4%	0.0%	11.9%	0.0%	23.0%	12.2%	0.0%	0.0%	100.0%

TABLE C-25

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
2-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	375	1.1	8	10		375												
B TO R	225	2.7	8	15				225										
Q TO R	120	4.3	8	15				120										
R TO N	810	7.0	8	18					810									
K TO E	510	0.6	8	8	510													
D TO E	150	1.8	8	10		150												
E TO F	265	2.3	10	12														
L TO F	470	1.2	8	10		470												
F TO G	130	2.6	12	15														
C TO H	640	0.8	8	8	640													
G TO H	155	6.1	12	21														
H TO M	140	6.6	12	21														
I TO J	200	2.2	8/10	12														
J TO M	340	5.2	10/12	18														
M TO N	190	10.6	14	30														
S TO P	775	0.6	8	8	775													
N TO P	350	19.9	16	33														
TOTAL:	5,845	8 INCH REPLACEMENT			1,925	995	0	345	810	0	0	0	0	0	0	0	0	4,075 LF

NOTE: I TO J: 90′ @ 8"; 110′ @ 10"
J TO M: 150′ @ 10"; 190′ @ 12"

TABLE C-26

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	1,925	995	90	345	810	0	0	0	0	0	0	0	0	4,165
EXISTING	10	0	0	375	0	150	0	0	0	0	0	0	0	0	525
EXISTING	12	0	0	0	130	190	295	0	0	0	0	0	0	0	615
EXISTING	14	0	0	0	0	0	0	0	0	190	0	0	0	0	190
EXISTING	16	0	0	0	0	0	0	0	0	0	350	0	0	0	350
TOTALS:		1,925	995	465	475	1,150	295	0	0	190	350	0	0	0	5,845

TABLE C-27

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	46.2%	23.9%	2.2%	8.3%	19.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	71.4%	0.0%	28.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	21.1%	30.9%	48.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		32.9%	17.0%	8.0%	8.1%	19.7%	5.0%	0.0%	0.0%	3.3%	6.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-28

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
5-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	375	1.8	8	12			375											
B TO R	225	4.3	8	18					225									
Q TO R	120	6.7	8	18					120									
R TO N	810	10.6	8	21						810								
K TO E	510	1.0	8	10		510												
D TO E	150	2.7	8	12			150											
E TO F	265	3.1	10	15														
L TO F	470	1.8	8	12			470											
F TO G	130	4.1	12	18														
C TO H	640	1.2	8	10		640												
G TO H	155	9.5	12	24														
H TO M	140	10.3	12	24														
I TO J	200	3.3	8/10	15														
J TO M	340	7.8	10/12	21														
M TO N	190	16.4	14	36														
S TO P	775	1.0	8	10		775												
N TO P	350	30.6	16	42														
TOTAL:	5,845	8 INCH REPLACEMENT			0	1,925	995	0	345	810	0	0	0	0	0	0	0	4,075 LF

NOTE: I TO J: 90' @ 8"; 110' @ 10"
J TO M: 150' @ 10"; 190' @ 12"

TABLE C-29

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	1,925	995	90	345	810	0	0	0	0	0	0	0	4,165
EXISTING	10	0	0	0	375	0	150	0	0	0	0	0	0	0	525
EXISTING	12	0	0	0	0	130	190	295	0	0	0	0	0	0	615
EXISTING	14	0	0	0	0	0	0	0	0	0	0	190	0	0	190
EXISTING	16	0	0	0	0	0	0	0	0	0	0	0	350	0	350
TOTALS:		0	1,925	995	465	475	1,150	295	0	0	0	190	350	0	5,845

TABLE C-30

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	46.2%	23.9%	2.2%	8.3%	19.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	71.4%	0.0%	28.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	21.1%	30.9%	48.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	100.0%
TOTALS:		0.0%	32.9%	17.0%	8.0%	8.1%	19.7%	5.0%	0.0%	0.0%	0.0%	3.3%	6.0%	0.0%	100.0%

TABLE C-31

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
10-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	375	2.2	8	12			375											
B TO R	225	5.3	8	18					225									
Q TO R	120	8.4	8	18					120									
R TO N	810	13.1	8	24							810							
K TO E	510	1.3	8	10		510												
D TO E	150	3.3	8	12			150											
E TO F	265	3.9	10	15														
L TO F	470	2.3	8	12			470											
F TO G	130	5.1	12	21														
C TO H	640	1.5	8	10		640												
G TO H	155	11.8	12	24														
H TO M	140	12.7	12	24														
I TO J	200	4.1	8/10	18														
J TO M	340	9.8	10/12	21														
M TO N	190	20.2	14	42														
S TO P	775	1.3	8	10		775												
N TO P	350	38.2	16	42														
TOTAL:	5,845	8 INCH REPLACEMENT			0	1,925	995	0	345	0	810	0	0	0	0	0	0	4,075 LF

NOTE: I TO J: 90′ @ 8"; 110′ @ 10"
J TO M: 150′ @ 10"; 190′ @ 12"

TABLE C-32

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	1,925	995	0	435	0	810	0	0	0	0	0	0	4,165
EXISTING	10	0	0	0	265	110	150	0	0	0	0	0	0	0	525
EXISTING	12	0	0	0	0	0	320	295	0	0	0	0	0	0	615
EXISTING	14	0	0	0	0	0	0	0	0	0	0	0	190	0	190
EXISTING	16	0	0	0	0	0	0	0	0	0	0	0	350	0	350
TOTALS:		0	1,925	995	265	545	470	1,105	0	0	0	0	540	0	5,845

TABLE C-33

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	46.2%	23.9%	0.0%	10.4%	0.0%	19.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	50.5%	21.0%	28.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	52.0%	48.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	100.0%
TOTALS:		0.0%	32.9%	17.0%	4.5%	9.3%	8.0%	18.9%	0.0%	0.0%	0.0%	0.0%	9.2%	0.0%	100.0%

TABLE C-34

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
25-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	375	2.9	8	15				375										
B TO R	225	6.9	8	21						225								
Q TO R	120	10.7	8	18					120									
R TO N	810	16.8	8	27								810						
K TO E	510	1.7	8	12			510											
D TO E	150	4.2	8	12			150											
E TO F	265	5.0	10	18														
L TO F	470	3.0	8	15				470										
F TO G	130	6.6	12	21														
C TO H	640	1.8	8	12			640											
G TO H	155	15.2	12	27														
H TO M	140	16.3	12	27														
I TO J	200	5.3	8/10	18														
J TO M	340	12.5	10/12	24														
M TO N	190	26.2	14	42														
S TO P	775	1.7	8	12			775											
N TO P	350	49.4	16	48														
TOTAL:	5,845	8 INCH REPLACEMENT			0	0	2,075	845	120	225	0	810	0	0	0	0	0	4,075 LF

NOTE: I TO J: 90′ @ 8"; 110′ @ 10"
J TO M: 150′ @ 10"; 190′ @ 12"

TABLE C-35

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	0	2,075	845	210	225	0	810	0	0	0	0	0	4,165
EXISTING	10	0	0	0	0	375	0	150	0	0	0	0	0	0	525
EXISTING	12	0	0	0	0	0	130	190	295	0	0	0	0	0	615
EXISTING	14	0	0	0	0	0	0	0	0	0	0	0	190	0	190
EXISTING	16	0	0	0	0	0	0	0	0	0	0	0	0	350	350
TOTALS:		0	0	2,075	845	585	355	340	1,105	0	0	0	190	350	5,845

TABLE C-36

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	0.0%	49.8%	20.3%	5.0%	5.4%	0.0%	19.4%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	71.4%	0.0%	28.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	21.1%	30.9%	48.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	100.0%
TOTALS:		0.0%	0.0%	35.5%	14.5%	10.0%	6.1%	5.8%	18.9%	0.0%	0.0%	0.0%	3.3%	6.0%	100.0%

TABLE C-37

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
2-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO C	130	2.1	8	12			130											
B TO C	130	3.4	8	15				130										
C TO D	400	5.0	10	18														
I TO F	635	1.3	8	10		635												
F TO G	80	1.4	8	10		80												
E TO G	95	2.5	8	12			95											
G TO H	375	3.6	10	15														
J TO H	105	1.4	8	10		105												
H TO K	190	4.5	10	15														
M TO D	130	2.6	10	15														
D TO L	125	6.2	12	18														
L TO K	180	7.0	12/14	18														
K TO O	265	11.8	14/16	24														
Q TO R	85	0.3	8	8	85													
P TO R	125	2.5	8	15				125										
R TO S	735	2.6	8	15				735										
S TO O	260	4.1	8/10	15														
TOTAL:	4,045	8 INCH REPLACEMENT			85	820	225	990	0	0	0	0	0	0	0	0	0	2,120 LF

NOTE: L TO K: 135′ @ 12"; 45′ @ 14′
K TO O: 155′ @ 14"; 110′ @ 16"
S TO O: 125′ @ 8"; 135′ @ 10"

TABLE C-38

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	85	820	225	1,115	0	0	0	0	0	0	0	0	0	2,245
EXISTING	10	0	0	0	830	400	0	0	0	0	0	0	0	0	1,230
EXISTING	12	0	0	0	0	260	0	0	0	0	0	0	0	0	260
EXISTING	14	0	0	0	0	45	0	155	0	0	0	0	0	0	200
EXISTING	16	0	0	0	0	0	0	110	0	0	0	0	0	0	110
TOTALS:		85	820	225	1,945	705	0	265	0	0	0	0	0	0	4,045

TABLE C-39

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	3.8%	36.5%	10.0%	49.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	67.5%	32.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	22.5%	0.0%	77.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		2.1%	20.3%	5.6%	48.1%	17.4%	0.0%	6.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-40

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
5-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO C	130	3.1	8	15				130										
B TO C	130	5.3	8	18					130									
C TO D	400	7.8	10	21														
I TO F	635	2.0	8	12			635											
F TO G	80	2.2	8	12			80											
E TO G	95	3.8	8	15				95										
G TO H	375	5.6	10	18														
J TO H	105	2.2	8	12			105											
H TO K	190	6.9	10	15														
M TO D	130	3.9	10	15														
D TO L	125	9.5	12	21														
L TO K	180	10.6	12/14	21														
K TO O	265	17.9	14/16	27														
Q TO R	85	0.5	8	8	85													
P TO R	125	3.8	8	18					125									
R TO S	735	4.0	8	18					735									
S TO O	260	6.6	8/10	18														
TOTAL:	4,045	8 INCH REPLACEMENT			85	0	820	225	990	0	0	0	0	0	0	0	0	2,120 LF

NOTE: L TO K: 135′ @ 12"; 45′ @ 14′
K TO O: 155′ @ 14"; 110′ @ 16"
S TO O: 125′ @ 8"; 135′ @ 10"

TABLE C-41

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	85	0	820	225	1,115	0	0	0	0	0	0	0	0	2,245
EXISTING	10	0	0	0	320	510	400	0	0	0	0	0	0	0	1,230
EXISTING	12	0	0	0	0	0	260	0	0	0	0	0	0	0	260
EXISTING	14	0	0	0	0	0	45	0	155	0	0	0	0	0	200
EXISTING	16	0	0	0	0	0	0	0	110	0	0	0	0	0	110
TOTALS:		85	0	820	545	1,625	705	0	265	0	0	0	0	0	4,045

TABLE C-42

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	3.8%	0.0%	36.5%	10.0%	49.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	26.0%	41.5%	32.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	22.5%	0.0%	77.5%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		2.1%	0.0%	20.3%	13.5%	40.2%	17.4%	0.0%	6.6%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-43

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
10-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO C	130	3.9	8	15				130										
B TO C	130	6.5	8	18					130									
C TO D	400	9.5	10	21														
I TO F	635	2.4	8	12			635											
F TO G	80	2.7	8	12			80											
E TO G	95	4.7	8	15				95										
G TO H	375	7.0	10	18														
J TO H	105	2.7	8	15				105										
H TO K	190	8.6	10	21														
M TO D	130	4.9	10	18														
D TO L	125	11.7	12	24														
L TO K	180	13.1	12/14	24														
K TO O	265	22.1	14/16	30														
Q TO R	85	0.6	8	8	85													
P TO R	125	4.7	8	18					125									
R TO S	735	5.0	8	18					735									
S TO O	260	8.2	8/10	21														
TOTAL:	4,045	8 INCH REPLACEMENT			85	0	715	330	990	0	0	0	0	0	0	0	0	2,120 LF

NOTE: L TO K: 135′ @ 12"; 45′ @ 14′
K TO O: 155′ @ 14"; 110′ @ 16"
S TO O: 125′ @ 8"; 135′ @ 10"

TABLE C-44

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	85	0	715	330	990	125	0	0	0	0	0	0	0	2,245
EXISTING	10	0	0	0	0	505	725	0	0	0	0	0	0	0	1,230
EXISTING	12	0	0	0	0	0	0	260	0	0	0	0	0	0	260
EXISTING	14	0	0	0	0	0	0	45	0	155	0	0	0	0	200
EXISTING	16	0	0	0	0	0	0	0	0	110	0	0	0	0	110
TOTALS:		85	0	715	330	1,495	850	305	0	265	0	0	0	0	4,045

TABLE C-45

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	3.8%	0.0%	31.8%	14.7%	44.1%	5.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	41.1%	58.9%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	22.5%	0.0%	77.5%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		2.1%	0.0%	17.7%	8.2%	37.0%	21.0%	7.5%	0.0%	6.6%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-46

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
25-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO C	130	4.9	8	18					130									
B TO C	130	8.3	8	21						130								
C TO D	400	12.2	10	24														
I TO F	635	3.1	8	15				635										
F TO G	80	3.5	8	15				80										
E TO G	95	6.0	8	18					95									
G TO H	375	8.9	10	21														
J TO H	105	3.4	8	15				105										
H TO K	190	11.1	10	21														
M TO D	130	6.3	10	18														
D TO L	125	15.1	12	27														
L TO K	180	16.8	12/14	27														
K TO O	265	28.4	14/16	30														
Q TO R	85	0.7	8	8	85													
P TO R	125	6.1	8	21						125								
R TO S	735	6.4	8	21						735								
S TO O	260	10.6	8/10	21														
TOTAL:	4,045	8 INCH REPLACEMENT			85	0	0	820	225	990	0	0	0	0	0	0	0	2,120 LF

NOTE: L TO K: 135' @ 12"; 45' @ 14'
K TO O: 155' @ 14"; 110' @ 16"
S TO O: 125' @ 8"; 135' @ 10"

TABLE C-47

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	85	0	0	820	225	1,115	0	0	0	0	0	0	0	2,245
EXISTING	10	0	0	0	0	130	700	400	0	0	0	0	0	0	1,230
EXISTING	12	0	0	0	0	0	0	0	260	0	0	0	0	0	260
EXISTING	14	0	0	0	0	0	0	0	45	155	0	0	0	0	200
EXISTING	16	0	0	0	0	0	0	0	0	110	0	0	0	0	110
TOTALS:		85	0	0	820	355	1,815	400	305	265	0	0	0	0	4,045

TABLE C-48

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	3.8%	0.0%	0.0%	36.5%	10.0%	49.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	10.6%	56.9%	32.5%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	22.5%	77.5%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		2.1%	0.0%	0.0%	20.3%	8.8%	44.9%	9.9%	7.5%	6.6%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
2-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN STREET

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	170	0.8	8	10		170												
B TO C	220	1.8	8	12			220											
C TO D	195	2.8	8	15				195										
D TO E	140	3.3	8	15				140										
E TO F	160	3.5	8	15				160										
F TO H	265	3.8	8/10	18					120									
G TO H	120	1.2	8	10		120												
H TO I	400	5.2	10	18														
J TO L	120	1.5	8	10		120												
K TO L	70	0.8	8	8	70													
L TO M	385	2.1	8	12			385											
N TO M	105	0.4	8	8	105													
M TO P	110	2.2	8	12			110											
O TO P	185	0.9	8	8	185													
P TO R	210	3.3	8/10	15				115										
Q TO R	190	0.8	8	8	190													
R TO I	120	3.8	12	18														
I TO T	110	8.0	12	21														
S TO T	180	0.8	8	8	180													
T TO V	115	8.2	12	24														
U TO V	410	0.9	8	10		410												
V TO X	100	8.5	12	24														
W TO X	190	1.2	8	10		190												
Y TO AA	115	1.1	8	10		115												
Z TO AA	115	1.5	8	18					115									
AA TO BB	555	2.2	8	15				555										
CC TO BB	365	1.8	8	12			365											
BB TO EE	115	3.3	10	15														
DD TO EE	350	1.5	8	12			350											
EE TO FF	240	5.9	10/12	21														
GG TO FF	175	2.1	8	12			175											
FF TO II	120	6.8	12	21														
HH TO II	550	1.5	10	12														
II TO X	115	7.6	12	27														
X TO KK	120	17.0	16	33														
JJ TO KK	590	1.7	8	12			590											
KK TO LL	125	18.0	18	33														
LL TO MM	115	19.4	18	33														
TOTAL:	8,035	8 INCH REPLACEMENT			730	1,125	2,195	1,165	235	0	0	0	0	0	0	0	0	5,450 LF

NOTE: F TO H 120′ @ 8"; 145′ @ 10", P TO R 115′ @ 8"; 95′ @ 10", EE TO FF 110′ @ 10"; 130′ @ 12"

TABLE C-50

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 5: HARKNESS ANVENUE AND LARKIN STREET
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	730	1,125	2,195	1,165	235	0	0	0	0	0	0	0	0	5,450
EXISTING	10	0	0	550	210	545	110	0	0	0	0	0	0	0	1,415
EXISTING	12	0	0	0	0	120	360	215	115	0	0	0	0	0	810
EXISTING	16	0	0	0	0	0	0	0	0	0	120	0	0	0	120
EXISTING	18	0	0	0	0	0	0	0	0	0	240	0	0	0	240
TOTALS:		730	1,125	2,745	1,375	900	470	215	115	0	360	0	0	0	8,035

TABLE C-51

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 5: HARKNESS ANVENUE AND LARKIN STREET
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	13.4%	20.6%	40.3%	21.4%	4.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	38.9%	14.8%	38.5%	7.8%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	14.8%	44.4%	26.5%	14.2%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	18	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		9.1%	14.0%	34.2%	17.1%	11.2%	5.8%	2.7%	1.4%	0.0%	4.5%	0.0%	0.0%	0.0%	100.0%

TABLE C-

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
5-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN STREET

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	170	1.2	8	10		170												
B TO C	220	2.7	8	15				220										
C TO D	195	4.4	8	18					195									
D TO E	140	5.1	8	18					140									
E TO F	160	5.4	8	18					160									
F TO H	265	5.9	8/10	18					120									
G TO H	120	1.9	8	12			120											
H TO I	400	8.1	10	21														
J TO L	120	2.2	8	12			120											
K TO L	70	1.2	8	8	70													
L TO M	385	3.3	8	15				385										
N TO M	105	0.7	8	8	105													
M TO P	110	3.5	8	15				110										
O TO P	185	1.6	8	10		185												
P TO R	210	5.1	8/10	18					115									
Q TO R	190	1.3	8	10		190												
R TO I	120	6.1	12	21														
I TO T	110	12.5	12	24														
S TO T	180	1.4	8	10		180												
T TO V	115	12.9	12	30														
U TO V	410	1.4	8	10		410												
V TO X	100	13.4	12	30														
W TO X	190	1.9	8	10		190												
Y TO AA	115	1.8	8	12			115											
Z TO AA	115	2.3	8	21						115								
AA TO BB	555	3.4	8	15				555										
CC TO BB	365	2.7	8	15				365										
BB TO EE	115	5.1	10	18														
DD TO EE	350	2.3	8	15				350										
EE TO FF	240	9.1	10/12	24														
GG TO FF	175	3.4	8	15				175										
FF TO II	120	10.4	12	24														
HH TO II	550	2.3	10	12														
II TO X	115	11.9	12	33														
X TO KK	120	26.3	16	42														
JJ TO KK	590	2.7	8	12			590											
KK TO LL	125	27.9	18	42														
LL TO MM	115	30.1	18	42														
TOTAL:	8,035	8 INCH REPLACEMENT			175	1,325	945	2,160	730	115	0	0	0	0	0	0	0	5,450 LF

NOTE: F TO H 120′ @ 8"; 145′ @ 10", P TO R 115′ @ 8"; 95′ @ 10", EE TO FF 110′ @ 10"; 130′ @ 12"

TABLE C-53

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 5: HARKNESS ANVENUE AND LARKIN STREET
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	175	1,325	945	2,160	730	115	0	0	0	0	0	0	0	5,450
EXISTING	10	0	0	550	0	355	400	110	0	0	0	0	0	0	1,415
EXISTING	12	0	0	0	0	0	120	360	0	215	115	0	0	0	810
EXISTING	16	0	0	0	0	0	0	0	0	0	0	0	120	0	120
EXISTING	18	0	0	0	0	0	0	0	0	0	0	0	240	0	240
TOTALS:		175	1,325	1,495	2,160	1,085	635	470	0	215	115	0	360	0	8,035

TABLE C-54

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 5: HARKNESS ANVENUE AND LARKIN STREET
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	3.2%	24.3%	17.3%	39.6%	13.4%	2.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	38.9%	0.0%	25.1%	28.3%	7.8%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	14.8%	44.4%	0.0%	26.5%	14.2%	0.0%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	100.0%
EXISTING	18	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	100.0%
TOTALS:		2.2%	16.5%	18.6%	26.9%	13.5%	7.9%	5.8%	0.0%	2.7%	1.4%	0.0%	4.5%	0.0%	100.0%

TABLE C-55

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
10-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN STREET

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	170	1.5	8	12			170											
B TO C	220	3.4	8	15				220										
C TO D	195	5.5	8	18					195									
D TO E	140	6.4	8	21						140								
E TO F	160	6.8	8	21						160								
F TO H	265	7.5	8/10	21						120								
G TO H	120	2.3	8	12			120											
H TO I	400	10.1	10	24														
J TO L	120	2.8	8	15				120										
K TO L	70	1.5	8	8	70													
L TO M	385	4.2	8	15				385										
N TO M	105	0.8	8	8	105													
M TO P	110	4.4	8	15				110										
O TO P	185	2.0	8	10		185												
P TO R	210	6.4	8/10	21						115								
Q TO R	190	1.7	8	10		190												
R TO I	120	7.7	12	24														
I TO T	110	15.9	12	27														
S TO T	180	1.8	8	12			180											
T TO V	115	16.5	12	33														
U TO V	410	1.8	8	12			410											
V TO X	100	17.1	12	33														
W TO X	190	2.5	8	12			190											
Y TO AA	115	2.3	8	12			115											
Z TO AA	115	3.0	8	24							115							
AA TO BB	555	4.2	8	18					555									
CC TO BB	365	3.4	8	15				365										
BB TO EE	115	6.5	10	21														
DD TO EE	350	2.8	8	15				350										
EE TO FF	240	11.5	10/12	27														
GG TO FF	175	4.5	8	15				175										
FF TO II	120	13.1	12	27														
HH TO II	550	2.8	10	15														
II TO X	115	14.9	12	36														
X TO KK	120	33.4	16	42														
JJ TO KK	590	3.4	8	15				590										
KK TO LL	125	35.1	18	42														
LL TO MM	115	37.6	18	42														
TOTAL:	8,035	8 INCH REPLACEMENT			175	375	1,185	2,315	750	535	115	0	0	0	0	0	0	5,450 LF

NOTE: F TO H 120′ @ 8"; 145′ @ 10", P TO R 115′ @ 8"; 95′ @ 10", EE TO FF 110′ @ 10"; 130′ @ 12"

TABLE C-56

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 5: HARKNESS ANVENUE AND LARKIN STREET
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	175	375	1,185	2,315	750	535	115	0	0	0	0	0	0	5,450
EXISTING	10	0	0	0	550	0	355	400	110	0	0	0	0	0	1,415
EXISTING	12	0	0	0	0	0	0	120	360	0	215	115	0	0	810
EXISTING	16	0	0	0	0	0	0	0	0	0	0	0	120	0	120
EXISTING	18	0	0	0	0	0	0	0	0	0	0	0	240	0	240
TOTALS:		175	375	1,185	2,865	750	890	635	470	0	215	115	360	0	8,035

TABLE C-57

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 5: HARKNESS ANVENUE AND LARKIN STREET
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	3.2%	6.9%	21.7%	42.5%	13.8%	9.8%	2.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	38.9%	0.0%	25.1%	28.3%	7.8%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	14.8%	44.4%	0.0%	26.5%	14.2%	0.0%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	100.0%
EXISTING	18	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	100.0%
TOTALS:		2.2%	4.7%	14.7%	35.7%	9.3%	11.1%	7.9%	5.8%	0.0%	2.7%	1.4%	4.5%	0.0%	100.0%

TABLE C-58

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
25-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN STREET

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO B	170	1.9	8	12			170											
B TO C	220	4.4	8	18					220									
C TO D	195	7.1	8	21						195								
D TO E	140	8.3	8	21						140								
E TO F	160	8.9	8	21						160								
F TO H	265	9.7	8/10	24							120							
G TO H	120	3.0	8	15				120										
H TO I	400	13.2	10	27														
J TO L	120	3.6	8	15				120										
K TO L	70	1.9	8	10		70												
L TO M	385	5.3	8	18					385									
N TO M	105	1.1	8	10		105												
M TO P	110	5.6	8	18					110									
O TO P	185	2.7	8	12			185											
P TO R	210	8.4	8/10	21						115								
Q TO R	190	2.2	8	12			190											
R TO I	120	10.1	12	24														
I TO T	110	20.6	12	30														
S TO T	180	2.4	8	12			180											
T TO V	115	21.4	12	36														
U TO V	410	2.3	8	12			410											
V TO X	100	22.2	12	36														
W TO X	190	3.3	8	15				190										
Y TO AA	115	2.9	8	15				115										
Z TO AA	115	3.8	8	24							115							
AA TO BB	555	5.4	8	18					555									
CC TO BB	365	4.3	8	15				365										
BB TO EE	115	8.3	10	21														
DD TO EE	350	3.6	8	15				350										
EE TO FF	240	14.7	10/12	30														
GG TO FF	175	5.9	8	18					175									
FF TO II	120	16.8	12	30														
HH TO II	550	3.6	10	15														
II TO X	115	19.1	12	42														
X TO KK	120	43.3	16	48														
JJ TO KK	590	4.3	8	15				590										
KK TO LL	125	45.9	18	48														
LL TO MM	115	48.8	18	48														
TOTAL:	8,035	8 INCH REPLACEMENT			0	175	1,135	1,850	1,445	610	235	0	0	0	0	0	0	5,450 LF

NOTE: F TO H 120′ @ 8"; 145′ @ 10", P TO R 115′ @ 8"; 95′ @ 10", EE TO FF 110′ @ 10"; 130′ @ 12"

TABLE C-59

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 5: HARKNESS ANVENUE AND LARKIN STREET
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	175	1,135	1,850	1,445	610	235	0	0	0	0	0	0	5,450
EXISTING	10	0	0	0	550	0	210	145	400	110	0	0	0	0	1,415
EXISTING	12	0	0	0	0	0	0	120	0	360	0	215	115	0	810
EXISTING	16	0	0	0	0	0	0	0	0	0	0	0	0	120	120
EXISTING	18	0	0	0	0	0	0	0	0	0	0	0	0	240	240
TOTALS:		0	175	1,135	2,400	1,445	820	500	400	470	0	215	115	360	8,035

TABLE C-60

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 5: HARKNESS ANVENUE AND LARKIN STREET
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	3.2%	20.8%	33.9%	26.5%	11.2%	4.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	38.9%	0.0%	14.8%	10.2%	28.3%	7.8%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	14.8%	0.0%	44.4%	0.0%	26.5%	14.2%	0.0%	100.0%
EXISTING	16	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	100.0%
EXISTING	18	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	100.0%
TOTALS:		0.0%	2.2%	14.1%	29.9%	18.0%	10.2%	6.2%	5.0%	5.8%	0.0%	2.7%	1.4%	4.5%	100.0%

TABLE C-61

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
2-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO C	210	0.7	8	8	210													
B TO C	215	1.3	8	10		215												
C TO D	265	4.3	10	18														
D TO H	475	4.6	12	18														
E TO F	370	0.8	8	10		370												
F TO G	575	2.4	8	12			575											
G TO H	655	1.3	8	10		655												
H TO J	170	6.4	12	21														
L TO K	140	0.9	8	8	140													
K TO J	185	2.0	8	12			185											
I TO J	300	1.3	8	10		300												
J TO N	310	9.9	14	24														
O TO N	250	2.5	8	15				250										
M TO N	315	2.0	8	12			315											
N TO O	410	12.4	14	27														
TOTAL:	4,845	8 INCH REPLACEMENT			350	1,540	1,075	250	0	0	0	0	0	0	0	0	0	3,215 LF

TABLE C-62

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	350	1,540	1,075	250	0	0	0	0	0	0	0	0	0	3,215
EXISTING	10	0	0	0	0	265	0	0	0	0	0	0	0	0	265
EXISTING	12	0	0	0	0	475	170	0	0	0	0	0	0	0	645
EXISTING	14	0	0	0	0	0	0	310	410	0	0	0	0	0	720
TOTALS:		350	1,540	1,075	250	740	170	310	410	0	0	0	0	0	4,845

TABLE C-63

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
2-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	10.9%	47.9%	33.4%	7.8%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	73.6%	26.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	43.1%	56.9%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		7.2%	31.8%	22.2%	5.2%	15.3%	3.5%	6.4%	8.5%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-64

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
5-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO C	210	1.2	8	10		210												
B TO C	215	1.9	8	12			215											
C TO D	265	6.6	10	21														
D TO H	475	7.0	12	21														
E TO F	370	1.2	8	10		370												
F TO G	575	3.7	8	15				575										
G TO H	655	2.0	8	12			655											
H TO J	170	9.9	12	24														
L TO K	140	1.5	8	10		140												
K TO J	185	3.1	8	15				185										
I TO J	300	1.9	8	12			300											
J TO N	310	15.3	14	27														
O TO N	250	3.8	8	15				250										
M TO N	315	3.0	8	15				315										
N TO O	410	18.9	14	30														
TOTAL:	4,845	8 INCH REPLACEMENT			0	720	1,170	1,325	0	0	0	0	0	0	0	0	0	3,215 LF

TABLE C-65

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	720	1,170	1,325	0	0	0	0	0	0	0	0	0	3,215
EXISTING	10	0	0	0	0	0	265	0	0	0	0	0	0	0	265
EXISTING	12	0	0	0	0	0	475	170	0	0	0	0	0	0	645
EXISTING	14	0	0	0	0	0	0	0	310	410	0	0	0	0	720
TOTALS:		0	720	1,170	1,325	0	740	170	310	410	0	0	0	0	4,845

TABLE C-66

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
5-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	22.4%	36.4%	41.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	73.6%	26.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	43.1%	56.9%	0.0%	0.0%	0.0%	0.0%	100.0%
TOTALS:		0.0%	14.9%	24.1%	27.3%	0.0%	15.3%	3.5%	6.4%	8.5%	0.0%	0.0%	0.0%	0.0%	100.0%

TABLE C-67

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
10-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO C	210	1.5	8	10		210												
B TO C	215	2.4	8	12			215											
C TO D	265	8.2	10	21														
D TO H	475	8.7	12	21														
E TO F	370	1.6	8	12			370											
F TO G	575	4.7	8	15				575										
G TO H	655	2.5	8	15				655										
H TO J	170	12.4	12	27														
L TO K	140	1.8	8	12			140											
K TO J	185	3.9	8	15				185										
I TO J	300	2.4	8	15				300										
J TO N	310	19.1	14	30														
O TO N	250	4.8	8	18					250									
M TO N	315	3.7	8	15				315										
N TO O	410	23.7	14	33														
TOTAL:	4,845	8 INCH REPLACEMENT			0	210	725	2,030	250	0	0	0	0	0	0	0	0	3,215 LF

TABLE C-68

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	210	725	2,030	250	0	0	0	0	0	0	0	0	3,215
EXISTING	10	0	0	0	0	0	265	0	0	0	0	0	0	0	265
EXISTING	12	0	0	0	0	0	475	0	170	0	0	0	0	0	645
EXISTING	14	0	0	0	0	0	0	0	0	310	410	0	0	0	720
TOTALS:		0	210	725	2,030	250	740	0	170	310	410	0	0	0	4,845

TABLE C-69

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
10-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	6.5%	22.6%	63.1%	7.8%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	73.6%	0.0%	26.4%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	43.1%	56.9%	0.0%	0.0%	0.0%	100.0%
TOTALS:		0.0%	4.3%	15.0%	41.9%	5.2%	15.3%	0.0%	3.5%	6.4%	8.5%	0.0%	0.0%	0.0%	100.0%

TABLE C-70

COLLECTION SYSTEM IMPROVEMENT REPLACEMENT LENGTHS
25-YEAR RATIONAL METHOD ANALYSIS
EXISTING 8 INCH TO REPLACEMENT DIAMETER

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

PIPE SECTION	PIPE LENGTH (LF)	PEAK FLOW (CFS)	EXIST. PIPE DIA. (IN)	REPLACE PIPE DIA. (IN)	PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
					8	10	12	15	18	21	24	27	30	33	36	42	48	
A TO C	210	1.9	8	12			210											
B TO C	215	3.1	8	15				215										
C TO D	265	10.6	10	24														
D TO H	475	11.4	12	24														
E TO F	370	2.0	8	12			370											
F TO G	575	6.0	8	18					575									
G TO H	655	3.2	8	15				655										
H TO J	170	15.8	12	30														
L TO K	140	2.4	8	12			140											
K TO J	185	5.0	8	18					185									
I TO J	300	3.1	8	15				300										
J TO N	310	24.3	14	33														
O TO N	250	6.1	8	18					250									
M TO N	315	4.8	8	18					315									
N TO O	410	30.4	14	36														
TOTAL:	4,845	8 INCH REPLACEMENT			0	0	720	1,170	1,325	0	0	0	0	0	0	0	0	3,215 LF

TABLE C-71

SUMMARY TABLE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTAL (LF)
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0	0	720	1,170	1,325	0	0	0	0	0	0	0	0	3,215
EXISTING	10	0	0	0	0	0	0	265	0	0	0	0	0	0	265
EXISTING	12	0	0	0	0	0	0	475	0	170	0	0	0	0	645
EXISTING	14	0	0	0	0	0	0	0	0	0	310	410	0	0	720
TOTALS:		0	0	720	1,170	1,325	0	740	0	170	310	410	0	0	4,845

TABLE C-72

PERCENTAGE OF REPLACEMENT PIPES LENGTHS
REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
25-YEAR RATIONAL METHOD ANALYSIS

		PIPE REPLACEMENT DIAMETERS (INCHES)													TOTALS
		8	10	12	15	18	21	24	27	30	33	36	42	48	
EXISTING	8	0.0%	0.0%	22.4%	36.4%	41.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	10	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	12	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	73.6%	0.0%	26.4%	0.0%	0.0%	0.0%	0.0%	100.0%
EXISTING	14	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	43.1%	56.9%	0.0%	0.0%	100.0%
TOTALS:		0.0%	0.0%	14.9%	24.1%	27.3%	0.0%	15.3%	0.0%	3.5%	6.4%	8.5%	0.0%	0.0%	100.0%

TABLE C-73
INVENTORY OF BACKYARD EASEMENT LENGTHS (FEET)

	PIPE DIAMETER		
BASIN	6″	8″	10″
3281	350		
3311	100	200	
3321		250	
3581		375	
3361		800	
3991		200	200
5701		700	
5591		300	
9371	700	2950	
9291		2200	
9552		2550	
9221		1250	
9251	150	550	
9211		350	
9381	200	550	
9391		850	
9191		350	
9151	800	850	
9411		1550	
9451		1700	
9452		950	
9361		550	
9501	100	4250	
7623		1900	
7491	600	800	
7481	100	1550	
7961		100	
7962	500	1200	
7991	350	800	
7951	200	4200	
7871		150	
7941		600	
7912		500	
7291	250	400	
7851		800	
7241	500	1250	
7831	500	1000	
7433		600	
7381	100	350	
7361	300		
7411		2500	450

TABLE C-73
INVENTORY OF BACKYARD EASEMENT LENGTHS (FEET)

	PIPE DIAMETER		
BASIN	6″	8″	10″
7421	300		
7441	300		
9711	200	950	300
1331		200	200
1302	700	550	
1051		250	
1321	200	800	
1301		300	
1452	100	1300	
1241			550
1211		1050	
1421	600	1250	100
1422	200	1200	
1441		3100	
1661	700	1350	
1481		600	
5111		1350	100
1641		1100	
1651		2300	
1591	200	200	
1551	200		
1521			100
1511	300	150	
1531		800	
1561		3250	
1501		850	
1631		700	
5092		550	
5091	50	800	
1601		3200	
5081		150	
5061	50	1400	
5041	400	900	
5031	200	300	
5021	150	2300	300
5011		600	
7051	50	1700	
1821	50	550	
1771		1200	
1781		450	
1791		2000	

TABLE C-73
INVENTORY OF BACKYARD
EASEMENT LENGTHS (FEET)

	PIPE DIAMETER		
BASIN	6″	8″	10″
1831	100	600	
1581		800	
7161	100	400	
7151		3200	
7131		1400	
1871		250	
1911		2300	
1891		500	
7781		850	
7821		500	
7201		450	
7211	450		
7801		500	
7191		200	
1921			150
1951	300	700	150
1931	1100		
1431		700	
TOTALS	12800	96025	2600

APPENDIX D

ANALYSIS OF PER-ACRE IMPROVEMENTS

APPENDIX D

ANALYSIS OF PER-ACRE IMPROVEMENTS

TABLE NO.	DESCRIPTION
D-1	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 1: E AND 25TH STREETS
D-2	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 1: E AND 25TH STREETS
D-3	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 1: E AND 25TH STREETS
D-4	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 1: E AND 25TH STREETS
D-5	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 2: P AND 38TH STREETS
D-6	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 2: P AND 38TH STREETS
D-7	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 2: P AND 38TH STREETS
D-8	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 2: P AND 38TH STREETS
D-9	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE
D-10	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE
D-11	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE
D-12	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

APPENDIX D

ANALYSIS OF PER-ACRE IMPROVEMENTS

TABLE NO.	DESCRIPTION
D-13	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
D-14	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
D-15	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
D-16	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE
D-17	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY
D-18	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY
D-19	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY
D-20	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY
D-21	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 2-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
D-22	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 5-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
D-23	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 10-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY
D-24	COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE 25-YEAR RATIONAL METHOD ANALYSIS REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

TABLE D–1
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
2–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	360	0.0026	1.5	8	12	$94.77	$34,117
D TO C	370	0.0033	1.7	8	12	$94.77	$35,065
C TO B	395	0.0040	2.9	8	15	$111.35	$43,983
B TO F	230	0.0043	4.4	10	18	$129.35	$29,751
E TO I	205	0.0043	1.5	8	12	$94.77	$19,428
I TO J	375	0.0030	2.6	8	15	$111.35	$41,756
F TO J	175	0.0043	5.5	10	18	$129.35	$22,636
H TO L	200	0.0054	1.6	8	10	$79.85	$15,970
P TO L	160	0.0046	1.0	8	10	$79.85	$12,776
L TO K	360	0.0035	3.3	8	15	$111.35	$40,086
G TO K	200	0.0054	1.5	8	10	$79.85	$15,970
K TO J	390	0.0030	5.0	8	18	$129.35	$50,447
J TO N	230	0.0026	11.1	12	24	$209.41	$48,164
T TO S	360	0.0033	1.1	8	10	$79.85	$28,746
O TO S	190	0.0031	1.4	8	12	$94.77	$18,006
S TO R	395	0.0029	3.0	8	15	$111.35	$43,983
M TO Q	190	0.0031	1.2	8	10	$79.85	$15,171
Q TO R	360	0.0029	1.7	8	12	$94.77	$34,117
N TO R	170	0.0031	12.2	12	24	$209.41	$35,600
OUTLET (R)			16.1				
TOTAL PIPE REPLACEMENT COST							$585,773
PIPE REPLACEMENT – $/AC (36.7 TOTAL AC IN BASIN)							$15,961

TABLE D–2
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
5–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

PIPE SECTION	PIPE LENGTH (feet)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/lf)	COST ($)
A TO B	360	0.0026	2.3	8	15	$111.35	$40,086
D TO C	370	0.0033	2.6	8	15	$111.35	$41,200
C TO B	395	0.0040	4.3	8	18	$129.35	$51,093
B TO F	230	0.0043	6.9	10	18	$129.35	$29,751
E TO I	205	0.0043	2.3	8	12	$94.77	$19,428
I TO J	375	0.0030	4.2	8	18	$129.35	$48,506
F TO J	175	0.0043	8.6	10	21	$147.05	$25,734
H TO L	200	0.0054	2.5	8	12	$94.77	$18,954
P TO L	160	0.0046	1.6	8	12	$94.77	$15,163
L TO K	360	0.0035	5.0	8	18	$129.35	$46,566
G TO K	200	0.0054	2.3	8	12	$94.77	$18,954
K TO J	390	0.0030	7.8	8	21	$147.05	$57,350
J TO N	230	0.0026	17.3	12	30	$269.03	$61,877
T TO S	360	0.0033	1.7	8	12	$94.77	$34,117
O TO S	190	0.0031	2.2	8	15	$111.35	$21,157
S TO R	395	0.0029	4.7	8	18	$129.35	$51,093
M TO Q	190	0.0031	1.8	8	12	$94.77	$18,006
Q TO R	360	0.0029	2.7	8	15	$111.35	$40,086
N TO R	170	0.0031	18.8	12	30	$269.03	$45,735
OUTLET (R)			25.0				
TOTAL PIPE REPLACEMENT COST							$684,855
PIPE REPLACEMENT – $/AC (36.7 TOTAL AC IN BASIN)							$18,661

TABLE D–3
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
10–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	360	0.0026	2.8	8	15	$111.35	$40,086
D TO C	370	0.0033	3.3	8	15	$111.35	$41,200
C TO B	395	0.0040	5.3	8	18	$129.35	$51,093
B TO F	230	0.0043	8.5	10	21	$147.05	$33,822
E TO I	205	0.0043	2.9	8	15	$111.35	$22,827
I TO J	375	0.0030	5.0	8	18	$129.35	$48,506
F TO J	175	0.0043	10.7	10	24	$209.41	$36,647
H TO L	200	0.0054	3.1	8	15	$111.35	$22,270
P TO L	160	0.0046	2.0	8	12	$94.77	$15,163
L TO K	360	0.0035	6.4	8	21	$147.05	$52,938
G TO K	200	0.0054	2.8	8	15	$111.35	$22,270
K TO J	390	0.0030	9.9	8	24	$209.41	$81,670
J TO N	230	0.0026	21.6	12	33	$302.12	$69,488
T TO S	360	0.0033	2.1	8	15	$111.35	$40,086
O TO S	190	0.0031	2.8	8	15	$111.35	$21,157
S TO R	395	0.0029	5.9	8	21	$147.05	$58,085
M TO Q	190	0.0031	2.2	8	15	$111.35	$21,157
Q TO R	360	0.0029	3.4	8	15	$111.35	$40,086
N TO R	170	0.0031	23.1	12	33	$302.12	$51,360
OUTLET (R)			30.7				
TOTAL PIPE REPLACEMENT COST							$769,909
PIPE REPLACEMENT – $/AC (36.7 TOTAL AC IN BASIN)							$20,978

TABLE D–4
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
25–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 1: E AND 25TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	360	0.0026	3.6	8	18	$129.35	$46,566
D TO C	370	0.0033	4.2	8	18	$129.35	$47,860
C TO B	395	0.0040	6.9	8	21	$147.05	$58,085
B TO F	230	0.0043	11.1	10	24	$209.41	$48,164
E TO I	205	0.0043	3.7	8	15	$111.35	$22,827
I TO J	375	0.0030	6.4	8	21	$147.05	$55,144
F TO J	175	0.0043	14.0	10	24	$209.41	$36,647
H TO L	200	0.0054	4.0	8	15	$111.35	$22,270
P TO L	160	0.0046	2.6	8	15	$111.35	$17,816
L TO K	360	0.0035	8.1	8	21	$147.05	$52,938
G TO K	200	0.0054	3.6	8	15	$111.35	$22,270
K TO J	390	0.0030	12.6	8	27	$237.49	$92,621
J TO N	230	0.0026	28.2	12	36	$341.67	$78,584
T TO S	360	0.0033	2.7	8	15	$111.35	$40,086
O TO S	190	0.0031	3.6	8	15	$111.35	$21,157
S TO R	395	0.0029	7.5	8	21	$147.05	$58,085
M TO Q	190	0.0031	2.8	8	15	$111.35	$21,157
Q TO R	360	0.0029	4.4	8	18	$129.35	$46,566
N TO R	170	0.0031	30.1	12	36	$341.67	$58,084
OUTLET (R)			40.0				
TOTAL PIPE REPLACEMENT COST							$846,925
PIPE REPLACEMENT – $/AC (36.7 TOTAL AC IN BASIN)							$23,077

TABLE D–5

COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
2–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	160	0.0045	0.9	8	10	$79.85	$12,776
B TO E	200	0.0046	1.7	8	12	$94.77	$18,954
C TO D	220	0.0035	1.1	8	10	$79.85	$17,567
D TO E	185	0.0051	1.8	8	12	$94.77	$17,532
E TO F	210	0.0032	3.4	10	15	$111.35	$23,384
F TO G	220	0.0032	5.6	10	18	$129.35	$28,457
G TO H	210	0.0034	10.3	10	24	$209.41	$43,976
H TO I	215	0.0034	11.2	12	24	$209.41	$45,023
I TO J	225	0.0034	15.3	12	27	$237.49	$53,435
OUTLET (J)			15.0				
TOTAL PIPE REPLACEMENT COST							$261,104
PIPE REPLACEMENT – $/AC (34.1 TOTAL AC IN BASIN)							$7,657

TABLE D–6

COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
5–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	160	0.0045	1.4	8	10	$79.85	$12,776
B TO E	200	0.0046	2.5	8	15	$111.35	$22,270
C TO D	220	0.0035	1.7	8	12	$94.77	$20,849
D TO E	185	0.0051	2.8	8	15	$111.35	$20,600
E TO F	210	0.0032	5.5	10	18	$129.35	$27,164
F TO G	220	0.0032	8.8	10	21	$147.05	$32,351
G TO H	210	0.0034	16.5	10	27	$237.49	$49,873
H TO I	215	0.0034	17.8	12	27	$237.49	$51,060
I TO J	225	0.0034	23.7	12	30	$269.03	$60,532
OUTLET (J)			23.4				
TOTAL PIPE REPLACEMENT COST							$297,475
PIPE REPLACEMENT – $/AC (34.1 TOTAL AC IN BASIN)							$8,724

TABLE D–7

COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
10–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	160	0.0045	1.7	8	12	$94.77	$15,163
B TO E	200	0.0046	3.1	8	15	$111.35	$22,270
C TO D	220	0.0035	2.1	8	12	$94.77	$20,849
D TO E	185	0.0051	3.4	8	15	$111.35	$20,600
E TO F	210	0.0032	6.8	10	21	$147.05	$30,881
F TO G	220	0.0032	10.8	10	24	$209.41	$46,070
G TO H	210	0.0034	21.7	10	30	$269.03	$56,496
H TO I	215	0.0034	23.2	12	30	$269.03	$57,841
I TO J	225	0.0034	31.4	12	36	$341.67	$76,876
OUTLET (J)			30.3				
TOTAL PIPE REPLACEMENT COST							$347,047
PIPE REPLACEMENT – $/AC (34.1 TOTAL AC IN BASIN)							$10,177

TABLE D–8

COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
25–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 2: P AND 38TH STREETS

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	160	0.0045	2.2	8	12	$94.77	$15,163
B TO E	200	0.0046	4.0	8	15	$111.35	$22,270
C TO D	220	0.0035	2.7	8	15	$111.35	$24,497
D TO E	185	0.0051	4.4	8	15	$111.35	$20,600
E TO F	210	0.0032	8.7	10	21	$147.05	$30,881
F TO G	220	0.0032	14.0	10	27	$237.49	$52,248
G TO H	210	0.0034	26.2	10	33	$302.12	$63,445
H TO I	215	0.0034	28.4	12	33	$302.12	$64,956
I TO J	225	0.0034	37.9	12	36	$341.67	$76,876
OUTLET (J)			37.5				
TOTAL PIPE REPLACEMENT COST							$370,935
PIPE REPLACEMENT – $/AC (34.1 TOTAL AC IN BASIN)							$10,878

TABLE D–9
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
2–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	375	0.0038	1.1	8	10	$79.85	$29,944
B TO R	225	0.0037	2.7	8	15	$111.35	$25,054
Q TO R	120	0.0105	4.3	8	15	$111.35	$13,362
R TO N	810	0.0048	7.0	8	18	$129.35	$104,774
K TO E	510	0.0044	0.6	8	8	$66.58	$33,956
D TO E	150	0.0147	1.8	8	10	$79.85	$11,978
E TO F	265	0.0042	2.3	10	12	$94.77	$25,114
L TO F	470	0.0044	1.2	8	10	$79.85	$37,530
F TO G	130	0.0019	2.6	12	15	$111.35	$14,476
C TO H	640	0.0047	0.8	8	8	$66.58	$42,611
G TO H	155	0.0030	6.1	12	21	$147.05	$22,793
H TO M	140	0.0042	6.6	12	21	$147.05	$20,587
I TO J	200	0.0039	2.2	8/10	12	$94.77	$18,954
J TO M	340	0.0039	5.2	10/12	18	$129.35	$43,979
M TO N	190	0.0009	10.6	14	30	$269.03	$51,116
S TO P	775	0.0047	0.6	8	8	$66.58	$51,600
N TO P	350	0.0020	19.9	16	33	$302.12	$105,742
OUTLET (P)			19.2				
TOTAL PIPE REPLACEMENT COST							$653,567
PIPE REPLACEMENT – $/AC (43.7 TOTAL AC IN BASIN)							$14,956

TABLE D–10
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
5–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	375	0.0038	1.8	8	12	$94.77	$35,539
B TO R	225	0.0037	4.3	8	18	$129.35	$29,104
Q TO R	120	0.0105	6.7	8	18	$129.35	$15,522
R TO N	810	0.0048	10.6	8	21	$147.05	$119,111
K TO E	510	0.0044	1.0	8	10	$79.85	$40,724
D TO E	150	0.0147	2.7	8	12	$94.77	$14,216
E TO F	265	0.0042	3.1	10	15	$111.35	$29,508
L TO F	470	0.0044	1.8	8	12	$94.77	$44,542
F TO G	130	0.0019	4.1	12	18	$129.35	$16,816
C TO H	640	0.0047	1.2	8	10	$79.85	$51,104
G TO H	155	0.0030	9.5	12	24	$209.41	$32,459
H TO M	140	0.0042	10.3	12	24	$209.41	$29,317
I TO J	200	0.0039	3.3	8/10	15	$111.35	$22,270
J TO M	340	0.0039	7.8	10/12	21	$147.05	$49,997
M TO N	190	0.0009	16.4	14	36	$341.67	$64,917
S TO P	775	0.0047	1.0	8	10	$79.85	$61,884
N TO P	350	0.0020	30.6	16	42	$416.47	$145,765
OUTLET (P)			29.9				
TOTAL PIPE REPLACEMENT COST							$802,792
PIPE REPLACEMENT – $/AC (43.7 TOTAL AC IN BASIN)							$18,371

TABLE D-11
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	375	0.0038	2.2	8	12	$94.77	$35,539
B TO R	225	0.0037	5.3	8	18	$129.35	$29,104
Q TO R	120	0.0105	8.4	8	18	$129.35	$15,522
R TO N	810	0.0048	13.1	8	24	$209.41	$169,622
K TO E	510	0.0044	1.3	8	10	$79.85	$40,724
D TO E	150	0.0147	3.3	8	12	$94.77	$14,216
E TO F	265	0.0042	3.9	10	15	$111.35	$29,508
L TO F	470	0.0044	2.3	8	12	$94.77	$44,542
F TO G	130	0.0019	5.1	12	21	$147.05	$19,117
C TO H	640	0.0047	1.5	8	10	$79.85	$51,104
G TO H	155	0.0030	11.8	12	24	$209.41	$32,459
H TO M	140	0.0042	12.7	12	24	$209.41	$29,317
I TO J	200	0.0039	4.1	8/10	18	$129.35	$25,870
J TO M	340	0.0039	9.8	10/12	21	$147.05	$49,997
M TO N	190	0.0009	20.2	14	42	$416.47	$79,129
S TO P	775	0.0047	1.3	8	10	$79.85	$61,884
N TO P	350	0.0020	38.2	16	42	$416.47	$145,765
OUTLET			36.9				
TOTAL PIPE REPLACEMENT COST							$873,416
PIPE REPLACEMENT – $/AC (43.7 TOTAL AC IN BASIN)							$19,987

TABLE D-12
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 3: 14TH STREET AND 2ND AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	375	0.0038	2.9	8	15	$111.35	$41,756
B TO R	225	0.0037	6.9	8	21	$147.05	$33,086
Q TO R	120	0.0105	10.7	8	18	$129.35	$15,522
R TO N	810	0.0048	16.8	8	27	$237.49	$192,367
K TO E	510	0.0044	1.7	8	12	$94.77	$48,333
D TO E	150	0.0147	4.2	8	12	$94.77	$14,216
E TO F	265	0.0042	5.0	10	18	$129.35	$34,278
L TO F	470	0.0044	3.0	8	15	$111.35	$52,335
F TO G	130	0.0019	6.6	12	21	$147.05	$19,117
C TO H	640	0.0047	1.8	8	12	$94.77	$60,653
G TO H	155	0.0030	15.2	12	27	$237.49	$36,811
H TO M	140	0.0042	16.3	12	27	$237.49	$33,249
I TO J	200	0.0039	5.3	8/10	18	$129.35	$25,870
J TO M	340	0.0039	12.5	10/12	24	$209.41	$71,199
M TO N	190	0.0009	26.2	14	42	$416.47	$79,129
S TO P	775	0.0047	1.7	8	12	$94.77	$73,447
N TO P	350	0.0020	49.4	16	48	$500.82	$175,287
OUTLET (P)			47.4				
TOTAL PIPE REPLACEMENT COST							$1,006,653
PIPE REPLACEMENT – $/AC (43.7 TOTAL AC IN BASIN)							$23,036

TABLE D–13
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
2–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO C	130	0.0040	2.1	8	12	$94.77	$12,320
B TO C	130	0.0038	3.4	8	15	$111.35	$14,476
C TO D	400	0.0046	5.0	10	18	$129.35	$51,740
I TO F	635	0.0045	1.3	8	10	$79.85	$50,705
F TO G	80	0.0065	1.4	8	10	$79.85	$6,388
E TO G	95	0.0057	2.5	8	12	$94.77	$9,003
G TO H	375	0.0059	3.6	10	15	$111.35	$41,756
J TO H	105	0.0055	1.4	8	10	$79.85	$8,384
H TO K	190	0.0060	4.5	10	15	$111.35	$21,157
M TO D	130	0.0044	2.6	10	15	$111.35	$14,476
D TO L	125	0.0042	6.2	12	18	$129.35	$16,169
L TO K	180	0.0051	7.0	12/14	18	$129.35	$23,283
K TO O	265	0.0048	11.8	14/16	24	$209.41	$55,494
Q TO R	85	0.0036	0.3	8	8	$63.11	$5,364
P TO R	125	0.0030	2.5	8	15	$111.35	$13,919
R TO S	735	0.0062	2.6	8	15	$111.35	$81,842
S TO O	260	0.0058	4.1	8/10	15	$111.35	$28,951
TOTAL PIPE REPLACEMENT COST							$455,426
PIPE REPLACEMENT – $/AC (33.7 TOTAL AC IN BASIN)							$13,514

TABLE D-14
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO C	130	0.0040	3.1	8	15	$111.35	$14,476
B TO C	130	0.0038	5.3	8	18	$129.35	$16,816
C TO D	400	0.0046	7.8	10	21	$147.05	$58,820
I TO F	635	0.0045	2.0	8	12	$94.77	$60,179
F TO G	80	0.0065	2.2	8	12	$94.77	$7,582
E TO G	95	0.0057	3.8	8	15	$111.35	$10,578
G TO H	375	0.0059	5.6	10	18	$129.35	$48,506
J TO H	105	0.0055	2.2	8	12	$94.77	$9,951
H TO K	190	0.0060	6.9	10	15	$111.35	$21,157
M TO D	130	0.0044	3.9	10	15	$111.35	$14,476
D TO L	125	0.0042	9.5	12	21	$147.05	$18,381
L TO K	180	0.0051	10.6	12/14	21	$147.05	$26,469
K TO O	265	0.0048	17.9	14/16	27	$237.49	$62,935
Q TO R	85	0.0036	0.5	8	8	$63.11	$5,364
P TO R	125	0.0030	3.8	8	18	$129.35	$16,169
R TO S	735	0.0062	4.0	8	18	$129.35	$95,072
S TO O	260	0.0058	6.6	8/10	18	$129.35	$33,631
TOTAL PIPE REPLACEMENT COST							$520,560
PIPE REPLACEMENT – $/AC (33.7 TOTAL AC IN BASIN)							$15,447

TABLE D–15
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
10–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO C	130	0.0040	3.9	8	15	$111.35	$14,476
B TO C	130	0.0038	6.5	8	18	$129.35	$16,816
C TO D	400	0.0046	9.5	10	21	$147.05	$58,820
I TO F	635	0.0045	2.4	8	12	$94.77	$60,179
F TO G	80	0.0065	2.7	8	12	$94.77	$7,582
E TO G	95	0.0057	4.7	8	15	$111.35	$10,578
G TO H	375	0.0059	7.0	10	18	$129.35	$48,506
J TO H	105	0.0055	2.7	8	15	$111.35	$11,692
H TO K	190	0.0060	8.6	10	21	$147.05	$27,940
M TO D	130	0.0044	4.9	10	18	$129.35	$16,816
D TO L	125	0.0042	11.7	12	24	$209.41	$26,176
L TO K	180	0.0051	13.1	12/14	24	$209.41	$37,694
K TO O	265	0.0048	22.1	14/16	30	$269.03	$71,293
Q TO R	85	0.0036	0.6	8	8	$63.11	$5,364
P TO R	125	0.0030	4.7	8	18	$129.35	$16,169
R TO S	735	0.0062	5.0	8	18	$129.35	$95,072
S TO O	260	0.0058	8.2	8/10	21	$147.05	$38,233
TOTAL PIPE REPLACEMENT COST							$563,404
PIPE REPLACEMENT – $/AC (33.7 TOTAL AC IN BASIN)							$16,718

TABLE D-16
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 4: 25TH STREET AND 4TH AVENUE

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO C	130	0.0040	4.9	8	18	$129.35	$16,816
B TO C	130	0.0038	8.3	8	21	$147.05	$19,117
C TO D	400	0.0046	12.2	10	24	$209.41	$83,764
I TO F	635	0.0045	3.1	8	15	$111.35	$70,707
F TO G	80	0.0065	3.5	8	15	$111.35	$8,908
E TO G	95	0.0057	6.0	8	18	$129.35	$12,288
G TO H	375	0.0059	8.9	10	21	$147.05	$55,144
J TO H	105	0.0055	3.4	8	15	$111.35	$11,692
H TO K	190	0.0060	11.1	10	21	$147.05	$27,940
M TO D	130	0.0044	6.3	10	18	$129.35	$16,816
D TO L	125	0.0042	15.1	12	27	$237.49	$29,686
L TO K	180	0.0051	16.8	12/14	27	$237.49	$42,748
K TO O	265	0.0048	28.4	14/16	30	$269.03	$71,293
Q TO R	85	0.0036	0.7	8	8	$63.11	$5,364
P TO R	125	0.0030	6.1	8	21	$147.05	$18,381
R TO S	735	0.0062	6.4	8	21	$147.05	$108,082
S TO O	260	0.0058	10.6	8/10	21	$147.05	$38,233
TOTAL PIPE REPLACEMENT COST							$636,978
PIPE REPLACEMENT – $/AC (33.7 TOTAL AC IN BASIN)							$18,901

TABLE D-17
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	170	0.0032	0.8	8	10	$79.85	$13,574
B TO C	220	0.0032	1.8	8	12	$94.77	$20,849
C TO D	195	0.0032	2.8	8	15	$111.35	$21,713
D TO E	140	0.0032	3.3	8	15	$111.35	$15,589
E TO F	160	0.0032	3.5	8	15	$111.35	$17,816
F TO H	265	0.0032	3.8	8/10	18	$129.35	$34,278
G TO H	120	0.0066	1.2	8	10	$79.85	$9,582
H TO I	400	0.0033	5.2	10	18	$129.35	$51,740
J TO L	120	0.0053	1.5	8	10	$79.85	$9,582
K TO L	70	0.0154	0.8	8	8	$66.11	$4,628
L TO M	385	0.0044	2.1	8	12	$94.77	$36,486
N TO M	105	0.0053	0.4	8	8	$66.11	$6,942
M TO P	110	0.0050	2.2	8	12	$94.77	$10,425
O TO P	185	0.0084	0.9	8	8	$66.11	$12,230
P TO R	210	0.0029	3.3	8/10	15	$111.35	$23,384
Q TO R	190	0.0068	0.8	8	8	$66.11	$12,561
R TO I	120	0.0020	3.8	12	18	$129.35	$15,522
I TO T	110	0.0031	8.0	12	21	$147.05	$16,176
S TO T	180	0.0063	0.8	8	8	$66.11	$11,900
T TO V	115	0.0013	8.2	12	24	$209.41	$24,082
U TO V	410	0.0048	0.9	8	10	$79.85	$32,738
V TO X	100	0.0024	8.5	12	24	$209.41	$20,941
W TO X	190	0.0083	1.2	8	10	$79.85	$15,171
Y TO AA	115	0.0063	1.1	8	10	$79.85	$9,183
Z TO AA	115	0.0003	1.5	8	18	$129.35	$14,875
AA TO BB	555	0.0032	2.2	8	15	$111.35	$61,799
CC TO BB	365	0.0045	1.8	8	12	$94.77	$34,591
BB TO EE	115	0.0034	3.3	10	15	$111.35	$12,805
DD TO EE	350	0.0031	1.5	8	12	$94.77	$33,170
EE TO FF	240	0.0021	5.9	10/12	21	$147.05	$35,292
GG TO FF	175	0.0057	2.1	8	12	$94.77	$16,585
FF TO II	120	0.0029	6.8	12	21	$147.05	$17,646
HH TO II	550	0.0041	1.5	10	12	$94.77	$52,124
II TO X	115	0.0007	7.6	12	27	$237.49	$27,311
X TO KK	120	0.0011	17.0	16	33	$302.12	$36,254
JJ TO KK	590	0.0057	1.7	8	12	$94.77	$55,914
KK TO LL	125	0.0022	18.0	18	33	$302.12	$37,765
LL TO MM	115	0.0022	19.4	18	33	$302.12	$34,744
TOTAL PIPE REPLACEMENT COST							$917,968
PIPE REPLACEMENT – $/AC (44.0 TOTAL AC IN BASIN)							$20,863

TABLE D-18
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
5-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	170	0.0032	1.2	8	10	$79.85	$13,574
B TO C	220	0.0032	2.7	8	15	$111.35	$24,497
C TO D	195	0.0032	4.4	8	18	$129.35	$25,223
D TO E	140	0.0032	5.1	8	18	$129.35	$18,109
E TO F	160	0.0032	5.4	8	18	$129.35	$20,696
F TO H	265	0.0032	5.9	8/10	18	$129.35	$34,278
G TO H	120	0.0066	1.9	8	12	$94.77	$11,372
H TO I	400	0.0033	8.1	10	21	$147.05	$58,820
J TO L	120	0.0053	2.2	8	12	$94.77	$11,372
K TO L	70	0.0154	1.2	8	8	$66.11	$4,628
L TO M	385	0.0044	3.3	8	15	$111.35	$42,870
N TO M	105	0.0053	0.7	8	8	$66.11	$6,942
M TO P	110	0.0050	3.5	8	15	$111.35	$12,249
O TO P	185	0.0084	1.6	8	10	$79.85	$14,772
P TO R	210	0.0029	5.1	8/10	18	$129.35	$27,164
Q TO R	190	0.0068	1.3	8	10	$79.85	$15,171
R TO I	120	0.0020	6.1	12	21	$147.05	$17,646
I TO T	110	0.0031	12.5	12	24	$209.41	$23,035
S TO T	180	0.0063	1.4	8	10	$79.85	$14,373
T TO V	115	0.0013	12.9	12	30	$269.03	$30,938
U TO V	410	0.0048	1.4	8	10	$79.85	$32,738
V TO X	100	0.0024	13.4	12	30	$269.03	$26,903
W TO X	190	0.0083	1.9	8	10	$79.85	$15,171
Y TO AA	115	0.0063	1.8	8	12	$94.77	$10,899
Z TO AA	115	0.0003	2.3	8	21	$147.05	$16,911
AA TO BB	555	0.0032	3.4	8	15	$111.35	$61,799
CC TO BB	365	0.0045	2.7	8	15	$111.35	$40,643
BB TO EE	115	0.0034	5.1	10	18	$129.35	$14,875
DD TO EE	350	0.0031	2.3	8	15	$111.35	$38,973
EE TO FF	240	0.0021	9.1	10/12	24	$209.41	$50,258
GG TO FF	175	0.0057	3.4	8	15	$111.35	$19,486
FF TO II	120	0.0029	10.4	12	24	$209.41	$25,129
HH TO II	550	0.0041	2.3	10	12	$94.77	$52,124
II TO X	115	0.0007	11.9	12	33	$302.12	$34,744
X TO KK	120	0.0011	26.3	16	42	$416.47	$49,976
JJ TO KK	590	0.0057	2.7	8	12	$94.77	$55,914
KK TO LL	125	0.0022	27.9	18	42	$416.47	$52,059
LL TO MM	115	0.0022	30.1	18	42	$416.47	$47,894
TOTAL PIPE REPLACEMENT COST							$1,074,226
PIPE REPLACEMENT – $/AC (44.0 TOTAL AC IN BASIN)							$24,414

TABLE D-19
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
10-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	170	0.0032	1.5	8	12	$94.77	$16,111
B TO C	220	0.0032	3.4	8	15	$111.35	$24,497
C TO D	195	0.0032	5.5	8	18	$129.35	$25,223
D TO E	140	0.0032	6.4	8	21	$147.05	$20,587
E TO F	160	0.0032	6.8	8	21	$147.05	$23,528
F TO H	265	0.0032	7.5	8/10	21	$147.05	$38,968
G TO H	120	0.0066	2.3	8	12	$94.77	$11,372
H TO I	400	0.0033	10.1	10	24	$209.41	$83,764
J TO L	120	0.0053	2.8	8	15	$111.35	$13,362
K TO L	70	0.0154	1.5	8	8	$66.11	$4,628
L TO M	385	0.0044	4.2	8	15	$111.35	$42,870
N TO M	105	0.0053	0.8	8	8	$66.11	$6,942
M TO P	110	0.0050	4.4	8	15	$111.35	$12,249
O TO P	185	0.0084	2.0	8	10	$79.85	$14,772
P TO R	210	0.0029	6.4	8/10	21	$147.05	$30,881
Q TO R	190	0.0068	1.7	8	10	$79.85	$15,171
R TO I	120	0.0020	7.7	12	24	$209.41	$25,129
I TO T	110	0.0031	15.9	12	27	$237.49	$26,124
S TO T	180	0.0063	1.8	8	12	$94.77	$17,059
T TO V	115	0.0013	16.5	12	33	$302.12	$34,744
U TO V	410	0.0048	1.8	8	12	$94.77	$38,856
V TO X	100	0.0024	17.1	12	33	$302.12	$30,212
W TO X	190	0.0083	2.5	8	12	$94.77	$18,006
Y TO AA	115	0.0063	2.3	8	12	$94.77	$10,899
Z TO AA	115	0.0003	3.0	8	24	$209.41	$24,082
AA TO BB	555	0.0032	4.2	8	18	$129.35	$71,789
CC TO BB	365	0.0045	3.4	8	15	$111.35	$40,643
BB TO EE	115	0.0034	6.5	10	21	$147.05	$16,911
DD TO EE	350	0.0031	2.8	8	15	$111.35	$38,973
EE TO FF	240	0.0021	11.5	10/12	27	$237.49	$56,998
GG TO FF	175	0.0057	4.5	8	15	$111.35	$19,486
FF TO II	120	0.0029	13.1	12	27	$237.49	$28,499
HH TO II	550	0.0041	2.8	10	15	$111.35	$61,243
II TO X	115	0.0007	14.9	12	36	$341.67	$39,292
X TO KK	120	0.0011	33.4	16	42	$416.47	$49,976
JJ TO KK	590	0.0057	3.4	8	15	$111.35	$65,697
KK TO LL	125	0.0022	35.1	18	42	$416.47	$52,059
LL TO MM	115	0.0022	37.6	18	42	$416.47	$47,894
TOTAL PIPE REPLACEMENT COST							$1,199,493
PIPE REPLACEMENT – $/AC (44.0 TOTAL AC IN BASIN)							$27,261

TABLE D-20
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
25-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 5: HARKNESS AVENUE AND LARKIN WAY

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO B	170	0.0032	1.9	8	12	$94.77	$16,111
B TO C	220	0.0032	4.4	8	18	$129.35	$28,457
C TO D	195	0.0032	7.1	8	21	$147.05	$28,675
D TO E	140	0.0032	8.3	8	21	$147.05	$20,587
E TO F	160	0.0032	8.9	8	21	$147.05	$23,528
F TO H	265	0.0032	9.7	8/10	24	$209.41	$55,494
G TO H	120	0.0066	3.0	8	15	$111.35	$13,362
H TO I	400	0.0033	13.2	10	27	$237.49	$94,996
J TO L	120	0.0053	3.6	8	15	$111.35	$13,362
K TO L	70	0.0154	1.9	8	10	$79.85	$5,590
L TO M	385	0.0044	5.3	8	18	$129.35	$49,800
N TO M	105	0.0053	1.1	8	10	$79.85	$8,384
M TO P	110	0.0050	5.6	8	18	$129.35	$14,229
O TO P	185	0.0084	2.7	8	12	$94.77	$17,532
P TO R	210	0.0029	8.4	8/10	21	$147.05	$30,881
Q TO R	190	0.0068	2.2	8	12	$94.77	$18,006
R TO I	120	0.0020	10.1	12	24	$209.41	$25,129
I TO T	110	0.0031	20.6	12	30	$269.03	$29,593
S TO T	180	0.0063	2.4	8	12	$94.77	$17,059
T TO V	115	0.0013	21.4	12	36	$341.67	$39,292
U TO V	410	0.0048	2.3	8	12	$94.77	$38,856
V TO X	100	0.0024	22.2	12	36	$341.67	$34,167
W TO X	190	0.0083	3.3	8	15	$111.35	$21,157
Y TO AA	115	0.0063	2.9	8	15	$111.35	$12,805
Z TO AA	115	0.0003	3.8	8	24	$209.41	$24,082
AA TO BB	555	0.0032	5.4	8	18	$129.35	$71,789
CC TO BB	365	0.0045	4.3	8	15	$111.35	$40,643
BB TO EE	115	0.0034	8.3	10	21	$147.05	$16,911
DD TO EE	350	0.0031	3.6	8	15	$111.35	$38,973
EE TO FF	240	0.0021	14.7	10/12	30	$269.03	$64,567
GG TO FF	175	0.0057	5.9	8	18	$129.35	$22,636
FF TO II	120	0.0029	16.8	12	30	$269.03	$32,284
HH TO II	550	0.0041	3.6	10	15	$111.35	$61,243
II TO X	115	0.0007	19.1	12	42	$416.47	$47,894
X TO KK	120	0.0011	43.3	16	48	$500.82	$60,098
JJ TO KK	590	0.0057	4.3	8	15	$111.35	$65,697
KK TO LL	125	0.0022	45.9	18	48	$500.82	$62,603
LL TO MM	115	0.0022	48.8	18	48	$500.82	$57,594
TOTAL PIPE REPLACEMENT COST							$1,324,063
PIPE REPLACEMENT – $/AC (44.0 TOTAL AC IN BASIN)							$30,092

TABLE D-21
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
2-YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO C	210	0.0047	0.7	8	8	$66.11	$13,883
B TO C	215	0.0045	1.3	8	10	$79.85	$17,168
C TO D	265	0.0035	4.3	10	18	$129.35	$34,278
D TO H	475	0.0034	4.6	12	18	$129.35	$61,441
E TO F	370	0.0039	0.8	8	10	$79.85	$29,544
F TO G	575	0.0053	2.4	8	12	$94.77	$54,493
G TO H	655	0.0047	1.3	8	10	$79.85	$52,302
H TO J	170	0.0025	6.4	12	21	$147.05	$24,999
L TO K	140	0.0062	0.9	8	8	$66.11	$9,255
K TO J	185	0.0037	2.0	8	12	$94.77	$17,532
I TO J	300	0.0036	1.3	8	10	$79.85	$23,955
J TO N	310	0.0025	9.9	14	24	$209.41	$64,917
O TO N	250	0.0044	2.5	8	15	$111.35	$27,838
M TO N	315	0.0047	2.0	8	12	$94.77	$29,853
N TO O	410	0.0026	12.4	14	27	$237.49	$97,371
TOTAL PIPE REPLACEMENT COST							$558,828
PIPE REPLACEMENT – $/AC (28.5 TOTAL AC IN BASIN)							$19,608

TABLE D–22
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
5–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO C	210	0.0047	1.2	8	10	$79.85	$16,769
B TO C	215	0.0045	1.9	8	12	$94.77	$20,376
C TO D	265	0.0035	6.6	10	21	$147.05	$38,968
D TO H	475	0.0034	7.0	12	21	$147.05	$69,849
E TO F	370	0.0039	1.2	8	10	$79.85	$29,544
F TO G	575	0.0053	3.7	8	15	$111.35	$64,026
G TO H	655	0.0047	2.0	8	12	$94.77	$62,074
H TO J	170	0.0025	9.9	12	24	$209.41	$35,600
L TO K	140	0.0062	1.5	8	10	$79.85	$11,179
K TO J	185	0.0037	3.1	8	15	$111.35	$20,600
I TO J	300	0.0036	1.9	8	12	$94.77	$28,431
J TO N	310	0.0025	15.3	14	27	$237.49	$73,622
O TO N	250	0.0044	3.8	8	15	$111.35	$27,838
M TO N	315	0.0047	3.0	8	15	$111.35	$35,075
N TO O	410	0.0026	18.9	14	30	$269.03	$110,302
TOTAL PIPE REPLACEMENT COST							$644,253
PIPE REPLACEMENT – $/AC (28.5 TOTAL AC IN BASIN)							$22,605

TABLE D–23
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
10–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO C	210	0.0047	1.5	8	10	$79.85	$16,769
B TO C	215	0.0045	2.4	8	12	$94.77	$20,376
C TO D	265	0.0035	8.2	10	21	$147.05	$38,968
D TO H	475	0.0034	8.7	12	21	$147.05	$69,849
E TO F	370	0.0039	1.6	8	12	$94.77	$35,065
F TO G	575	0.0053	4.7	8	15	$111.35	$64,026
G TO H	655	0.0047	2.5	8	15	$111.35	$72,934
H TO J	170	0.0025	12.4	12	27	$237.49	$40,373
L TO K	140	0.0062	1.8	8	12	$94.77	$13,268
K TO J	185	0.0037	3.9	8	15	$111.35	$20,600
I TO J	300	0.0036	2.4	8	15	$111.35	$33,405
J TO N	310	0.0025	19.1	14	30	$269.03	$83,399
O TO N	250	0.0044	4.8	8	18	$129.35	$32,338
M TO N	315	0.0047	3.7	8	15	$111.35	$35,075
N TO O	410	0.0026	23.7	14	33	$302.12	$123,869
TOTAL PIPE REPLACEMENT COST							$700,314
PIPE REPLACEMENT – $/AC (28.5 TOTAL AC IN BASIN)							$24,572

TABLE D–24
COLLECTION SYSTEM IMPROVEMENT COST ESTIMATE
25–YEAR RATIONAL METHOD ANALYSIS

REPRESENTATIVE BASIN 6: 56TH STREET AND BROADWAY

PIPE SECTION	PIPE LENGTH (LF)	PIPE SLOPE (ft/ft)	PEAK FLOW (cfs)	EXISTING PIPE DIAMETER (inches)	REPLACE PIPE DIAMETER (inches)	UNIT COST ($/LF)	COST ($)
A TO C	210	0.0047	1.9	8	12	$94.77	$19,902
B TO C	215	0.0045	3.1	8	15	$111.35	$23,940
C TO D	265	0.0035	10.6	10	24	$209.41	$55,494
D TO H	475	0.0034	11.4	12	24	$209.41	$99,470
E TO F	370	0.0039	2.0	8	12	$94.77	$35,065
F TO G	575	0.0053	6.0	8	18	$129.35	$74,376
G TO H	655	0.0047	3.2	8	15	$111.35	$72,934
H TO J	170	0.0025	15.8	12	30	$269.03	$45,735
L TO K	140	0.0062	2.4	8	12	$94.77	$13,268
K TO J	185	0.0037	5.0	8	18	$129.35	$23,930
I TO J	300	0.0036	3.1	8	15	$111.35	$33,405
J TO N	310	0.0025	24.3	14	33	$302.12	$93,657
O TO N	250	0.0044	6.1	8	18	$129.35	$32,338
M TO N	315	0.0047	4.8	8	18	$129.35	$40,745
N TO O	410	0.0026	30.4	14	36	$341.67	$140,085
TOTAL PIPE REPLACEMENT COST							$804,343
PIPE REPLACEMENT – $/AC (28.5 TOTAL AC IN BASIN)							$28,223

APPENDIX E

DETAILED RATIONAL METHOD CALCULATIONS

APPENDIX E

DETAILED RATIONAL METHOD CALCULATIONS

BASIN 1	E AND 25TH STREETS
BASIN 2	P AND 38TH STREETS
BASIN 3	14TH STREET AND 2ND AVENUE
BASIN 4	25TH STREET AND 4TH AVENUE
BASIN 5	HARKNESS AVENUE AND LARKIN WAY
BASIN 6	56TH STREET AND BROADWAY

Representative Basin No. 1

E and 25th Streets

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: LAC Date: 12/12/91
Checked by: _____ Date: _____
E Street and 25th St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope 3 1/1	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	2.0	2.0	0.5	i @ 6 min		1.5	1.5		0.0026	0.013	11.2"	Use	12"				
	Overland	150	0.85	fps			2.9					V_{12} =	2.3	fps						
	Street	150	0.8	fps			3.1													
							6.0	min				t_{pipe} =	360/2.3(60) =			2.6 min				
	Street Slope:	(9d-67)	-(9d-101)	RIMS								t_c =	6+2.6 =			8.6 min				
	(24.1-23.1)/575 =	0.0017 1/1																		
	Design Slope:	(9d-67)	-(9d-101)	inverts																
	(20.9-19.4)/575 =	0.0026 1/1																		
4		D	C	2.9	2.9	0.5	i @ 9.3 min		1.2	1.7		0.0033	0.013	11.2"	Use 12"					
	Overland	150	0.85	fps			2.9													
	Street	300	0.78	fps			6.4													
							9.3	min				V_{12} =	2.6	fps						
	Street Slope:	(9c-20)	-(9c-2)	RIMS								t_{pipe} =	370/2.6(60) =			2.4 min				
	(24.74-24.18)/370 =	0.0015 1/1																		
	Design Slope:	(9c-20)	-(9c-2)	Inverts								t_c =	9.3 + 2.4 =			11.7 min				
	(22.19-20.98)/370	= 0.0033 1/1																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LHC Date 12/12/91
Checked by _____ Date _____
E Street and 25th St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3		C	B	2.3	5.2	0.5	i @ 11.	7 min	1.1	2.9		0.004	0.013	13.2"	Use	15"				
	Contributing Drainage Areas 3 and 4																			
	Overland	250'	0.85				4.9	min				V_{15} = 3.3 fps								
	Design Slope:	(9c-2) -	(9d-101)									t_{pipe} = 395/3.3(60) =				2.0 min				
	(20.98-19.4)/395 =	0.004'/'										t_c = 11.7 + 2 = 13.7 min.								
2		B	F		9.0	0.5	i @ 13.	7 min	0.98	4.4		0.0043	0.013	15.2"	Use	18"				
	Contributing Drainage Areas 1, 2, 3 and 4											V_{18} = 3.9 fps								
	t_c A to B = 8.6 min											t_{pipe} = 230/3.9(60) =				1 min				
	t_c C to B = 13.7 min											t_c = 13.7 + 1 =				14.7 min				
	t_c 13.7 min governs																			
	Design Slope:	(9d-101)	-(9d-103)																	
	(19.4-18.4)/230 =	0.0043'/'																		
5		E	I	2.0	2.0	0.5	i @ 5.	7 min	1.5	1.5		0.0043	0.013	10.1"	Use	12"				
	Overland	150	0.85	fps			2.9					V_{12} = 3.0 fps								
	Street	150	0.9	fps			2.8													
							5.7					t_{pipe} = 205/3(60) =				1.1 min				
	Street Slope: Assume same as street slope from F to J																			
	(9d-103) - (9d-	104) RIMS										t_c = 5.7 + 1.1 =				6.8 min				
	(22.89-22.52)/175	= 0.0021	'/'																	
	Design Slope: Assume same as invert slope from F to J																			
	-103) - (9d-	104) Inverts																		
	39-17.64)/175	= 0.0043	'/'																	

Computation Form
Drainage and Storm Sewer Design
Rational Method

Project Sacramento CSO - Phase 3
Computed by JAC Date 12/12/91
Checked by _____ Date _____
E Street and 25th St - 2 year

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
9		I	J	1.7	3.7	0.5	i@	6.8 min	1.4	2.6		0.003	0.013	13.3'	Use	15"				
	Overland	150'	0.85	fps			2.9				t_c	E to I	= 6.8 min							
	8" Sewer	375	1.9	fps			3.3				t_c	Area	9 = 6.2 min							
							6.2	min				t_c =	6.8 govern							
	8" Sewer: (9d-73)-(9d-104)			Drainage Inlets			In alley						V_{15} =	2.9	fps					
	(1945-17.64)/600 = 0.0030'/'																			
	Design Slope:	Same as 8" sewer slope											t_{pipe} =	3.75/60(2.9) =		2.2 min				
	Slope = 0.0030 '/'												t_c =	6.7 + 2.2 =		8.9 min				
6		F	J	2.8	11.8	0.5	i@14.7 min		0.93	5.5		0.0043	0.013	16.5'	Use	18"				
	t_c B to F =	14.7 min											V_{18} =	3.9	fps					
	No t_c computations for Area,												t_{pipe} =	175/3.9(60) =		0.7 min				
	upstream basins	t_c govern																		
	Design Slope	(9d-103) - (9d-104)											t_c =	14.7 + 0.7 =		15.4 min				
	(18.39-17.64)/175 = 0.0043'/'																			
8		H	L	2.1	2.1	0.5	i@5.2 min		1.5	1.6		0.0054	0.013	10.0"	Use	10"				
	Overland	150'	0.85	fps			2.9						V_{10} =	3.0	fps					
	Street	150	1.1	fps			2.3						t_{pipe} =	200/3(60) =		1.1 min				
							5.2	min												
	Street Slope:	(9c-21)	-(9c-22)	RIMS									t_c =	5.2 + 1.1 =		6.3 min				
	(24.85-24.25)/200 =		0.003 '/'																	
	Design Slope:	(9c-21)	-(9c-22)	Inverts																
	(21.75-20.67)/200 =		0.0054 '/'																	

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LHC Date 12/15/91
Checked by _____ Date _____
E Street and 25th St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope, %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
16		P	L	1.2	1.2	0.5	i @ 4.4	min	1.7	1.0		0.0046	0.013	8.6"	Use	10"				
	Overland	150'	0.85	fps			2.9	min					V_{10} =	2.7	fps					
	Street	100'	1.1	fps			1.5													
							4.4	min					t_{pipe} =	160/2.7(60) =		1.0	min			
	Street Slope:	Adverse	street	grades	from	P to L														
	Assume same slope as	H to L		Slope = 0.003									t_c =	4.4 + 1 =		5.4	min			
	Design Slope:	(9c-25)	-(9c-22)		Inverts															
	(21.44 - 20.7)/160	= 0.0046																		
12		L	K	1.2	4.5	0.5	i @ 6.3	min	1.45	3.3		0.0035	0.013	14.2"	Use	15"				
	Contributing Drainage Areas		8, 12 and 16																	
	Design Slope:	(9c-23)	-(9c-6)										$V_{1.5}$ =	3.1	fps					
	(20.65 - 19.38)/360	= 0.0035											t_{pipe} =	360/60(3.1) =		1.9	min			
													t_c =	6.3 + 1.9 =		8.2	min			
	t_c P to L =	5.4 min																		
	t_c H to L =	6.3 min																		
	6.3 > 5.4	∴ 6.3 t_c	governs																	
7		G	K	1.9	1.9	0.5	i @	5.2 min	1.54	1.5		0.0054	0.013	9.7"	Use	10"				
	Overland	150	0.85	fps			2.9						V_{10} =	3.0	fps					
	Street	150	1.1	fps			2.3													
							5.2	min					t_{pipe} =	200/3(60) =		1.1	min			
	Street Slope: Assume same slope as H to L (street)																			
	Slope = 0.003'												t_c =	5.2 + 1.1 =		6.3	min			
	Design Slope: Assume same slope as H to L (pipe)																			
	Slope = 0.0054																			

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO-Phase 3
Computed by: LAC Date: 12/15/91
Checked by: _____ Date: _____
E Street and 25th St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11		K	J	1.3	7.7	0.5	1 @ 8	2 min	1.3	5.0		0.003	0.013	17.1"	Use	18"				
	No t_c computations made for Area 11,												V_{18}	= 3.2	fps					
	upstream basins will govern																			
	t_c G to K =	6.3 min											t_{pipe}	= 390/3.2(60) =			2 min			
	t_c L to K =	8.2 min											t_c	= 8.2 + 2 =			10.2 min			
	Contributing Areas 8, 11, 12, 16																			
	Design Slope:	(9c-6) -	(9d-104)																	
	(19.38 - 18.22)/390	= 0.003	ft/ft																	
10		J	N	1.3	24.5	0.5	1 @ 15	4 min	0.91	11.1		0.0026	0.013	23.6"	Use 24"					
	t_c from F to J	15.4 min												V_{24} =	3.7 fps					
	t_c from I to J	8.9 min												t_{pipe} =	230/3.7(60) =		1 min			
	t_c from K to J	= 10.2 min												t_c =	15.4 + 1 = 16.4 min					
	governing	t_c = 15.4 min																		
	Contributing areas 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12 and 16																			
	Design Slope:	(9d-104)	- (9d-106)																	
	(17.02 - 17.02)/	230 =	0.0026	ft/ft																

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Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/15/91
Checked by _____ Date _____
E Street and 25th St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope 3%	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
20		T	S	1.5	1.5	0.5	i @	6.2 min	1.46	1.1		0.0033	0.013	9.5"	Use	10"				
	Overland	100	0.85	fps			2.0						V_{10} =	2.3	fps					
	Street	250	1.0	fps			4.2													
							6.2	min					t_{pipe} =	360/2.3(60) =		2.6 min				
	Street Slope:	No data,	assume	velocity	of 1fps								t_c =	6.2 + 2.6 =		8.8 min				
	Design Slope:	(9c-26)	-(9c-10)																	
	(19.64-18.44)/360	= 0.0033	1/'																	
15		O	S	2.2	2.2	0.5	i @	7.8 min	1.3	1.4		0.0031	0.013	10.5"	Use	12"				
	Overland	150	0.85	fps			2.9						V_{12} =	2.5	fps					
	Street	250	0.85	fps			4.9													
							7.8	min					t_{pipe} =	190/2.5(60) =		1.3 min				
	Street Slope:	Assume	same as N to R										t_c =	7.8 + 1.3 =		9.1 min				
	(9c-106) - (9c	-107)	RIMS																	
	(22.36-22.04)/	170	0.0019'/'																	
	Design Slope:	Assume	same as N to R																	
	(9c-106) - (9c-	107)	Inverts																	
	(17.02-16.49)/170	= 0.	0031'/'																	
19		S	R	1.3	5.0	0.5	i @	9.1 min	1.2	3.0		0.0029	0.013	14.2"	Use	15"				
	No t_c computations made for Area 19, Use governing t_c from upstream												V_{15} =	2.8 fps						
	Design Slope:	(9c-10)	-(9d-107)			t_c	from	T to S =	8.8 min				t_{pipe} =	395/2.8(60) =		2.4 min				
	(18.44-17.29)/395' =		0.0029	1/'		t_c	from	O to S =	9.1 min											
						governing	t_c =		9.1 min				t_c =	9.1 + 2.4 =		11.5 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DMC Date 12/15/91
Checked by _____ Date _____
E Street and 25th St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
13		M	Q	2.1	2.1	0.5	i@	10.7 min	1.1	1.2		0.0031	0.013	9.9"	Use	10"				
	Overland	150	0.85	fps			2.9					V_{10} = 2.2 fps								
	Street	400	0.85	fps			7.8													
							10.7 min					t_{pipe} = 190/2.2(60) =				1.4 min				
	Street Slope:	Assume	same as	N to R								t_c = 10.7 + 1.4 =				12.1 min				
	(9c-106) - (9c-107) RIMS																			
	Slope = 0.0019 ft/ft																			
	Design Slope:	Assume	same as N to R																	
	(9c-106) - (9c-107) Inverts																			
	Slope = 0.0031 ft/ft																			
17		Q	R	1.3	3.4	0.5	i@ 12.1 min		1.0	1.7		0.0029	0.013	11.4"	Use	12"				
	No t_c computation, governing t_c from upstream											V_{12} = 2.4 fps								
												t_{pipe} = 360/2.4(60) =				2.5 min				
	Design Slope:	No data, Assume same slope as S to R										t_c = 12.1 + 2.5 =				14.6 min				
	Slope = 0.0029 ft/ft																			
14		N	R	2.7	27.2	0.5	i@ 16.4 min		0.9	12.2		0.0031	0.013	23.7	Use	24"				
	No t_c computation for Area 14, governing t_c from upstream J to N																			
	Contributing Areas - 1-12, 14.											V_{24} = 4.0								
	Design Slope: (9d-106) - (9d-107)																			
	(17.02 - 16.49)/170 = 0.0031 ft/ft											t_{pipe} = 170/4.0(60) =				0.7 min				
												t_c = 16.4 + 0.7 =				17.1 min				
18			R	1.1	36.7	0.5	i@ 17.1 min		0.88	16.1										

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Computation Form
Drainage and Storm Sewer Design
Rational Method

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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO-Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - [illegible]

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1	24th to 25th Sts	A	B	2.0	2.0	0.5	i@	6 min	2.27	2.3		0.0026	0.013	13.1"	Use	15"				
												V_{15} =	2.7 fps							
												t_{pipe} =	360/60(2.7) =			2.2 min				
												t_c =	6+	2.2 =		8.2 min				
4	27th to 26th Sts	D	C	2.9	2.9	0.5	i@	9.3 min	1.83	2.6		0.0033	0.013	13.1"	Use	15"				
												V_{15} =	3.0 fps							
												t_{pipe} =	370/60(3) =			2.1 min				
												t_c =	9.3+	2.1 =		11.4 min				
3	26th to 25th Sts	C	B	2.3	5.2	0.5	i@	11.4 min	1.64	4.3		0.004	0.013	15.3"	Use	18"				
												V_{18} =	3.8 fps							
												t_{pipe} =	395/60(3.8) =			1.7 min				
												t_c =	11.4+1.7 =			13.1 min				
2	Alley at 25th St to "C" St	B	F	–	9.0	0.5	i@	13.1 min	1.53	6.9		0.0044	0.013	17.9"	Use	18"				
												V_{18} =	4.0 fps							
												t_{pipe} =	230/60(4) =			1 min				
												t_c =	13.1+1 =			14.1 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 5year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5	24th + "C" to 24th + alley	E	I	2.0	2.0	0.5	i @ 5.7 min		2.31	2.3		0.0043	0.013	11.9"	Use	12"				
												V_{12} =	3.0 fps							
												t_{pipe} =	205	/60(3) =		0.4 min				
												t_c =	5.7	+ 0.4 =		6.1 min				
9	24th + Alley to 25th + Alley	I	J	1.7	3.7	0.5	i @	6.1 min	2.25	4.2		0.003	0.013	16.0"	Use	18"				
												V_{18} =	3.3 fps							
												t_{pipe} =	375	/60(3.3) =		1.9 min				
												t_c =	6.1	+ 1.9 =		8 min				
6	25th + "C" Sts to 25th + Alley	F	J	2.8	11.8	0.5	i @	14.1 min	1.46	8.6		0.0043	0.013	19.5"	Use	21"				
												V_{21} =	4.3 fps							
												t_{pipe} =	175	/60(4.3) =		0.7 min				
												t_c =	14.1	+ 0.7 =		14.8 min				
8	27th + "C" Sts to 27th + Alley	H	L	2.1	2.1	0.5	i @	5.2 min	2.37	2.5		0.0054	0.013	11.8"	Use	12"				
												V_{12} =	3.3 fps							
												t_{pipe} =	200	/60(3.3) =		1 min				
												t_c =	5.2 +	1 =		6.2 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
16	27th + "D" Sts to 27th + Alley	P	L	1.2	1.2	0.5	i @ 4	4 min	2.64	1.6		0.0046	0.013	10.3"	Use	12"				
												V_{12} =	3.1	fps						
												t_{pipe} =	160/60(3.1) =			0.9 min				
												t_c =	4.4 +	0.9 =		5.3 min				
12	27th + Alley to 26th + Alley	L	K	1.2	4.5	0.5	i @	6.2 min	2.24	5.0		0.0035	0.013	16.6"	Use	18"				
												V_{18} =	3.5	fps						
												t_{pipe} =	360/60(3.5) =			1.7 min				
												t_c =	6.2 +	1.7 =		7.9 min				
7	26th + "C" to 26th + Alley	G	K	1.9	1.9	0.5	i @	5.2 min	2.37	2.3		0.0054	0.013	11.4"	Use	12"				
												V_{12} =	3.3	fps						
												t_{pipe} =	200/(60)(3.3) =			1 min				
												t_c =	5.2 +	1 =		6.2 min				
11	26th + Alley to 25th + Alley	K	J	1.3	7.7	0.5	i @	7.9 min	2.02	7.8		0.003	0.013	20.2"	Use	21"				
												V_{21} =	3.6	fps						
												t_{pipe} =	390/60(3.6) =			1.8 min				
												t_c =	7.9 + 1.8 =			9.7 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: LAL Date: 12/17/91
Checked by: _____ Date: _____

E and 25th Sts - 5year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
10	25th + Alley to 25th + "D"	J	N	1.3	24.5	0.5	i @	14.8 min	1.41	17.3		0.0026	0.013	27.9"	Use	30"				
												V_{30} =	4.3	fps						
												t_{pipe} =	230	/60 (4.3) =		0.9 min				
												t_c =	14.8 + 0.9 =			15.7 min				
20	27th + Alley to 26th + Alley	T	S	1.5	1.5	0.5	i @ 6.2 min		2.24	1.7		0.0033	0.013	11.2"	Use	12"				
												V_{12} =	2.6	fps						
												t_{pipe} =	360	/60 (2.6) =		2.3 min				
												t_c =	6.2 + 2.3 =			8.5 min				
15	26th + "D" Sts to 26th + Alley	O	S	2.2	2.2	0.5	i @ 7.8 min		2.03	2.2		0.0031	0.013	12.5"	Use	15"				
												V_{15} =	2.9	fps						
												t_{pipe} =	190	/60 (2.9) =		1.1 min				
												t_c =	7.8 + 1.1 =			8.9 min				
19	26th St + Alley to 25th St + Alley	S	R	1.3	5.0	0.5	i @	8.9 min	1.89	4.7		0.0029	0.013	16.8"	Use	18"				
												V_{18} =	3.2	fps						
												t_{pipe} =	395	/60 (3.2) =		2.1 min				
												t_c =	8.9 + 2.1 =			11.0 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
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I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
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t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 5year

Drainage Area Number	Location			A		C	Time of Concentration		I	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
13	24th + "D" Sts to 24th + Alley	M	Q	2.1	2.1	0.5	i @ 10.7 min		1.69	1.8		0.0031	0.013	11.6"	Use	12"				
												V_{12} =	2.5	fps						
												t_{pipe} =	190/60(2.5) =			1.3 min				
												t_c =	10.7 + 1.3 =			12.0 min				
17	24th + Alley to 25th + Alley	Q	R	1.3	3.4	0.5	i @	12.0 min	1.60	2.7		0.0029	0.013	13.6"	Use	15"				
												V_{15} =	2.8	fps						
												t_{pipe} =	360/60(2.8) =			2.1 min				
												t_c =	12.0 + 2.1 =			14 min				
14	25th + "D" Sts to 25th + Alley	N	R	2.7	27.2	0.5	i @	15.7 min	1.38	18.8		0.0031	0.013	27.9"	Use	30"				
												V_{30} =	4.7	fps						
												t_{pipe} =	170/60(4.7) =			0.6 min				
												t_c =	15.7 + 0.6 =			16.3 min				
18	25th St + Alley		R	1.1	36.7	0.5	i @	16.3 min	1.36	25.0										

Computation Form
Drainage and Storm Sewer Design
Rational Method

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V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO-Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 10year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1	24th to 25th Sts	A	B	2.0	2.0	0.5	i@	6 min	2.83	2.8		0.0026	0.013	14.1"	Use	15"				
												V_{15} =	2.7	fps						
												t_{pipe} =	360/	60(2.7) =		2.2 min				
												t_c =	6+	2.2 =		8.2 min				
4	27th to 26th Sts	D	C	2.9	2.9	0.5	i@	9.3 min	2.28	3.3		0.0033	0.013	14.3"	Use	15"				
												V_{15} =	3.0	fps						
												t_{pipe} =	370	/60(3) =		2.1 min				
												t_c =	9.3+	2.1 =		11.4 min				
3	26th to 25th Sts	C	B	2.3	5.2	0.5	i@	11.4 min	2.04	5.3		0.004	0.013	16.5"	Use	18"				
												V_{18} =	3.8	fps						
												t_{pipe} =	395	/60(3.8) =		1.7 min				
												t_c =	11.4	+1.7 =		13.1 min				
2	Alley at 25th St to "C" St	B	F	–	9.0	0.5	i@	13.1 min	1.89	8.5		0.0043	0.013	19.4"	Use	21"				
												V_{21} =	4.4	fps						
												t_{pipe} =	230	/60(4.4) =		0.9 min				
												t_c =	13.1	+0.9 =		14.0 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope 1/1	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5	24th + "C" to 24th + alley	E	I	2.0	2.0	0.5	i @ 5	7 min	2.88	2.9		0.0043	0.013	13.0"	Use	15"				
												V_{15} =	3.5	fps						
												t_{pipe} =	205	/60(3.5) =		1 min				
												t_c =	5.7	+ 1.0	= 6.7	min				
9	24th + Alley to 25th + Alley	I	J	1.7	3.7	0.5	i @	6.7 min	2.71	5.0		0.003	0.013	17.1"	Use	18				
												V_{18} =	3.3	fps						
												t_{pipe} =	375	/60(3.3) =		1.9 min				
												t_c =	6.7	+ 1.9	= 8.6	min				
6	25th + "C" Sts to 25th + Alley	F	J	2.8	11.8	0.5	i @	14.0 min	1.81	10.7		0.0043	0.013	21.2"	Use	24				
												V_{24} =	4.7	fps						
												t_{pipe} =	175	/60(4.7) =		0.6 min				
												t_c =	14	+ 0.6 =	14.6	min				
8	27th + "C" Sts to 27th + Alley	H	L	2.1	2.1	0.5	i @	5.2 min	2.97	3.1		0.0054	0.013	12.8"	Use	15"				
												V_{15} =	3.9	fps						
												t_{pipe} =	200	/60(3.9) =		0.8				
												t_c =	5.2	+ 0.8	= 6.0	min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 10year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
16	27th & "D" Sts to 27th & Alley	P	L	1.2	1.2	0.5	i @ 4	4 min	3.36	2.0		0.0046	0.013	11.2"	Use	12"				
												V_{12} = 3.1 fps								
												t_{pipe} = 160/60(3.1) =				0.9 min				
												t_c = 4.4 + 0.9 =				5.3 min				
12	27th & Alley to 26th & Alley	L	K	1.2	4.5	0.5	i @	6.0 min	2.83	6.4		0.0035	0.013	18.2"	Use	21"				
												V_{21} = 3.9 fps								
												t_{pipe} = 360/60(3.9) =				1.5 min				
												t_c = 6 + 1.5 =				7.5 min				
7	26th & "C" to 26th & Alley	G	K	1.9	1.9	0.5	i @	5.2 min	2.97	2.8		0.0054	0.013	12.2"	Use	15"				
												V_{15} = 3.9 fps								
												t_{pipe} = 200/(60)(3.9) =				0.9 min				
												t_c = 5.2 + 0.9 =				6.1 min				
11	26th & Alley to 25th & Alley	K	J	1.3	7.7	0.5	i @	7.5 min	2.58	9.9		0.003	0.013	22.0"	Use	24"				
												V_{24} = 4.0 fps								
												t_{pipe} = 390/60(4) =				1.6 min				
												t_c = 7.5 + 1.6 =				9.1 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
10	25th + Alley to 25th + "D"	J	N	1.3	24.5	0.5	i @	14.6 min	1.76	21.6		0.0026	0.013	30.3"	Use	33"				
												V_{33} =	4.6	fps						
												t_{pipe} =	230	/60 (4.6) =		0.8 min				
												t_c =	14.6	+ 0.8 =		15.4 min				
20	27th + Alley to 26th + Alley	T	S	1.5	1.5	0.5	i @ 6	2 min	2.80	2.1		0.0033	0.013	12.1"	Use	15"				
												V_{15} =	3.0	fps						
												t_{pipe} =	360	/60 (3) =		2 min				
												t_c =	6.2	+ 2 =		8.2 min				
15	26th + "D" Sts to 26th + Alley	O	S	2.2	2.2	0.5	i @ 7	8 min	2.53	2.8		0.0031	0.013	13.6"	Use	15"				
												V_{15} =	2.9	fps						
												t_{pipe} =	190	/60 (2.9) =		1.1 min				
												t_c =	7.8	+ 1.1 =		8.9 min				
19	26th St + Alley to 25th St + Alley	S	R	1.3	5.0	0.5	i @	8.9 min	2.34	5.9		0.0029	0.013	18.3"	Use	21"				
												V_{21} =	3.6	fps						
												t_{pipe} =	395	/60 (3.6) =		1.8 min				
												t_c =	8.9	+ 1.8 =		10.7 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
13	24th + "D" Sts to 24th + Alley	M	Q	2.1	2.1	0.5	i @ 10.7 min		2.10	2.2		0.0031	0.013	12.5"	Use 15"					
												V_{15} = 2.9 fps								
												t_{pipe} = 190/60(2.9) = 1.1								
												t_c = 10.7 + 1.1 = 11.8 min								
17	24th + Alley to 25th + Alley	Q	R	1.3	3.4	0.5	i @	11.8 min	2.0	3.4		0.0029	0.013	14.9"	Use 15"					
												V_{15} = 2.8 fps								
												t_{pipe} = 360/60(2.8) = 2.1 min								
												t_c = 11.8 + 2.1 = 13.9 min								
14	25th + "D" Sts to 25th + Alley	N	R	2.7	27.2	0.5	i @	15.4 min	1.7	23.1		0.0031	0.013	30.1"	Use 33"					
												V_{33} = 5.0 fps								
												t_{pipe} = 170/60(5.0) = 0.6 min								
												t_c = 15.4 + 0.6 = 16 min								
18	25th St + Alley		R	1.1	36.7	0.5	i @	16.0	1.68	30.8										

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
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V = Velocity, in fps
t = Time, in minutes
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Project Sacramento CSO-Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1	24th to 25th Sts	A	B	2.0	2.0	0.5	i@	6 min	3.62	3.6		0.0026	0.013	15.5"	Use	18"				
												V_{18} =	3.0 fps							
												t_{pipe} =	360/60(3) = 2 min							
												t_c =	6+2 = 8 min							
4	27th to 26th Sts	D	C	2.9	2.9	0.5	i@	9.3 min	2.91	4.2		0.0033	0.013	15.7"	Use	18"				
												V_{18} =	3.4 fps							
												t_{pipe} =	370/60(3.4) = 1.8 min							
												t_c =	9.3+1.8 = 11.1 min							
3	26th to 25th Sts	C	B	2.3	5.2	0.5	i@	11.1 min	2.64	6.9		0.004	0.013	18.2"	Use	21"				
												V_{21} =	4.2 fps							
												t_{pipe} =	395/60(4.2) = 1.6 min							
												t_c =	11.1+1.6 = 12.7							
2	Alley at 25th St to "C" St	B	F	—	9.0	0.5	i@	12.7 min	2.46	11.1		0.0048	0.013	21.4"	Use	24"				
												V_{24} =	4.8 fps							
												t_{pipe} =	230/60(4.8) = 0.8 min							
												t_c =	12.7+0.8 = 13.5 min							

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5	24th + "C" to 24th + alley	E	I	2.0	2.0	0.5	i @ 5.7min		3.69	3.7		0.0043	0.013	14.2"	Use	15"				
												V_{15} =	3.5	fps						
												t_{pipe} =	205	/60(3.5) =		1 min				
												t_c =	5.7	+ 1 =		6.7 min				
9	24th + Alley to 25th + Alley	I	J	1.7	3.7	0.5	i @	6.7min	3.47	6.4		0.003	0.013	18.7"	Use	21"				
												V_{21} =	3.6	fps						
												t_{pipe} =	375	/60(3.6) =		1.7 min				
												t_c =	6.7	+ 1.7 =		8.4 min				
6	25th + "C" Sts to 25th + Alley	F	J	2.8	11.8	0.5	i @	13.5min	2.37	14.0		0.0043	0.013	23.5"	Use	24"				
												V_{24} =	4.7	fps						
												t_{pipe} =	175	/60(4.7) =		0.6 min				
												t_c =	13.5	+ 0.6 =		14.1 min				
8	27th + "C" Sts to 27th + Alley	H	L	2.1	2.1	0.5	i @	5.2min	3.80	4.0		0.0054	0.013	14.1"	Use	15"				
												V_{15} =	3.9	fps						
												t_{pipe} =	200	/60(3.9) =		0.9 min				
												t_c =	5.2	+ 0.9 =		6.1 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
16	27th & "D" Sts to 27th & Alley	P	L	1.2	1.2	0.5	i @ 4	4 min	4.33	2.6		0.0046	0.013	12.3"	Use	15"				
												V_{15} =	3.6	fps						
												t_{pipe} =	160/	60(3.6) =		0.7 min				
												t_c =	4.4 +	0.7 =		5.1 min				
12	27th & Alley to 26th & Alley	L	K	1.2	4.5	0.5	i @	6.1 min	3.6	8.1		0.0035	0.013	19.9"	Use	21"				
												V_{21} =	3.9	fps						
												t_{pipe} =	360/	60(3.9) =		1.5 min				
												t_c =	6.1 +	1.5 =		7.6 min				
7	26th & "C" to 26th & Alley	G	K	1.9	1.9	0.5	i @	5.2 min	3.80	3.6		0.0054	0.013	13.5"	Use	15"				
												V_{15} =	3.9	fps						
												t_{pipe} =	200/	(60)(3.9) =		0.9 min				
												t_c =	5.2 +	0.9 =		6.1 min				
11	26th & Alley to 25th & Alley	K	J	1.3	7.7	0.5	i @	7.6 min	3.28	12.6		0.003	0.013	24.1"	Use	27"				
												V_{27} =	4.3	fps						
												t_{pipe} =	390/	60(4.3) =		1.5 min				
												t_c =	7.6 +	1.5 =		9.1 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: LAL Date: 12/17/91
Checked by: _____ Date: _____

E and 25th Sts - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
10	25th + Alley to 25th + "D"	J	N	1.3	24.5	0.5	i @	14.1 min	2.30	28.2		0.0026	0.013	33.5"	Use	36"				
												V_{36} =	4.8	fps						
												t_{pipe} =	230	/60(4.8) =		0.8 min				
												t_c =	14.1	+ 0.8 =		14.9 min				
20	27th + Alley to 26th + Alley	T	S	1.5	1.5	0.5	i @ 6	2 min	3.58	2.7		0.0033	0.013	13.3"	Use	15"				
												V_{15} =	3.0	fps						
												t_{pipe} =	360	/60(3) =		2 min				
												t_c =	6.2	+ 2.0 =		8.2 min				
15	26th + "D" Sts to 26th + Alley	O	S	2.2	2.2	0.5	i @ 7	8 min	3.24	3.6		0.0031	0.013	15.0"	Use	15"				
												V_{15} =	2.9	fps						
												t_{pipe} =	190	/60(2.9) =		1.1 min				
												t_c =	7.8	+ 1.1 =		8.9 min				
19	26th St + Alley to 25th St + Alley	S	R	1.3	5.0	0.5	i @	8.9 min	3.00	7.5		0.0029	0.013	20.0"	Use	21"				
												V_{21} =	3.6	fps						
												t_{pipe} =	395	/60(3.6) =		1.8 min				
												t_c =	8.9	+ 1.8 =		10.7 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
E and 25th Sts - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
13	24th + "D" Sts to 24th + Alley	M	Q	2.1	2.1	0.5	i @ 10.7 min		2.68	2.8		0.0031	0.013	13.6"	Use	15"				
												V_{15} = 2.9 fps								
												t_{pipe} = 190/60(2.9) =				1.1 min				
												t_c = 10.7 + 1.1 =				11.8 min				
17	24th + Alley to 25th + Alley	Q	R	1.3	3.4	0.5	i @	11.8 min	2.56	4.4		0.0029	0.013	16.4"	Use	18"				
												V_{18} = 3.2 fps								
												t_{pipe} = 360/60(3.2) =				1.9 min				
												t_c = 11.8 + 1.9 =				13.7				
14	25th + "D" Sts to 25th + Alley	N	R	2.7	27.2	0.5	i @	14.9 min	2.21	30.1		0.0031	0.013	33.2"	Use	36"				
												V_{36} = 5.3 fps								
												t_{pipe} = 170/60(5.3) =				0.5 min				
												t_c = 14.9 + 0.5 =				15.4				
18	25th St + Alley		R	1.1	36.7	0.5	i @	15.4 min	2.18	40.0										

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
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Project Sacramento CSO-Phase 3
Computed by DAC Date 12/15/91
Checked by _____ Date _____
E Street and 25th St- Summary

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
	Summary			2	4A		5	4A			10	4A		25	4A		No			
Existing Ø (in)	Reach	Slope (ft/ft)	Length (ft)	Dischage (cfs)	Dia Ø (in)		Dischage (cfs)	Dia Ø (in)			Dischage (cfs)	Dia Ø (in)		Dischage (cfs)	Dia Ø (in)		DI			
8	A to B	0.0026	360	1.5	12		2.3	15			2.8	15		3.6	18					
8	D to C	0.0033	370	1.7	12		2.6	15			3.3	15		4.2	18					
8	C to B	0.004	395	2.9	15		4.3	18			5.3	18		6.9	21					
10	B to F	0.0043	230	4.4	18		6.9	18			8.5	21		11.1	24					
8	E to I	0.0043	205	1.5	12		2.3	12			2.9	15		3.7	15					
8	I to J	0.003	375	2.6	15		4.2	18			5.0	18		6.4	21					
10	F to J	0.0043	175	5.5	18		8.6	21			10.7	24		14.0	24					
8	H to L	0.0054	200	1.6	10		2.5	12			3.1	15		4.0	15					
8	P to L	0.0046	160	1.0	10		1.6	12			2.0	12		2.6	15					
8	L to K	0.0035	360	3.3	15		5.0	18			6.4	21		8.1	21					
8	G to K	0.0054	200	1.5	10		2.3	12			2.8	15		3.6	15					
8	K to J	0.003	390	5.0	18		7.8	21			9.9	24		12.6	27					
12	J to N	0.0026	230	11.1	24		17.3	30			21.6	33		28.2	36					
8	T to S	0.0033	360	1.1	10		1.7	12			2.1	15		2.7	15					
8	O to S	0.0031	190	1.4	12		2.2	15			2.8	15		3.6	15					
	S to R	0.0029	395	3.0	15		4.7	18			5.9	21		7.5	21					
	M to Q	0.0031	190	1.2	10		1.8	12			2.2	15		2.8	15					
	Q to R	0.0029	360	1.7	12		2.7	15			3.4	15		4.4	18					
12	N to R	0.0031	170	12.2	24		18.8	30			23.1	33		30.1	36					
	R	—	—	16.1	—		25.0	—			30.8	—		40.0	—					

Representative Basin No. 2

P and 38th Streets

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: [illegible]
Computed by: [illegible] Date: [illegible]
Checked by: Date:
P and 38th Sts - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.2	1.2	0.5	i @ 5.6 min		1.51	0.91		0.0045	0.013	8.3"	Use	10"				
	Overland	75	0.85	fps			1.5					V_{10} =	2.7	fps						
	Street	175	0.72	fps			4.1													
							5.6	min				t_{pipe} =	160/2.7(60) =			1.0 min				
	Street Slope:	Adverse	street	slope	A to B							t_c =	5.6 + 1 =			6.6 min				
	Assume Slope C to	E RIMS	(21b-12)	-(15c-45)																
	(29.99 - 29.91)/100 =	0.0013'/'																		
	Design Slope:	(15c-42)	-(15c-43)	Inverts																
	(25.71 - 24.97)/160 =	0.0045'/'																		
2		B	E	1.4	2.6	0.5	i @ 8.4 min		1.3	1.7		0.0046	0.013	10.5"	Use	12"				
	Overland	75	0.85	fps			1.5													
	Street	300	0.72	fps			6.9					V_{12} =	3.1	fps						
							8.4	min												
	Street Slope:	Adverse	street slope B to E									t_{pipe} =	200/3.1(60) =			1.1 min				
	Assume Slope C to	E RIMS	(21b-18)	-(15c-45)			t_c from A to B = 6.6 min													
	Slope = 0.0013'/'						t_c from Area 2 > 6.6 min					t_c =	8.4 + 1.1 =			9.5 min				
							8.4 min governing t_c													
	Design Slope:	(15c-43)	-(15c-45)																	
	(24.99 - 24.07)/200 =	0.0046'/'																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSD - Phase 5
Computed by [illegible] Date 12/16/91
Checked by _____ Date _____
[illegible] 8th St's - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5		C	D	1.9	1.9	0.5	i@	9.7 min	1.2	1.1		0.0035	0.013	9.4"	Use	10"				
	Overland	75'	0.85	fps			1.5					V_{10} = 2.4	fps							
	Street	275'	0.56	fps			8.2													
							9.7	min				t_{pipe} = 220/(2.4(60)) =				1.5 min				
	Street Slope	(21b-18)-	(21b-8)	RIMS																
	(29.99-29.81)/220=	0.0008'/'										t_c = 9.7+1.5 =				11.2 min				
	Design Slope:	(21b-18)-	(21b-8)	Inverts																
	(25.77-25.01)/220=	0.0035'/'																		
4		D	E	1.4	3.3	0.5	i@	11.2 min	1.1	1.8		0.0051	0.013	10.5"	Use	12"				
	Overland	75'	0.85	fps			1.5					V_{12} = 3.2	fps							
	Street	175'	0.85	fps			3.4													
							4.9 min					t_{pipe} = 185/3.2(60) =				1 min				
	Street Slope:	(21b-8)	(15c-45)	RIMS																
	(29.81-29.47)/185	= 0.0018'/'										t_c = 11.2+1 =				12.2 min				
	Design Slope	(21b-8)-	(15c-45)	Inverts																
	(25.01-24.07)/185	= 0.0051'/'																		
	t_c from C to D = 11.2 min																			
	t_c from Area 4 = 4.9 min																			
	t_c from C to D governs		∴ t_c = 11.2 min																	

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CIA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/16/91
Checked by _____ Date _____
P and 38th Sts - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3		E	F	0.86	6.8	0.5	i@ 12.2 min		1.0	3.4		0.0032	0.013	14.6"	Use	15"				
	t_c from B to E =	9.5 min		Contributing Area =			1, 2, 3	4, 5				V_{15} =	3.0 fps							
	t_c from D to E =	12.2 min																		
	Note t_c computations	for Area 3										t_{pipe} =	210 / 3(60) =			1.2 min				
	governing t_c = 12.2 min											t_c =	12.2 + 1.2 =			13.4 min				
	Design Slope:	No data.	Interpolate from	(15c-45) -	(5c-30)	E to G														
	(24.06 - 22.73)/420	= 0.0032 '/'																		
6		F	G	4.5	11.3	0.5	i @ 13.4 min		0.99	5.6		0.0032	0.013	17.6"	Use	18"				
	Overland	75'	0.85				1.5					V_{18} =	3.4 fps							
	Street	700'	1.3				9.0													
							10.5 min					t_{pipe} =	220 / 3.4(60) =			1.1 min				
	Street Slope:	(15c-40)	- (15c-44)	RIM																
	(31.16 - 28.70)/600	= 0.0041 '/'										t_c =	13.4 + 1.1 =			14.5 min				
	t_c from E to F	= 13.4 min																		
	t_c from Area 6	= 10.5 min																		
	governing t_c =	13.4 min																		
	Design Slope:	Assume same as		(15c-45) to	(5c-30)	E to G														
	Slope = 0.0032 '/'																			

Computation Form
Drainage and Storm Sewer Design
Rational Method

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Project Sacramento CSO-Phase 3
Computed by LAC Date 12/16/91
Checked by _____ Date _____
Pand 38th Sts - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
7		G	H	10.7	22.0	0.5	i@	14.5min	0.94	10.3		0.0034	0.013	21.9"	Use	24"				
	Overland	150	0.85				2.9					V_{24} = 4.2 fps								
	Alley	700	1.3				9.0													
							11.9	min				t_{pipe} = 210/4.2(60) =				0.8 min				
	Alley Slope: No data. Assume same as (15c-40) to (15c-44) & MS																			
	(31.10-28.7)/600 = 0.0041'											t_c = 14.5+0.8 = 15.3 min								
	t_c from F to G = 14.5min																			
	t_c from Area 7 = 11.9min																			
	governing t_c = 14.5min																			
	Design Slope:	(15c-30)	-(15c-29)																	
	(22.73-22.02)/210	= 0.0034'/'																		
8		H	I	2.7	24.7	0.5	i@	15.3min	0.91	11.2		0.0034	0.013	22.5"	Use	24"				
	No t_c computation made for Area 8											V_{24} = 4.2 fps								
	governing t_c from G to H = 15.3min																			
												t_{pipe} =	215/4.2(60) =			0.9 min				
	Design Slope:	(15c-29)	-(15c-81)	Assume same as H to J																
	(22.02-20.52)/440	= 0.0034'/'										t_c = 15.3+0.9 = 16.2 min								
9		I	J	9.7	34.1	0.5	i@	16.2min	0.9	15.3		0.0034	0.013	25.3"	Use	27"				
	No t_c computation made for Area 9											V_{29} = 4.6 fps								
	governing t_c from H to I = 16.2min																			
												t_{pipe} = 225/60(4.6) =				0.8 min				
	Design slope:	Assume	same as	H to J = 0.0034'/'								t_c = 16.2+0.8 = 17.0min								
At J				—	34.1	0.5	i@	17.0	0.88	15										

Computation Form
Drainage and Storm Sewer Design
Rational Method

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i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO-Phase 3
Computed by DAC Date 12/16/91
Checked by _____ Date _____
P and 38th Sts - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.2	1.2	0.5	i @ 5.6 min		2.3	1.4		0.0045	0.013	9.8"	Use	10"				
												V_{10} = 2.7 fps								
												t_{pipe} = 160/2.7(60) =				1 min				
												t_c = 5.6 + 1 = 6.6 min								
2		B	E	1.4	2.6	0.5	i @ 8.4 min		1.95	2.5		0.0046	0.013	12.1"	Use	15"				
												V_{15} = 3.6 fps								
												t_{pipe} = 200/3.6(60) =				0.9 min				
												t_c = 8.4 + 0.9 = 9.3 min								
5		C	D	1.9	1.9	0.5	i @ 9.7 min		1.78	1.7		0.0035	0.013	11.1"	Use	12"				
												V_{12} = 2.7 fps								
												t_{pipe} = 220/2.7(60) =				1.4 min				
												t_c = 9.7 + 1.4 = 11.1 min								
6		D	E	1.4	3.3	0.5	i @ 11.1 min		1.67	2.8		0.0051	0.013	12.4"	Use	15"				
												V_{15} = 3.8 fps								
												t_{pipe} = 185/3.8(60) =				0.8 min				
												t_c = 10.8 + 0.8 = 11.6 min								

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CIA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/16/91
Checked by _____ Date _____
Pond 38th sts - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3		E	F	0.86	6.8	0.5	iQ	11.6 min	1.63	5.5		0.0032	0.013	17.5"	Use	18"				
												V_{18} = 3.4 fps								
												t_{pipe} = 210/3.4(60) =				1 min				
												t_c = 11.6 + 1 = 12.6 min								
6		F	G	4.5	11.3	0.5	iQ 12.6 min		1.56	8.8		0.0032	0.013	20.8"	Use	21"				
												V_{21} = 3.7 fps								
												t_{pipe} = 220/3.7(60) =				1 min				
												t_c = 12.6 + 1 = 13.6 min								
7		G	H	10.7	22.0	0.5	iQ	13.6 min	1.5	16.5		0.0034	0.013	26.1"	Use	27"				
												V_{27} = 4.6 fps								
												t_{pipe} = 210/4.6(60) = 0.8 min								
												t_c = 13.6 + 0.8 = 14.4 min								
8		H	I	2.7	24.7	0.5	iQ	14.4 min	1.44	17.8		0.0034	0.013	26.8"	Use	27"				
												V_{27} = 4.6 fps								
												t_{pipe} = 215/60(4.6) = 0.8 min								
												t_c = 14.4 + 0.8 = 15.2 min								

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CIA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/16/91
Checked by _____ Date _____
Pond 38th Sts - 5 year

Drainage Area Number	Location			A		C	Time of Concentration		I	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
9		I	J	9.7	34.1	0.5	i @ 15.2 min		1.39	23.7		0.0034	0.013	29.9	Use	30"				
												V_{30} = 4.9 fps								
												t_{pipe} = 225/60(4.9) = 0.8 min								
												t_c = 15.2 + 0.8 = 16 min								
At J				—	34.1	0.5	i @ 16.0 min		1.37	23.4										

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CIA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento - CSO - Phase 3
Computed by LAC Date 12/16/91
Checked by _____ Date _____
P and 38th Sts - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope B/1	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.2	1.2	0.5	i@ 5.6 min		2.9	1.7		0.0045	0.013	10.6"	Use	12"				
												V_{12} =	3.1	fps						
												t_{pipe}	160/3.1(60) =		0.9 min					
												t_c =	5.6 + 0.9 =		6.5 min					
2		B	E	1.4	2.6	0.5	i@ 8.4 min		2.4	3.1		0.0046	0.013	13.2"	Use	15"				
												V_{15} =	3.6	fps						
												t_{pipe} =	200/3.6(60) =		0.9 min					
												t_c =	6.5 + 0.9 =		7.4 min					
5		C	D	1.9	1.9	0.5	i@ 9.7 min		2.2	2.1		0.0035	0.013	12.0"	Use	12"				
												V_{12} =	2.7	fps						
												t_{pipe} =	220/2.7(60) =		1.4 min					
												t_c =	9.7 + 1.4 =		11.1 min					
4		D	E	1.4	3.3	0.5	i@	11.1 min	2.06	3.4		0.0051	0.013	13.4"	Use	15"				
												V_{15} =	3.8	fps						
												t_{pipe} =	185/3.8(60) =		0.8 min					
												t_c =	11.1 + 0.8 =		11.9 min					

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CIA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall Intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: dHC Date: 12/16/91
Checked by: _____ Date: _____
P and 38th Sts - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3		E	F	0.86	6.8	0.5	i@	11.9 min	1.99	6.8		0.0032	0.013	18.9"	Use	21"				
												V_{21} = 3.7 fps								
												t_{pipe} = 210/3.7(60) =				0.9 min				
												t_c = 11.9 + 0.9 =				12.8 min				
6		F	G	4.5	11.3	0.5	i@	12.8 min	1.91	10.8		0.0032	0.013	22.5"	Use	24"				
												V_{24} = 4.1 fps								
												t_{pipe} = 220/4.1(60) =				0.9 min				
												t_c = 11.3 + 0.9 =				12.2 min				
7		G	H	10.7	22.0	0.5	i@	12.2 min	1.97	21.7		0.0034	0.013	28.9"	Use	30"				
												V_{30} = 4.9 fps								
												t_{pipe} = 210/4.9(60) =				0.7 min				
												t_c = 12.2 + 0.7 =				12.9 min				
8		H	I	2.7	24.4	0.5	i@	12.9 min	1.90	23.2		0.0034	0.013	29.6"	Use	30"				
												V_{30} = 4.9 fps								
												t_{pipe} = 215/4.9(60) =				0.7 min				
												t_c = 12.9 + 0.7 =				13.6 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO Phase 3
Computed by JAK Date 12/16/91
Checked by _____ Date _____
P and 38th Sts - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
9		I	J	9.7	34.1	0.5	i@	13.6 min	1.84	31.4		0.0034	0.013	33.2	Use	36"				
												V_{36} = 5.5 fps								
												t_{pipe} = 225/5.5(60) =				0.7 min				
												t_c = 13.6 + 0.7 =				14.3 min				
A+J				—	34.1	0.5	i @ 14.3 min		1.78	30.3										

Computation Form
Drainage and Storm Sewer Design
Rational Method

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/16/91
Checked by _____ Date _____
Pand 38th Sts - 25 year

$Q = CIA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Drainage Area Number	Location			A		C	Time of Concentration		I	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
1		A	B	1.2	1.2	0.5	i @ 5.6 min		3.71	2.2		0.0045	0.013	11.6"	Use	12"				
												$V_{1/2}$ = 3.05 fps								
												t_{pipe} = 160/3.05(60) = 0.9 min								
												t_c = 5.6 + 0.9 = 6.5 min								
2		B	E	1.4	2.6	0.5	i @ 8.4 min		3.11	4.0		0.0046	0.013	14.5"	Use	15"				
												$V_{1/5}$ = 3.6 fps								
												t_{pipe} = 200/3.6(60) = 0.9 min								
												t_c = 4 + 0.9 = 4.9 min								
5		C	D	1.9	1.9	0.5	i @ 9.7 min		2.8	2.7		0.0035	0.013	13.2"	Use	15"				
												$V_{1/5}$ = 3.1 fps								
												t_{pipe} = 220/3.1(60) = 1.2 min								
												t_c = 9.7 + 1.2 = 10.9 min								
4		D	E	1.4	3.3	0.5	i @ 10.9 min		2.66	4.4		0.0051	0.013	14.7"	Use	15"				
												$V_{1/5}$ = 3.8 fps								
												t_{pipe} = 185/3.8(60) = 0.8 min								
												t_c = 10.9 + 0.8 = 11.7 min								

Computation Form
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Project Sacramento CSO - Phase 3
Computed by LAC Date 12/16/91
Checked by _____ Date _____
P and 38th Sts - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3		E	F	0.86	6.8	0.5	10	11.7 min	2.57	8.7		0.0032	0.013	20.7"	Use	21"				
												V_{21} = 3.7 fps								
												t_{pipe} = 210/3.7(60) = 0.9 min								
												t_c = 11.7 + 0.9 = 12.6 min								
6		F	G	4.5	11.3	0.5	10	12.6 min	2.47	14.0		0.0032	0.013	24.8"	Use	27"				
												V_{27} = 4.4 fps								
												t_{pipe} = 220/4.4(60) = 0.8 min								
												t_c = 12.6 + 0.8 = 13.4 min								
7		G	H	10.7	22.0	0.5	10	13.4 min	2.38	26.2		0.0034	0.013	31.0"	Use	33"				
												V_{33} = 5.2 fps								
												t_{pipe} = 210/5.2(60) = 0.7 min								
												t_c = 13.4 + 0.7 = 14.1 min								
8		H	I	2.7	24.7	0.5	10	14.1 min	2.30	28.4		0.0034	0.013	32.0"	Use	33"				
												V_{33} = 5.2 fps								
												t_{pipe} = 215/5.2(60) = 0.7 min								
												t_c = 14.1 + 0.7 = 14.8 min								

Computation Form
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V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 12/16/91
Checked by _____ Date _____
P and 38th Sts - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
9		I	J	9.7	34.1	0.5	i@	14.8 min	2.22	37.9		0.0034	0.013	35.6"	Use	36"				
												V_{36} =	5.5	fps						
												t_{pipe} =	225/5.5(60) =			0.7 min				
												t_c =	14.8	+0.7	= 15.5 min					
At J				—	34.1	0.5	i@	15.5 min	2.2	37.5										

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Computation Form
Drainage and Storm Sewer Design
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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/6/91
Checked by _____ Date _____
Pond 38th Sts - Summary

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
	Summary			2	YR		5	YR			10	YR		25	YR					
	Reach	Slope (ft/ft)	Length (ft)	Discharge (cfs)	Dia ϕ (in)		Discharge (cfs)	Dia ϕ (in)			Discharge (cfs)	Dia ϕ (in)		Discharge (cfs)	Dia ϕ (in)					
Existing ϕ																				
8	A to B	0.0045	160	0.91	10		1.4	10			1.7	12		2.2	12					
8	B to E	0.0046	200	1.7	12		2.5	15			3.1	15		4.0	15					
8	C to D	0.0035	220	1.1	10		1.7	12			2.1	12		2.7	15					
8	D to E	0.0051	185	1.8	12		2.8	15			3.4	15		4.4	15					
10	E to F	0.0032	210	3.4	15		5.5	18			6.8	21		8.7	21					
10	F to G	0.0032	220	5.6	18		8.8	21			10.8	24		14.0	27					
10	G to H	0.0034	210	10.3	24		16.5	27			21.7	30		26.2	33					
12	H to I	0.0034	215	11.2	24		17.8	27			23.2	30		28.4	33					
12	I to J	0.0034	225	15.3	27		23.7	30			31.4	36		37.9	36					
	At J	—	—	15	—		23.4	—			30.3	—		37.5	—					

Representative Basin No. 3

14th Street and 2nd Avenue

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO Phase 3
Computed by: LAC Date: 12/12/91
Checked by: _____ Date: _____
14th Street and 2nd Ave. /2 year design

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1	13th Street to Markham Wy	A	B	1.6	1.6	0.5	i @	6.2 min	1.42	1.14		0.0038	0.013	9.4"	Use	10"				
	Overland	150'	0.85	fps			2.9						V_{10} =	2.5	fps					
	Street	200'	1.0	fps			3.3													
							6.2	min					t_{pipe} =	375/60(2.5) =			2.5	min		
													t_c =	6.3 + 2.5 =		8.8	min			
	Street Slope:	(26a-52)	-(26d-29)		RIMS															
	(13.55-12.56)/375	= 0.0026	'/'																	
	Design Slope:	(26a-52)	-(26d-29)		Inverts															
	(9.36-7.95)/375 =	0.0038'/'																		
1+2	Markham Way	B	R	2.8	4.4	0.5	i @ 8.8 min		1.24	2.7		0.0037	0.013	13.0	Use	15"				
	Street Slope:	Use same as A to B											V_{10} =	3.2 fps						
	Slope = 0.0026'/'																			
													t_{pipe} =	225/60(3.2) =			0.9 min			
	Design Slope:	(26d-29)	-(26d-36)										t_c =	8.8 + 0.9 =		9.7	min			
	(7.95-7.12)/225	= 0.0037	'/'																	
10	Markham Way	Q	R	7.2	7.2	0.5	i @	9.0 min	1.2	4.3		0.0105	0.013	12.8'	Use	15"				
	Overland	150	0.85	fps			2.9						V_{15} =	5.4	fps					
	Street	375	1.02	fps			6.1													
							9.0						t_{pipe} =	120/5.4(60) =			0.4 min			
	Street Slope:	(26a-52)	-(26d-29)		use slope from A to B			RIMS												
	Slope =	0.0026'/'											t_c =	9 + 0.4 =		9.4	min			
	Design Slope:	(26d-37)	-(26d-36)																	
	(8.37-7.11)/120	= 0.0105	'/'																	

Computation Form
Drainage and Storm Sewer Design
Rational Method

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Project Sacramento CSO Phase 3
Computed by LAC Date 12/12/91
Checked by _____ Date _____
14th Street and 2nd Ave / 2 year design

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
	Markham Wy to 2nd Ave	R	N	—	11.6	0.5	i @ 9.7 min		1.2	7.0		0.0048	0.013	17.7"	Use	18"				
	Contributing Drainage Areas			1, 2, and 10																
											9.7 min		>9.4 min ∴		Use	t_c = 9.7 min to size Rt to N				
	Design Slope:	(26d-36)	-(26a-50)	Assume	invert		+8" IN S = 3.22													
	7.1-(-3.22)/810 =	0.0048'/'											V_{18} =	4.1 fps						
													t_{pipe} =	810/60(4.1) =		3.3 min				
													t_c =	9.7+3.3 =		13 min				
6	Markham Wy to 12th St	K	E	0.6	0.6	0.5	i @	2.9 min	1.9	0.6		0.0044	0.013	7.2"	Use	8"				
	Overland	150	0.85	fps			2.9													
													V_8 =	2.3 fps						
	Design Slope:	Use (26a-20) to		(26a-21)									t_{pipe} =	510/60(2.3) =		3.7 min				
	(7.52-6.03)/340	= 0.0044'/'											t_c =	2.9+3.7 =		6.6 min				
5	Markham Wy 12th St	D	E	3.1	3.1	0.5	i @	10.1 min	1.14	1.8		0.0147	0.013	8.6"	Use	10"				
	Overland	150	0.85	fps			2.9						V_{10} =	4.9 fps						
	Street	430	1.0	fps			7.2													
							10.1	min					t_{pipe} =	150/4.9(60) =		0.5 min				
	Street Slope:																			
	No data - assume velocity = 1 fps												t_c =	10.1+0.5 =		10.6 min				
	Design slope:	(26a-28)	-(26a-27)		inverts															
	(8.91-6.71)/150	= 0.0147'/'																		

Computation Form
Drainage and Storm Sewer Design
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Project Sacramento CSD-Phase 3
Computed by LAC Date 12/12/91
Checked by _____ Date _____
14th Street and 2nd Ave / 2 year design

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5, 6	12th Street	E	F		3.7	0.5	i @ 10	6 min	1.1	2.3		0.0042	0.013	12.0"	Use	12"				
	Contributing Drainage Areas			5 and 6							10.6 min	>6.6 min	Use 10.6 to size E to F							
	Design Slope	(26a-27)	- (26a-21)									V_{12} =	2.9 fps							
	(6.66-5.55)/265 =	0.0042	1/1									t_{pipe} =	265/60(2.9) =	1.5 min						
												t_c =	10.6 + 1.5 =	12.1 min						
7	Castro Way to 12th Street	L	F		1.4	0.5	i @	4.5 min	1.7	1.2		0.0044	0.013	9.3"	Use	10"				
	Overland	100	0.85 fps				2.0													
	Street	150	1.0 fps				2.5						V_{10} =	2.7 fps						
							4.5	min												
	Street Slope:	No data	Assume	1 fps									t_{pipe} =	470/2.7(60) =	2.9 min					
	Design Slope:	Use	(26a-20) to (26a-21)									t_c =	4.5 + 2.9 =	7.4 min						
	(7.92 - 6.03)/340	= 0.0044	1/1																	
5, 6 + 7	12th St to 2nd Ave	F	G		5.1	0.5	i @	12.1 min	1.0	2.6		0.0019	0.013	14.5"	Use	15"				
	Contributing	drainage	areas	5, 6 and 7																
												12.1 min >	7.4 min	Use 12.1 to size F to I						
	Design Slope:	(26a-21)	- (26a-17)										V_{15} =	2.3 fps						
	(5.51 - 5.26)/130 =	0.0019	1/1										t_{pipe}	130/2.3(60) =	0.9 min					
													t_c =	12.1 + 0.9 =	13.0 min					

Computation Form
Drainage and Storm Sewer Design
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Project Sacramento CSO - Phase 3
Computed by JAC Date 12/12/91
Checked by _____ Date _____
14th Street and 2nd Ave / 2 year design

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3	Markham Wy to 2nd Ave	C	H	1.3	1.3	0.5	i @ 9.1 min		1.2	0.8		0.0047	0.013	7.9"	Use 8"					
	Overland	120'	0.85	fps			2.4					V_8 = 2.4 fps								
	Street	400	1.0	fps			6.7													
							9.1	min				t_{pipe} = 640/2.4(60) = 4.4 min								
	Street Slope -	No Data	assume	1.0 fps																
	Design Slope:	(26d-28)	-(26a-18)									t_c = 9.1 + 4.4 = 13.5 min								
	(9.09-6.09)/640	= 0.0047	'/'																	
8	2nd Ave	G	H	7.1	12.2	0.5	i @	13.0 min	1.0	6.1		0.003	0.013	18.4"	Use 21"					
												Contributing drainage areas 5, 6 + 7								
	Overland	120	0.85	fps			2.4					t_c from 5, 6, 7 > t_c 8								
	Street	550	0.9	fps			10.2					13.0 min > 12.6 min Use 13.0 to size G to H								
							12.6	min	Area 8											
	Street Slope:	(26a-28)	-(26a-17)		RIMS															
	(13.61 - 12.46)/	550 =	0.0021'/'									V_{21} = 3.6 fps								
	Design Slope	(26a-17)	-(26a-18)		Inverts															
	(5.26 - 4.79)/155	= 0.0030'/'										t_{pipe} = 155/3.6(60) = 0.7 min								
												t_c = 13.0 + 0.7 = 13.7 min								
At "H"	2nd Ave	H	M		13.5	0.5	i @ 13.7 min		0.98	6.6		0.0042	0.013	17.8"	Use 21"	to match G to H				
	Contributing Drainage Areas 3, 5, 6, 7 and 8																			
	t_c C to H =	13.5 min										V_{21} = 4.3 fps								
	t_c G to H =	13.7 min	Use	t_c = 13.7 min as governing t_c																
	Design Slope:	(26a-18)	-(26a-49)									t_{pipe} = 140/4.3(60) = 0.5 min								
	(4.79-4.20)/140 =	0.0042'/'										t_c = 13.7 + 0.5 = 14.2 min								

Computation Form
Drainage and Storm Sewer Design
Rational Method

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/12/91
Checked by _____ Date _____
14th Street and 2nd Ave / 2 year design

Q = CiA

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Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11	Beverly Way to 13th St	I	J	3.4	3.4	0.5	i @ 8.4 min		1.3	2.2		0.0039	0.013	11.9"	Use	12"				
	Overland	100'	0.85 fps				2.0					V_{12} = 2.8 fps								
	Street	500	1.3 fps				6.4													
							8.4	min												
												t_{pipe} = 200/2.8(60) =				1.2 min				
	Street Slope - Assume same slope as (26a-91) to (26a-46) RIMS																			
	(13.62-11.84)/400 =	0.0045'/'										t_c = 8.4+1.2 =				9.6 min				
	Design Slope:	(26a-48)	- (26a-46)																	
	(5.34-4.32)/260	= 0.0039'/'																		
12	13th St	J	M	5.2	8.6	0.5	i @	9.6 min	1.2	5.2		0.0039	0.013	16.5"	Use	18"				
	t_c for Area 12																			
	Overland	150	0.85 fps				2.9													
	Alley	500	1.3 fps				6.4					t_c Area 12 = 9.3 min								
							9.3 min					t_c I to J = 9.6 min								
	Street Slope:	Assume same slope as used for Area 11										9.6 > 9.3 use 9.6 min								
	Slope: 0.0045'/'																			
												V_{18} = 3.7 fps								
	Design Slope:	Assume same slope as used for Area 11																		
	Slope = 0.0039	'/'										t_{pipe} = 340/3.7(60) =				1.5 min				
												t_c = 9.6 + 1.5 =				11.1 min				

Computation Form
Drainage and Storm Sewer Design
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Project Sacramento CSO - Phase 3
Computed by LAC Date 12/12/91
Checked by _____ Date _____
14th Street and 2nd Ave / 2 year design

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
At "M"	2nd Ave	M	N		22.1	0.5	i @ 14.2 min		0.96	10.6		0.0009	0.013	28.1"	Use	30"				
	t_c J to M = 11.1 min					Contributing Drainage Areas						3, 5, 6, 7, 8, 11, 12								
	t_c H to M = 14.2 min																			
	Use t_c = 14.2 min as	governing	t_c									V_{30} = 2.6 fps								
	Design Slope	(26a-49)	-(26a-50)									t_{pipe} = 190/2.6(60) = 1.2 min								
	(3.4-3.22)/190 =	0.0009																		
												t_c = 14.2 + 1.2 = 15.4 min								
4	Markham Wy to 2nd Ave	S	P	0.63	0.63	0.5	i @ 3.2 min		1.9	0.6		0.0047	0.013	7.1"	Use 8"					
	Overland	75'	0.85	fps			1.5					V_8 = 2.4 fps								
	Street	100'	1.0	fps			1.7													
							3.2 min					t_{pipe} = 775/2.4(60) = 5.4 min								
	Street slope:	No data	Assume	velocity of 1 fps																
												t_c = 3.2 + 5.4 = 8.6 min								
	Design Slope:	(26d-113)	-(26a-59)																	
	(8.03-4.38)/775 =	0.0047 '/'																		
9	2nd Ave	N	P	10.0	43.7	0.5	i @	15.4 min	0.91	19.9		0.002	0.013	30.9"	Use	33"				
	t_c M to N	15.4 min				Contributing Drainage Areas					1, 2, 3, 5, 6, 7, 8, 9, 10, 11, 12									
	t_c R to N	13.0 min																		
	t_c = 15.4 min	governs										V_{33} = 4 fps								
	No calculations	for t_c	in Area 9	; upstream basins				will govern				t_{pipe} = 350/4(60) = 1.5 min								
	Design slope:	(26a-50)	-(26a-59)		inverts							t_c = 15.4 + 1.5 = 16.9 min								
	(3.98-2.48)/350	= 0.002 '/'																		
			D	-	43.7	0.5	i @ 16.9 min		0.88	19.2										

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1 of

Computation Form
Drainage and Storm Sewer Design
Rational Method

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Project Sacramento CSO - Phase 3
Computed by JAC Date 12/16/91
Checked by _____ Date _____
14th Street and 2nd Ave - 5year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.6	1.6	0.5	i @ 6	2 min	2.24	1.8		0.0038	0.013	11.1"	Use	12"				
												V_{12} = 2.8 fps								
												t_{pipe} = 375/60(2.8) = 2.2 min								
												t_c = 6.2 + 2.2 = 8.4 min								
2		B	R	2.8	4.4	0.5	i @	8.4 min	1.95	4.3		0.0037	0.013	15.5"	Use	18"				
												V_{18} = 3.6 fps								
												t_{pipe} = 225/60(3.6) = 1 min								
												t_c = 8.4 + 1 = 9.4 min								
10		Q	R	7.2	7.2	0.5	i @ 9	0 min	1.87	6.7		0.0105	0.013	15.1"	Use	18"				
												V_{18} = 6.1 fps								
												t_{pipe} = 120/(60)(6.1) = 0.3 min								
												t_c = 9.0 + 0.3 = 9.3 min								
1,2+10		R	N	—	11.6	0.5	i @	9.4 min	1.82	10.6		0.0048	0.013	20.7"	Use	21"				
												V_{21} = 4.6 fps								
												t_{pipe} = 810/60(4.6) = 2.9 min								
												t_c = 9.4 + 2.9 = 12.3 min								

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Computation Form
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Project Sacramento CSO - Phase 3
Computed by LHC Date 12/16/91
Checked by _____ Date _____
14th Street and 2nd Ave - 5year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
6		R	E	0.6	0.6	0.5	i@2.9min		3.24	0.97		0.0044	0.013	8.6"	Use 10"					
												V_{10} = 2.7 fps								
												t_{pipe} = 510/(60)(2.7) = 3.1 min								
												t_c = 2.9 + 3.1 = 6 min								
5		D	E	3.1	3.1	0.5	i@	10.1 min	1.73	2.7		0.0147	0.013	10.1"	Use 12"					
												V_{12} = 5.5 fps								
												t_{pipe} = 150/60(5.5)								
												t_c = 10.1 + 0.5 = 10.6 min								
5+6		E	F	—	3.7	0.5	i@	10.6 min	1.7	3.1		0.0042	0.013	13.4"	Use 15"					
												V_{15} = 3.4 fps								
												t_{pipe} = 265/60(3.4) = 1.3 min								
												t_c = 10.6 + 1.3 = 11.9 min								
7		L	F	1.4	1.4	0.5	i@	4.5 min	2.6	1.8		0.0044	0.013	10.8"	Use 12"					
												V = 3.0 fps								
												t_{pipe} = 470/(60)(3) = 2.6 min								
												t_c = 4.5 + 2.6 = 7.1 min								

Computation Form
Drainage and Storm Sewer Design
Rational Method

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/16/91
Checked by _____ Date _____
14th Street and 2nd Ave

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Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5,6,7		F	G	—	5.1	0.5	i@	11.9 min	1.61	4.1		0.0019	0.013	17.2"	Use	18"				
												V_{18} =	2.6	fps						
												t_{pipe} =	130	/(60)	(2.6)	= 0.8 min				
												t_c =	11.9	+ 0.8 =	12.7	min				
3		C	H	1.3	1.3	0.5	i@ 9.1	min	1.8	1.2		0.0047	0.013	9.2"	Use	10"				
												V_{10} =	2.8	fps						
												t_{pipe} =	640	/(60)	(2.8)	= 3.8 min				
												t_c =	9.1	+ 3.8 =	12.9	min				
8		G	H	7.1	12.2	0.5	i@	12.7 min	1.56	9.5		0.003	0.013	21.7"	Use	24"				
												V_{24} =	4.0	fps						
												t_{pipe} =	155	/(60)	(4) =	0.6 min				
												t_c =	12.7	+ 0.6 =	13.3	min				
at "H"		H	M		13.5	0.5	i@	13.3	1.52	10.3		0.0042	0.013	21.0"	Use	24"	to match G to H			
												V_{24} =	4.7	fps						
												t_{pipe} =	140	/(60)	(4.7)	= 0.5 min				
												t_c =	13.3	+ 0.5 =	13.8	min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CIA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: LAC Date: 12/16/91
Checked by: _____ Date: _____

14th Street and 2nd Ave - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11		I	J	3.4	3.4	0.5	i@	8.4 min	1.95	3.3		0.0039	0.013	13.9"	Use 15"					
												V_{12} =	3.3	fps						
												t_{pipe} =	200	/60(3.3) =	1 min					
												t_c =	8.4	+1 =	9.4 min					
12		J	M	5.2	8.6	0.5	i@	9.4 min	1.82	7.8		0.0039	0.013	19.2"	Use 21"					
												V_{21} =	4.1	fps						
												t_{pipe} =	340	/60(4.1) =	1.4 min					
												t_c =	9.4	+1.4 =	10.8 min					
AE "M"		M	N	–	22.1	0.5	i@	13.8 min	1.48	16.4		0.0009	0.013	33.3"	Use 36"					
												V_{36} =	2.8	fps						
												t_{pipe} =	190	/(60)(2.8) =	1.1 min					
												t_c =	13.8	+1.1 =	14.9 min					
4		S	P	0.63	0.63	0.5	i@	3.2 min	3.12	0.98		0.0047	0.013	8.5"	Use 10"					
												V_{10} =	2.8	fps						
												t_{pipe} =	775	/60(2.8) =	4.6 min					
												t_c =	3.2	+ 4.6 =	7.8 min					

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Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CIA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: dAc Date: 12/10/91
Checked by: _____ Date: _____
14th Street and 2nd Ave - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
9		N	P	10.0	43.7	0.5	i @	14.9 min	1.4	30.6		0.002	0.013	36.3	Use	42"				
												V_{42} =	4.7 fps							
												t_{pipe} =	350/60(4.7) =			1.2 min				
												t_c =	14.9 + 1.2 =			16.1 min				
			P	–	43.7	0.5	i @ 16.1 min		1.37	29.9										

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CIA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: SAC Date: 12/17/91
Checked by: _____ Date: _____

14th Street and 2nd Ave - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.6	1.6	0.5	i@	6.2 min	2.8	2.2		0.0038	0.013	12.0"	Use 12"					
												V_{12} = 2.8 fps								
												t_{pipe} = 375/(60)(2.8)								
												t_c = 6.2 + 2.2 = 8.4 min								
2		B	R	2.8	4.4	0.5	i@	8.4 min	2.43	5.3		0.0037	0.013	16.8"	Use 18"					
												V_{18} = 3.6 fps								
												t_{pipe} = 225/60(3.6) = 1 min								
												t_c = 8.4 + 1 = 9.4 min								
10		Q	R	7.2	7.2	0.5	i@	9.0 min	2.33	8.4		0.0105	0.013	16.4"	Use 18"					
												V_{18} = 6.1 fps								
												t_{pipe} = 120/60(6.1) = 0.3 min								
												t_c = 9.0 + 0.3 = 9.3 min								
1,2+10		R	N	–	11.6	0.5	i@	9.4 min	2.26	13.1		0.0048	0.013	22.4"	Use 24"					
												V_{24} = 5 fps								
												t_{pipe} = 810/60(5) = 2.7 min								
												t_c = 9.4 + 2.7 = 12.1 min								

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CIA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall Intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LHC Date 12/17/91
Checked by _____ Date _____
14th Street and 2nd Ave - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
6		K	E	0.6	0.6	0.5	i @	2.9 min	4.26	1.3		0.0044	0.013	9.6"	Use 10"					
												V_{10} =	2.7 fps							
												t_{pipe} =	510/60(2.7) =			3.1 min				
												t_c =	2.9 + 3.1 =			6 min				
5		D	E	3.1	3.1	0.5	i @	10.1 min	2.15	3.3		0.0147	0.013	10.8"	Use 12"					
												V_{12} =	5.5 fps							
												t_{pipe} =	150/60(5.5) =			0.5 min				
												t_c =	10.1 + 0.5 =			10.6 min				
5+6		E	F	-	3.7	0.5	i @	10.6 min	2.11	3.9		0.0042	0.013	14.6"	Use 15"					
												V_{15} =	3.4 fps							
												t_{pipe} =	265/60(3.4) =			1.3 min				
												t_c =	10.6 + 1.3 =			11.9 min				
7		L	F	1.4	1.4	0.5	i @	4.5 min	3.3	2.3		0.0044	0.013	11.9"	Use 12"					
												V_{12} =	3.0 fps							
												t_{pipe} =	470/60(3) =			2.6 min				
												t_c =	4.5 + 2.6 =			7.1 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CIA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSU - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
14th Street and 2nd Ave - 10year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5,6,7		F	G	–	5.1	0.5	i@	11.9min	1.99	5.1		0.0019	0.013	18.7"	Use	21"				
												V_{21} =	2.9	fps						
												t_{pipe} = 130/60(2.9) =				0.7 min				
												t_c = 11.9 + 0.7 = 12.6 min								
3		C	H	1.3	1.3	0.5	i@	9.1min	2.31	1.5		0.0047	0.013	10.8"	Use	10"				
												V_{10} =	2.8	fps						
												t_{pipe} = 640/60 (2.8) =				3.8 min				
												t_c = 9.1 + 3.8 = 12.9 min								
8		G	H	7.1	12.2	0.5	i@	12.6min	1.93	11.8		0.003	0.013	23.5"	Use	24"				
												V_{24} =	4.0	fps						
												t_{pipe} = 155/60(4) =				0.6 min				
												t_c = 12.6 + 0.6 = 13.2 min								
at "H"		H	M	–	13.5	0.5	i@	13.2min	1.88	12.7		0.0042	0.013	22.7"	Use	24"				
												V_{24} =	4.8	fps						
												t_{pipe} = 140/60(4.8) =				0.5 min				
												t_c = 13.2 + 0.5 = 13.7 min								

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CIA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO Phase 3
Computed by: LAC Date: 12/17/91
Checked by: _____ Date: _____
14th Street and 2nd Ave - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11		I	J	3.4	3.4	0.5	i @	8.4 min	2.43	4.1		0.0039	0.013	15.1"	Use 18"					
												V_{18} = 3.7 fps								
												t_{pipe} = 200/60(3.7) = 0.9 min								
												t_c = 8.4 + 0.9 = 9.3 min								
12		J	M	5.2	8.6	0.5	i @	9.3 min	2.28	9.8		0.0039	0.013	20.9"	Use 21"					
												V_{21} = 4.1 fps								
												t_{pipe} = 340/60(4.1) = 1.4 min								
												t_c = 9.3 + 1.4 = 10.7 min								
A "M"		M	N	-	22.1	0.5	i @	13.7 min	1.83	20.2		0.0009	0.013	36.1"	Use 42"					
												V_{42} = 3.1 fps								
												t_{pipe} = 190/60(3.1) = 1 min								
												t_c = 13.7 + 1 = 14.7 min								
4		S	P	0.63	0.63	0.5	i @	3.2 min	4.08	1.3		0.0047	0.013	9.5"	Use 10"					
												V_{10} = 2.8 fps								
												t_{pipe} = 775/60(2.8) = 4.6 min								
												t_c = 3.2 + 4.6 = 7.8 min								

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CIA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LHC Date
Checked by Date
14th Street and 2nd Ave - 10year

Drainage Area Number	Location			A		C	Time of Concentration		I	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
9		N	P	10.0	43.7	0.5	i @	14.7min	1.75	38.2		0.002	0.013	39.4"	Use	42"				
												V_{42} = 4.7 fps								
												t_{pipe} = 350/(60)(4.7) = 1.2 min								
												t_c = 14.7+1.2 = 15.9 min								
		P		-	43.7	0.5	i @ 15.9min		1.69	36.9										

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CIA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
14th Street and 2nd Ave - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.6	1.6	0.5	i @	6.2 min	3.58	2.9		0.0038	0.013	13.3"	Use 15"					
												V_{15} = 3.3 fps								
												t_{pipe} = 375/60 (3.3) = 2.0 min								
												t_c = 6.2 + 2 = 8.2 min								
2		B	R	2.8	4.4	0.5	i @	8.2 min	3.15	6.9		0.0037	0.013	18.5"	Use 21"					
												V_{21} = 4.0 fps								
												t_{pipe} = 225/60(4) = 0.9 min								
												t_c = 8.2 + 0.9 = 9.1 min								
10		Q	R	7.2	7.2	0.5	i @	9.0 min	2.98	10.7		0.0105	0.013	17.9"	Use 18"					
												V_{18} = 6.1 fps								
												t_{pipe} = 120/60(6.1) = 0.3 min								
												t_c = 9 + 0.3 = 9.3 min								
1,2+10		R	N		11.6	0.5	i @	9.3 min	2.9	16.8		0.0048	0.013	24.6"	Use 27"					
												V_{27} = 5.4 fps								
												t_{pipe} = 810/60(5.4) = 2.5 min								
												t_c = 9.3 + 2.5 = 11.8 min								

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CIA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
14th Street and 2nd Ave - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
6		K	E	0.6	0.6	0.5	i @	2.9 min	5.56	1.7		0.0044	0.013	10.6"	Use	12"				
												V_{12} =	3.0	fps						
												t_{pipe} =	510/60(3) =			2.8 min				
												t_c =	2.9 + 2.8 =	5.7 min						
5		D	E	3.1	3.1	0.5	i @	10.1 min	2.74	4.2		0.0147	0.013	11.9"	Use	12"				
												V_{12} =	5.5	fps						
												t_{pipe} =	150/60(5.5) =			0.5 min				
												t_c =	10.1 + 0.5 =	10.6 min						
5+6		E	F	-	3.7	0.5	i @	10.6 min	2.69	5.0		0.0042	0.013	16.0"	Use	18"				
												V_{18} =	3.9	fps						
												t_{pipe} =	265/60(3.9) =			1.1 min				
												t_c =	10.6 + 1.1 =	11.7 min						
7		L	F	1.4	1.4	0.5	i @	4.5 min	4.25	3.0		0.0044	0.013	13.1"	Use	15"				
												V_{15} =	3.5	fps						
												t_{pipe} =	470/60	(3.5)	=	2.2 min				
												t_c =	4.5 + 2.2 =	6.7 min						

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
14th Street and 2nd Ave - 25 year

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
5, to 7		F	G	-	5.1	0.5	i@	11.7 min	2.57	6.6		0.0019	0.013	20.6"	Use	21"				
												V_{21} =	2.9 fps							
												t_{pipe} =	130/60(2.9) =			0.7 min				
												t_c =	11.7 + 0.7 =			12.4 min				
3		C	H	1.3	1.3	0.5	i@	9.1 min	2.77	1.8		0.0047	0.013	10.7"	Use	12"				
												V_{12} =	3.1 fps							
												t_{pipe} =	640/60(3.1) =			3.4 min				
												t_c =	9.1 + 3.4 =			12.5 min				
8		G	H	7.1	12.2	0.5	i@	12.4 min	2.49	15.2		0.003	0.013	25.9"	Use	27"				
												V_{27} =	4.3 fps							
												t_{pipe} =	155/60(4.3) =			0.6 min				
												t_c =	12.4 + 0.6 =			13 min				
at "H"		H	M	-	13.5	0.5	i@	13.0 min	2.42	16.3		0.0042	0.013	24.9"	Use	27"				
												V_{27} =	5.1 fps							
												t_{pipe} =	140/60(5.1) =			0.5 min				
												t_c =	13.0 + 0.5 =			13.5 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by JAC Date 12/17/91
Checked by _____ Date _____
14th Street and 2nd Ave - 25 year

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
11		I	J	3.4	3.4	0.5	i@	8.4 min	3.11	5.3		0.0039	0.013	16.6"	Use	18"				
												V_{18} =	3.7	fps						
												t_{pipe} =	200/60 (3.7)		=	0.9 min				
												t_c =	8.4 + 0.9		=	9.3 min				
12		J	M	5.2	8.6	0.5	i@	9.3 min	2.91	12.5		0.0039	0.013	22.9"	Use	24"				
												V_{24} =	4.5	fps						
												t_{pipe} =	340/(60)(4.5)		=	1.3 min				
												t_c =	9.3 + 1.3		=	10.6 min				
At "M"		M	N	-	22.1	0.5	i@	13.5 min	2.37	26.2		0.0009	0.013	39.8"	Use	42"				
												V_{42} =	3.1	fps						
												t_{pipe} =	190/60 (3.1)		=	1.0				
												t_c =	13.5 + 1		=	14.5 min				
4		S	P	0.63	0.63	0.5	i@3	2 min	5.32	1.7		0.0047	0.013	10.5"	Use	12"				
												V =	3.1	fps						
												t_{pipe} =	775/60(3.1)		=	4.2 min				
												t_c =	3.2 + 4.2		=	7.4 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/17/91
Checked by _____ Date _____
14th Street and 2nd Ave - 25year

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
9		N	P	10.0	43.7	0.5	i@	14.5 min	2.26	49.4		0.002	0.013	43.4"	Use	48"				
												V_{full}	5.1 fps							
												t_{pipe}	=350/60(5.1) =			1.1 min				
												t_c	14.5 + 1.1 =			15.6 min				
		P		-	43.7	0.5	i@	15.6 min	2.17	47.4										

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO
Computed by DTC Date 12/12/91
Checked by _____ Date _____
14th Street and 2nd Ave - Summary

Drainage Area Number	Location			A		C	Time of Concentration		I	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
	Summary -			2	YR		5	YR			10	YR		25	YR					
Existing ϕ (in)	Reach	Slope (ft/ft)	Length (ft)	Discharge (cfs)	Dia ϕ (in)		Discharge (cfs)	Dia ϕ (in)			Discharge (cfs)	Dia ϕ (in)		Discharge (cfs)	Dia ϕ (in)					
8	A to B	0.0038	375	1.14	10		1.8	12			2.2	12		2.9	15					
8	B to R	0.0037	225	2.7	15		4.3	18			5.3	18		6.9	21					
8	Q to R	0.0105	120	4.3	15		6.7	18			8.4	18		10.7	18					
8	R to N	0.0048	810	7.0	18		10.6	21			13.1	24		16.8	27					
8	K to E	0.0044	510	0.6	8		0.97	10			1.3	10		1.7	12					
8	D to E	0.0147	150	1.8	10		2.7	12			3.3	12		4.2	12					
10	E to F	0.0042	265	2.3	12		3.1	15			3.9	15		5.0	18					
8	L to F	0.0044	470	1.2	10		1.8	12			2.3	12		3.0	15					
12	F to G	0.0019	130	2.6	15		4.1	18			5.1	21		6.6	21					
8	C to H	0.0047	640	0.8	8		1.2	10			1.5	10		1.8	12					
12	G to H	0.003	155	6.1	21		9.5	24			11.8	24		15.2	27					
12	H to M	0.0042	140	6.6	21		10.3	24			12.7	24		16.3	27					
8/10	I to J	0.0039	200	2.2	12		3.3	15			4.1	18		5.3	18	110' -	10"	20' -		
10/12	J to M	0.0039	340	5.2	18		7.8	21			9.8	21		12.5	24	150 -	10"	190' -	12"	
14	M to N	0.0009	190	10.6	30		16.4	36			20.2	42		26.2	42					
8	S to P	0.0047	775	0.6	8		0.98	10			1.3	10		1.7	12					
16	N to P	0.002	350	19.9	33		30.6	42			38.2	42		49.4	48					
	P	—	—	19.2	—		29.9	—			36.9	—		47.4	—					

Representative Basin No. 4

25th Street and 4th Avenue

Computation Form
Drainage and Storm Sewer Design
Rational Method

Project: Sacramento CSO - Phase 3
Computed by: JAC Date: 12/26/91
Checked by: _____ Date: _____
25th Street and 44th Ave - 2 year

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope 3:1	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	C	4.0	4.0	0.5	i @ 12.6 min		1.03	2.1		0.0040	0.013	11.7"	Use	12"				
	Overland	110'	0.85	fps			2.1					V_{12} =	2.9	fps						
	Street	785'	1.25	fps			10.5													
							12.6 min					t_{pipe} =	130/60	(2.9)		= 0.7 min				
	Street Slope: (23.96-20.9)/785	Gutter near (27b-48)	= 0.0039 '/'	- Gutter near (27b-12)								t_c =	12.6	+ 0.7	=	13.3 min				
	Design Slope: (17.27-16.75)/130	(27b-12)	= 0.0040 '/'	- (27b-45)																
2		B	C	6.2	6.2	0.5	i @	10.6 min	1.11	3.4		0.0038	0.013	14.1"	Use	15"				
	Overland	110'	0.85	fps			2.2					V_{15} =	3.2	fps						
	Street	785'	1.55	fps			8.4													
							10.6	min				t_{pipe} =	130/60(3.2) =			0.7	min			
	Street Slope: (24.47-19.74)/785	Gutter near (27b-53)	= 0.006 '/'	- Gutter near (27b-18)								t_c =	10.6	+ 0.7	= 11.3	min				
	Design Slope: (16.67-16.18)/130 =	(27b-18)	0.0038 '/'	- (27b-45)																
		C	D	—	10.2	0.5	i @	13.3 min	0.99	5.0		0.0046	0.013	15.7"	Use	18"				
	t_c A to C = 13.3 min											V_{18} =	4 fps							
	t_c B to C = 11.3 min																			
	governing t_c =	13.3 min										t_{pipe} =	400/60(4) =			1.7 min				
	Design Slope: (16.10-14.25)/400	(27b-45)	= 0.0046 '/'	-(27a-91)								t_c =	13.3	+ 1.7	=	15 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSD - Phase 3
Computed by LAC Date 12/26/91
Checked by _____ Date _____
23th Street and 4th Ave - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
9		I	F	1.6	1.6	0.5	i @ 4.9 min		1.58	1.3		0.0045	0.013	9.5"	Use 10"					
	Overland	100	0.85	fps			2.0					V_{10} = 2.7 fps								
	Street	200	1.13	fps			2.9													
							4.9 min					t_{pipe} = 635/2.7(60) =				3.9 min				
	Street Slope:	Gutter near	(27e-25)	- Gutter near		(27e-13)						t_c = 4.9 + 3.9 =				8.7 min				
	(24.88-24.32)/175' =	0.0032'/'																		
	Design Slope:	(27e-11)	-(27e-12)																	
	(19.32-17.62)/375 =	0.0045'/'																		
	Length = 110+375+150 = 635'																			
4		F	G	0.6	2.2	0.5	i @	8.7 min	1.25	1.4		0.0065	0.013	9.2"	Use	10"				
	t_c from I to F =	8.7 min										V_{10} = 3.2 fps								
	No t_c computations	made	for Area	4								t_{pipe} = 80/3.2(60) =				0.4 min				
	Design Slope:	(27e-56)	-(27e-8)									t_c = 8.7 + 0.4 =				9.1 min				
	(17.19-16.62)/80	= 0.0065	'/'																	

TIDH • Ver 1.0 • 1989

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: JAC Date: 2/26/91
Checked by: _____ Date: _____
25th Street and 4th Ave - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3		E	G	4.2	4.2	0.5	i @ 9.5 min		1.18	2.5		0.0057	0.013	11.7"	Use	12"				
	Overland	110	0.85	fps			2.2					V_{12} =	3.4	fps						
	Street	675	1.55	fps			7.3													
							9.5	min				t_{pipe} =	95/3.4(60)	= 0.5		min				
	Street Slope:	No data	- Assume	same	as street		slope	used in	B to	C		t_c =	9.5	+ 0.5	= 10 min					
	Slope = 0.006%																			
	Design Slope:	(27e-5)	- (27e-8)																	
	(17.27 - 16.73)/95 =	0.0057%																		
		G	H	-	6.4	0.5	i @ 10 min		1.14	3.6		0.0059	0.013	13.3	Use	15"				
	t_c from E to G =	10 min																		
	t_c from F to G =	9.1 min	Drainage Areas 3, 4 and 9									V_{15} =	4.0	fps						
	t_c = 10 min governs																			
												t_{pipe} =	375/4(60)	= 1.6	min					
	Design Slope:	(27e-8)	- (27e-6)									t_c =	10 + 1.6	= 11.6	min					
	(16.48 - 14.27)/375 =	0.0059%																		

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: [illegible] Date: 12/26/91
Checked by: _____ Date: _____
25th Street and 4th Ave - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5		J	H	2.0	2.0	0.5	i @	6.9 min	1.40	1.4		0.0055	0.013	9.4"	Use 10"					
	Overland	100	0.85	fps			2.0					V_{10} =	3.0	fps						
	Street	325	1.1	fps			4.9													
							6.9	min				t_{pipe} =	105	/3(60) =	0.6 min					
	Street Slope:	Use RIMS	(27e-5b)	-(27e-7)																
	(23.30-22.11)/375 =	0.0029%										t_c =	6.9 +	0.6 =	7.5 min					
	Design Slope:	(27e-7) -	(27e-6)	inverts																
	(17.87 - 17.23)/105 =	0.0055%																		
		H	K	—	8.4	0.5	i @ 11.6 min		1.07	4.5		0.0060	0.013	14.4"	Use 15"					
	t_c from J to H =	7.5 min										V_{15} =	4.1	fps						
	t_c from G to H =	11.6 min																		
	Use 11.6 min as	governing t_c										t_{pipe} =	190	/4.1(60) =	0.8 min					
	Contributing Drainage area =	3, 4, 5 and 9										t_c =	11.6 +	0.8 =	12.4 min					
	Design Slope:	(27e-6)	-(27e-3)	inverts																
	(14.27 - 13.12)/190 =	0.0060	%																	

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO
Computed by LAC Date 12/26/91
Checked by Date
25th Street and 7th Ave - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
8		M	D	3.3	3.3	0.5	i @ 5.1 min		1.55	2.6		0.0044	0.013	12.4"	Use 15"					
	Overland	100	0.85	fps			2.0					V_{15} = 3.5 fps								
	Street	320	1.7	fps			3.1					t_{pipe} = 130/3.5(60) =				0.6 min				
							5.1 min													
	Street Slope: Gutter near (27b-12) - Gutter near (27a-90)											t_c = 5.1 + 0.6 =				5.7 min				
	(2088 - 18.28)/350 = 0.0074 ft/ft																			
	Design Slope:	(27a-90)	- (27a-91)	inverts																
	(14.72 - 14.15)/130 =	0.0044 ft/ft																		
	Note: 10" overflow pipe enters (27a-90) from the NW at 14.72. Neglect contribution from 10" sewer for analysis.																			
		D	L	–	13.5	0.5	i @ 15 min		0.92	6.2		0.0042	0.013	17.4"	Use 18"					
	t_c from M to D =	5.7 min										V_{18} = 3.8 fps								
	t_c from C to D =	15 min																		
	governing t_c = 15 min											t_{pipe} = 125/3.8(60) =				0.5 min				
	Contributing Drainage Areas: 1, 2, 8											t_c = 15 + 0.5 =				15.5 min				
	Design Slope:	(27a-91)	- (27c-1)																	
	(14.15 - 13.63)/125 =	0.0042 ft/ft																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CIA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: LAC Date: 12/26/91
Checked by: _____ Date: _____
25th Street and 4th Ave - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
7		L	K	1.8	15.3	0.5	i@15.5min		0.91	7.0		0.0051	0.013	17.5"	Use	18"				
	No t_c calculation made for Area 7.											V_{18} = 4.3 fps								
	Governing t_c from upstream basins											t_{pipe} = 180/4.3(60) = 0.7 min								
	t_c From D to L = 15.5 min											t_c = 15.5 + 0.7 = 16.2 min								
	Contributing Drainage Areas: 1, 2, 8, 7																			
	Design Slope: (27e-1) - (27e-3) (13.63 - 12.71)/180 = 0.0051 ft/ft																			
6		K	O	2.5	26.2	0.5	i@16.2min		0.90	11.8		0.0048	0.013	21.6"	Use	24"				
	t_c from L to K = 16.2 min											V_{24} = 5 fps								
	t_c from H to K = 12.4 min																			
	t_c from Area 6 neglected, t_c 16.2 governs											t_{pipe} = 265/5(60) = 0.9 min								
	Contributing Drainage Areas: 1, 2, 3, 4, 5, 6, 7, 8, 9											t_c = 16.2 + 0.9 = 17.1 min								
	Design Slope: (27e-3) - (27f-27) (12.30 - 11.04)/265 = 0.0048 ft/ft																			

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO Phase 3
Computed by: LAC Date: 12/26/91
Checked by: _____ Date: _____
25th Street and 4th Ave - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
13		Q	R	0.3	0.3	0.5	i @	3.7 min	1.79	0.3		0.0036	0.013	5.7"	Use	8"				
	Overland	100	0.85	fps			2.0					V_g =	2.1 fps							
	Street	130	1.3	fps			1.7													
							3.7	min				t_{pipe} =	85/2.1(60) =			0.7 min				
	Street Slope: Gutter near (27f-40) - Gutter near (27f-38)											t_c =	3.7 + 0.7 =			4.4 min				
	(23.47-22.81)/150 =	0.0040 ft/ft																		
	Design Slope:	(27f-38) -	(27f-39)	Inverts																
	(19.64-19.33)/85 =	0.0036 ft/ft																		
14		P	R	3.5	3.5	0.5	i @	6.7 min	1.42	2.5		0.0030	0.013	13.2"	Use	15"				
	Overland	100	0.85	fps			2.0					V_{15} =	2.9 fps							
	Street	280	1.0	fps			4.7													
							6.7					t_{pipe} =	125/2.9(60) =			0.7 min				
	Street Slope: Gutter near (27f-74) - Gutter near (27f-40)											t_c =	6.7 + 0.7 =			7.4 min				
	(24.48-23.77)/280 =	0.0025 ft/ft																		
	Design Slope:	(27f-40)	-(27f-39)																	
	(19.7-19.33)/125 =	0.0030 ft/ft																		
—		R	S	—	3.8	0.5	i @ 7.4 min		1.36	2.6		0.0062	0.013	11.6"	Use	15"	to match P to R			
	t_c from Q to R =	4.4 min																		
	t_c from P to R =	7.4 min										V_{15} =	4.2 fps							
	t_c = 7.4 governs											t_{pipe} =	735/4.2(60) =			2.9 min				
	Design Slope:	(27f-39)	-(27f-25)																	
	(19.33-14.15)/735 =	0.0062 ft/ft										t_c =	7.4 + 2.9 =			10.3 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/26/91
Checked by _____ Date _____
25th Street and 4th Ave - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
12		5	0	3.7	7.5	0.5	i@ 10.3 min		1.13	4.2		0.0058	0.013	14.1"	Use	15"				
	No tc calculation for Area 12, upstream basin control.											V_{15} = 4 fps								
	tc from 8 to 5 =	10.3 min										t_{pipe} = 260/60(4) =				1.1 min				
	Contributing Areas = 12, 13 & 14											tc = 10.8 + 1.1 = 11.9 min								
	Design Slope: (27 - 25) - (27 - 27)																			
	(14.75 - 13.23)/260 =	0.0058%																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/30/99
Checked by _____ Date _____
25th Street & 4th Ave - Sump

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	C	4.0	4.0	0.5	i @ 12.6 min		1.56	3.12		0.0040	0.013	13.5"	Use	15"				
												V_{15} =	3.3 fps							
												t_{pipe} =	130/60(3.3) =			0.7 min				
												t_c =	12.6 + 0.7 =			13.3 min				
2		B	C	6.2	6.2	0.5	i @ 10.6 min		1.70	5.3		0.0038	0.013	16.6"	Use	18"				
												V_{18} =	3.7 fps							
												t_{pipe} =	130/60(3.7) =			0.6 min				
												t_c =	10.6 + 0.6 =			11.2 min				
—		C	D	—	10.2	0.5	i @ 13.3 min		1.52	7.8		0.0046	0.013	18.6"	Use	21"				
	t_c from A to C =	13.3 min										V_{21} =	4.5 fps							
	t_c from B to C =	11.2 min										t_{pipe} =	400/60(4.5) =			1.5 min				
	governing t_c =	13.3 min										t_c =	13.3 + 1.5 =			14.8 min				
9		I	F	1.6	1.6	0.5	i @ 4.9 min		2.44	2.0		0.0045	0.013	11.2"	Use	12"				
												V_{12} =	3.1 fps							
												t_{pipe} =	635/60(3.1) =			3.4 min				
												t_c =	4.9 + 3.4 =			8.3 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 5
Computed by: LAC Date: 1/2/92
Checked by: _____ Date: _____
25th Street and 4th Ave - 5year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
4		F	G	0.6	2.2	0.5	i@	8.3min	1.96	2.2		0.0065	0.013	10.8"	Use	12"				
	t_c from I to F =	8.3min																		
												V_{12} =	3.7 fps							
												t_{pipe} =	80/60(3.7) =			0.4min				
												t_c =	8.3 + 0.4 =			8.7min				
3		E	G	4.2	4.2	0.5	i@ 9.5min		1.81	3.8		0.0057	0.013	13.6"	Use	15"				
												V_{15} =	4.0 fps							
												t_{pipe} =	95/60(4) =			0.4min				
												t_c =	9.5 + 0.4 =			9.9min				
—		G	H	—	6.4	0.5	i@	9.9min	1.75	5.6		0.0059	0.013	15.7"	Use	18"				
	t_c from E to G =	9.9																		
	t_c from F to G =	8.7										V_{18} =	4.6 fps							
	governing t_c =	9.9										t_{pipe} =	375/60(4.6) =			1.4min				
												t_c =	9.9 + 1.4 =			11.3min				
5		J	H	2.0	2.0	0.5	i@ 6.9min		2.15	2.2		0.0055	0.013	11.2"	Use	12"				
												V_{12} =	3.4 fps							
												t_{pipe} =	105/(60)(3.4) =			0.5min				
												t_c =	6.9 + 0.5 =			7.4min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/2/92
Checked by Date
25th Street and 4th Ave - 5year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
–		H	K	–	8.4	0.5	i@	11.3 min	1.65	6.9		0.0060	0.013	13.6"	Use	15"				
	t_c from J to H =	7.4										V_{15} =	4.1 fps							
	t_c from G to H =	11.3										t_{pipe} =	190/60(4.1) =			0.8 min				
	governing t_c =	11.3										t_c =	11.3 + 0.8 =			12.1 min				
8		M	D	3.3	3.3	0.5	i@ 5	min	2.39	3.9		0.0044	0.013	14.5"	Use	15"				
												V_{15} =	3.5 fps							
												t_{pipe} =	130/60(3.5) =			0.6 min				
												t_c =	5.1 + 0.6 =			5.7 min				
–		D	L	–	13.5	0.5	i@	14.8 min	1.41	9.5		0.0042	0.013	20.4"	Use	21"				
	t_c from M to D =	5.7 min										V_{21} =	4.3 fps							
	t_c from C to D =	14.8 min										t_{pipe} =	125/60(4.3) =			0.5 min				
	governing t_c =	14.8 min										t_c =	14.8 + 0.5 =			15.3 min				
7		L	K	1.8	15.3	0.5	i@ 15.3 min		1.39	10.6		0.0051	0.013	20.5"	Use	21"				
	t_c from D to L =	15.3 min										V_{21} =	4.7 fps							
												t_{pipe} =	180/60(4.7) =			0.6 min				
												t_c =	15.3 + 0.6 =			15.9 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: LHC Date: 1/2/92
Checked by: _____ Date: _____
25th Street and 4th Ave - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
6		K	O	2.5	26.2	0.5	i @	15.9 min	1.37	17.9		0.0048	0.013	25.2"	Use	27"				
	t_c from L to K =	15.9 min																		
	t_c from H to K =	12.1 min										V_{27} =	5.4 fps							
	governing t_c =	15.9 min																		
												t_{pipe} =	265 / 60(5.4) =			0.8 min				
												t_c =	15.9 + 0.8 =			16.7 min				
13		Q	R	0.3	0.3	0.5	i @ 3	7 min	3.51	0.5		0.0036	0.013	7.0"	Use	8"				
												V_8 =	2.1 fps							
												t_{pipe} =	85 / 60(2.1) =			0.7 min				
												t_c =	3.7 + 0.7 =			4.4 min				
14		P	R	3.5	3.5	0.5	i @ 6	7 min	2.18	3.8		0.0030	0.013	15.4"	Use	18"				
												V_{18} =	3.3 fps							
												t_{pipe} =	125 / 60(3.3) =			0.6 min				
												t_c =	6.7 + 0.6 =			7.3 min				
—		R	S	—	3.8	0.5	i @	7.3 min	2.10	4.0		0.0062	0.013	13.6"	Use	18" to match		P to R		
	t_c from Q to R =	4.4 min										V_{18} =	4.7 fps							
	t_c from P to R =	7.3 min										t_{pipe} =	735 / 60(4.7) =			2.6 min				
	governing t_c = 7.3	min										t_c =	7.3 + 2.6 =			9.9 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: AC Date: 1/2/92
Checked by: _____ Date: _____
25th Street and 4th Ave -

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
12		5	0	3.7	7.5	0.5	i@	9.9 min	1.75	6.6		0.0058	0.013	16.7"	Use	18" to match RA 5				
	t_c from R to 5 =	9.9 min										V_{18} =	4.5 fps							
												t_{pipe} =	260/60(4.5) =			1 min				
												t_c =	9.9 + 1 =			10.9 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/2/92
Checked by _____ Date _____
25th Street & 4th Ave - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	C	4.0	4.0	0.5	i @ 12.6 min		1.93	3.9		0.0040	0.013	14.7"	Use	15"				
												V_{15} =	3.3	fps						
												t_{pipe} =	130/60(3.3) =			0.7 min				
												t_c =	12.6 + 0.7 =			13.3 min				
2		B	C	6.2	6.2	0.5	i @ 10.6 min		2.11	6.5		0.0035	0.013	18.0"	Use	18"				
												V_{8} =	3.7	fps						
												t_{pipe} =	130/60(3.7) =			0.6 min				
												t_c =	10.6 + 0.6 =			11.2 min				
—		C	D	—	10.2	0.5	i @ 13.3 min		1.87	9.5		0.0046	0.013	20.0"	Use	21"				
	t_c from A to C =	13.3 min																		
	t_c from B to C =	11.2 min																		
	governing t_c =	13.3 min										V_{21} =	4.5	fps						
												t_{pipe} =	400/60(4.5) =			1.5 min				
												t_c =	13.3 + 1.5 =			14.8 min				
9		I	F	1.6	1.6	0.5	i @ 4.9 min		3.06	2.4		0.0045	0.013	12.0"	Use	12"				
												V_{12} =	3.1 fps							
												t_{pipe} =	635/60(3.1) =			3.4 min				
												t_c =	4.9 + 3.4 =			8.3 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 5
Computed by DAC Date 1/2/92
Checked by Date
25th Street and 4th Ave - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
4		F	G	0.6	2.2	0.5	i@	8.3min	2.44	2.7		0.0065	0.013	11.7"	Use	12"				
	t_c from I to F =	8.3min										V_{12} =	3.7 fps							
												t_{pipe} =	80/60(3.7) =			0.4min				
												t_c =	8.3 + 0.4 =			8.7min				
3		E	G	4.2	4.2	0.5	i@ 9.	5min	2.24	4.7		0.0057	0.013	14.8"	Use	15"				
												V_{15} =	4.0 fps							
												t_{pipe} =	95/60(4) =			0.4min				
												t_c =	9.5 + 0.4 =			9.9min				
—		G	H	—	6.4	0.5	i@	9.9min	2.18	7.0		0.0059	0.013	17.0"	Use	18"				
	t_c from E to G =	9.9min										V_{18} =	4.6 fps							
	t_c from F to G =	8.7min										t_{pipe} =	375/60(4.6) =			1.4min				
	governing t_c =	9.9min										t_c =	9.9 + 1.4 =			11.3min				
5		J	H	2.0	2.0	0.5	i@ 6.	9min	2.68	2.7		0.0055	0.013	12.1"	Use	15"				
												V_{15} =	3.9 fps							
												t_{pipe} =	105/(60)(3.9) =			0.4min				
												t_c =	6.9 + 0.4 =			7.3min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/2/92
Checked by _____ Date _____
25th Street and 4th Ave - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
–		H	K	–	8.4	0.5	i@ 11.3 min		2.05	8.6		0.0060	0.013	18.4"	Use	21"				
	t_c from J to H =	7.3 min																		
	t_c from G to H =	11.3 min										V_{21} =	5.1	fps						
	governing t_c =	11.3 min										t_{pipe} =	190/60(5.1)	=	0.6 min					
												t_c =	11.3	+ 0.6 =	11.9 min					
8		M	D	3.3	3.3	0.5	i@ 5	min	2.98	4.9		0.0044	0.013	15.8"	Use	18"				
												V_{18} =	4.0 fps							
												t_{pipe} =	130/60(4)	=	0.5 min					
												t_c =	5.1 +	0.5 =	5.6 min					
–		D	L	–	13.5	0.5	i@	14.8 min	1.74	11.7		0.0042	0.013	22.0"	Use	24"				
	t_c from M to D =	5.6 min																		
	t_c from C to D =	14.8 min										V_{24} =	4.7 fps							
	governing t_c =	14.8 min										t_{pipe} =	125/60(4.7)	=	0.4 min					
												t_c =	14.8 +	0.4 =	15.2 min					
7		L	K	1.8	15.3	0.5	i@	15.2 min	1.71	13.1		0.0051	0.013	22.2"	Use	24"				
	t_c from D to L =	15.2 min										V_{24} =	5.2 fps							
												t_{pipe} =	180/60(5.2)	=	0.6 min					
												t_c =	15.2 +	0.6 =	15.8 min					

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LHC Date 1/2/92
Checked by _____ Date _____
25th Street and 4th Ave - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
6		K	O	2.5	26.2	0.5	i @	15.8 min	1.69	22.1		0.0048	0.013	27.3"	Use	30"				
	t_c from L to K =	15.8 min										V_{30} =	5.8 fps							
	t_c from H to K =	11.9 min										t_{pipe} =	265/60(5.8) =			0.8 min				
	governing t_c =	15.8 min										t_c =	15.8 + 0.8 =			16.6 min				
13		Q	R	0.3	0.3	0.5	i @ 3	7 min	3.78	0.6		0.0036	0.013	7.4"	Use	8"				
												V_8 =	2.1 fps							
												t_{pipe} =	85/60(2.1) =			0.7 min				
												t_c =	3.7 + 0.7 =			4.4 min				
14		P	R	3.5	3.5	0.5	i @ 6	7 min	2.71	4.7		0.0030	0.013	16.7"	Use	18"				
												V_{18} =	3.3 fps							
												t_{pipe} =	125/60(3.3) =			0.6 min				
												t_c =	6.7 + 0.6 =			7.3 min				
–		R	S	–	3.8	0.5	i @	7.3 min	2.61	5.0		0.0062	0.013	14.9"	Use	18" to match P to R				
	t_c from Q to R =	4.4										V_{18} =	4.7 fps							
	t_c from P to R =	7.3										t_{pipe} =	735/60(4.7) =			2.6 min				
												t_c =	7.3 + 2.6 =			9.9 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by AC Date
Checked by Date
25th Street and 4th Ave - 10 year

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope ft/ft	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
12		5	0	3.7	7.5	0.5	i@	9.9 min	2.18	8.2		0.0058	0.013	18.1"	Use	21"				
	t_c from R to 5 =	9.9 min																		
												V_{21} =	5.0 fps							
												t_{pipe} =	260/60(5) = 0.9 min							
												t_c =	9.9 + 0.9 =	10.8 min						

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/2/92
Checked by _____ Date _____
25th Street & 4th Ave - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope, %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	C	4.0	4.0	0.5	i @ 12.6 min		2.47	4.9		0.0040	0.013	16.0"	Use	18"				
												V_{18} =	3.8 fps							
												t_{pipe} =	130/60(3.8) =			0.6 min				
												t_c =	12.6 + 0.6 =			13.2 min				
2		B	C	6.2	6.2	0.5	i @ 10.6 min		2.69	8.3		0.0036	0.013	19.7"	Use	21"				
												V_{21} =	4.1 fps							
												t_{pipe} =	130/60(4.1) =			0.5 min				
												t_c =	10.6 + 0.5 =			11.1 min				
4		C	D	—	10.2	0.5	i @ 13.2 min		2.40	12.2		0.0046	0.013	22.0"	Use	24"				
	t_c from A to C =	13.2 min										V_{24} =	4.9 fps							
	t_c from B to C =	11.1 min										t_{pipe} =	400/60(4.9) =			1.4 min				
	governing t_c =	13.2 min										t_c =	13.2 + 1.4 =			14.6 min				
9		I	F	1.6	1.6	0.5	i @ 4.9 min		3.92	3.1		0.0045	0.013	13.2"	Use	15"				
												V_{15} =	3.5 fps							
												t_{pipe} =	635/60(3.5) =			3 min				
												t_c =	4.9 + 3 =			7.9 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 5
Computed by LAC Date 1/2/92
Checked by Date
25th Street and 4th Ave - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
4		F	G	0.6	2.2	0.5	i@	7.9min	3.21	3.5		0.0065	0.013	12.9"	Use	15"				
	t_c from I to F =	7.9min																		
												V_{15} =	4.3 fps							
												t_{pipe} =	80/60(4.3) =			0.3min				
												t_c =	7.9+0.3 =			8.2min				
3		E	G	4.2	4.2	0.5	i@9.5min		2.87	6.0		0.0057	0.013	16.2"	Use	18"				
												V_{18} =	4.5 fps							
												t_{pipe} =	95/60(4.5) =			0.4min				
												t_c =	9.5+0.4 =			9.9min				
—		G	H	—	6.4	0.5	i@	9.9min	2.78	8.9		0.0059	0.013	18.7"	Use	21"				
	t_c from E to G =	9.9min																		
	t_c from F to G =	8.2min										V_{21} =	5.1 fps							
	governing t_c =	9.9min										t_{pipe} =	375/60(5.1) =			1.2min				
												t_c =	9.9+1.2 =			11.1min				
5		J	H	2.0	2.0	0.5	i@6.9min		3.43	3.4		0.0055	0.013	13.2"	Use	15"				
												V_{15} =	3.9 fps							
												t_{pipe} =	105/(60)(3.9) =			0.4min				
												t_c =	6.9+0.4 =			7.3min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by ZAC Date 1/2/92
Checked by Date
25th Street and 4th Ave - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
–		H	K	–	8.4	0.5	i@	11.1min	2.64	11.1		0.0060	0.013	20.2"	Use	21"				
	t_c from J to H =	7.3min																		
	t_c from G to H =	11.1min										V_{21} =	5.1fps							
	governing t_c =	11.1min										t_{pipe} =	190/60(5.1) =			0.6min				
												t_c =	11.1	+0.6 =		11.7min				
8		M	D	3.3	3.3	0.5	i@5/min		3.82	6.3		0.0044	0.013	17.3"	Use	18"				
												V_{18} =	4.0	fps						
												t_{pipe} =	130/60(4) =			0.5min				
												t_c =	5.1+	0.5	=	5.6min				
–		D	L	–	13.5	0.5	i@	14.6min	2.24	15.1		0.0042	0.013	24.2"	Use	27"				
	t_c from M to D =	5.6min										V_{27} =	5.1fps							
	t_c from C to D =	14.6min										t_{pipe} =	125/60(5.1) =			0.4min				
	governing t_c =	14.6min										t_c =	14.6	+0.4	=	15min				
7		L	K	1.8	15.3	0.5	i@	15.0min	2.2	16.8		0.0051	0.013	24.3"	Use	27"				
	t_c from D to L =	15.0min										V_{27} =	5.6fps							
												t_{pipe} =	180/60(5.6) =			0.5min				
												t_c =	15+	0.5	=	15.5min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LHC Date 1/2/92
Checked by _____ Date _____
25th Street and 4th Ave - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
6		K	O	2.5	26.2	0.5	i@	15.5min	2.17	28.4		0.0048	0.013	30"	Use 30"					
	t_c from L to K =	15.5min																		
	t_c from H to K =	11.7min										V_{30} =	5.8 fps							
	governing t_c =	15.5min																		
												t_{pipe} =	265	/60(5.8) =		0.8 min				
												t_c =	15.5 +	0.8 =		16.3min				
13		Q	R	0.3	0.3	0.5	i@3	7min	4.91	0.7		0.0036	0.013	7.9"	Use	8"				
												V_8 =	2.1fps							
												t_{pipe} =	85	/60(2.1) =		0.7 min				
												t_c =	3.7 +	0.7 =		4.4min				
14		P	R	3.5	3.5	0.5	i@6	7min	3.47	6.1		0.0030	0.013	18.4"	Use	21"				
												V_{21} =	3.6 fps							
												t_{pipe} =	125	/60(3.6) =		0.6 min				
												t_c =	6.7 +	0.6		7.3 min				
—		R	S	—	3.8	0.5	i@	7.3min	3.34	6.4		0.0062	0.013	16.3"		Use 21" from Jch P to R				
	t_c from Q to R =	4.4min																		
	t_c from P to R =	7.3min										V_{21} =	5.2 fps							
												t_{pipe} =	735	/60(5.2) =		2.4 min				
												t_c =	7.3 +	2.4 =		9.7 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 1/2/92
Checked by _____ Date _____
25th Street and 4th Ave - 25 year.

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope ft/ft	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
12		5	0	3.7	7.5	0.5	i@	9.7min	2.82	10.6		0.0058	0.013	20.0"	Use	21"				
	t_c from R to 5 =	9.7min										V_{cd} =	5.0							
												t_{pipe} =	260/60(5) = 0.9min							
												t_c =	9.7 + 0.9 =	10.6 min						

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Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: DAC Date: 12/26/91
Checked by: _____ Date: _____
25th Street and 4th Ave

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
	Summary			2 YR			5 YR				10 YR			25 YR						
Existing Φ (in)	Reach	Slope (ft/ft)	Length (ft)	Discharge (cfs)	Dia (Φ) (in)		Discharge (cfs)	Dia Φ (in)			Discharge (cfs)	Dia Φ (in)		Discharge (cfs)	Dia Φ (in)					
8	A to C	0.0040	130	2.1	12		3.1	15			3.9	15		4.9	18					
8	B to C	0.0038	130	3.4	15		5.3	18			6.5	18		8.3	21					
10	C to D	0.0046	400	5.0	18		7.8	21			9.5	21		12.2	24					
8	I to F	0.0045	635	1.3	10		2.0	12			2.4	12		3.1	15					
8	F to G	0.0065	80	1.4	10		2.2	12			2.7	12		3.5	15					
8	E to G	0.0057	95	2.5	12		3.8	15			4.7	15		6.0	18					
10	G to H	0.0059	375	3.6	15		5.6	18			7.0	18		8.9	21					
8	J to H	0.0055	105	1.4	10		2.2	12			2.7	15		3.4	15					
10	H to K	0.0060	190	4.5	15		6.9	15			8.6	21		11.1	21					
10	M to D	0.0044	130	2.6	15		3.9	15			4.9	18		6.3	18					
12	D to L	0.0042	125	6.2	18		9.5	21			11.7	24		15.1	27					
12/14	L to K	0.0051	180	7.0	18		10.6	21			13.1	24		16.8	27	135'	12'	45	14'	
14/16	K to O	0.0048	265	11.8	24		17.9	27			22.1	30		28.4	30	155'	14'	110'	16"	
8	Q to R	0.0036	85	0.3	8		0.5	8			0.6	8		0.7	8					
8	P to R	0.0030	125	2.5	15		3.8	18			4.7	18		6.1	21					
8	R to S	0.0062	735	2.6	15		4.0	18			5.0	18		6.4	21					
8/10	S to O	0.0058	260	4.1	15		6.6	18			8.2	21		10.6	21	125'	8'	135'	10"	

Representative Basin No. 5

Harkness Avenue and Larkin Way

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by SAC Date 1/6/92
Checked by _____ Date _____
Harkness Ave + Larkin St - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.0	1.0	0.5	i @	5.4 min	1.53	0.8		0.0032	0.013	8.5'	Use	10"				
	Overland	110'	0.85	fps			2.1					V_{10} =	2.3	fps						
	Street	200'	1.0	fps			3.3													
							5.4	min				t_{pipe} =	170/2.3	(60)	= 1.2 min					
	Street Slope:	No survey										t_c =	5.4 + 1.2	= 6.6 min						
	Assume velocity	of 1 fps																		
	Design Slope:	No survey																		
	Assume slope of	0.0032 ft/ft																		
	Avg slope M to R, P to R, R to I			inverts																
	(8.46 - 7.91) / 105 = 0.0052																			
	(7.91 - 7.31) / 210 = 0.0029																			
	(7.31 - 7.07) / 120 = 0.0020																			
	0.0052(105) + 210(0.0029) + 120(0.0020) / 105 + 210 + 120 = 0.0032 ft/ft																			
2		B	C	1.5	2.5	0.5	i @ 6.6 min		1.43	1.8		0.0032	0.013	11.5"	Use 12"					
	Overland	100	0.85	fps			2.0					V_{12} =	2.6	fps						
	Street	180	1.0	fps			3.0													
							5.0	min				t_{pipe} =	220/2.6	(60)	= 1.4 min					
	Street Slope:	No survey										t_c =	6.6 + 1.4	= 8.0 min						
	Assume velocity	of 1 fps																		
	Design Slope:	No survey																		
	Assume slope of	0.0032 ft/ft																		
	t_c A to B =	6.6 min																		
	governing t_c =	6.6 min																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/6/90
Checked by _____ Date _____
Harkness Ave + Larkin St - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3		C	D	1.8	4.3	0.5	i@	8 min	1.31	2.8		0.0032	0.013	13.6"	Usc	15"				
	Overland	190'	0.85	fps			3.7						V_{15} =	3.0 fps						
	Street	125'	1.0	fps			2.1													
							5.8	min					t_{pipe} =	195/3(60) =			1.1 min			
	Street Slope:	No survey																		
	Assume velocity	of 1fps											t_c =	8 + 1.1 =		9.1 min				
	Design Slope:	No survey																		
	Assume slope of	0.0032'/'																		
	t_c B to C = 8.0 min																			
	governing t_c = 8. min																			
4		D	E	1.1	5.4	0.5	i@ 9.1 min		1.22	3.3		0.0032	0.013	14.4"	Usc	15"				
	No t_c computation for Area 4												V_{15} =	3.0 fps						
	governing t_c from upstream																			
													t_{pipe} =	140/3(60) =			0.8 min			
	t_c C to D = 9.1 min ∴ governing t_c																			
													t_c =	9.1 + 0.8		= 9.9 min				
	Design slope: No survey		Assume slope of 0.0032'/'																	
5		E	F	0.6	6.0	0.5	i@ 9.9 min		1.15	3.5		0.0032	0.013	14.7"	Usc	15"				
	No t_c computation for Area 5												V_{15} =	3.0 fps						
	governing t_c from upstream																			
													t_{pipe} =	160/3(60) =			0.9 min			
	t_c D to E = 9.9 min																			
													t_c =	9.9 + 0.9 =		10.8 min				
	Design Slope:	No survey																		
	As[illegible] slope of	0.0032'/'																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/6/92
Checked by _____ Date _____
Harkness Ave & Larkin St - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
6		F	H	0.9	6.9	0.5	i @ 10.8 min		1.10	3.8		0.0032	0.013	15.2"	Use	18"				
	Note computation for Area 6 governing t_c from upstream											V_{18} = 3.4 fps								
												t_{pipe} = 265/3.4(60) = 1.3 min								
	t_c from E to F = 10.8 min											t_c = 10.8 + 1.3 = 12.1 min								
	Design Slope: Assume slope of	No survey 0.0032 %																		
7		G	H	1.9	1.9	0.5	i @ 8.3 min		1.28	1.2		0.0066	0.013	8.6"	Use	10"				
	Overland	185'	0.55 fps				3.6					V_{10} = 3.3 fps								
	Street	240'	0.85 fps				4.7													
							8.3 min					t_{pipe} = 120/3.3(60) = 0.6 min								
	Street Slope: (14.04 - 13.84)/110 = 0.0018	Gutter near (266 - 57)					Gutter near (266 - 126)					t_c = 8.3 + 0.6 = 8.9 min								
	Design Slope: (10.54 - 9.75)/120 = 0.0066	(266 - 126)	- (266 - 62)																	
8		H	I	1.1	9.9	0.5	i @ 12.1 min		1.05	5.2		0.0033	0.013	17.0"	Use	18"				
	Note computation for Area 8, governing t_c from upstream											V_{18} = 3.4 fps								
	t_c F to H = 12.1 min																			
	t_c G to H = 8.9 min governing t_c = 12.1 min											t_{pipe} = 400/3.4(60) = 2 min								
	Contributing area 1 - 8 = 9.9 Ac											t_c = 12.1 + 2 = 14.1 min								
	Design Slope: (8.36 - 7.04)/400 = 0.0033 %	(266 - 52)	- (266 - 61)																	

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by SHC Date 1/6/92
Checked by _____ Date _____
Harkness Ave + Larkin St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
9		J	L	1.9	1.9	0.5	i@	5.3 min	1.53	1.5		0.0053	0.013	9.8"	Use	10"				
	Overland	200	0.85	fps			3.9					V_{10} = 2.9 fps								
	Street	80	0.97	fps			1.4													
							5.3 min					t_{pipe} = 120/2.9(60) = 0.7 min								
	Street Slope: Gutters near (25c-115)																			
	(15.84-15.46)/160 = 0.0024 1/1											t_c = 5.3 + 0.7 = 6 min								
	Design Slope: (23c-115) - (26b-46)			Inverts																
	(10.81-10.17)/120 = 0.0053 1/1																			
10		K	L	0.98	0.98	0.5	i@	5.0 min	1.56	0.8		0.0154	0.013	6.3"	Use	8"				
	Overland	200	0.85	fps			3.9					V_8 = 4.3 fps								
	Street	110	1.6	fps			1.1													
							5.0 min					t_{pipe} = 70/4.3(60) = 0.3 min								
	Street Slope: Gutter near (25c-115) - Gutter near (26b-52)																			
	(15.46-14.53)/150 = 0.0062 1/1											t_c = 5 + 0.3 = 5.3 min								
	Design Slope: (26b-52) - (26b-46)																			
	(11.25-10.17)/70 = 0.0154 1/1																			
—		L	M	—	2.9	0.5	i @	6 min	1.48	2.1		0.0044	0.013	11.5"	Use	12"				
	t_c from J to L =	6.0 min										V_{12} = 3.0 fps								
	t_c from K to L =	5.3 min																		
	governing t_c = 6.0	min										t_{pipe} = 385/3(60) = 2.1 min								
	Design Slope: (26b-46) - (26b-45)																			
	(10.17-8.46)/385 = 0.0044 1/1											t_c = 6 + 2.1 = 8.1 min								

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 5
Computed by SAL Date 1/6/92
Checked by _____ Date _____
Harkness Ave + Larkin St - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11		N	M	0.52	0.52	0.5	i @ 4.7 min		1.61	0.4		0.0053	0.013	5.9"	Use	8"				
	Overland	90'	0.85	fps			1.8					V_8 =	2.5 fps							
	Street	170'	0.97	fps			2.9													
							4.7 min					t_{pipe} =	105/2.5(60) =			0.7 min				
	Street Slope:	Gutters	near (23	c-91)								t_c =	4.7 + 0.7 =			5.4 min				
	(15.84 - 15.46)/160	= 0.0024	1/1																	
	Design Slope:	No survey																		
	Assume same slope as J to L =			0.0053	1/'															
—		M	P	—	3.4	0.5	i @ 8.1 min		1.30	2.2		0.0050	0.013	11.4"	Use	12"				
	Contributing Areas:	9, 10, 11										V_{12} =	3.2 fps							
	t_c from L to M	= 8.1 min										t_{pipe} =	110/3.2(60) =			0.6 min				
	Design Slope:	(26b - 45)	-(26b - 50)									t_c =	8.1 + 0.6 =			8.7 min				
	(18.46 - 7.91)/110 =	0.0050 1/'																		
13		O	P	1.0	1.0	0.5	i @ 3.2 min		1.88	0.9		0.0084	0.013	7.4"	Use	8"				
	Overland	85'	0.85	fps			1.7					V_8 =	3.2 fps							
	Street	110'	1.2	fps			1.5													
							3.2	min				t_{pipe} =	185/3.2(60) =			1 min				
	Street Slope:	Gutters	near (26	b-51)								t_c =	3.2 + 1 =			4.2 min				
	(14.25 - 13.97)/80 =	0.0035 1/'																		
	Design Slope:	(26b - 51)	-(26b - 50)																	
	(11.96 - 9.61)/185'	= 0.0084	1/'																	

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 4/6/99
Checked by _____ Date _____
Harkness Ave & Larkin St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
12		P	R	0.8	5.2	0.5	i @ 8.7min		1.25	3.3		0.0029	0.013	14.7"	Use	15"				
	Contributing areas	9, 10, 11,	12, 13									V_{15} = 2.8 fps								
	t_c from M to P:	8.7min																		
	t_c from O to P	4.2min										t_{pipe} = 210/2.8(60) =				1.3 min				
	Design Slope:	(26b-50)	-(26b-58)									t_c = 8.7	+1.3	= 10 min						
	(7.91-7.31)/210	= 0.0029	ft/ft																	
16		Q	R	0.9	0.9	0.5	i@	3.6min	1.81	0.8		0.0068	0.013	7.4"	Use	8"				
	Overland	90	0.85	fps			1.8					V_8 = 2.9 fps								
	Street	130	1.2	fps			1.8													
							3.6					t_{pipe} = 190/2.9(60) =				1.1 min				
	Street Slope:	Gutters	near (26b-59)									t_c = 3.6	+1.1	4.7 min						
	(14.05 13.40)/190	= 0.0038	ft/ft																	
	Design Slope:																			
	(10.46-9.16)/190	= 0.0068	ft/ft																	
14		R	I	0.6	6.7	0.5	i @	10min	1.14	3.8		0.0020	0.013	16.6"	Use	18"				
	Contributing areas	9, 10,	11, 12, 13	14, 16								V_{18} = 2.7 fps								
	t_c from P to R =	10min																		
	t_c from O to R =	4.7min	∴ governing t_c =	10min								t_{pipe} = 120/2.7(60) =				0.7 min				
	Design Slope:	(26b-58)	-(26b-61)									t_c = 10+	0.7	= 10.7 min						
	(7.31-7.07)/120	= 0.0020	ft/ft																	

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by AHC Date 1/6/92
Checked by _____ Date _____
Harkness Ave + Larkin St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
—		J	T	—	16.6	0.5	i @ 14.1	/min	0.96	8.0		0.0031	0.013	20.2"	Use 21"					
	Contributing Areas 1-	14, 16										V_{21} = 3.7 fps								
	t_c from H to J	= 14.1 min										t_{pipe} = 110/3.7(60) = 0.5 min								
	t_c from R to J =	10.7 min																		
	governing t_c =	14.1 min										t_c = 14.1 + 0.5 = 14.6 min								
	Design Slope:	(26c-61)	-(26c-69)																	
	(7.07-6.73)/110 =	0.0031 ft/ft																		
17		S	T	0.9	0.9	0.5	i @	3.3 min	1.87	0.8		0.0063	0.013	7.5"	Use 8"					
	Overland	90'	0.85	fps			1.8					V_8 = 2.8 fps								
	Street	150'	1.7	fps			1.5													
							3.3	min				t_{pipe} = 180/2.8(60) = 1.1 min								
	Street Slope:	Gutters	near (26c-68)																	
	(14.10-12.80)/180 =	0.0072 ft/ft										t_c = 3.3 + 1.1 = 4.4 min								
	Design Slope:	(26c-68)	-(26c-69)																	
	(9.99-8.85)/180 =	0.0063 ft/ft																		

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by SAC Date 1/6/92
Checked by _____ Date _____
Harkness Ave to Harkin St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
15		7	V	0.7	17.5	0.5	i @	14.6 min	0.94	8.2		0.0013	0.013	24.0"	Use 24"					
	Contributing Areas: 1-17											V_{24} = 2.6 fps								
	t_c from I to T = 14.6 min											t_{pipe} = 115/2.6(60) = 0.7 min								
	Design Slope: (26b-69) - (26b-70)											t_c = 14.6 + 0.7 = 15.3 min								
	(6.73-6.58)/115 = 0.0013 ft/ft																			
19		U	V	1.2	1.2	0.5	i @ 5.1 min		1.55	0.9		0.0048	0.013	8.2"	Use 10"					
	Overland	175'	0.85 fps				3.4					V_{10} = 2.8 fps								
	Street	100'	0.97 fps				1.7													
							5.1 min					t_{pipe} = 410/2.8(60) = 2.4 min								
	Street Slope: Gutters near (26b-71)											t_c = 5.1 + 2.4 = 7.5 min								
	(13.94-13.68)/110' = 0.0024 ft/ft																			
	Design Slope: (26b-71) - (26b-70)																			
	(10.41-8.45)/410 = 0.0048 ft/ft																			
—		V	X	—	18.7	0.5	i @	15.3 min	0.91	8.5		0.0024	0.013	21.7"	Use 24"					
	Contributing Areas 1-17, 19											V_{24} = 3.5 fps								
	t_c from T to V = 15.3 min											t_{pipe} = 100/3.5(60) = 0.5 min								
	Design Slope: (26b-70) - (26b-72)											t_c = 15.3 + 0.5 = 15.8 min								
	(6.58-6.34)/100 = 0.0024 ft/ft																			

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSD - Phase 3
Computed by: [illegible] Date: [illegible]
Checked by: _____ Date: _____
Harvness Ave Harvest St - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
20		W	X	1.2	1.2	0.5	i @	3.0 min	1.92	1.2		0.0083	0.013	8.3"	Use	10"				
	Overland	80'	0.85	fps			1.6					V_{10} = 3.7 fps								
	Street	150'	1.8	fps			1.4													
							3.0 min					t_{pipe} = 190/3.7(60) = 0.9 min								
	Street Slope:	Gutter near (26b-74) - Gutter near (26b-73)																		
	(14.18 - 12.64)/180 =	0.0086'/'										t_c = 3.0 + 0.9 = 3.9 min								
	Design Slope:	(26b-73) - (26b-72)																		
	(10.17 - 8.59)/190 =	0.0083'/'																		
23		Y	AA	1.2	1.2	0.5	i @	3.7 min	1.79	1.1		0.0063	0.013	8.4"	Use	10"				
	Overland	150	0.85	fps			2.9					V_{10} = 3.2 fps								
	Street	120	2.6	fps			0.8													
							3.7 min					t_{pipe} = 115/3.2(60) = 0.6 min								
	Street Slope:	Gutters near (26b-95)																		
	(16.23 - 14.75)/85 =	0.0174'/'										t_c = 3.7 + 0.6 = 4.3 min								
	Design Slope:	(26b-95) - (26b-105)																		
	(11.08 - 10.35)/115 =	0.0063'/'																		
24		Z	AA	1.8	1.8	0.5	i @	4.5 min	1.65	1.5		0.0003	0.013	16.7"	Use	18"				
	Overland	180	0.85	fps			3.5					V_{18} = 1.0 fps								
	Street	160	2.6	fps			1.0													
							4.5 min					t_{pipe} = 115/1(60) = 1.9 min								
	Street Slope:	No survey data																		
	Assume same slope as from Y to AA = 0.0174'/'											t_c = 4.5 + 1.9 = 6.4 min								
	Design Slope:	(26b-109) - (26b-105)																		
	(9.49 - 9.45)/115 =	0.0003'/'																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 4/6/92
Checked by _____ Date _____
Harkness Ave & Larkin St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
—		AA	BB	—	3.0	0.5	i @ 6	4 min	1.44	2.2		0.0032	0.013	12.4"	Use	15"	(Did not match Z to AA, the slope on this segment is not atypical)			
	t_c from Y to AA:	4.3 min																		
	t_c from Z to AA:	6.4 min										V_{15} =	3.0	fps						
	Design Slope	(26b-105)	-(26b-106)									t_{pipe} =	555/3	(60) =	5.1 min					
	(9.45-7.66)/555 =	0.0032	'/'									t_c =	6.4	+3.1	= 9.5 min					
26		CC	BB	2.6	2.6	0.5	i @	7.4 min	1.36	1.8		0.0045	0.013	10.8"	Use	12"				
	Overland	100	0.85	fps			2.0					V_{12} =	3.1	fps						
	Street	300	0.93	fps			5.4													
							7.4 min					t_{pipe} =	365/3.1	(60) =	2.0 min					
	Street Slope:	Gutter	near	(26b-109)	-(26b-108)															
	(12.35-11.76)/270 =	0.0022	'/'									t_c =	7.4	+2 =	9.4 min					
	Design Slope:	(26b-108)	-(26b-106)																	
	(9.32-7.66)/365 =	0.0045	'/'																	
—		BB	EE	—	5.6	0.5	i @ 9.5 min		1.18	3.3		0.0034	0.013	14.2"	Use	15"				
	t_c from CC to BB =	9.4 min										V_{15} =	3.1	fps						
	t_c from AA to BB:	9.5 min																		
	governing t_c:	9.5 min										t_{pipe} =	115/3.1	(60) =	0.6 min					
	Design Slope:	(26b-106)	-(26b-93)									t_c =	9.5	+0.6 =	10.1 min					
	(7.66-7.27)/115 =	0.0034																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by ZAC Date 1/6/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
25		DD	EE	2.1	2.1	0.5	i@	6.9 min	1.40	1.5		0.0031	0.013	10.8"	Use	12"				
	Overland	85	0.85	fps			1.7					V_{12} =	2.5	fps						
	Street	270	0.87	fps			5.2													
							6.9	min				t_{pipe} =	350/2.5(60) =			2.3 min				
	Street Slope:	Gutter near (26b-94)				- Gutter near (26b-94)														
	(12.83 - 12.24)/310 =	0.0019	'/'									t_c =	6.9 + 2.3 =			9.2 min				
	Design Slope:	(26b-94) - (26b-93)																		
	(9.31 - 8.21)/350 =	0.0031	'/'																	
27		EE	FF	2.6	10.3	0.5	i@ 10.1 min		1.14	5.9		0.0021	0.013	19.4"	Use	21"				
	Contributing Areas 23, 24, 25, 26, 27											V_{21} =	3.0	fps						
	t_c from DD to EE =	9.2 min										t_{pipe} =	240/3(60) =			1.3 min				
	t_c from BB to EE =	10.1 min										t_c =	10.1 + 1.3 =			11.4 min				
	governing t_c =	10.1 min																		
	Design Slope:	(26b-93) - (26b-83)																		
	(7.29 - 6.79)/240 =	0.0021	'/'																	
22		GG	FF	2.2	2.2	0.5	i@ 3.2 min		1.88	2.1		0.0057	0.013	10.9"	Use	12"				
	Overland	75'	0.85	fps			1.5					V_{12} =	3.4	fps						
	Street	230'	2.2	fps			1.7													
							3.2	min				t_{pipe} =	175/3.4(60) =			0.9 min				
	Street Slope:	Gutters near (26b-84)																		
	(15.58 - 12.63)/250 =	0.0118	'/'									t_c =	3.2 + 0.9 =			4.1 min				
	Design Slope:	(26b-84) - (26b-83)																		
	(9.4 - 8.41)/175 =	0.0057	'/'																	

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
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i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSD - Phase 3
Computed by: JHC Date: 1/6/92
Checked by: _____ Date: _____
Harkness Ave & Larkin St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
–		FF	II	–	12.5	0.5	i @	11.4 min	1.08	6.8		0.0029	0.013	19.3"	Use	21"				
	Contributing Areas 22-27											V_{21} = 3.6 fps								
	t_c from EE to FF = 11.4 min											t_{pipe} = 120/3.6(60) = 0.6 min								
	t_c from GG to FF 4.1 min																			
	governing t_c = 11.4 min											t_c = 11.4 + 0.6 = 12 min								
	Design Slope: (26b-83) - (26b-82)																			
	(6.77-6.42)/120 = 0.0029 ft/ft																			
21		HH	II	2.0	2.0	0.5	i @ 6.0 min		1.48	1.5		0.0041	0.013	10.2"	Use	12"				
	Overland	180'	0.85	fps			3.5					V_{12} = 2.9 fps								
	Street	300	1.97	fps			2.5													
							6.0					t_{pipe} = 550/60(2.9) = 3.2 min								
	Street Slope: Gutter near (26b-84) - Gutter near (26b-74)																			
	(15.58-14.02)/160 = 0.0098 ft/ft											t_c = 6 + 3.2 = 9.2 min								
	Design Slope: (26b-81) - (26b-82)																			
	(10.8-8.2)/435 = 0.0041 ft/ft																			
	435 + 115 = 550 for pipe length																			
–		II	X	–	14.5	0.5	i @ 12.0 min		1.05	7.6		0.0007	0.013	26.2"	Use	27"				
	Contributing Areas: 21-27											V_{27} = 2.1 fps								
	t_c from FF to JJ = 12 min											t_{pipe} = 115/2.1(60) = 0.9 min								
	Design Slope: (26b-82) - (26b-72)											t_c = 12.0 + 0.9 = 12.9 min								
	(6.[illegible] - 6.34)/115 = 0.0007																			

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
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i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/6/92
Checked by _____ Date _____
Harkness Ave + Larkin St - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope, ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
18		X	KK	2.9	37.3	0.5	i @ 15.8 min		0.91	17.0		0.0011	0.013	32.5"	Use	33'				
	t_c from JJ to X = 12.9 min											$V_{33'}$ = 3.0 fps								
	t_c from V to X = 15.8 min											t_{pipe} = 120/3.0(60) = 0.7 min								
	t_c from W to X = 3.9 min																			
	governing t_c = 15.8 min											t_c = 15.8 + 0.7 = 16.5 min								
	Contributing Areas 1-27																			
	Design Slope: (266-72) - (266-37)																			
	(6.35 - 6.22)/120 = 0.0011 ft/ft																			
28		JJ	KK	3.1	3.1	0.5	i @	9.8 min	1.12	1.7		0.0057	0.013	10.1"	Use	12"				
	Overland	80'	0.85 fps				1.6					V_{12} = 3.4 fps								
	Street	375'	0.76 fps				8.2													
							9.8					t_{pipe}: 590/3.4(60) = 2.9 min								
	Street Slope: Gutter near (266-31) - Gutter near (266-34)																			
	(13.61 - 13.06)/375 = 0.0015 ft/ft											t_c: 9.8 + 2.9 = 12.7 min								
	Design Slope: (266-38) - (266-37)																			
	(7.90 - 5.24)/465 = 0.0057 ft/ft																			
	465 + 125 = 590' for pipe length																			

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Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 1/6/92
Checked by _____ Date _____
Harkness Ave & Larkin St - 2year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
—		KK	LL	—	40.4	0.5	i @ 16.5 min		0.89	18.0		0.0022	0.013	29.2"	Use 33"					
	t_c from X to KK = 16.5 min											V_{33} = 4.2 fps								
	t_c from JJ to KK = 12.7 min																			
												t_{pipe} = 125/4.2(60) = 0.5 min								
	Design Slope: (266-37)-(266-31)																			
	(6.22-5.94)/125 =	0.0022'/'										t_c = 16.5 + 0.5 =				17. min				
29		LL	MM	3.6	44.0	0.5	i @ 17.0 min		0.88	19.4		0.0022	0.013	30.1"	Use 33"					
	Design Slope:	No survey data										V_{33} = 4.2 fps								
	Assume same slope as	KK to LL																		
	Slope = 0.0022											t_{pipe} = 115/60(4.2) = 0.5 min								
												t_c = 17 + 0.5 =				17.5 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by Date
Harkness Ave and Larkin St - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.0	1.0	0.5	i @	5.4 min	2.35	1.2		0.0032	0.013	9.9"	Use	10"				
												V_{10} =	2.3	fps						
												t_{pipe} =	170/60(2.3) =			1.2 min				
												t_c =	5.4 + 1.2 =			6.6 min				
2		B	C	1.5	2.5	0.5	i @	6.6 min	2.19	2.7		0.0032	0.013	13.4"	Use	15"				
	t_c from A to B =	6.6 min																		
												V_{15} =	3.0	fps						
												t_{pipe} =	220/60(3) =			1.2 min				
												t_c =	6.6 + 1.2 =			7.8 min				
3		C	D	1.8	4.3	0.5	i @	7.8 min	2.03	4.4		0.0032	0.013	16.1"	Use	18"				
	t_c from B to C =	7.8 min																		
												V_{18} =	3.4	fps						
												t_{pipe} =	195/60(3.4) =			1 min				
												t_c =	7.8 + 1 =			8.8 min				
4		D	E	1.1	5.4	0.5	i @	8.8 min	1.90	5.1		0.0032	0.013	17.0"	Use	18"				
	t_c from C to D =	8.8 min																		
												V_{18} =	3.4	fps						
												t_{pipe} =	140/3.4 (60) =			0.7 min				
												t_c =	8.8 + 0.7 =			9.5 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 7/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 5year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5		E	F	0.6	6.0	0.5	i@	9.5min	1.81	5.4		0.0032	0.013	17.3"	Use	18"				
	t_c from D to E =																			
												V_{18} =	3.4	fps						
												t_{pipe} =	160	/60(3.4)	= 0.8 min					
												t_c =	9.5	+ 0.8	= 10.3 min					
6		F	H	0.9	6.9	0.5	i@	10.3min	1.72	5.9		0.0032	0.013	17.9"	Use	18"				
	t_c from E to F =	10.3 min																		
												V_{18} =	3.4	fps						
												t_{pipe} =	265	/60(3.4) =	1.3 min					
												t_c =	10.3	+1.3	= 11.6 min					
7		G	H.	1.9	1.9	0.5	i@ 8.3min		1.96	1.9		0.0066	0.013	10.2"	Use	12"				
												V_{12} =	3.7	fps						
												t_{pipe} =	120	/60(3.7) =	0.5 min					
												t_c =	8.3 +	0.5	= 8.8 min					
8		H	I	1.1	9.9	0.5	i@ 11.6min		1.63	8.1		0.0033	0.013	20.1"	Use	21"				
	t_c from F to H =	11.6 min																		
	t_c from G to H =	8.8 min										V_{21} =	3.8	fps						
	governing t_c =	11.6 min										t_{pipe}	400	/60(3.8) =	1.8 min					
												t_c =	11.6	+1.8 =	13.4 min					

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by JAC Date 7/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
9		J	L	1.9	1.9	0.5	i@5.3 min		2.36	2.2		0.0053	0.013	11.3"	Use	12"				
												V_{12} =	3.3	fps						
												t_{pipe} = 120/60(3.3) =				0.6 min				
												t_c = 5.3 + 0.6 =				5.9 min				
10		K	L	0.98	0.98	0.5	i@5.0 min		2.40	1.2		0.0154	0.013	7.4"	Use	8"				
												V_8 =	4.3	fps						
												t_{pipe} = 70/60(4.3) =				0.3 min				
												t_c = 5.0 + 0.3 =				5.3 min				
—		L	M	—	2.9	0.5	i@ 5.9 min		2.28	3.3		0.0044	0.013	13.6"	Use	15"				
	t_c from J to L =	5.9 min										V_{15} =	3.5	fps						
	t_c from K to L =	5.3 min										t_{pipe} = 385/60(3.5) =				1.8 min				
	governing t_c =	5.9 min										t_c = 5.9 + 1.8 =				7.7 min				
11		N	M	0.52	0.52	0.5	i@4.7 min		2.52	0.7		0.0053	0.013	7.3"	Use	8"				
												V_8 =	2.5 fps							
												t_{pipe} = 105/60(2.5) =				0.7 min				
												t_c = 4.7 + 0.7 =				5.4 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

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t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO · Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
–		M	P	–	3.4	0.5	i @	7.7 min	2.04	3.5		0.0050	0.013	13.6"	Use	15"				
	t_c from L to M =	7.7 min																		
												V_{15} =	3.7 fps							
												t_{pipe} =	110/60(3.7) =			0.5 min				
												t_c =	7.7 + 0.5 =			8.2 min				
13		O	P	1.0	1.0	0.5	i @ 3	2 min	3.12	1.6		0.0084	0.013	9.2"	Use	10"				
												V_{10} =	3.7 fps							
												t_{pipe} =	185/60(3.7) =			0.8 min				
												t_c =	3.2 + 0.8 =			4.0 min				
12		P	R	0.8	5.2	0.5	i @ 8.2 min		1.98	5.1		0.0029	0.013	17.3"	Use	18"				
	t_c from M to P =	8.2 min																		
	t_c from O to P =	4.0 min										V_{18} =	3.2 fps							
	governing t_c =	8.2 min										t_{pipe} =	210/60(3.2) =			1.1 min				
												t_c =	8.2 + 1.1 =			9.3 min				
16		Q	R	0.9	0.9	0.5	i @ 3	6 min	2.96	1.3		0.0068	0.013	8.8"	Use	10"				
												V_{10} =	3.3 fps							
												t_{pipe} =	190/60(3.3) =			1 min				
												t_c =	3.6 + 1 =			4.6 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

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i = Rainfall intensity, in inches/hour
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V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project: Sacramento CSO - Phase 3
Computed by: LAC Date: 1/7/02
Checked by: _____ Date: _____
Harkness Ave and Larkin St - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
14		R	I	0.6	6.7	0.5	i @	9.3 min	1.83	6.1		0.0020	0.013	19.8	Use	21"				
	t_c from P to R =	9.3 min										V_{21} =	3.0	fps						
	t_c from Q to R =	4.6 min																		
	governing t_c =	9.3 min										t_{pipe} =	120 / 60(3)	=	0.7 min					
												t_c =	9.3	+ 0.7	=	10.0 min				
—		I	T	—	16.6	0.5	i @	13.4 min	1.51	12.5		0.0031	0.015	23.9"	Use	24"				
	t_c from H to I =	13.4 min										V_{24} =	4.0	fps						
	t_c from R to I =	10 min										t_{pipe} =	110 / 60(4)	=	0.5 min					
	governing t_c =	13.4 min										t_c =	13.4	+ 0.5	=	13.9 min				
17		S	T	0.9	0.9	0.5	i @	3.3 min	3.08	1.4		0.0023	0.013	9.2"	Use	10"				
												V_{10} =	3.2	fps						
												t_{pipe} =	180 / 60(3.2)	=	0.9 min					
												t_c =	3.3	+ 0.9	=	4.2 min				
15		T	V	0.7	17.5	0.5	i @	13.9 min	1.47	12.9		0.0013	0.013	28.5"	Use	30"				
	t_c from I to T =	13.9 min										V_{30} =	3.0	fps						
												t_{pipe} =	115 / 60(3)	=	0.6 min					
												t_c =	13.9	+ 0.6	=	14.5 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
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i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by JAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave + Larkin St - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope -S/L	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
19		U	V	1.2	1.2	0.5	i @ 5.1 min		2.39	1.4		0.0048	0.013	9.7"	Use	10"				
												V_{10} =	2.8 fps							
												t_{pipe} =	410/60(2.8) =			2.4 min				
												t_c =	5.1 + 2.4 =			7.5 min				
—		V	X	—	18.7	0.5	i @	14.5 min	1.43	13.4		0.0024	0.013	25.7"	Use	30"	to match		T to V	
	t_c from T to V =	14.5 min										V_{30} =	4.1 fps							
												t_{pipe} =	100/60(4.1) =			0.4 min				
												t_c =	14.5 + 0.4 =			14.9 min				
20		W	X	1.2	1.2	0.5	i @ 3.0 min		3.20	1.9		0.0083	0.013	9.8"	Use	10"				
												V_{10} =	3.7 fps							
												t_{pipe} =	190/60(3.7) =			0.9 min				
												t_c =	3 + 0.9 =			3.9 min				
23		Y	AA	1.2	1.2	0.5	i @ 3.7 min		2.92	1.8		0.0063	0.013	10.1"	Use	12"				
												V_{12} =	3.6 fps							
												t_{pipe} =	115/60(3.6) =			0.5 min				
												t_c =	3.7 + 0.5 =			4.2 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by RAC Date 4/7/02
Checked by _____ Date _____
Harkness Ave & Larkin St - 2 year

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope 3%	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
24		Z	AA	1.8	1.8	0.5	i@ 4.5 min		2.60	2.3		0.0003	0.013	19.6"	Use	21"				
												V_{21} =	1.1	fps						
												t_{pipe} =	115/60(1.1)	=	1.7 min					
												t_c =	4.5 +	1.7	= 6.2 min					
–		AA	BB	–	3.0	0.5	i@	6.2 min	2.2	3.4		0.0032	0.013	14.6"	Use	15"				
	t_c from Y to AA	4.2 min										V_{15} =	3.0 fps							
	t_c from Z to AA =	6.2 min										t_{pipe} =	555/60(3)	=	3.1 min					
	governing t_c = 6.2 min											t_c =	6.2 +	3.1	= 9.3 min					
26		CC	BB	2.6	2.6	0.5	i@ 7.4 min		2.08	2.7		0.0045	0.013	12.5"	Use	15"				
												V_{15} =	3.5	fps						
												t_{pipe} =	365/60(3.5)	=	1.7 min					
												t_c =	7.4 +	1.7	= 9.1 min					
–		BB	EE	–	5.6	0.5	i@	9.3 min	1.83	5.1		0.0034	0.013	16.8"	Use	18"				
	t_c from CC to BB =	9.1 min										V_{18} =	3.5	fps						
	t_c from AA to BB =	9.3 min										t_{pipe} =	115/60(3.5)	=	0.5 min					
	governing t_c =	9.3 min										t_c =	9.3 +	0.5	= 9.8 min					

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Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by AHC Date 1/7/92
Checked by Date
Harkness Ave + Larkin St. - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
25		DD	EE	2.1	2.1	0.5	i @ 6.9 min		2.15	2.3		0.0031	0.013	12.7"	Use	15"				
												V_{15} =	2.9	fps						
												t_{pipe} =	350/60(2.9) =			2.0 min				
												t_c =	6.9 + 2.0	min =		8.9 min				
27		EE	FF	2.6	10.3	0.5	i @	9.8 min	1.77	9.1		0.0021	0.013	22.8	Use	24"				
	t_c from DD to EE =	8.9 min																		
	t_c from BB to EE =	9.8 min										V_{24} =	3.3	fps						
	governing t_c =	9.8 min										t_{pipe} =	240/60(3.3) =			1.2 min				
												t_c =	9.8 + 1.2 =			11.0 min				
22		GG	FF	2.2	2.2	0.5	i @ 3.2 min		3.12	3.4		0.0059	0.013	13.1"	Use	15"				
												V_{15} =	4.0	fps						
												t_{pipe} =	175/60(4) =			0.7 min				
												t_c =	3.2 + 0.7 =			3.9 min				
—		FF	II	—	12.5	0.5	i @	11.0 min	1.67	10.4		0.0029	0.013	22.6"	Use	24"				
	t_c from EE to FF =	11.0 min										V_{24} =	3.9	fps						
	t_c from GG to FF =	3.9 min										t_{pipe} =	120/60(3.9) =			0.5 min				
	governing t_c =	11.0 min										t_c =	11 + 0.5 =			11.5 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave. + Larkin St - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
21		HH	II	2.0	2.0	0.5	i @ 6.0 min		2.27	2.3		0.0041	0.013	12.0"	Use	12"				
												V_{12} =	2.9	fps						
												t_{pipe} =	550	/60(2.9) =		3.2	min			
												t_c =	6.0	+ 3.2	=	9.2 min				
—		II	X	—	14.5	0.5	i @ 11.5 min		1.64	11.9		0.0007	0.013	31.0"	Use	33"				
	t_c from FF to II =	11.5 min										V_{33} =	2.4	fps						
												t_{pipe} =	115	/60(2.4) =		0.8	min			
												t_c =	11.5	+ 0.8	=	12.3 min				
18		X	KK	2.9	37.3	0.5	i @	14.9 min	1.41	26.3		0.0011	0.013	38.4"	Use	42"				
	t_c from II to X =	12.3 min										V_{42} =	3.5	fps						
	t_c from V to X =	14.9 min										t_{pipe} =	120	/60(3.5) =		0.6	min			
	t_c from W to X =	3.9 min										t_c =	14.9	+ 0.6	=	15.5 min				
	governing t_c =	14.9 min																		
85		JJ	KK	3.1	3.1	0.5	i @	9.8 min	1.77	2.7		0.0057	0.013	12.0"	Use	12"				
												V_{12} =	3.4	fps						
												t_{pipe} =	590	/60(3.4) =		2.9	min			
												t_c =	9.8	+ 2.9	=	12.7 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by AC Date 11/92
Checked by Date
Harkness Ave & Larkin St - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
—		KK	LL	—	40.4	0.5	i @	15.5 min	1.38	27.9		0.0022	0.013	34.5	" Use	42"	to match	X	to KK	
	t_c from X to KK =	15.5 min																		
	t_c from JJ to KK =	12.7 min										$V_{1/2}$ =	4.9	fps						
	governing t_c =	15.5 min										t_{pipe} =	125	/60(4.9) =	0.4	min				
												t_c =	15.5	+0.4	= 15.9	min				
29		LL	MM	3.6	44.0	0.5	i @	15.9 min	1.37	30.1		0.0022	0.013	35.4	" Use	42"	to match	KK	to LL	
	t_c from KK to LL =																			
												$V_{1/2}$ =	4.9	fps						
												t_{pipe} =	115	/60(4.9) =	0.4	min				
												t_c =	15.9	+0.4	= 16.3	min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 10year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.0	1.0	0.5	i@	5.4min	2.93	1.5		0.0032	0.013	10.7"	Use	12"				
												V_{12} =	2.6	fps						
												t_{pipe} = 170/60(2.6) =				1.1 min				
												t_c =	5.4	+ 1.1	=	6.5 min				
2		B	C	1.5	2.5	0.5	i@	6.5min	2.75	3.4		0.0032	0.013	14.6"	Use	15"				
	t_c from A to B =	6.5min										V_{15} =	3.0 fps							
												t_{pipe} = 220/60(3) =				1.2 min				
												t_c =	6.5	+ 1.2	=	7.7 min				
3		C	D	1.8	4.3	0.5	i@	7.7min	2.55	5.5		0.0032	0.013	17.5"	Use	18"				
	t_c from B to C =	7.7min										V_{18} =	3.4	fps						
												t_{pipe} = 195/60(3.4) =				1 min				
												t_c =	7.7	+ 1	=	8.7 min				
4		D	E	1.1	5.4	0.5	i@	8.7min	2.38	6.4		0.0032	0.013	18.5"	Use	21"				
	t_c from C to D =	8.7min										V_{21} =	3.7	fps						
												t_{pipe} = 140/60(3.7) =				0.6 min				
												t_c =	8.7	+ 0.6	=	9.3 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by ZAC Date 7/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5		E	F	0.6	6.0	0.5	i@	9.3 min	2.28	6.8		0.0032	0.013	18.9"	Use	21"				
	t_c from D to E =	9.3 min																		
												V_{21} =	3.7	fps						
												t_{pipe} =	160	/60	(3.7) =	0.7 min				
												t_c =	9.3	+ 0.7	= 10 min					
6		F	H	0.9	6.9	0.5	i@	10 min	2.16	7.5		0.0032	0.013	19.6"	Use	21"				
	t_c from E to F =	10 min																		
												V_{21} =	3.7	fps						
												t_{pipe} =	265	/60	(3.7) =	1.2 min				
												t_c =	10 +	1.2 =	11.2 min					
7		G	H	1.9	1.9	0.5	i@ 8.3 min		2.45	2.3		0.0066	0.013	11.0"	Use	12"				
												V_{12} =	3.7	fps						
												t_{pipe} =	120	/60	(3.7) =	0.5 min				
												t_c =	8.3 +	0.5 =	8.8 min					
8		H	I	1.1	9.9	0.5	i@ 11.2 min		2.05	10.1		0.0033	0.013	21.8"	Use	24"				
	t_c from F to H =	11.2 min																		
	t_c from G to H =	8.8 min										V_{24} =	4.1	fps						
	governing t_c =	11.2 min										t_{pipe}	400	/60	(4.1) =	1.6 min				
												t_c =	11.2	+ 1.6	= 12.8 min					

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

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V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by JAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
9		J	L	1.9	1.9	0.5	i@5.3 min		2.95	2.8		0.0053	0.013	12.3"	Use	15"				
												V_{15} =	3.8 fps							
												t_{pipe} =	120/60(3.8) =			0.5 min				
												t_c =	5.3 + 0.5 =			5.8 min				
10		K	L	0.98	0.98	0.5	i@5.0 min		3.00	1.5		0.0154	0.013	8.0"	Use	8"				
												V_8 =	4.3 fps							
												t_{pipe} =	70/60(4.3) =			0.3 min				
												t_c =	5.0 + 0.3 =			5.3 min				
—		L	M	—	2.9	0.5	i@ 5.8 min		2.87	4.2		0.0044	0.013	14.9"	Use	15"				
	t_c from J to L =	5.8 min										V_{15} =	3.5 fps							
	t_c from K to L =	5.3 min										t_{pipe} =	385/60(3.5) =			1.8 min				
	governing t_c =	5.8 min										t_c =	5.8 + 1.8 =			7.6 min				
11		N	M	0.52	0.52	0.5	i@4.7 min		3.18	0.8		0.0053	0.013	7.7"	Use	8"				
												V_8 =	2.5 fps							
												t_{pipe} =	102/2.5(60) =			0.7 min				
												t_c =	4.7 + 0.7 =			5.4 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
–		M	P	–	3.4	0.5	i @	7.6 min	2.56	4.4		0.0050	0.013	14.8"	Use	15"				
	t_c from L to M =	7.6 min																		
												V_{15} =	3.7 fps							
												t_{pipe} =	110/60(3.7) =			0.5 min				
												t_c =	7.6	+ 0.5	= 8.1	min				
13		O	P	1.0	1.0	0.5	i @ 3	2 min	4.08	2.0		0.0084	0.013	10.0"	Use	10"				
												V_{10} =	3.7	fps						
												t_{pipe} =	185/60	(3.7) =		0.8 min				
												t_c =	3.2	+ 0.8	= 4.0	min				
12		P	R	0.8	5.2	0.5	i @	8.1 min	2.48	6.4		0.0029	0.013	18.8"	Use	21"				
	t_c from M to P =	8.1 min										V_{21} =	3.6 fps							
	t_c from O to P =	4.0 min										t_{pipe} =	210/60	(3.6) =		1 min				
	governing t_c =	8.1 min										t_c =	8.1	+ 1 =	9.1	min				
16		Q	R	0.9	0.9	0.5	i @ 3	6 min	3.84	1.7		0.0068	0.013	9.8"	Use	10"				
												V_{10} =	3.3	fps						
												t_{pipe} =	190/60	(3.3) =		1 min				
												t_c =	3.6	+ 1 =	4.6	min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ~~%~~	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
14		R	I	0.6	6.7	0.5	i@	9.1 min	2.31	7.7		0.0020	0.013	21.6"	Use	24"				
	t_c from P to R =	9.1 min																		
	t_c from Q to R =	4.6 min										V_{24} =	3.2	fps						
	governing t_c =	9.1 min										t_{pipe} =	120/60(3.2) =			0.6 min				
												t_c =	9.1 +	0.6 =		9.7 min				
—		I	T	—	16.6	0.5	i@	12.8 min	1.91	15.9		0.0031	0.015	26.2"	Use	27"				
	t_c from H to J =	12.8 min										V_{27} =	4.3	fps						
	t_c from R to I =	9.7 min										t_{pipe} =	110/60(4.3) =			0.4 min				
	governing t_c =	12.8 min										t_c =	12.8 + 0.4 =			13.2 min				
17		S	T	0.9	0.9	0.5	i@	3.3 min	4.02	1.8		0.0063	0.013	10.1"	Use	12"				
												V_{12} =	3.6	fps						
												t_{pipe} =	180/60(3.6) =			0.8 min				
												t_c =	3.3 + 0.8 =			4.1 min				
15		T	V	0.7	17.5	0.5	i@	13.2 min	1.88	16.5		0.0013	0.013	31.2"	Use	33"				
	t_c from I to T =	13.2 min										V_{33} =	3.2	fps						
												t_{pipe} =	115/60(3.2) =			0.6 min				
												t_c =	13.2 + 0.6 =			13.8 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

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t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by JAC Date 1/7/92
Checked by Date
Harkness Ave & Larkin St - 10 year

Drainage Area Number	Location			A		C	Time of Concentration		I	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
19		U	V	1.2	1.2	0.5	i @ 5.1 min		2.98	1.8		0.0048	0.013	10.6"	Use	12"				
												V_{12} =	3.2 fps							
												t_{pipe} =	410/60(3.2) =			2.1 min				
												t_c =	5.1 +	2.1 =		7.2 min				
—		V	X	—	18.7	0.5	i @	13.8 min	1.83	17.1		0.0024	0.013	28.2"	Use	33"				
	t_c from T to V =	13.8 min										V_{33} =	4.4	fps						
												t_{pipe} =	100/60(4.4) =			0.4 min				
												t_c =	13.8	+0.4 =		14.2 min				
20		W	X	1.2	1.2	0.5	i @ 3.0 min		4.20	2.5		0.0083	0.013	11.0"	Use	12"				
												V_{12} =	4.1 fps							
												t_{pipe} =	190/60(4.1) =			0.8 min				
												t_c =	3 +	0.8 =		3.8 min				
23		Y	AA	1.2	1.2	0.5	i @ 3.7 min		3.78	2.3		0.0063	0.013	11.1"	Use	12"				
												V_{12} =	3.6 fps							
												t_{pipe} =	115/60(3.6) =			0.5 min				
												t_c =	3.7	+0.5 =		4.2 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave + Larkin St - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
24		Z	AA	1.8	1.8	0.5	i@4.5min		3.30	3.0		0.0003	0.013	21.7"	Use	24"				
												V_{24} =	1.3	fps						
												t_{pipe} =	115/60(1.3) =			1.5 min				
												t_c =	4.5 +	1.5 =	6.0 min					
–		AA	BB	–	3.0	0.5	i@	6.0min	2.83	4.2		0.0032	0.013	15.8"	Use	18"				
	t_c from Y to AA = 4.2min											V_{18} =	3.4	fps						
	t_c from Z to AA = 6.0min											t_{pipe} =	555/60(3.4) =			2.7 min				
	governing t_c = 6.0 min											t_c =	6 +	2.7 =	8.7 min					
26		CC	BB	2.6	2.6	0.5	i@9.4min		2.60	3.4		0.0045	0.013	13.7"	Use	15"				
												V_{15} =	3.5	fps						
												t_{pipe} =	365/60(3.5) =			1.7 min				
												t_c =	9.4 +	1.7 =	9.1 min					
–		BB	EE	–	5.6	0.5	i@	9.1min	2.31	6.5		0.0034	0.013	18.4"	Use	21"				
	t_c from CC to BB = 9.1min											V_{21} =	3.9	fps						
	t_c from AA to BB = 8.7min											t_{pipe} =	115/60(3.9) =			0.5 min				
	governing t_c = 9.1min											t_c =	9.1 +	0.5 =	9.6 min					

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Computation Form
Drainage and Storm Sewer Design
Rational Method

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n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by AHC Date 1/7/92
Checked by _____ Date _____
Harkness Ave + Larkin St. - 10year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
25		DD	EE	2.1	2.1	0.5	i@ 6	9min	2.68	2.8		0.0031	0.013	13.6	Use	15"				
												V_{15} =	2.9	fps						
												t_{pipe} =	350/60(2.9) =			2.0 min				
												t_c =	6.9 + 2.0 =			8.9 min				
27		EE	FF	2.6	10.3	0.5	i @	9.6min	2.23	11.5		0.0021	0.013	24.9"	Use	27"				
	t_c from DD to EE =	8.9 min										V_{27} =	3.6	fps						
	t_c from BB to EE =	9.6 min										t_{pipe} =	240/60(3.6) =			1.1 min				
	governing t_c =	9.6 min										t_c =	9.6 + 1.1 =			10.7 min				
22		GG	FF	2.2	2.2	0.5	i @ 3	2min	4.08	4.5		0.0057	0.013	14.5"	Use	15"				
												V_{15} =	4.0 fps							
												t_{pipe} =	175/60(4) =			0.7 min				
												t_c =	3.2 + 0.7 =			3.9 min				
—		FF	II	—	12.5	0.5	i @ 1	0.7min	2.10	13.1		0.0029	0.013	24.6	Use	27"				
	t_c from EE to FF =	10.7 min										V_{27} =	4.2	fps						
	t_c from GG to FF =	3.9 min										t_{pipe}	120/60(4.2) =			0.5 min				
	governing t_c =	10.7 min										t_c =	10.7 + 0.5 =			11.2 min				

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Computation Form
Drainage and Storm Sewer Design
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t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave. & Larkin St - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
21		HH	II	2.0	2.0	0.5	i @ 6	0 min	2.83	2.8		0.0041	0.013	12.9"	Use	15"				
												V_{15} =	3.4 fps							
												t_{pipe} =	550	/60(3.4) =		2.7 min				
												t_c =	6.0	+ 2.7 =		8.7 min				
—		II	X	—	14.5	0.5	i @ 11	2 min	2.05	14.9		0.0007	0.013	33.8"	Use	36"				
	t_c from FF to II =	11.2 min										V_{36} =	2.5 fps							
												t_{pipe} =	115	/60(2.5) =		0.8 min				
												t_c =	11.2	+ 0.8 =		12.0 min				
18		X	KK	2.9	37.3	0.5	i @	14.2 min	1.79	33.4		0.0011	0.013	42.0"	Use	42"				
	t_c from II to X =	12.0 min										V_{42} =	3.5 fps							
	t_c from V to X =	14.2 min										t_{pipe} =	120	/60(3.5) =		0.6 min				
	t_c from W to X =	3.8 min										t_c =	14.2	+ 0.6 =		14.8 min				
	governing t_c =	14.2 min																		
29		JJ	KK	3.1	3.1	0.5	i @	9.8 min	2.19	3.4		0.0057	0.013	13.1"	Use	15"				
												V_{15} =	4.0 fps							
												t_{pipe} =	590	/60(4) =		2.5 min				
												t_c =	9.8	+ 2.5 =		12.3 min				

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Project Sacramento CSO - Phase 3
Computed by JAC Date 7/7/92
Checked by Date
Harkness Ave + Larkin St - 10year

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
—		KK	LL	—	40.4	0.5	i @	14.8 min	1.74	35.1		0.0022	0.013	37.6"	Use	42"				
	t_c from X to KK = 14.8 min																			
	t_c from JJ to KK = 12.8 min											V_{42} =	4.9	fps						
	governing t_c = 14.8 min											t_{pipe} =	125 / 60 (4.9) =			0.4	min			
												t_c =	14.8	+ 0.4 =		15.2 min				
29		LL	MM	3.6	44.0	0.5	i @	15.2 min	1.71	37.6		0.0022	0.013	38.5"	Use	42"				
	t_c from KK to LL											V_{42} =	4.9	fps						
												t_{pipe} =	115 / 60 (4.9) =			0.4	min			
												t_c =	15.2	+ 0.4 =		15.6 min				

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Computation Form
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Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	B	1.0	1.0	0.5	i@	5.4min	3.75	1.9		0.0032	0.013	11.7'	Use	12"				
												V_{12} =	2.6	fps						
												t_{pipe} =	170/60(2.6) =			1.1min				
												t_c =	5.4	+ 1.1	= 6.5	min				
2		B	C	1.5	2.5	0.5	i@	6.5min	3.52	4.4		0.0032	0.013	16.1"	Use	18"				
	t_c from A to B =	6.5min																		
												V_{18} =	3.4fps							
												t_{pipe} =	220/60(3.4) =			1.1min				
												t_c =	6.5	+1.1 =	7.6	min				
3		C	D	1.8	4.3	0.5	i@	7.6min	3.28	7.1		0.0032	0.013	19.2"	Use	21"				
	t_c from B to C =	7.6min																		
												V_{21} =	3.7	fps						
												t_{pipe} =	195/60(3.7) =			0.9min				
												t_c =	7.6	+0.9 =	8.5	min				
4		D	E	1.1	5.4	0.5	i@	8.5min	3.08	8.3		0.0032	0.013	20.3"	Use	21"				
	t_c from C to D =	8.5min																		
												V_{21} =	3.7	fps						
												t_{pipe} =	140/3.7(60) =			0.6min				
												t_c =	8.5	+0.6 =	9.1	min				

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Computation Form
Drainage and Storm Sewer Design
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Project Sacramento CSO - Phase 3
Computed by JAC Date 7/7/92
Checked by Date
Harkness Ave and Larkin St - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
5		E	F	0.6	6.0	0.5	i@	9.1 min	2.95	8.9		0.0032	0.013	20.9"	Use	21"				
	t_c from D to E =	9.1 min										V_{21} =	3.7	fps						
												t_{pipe} =	160	/60	(3.7) =	0.7	min			
												t_c =	9.1	+ 0.7	=	9.8 min				
6		F	H	0.9	6.9	0.5	i@	9.8 min	2.80	9.7		0.0032	0.013	21.6"	Use	24"				
	t_c from E to F =	9.8 min										V_{27} =	4.1	fps						
												t_{pipe} =	265	/60	(4.1) =	1.1	min			
												t_c =	9.8	+ 1.1 =		10.9 min				
7		G	H	1.9	1.9	0.5	i@ 8.3 min		3.13	3.0		0.0066	0.013	12.2"	Use	15"				
												V_{15} =	4.3	fps						
												t_{pipe} =	120	/60	(4.3) =	0.5 min				
												t_c =	8.3 +	0.5 =		8.8 min				
8		H	I	1.1	9.9	0.5	i@ 10.9 min		2.66	13.2		0.0033	0.013	24.1"	Use	27"				
	t_c from F to H =	10.9 min										V_{27} =	4.1	fps						
	t_c from G to H =	8.8 min										t_{pipe}	400	/60	(4.1) =	1.6	min			
	governing t_c =	10.9 min										t_c =	10.9	+ 1.6	=	12.5 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
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n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by JAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 25 year

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope ft/ft	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
9		J	L	1.9	1.9	0.5	i@5.3 min		3.78	3.6		0.0053	0.013	13.6"	Use	15"				
												V_{15} =	3.8 fps							
												t_{pipe} =	120/60(3.8) =			0.5 min				
												t_c =	5.3 + 0.5 =			5.8 min				
10		K	L	0.98	0.98	0.5	i@5.0 min		3.84	1.9		0.0154	0.013	8.7"	Use	10"				
												V_{10} =	5.0 fps							
												t_{pipe} =	70/60(5) =			0.2 min				
												t_c =	5.0 + 0.2 =			5.2 min				
—		L	M	—	2.9	0.5	i@	5.8 min	3.67	5.3		0.0044	0.013	16.2"	Use	18"				
	t_c from J to L =	5.8 min																		
	t_c from K to L =	5.2 min																		
	governing t_c =	5.8 min										V_{18} =	4.0 fps							
												t_{pipe} =	385/60(4) =			1.6 min				
												t_c =	5.8 + 1.6 =			7.4 min				
11		N	M	0.52	0.52	0.5	i@4.7 min		4.09	1.1		0.0053	0.013	8.7"	Use	10"				
												V_{10} =	2.9 fps							
												t_{pipe} =	105/2.9(60) =			0.6 min				
												t_c =	4.7 + 0.6 =			5.3 min				

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Computation Form
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Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
—		M	P	—	3.4	0.5	i @	7.4 min	3.32	5.6		0.0050	0.013	16.2"	Use	18"				
	t_c from L to M =	7.4 min																		
												V_{18} =	4.2 fps							
												L_{pipe} =	110/60(4.2) =			0.4 min				
												t_c =	7.4	+ 0.4	= 7.8	min				
13		O	P	1.0	1.0	0.5	i @ 3	2 min	5.32	2.7		0.0084	0.013	11.2"	Use	12"				
												V_{12} =	4.2	fps						
												t_{pipe} =	185/60 (4.2) =			0.7 min				
												t_c =	3.2	+ 0.7	= 3.9	min				
12		P	R	0.8	5.2	0.5	i @	7.8 min	3.24	8.4		0.0029	0.013	20.9"	Use	21"				
	t_c from M to P =	7.8 min																		
	t_c from O to P =	3.9 min										V_{21} =	3.6 fps							
	governing t_c =	7.8 min										t_{pipe} =	210/60 (3.6) =			1 min				
												t_c =	7.8	+ 1 =	8.8	min				
16		Q	R	0.9	0.9	0.5	i @ 3	6 min	4.99	2.2		0.0068	0.013	10.8"	Use	12"				
												V_{12} =	3.8	fps						
												t_{pipe} =	190/60 (3.8) =			0.8 min				
												t_c =	3.6	+ 0.8	= 4.4 min					

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Computation Form
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Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave and Larkin St - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
14		R	I	0.6	6.7	0.5	i @	8.8 min	3.02	10.1		0.0020	0.013	23.9"	Use	24"				
	t_c from P to R =	8.8 min										V_{24} =	3.2	fps						
	t_c from Q to R =	4.4 min										t_{pipe} =	120/60(3.2) =			0.6 min				
	governing t_c =	8.8 min										t_c =	8.8 + 0.6 =			9.4 min				
—		I	T	—	16.6	0.5	i @	12.5 min	2.48	20.6		0.0031	0.015	28.8"	Use	30"				
	t_c from H to J =	12.5 min										V_{30} =	4.7	fps						
	t_c from R to I =	9.4 min										t_{pipe} =	110/60(4.7) =			0.4 min				
	governing t_c =	12.5 min										t_c	= 12.5 + 0.4 =			12.9 min				
17		S	T	0.9	0.9	0.5	i @	3.3 min	5.23	2.4		0.0013	0.013	11.3"	Use	12"				
												V_{12} =	3.6	fps						
												t_{pipe} =	180/60(3.6) =			0.8 min				
												t_c =	3.3 + 0.8 =			4.1 min				
15		T	V	0.7	17.5	0.5	i @	12.9 min	2.44	[illegible]		0.0013	0.013	34.4"	Use	36"				
	t_c from I to T =	12.9 min										V_{36} =	3.4	fps						
												t_{pipe} =	115/60(3.4) =			0.6 min				
												t_c =	12.9 + 0.6 =			13.5 min				

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Project Sacramento CSO - Phase 3
Computed by JAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave & Larkin St - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
19		U	V	1.2	1.2	0.5	i @ 5.1min		3.82	2.3		0.0048	0.013	11.7"	Use	12"				
												V_{12} =	3.2	fps						
												t_{pipe} =	410/60(3.2) =			2.1 min				
												t_c =	5.1	+ 2.1	= 7.2 min					
—		V	X	—	18.7	0.5	i @	13.5min	2.37	22.2		0.0024	0.013	31.1"	Use	36"	to match T to V			
	t_c from T to V =	13.5min																		
												V_{36} =	4.6	fps						
												t_{pipe} =	100/60(4.6) =			0.4 min				
												t_c =	13.5	+ 0.4	= 13.9min					
20		W	X	1.2	1.2	0.5	i @ 3.0min		5.48	3.3		0.0083	0.013	12.1"	Use	15"				
												V_{15} =	4.8	fps						
												t_{pipe} =	190/60(4.8) =			0.7 min				
												t_c =	3 +	0.7 =	3.7 min					
23		Y	AA	1.2	1.2	0.5	i @ 3.7min		4.91	2.9		0.0063	0.013	12.1"	Use	15"				
												V_{15} =	4.2	fps						
												t_{pipe} =	115/60(4.2) =			0.5 min				
												t_c =	3.7	+ 0.5	= 4.2 min					

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave + Larkin St - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
24		Z	AA	1.8	1.8	0.5	i@ 4.5 min		4.25	3.8		0.0003	0.013	23.7"	Use	24"				
												V_{24} =	1.3	fps						
												t_{pipe} =	115/60(1.3) =			1.5 min				
												t_c =	4.5 + 1.5 =			6.0 min				
–		AA	BB	–	3.0	0.5	i@	6.0 min	3.62	5.4		0.0032	0.013	17.3"	Use	18"				
	t_c from Y to AA = 4.2 min											V_{18} =	3.4	fps						
	t_c from Z to AA = 6.0 min											t_{pipe} =	555/60(3.4) =			2.7 min				
	governing t_c = 6.0 min											t_c =	6.0 + 2.7 =			8.7 min				
26		CC	BB	2.6	2.6	0.5	i@ 7.4 min		3.32	4.3		0.0045	0.013	14.9"	Use	15"				
												V_{15} =	3.5	fps						
												t_{pipe} =	365/60(3.5) =			1.7 min				
												t_c =	7.4 + 1.7 =			9.1 min				
–		BB	EE	–	5.6	0.5	i@	9.1 min	2.95	8.3		0.0034	0.013	20.2"	Use	21"				
	t_c from CC to BB = 9.1 min											V_{21} =	3.9	fps						
	t_c from AA to BB = 8.7 min											t_{pipe} =	115/60(3.9) =			0.5 min				
	governing t_c = 9.1 min											t_c =	9.1 + 0.5 =			9.6 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

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t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by AHC Date 1/7/92
Checked by Date
Harkness Ave + Larkin St. - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
25		DD	EE	2.1	2.1	0.5	i @ 6.9 min		3.43	3.6		0.0031	0.013	15.0	Use	15"				
												V_{15} =	2.9	fps						
												t_{pipe} =	350/60(2.9) =			2 min				
												t_c =	6.9	+ 2 =	8.9 min					
27		EE	FF	2.6	10.3	0.5	i @	9.6 min	2.85	14.7		0.0021	0.013	27.3"	Use	30"				
	t_c from DD to EE =	8.9 min																		
	t_c from BB to EE =	9.6 min										V_{30} =	3.8	fps						
	governing t_c =	9.6 min										t_{pipe} =	240/60(3.8) =			1.1 min				
												t_c =	9.6	+ 1.1 =	10.7 min					
22		GG	FF	2.2	2.2	0.5	i @ 3.2 min		5.32	5.9		0.0057	0.013	16.1"	Use	18"				
												V_{18} =	4.5	fps						
												t_{pipe} =	175/60(4.5) =			0.6 min				
												t_c =	3.2	+ 0.6 =	3.8 min					
—		FF	II	—	12.5	0.5	i @	10.7 min	2.68	16.8		0.0029	0.013	27.0"	Use	30"	to match EE to FF			
	t_c from EE to FF =	10.7 min																		
	t_c from GG to FF =	3.8 min										V_{30} =	4.5	fps						
	governing t_c =	10.7 min										t_{pipe}	120/60(4.5) =			0.4 min				
												t_c =	10.7	+ 0.4	= 11.1 min					

Computation Form
Drainage and Storm Sewer Design
Rational Method

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Project Sacramento CSO - Phase 3
Computed by LAC Date 1/7/92
Checked by _____ Date _____
Harkness Ave & Larkin St - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
21		HH	II	2.0	2.0	0.5	i @ 6.0 min		3.62	3.6		0.0041	0.013	14.2"	Use	15"				
												V_{15} = 3.4 fps								
												t_{pipe} = 550/60(3.4) = 2.7 min								
												t_c = 6.0 + 2.7 = 8.7 min								
—		II	X	—	14.5	0.5	i @ 11.1 min		2.64	19.1		0.0007	0.013	37.0"	Use	42"				
	t_c from FF to II = 11.1 min											V_{42} = 2.8 fps								
												t_{pipe} = 115/60(2.8) = 0.7 min								
												t_c = 11.1 + 0.7 = 11.8 min								
18		X	KK	2.9	37.3	0.5	i @ 13.9 min		2.32	43.3		0.0011	0.013	46.3"	Use	48"				
	t_c from II to X = 11.8 min											V_{48} = 3.8 fps								
	t_c from V to X = 13.9 min											t_{pipe} = 120/60(3.8) = 0.5 min								
	t_c from W to X = 3.7 min											t_c = 13.9 + 0.5 = 14.4 min								
	governing t_c = 13.9 min																			
28		JJ	KK	3.1	3.1	0.5	i @ 9.8 min		2.80	4.3		0.0057	0.013	14.3"	Use	15"				
												V_{15} = 4.0 fps								
												t_{pipe} = 590/60(4) = 2.5 min								
												t_c = 9.8 + 2.5 = 12.3 min								

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Computation Form
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Project Sacramento CSO - Phase 3
Computed by ZAC Date 1/7/92
Checked by Date
Harkness Ave + Larkin St - 25 year

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope ft/ft	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
—		KK	LL	—	40.4	0.5	i @	14.4 min	2.27	45.9		0.0022	0.013	41.5	"Use	48"	to match		X to	KK
	t_c from X to KK =	14.4 min																		
	t_c from JJ to KK =	12.3 min										V_{48} =	5.4	fps						
	governing t_c = 14.4 min											t_{pipe} =	125/60	(5.4) =		0.4	min			
												t_c =	14.4	+0.4 =		14.8 min				
29		LL	MM	3.6	44.0	0.5	i @	14.8 min	2.22	48.8		0.0022	0.013	42.5	"Use	48"				
												V_{48} =	5.4	fps						
												t_{pipe} =	118/60	(5.4) =		0.4	min			
												t_c =	14.8	+0.4 =		15.2 min				

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Project Sacramento CSO - Phase 3
Computed by JAC Date 7/6/92
Checked by _____ Date _____
Harkness Ave + Larkin St - Summary

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
				2 year			5 year				10 year			25 year						
		Slope	Length	Discharge	Dia ϕ		Discharge	Dia ϕ			Discharge	Dia ϕ		Discharge	Dia ϕ					
Existing ϕ	Reach	(ft/ft)	(ft)	(cfs)	(in)		(cfs)	(in)			(cfs)	(in)		(cfs)	(in)					
8	A to B	0.0032	170	0.8	10		1.2	10			1.5	12		1.9	12					
8	B to C	0.0032	220	1.8	12		2.7	15			3.4	15		4.4	18					
8	C to D	0.0032	195	2.8	15		4.4	18			5.5	18		7.1	21					
8	D to E	0.0032	140	3.3	15		5.1	18			6.4	21		8.3	21					
8	E to F	0.0032	160	3.5	15		5.4	18			6.8	21		8.9	21					
8/10	F to H	0.0032	265	3.8	18		5.9	18			7.5	21		9.7	24					
8	G to H	0.0066	120	1.2	10		1.9	12			2.3	12		3.0	15					
10	H to J	0.0033	400	5.2	18		8.1	21			10.1	24		13.2	27					
8	J to L	0.0053	120	1.5	10		2.2	12			2.8	15		3.6	15					
8	K to L	0.0154	70	0.8	8		1.2	8			1.5	8		1.9	10					
8	L to M	0.0047	385	2.1	12		3.3	15			4.2	15		5.3	18					
8	N to M	0.0053	105	0.4	8		0.7	8			0.8	8		1.1	10					
8	M to P	0.0050	110	2.2	12		3.5	15			4.4	15		5.6	18					
8	O to P	0.0084	185	0.9	8		1.6	10			2.0	10		2.7	12					
8/10	P to R	0.0029	210	3.3	15		5.1	18			6.4	21		8.4	21					
8	Q to R	0.0068	190	0.8	8		1.3	10			1.7	10		2.2	12					
12	R to I	0.0020	120	3.8	18		6.1	21			7.7	24		10.1	24					
12	I to T	0.0031	110	8.0	21		12.5	24			15.9	27		20.6	30					
8	S to T	0.0063	180	0.8	8		1.4	10			1.8	12		2.4	12					
12	T to V	0.0013	115	8.2	24		12.9	30			16.5	33		21.4	36					
8	U to V	0.0048	410	0.9	10		1.4	10			1.8	12		2.3	12					
12	V to X	0.0024	100	8.5	24		13.4	30			17.1	33		22.2	36					
8	W to X	0.0083	190	1.2	10		1.9	10			2.5	12		3.3	15					
8	Y to AA	0.0063	115	1.1	10		1.8	12			2.3	12		2.9	15					
8	Z to AA	0.0003	115	1.5	18		2.3	21			3.0	24		3.8	24					

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n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by ZAC Date 1/6/92
Checked by _____ Date _____
Harkness Ave + Larkin St - Summary

Drainage Area Number	Location			A		C	Time of Concentration		i	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
				2 year			5 year				10 year			25 year						
Existing ∅	Reach	Slope (ft/ft)	Length (ft)	Discharge (cfs)	Dia ∅ (in)		Discharge (cfs)	Dia ∅ (in)			Discharge (cfs)	Dia ∅ (in)		Discharge (cfs)	Dia ∅ (in)					
8	AA to BB	0.0032	555	2.2	15		3.4	15			4.2	18		5.4	18					
8	CC to BB	0.0045	365	1.8	12		2.7	15			3.4	15		4.3	15					
10	BB to EE	0.0034	115	3.3	15		5.1	18			6.5	21		8.3	21					
8	DD to EE	0.0031	350	1.5	12		2.3	15			2.8	15		3.6	15					
10/12	EE to FF	0.0021	240	5.9	21		9.1	24			11.5	27		14.7	30					
8	GG to FF	0.0057	175	2.1	12		3.4	15			4.5	15		5.9	18					
12	FF to II	0.0029	120	6.8	21		10.4	24			13.1	27		16.8	30					
10	HH to II	0.0041	550	1.5	12		2.3	12			2.8	15		3.6	15					
12	II to X	0.0007	115	7.6	27		11.9	33			14.9	36		19.1	42					
16	X to KK	0.0011	120	17.0	33		26.3	42			33.4	42		43.3	48					
8	JJ to KK	0.0057	590	1.7	12		2.7	12			3.4	15		4.3	15					
18	KK to LL	0.0022	125	18.0	33		27.9	42			35.1	42		45.9	48					
18	LL to MM	0.0022	115	19.4	33		30.1	42			37.6	42		48.8	48					

Representative Basin No. 6

56th Street and Broadway

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 12/31/91
Checked by _____ Date _____
56th Street and Broadway - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	C	0.9	0.9	0.5	i @	4.6 min	1.63	0.7		0.0047	0.013	7.5"	Use	8"				
	Overland	80	0.85	fps			1.6					V_8 =	2.4	fps						
	Street	160	0.88	fps			3.0													
							4.6	min				t_{pipe} =	210/	2.4(60)	= 1.5 min					
	Street Slope:	Gutter near (30a-58) - Gutter near (30a-64)																		
	(31.19-30.88)/160=	0.002'/'										t_c =	4.6 +	1.5 =	6.1 min					
	Design Slope:	(30a-64)	-(30a-68)	inverts																
	(26.18-25.17)/210	0.0047	'/'																	
2		B	C	1.9	1.9	0.5	i @	7.7 min	1.33	1.3		0.0045	0.013	9.5"	Use	10"				
	Overland	80	0.85	fps			1.6					V_{10} =	2.7	fps						
	Street	175	0.48	fps			6.1													
							7.7	min				t_{pipe} =	215	/2.7(60) =	1.3 min					
	Street Slope:	Gutter near (30f-29) - Gutter near (30a-71)																		
	(31.09-30.99)/175=	0.0006'/'										t_c =	7.7 +	1.3 =	9.0 min					
	Design Slope	(30a-71)	-(30a-68)	inverts																
	(26.21-25.24)/215	= 0.0045'/'																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by [illegible] Date 12/31/91
Checked by _____ Date _____
56th Street and Broadway - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope 3 %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
3		C	D	4.2	7.0	0.5	i @	9 min	1.22	4.3		0.0035	0.013	15.7"	Use	18"				
	Overland	75	0.85	fps			1.5					V_{18} =	3.5	fps						
	Street	300	0.77	fps			6.5													
							8.0	min				t_{pipe} =	265	/3.5	(60) =	1.3 min				
	Street Slope:	Gutter near	(30f-24)	- Gutter	near	(30a-68)														
	(30.97-30.52)/300 =	0.0015 %										t_c =	9 + 1.3 =	10.3 min						
	t_c Area 3 =	8 min																		
	t_c A to C =	6.1 min																		
	t_c B to C =	9 min																		
	governing t_c =	9 min																		
	Design Slope:	(30a-68)	-(30a-66)																	
	(24.87-23.94)/265	= 0.0035 %																		
7		D	H	1.1	8.1	0.5	i @ 10.3 min		1.13	4.6		0.0034	0.013	16.2"	Use	18"				
	t_c from C to D =	10.3 min										V_{18} =	3.5	fps						
	No t_c calculation	for Area	7																	
	Design Slope:	(30a-66)	-(30a-39)									t_{pipe} =	475	/60(3.5) =		2.3 min				
	(23.94-22.34)/475	= 0.0034 %										t_c =	10.3 + 2.3	= 12.6 min						

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 5
Computed by KAC Date 12/31/91
Checked by _____ Date _____
56th Street and Broadway - 2 year.

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
4		E	F	0.96	0.96	0.5	1@	4.6 min	1.63	0.8		0.0039	0.013	8.2"	Use 10"					
	Overland	80	0.85	fps			1.6					V_{10} =	2.5	fps						
	Street	150	0.83	fps			3.0													
							4.6	min				t_{pipe}	370/2.5(60) =		2.5 min					
	Street Slope:	Gutter near (30a-58)										t_c =	4.6 + 2.5 =		7.1 min					
	(31.45-31.19)/150 = 0.0017 ft/ft																			
	Design Slope:	(30a-57)	-(30a-52)	inverts																
	(26.88-25.74)/290 = 0.0039 ft/ft																			
	290+80 = 370'	for pipe length																		
5		F	H	2.5	3.5	0.5	1@	7.1 min	1.38	2.4		0.0053	0.013	11.6"	Use 12"					
	Overland	125	0.85	fps			2.5					V_{12} =	3.3	fps						
	Street	150	1.1	fps			2.3													
							4.8	min				t_{pipe} =	575/60(3.3) =		2.9 min					
	Street Slope:	Gutter near (30a-53)					t_c from E to F =		7.1	min		t_c =	7.1 + 2.9 =		10 min					
	(31.45-31.02)/150 = 0.0029 ft/ft						governing t_c =		7.1											
	Design Slope:	(30a-52)	-(30a-53)																	
	(25.74-23.10)/495 = 0.0053 ft/ft																			
	495+80 = 575 pipe length																			

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 5
Computed by JAC Date 12/31/91
Checked by _____ Date _____
56th Street and Broadway - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
6		G	H	2.5	2.5	0.5	i @	11.6 min	1.07	1.3		0.0047	0.013	9.4"	Use 10"					
	Overland	80	0.55	fps			1.6					V_{10} = 2.8 fps								
	Street	300	0.5	fps			10													
							11.6	min				t_{pipe} = 655/2.8(60) =				3.9	min			
	Street Slope: Gutter nor		(30a-75) - (30a-72)				KIM													
	(31.48-31.30)/300 =	0.0006 1/1										t_c = 11.6 + 3.9 =				15.5 min				
	Design Slope:	(30a-72)	-(30a-39)																	
	(27.88-24.87)/655	= 0.0047 1/1																		
—		H	J	—	14.1	0.5	i @ 15.5 min		0.91	6.4		0.0025	0.013	19.4"	Use 21"					
	t_c from G to H =	15.5 min																		
	t_c from F to H =	10 min										V_{21} = 3.3 fps								
	t_c from D to H =	12.6 min																		
	t_c = 15.5 min governs											t_{pipe} = 170/3.3(60) =				0.9 min				
	Contributing areas	1, 2, 3, 4, 5	6, 7									t_c = 15.5 + 0.9 =				16.4 min				
	Design Slope:	(30a-39)	-(30a-31)																	
	(22.34-21.92)/170	= 0.0025 1/1																		

Computation Form
Drainage and Storm Sewer Design
Rational Method

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C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/31/91
Checked by _____ Date _____
56th Street and Broadway - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope 3%	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11		L	K	1.1	1.1	0.5	i @	4.4 min	1.67	0.92		0.0062	0.013	7.9"	Use 8"					
	Overland	140	0.85	fps			2.7					V_8 =	2.7 fps							
	Street	110	1.1	fps			1.7					t_{pipe} =	140/2.7(60) =		0.9 min					
							4.4	min												
	Street Slope: Assume same as RIMS (30a-33) - (30a-32)											t_c =	4.4 + 0.9 =		5.3 min					
	(32.01-31.59)/140 = 0.0030'/'																			
	Design Slope: (30a-33) - (30a-32)																			
	(24.87-24.00)/140 = 0.0062'/'																			
10		K	J	1.5	2.6	0.5	i @	5.3 min	1.53	2.0		0.0037	0.013	11.6"	Use 12"					
	Overland	160	0.85	fps			3.1					V_{12} =	2.8 fps							
	Street	125	1.1	fps			1.8					t_{pipe} =	185/2.8(60) =		1.1 min					
							4.9	min												
	Street Slope: Assume same as RIMS (30a-33) - (30a-32)						governing tc from I to K = 5.3 min.					t_c =	5.3 + 1.1 =		6.4 min					
	(32.01-31.59)/140 = 0.0030 '/'																			
	Design Slope: (30a-32) - (30a-31)																			
	(24.00-23.32)/185 = 0.0037 '/'																			

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Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

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I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 12/31/91
Checked by _____ Date _____
56th Street and Broadway - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
8		I	J	1.7	1.7	0.5	i @	6.0 min	1.48	1.3		0.0036	0.013	9.9"	Use 10"					
	Overland	140	0.85	fps			2.7					V_{10} =	2.4 fps							
	Street	170	0.85	fps			3.3													
							6.0	min				t_{pipe}	300/2.4(60)	= 2.1	min					
	Street Slope: Gutter near (30a-30)											t_c =	6 + 2.1	= 8.1	min					
	(31.85 - 31.54)/170 = 0.0018 ft/ft																			
	Design Slope: (30a-30) - (30a-31)																			
	(24.42 - 23.84)/300 = 0.0036 ft/ft																			
9		J	N	3.9	22.3	0.5	i @ 16.4	min	0.89	9.9		0.0025	0.013	22.8"	Use 24"					
	t_c H to J = 16.4 min											V_{24} =	3.6 fps							
	Contributing Areas 1-11											t_{pipe}	310/3.6(60)	= 1.4	min					
	Design Slope: (30a-31) - (30a-16)											t_c =	16.4 + 1.4	= 17.8	min					
	21.92 - 21.14)/310 = 0.0025 ft/ft																			

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LHC Date 12/31/91
Checked by _____ Date _____
56th Street and Broadway - 2 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
13		0	N	3.3	3.3	0.5	i@	5.7 min	1.50	2.5		0.0044	0.013	12.2"	Use	15"				
	Overland	170	0.85	fps			3.3					V_{15} =	3.5	fps						
	Street	190	1.3	fps			2.4													
							5.7	min				t_{pipe} =	250/3.5(60) =			1.2 min				
	Street Slope:	Gutters	near (30a-17)																	
	(32.37-31.53)/190	= 0.0044	'/'									t_c =	5.7 + 1.2 =			6.9 min				
	Design Slope:	(30a-17)	-(30a-16)																	
	(28.26-27.17)/250	= 0.0044	'/'																	
12		M	N	2.9	2.9	0.5	i@	7.5 min	1.35	2.0		0.0047	0.013	11.1"	Use	12"				
	Overland	200	0.85	fps			3.9					V_{12} =	3.1	fps						
	Street	175	0.81	fps			3.6													
							7.5	min				t_{pipe} =	315/3.1(60) =			1.7 min				
	Street Slope:	Gutter near (30a-9) - Gutter near (30a-15)																		
	(31.80-31.51)/175	= 0.0017	'/'									t_c =	7.5 + 1.7 =			9.2 min				
	Design Slope:	(30a-15)	-(30a-16)																	
	(27.52-26.04)/315	= 0.0047	'/'																	
—		N	O	—	28.5	0.5	i@	17.8 min	0.87	12.4		0.0026	0.013	24.6"	Use	27"				
	t_c from J to N	= 17.8 min										V_{27} =	4.0	fps						
	Design Slope:	(30a-16)	-Broadway MH									t_{pipe} =	410/4(60) =			1.7 min				
	(21.14-20.06)/410	= 0.0026	'/'									t_c =	17.8 + 1.7 =			19.5 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - Sewer

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	C	0.9	0.9	0.5	i @ 4.6 min		2.56	1.2		0.0047	0.013	9.2"	Use	10"				
												V_{10} =	2.8	fps						
												t_{pipe} =	210/60(2.8) =			1.3 min				
												t_c =	4.6 + 1.3 =			5.9 min				
2		B	C	1.9	1.9	0.5	i @ 7.7 min		2.04	1.9		0.0045	0.013	11.0"	Use	12"				
												V_{12} =	3.1	fps						
												t_{pipe} =	215/60(3.1) =			1.2 min				
												t_c =	7.7 + 1.2 =			8.9 min				
3		C	D	4.2	7.0	0.5	i @	8.9 min	1.89	6.6		0.0035	0.013	18.4"	Use	21"				
	t_c from A to C = 5.9 min																			
	t_c from B to C = 8.9 min											V_{21} =	3.9 fps							
												t_{pipe} =	265/60(3.9) =			1.1 min				
												t_c =	8.9 + 1.1 =			10 min				
4		D	H	1.1	8.1	0.5	i @ 10 min		1.74	7.0		0.0034	0.013	18.9"	Use	21"				
	t_c from C to D = 10 min																			
												V_{21} =	3.9 fps							
												t_{pipe} =	475/60(3.9) =			2 min				
												t_c =	10 + 2 =			12 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 5year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
4		E	F	0.96	0.96	0.5	i @ 4.6 min		2.56	1.2		0.0039	0.013	9.5"	Use	10"				
												V_{10} =	2.5 fps							
												t_{pipe} =	370/60(2.5) =			2.5 min				
												t_c =	4.6 + 2.5 = 7.1 min							
5		F	H	2.5	3.5	0.5	i @	7.1 min	2.12	3.7		0.0053	0.013	13.7"	Use	15"				
	t_c from E to F =	7.1 min										V_{15} =	3.8 fps							
												t_{pipe} =	575/60(3.8) =			2.5 min				
												t_c =	7.1 + 2.5 = 9.6 min							
6		G	H	2.5	2.5	0.5	i @ 11.6 min		1.63	2.0		0.0047	0.013	11.1"	Use	12"				
												V_{12} =	3.1 fps							
												t_{pipe} =	655/60(3.1) =			3.5 min				
												t_c =	11.6 + 3.5 = 15.1 min							
-		H	J	-	14.1	0.5	i @ 15.1 min		1.40	9.9		0.0025	0.013	22.8"	Use	24"				
	t_c from G to H =	15.1 min										V_{24} =	3.6 fps							
	t_c from F to H =	9.6 min										t_{pipe} =	170/60(3.6) =			0.8 min				
	t_c from D to H	12.0 min										t_c =	15.1 + 0.8 = 15.9 min							

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Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

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C = Runoff coefficient
i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DHC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 5 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11		L	K	1.1	1.1	0.5	i @	4.4 min	2.4	1.5		0.0062	0.013	9.5"	Use	10"				
												V_{10} =	3.2	fps						
												t_{pipe} =	140/	60(3.2) =		0.7 min				
												t_c =	4.4 +	0.7 =		5.1 min				
10	t_c from L to K = 5.1 min	K	J	1.5	2.6	0.5	i @	5.1 min	2.39	3.1		0.0037	0.013	13.7"	Use	15"				
												V_{15} =	3.2	fps						
												t_{pipe} =	185/	60(3.2) =		1 min				
												t_c =	5.1 +	1 =		6.1 min				
8		I	J	1.7	1.7	0.5	i @	6.0 min	2.27	1.9		0.0036	0.013	11.5	Use	12"				
												V_{12} =	2.7	fps						
												t_{pipe} =	300/	60(2.7) =		1.9 min				
												t_c =	6.0 +	1.9 =		7.9 min				
9	t_c from H to J	J	N	3.9	22.3	0.5	i @	15.9 min	1.37	15.3		0.0025	0.013	26.9"	Use	27"				
												V_{27} =	3.9	fps						
												t_{pipe} =	310/	60(3.9) =		1.3 min				
												t_c =	15.9 +	1.3 =		17.2 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - Sewer

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
13		O	N	3.3	3.3	0.5	i@ 5.7min		2.31	3.8		0.0044	0.013	14.3"	Use	15"				
												V_{15} =	3.5	fps						
												t_{pipe} =	250/60(3.5) =			1.2 min				
												t_c =	5.7 +	1.2 =		6.9 min				
12		M	N	2.9	2.9	0.5	i@	7.5min	2.07	3.0		0.0047	0.013	12.9"	Use	15"				
												V_{15} =	3.6 fps							
												t_{pipe} =	315/60(3.6) =			1.5 min				
												t_c =	7.5 +	1.5 =		9.0 min				
–		N	O	–	28.5	0.5	i@	17.2min	1.33	18.9		0.0026	0.013	28.9"	Use	30"				
	t_c from J to N =	17.2 min										V_{30} =	4.3 fps							
												t_{pipe} =	410/60(4.3) =			1.6 min				
												t_c =	17.2 + 1.6 =			18.8 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

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A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 10 year

Drainage Area Number	Location			A		C	Time of Concentration		I	Q	Design					Profile				
	Street	From	To	Increment	Total		To Inlet	Pipe or Channel			Size	Slope %	n	Cap. Full	V	Length ft.	Fall ft.	Other Losses	Upper Inv. El.	Lower Inv. El.
1		A	C	0.9	0.9	0.5	i @ 4.6	min	3.24	1.5		0.0047	0.013	10"	Use	10"				
												V_{10} =	2.8	fps						
												t_{pipe} =	210 / 60(2.8) =			1.3 min				
												t_c =	4.6 + 1.3 =			5.9 min				
2		B	C	1.9	1.9	0.5	i @ 7.7	min	2.55	2.4		0.0045	0.013	12.0"	Use	12.0"				
												V_{12} =	3.1	fps						
												t_{pipe} =	215 / 60(3.1) =			1.2 min				
												t_c =	7.7 + 1.2 =			8.9 min				
3		C	D	4.2	7.0	0.5	i @	8.9 min	2.34	8.2		0.0035	0.013	20.0"	Use	21"				
	t_c from A to C =	5.9 min										V_{21} =	3.9 fps							
	t_c from B to C =	8.9 min										t_{pipe} =	265 / 60(3.9) =			1.1 min				
												t_c =	8.9 + 1.1 =			10 min				
7		D	H	1.1	8.1	0.5	i @	10 min	2.16	8.7		0.0034	0.013	20.5"	Use	21"				
	t_c from C to D =	10 min										V_{21} =	3.9 fps							
												t_{pipe} =	475 / 60(3.9) =			2.0 min				
												t_c =	10 + 2 =			12 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

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I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
4		E	F	0.96	0.96	0.5	i @ 4.6 min		3.24	1.6		0.0039	0.013	10.6"	Use	12"				
												V_{12} =	2.8 fps							
												t_{pipe} =	370/60(2.8) =			2.2 min				
												t_c =	4.6 + 2.2 =			6.8 min				
5		F	H	2.5	3.5	0.5	i @	6.8 min	2.70	4.7		0.0053	0.013	15.0"	Use	15"				
	t_c from E to F =	6.8 min										V_{15} =	3.8	fps						
												t_{pipe} =	575/60(3.8) =			2.5 min				
												t_c =	6.8 + 2.5 =			9.3 min				
6		G	H	2.5	2.5	0.5	i @ 11.6 min		2.02	2.5		0.0047	0.013	12.1"	Use	15"				
												V_{15} =	3.6	fps						
												t_{pipe} =	655/60(3.6) =			3 min				
												t_c =	11.6 + 3 =			14.6 min				
-		H	J	-	14.1	0.5	i @	14.6 min	1.76	12.4		0.0025	0.013	24.8"	Use	27"				
	t_c from G to H	14.6 min										V_{27} =	3.9	fps						
	t_c from F to H	9.3 min										t_{pipe} =	170/60(3.9) =			0.7 min				
	t_c from D to H	12 min										t_c =	14.6 min + 0.7 =			15.3 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DHC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 10 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11		L	K	1.1	1.1	0.5	i @	4.4 min	3.36	1.8		0.0062	0.013	10.2"	Use	12"				
												V_{12} =	3.6 fps							
												t_{pipe} =	140/60(3.6) =			0.6 min				
												t_c =	4.4 + 0.6 =			5 min				
10	t_c from L to K = 5 min	K	J	1.5	2.6	0.5	i @ 5	min	3.0	3.9		0.0037	0.013	14.9"	Use	15"				
												V_{15} =	3.2 fps							
												t_{pipe} =	185/60(3.2) =			1 min				
												t_c =	5 + 1 =			6 min				
8		I	J	1.7	1.7	0.5	i @ 6.0	min	2.83	2.4		0.0036	0.013	12.5"	Use	15"				
												V_{15} =	3.2 fps							
												t_{pipe} =	300/60(3.2) =			1.6 min				
												t_c =	6.0 + 1.6 =			7.6 min				
9	t_c from H to J	J	N	3.9	22.3	0.5	i @	15.3 min	1.71	19.1		0.0025	0.013	29.2"	Use	30"				
												V_{30} =	4.2 fps							
												t_{pipe} =	310/60(4.2) =			1.2 min				
												t_c =	15.3 + 1.2 =			16.5 min				

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Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO-Phase 3
Computed by LAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 10year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope ft/ft	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
13		O	N	3.3	3.3	0.5	i @ 5.7 min		2.88	4.8		0.0044	0.013	15.6"	Use	18"				
												V_{18} =	4 fps							
												t_{pipe} =	250/60(4) =			1 min				
												t_c =	5.7 + 1 = 6.7 min							
12		M	N	2.9	2.9	0.5	i @	7.5 min	2.58	3.7		0.0047	0.013	14.0"	Use	15"				
												V_{15} =	3.6 fps							
												t_{pipe} =	315/60(3.6) =			1.5 min				
												t_c =	7.5 + 1.5 = 9 min							
–		N	O	–	28.5	0.5	i @	16.5 min	1.66	23.7		0.0026	0.013	31.4"	Use	33"				
	t_c from J to N =											V_{33} =	4.6 fps							
												t_{pipe} =	410/60(4.6) =			1.5 min				
												t_c =	16.5 + 1.5 = 18 min							

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
1		A	C	0.9	0.9	0.5	i @ 4.6	min	4.17	1.9		0.0047	0.013	10.9"	Use	12'				
												V_{12} =	3.1	fps						
												t_{pipe} =	210/60(3.1) =			1.1 min				
												t_c =	4.6 + 1.1 =			5.7 min				
2		B	C	1.9	1.9	0.5	i @ 7.7	min	3.26	3.1		0.0045	0.013	13.2"	Use	15"				
												V_{15} =	3.5	fps						
												t_{pipe} =	215/60(3.5) =			1 min				
												t_c =	7.7 + 1 =			8.7 min				
3		C	D	4.2	7.0	0.5	i @	8.7 min	3.04	10.6		0.0035	0.013	22.0"	Use	24"				
	t_c from A to C =	5.7 min										V_{24} =	4.3	fps						
	t_c from B to C =	8.7 min										t_{pipe} =	265/60(4.3) =			1 min				
												t_c =	8.7 + 1 =			9.7 min				
7		D	H	1.1	8.1	0.5	i @	9.7 min	2.82	11.4		0.0034	0.013	22.7"	Use	24"				
	t_c from C to D =	9.7 min										V_{24} =	4.2 fps							
												t_{pipe} =	475/60(4.2) =			1.9 min				
												t_c =	9.7 + 1.9 =			11.6 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

$Q = CiA$

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
4		E	F	0.96	0.96	0.5	i @ 4.6 min		4.17	2.0		0.0039	0.013	11.5"	Use	12"				
												$V_{\frac{1}{2}}$ =	2.8 fps							
												t_{pipe} =	370/60(2.8) =			2.2 min				
												t_c =	4.6 + 2.2 =			6.8 min				
5		F	H	2.5	3.5	0.5	i @	6.8 min	3.45	6.0		0.0053	0.013	16.4"	Use	18"				
	t_c from E to F =	6.8 min										$V_{\frac{1}{8}}$ =	4.3 fps							
												t_{pipe} =	575/60(4.3) =			2.2 min				
												t_c =	6.8 + 2.2 =			9 min				
6		G	H	2.5	2.5	0.5	i @ 11.6 min		2.58	3.2		0.0047	0.013	13.3"	Use	15"				
												$V_{\frac{1}{5}}$ =	3.6 fps							
												t_{pipe} =	655/60(3.6) =			3.0 min				
												t_c =	11.6 + 3 =			14.6 min				
-		H	J	-	14.1	0.5	i @	14.6 min	2.24	15.8		0.0025	0.013	27.2"	Use	30"				
	t_c from G to H	14.6 min										$V_{\frac{1}{30}}$ =	4.2 fps							
	t_c from F to H	9.0 min										t_{pipe} =	170/60(4.2) =			0.7 min				
	t_c from D to H =	11.6 min										t_c =	14.6 + 0.7 =			15.3 min				
	governing t_c =	14.6 min																		

Computation Form
Drainage and Storm Sewer Design
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i = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by DHC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 25year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	i	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
11		L	K	1.1	1.1	0.5	i @	4.4min	4.33	2.4		0.0062	0.013	11.3"	Use	12"				
												V_{12} =	3.6fps							
												t_{pipe} =	140/60(3.6) =			0.6min				
												t_c =	4.4 + 0.6 =			5min				
10		K	J	1.5	2.6	0.5	i @	5min	3.84	5.0		0.0037	0.013	16.4"	Use	18"				
	t_c from L to K =	5min										V_{18} =	3.6fps							
												t_{pipe} =	185/60(3.6) =			0.9min				
												t_c =	5 + 0.9 =			5.9min				
8		I	J	1.7	1.7	0.5	i @	6.0min	3.62	3.1		0.0036	0.013	13.8"	Use	15"				
												V_{15} =	3.2 fps							
												t_{pipe} =	300/60(3.2) =			1.6min				
												t_c =	6.0 + 1.6 =			7.6min				
9		J	N	3.9	22.3	0.5	i @	15.3min	2.18	24.3		0.0025	0.013	31.9"	Use	33"				
	t_c from H to J	: 15.3min										V_{33}	4.5 fps							
												t_{pipe} =	310/60(4.5) =			1.1min				
												t_c =	15.3 + 1.1 =			16.4min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - 25 year

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
13		O	N	3.3	3.3	0.5	i @ 5.7 min		3.69	6.1		0.0044	0.013	17.1"	Use	18"				
												V_{18} =	4 fps							
												t_{pipe} =	250/60(4) =			1 min				
												t_c =	5.7 +	1 =		6.7 min				
12		M	N	2.9	2.9	0.5	i @	7.5 min	3.30	4.8		0.0047	0.013	15.4"	Use	18"				
												V_{18} =	4.1	fps						
												t_{pipe} =	315/60(4.1) =			1.3 min				
												t_c =	7.5 +	1.3 =		8.8 min				
—		N	O	—	28.5	0.5	i @	16.4 min	2.13	30.4		0.0026	0.013	34.5"	Use	36"				
	t_c from J to N =											V_{36} =	4.8	fps						
												t_{pipe} =	410/60(4.8) =			1.4 min				
												t_c =	16.4 + 1.4 =			17.8 min				

Computation Form
Drainage and Storm Sewer Design
Rational Method

Q = CiA

Q = Discharge, in cfs
C = Runoff coefficient
I = Rainfall intensity, in inches/hour
A = Drainage area, in acres
V = Velocity, in fps
t = Time, in minutes
n = Manning's roughness coefficient

Project Sacramento CSO - Phase 3
Computed by LAC Date 1/3/92
Checked by _____ Date _____
56th Street and Broadway - Summary

Drainage Area Number	Location: Street	Location: From	Location: To	A: Increment	A: Total	C	Time of Concentration: To Inlet	Time of Concentration: Pipe or Channel	I	Q	Design: Size	Design: Slope %	Design: n	Design: Cap. Full	Design: V	Profile: Length ft.	Profile: Fall ft.	Profile: Other Losses	Profile: Upper Inv. El.	Profile: Lower Inv. El.
	Summary			2	YR		5	YR			10	YR		25	YR					
Existing Φ (in)	Reach	Slope (ft/ft)	Length (ft)	Discharge (cfs)	Dia (Φ) (in)		Discharge (cfs)	Dia Φ (in)			Discharge (cfs)	Dia Φ (in)		Discharge (cfs)	Dia Φ (in)					
8	A to C	0.0047	210	0.7	8		1.2	10			1.5	10		1.9	12					
8	B to C	0.0045	215	1.3	10		1.9	12			2.4	12		3.1	15					
10	C to D	0.0035	265	4.3	18		6.6	21			8.2	21		10.6	24					
12	D to H	0.0034	475	4.6	18		7.0	21			8.7	21		11.4	24					
8	E to F	0.0039	370	0.8	10		1.2	10			1.6	12		2.0	12					
8	F to H	0.0053	575	2.4	12		3.7	15			4.7	15		6.0	18					
8	G to H	0.0047	655	1.3	10		2.0	12			2.5	15		3.2	15					
12	H to J	0.0025	170	6.4	21		9.9	24			12.4	27		15.8	30					
8	L to K	0.0062	140	0.92	8		1.5	10			1.8	12		2.4	12					
8	K to J	0.0037	185	2.0	12		3.1	15			3.9	15		5.0	18					
8	I to J	0.0036	300	1.3	10		1.9	12			2.4	15		3.1	15					
14	J to N	0.0025	310	9.9	24		15.3	27			19.1	30		24.3	33					
8	O to N	0.0044	250	2.5	15		3.8	15			4.8	18		6.1	18					
8	M to N	0.0047	315	2.0	12		3.0	15			3.7	15		4.8	18					
14	N to O	0.0026	410	12.4	27		18.9	30			23.7	33		30.4	36					

APPENDIX F

PHASE 3 SEWERED ACREAGE ANALYSIS

PHASE 3 BASIN ANALYSIS FOR COLLECTION SYSTEM IMPROVEMENTS			
Basin No.	Basin Area (acres)	Applicable Area (acres)	Comments
1001	55.6	37.1	
1002	26.6	0.0	FREEWAY
1003	18.6	0.0	FREEWAY
1031	13.9	7.0	
1051	13.0	13.0	
1061	24.6	24.6	
1062	34.5	0.0	SCHOOL
1071	22.5	22.5	
1072	5.9	5.9	
1081	23.4	23.4	
1082	7.3	7.3	
1083	9.6	0.0	FREEWAY
1101	24.4	24.4	
1102	16.5	16.5	
1211	10.9	10.9	
1241	30.1	30.1	
1301	20.5	10.3	
1302	29.3	29.3	
1321	23.3	23.3	
1331	43.0	21.5	
1361	5.7	0.0	OPEN AREA
1401	25.4	25.4	
1421	29.7	29.7	
1422	34.0	34.0	
1431	12.6	12.6	
1441	35.8	35.8	
1451	43.5	0.0	CEMETARY
1452	23.6	23.6	
1481	21.4	21.4	
1501	9.2	9.2	
1502	17.3	17.3	
1511	29.6	29.6	
1521	28.2	28.2	
1531	7.5	7.5	
1551	27.8	27.8	
1561	43.3	43.3	
1581	15.1	15.1	
1591	29.8	14.9	
1601	48.1	48.1	
1631	69.5	46.3	
1641	35.0	35.0	
1651	39.9	39.9	
1661	32.8	32.8	
1721	29.8	0.0	SCHOOL
1731	26.2	13.1	
1741	17.7	17.7	
1751	15.0	15.0	
1771	26.1	26.1	

PHASE 3 BASIN ANALYSIS FOR COLLECTION SYSTEM IMPROVEMENTS			
Basin No.	Basin Area (acres)	Applicable Area (acres)	Comments
1781	16.0	16.0	
1791	38.2	38.2	
1821	17.9	17.9	
1831	48.0	32.0	
1861	46.5	0.0	PARK
1871	10.6	10.6	
1881	14.2	14.2	
1891	8.8	8.8	
1911	31.9	31.9	
1921	40.5	20.3	
1922	51.7	0.0	PARK
1923	9.0	0.0	PARK
1931	56.7	28.4	
1932	32.0	16.0	
1941	12.3	6.2	
1951	35.0	35.0	
2001	54.3	54.3	
2002	44.0	0.0	FREEWAY
2003	23.8	0.0	FREEWAY
3011	10.3	5.1	
3021	18.9	18.9	
3031	13.8	13.8	
3041	21.8	21.8	
3042	24.0	24.0	
3071	5.1	5.1	
3081	20.5	20.5	
3091	13.0	13.0	
3121	23.2	23.2	
3151	8.3	8.3	
3161	7.4	7.4	
3191	5.2	5.2	
3201	21.9	21.9	
3231	35.2	35.2	
3241	17.4	17.4	
3281	13.3	13.3	
3291	7.6	7.6	
3311	26.9	26.9	
3321	18.3	18.3	
3331	37.1	18.6	
3341	21.5	21.5	
3351	19.4	19.4	
3361	30.2	30.2	
3371	11.7	11.7	
3411	27.4	27.4	
3421	12.2	12.2	
3431	37.0	37.0	
3451	22.5	11.3	
3461	45.8	45.8	

PHASE 3 BASIN ANALYSIS FOR COLLECTION SYSTEM IMPROVEMENTS			
Basin No.	Basin Area (acres)	Applicable Area (acres)	Comments
3491	33.1	33.1	
3501	27.0	27.0	
3511	20.1	20.1	
3531	16.1	16.1	
3541	9.7	0.0	PARK
3551	20.2	20.2	
3561	15.6	7.8	
3562	9.7	9.7	
3571	3.2	0.0	DIRECT DISCHARGE
3581	31.9	31.9	
3611	13.6	13.6	
3612	15.4	15.4	
3613	11.4	0.0	DIRECT DISCHARGE
3621	10.5	10.5	
3641	13.4	6.7	
3642	11.7	11.7	
3651	12.8	12.8	
3661	22.8	22.8	
3991	41.9	41.9	
5011	2.2	0.0	DIRECT DISCHARGE
5021	23.1	23.1	
5031	17.2	17.2	
5041	24.4	24.4	
5061	31.9	31.9	
5081	21.9	11.0	
5091	33.1	33.1	
5092	14.2	14.2	
5101	18.3	18.3	
5111	24.1	24.1	
5121	4.3	4.3	
5122	27.9	0.0	FREEWAY
5151	5.6	5.6	
5152	31.8	31.8	
5153	7.7	0.0	FREEWAY
5181	18.4	18.4	
5201	6.5	6.5	
5211	7.2	7.2	
5221	15.4	15.4	
5251	9.4	4.7	
5281	4.4	0.0	DIRECT DISCHARGE
5291	3.5	0.0	DIRECT DISCHARGE
5301	8.8	8.8	
5321	34.8	34.8	
5341	33.2	33.2	
5342	51.7	25.9	
5343	35.4	35.4	
5371	29.8	29.8	
5372	2.0	0.0	FREEWAY

PHASE 3 BASIN ANALYSIS FOR COLLECTION SYSTEM IMPROVEMENTS			
Basin No.	Basin Area (acres)	Applicable Area (acres)	Comments
5381	7.5	7.5	
5382	52.7	0.0	FREEWAY
5391	10.4	0.0	FREEWAY
5392	7.1	7.1	
5401	25.4	25.4	
5402	22.2	22.2	
5421	33.2	33.2	
5451	14.9	14.9	
5461	37.7	37.7	
5462	10.6	10.6	
5471	4.8	4.8	
5481	16.4	16.4	
5482	30.7	30.7	
5491	21.0	21.0	
5501	26.9	26.9	
5561	15.2	0.0	FREEWAY
5562	9.0	9.0	
5563	7.3	7.3	
5571	13.9	7.0	
5581	10.8	10.8	
5591	40.4	40.4	
5601	34.3	34.3	
5602	23.7	23.7	
5621	28.1	28.1	
5631	15.5	15.5	
5641	23.3	23.3	
5661	3.6	0.0	FREEWAY
5671	13.5	13.5	
5672	13.4	13.4	
5681	41.0	41.0	
5691	32.9	32.9	
5701	40.9	40.9	
5711	28.2	28.2	
5721	9.4	9.4	
5741	10.1	0.0	FREEWAY
5742	43.9	43.9	
5751	14.1	14.1	
5771	6.3	0.0	SANITARY
5781	3.9	0.0	SANITARY
5791	4.4	0.0	SANITARY
5841	17.4	17.4	
5851	17.7	17.7	
5861	15.4	15.4	
5871	10.6	10.6	
5881	12.4	12.4	
5901	24.4	24.4	
5921	22.6	22.6	
5931	40.9	40.9	

PHASE 3 BASIN ANALYSIS FOR COLLECTION SYSTEM IMPROVEMENTS			
Basin No.	Basin Area (acres)	Applicable Area (acres)	Comments
5941	54.1	54.1	
5961	14.4	14.4	
5981	19.1	19.1	
5991	25.6	25.6	
7011	2.5	0.0	DIRECT DISCHARGE
7031	5.4	0.0	PARK
7041	5.7	0.0	PARK
7051	56.6	37.7	
7101	10.2	0.0	SCHOOL
7111	25.5	25.5	
7121	31.2	0.0	RAILROAD YARD
7122	27.0	0.0	RAILROAD YARD
7123	15.2	7.6	
7131	17.7	17.7	
7141	5.4	5.4	
7151	36.7	36.7	
7161	28.7	28.7	
7162	6.0	0.0	FREEWAY
7181	22.3	22.3	
7191	20.7	20.7	
7201	15.2	15.2	
7211	25.4	25.4	
7221	24.4	24.4	
7231	24.9	12.5	
7241	72.5	72.5	
7242	9.1	9.1	
7291	10.0	10.0	
7321	11.6	11.6	
7322	22.3	0.0	SEPARATED
7323	33.3	0.0	SEPARATED
7331	23.4	23.4	
7341	7.2	7.2	
7351	15.2	15.2	
7921	6.3	6.3	
7361	39.8	19.9	
7371	14.9	14.9	
7372	21.2	21.2	
7381	30.4	30.4	
7382	13.9	13.9	
7401	21.2	21.2	
7411	38.1	38.1	
7421	34.9	34.9	
7431	31.8	31.8	
7432	43.2	21.6	
7433	33.5	16.8	
7441	48.2	48.2	
7442	26.3	26.3	
7451	8.6	0.0	DIRECT DISCHARGE

PHASE 3 BASIN ANALYSIS FOR COLLECTION SYSTEM IMPROVEMENTS			
Basin No.	Basin Area (acres)	Applicable Area (acres)	Comments
7461	18.0	18.0	
7481	33.2	33.2	
7482	27.4	27.4	
7491	51.6	51.6	
7492	17.6	17.6	
7521	3.4	3.4	
7522	1.4	0.0	SANITARY
7541	8.9	0.0	SANITARY
7561	4.2	0.0	SANITARY
7581	7.4	0.0	SANITARY
7601	21.6	21.6	
7622	42.6	0.0	CEMETARY
7623	29.7	29.7	
7641	2.0	0.0	SANITARY
7651	4.0	0.0	SANITARY
7681	4.0	0.0	SANITARY
7682	2.4	0.0	SANITARY
7683	3.7	0.0	SANITARY
7684	1.6	0.0	SANITARY
7691	1.8	0.0	SANITARY
7701	3.3	0.0	SANITARY
7711	4.9	0.0	SANITARY
7741	2.8	0.0	SANITARY
7751	5.1	0.0	SANITARY
7761	2.1	0.0	SANITARY
7771	4.2	0.0	SANITARY
7781	16.5	16.5	
7791	18.6	18.6	
7801	19.6	19.6	
7811	39.0	39.0	
7821	38.1	38.1	
7831	29.8	29.8	
7851	27.6	27.6	
7861	22.1	22.1	
7871	15.4	15.4	
7872	10.7	10.7	
7901	49.9	49.9	
7911	10.9	0.0	DIRECT DISCHARGE
7912	68.4	68.4	
7921	6.3	6.3	
7931	9.2	0.0	DIRECT DISCHARGE
7941	30.8	30.8	
7951	38.5	38.5	
7952	8.6	8.6	
7961	19.5	19.5	
7962	27.8	27.8	
7971	5.2	0.0	SANITARY
7972	7.2	0.0	SANITARY

PHASE 3 BASIN ANALYSIS FOR COLLECTION SYSTEM IMPROVEMENTS			
Basin No.	Basin Area (acres)	Applicable Area (acres)	Comments
7973	16.2	0.0	SANITARY
7974	14.9	0.0	SANITARY
7991	26.2	26.2	
7992	8.7	0.0	SANITARY
9021	26.6	26.6	
9031	2.7	0.0	SANITARY
9032	7.0	0.0	FREEWAY
9051	23.0	23.0	
9052	16.9	0.0	DIRECT DISCHARGE
9061	27.8	27.8	
9071	15.9	0.0	FREEWAY
9081	8.8	8.8	
9101	20.6	0.0	FREEWAY
9131	23.8	0.0	FREEWAY
9151	47.3	47.3	
9181	14.9	14.9	
9191	9.8	9.8	
9201	12.5	12.5	
9211	12.8	6.4	
9221	23.0	11.5	
9241	30.0	0.0	PARK
9251	35.7	35.7	
9281	1.0	0.0	SANITARY
9282	36.2	18.1	
9291	31.3	31.3	
9301	2.8	0.0	SANITARY
9311	3.5	0.0	SANITARY
9331	0.8	0.0	SANITARY
9341	2.8	0.0	SANITARY
9342	7.1	0.0	SANITARY
9361	25.0	25.0	
9371	45.1	45.1	
9381	19.0	19.0	
9382	22.4	22.4	
9391	46.0	46.0	
9401	18.0	18.0	
9411	48.3	48.3	
9451	29.4	29.4	
9452	15.7	0.0	DIRECT DISCHARGE
9471	20.6	20.6	
9501	39.4	39.4	
9481	16.3	16.3	
9491	26.4	26.4	
9551	4.3	0.0	SANITARY
9552	24.7	24.7	
9581	6.7	0.0	SANITARY
9591	3.0	0.0	SANITARY
9611	1.5	0.0	SANITARY

PHASE 3 BASIN ANALYSIS FOR COLLECTION SYSTEM IMPROVEMENTS			
Basin No.	Basin Area (acres)	Applicable Area (acres)	Comments
9612	1.5	0.0	SANITARY
9621	2.0	0.0	SANITARY
9631	2.8	0.0	SANITARY
9641	7.1	0.0	SANITARY
9661	5.5	0.0	SANITARY
9701	37.4	18.7	
9702	6.0	6.0	
9711	23.6	23.6	
9721	16.5	16.5	
9731	27.0	27.0	
9741	40.2	40.2	
TOTALS	7317.1	5834.9	

APPENDIX G

REPRESENTATIVE BASIN ANALYSIS BY MR.JOHN TOMKO

TECHNICAL MEMORANDUM
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN
PHASE 3, TASK 5

PREPARED BY: **John Tomko, Consulting Engineer**

DATE: **June 23, 1992**

SUBJECT: **Combined Sewer Collection System Improvement Costs for the 2-, 5-, 10- and 25-Year Storms**

A. INTRODUCTION

During Phase 2 of the Combined Sewer Improvement Plan Study, HDR conducted a hydraulic and cost estimating analysis on three representative collection system basins for the 10 year storm event. Cost estimates for these three basins were then converted into per-acre cost estimates, and these cost data were then extrapolated to the entire combined collection system by multiplying the per-acre cost estimate for a specific basin by the total acreage of that specific basin type within the entire service area.

The major cost estimating assumption in Phase 2 was that there are significant differences (i.e., hydraulic capability, pipe layout, physical condition, age of sewers, types of sewers, and presence of backyard easements) between the combined sewer collection system's basins to warrant estimating improvement costs for three different representative types.

During Phase 3, HDR abandoned the Phase 2 basin-type approach for estimating collection system improvements for two different approaches. The first approach, which they called "An Analysis of Pipe Replacement Based Upon Pipe Lengths" (Pipe Length Method), uses average replacement percentages (over 6 basins) to compute system-wide replacement estimates, based upon total system-wide pipe lengths (within diameter categories). The second approach, which they called "An Analysis of Pipe Replacement Based Upon Area" (Area Method), uses an average per-acre cost (over 6 basins) to compute system-wide replacement estimates, based upon a total collection system acreage of 6406 acres.

HDR's new "system-wide averaged" approach uses the six representative basins as representative of the entire combined sewer collection system. However, four of the six basins selected by HDR contain a significant portion of backyard easement sewers. The concern is that "backyard easement" sewers represent only 11% of the total system-wide collection sewers. Therefore, using these representative basins with the new "system-wide averaged" approaches may tend to grossly bias the overall collection system cost estimate toward "backyard easement" systems.

This technical memorandum addresses this concern by presenting the results of a Phase 2 (Representative Basin) cost analysis of the improvement cost data developed by HDR for the six representative basins studied in Phase 3. However, rather than conducting the Phase 2 representative type cost analysis using the total area of the collection system (as was done in Phase 2), this analysis uses total pipe lengths.

For purposes of this report, collection system basins are categorized as follows:

Type 1 - Downtown basins with square blocks and alleys.

Type 2 - Non-downtown basins without backyard easements.

Type 3 - Non-downtown basins with backyard easements.

Attachment 1 categorizes the areal distribution for the combined sewer system's basins based upon basin type. This breakdown of basins is based predominantly upon HDR's categorization of basins in their Phase 2 report, and Table 13 of their Phase 3 Technical Memorandum entitled, "Hydraulic Evaluation and 10-Year Storm Improvements for the Combined Sewer Collection System".

B. PIPE REPLACEMENT COSTS

REPRESENTATIVE BASINS - The same six representative basins as in the HDR Phase 3, Task 5 Technical Memorandum were used in this analysis. However, for purposes of this analysis the six representative basins were classified into one of three types (see Table 1).

TABLE 1
REPRESENTATIVE BASINS

BASIN #	TYPE #	DESCRIPTION	AREA (ACRES)	TOTAL [1] PIPE (LF)	PIPE PER ACRE (LF/AC)
1	1	DOWNTOWN	36.7	5315	144.8
2	2	NON-DOWNTOWN W/O EASEMENTS	34.1	1845	54.1
3	3	NON-DOWNTOWN W/ EASEMENTS	43.7	7895	180.7
4	3	"	33.7	7485	222.1
5	3	"	44.0	10945	248.8
6	3	"	28.5	7250	254.4

Footnote 1 = Total pipe length includes backyard easement pipes.

PIPE REPLACEMENT COSTS PER REPRESENTATIVE BASIN - Pipe replacement costs per acre for each of the six representative basins are shown in Table 2. The improvement costs were obtained from Appendix D of the HDR Phase 3, Task 5 Technical Memorandum.

TABLE 2
PIPE REPLACEMENT COSTS
($ PER ACRE)

BASIN	2-YEAR	5-YEAR	10-YEAR	25-YEAR
1	15,961	18,661	20,978	23,077
2	7,657	8,724	10,177	10,878
3	14,956	18,371	19,987	23,036
4	13,514	15,447	16,718	18,901
5	20,863	24,414	27,261	30,062
6	19,608	22,605	24,572	28,223

PIPE REPLACEMENT DATA PER BASIN TYPE - Table 3 presents the pipe length and pipe replacement cost data contained in Tables 1 and 2 based upon basin type. Table 4 shows the pipe replacement cost data converted into replacement costs per total lineal feet of pipe within each basin type. The data for basin type 3 represents the average of Representative Basins 3 through 6.

TABLE 3
PIPE LENGTHS AND REPLACEMENT COSTS PER AREA

TYPE #	TOTAL PIPE (LF)	PIPE PER ACRE (LF/AC)	2-YEAR ($ PER ACRE)	5-YEAR ($ PER ACRE)	10-YEAR ($ PER ACRE)	25-YEAR ($ PER ACRE)
1	5315	144.82	15,961	18,661	20,978	23,077
2	1845	54.11	7,657	8,724	10,177	10,878
3	8394	226.48	17,235	20,209	22,135	25,056

TABLE 4
REPLACEMENT COSTS PER LINEAL FEET

TYPE #	2-YEAR ($/LF)	5-YEAR ($/LF)	10-YEAR ($/LF)	25-YEAR ($/LF)
1	110.21	128.86	144.86	159.35
2	141.51	161.23	188.08	201.03
3	76.10	89.23	97.73	110.63

TOTAL PIPE REPLACEMENT PER BASIN TYPE - Using the areal distribution of the three representative type basins as shown in Attachment 1, the amount of pipe requiring replacement for each basin type was calculated by multiplying the total area of a basin type by the appropriate "Pipe Per Acre" value in Table 3 above. Table 5 shows the "Total Pipe" in lineal feet requiring replacement for each representative basin type.

The March 1977 City survey data contained in a report entitled "Report on Pipe Diameter and Length, Sewer and Drainage Collection System", Attachment 2, was then used to normalize the lengths of pipe requiring replacement for each basin type.

TABLE 5
REPLACEMENT PIPES PER REPRESENTATIVE BASIN TYPE

TYPE #	AREA (AC)	PIPE (LF)	PIPE % OF TOTAL	NORMAL. PIPE (LF)
1	2091.1	302,833	38.1	399,263
2	2585.5	139,901	17.6	184,449
3	1558.8	353,037	44.4	465,453
TOTAL	**6235.4**	**795,772**	**100**	**1,049,165**

TOTAL PIPE REPLACEMENT COSTS PER BASIN TYPE - The pipe replacement costs for each representative basin type (Table 6) was computed by multiplying the normalized pipe lengths for each basin type (Table 5) by the replacement costs per lineal foot for each basin type (Table 4).

TABLE 6
PIPE REPLACEMENT COSTS PER REPRESENTATIVE BASIN TYPE
($ MILLION)

TYPE #	2-YEAR	5-YEAR	10-YEAR	25-YEAR
1	44.00	51.45	57.84	63.62
2	26.10	29.74	34.69	37.08
3	35.42	41.53	45.49	51.49
TOTAL	**105.53**	**122.72**	**138.02**	**152.20**

C. DROP INLET COSTS

HDR's estimate for DI replacement costs are appropriate and are shown in Table 7.

TABLE 7
DROP INLET COST

DESIGN STORM (YEAR)	$ (MILLION)
2	5.66
5	11.16
10	15.82
25	18.82

D. CONNECTION LATERAL COSTS

The connection lateral costs were estimated in a similar fashion to HDR's Phase 3 estimate, except that the costs were estimated for each basin type and then extrapolated to the total acreage for each representative basin type, rather than averaging the six representative basins over the entire service area. In addition, the total pipe lengths within each representative basin was used in this analysis (i.e., HDR's Assumption # 3, in their Phase 3, Task 5 memorandum, page 23, was not used). Assumptions used in this analysis include:

1. Total length of 8 inch pipes = 762,100 feet.

2. Total length of 10 inch pipes = 120,630 feet.

3. All service laterals are connected to either 8 inch or 10 inch pipes.

4. The number of service laterals connected to pipes of either size is proportional to the total lengths of pipe within the system. Therefore, 86.3 % of the connections are to 8 inch lines, and 13.7 % are to 10 inch lines.

5. The total number of connections, using HDR's estimate of 6 connections per acre, and the revised total collection system acreage of 6235.4 acres (Attachment 1), is 24,438. Therefore, there are 21,090 service lateral connections to 8 inch lines, and 3,348 service lateral connections to 10 inch lines.

8 INCH PIPE REPLACEMENT - Table 8 shows, for each representative basin, the total length of 8-inch pipes, the length of 8-inch pipes that HDR considered for replacement, and the length of 8-inch pipes that HDR did replace. It is assumed that the pipes that HDR did not consider in their analysis were 8-inch pipes in backyard easements. Table 9 shows the total 8-inch pipe lengths per acre and the percentage of pipes replaced for each representative basin type. Type 3 is an average of representative basins 3 through 6.

TABLE 8
8-INCH PIPE DATA PER REPRESENTATIVE BASIN

BASIN #	TOTAL PIPE (LF)	CONSID. FOR REPLACE. (LF)	2-YEAR REPLACED (LF)	5-YEAR REPLACED (LF)	10-YEAR REPLACED (LF)	25-YEAR REPLACED (LF)
1	4510	4510	4510	4510	4510	4510
2	765	765	765	765	765	765
3	6215	4165	2240	4165	4165	4165
4	5685	2245	2160	2160	2160	2160
5	8360	5450	4720	5275	5275	5450
6	5965	3215	2865	3215	3215	3215
TOTAL	**31,500**	**20,350**	**17,260**	**20,090**	**20,090**	**20,265**

TABLE 9
8 INCH PIPE DATA PER BASIN TYPE

TYPE	TOTAL PIPE (LF)	PIPE PER ACRE (LF/AC)	2-YEAR REPLACED (%)	5-YEAR REPLACED (%)	10-YEAR REPLACED (%)	25-YEAR REPLACED (%)
1	4510	122.89	100.00	100.00	100.00	100.00
2	765	22.43	100.00	100.00	100.00	100.00
3	6556	174.95	45.70	56.49	56.49	57.16

TOTAL 8-INCH PIPE WITHIN EACH BASIN TYPE - Using the areal distribution of the three representative type basins as shown in Attachment 1, the amount of 8-inch pipe within each basin type was calculated by multiplying the total area of a basin type by the appropriate "Pipe Per Acre" value in Table 9 above. Table 10 shows the "Total Pipe" in lineal feet within each representative basin type. The March 1977 City survey data contained in a report entitled "Report on Pipe Diameter and Length, Sewer and Drainage Collection System", Attachment 2, was then used to normalize the lengths of 8-inch pipe within each basin type.

TABLE 10
8-INCH PIPE LENGTHS PER REPRESENTATIVE TYPE

TYPE	AREA (AC)	TOTAL PIPE (LF)	PIPE (% OF TOTAL)	NORMAL. PIPE (LF)
1	2091.1	256,975	43.7	333,038
2	2585.5	57,993	9.9	75,448
3	1558.8	272,712	46.4	353,614
TOTAL	**6,235.4**	**587,680**	**100**	**762,100**

TOTAL 8-INCH PIPES REPLACED WITHIN EACH BASIN TYPE - Table 11 shows the total 8-inch pipes replaced within each basin type for each design storm. These values were obtained by multiplying the % replaced values in Table 9 by the normalized 8-inch pipe lengths in Table 10. The % replaced value for basin type 3 is the average of basins 3 through 6.

TABLE 11
8-INCH PIPES REPLACED IN EACH BASIN TYPE

TYPE	2-YEAR REPLACED (LF)	5-YEAR REPLACED (LF)	10-YEAR REPLACED (LF)	25-YEAR REPLACED (LF)
1	333,038	333,038	333,038	333,038
2	75,448	75,448	75,448	75,448
3	161,602	199,757	199,757	202,126
TOTAL	**570,088**	**608,243**	**608,243**	**610,612**
% REPLACED	**74.8**	**79.8**	**79.8**	**80.1**

TOTAL 10-INCH PIPES REPLACED WITHIN EACH BASIN TYPE - Based upon HDR's analysis in the Phase 3, Task 5 TM, all of the service laterals connected to 10-inch pipes will be replaced.

SERVICE LATERAL CONNECTION COSTS - Table 12 shows the service lateral connection costs for both the 8 and 10-inch pipes for each design storm. The costs are equal to the number of service lateral connections to each diameter-sized line multipled by the percent of that diameter pipe replaced multiplied by $2400 per connection.

TABLE 12
SERVICE LATERAL CONNECTION COSTS
($ MILLION)

PIPE DIA.	2-YEAR	5-YEAR	10-YEAR	25-YEAR
8-INCH	37.86	40.39	40.39	40.54
10-INCH	8.04	8.04	8.04	8.04
TOTAL	**45.90**	**48.43**	**48.43**	**48.58**

E. **OVERALL COLLECTION SYSTEM REPLACEMENT COSTS** - Table 13 shows the overall collection system replacement costs for each design storm. These costs are the sum of the pipe replacement costs (Section B), the drop inlet replacement costs (Section C), and the service lateral connection costs (Section D).

TABLE 13
OVERALL COLLECTION SYSTEM REPLACEMENT COSTS
($ MILLION)

COST ITEM	2-YEAR	5-YEAR	10-YEAR	25-YEAR
PIPE	105.53	122.72	138.02	152.20
DROP INLETS	5.66	11.16	15.82	18.82
SERVICE LATERALS	45.90	48.43	48.43	48.58
TOTAL	**157.09**	**182.31**	**202.27**	**219.60**

ATTACHMENT 1

BASIN CATEGORIZATION BY TYPE

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
1001	1	55.6	37.1
1101	1	24.4	24.4
1102	1	16.5	16.5
1951	1	35.0	35.0
2001	1	54.3	54.3
3011	1	10.3	5.1
3021	1	18.9	18.9
3031	1	13.8	13.8
3041	1	21.8	21.8
3042	1	24.0	24.0
3071	1	5.1	5.1
3081	1	20.5	20.5
3091	1	13.0	13.0
3121	1	23.2	23.2
3151	1	8.3	8.3
3161	1	7.4	7.4
3191	1	5.2	5.2
3201	1	21.9	21.9
3231	1	35.2	35.2
3241	1	17.4	17.4
3281	1	13.3	13.3
3311	1	26.9	26.9
3321	1	18.3	18.3
3331	1	37.1	18.6
3341	1	21.5	21.5
3351	1	19.4	19.4
3361	1	30.2	30.2
3371	1	11.7	11.7
3411	1	27.4	27.4
3421	1	12.2	12.2
3431	1	37.0	37.0
3451	1	22.5	11.3
3461	1	45.8	45.8
3491	1	33.1	33.1
3501	1	27.0	27.0
3511	1	20.1	20.1
3531	1	16.1	16.1
3551	1	20.2	20.2
3561	1	15.6	7.8
3562	1	9.7	9.7
3581	1	31.9	31.9
3611	1	13.6	13.6
3612	1	15.4	15.4

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
3621	1	10.5	10.5
3641	1	13.4	6.7
3642	1	11.7	11.7
3651	1	12.8	12.8
3661	1	22.8	22.8
3991	1	41.9	41.9
5121	1	4.3	4.3
5151	1	5.6	5.6
5152	1	31.8	31.8
5181	1	18.4	18.4
5201	1	6.5	6.5
5211	1	7.2	7.2
5221	1	15.4	15.4
5251	1	9.4	4.7
5301	1	8.8	8.8
5371	1	29.8	29.8
5381	1	7.5	7.5
5392	1	7.1	7.1
5401	1	25.4	25.4
5402	1	22.2	22.2
5421	1	33.2	33.2
5451	1	14.9	14.9
5461	1	37.7	37.7
5471	1	4.8	4.8
5481	1	16.4	16.4
5482	1	30.7	30.7
5491	1	21.0	21.0
5501	1	26.9	26.9
5562	1	9.0	9.0
5563	1	7.3	7.3
5571	1	13.9	7.0
5581	1	10.8	10.8
5591	1	40.4	40.4
5601	1	34.3	34.3
5621	1	28.1	28.1
5631	1	15.5	15.5
5641	1	23.3	23.3
5671	1	13.5	13.5
5672	1	13.4	13.4
5681	1	41.0	41.0
5691	1	32.9	32.9
5701	1	40.9	40.9
5711	1	28.2	28.2

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

	BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
	5721	1	9.4	9.4
	5751	1	14.1	14.1
	5841	1	17.4	17.4
	5851	1	17.7	17.7
	5871	1	10.6	10.6
	5881	1	12.4	12.4
	5901	1	24.4	24.4
	5921	1	22.6	22.6
	5931	1	40.9	40.9
	5941	1	54.1	54.1
	5961	1	14.4	14.4
	5981	1	19.1	19.1
	5991	1	25.6	25.6
NUMBER	99		2091.1	2011.6
	1031	2	13.9	7.0
	1071	2	22.5	22.5
	1072	2	5.9	5.9
	1081	2	23.4	23.4
	1082	2	7.3	7.3
	1301	2	20.5	10.3
	1331	2	43.0	21.5
	1481	2	21.4	21.4
	1531	2	7.5	7.5
	1601	2	48.1	48.1
	1731	2	26.2	13.1
	1911	2	31.9	31.9
	1921	2	40.5	20.3
	1931	2	56.7	28.4
	1932	2	32.0	16.0
	1941	2	12.3	6.2
	3291	2	7.6	7.6
	5321	2	34.8	34.8
	5341	2	33.2	33.2
	5342	2	51.7	25.9
	5343	2	35.4	35.4
	5462	2	10.6	10.6
	5602	2	23.7	23.7
	5861	2	15.4	15.4
	7111	2	25.5	25.5
	7123	2	15.2	7.6

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
7141	2	5.4	5.4
7151	2	36.7	36.7
7161	2	28.7	28.7
7181	2	22.3	22.3
7191	2	20.7	20.7
7201	2	15.2	15.2
7211	2	25.4	25.4
7221	2	24.4	24.4
7231	2	24.9	12.5
7241	2	72.5	72.5
7242	2	9.1	9.1
7291	2	10.0	10.0
7321	2	11.6	11.6
7331	2	23.4	23.4
7341	2	7.2	7.2
7351	2	15.2	15.2
7361	2	39.8	19.9
7371	2	14.9	14.9
7372	2	21.2	21.2
7381	2	30.4	30.4
7382	2	13.9	13.9
7401	2	21.2	21.2
7411	2	38.1	38.1
7421	2	34.9	34.9
7432	2	43.2	21.6
7433	2	33.5	16.8
7442	2	26.3	26.3
7461	2	18.0	18.0
7481	2	33.2	33.2
7482	2	27.4	27.4
7491	2	51.6	51.6
7492	2	17.6	17.6
7521	2	3.4	3.4
7623	2	29.7	29.7
7781	2	16.5	16.5
7791	2	18.6	18.6
7801	2	19.6	19.6
7821	2	38.1	38.1
7851	2	27.6	27.6
7861	2	22.1	22.1
7872	2	10.7	10.7
7901	2	49.9	49.9
7921	2	6.3	6.3

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

	BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
	7921	2	6.3	6.3
	7941	2	30.8	30.8
	7952	2	8.6	8.6
	7961	2	19.5	19.5
	9021	2	26.6	26.6
	9051	2	23.0	23.0
	9061	2	27.8	27.8
	9081	2	8.8	8.8
	9181	2	14.9	14.9
	9191	2	9.8	9.8
	9201	2	12.5	12.5
	9211	2	12.8	6.4
	9221	2	23.0	11.5
	9251	2	35.7	35.7
	9282	2	36.2	18.1
	9291	2	31.3	31.3
	9361	2	25.0	25.0
	9371	2	45.1	45.1
	9381	2	19.0	19.0
	9382	2	22.4	22.4
	9391	2	46.0	46.0
	9401	2	18.0	18.0
	9411	2	48.3	48.3
	9451	2	29.4	29.4
	9471	2	20.6	20.6
	9481	2	16.3	16.3
	9491	2	26.4	26.4
	9501	2	39.4	39.4
	9552	2	24.7	24.7
	9701	2	37.4	18.7
	9702	2	6.0	6.0
	9711	2	23.6	23.6
	9721	2	16.5	16.5
	9731	2	27.0	27.0
	9741	2	40.2	40.2
NUMBER	104		2585.5	2304.5
	1051	3	13.0	13.0
	1061	3	24.6	24.6
	1211	3	10.9	10.9
	1241	3	30.1	30.1

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
1302	3	29.3	29.3
1321	3	23.3	23.3
1401	3	25.4	25.4
1421	3	29.7	29.7
1422	3	34.0	34.0
1431	3	12.6	12.6
1441	3	35.8	35.8
1452	3	23.6	23.6
1501	3	9.2	9.2
1502	3	17.3	17.3
1511	3	29.6	29.6
1521	3	28.2	28.2
1551	3	27.8	27.8
1561	3	43.3	43.3
1581	3	15.1	15.1
1591	3	29.8	14.9
1631	3	69.5	46.3
1641	3	35.0	35.0
1651	3	39.9	39.9
1661	3	32.8	32.8
1741	3	17.7	17.7
1751	3	15.0	15.0
1771	3	26.1	26.1
1781	3	16.0	16.0
1791	3	38.2	38.2
1821	3	17.9	17.9
1831	3	48.0	32.0
1871	3	10.6	10.6
1881	3	14.2	14.2
1891	3	8.8	8.8
5021	3	23.1	23.1
5031	3	17.2	17.2
5041	3	24.4	24.4
5061	3	31.9	31.9
5081	3	21.9	11.0
5091	3	33.1	33.1
5092	3	14.2	14.2
5101	3	18.3	18.3
5111	3	24.1	24.1
7051	3	56.6	37.7
7131	3	17.7	17.7
7431	3	31.8	31.8
7441	3	48.2	48.2

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

	BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
	7601	3	21.6	21.6
	7811	3	39.0	39.0
	7831	3	29.8	29.8
	7871	3	15.4	15.4
	7912	3	68.4	68.4
	7951	3	38.5	38.5
	7962	3	27.8	27.8
	7991	3	26.2	26.2
	9151	3	47.3	47.3
NUMBER	56		1558.8	1474.9
	7622	CEMETERY (C)	42.6	0.0
	1451	C	43.5	0.0
NUMBER	2		86.1	0
	1003	FREEWAY (F)	18.6	0.0
	1083	F	9.6	0.0
	2002	F	44.0	0.0
	2003	F	23.8	0.0
	5122	F	27.9	0.0
	5153	F	7.7	0.0
	5372	F	2.0	0.0
	5382	F	52.7	0.0
	5391	F	10.4	0.0
	5561	F	15.2	0.0
	5661	F	3.6	0.0
	5741	F	10.1	0.0
	7162	F	6.0	0.0
	9032	F	7.0	0.0
	9071	F	15.9	0.0
	9101	F	20.6	0.0
	9131	F	23.8	0.0
	1002	F	26.6	0.0
NUMBER	18		325.5	0
	3613	TO MAIN (M)	11.4	0.0
	5011	M	2.2	0.0

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

	BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
	5281	M	4.4	0.0
	5291	M	3.5	0.0
	7011	M	2.5	0.0
	7451	M	8.6	0.0
	7911	M	10.9	0.0
	7931	M	9.2	0.0
	9052	M	16.9	0.0
	9452	M	15.7	0.0
	3571	M	3.2	0.0
NUMBER	11		88.5	0
	1361	OPEN AREA (OA)	5.7	0.0
NUMBER	1		5.7	0
	1922	PARK (P)	51.7	0.0
	1923	P	9.0	0.0
	3541	P	9.7	0.0
	5742	P	43.9	0.0
	7031	P	5.4	0.0
	7041	P	5.7	0.0
	9241	P	30.0	0.0
	1861	P	46.5	0.0
NUMBER	8	0	201.9	0
	7121	RAILYARD (RY)	31.2	0.0
	7122	RY	27.0	0.0
NUMBER	2	0	58.2	0
	1721	SCHOOL (S)	29.8	0.0
	5771	S	6.3	0.0
	5781	S	3.9	0.0
	5791	S	4.4	0.0
	7101	S	10.2	0.0
	7522	S	1.4	0.0
	7541	S	8.9	0.0

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

	BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
	7561	S	4.2	0.0
	7581	S	7.4	0.0
	7641	S	2.0	0.0
	7651	S	4.0	0.0
	7681	S	4.0	0.0
	7682	S	2.4	0.0
	7683	S	3.7	0.0
	7684	S	1.6	0.0
	7691	S	1.8	0.0
	7701	S	3.3	0.0
	7711	S	4.9	0.0
	7741	S	2.8	0.0
	7751	S	5.1	0.0
	7761	S	2.1	0.0
	7771	S	4.2	0.0
	7971	S	5.2	0.0
	7972	S	7.2	0.0
	7973	S	16.2	0.0
	7974	S	14.9	0.0
	7992	S	8.7	0.0
	9031	S	2.7	0.0
	9281	S	1.0	0.0
	9301	S	2.8	0.0
	9311	S	3.5	0.0
	9331	S	0.8	0.0
	9341	S	2.8	0.0
	9342	S	7.1	0.0
	9551	S	4.3	0.0
	9581	S	6.7	0.0
	9591	S	3.0	0.0
	9611	S	1.5	0.0
	9612	S	1.5	0.0
	9621	S	2.0	0.0
	9631	S	2.8	0.0
	9641	S	7.1	0.0
	9661	S	5.5	0.0
	1062	S	34.5	0.0
NUMBER	44	0	260.2	0
	7322	STORM WATER (SW)	22.3	0.0
	7323	SW	33.3	0.0

ATTACHMENT 1
BASIN CATEGORIZATION BY TYPE
COMBINED SEWER COLLECTION SYSTEM
CITY OF SACRAMENTO

	BASIN #	REPRESENT. TYPE	AREA (AC)	APPLICABLE AREA (AC)
NUMBER	2	0	55.6	0
NUMBER	347	TOTAL =	7317	5791

ATTACHMENT 2

1977 CITY REPORT ON PIPE DIAMETERS

REPORT

ON

PIPE DIAMETER AND LENGTH

SEWER AND DRAINAGE COLLECTIONS SYSTEMS

CITY OF SACRAMENTO

DIVISION OF WATER AND SEWERS

927 - 10TH ST.
SUITE #201

SACRAMENTO, CALIFORNIA 95814
TELEPHONE (916) 449-5271

HARRY G. BEHRENS
MANAGER

ROBERT W. JOHNSTON
ASSISTANT MANAGER

cc: H. Behrens
R. Johnston
I. Clark
D. Kness
W. Hetland

March 1977

Total Combined Sanitary Storm	
Pipe Dia., In.	Pipe Length -Ft.
4	2,420
6	237,760
8	762,100
10	120,630
12	89,190
14	18,395
15	32,080
16	26,770
18	38,570
20	12,490
21	6,970
22	11,400
24	40,440
27	9,130
28	450
30	10,150
33	9,530
36	7,500
39	6,100
42	12,180
44	2,470
45	11,180
48	11,440
51	2,560
52	2,000
54	5,100

Σ = 1,049,165

APPENDIX K

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 6

DESIGN AND ANALYSIS ASSUMPTIONS

Greg Jones
HDR Engineering, Inc.

August 13, 1992

TECHNICAL MEMORANDUM
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN
PHASE 3, TASK 6

PREPARED BY: Greg Jones, HDR Engineering, Inc.
DATE: July 1, 1991
SUBJECT: Design and Analysis Assumptions

A. INTRODUCTION

The assumptions and procedures outlined in this memorandum were used in the Phase 3 study effort. This memorandum:

- Gives an overview of the SWMM program used to evaluate alternative improvement plans,
- Describes the basis for the design storms chosen for alternative sizing,
- Identifies the Phase 3 alternative improvement plans,
- Summarizes the design criteria and assumptions applied to the alternatives evaluated, and
- Discusses the modeling of CSO operational criteria and the modeling of existing facilities.

B. THE SACRAMENTO SWMM PROGRAM

The City of Sacramento's combined sewer system has been modeled with a modified version of the EPA's Storm Water Management Model, SWMM, a computer model used to simulate hydrologic/hydraulic systems, including conveyance, storage and treatment facilities.

The Sacramento SWMM computer program, called SSWMM91, consists of two sub-programs that are executed separately. The first program is called the RUNOFF block. The RUNOFF block has the capability of simulating runoff from defined drainage basins, given a precipitation event and the physical parameters of the

drainage basins such as the basin area, basin width, impervious percentage, and infiltration capacity.

The second program is called the EXTRAN block. This program has the ability to simulate the entry of runoff into a piped collection system and the gravity or pressure flow of water in that collection system. EXTRAN input consists of the output hydrographs generated by the RUNOFF block, sanitary flow rates, and data on the lengths, diameters, slopes and Manning's roughness coefficients for the conduits. In addition, a surface "overflow channel" system can also be modeled that represents the routing of floodwater through the streets. The model provides a dynamic simulation; flow rates, velocities and water surface elevations are reported for the collection system in a time history format over the defined duration of the rainstorm event.

A more detailed review of SSWMM91 is provided in the Phase 2 technical memorandum entitled *SWMM Expansion, Calibration, and Baseline Flooding Results for the 10-Year 6-Hour Storm*, dated July 1, 1991. That document is Appendix E to the *Phase 2 Detailed Technical Report*.

The EPA program (unmodified version) is described in detail in the current version of the EPA's *Users Manual*, including appendices available from the EPA Office of Municipal Environmental Research Center, Office of Research and Development, Cincinnati, Ohio. The following discussion assumes a limited knowledge of the model. Details of the model configuration, program code and limitations can be reviewed in the EPA *Users Manual*.

C. FLOOD AND CSO CONTROL DESIGN STORMS

Design storms are a hypothetical distribution of rainfall depths or intensities over a time increment for a given storm duration and frequency. Synthetic design storms were used to determine the performance of proposed improvement plans measured in terms of flood and CSO control. For this planning study, the flood control design storms were also used as a measure of the effectiveness of the alternatives to control street outflows.

Two sets of design storms were used to analyze alternative improvement plans. The first involves design storms for flood/outflow control project sizing. Design storms for flood control are six hours in duration with return periods of 2, 5, 10 and 25 years. The second involves design storms for sizing CSO control facilities, such as new storage facilities and treatment plants. Storm frequencies of 1/year and 5/year were used for CSO control project sizing.

The use of design storms in designing conveyance and storage facilities is widespread. Design storms are valuable in that they provide a relative measure by which several alternatives can be compared. The use of design storms, however, does require some assumptions that may not apply in all cases.

The most critical of these assumptions is that the frequency of the precipitation will produce runoff and facility response of the same frequency. This assumption may not always be true due to varying conditions in the drainage basin and conveyance system. For example, the same precipitation may be applied to a basin and the timing and volume of the runoff may differ as a result of the initial moisture in the soil from previous events. The soil will infiltrate much more precipitation and produce less runoff in a fall storm with little previous rainfall than in a spring storm with previous rainfall. The assumption that a storm event of a particular frequency will produce a conveyance system response of the same frequency can also be invalid in the case of a conveyance system that has a significant amount of storage.

These types of problems associated with use of synthetic design storms in analyzing a system as hydraulically complex as the CWCS can be solved through the use of continuous simulation. Continuous simulation involves modeling a system using a long-term rainfall record (usually hourly) or a selected representative period (e.g. 10 years) of that record. Statistics, such a CSO or flooding frequency, can then be derived directly from the simulation under actual, historical rainfall conditions rather than extrapolated from the use of single design events. It was determined by City staff, that this approach would be a refinement to the current study, and would be completed during future evaluations.

C.1. Flood Control Design Storms

Improvement plans were configured so as to eliminate the occurrence of street floodwater when subjected to a synthetic design storm. The level of flood control afforded by a particular improvement plan was defined by the recurrence interval of the design storm used to configure that improvement plan.

The flood control design storm recurrence intervals of 2, 5, 10, and 25 years were established by the City based on the results of the Phase 2 analysis, and based on comments received from the Regional Board on the *Phase 2 Detailed Technical Report*. The design storms were developed and input to the RUNOFF block for generating hydrographs for the combined service area basins. Conveyance and storage improvements to control flooding were then sized and configured to convey the resulting flows in the system.

C.2. CSO Control Design Storms

In addition to flood control design storms, alternative improvement plans were evaluated for the control of design events representing frequencies of 1/year and 5/year storms. The underlying assumption made by the City was that the design storm frequency for the system would correspond to the CSO frequency for the existing system. As previously discussed, this underlying assumption may not necessarily be correct for the combined sewer system.

The use of the 1/year design storm is based on a Phase 2 technical memorandum *Survey of Cities Developing or Implementing CSO Control Plans*, dated July 1, 1991. This study reported that a number of states impose a limitation of one CSO event per year, or control of CSOs for the 1/year storm event. One of the objectives of the Phase 3 study was to identify the costs of providing CSO control to a similar level, i.e. the 1/year storm, for Sacramento.

The objective of the 5/year design storm analysis was intended to assess the CSO upgrade costs, if any, to provide a CSO control protection level similar to the existing CWCS. Based on 4 years of recent historical data the existing CWCS spills approximately 4 to 5 times per year to the Sacramento River. The use of

the 5/year storm was intended to replicate existing CSO conditions, again based on the City's assumption of equating rainfall and CSO frequency.

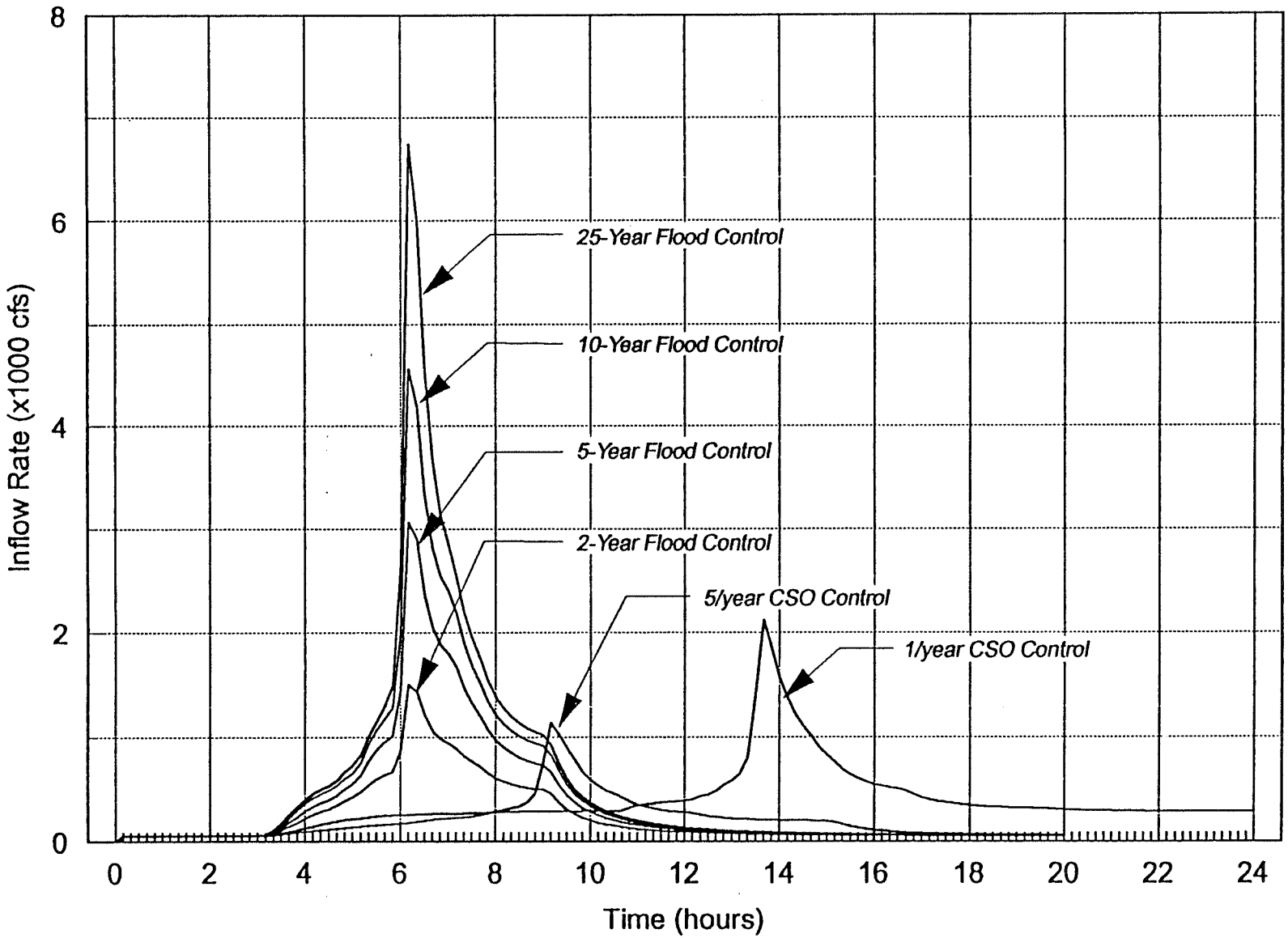
The flood control and CSO design storms were derived in accordance with the procedures outlined in the Phase 3, Task 3 technical memorandum (TM) *Design Storms, Rainfall Analysis and Flood Volume Maps for the Phase 3 Analysis*.

The runoff hydrographs into the CWCS resulting from the above discussed design storms are shown in Figure 1.

C.3. Discussion of Outflow Control

The Cease and Desist Order levied on the City by the Regional Water Quality Board specifically addressed the control of potential health threats posed by combined sewage outflows. Quantifying existing and future reduced outflows associated with alternative plans would provide a criterion, in addition to flood and CSO control, for selecting an optimum plan. One of the initial efforts in the Phase 3 study was to determine whether outflows from the system could be quantified, given the analytical tools available. Following the submittal of the *Phase 2 Detailed Technical Report*, the Regional Board also requested that the City determine the amount of outflows, and to differentiate floodwater caused by stormwater incapable of entering the system from floodwater caused by outflows from the surcharged system. The City was asked to develop costs and improvements associated only with the control of system outflows. Based on an extensive review of available analytical tools and the technical aspects of modeling outflows, it was determined that the requested analysis could not be completed within the time constraints of the Phase 3 study. Furthermore, it was estimated that the time and expense necessary to develop a theoretical computer model and cost allocation procedures would be unreasonable. Finally, the modeling effort could prove impossible to carry out satisfactorily to satisfy the Regional Board's request.

For these reasons, it was determined that outflow control benefits could only be measured relative to flood control benefits. That is, if an alternative improvement plan was developed to control street flooding for a given design storm, then it was assumed that outflows to the street would also be controlled



SYSTEM INFLOW HYDROGRAPHS FOR THE CSO AND FLOOD CONTROL DESIGN STORMS

HDR

Figure 1

for that event. The development of improvement plans and the preparation of the benefit/cost analysis was based on this assumption.

D. OVERVIEW OF ALTERNATIVE IMPROVEMENT PLANS EVALUATED FOR PHASE 3

Improvement plan alternatives can be classified according to the type of facilities incorporated into the proposed plan. The types of improvement plans evaluated include:

- Tunnel Alternatives.
- Storage Alternatives.
- Treatment Alternatives.
- Separate Sanitary Sewer Alternative.
- Separate Storm Drain Alternative.
- Inflow Control Alternative.
- Localized Flooding Improvement Alternative.
- Baseline Rehabilitation Alternative.

The Phase 3 improvement plan alternatives were configured to meet both flood and CSO control design storm criteria. The performance of proposed improvements for providing flood and CSO control was verified using the SSWMM91.

For the storage, treatment and tunnel alternatives, flood control was achieved by upsizing the existing conveyance system, and by relieving excess flows from the existing system using weirs to divert flow into a network of consolidation conduits. The network of consolidation conduits drain to designated storage and/or treatment sites, or into a tunnel system. The conduits were sized to eliminate flooding in the combined sewer service area for a given design storm.

To configure alternatives to also provide CSO control, the 1/year and 5/year CSO design storms were applied to the individual improvement plans configured for the 2-, 5-, and 10-year flood control design storm events. The combined wastewater flows and volumes resulting from the application of the CSO storms were used in determining the sizes of storage, treatment, and tunnel storage needed for CSO control.

It was determined from model runs that CSO control for the 5/year design storm is provided by the existing CWCS facilities, operating under the original plan of operations. Therefore, all alternative improvement plans configured for the 5/year CSO design storm consisted of only the facilities that would be needed for flood control. CSO control for the 1/year design storm required storage and/or treatment improvements in addition to the flood control improvements.

Five of the alternatives evaluated during the Phase 3 study were not developed to control both a predefined CSO and flood design storm, e.g. the 5-year flood control and 1/year CSO control design storms. Furthermore, some of these alternatives were evaluated only from a general feasibility standpoint. These include:

- Separate Sanitary Sewer Alternative.
- Separate Storm Drain Alternative.
- Inflow Control Alternative.
- Localized Flooding Improvement Alternative.
- Baseline Rehabilitation Alternative.

Descriptions of the alternative improvement plans and design considerations unique to each are summarized in the sections below. Detailed information regarding configurations, materials, and mechanical and electrical equipment pertinent to these alternatives and components are included in two other Phase 3 TMs, i.e., *Unit Cost Refinements*, and *Improvement Plans*.

D.1. Tunnels

The tunnel alternatives consist of three components used to convey or store combined sewage. The first component consists of replaced service laterals and collectors. These replaced conduits convey flow to the mains in the existing combined sewer system. Excess flow that cannot be handled by these mains is drawn off by diversion weirs and directed into the second main component, a network of new consolidation conduits. The consolidation conduits receive excess flows at one or more locations and convey the flow to tunnel drop shafts. These drop shafts empty into the tunnel system, the third component of the system.

Flows in the tunnel are directed to terminal points near Sumps 1A and 2. A portion of the total inflow into the tunnels was retained within the tunnels by discharge control structures located at the ends of the tunnels at Sumps 1A and 2. These discharge control structures were set to retain the design CSO control volumes within the tunnels. Provisions were also included to allow diversion of Pioneer Reservoir overflows into the tunnel network for CSO control. Tunnel pump stations discharge all flows in excess of the CSO volume directly to the river without treatment up to the point where the combined sewer service area no longer contributes significant runoff to the tunnel system. At that point, pumped discharge to the river ends and the contents of the tunnel system are stored for secondary treatment at SRWTP during off-peak flow conditions.

The analysis of the tunnel alternatives included variations in tunnel diameters, storage, and discharge control. These parameters were varied to configure the alternatives for the flood control (5-, 10-, and 25-year) and the CSO control (1/year and 5/year) design storm events. The sensitivity of these design parameters to cost was also considered. For the 2-year design storm, tunnels were not cost effective, since most of the existing piping system can convey the 2-year storm without flooding.

Tunnel system configurations were influenced by constructibility limitations. One of these limitations was that the tunnels should be a minimum of one mile long if they are to be cost competitive with other improvement plan alternatives. A major portion of the tunnel construction cost is in contractor mobilization and construction of the tunnel access shaft. Minimum constant-diameter tunnel sections of one mile are needed to distribute these front-end costs over the entire project.

Another construction limitation involved tunnel alignment. Tunnel boring machines cannot construct bends with a centerline radius smaller than roughly 200 feet. Consequently, the right-of-way for tunnel projects had to be relatively straight, using only long radius bends where required.

The Phase 2 recommendation for a tunnel depth of approximately 50 feet was used in configuring the tunnel networks considered for the Phase 3 study.

D.2. Storage

As with the tunnel alternatives, the storage alternatives were configured assuming new diversion weirs installed on the existing collection system. Instead of conveyance to a tunnel system, diverted flows were conveyed to storage sites by consolidation conduits.

The analysis of storage alternatives involved the use of both flood control and CSO control design storms run in the SSWMM91, similar to the analysis of tunnel alternatives. Storage volumes and pumping capacities required for both flood and CSO control were sized from the model output file data on peak flows and inflow volumes to storage sites.

Three sites were considered for proposed storage reservoirs: Pioneer Reservoir, Western Pacific Railroad and UC Davis Medical Center. Combinations of these sites were considered in improvement plan alternatives. These combinations are listed in Table 1.

TABLE 1. STORAGE ALTERNATIVE NOMENCLATURE

Alternative Abbreviation	Storage Sites Used in Alternative
UCDP	UC Davis and Pioneer Sites
WPRR	Western Pacific Site
WPP	Western Pacific and Pioneer Sites
WPUCD	Western Pacific and UC Davis Sites
WPUCDP	Western Pacific, UC Davis and Pioneer Sites

In addition to the alternatives listed in Table 1, two other conveyance alternatives were configured for the 2-year design storm. These consisted of using the Pioneer Reservoir and WPRR sites for locating storage facilities, with flow delivered by upsizing the existing CWCS without using side overflow

diversion weirs and consolidation conduits. These two alternatives are referred to as the "2-Year Upsized CWCS" alternatives.

The WPRR and Pioneer Reservoir sites are close enough to the Sacramento River that combined wastewater exceeding the volume produced by the CSO design storm could be discharged to the river. River discharge configurations have potentially lower costs than storing the entire design storm volume at these sites. Flows directed to the UCD Medical Center would be too far from the river for discharge. The UCD Medical Center requires flow to be stored at the site and discharged to the existing collection system during off-peak flow conditions for secondary treatment.

Overflow from the existing Pioneer Reservoir was diverted back into the network of consolidation conduits for the 5- and 10-year alternatives, in order to meet CSO control objectives. The exception was for alternatives that include a facility at Pioneer Reservoir. For the 2-year alternatives, CSO control was provided either by storage at the Pioneer Reservoir site and/or by storage at the WPRR site. A diversion structure was located on the existing Donner trunk at the WPRR site to divert stormwater flow into the facilities.

Storage reservoirs used in improvement plans were assumed to be similar in construction to the existing Pioneer Reservoir. The elevations of the proposed storage reservoirs were assumed to be high enough to avoid construction below the local area water table. This assumption necessitated the use of an influent pumping station to lift wastewater from the consolidation conduits up into the storage reservoirs.

D.3. Treatment

The treatment alternatives utilized the same combinations of sites as the storage alternatives as listed in Table 1. Flood control for the 2-, 5- and 10-year design storms was provided by upgrading the existing CWCS and/or providing networks of weirs and consolidation conduits to divert flows to the sites. Three of the alternatives also considered storage at the UCD Medical Center site for flood control. The treatment facilities were sized to control the CSO design

storms. Flows in excess of that required to control the CSO storms were pumped to the river without treatment.

The analysis of treatment alternatives considered treatment at four sites: Pioneer Reservoir, WPRR, Meadowview and SRWTP. Conveyance to the Meadowview and SRWTP sites was provided by a pumping station at the WPRR site and a long force main connecting the pumping station to the candidate treatment sites.

WPRR and Pioneer sites were evaluated to determine if the addition of flow equalization storage would lower the overall cost of the alternative. Inflow hydrographs to the facilities generated from the CSO design storms were used to perform a cost optimization of the treatment facility flow rate versus storage facility volume. From this optimization, a rated treatment capacity and storage volume was determined for each site. All flows less than or equal to the rated capacity of the treatment facility were treated. All flows in excess of the treatment rate were stored until the available storage is filled. Inflow rates and volumes in excess of the primary treatment and storage facilities respectively are discharged to the river without treatment.

The use of available flow equalization storage was maximized throughout the storm, and all water remaining in storage after the storm would receive primary treatment. Primary treatment consists of chlorination, sedimentation with a minimum of 1.5 hours of detention time for the peak operational flow rate, and dechlorination prior to discharge to the river. This assumed mode of operation for flow equalization basins differs slightly from the "storage-only" alternatives. For the latter type of alternative plan, water remaining in the storage facilities was discharged to the existing conveyance system for treatment at the SRWTP.

Effluent quality for combined wastewater treatment facilities will comply with the existing City of Sacramento NPDES discharge permit. This permit requires a minimum of primary treatment with disinfection and dechlorination.

D.4. Separation

Two improvement plan alternatives were configured based on the concept of separating the combined wastewater conveyance system. One assumed construction of a new sanitary sewer system, the other a new storm drainage system. The area served by these new sewer systems would be coincident with the existing combined system.

D.4.a. New Storm Drain System--

The Separate Storm Drain Alternative was configured by assuming the existing combined sewer system would remain in service as a sanitary sewer system. The new system provided drainage for the entire combined sewer service area, excluding the existing separated sewer service area. Sizing of new sewers was based on conveying the 10-year design storm. Analysis using CSO design storms was not needed, since this alternative eliminates CSOs. New storm sewers were configured to drain to two new stormwater pumping stations located near the existing pumping facilities at Sump 1A and Sump 2. Collected stormwater would be pumped to the Sacramento River. Routing of the new storm sewers followed the existing combined sewer conduits modeled in the SSWMM91.

D.4.b. New Sanitary Sewer System--

The new sanitary sewer system assumed the existing combined sewer collection system would remain in service as a storm drainage system. New sanitary sewers modeled in the SSWMM91 were sized using sanitary hydrographs. These were developed using a diurnal curve approach that included the peaking factors recommended in the City's *Design and Procedures Manual and Improvement Standards*. This approach was conservative, in that the resulting predicted peak flows included a design factor of safety. It is anticipated that actual peak flows would be lower. Individual drainage basin sanitary hydrographs were derived by imposing the diurnal curve in the average sanitary flow estimates prepared by the City. Added to this was an allowance of 200 gpd/acre to account for groundwater infiltration.

The network of new sanitary sewers needed to serve the basins modeled in the SSWMM91 was assumed to parallel the existing modeled combined sewers. New sewer

sizes were estimated by reducing the size of the existing system conduits to convey the estimated peak flows generated by the routing of the sanitary hydrographs. New sewers were configured to drain to Sump 2 for conveyance to SRWTP for secondary treatment.

By contrast, separation of the existing collection system sewers (pipes less than or equal to 16 inches in diameter) was assumed to be a network of uniformly sized 8-inch conduits paralleling the existing collection system. This estimate excluded the existing backyard easement sewers that convey only sanitary flows.

The Separate Sanitary Sewer Alternative includes only a minor flood control upgrade beyond the capacity of the existing system. However, the existing system provides flood control to a 2-year event in most areas. CSOs are eliminated, and flood control is slightly improved, by removing the sewage portion of flow from the conveyance system. This alternative also eliminates outflows.

D.5. Inflow Control Alternative

The Phase 3 study included a review of the feasibility of inflow control as an alternative to upsizing the existing conveyance system to control street outflows. Outflows occur when combined wastewater backs out of the conveyance system onto streets or into building structures.

The Inflow Control Alternative involves restricting stormwater inflow into the combined sewer system such that gravity flow conditions are maintained. This controls surcharging and subsequent outflows during storm events. Inflow control is accomplished by installing flow restriction devices at individual drop inlets in the collection system.

Only a feasibility-level evaluation was completed for this alternative, since sufficient data were not available for a complete analysis. The ability of this alternative to meet flood, outflow, or CSO control was not assessed.

D.6. Localized Flooding Improvement Alternative

The City Utilities Department staff developed an alternative that provides limited flood control for six key problem areas in the combined system. This alternative uses a diversion and consolidation conduit system, similar to the other alternatives, to draw off excess flows from the existing sewer system. These conduits lead to underground storage facilities that consist of parallel 12-foot diameter pipes. Because of the time constraints involved in analyzing this alternative, the six areas were isolated from the system for modeling and analysis. Therefore, flows that back up into these six areas (several of which are low in elevation) from adjacent areas were not considered.

D.7. Baseline Rehabilitation Alternative

Minimum improvements to the CWCS are represented in the Baseline Rehabilitation Alternative, which includes structural and other reliability upgrades to the existing pumping stations and conveyance system. No outflow, flood or CSO control improvements are included in this alternative, and no SWMM analysis was performed. Additional description of the components of this alternative is provided in the Phase 3 TM on *Improvement Plans*.

E. SSWMM91 INPUT PARAMETERS

E.1 SSWMM91 Model of the Existing System

All of the alternative improvement plan model networks developed in the Phase 3 study used the existing system model set-up as a foundation. The extent of the modeled study area includes the entire combined sewer service area and adjacent drainage basins that contribute either stormwater or sanitary sewage to the CWCS. All interceptors and most laterals/mains 16 inches and greater in diameter have been included in the EXTRAN input files. Most recently for the Phase 3 study, the operation of Pioneer Reservoir and its outfall, Pioneer Interceptor, and the Flow Control Structure at Sump 2 and its two outfalls, have been incorporated into the EXTRAN input files.

E.2. Revised SSWMM91 Model Parameters

Individual drainage basin characteristics influence the routing and runoff of rainfall incident upon those basins. The Phase 2 study recommended the re-evaluation of hydrologic parameters input to the RUNOFF block and sanitary flows input to the EXTRAN block setup.

For this Phase 3 study, the City Utilities Department staff developed the technical memorandum entitled *SWMM Parameter Refinements to Infiltration Rates, Impervious Percentages, and Sanitary Flows within the Combined Sewer System*. This memorandum is included in the Appendix. Aerial photos and Soil Conservation Service (SCS) soil classification mapping were examined for each basin in SSWMM91. Aerial photos were used to define land use, from which impervious percentages could be determined. By applying typical infiltration rates for each soil type within a basin, a total weighted infiltration rate was also developed for each basin in the RUNOFF block setup files.

This analysis also provided an opportunity to review the amount of sanitary base flow generated by each basin. During the examination of the aerial photos, the land use classification for each basin was established and correlated with the sewage generation rates listed in the City's *Design and Procedures Manual*. Sanitary flow data were input as constant flows to junctions in the EXTRAN block input file. The total average sanitary base flow rate for the combined sewer service area was estimated by City staff to be approximately 32 mgd.

E.3. Infiltration and Inflow Contribution from the Separated Sewer Area

Junction flow data in SSWMM91 for the separated sewer area consists of both sanitary base flow and rainfall-dependent infiltration and inflow (RDI/I). RDI/I is water that enters the sewer system in direct response to rainfall. RDI/I enters the system directly through cross-connections between the storm drain and combined sewer system or through area drains and downspouts illegally connected to the sanitary system. RDI/I also enters the sewer system indirectly through sewer defects, particularly in shallow pipes such as private building laterals.

Rainfall-dependent infiltration for the separated sewer area was selected to be 2,400 gallons per day per acre of drainage basin, for the 10-year design storm event. These rates agree with the draft technical memorandum *Design Flow Criteria*, prepared for the County of Sacramento Sanitation District Sewerage Expansion Study by James M. Montgomery Engineers, dated December 2, 1991. The value of 2,400 gal/acre/day was adjusted according to the storm event considered with the multipliers listed in Table 2. These multipliers were also adopted for the Sacramento County interceptor master planning documents.

TABLE 2. RAINFALL INDUCED I/I MULTIPLIERS

Flood Control Design Storm	Multiplier
2-Year	0.85
5-Year	0.85
10-Year	1.0
25-Year	1.2

F. DESIGN AND ANALYSIS ASSUMPTIONS

Modeling assumptions common to several alternative improvement plans analyzed using SSWMM91 are presented below. Modeling assumptions that are specific to a particular alternative are described in the Phase 3 technical memorandum, *Improvement Plans*.

F.1. General Design Criteria

All new sewers, whether they were part of the collection system, laterals, mains, interceptors or tunnels, were assumed to have a Manning's roughness coefficient of 0.014, which is conservative for large diameter pipes. Commonly used values range from 0.010 for very smooth pipe to 0.016 for older, rough, unfinished pipe. The differences in modeled flow rates resulting from variation of this parameter

are within the accuracy limits of any field flow monitoring and model calibration techniques.

Peak velocities for new sewers were limited to between 2 and 10 feet per second for the applied design storm event.

New sewers were modeled to provide a minimum cover of 2.5 feet. This allows for standard urban street construction practices of placing an aggregate base and pavement over the pipe in thicknesses typical for urban streets.

New sewers were almost exclusively modeled as circular conduits. Exceptions to this were for discharge from the tunnel system and modeling the operation of the Pioneer Reservoir and Flow Control Structure at Sump 2.

All new conduits with the exception of the replacement conduits were designed for approximately 80 percent of the total conduit capacity at the peak of the design storm. In SSWMM91, conduits are joined together at junctions. For new conduits, the crown elevations of the sewers entering and exiting a junction were modeled with the same crown elevations. This assures minimal restrictions in hydraulic transitions from one diameter sewer to another. In sewer design, this would be preferable where adequate grade is available. However, it is not uncommon practice to maintain a constant flow line invert elevation at straight-flow-through junctions where existing surface grades are relatively flat.

F.2. Replacement Pipes

Several modeling options exist for upgrading the existing collection system for flood control. The options selected for alleviating existing flooding conditions are replacement pipes, parallel pipes, and diversion weirs to a consolidation conduit. Factors that influenced the option used were the size of the conduit, the age and condition of the conduit, location of the nearest consolidation conduit and locations of nearby weirs and drop shafts.

Flooding was first alleviated where possible by upsizing existing conduits. This provided the City with a new replacement sewer with extended life for that

section of the sewer system. If a single new sewer could not be used to replace the existing sewer due to cover limitations, a new parallel sewer or a diversion weir was considered. If replacing the existing sewer with a larger sewer would not solve the flooding, or if the undersized sewer was a major interceptor, then a diversion weir was used to relieve flow from the existing system.

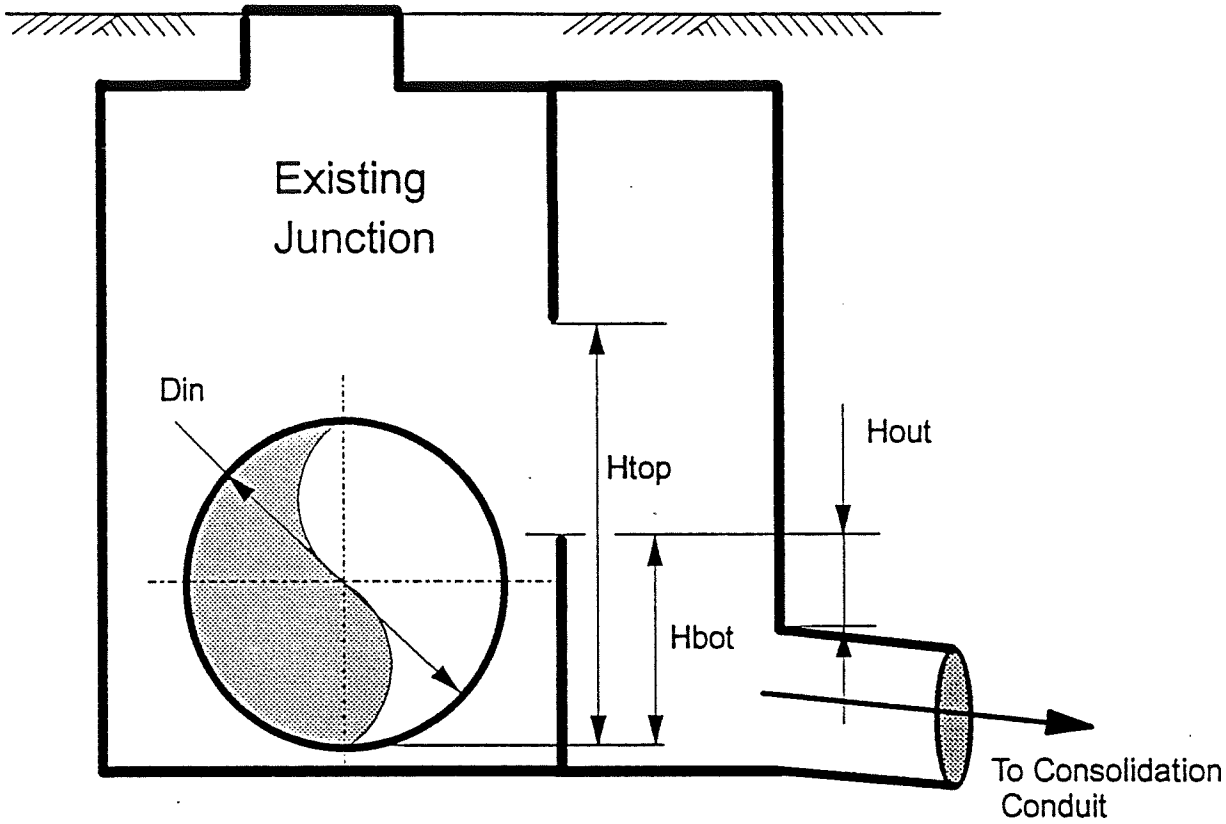
Much of the flooding in the existing combined sewer system is due to undersized laterals and collectors. To draw off excess flows from smaller conduits would not be cost effective. Therefore, these undersized conduits were upsized in order to convey flow to more central locations were the flow was diverted by means of diversion weirs.

The replacement conduits were designed to take advantage of higher conveyance due to surcharged conditions. This was done for several reasons. First, pipes designed for open channel flow conditions would end up being considerably larger than the mains to which they connect. Second, the larger diameters would have difficulty meeting cover requirements. New replacement conduits were consequently designed to be surcharged. The existing conduits remaining in place were also allowed to surcharge. However the pipes were sized to control the surcharge levels and to maintain the hydraulic grade line between 2 and 3 feet below the manhole lid elevation. Exceptions exist where manholes are shallow or where the cost of meeting the criteria outweighs the benefit.

F.3. Diversion Weirs

Diversion weirs were used to relieve the existing CWCS sewer system of excessive flows. These excess flows were collected in a new system of gravity sewers referred to in this study as "consolidation conduits". An illustration of a diversion weir as modeled in the SSWMM91 is provided in Figure 2.

Input data required for modeling the weir includes: the weir length, the weir coefficient, and the height of the bottom and top of the weir with respect to the invert of the junction (as shown in Figure 2). Weir lengths were set at a nominal 10 to 30 feet, with the majority of the weir lengths being 20 feet. The weirs were assumed to be double sided, meaning that an overflow would exist on each



<u>Weir Settings:</u>

Hbot, from 0.33(Din) to 0.80(Din)

Htop - Hbot $\geq$ 5.0 ft

Hout > 0.0 ft

SCHEMATIC ILLUSTRATION OF THE WEIR SETTINGS AS MODELED FOR ALL IMPROVEMENT PLANS UTILIZING DIVERSION WEIRS

HDR

Figure 2

side of the conduit. Therefore, most weirs would be 10 feet long on each side of the structure. The weir coefficient was set at 2.8, a conservative value used to account for a variety of hydraulic conditions. The use of this coefficient was also discussed with our SWMM modeling consultant.

The height of the weirs were set to be a function of the design storm at peak flow, in order to divert excess flows while utilizing the existing combined sewer system as much as possible. The weir height was chosen as a percentage of the diameter of the existing sewer from which flows were directed. For the larger storm events, lower weir heights were necessary to lower the hydraulic grade line for flow in the existing sewers. For the smaller storm events, the weirs were raised to maximize utilization of the existing collection system. As a general guide, the weir heights indicated in Table 3 were used to model the weirs. In some locations, these weir heights had to be lowered in order to control local flooding.

TABLE 3. DIVERSION WEIR CREST DEPTH SETTINGS

Design Storm Recurrence Interval	Weir Crest Depth As A Percentage of Existing Sewer Diameter
2	80
5	80
10	33
25	33

It must be noted that these weir heights are designed for the peak flow of an event, and that during the leading and trailing portions of the design storm, more water may be diverted to the consolidation conduits than necessary, especially with the 10- and 25-year weir settings.

In practice, these weirs would be constructed as weir gates with motor-driven operators and a telephone-signal telemetry control system. This would allow the operations staff to control the amount of water relieved from the existing

collection system. However, only fixed weirs can be modeled by SSWMM91 at this time.

There was concern that by allowing the weirs to be so low in the model during the less frequent design storms, the storage capacity of the existing sewer system would be underutilized. However, the flow into the combined sewer collection system at the peak of the storm exceeds the capacity of the existing system to such an extent, that full flow conditions were reached at one or two junctions downstream of a diversion weir. For the smaller design storm events, weir depths could be set higher and still make full use of the storage capacity of the existing collection system. Again, it is important to remember that the model only allows for simulation of static weir operation, rather than the dynamic operation conditions that would exist in reality.

F.4. Consolidation Conduits

The diversion weirs spill excess flows from the existing system into a network of consolidation conduits which convey the flow to tunnels, storage facilities, or treatment facilities. The consolidation conduits were designed in accordance with the design criteria as outlined in Subsection F.1. The conduits were also designed so that the top of the consolidation conduit would always lie below the edge of the overflow weir to avoid any flow from transferring back across the weir in open channel flow conditions. See Figure 2. Because of SSWMM91 limitations, consolidation conduits and tunnels cannot be surcharged to the extent that the weir becomes surcharged. This condition causes instability in the model results.

The trade-offs between increasing the slope of the pipe in order to decrease the needed diameter was examined to determine the most important consideration with respect to cost. It was determined that to attempt to place the conduits shallow in order to avoid reduced trenching costs resulted in pipe diameters which required the same or even more excavation. The slopes and diameters used for the conduit design are considered reasonable for the design flows.

The storage and treatment alternatives collect and convey large flows which require consolidation conduit diameters in excess of 12 feet. It was assumed that large diameter conduits, up to 12 feet, would be constructed using open trench construction techniques. Above that, the conduits were assumed to be constructed by tunneling. As previously described, tunnels are subject to the construction limitation of using only a single diameter for an entire tunnel segment. To be cost effective, the length of tunnel must be a minimum of a mile long. Consequently where tunneling was required for consolidation conduit diameters greater than 12 feet, a long run of tunnel of constant diameter was used.

G. MODELING CSO OPERATIONAL CRITERIA

The criteria for all new improvement plans used to determine whether combined wastewater would require treatment (either primary or secondary) versus direct discharge to the river approximates the City's permitted operations within the recent past. Below is a summary of the City of Sacramento current discharge prohibitions.

1) The discharge to the Sacramento River is prohibited at the following discharge points unless the specified conditions are met.

    a) CWTP. No discharge unless a flow of 60 mgd is being sent to the SRWTP.

    b) Sump 2 and Pioneer Reservoir. No discharge unless a flow of 130 mgd is being sent to the CWTP, and 60 mgd is being sent to SRWTP.

2) Other than as a result of wet weather, it is prohibited to discharge from the CWTP, Pioneer Reservoir or any sumps to surface waters or surface water drainage courses.

In the development of alternative improvements, the above criteria were adhered to as well as the following additional criteria for storage, treatment, and discharge of combined wastewater during and after a storm event.

1) During a storm event, all tunnel, storage, and treatment alternatives must store the CSO design storm volume, but may discharge to the river any volume in excess of the CSO design storm volume that reaches these facilities

2) After a storm event has ended, any combined wastewater remaining in the tunnel facilities, storage only facilities, and in Pioneer Reservoir must receive secondary treatment at the SRWTP. Any combined sewage remaining in the storage/treatment facilities after a storm event must receive primary treatment. In this context, a storage/treatment facility is used to describe a primary treatment facility with upstream storage for flow equalization.

3) Untreated combined wastewater discharged to the river is considered a CSO. Combined wastewater treated either at a primary treatment facility or at SRWTP and subsequently discharged to the river is not considered a CSO.

4) No new combined wastewater or stormwater outfalls are allowed to discharge into the American River. All new primary effluent, combined wastewater and stormwater discharges would be directed into the Sacramento River.

5) Volumes and flow rates for minimum sizing of storage and primary treatment facilities were estimated using the CSO control design storms (i.e. either 1/year or 5/year design storm frequencies).

H. MODELING EXISTING FACILITIES

The EXTRAN block setup was modified for the Phase 3 study to simulate the operation of major existing facilities in the CWCS. This included Pioneer Reservoir, Pioneer Interceptor, the Flow Control Structure at Sump 2, the interceptors carrying flows south from the Flow Control Structure to the CWTP, and the outfalls to the river at both Pioneer Reservoir and the Sump 2 Flow Control Structure. The capacity of the existing CWCS infrastructure is summarized in Table 4.

These data were used as input to the models simulating the performance and contribution of the existing system in controlling flooding and CSOs.

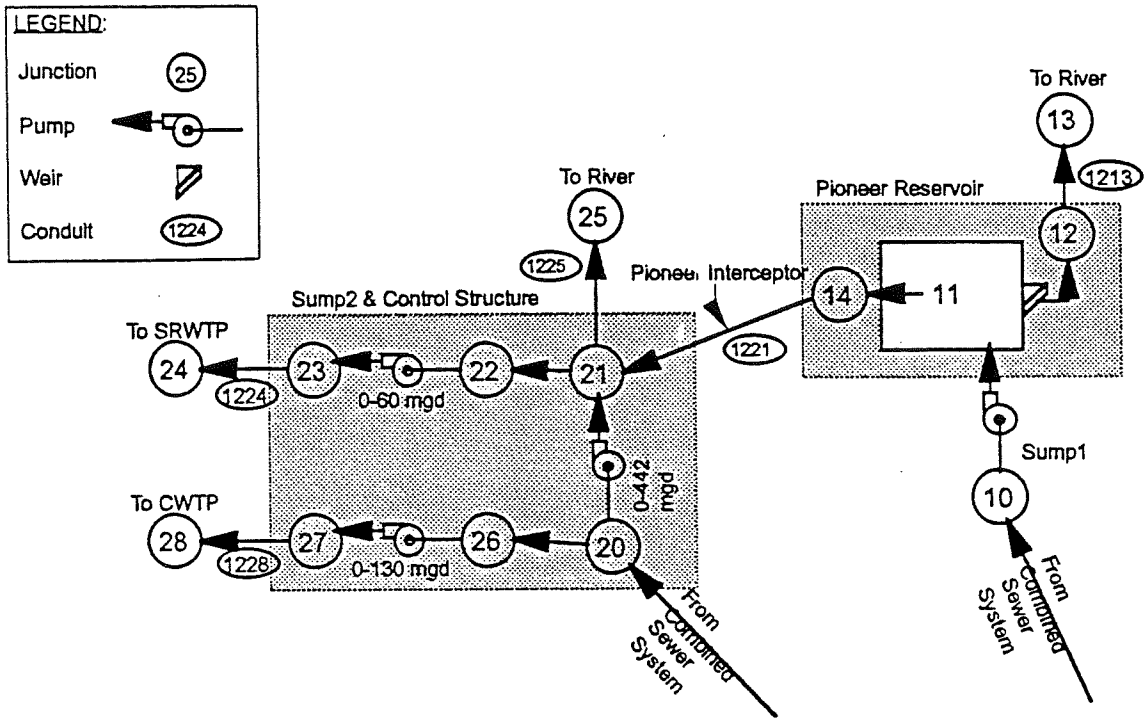
The modeling of these facilities was designed to reflect the original Plan of Operations, which minimized the volume of wastewater spilled to the river. The model network used to simulate this is shown schematically in Figure 3 and explained further in the following subsections.

TABLE 4. CAPACITIES AND PERFORMANCE RATINGS FOR EXISTING CWCS INFRASTRUCTURE

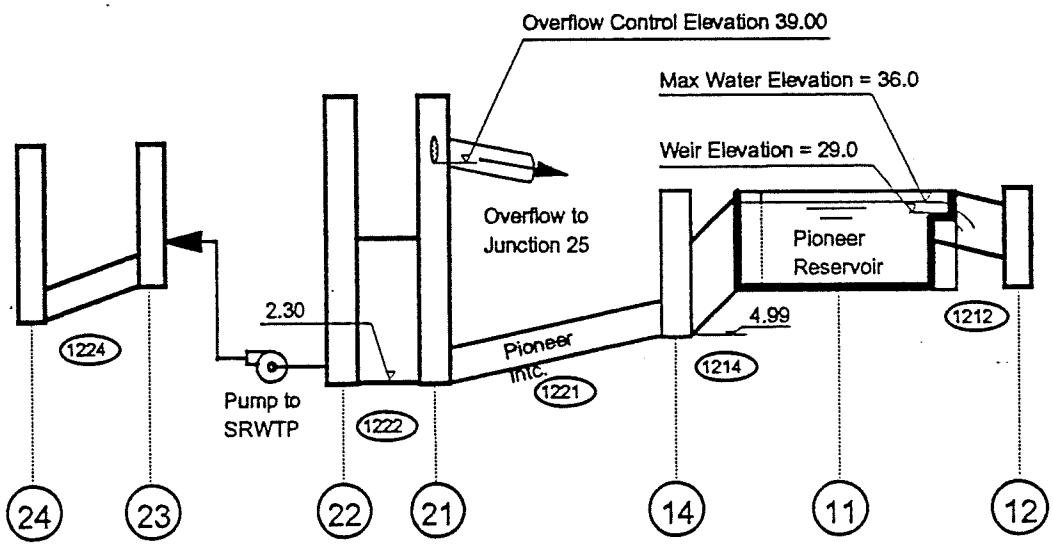
Description	Capacity Rating
Pioneer Reservoir	23 MG
Pioneer Interceptor	120-inch, 9100 lf
CWTP	130 mgd
City Allowance at SRWTP	60 mgd
Sump 1A	84 to 124 mgd Depending upon Pioneer Reservoir Level
Sump 2	572 mgd

G.1. Modeling the Operation of Sump 2

Previous versions of EXTRAN setup files for the CWCS included a pumping station (Sump 2) at Junction 20, which pumped flow to a free outfall. The rate of pumping was controlled by the water elevation at Junction 20, as recommended by the original Plan of Operations for Sump 2. During the Phase 2 study, setpoints for Sumps 1A and 2 were revised from previous work to be in accordance with the Interim Plan of Operations (adopted November 2, 1990). Previous modeling efforts did not provide for any simulation of Pioneer Interceptor conveyance, storage in Pioneer Reservoir, conveyance to treatment facilities, or overflows to the Sacramento River.To simulate pumping operations at Sump 2 and the Flow Control Structure, and provide diversion of pumped flows to treatment, storage and overflows to the river, a total of 3 pumps were used as indicated in Figure 3.



SCHEMATIC LAYOUT OF THE SUMP 2 AND PIONEER RESERVOIR SYSTEM MODEL



SCHEMATIC SECTIONAL VIEW OF THE SUMP 2 AND PIONEER RESERVOIR MODEL SETUP

HDR

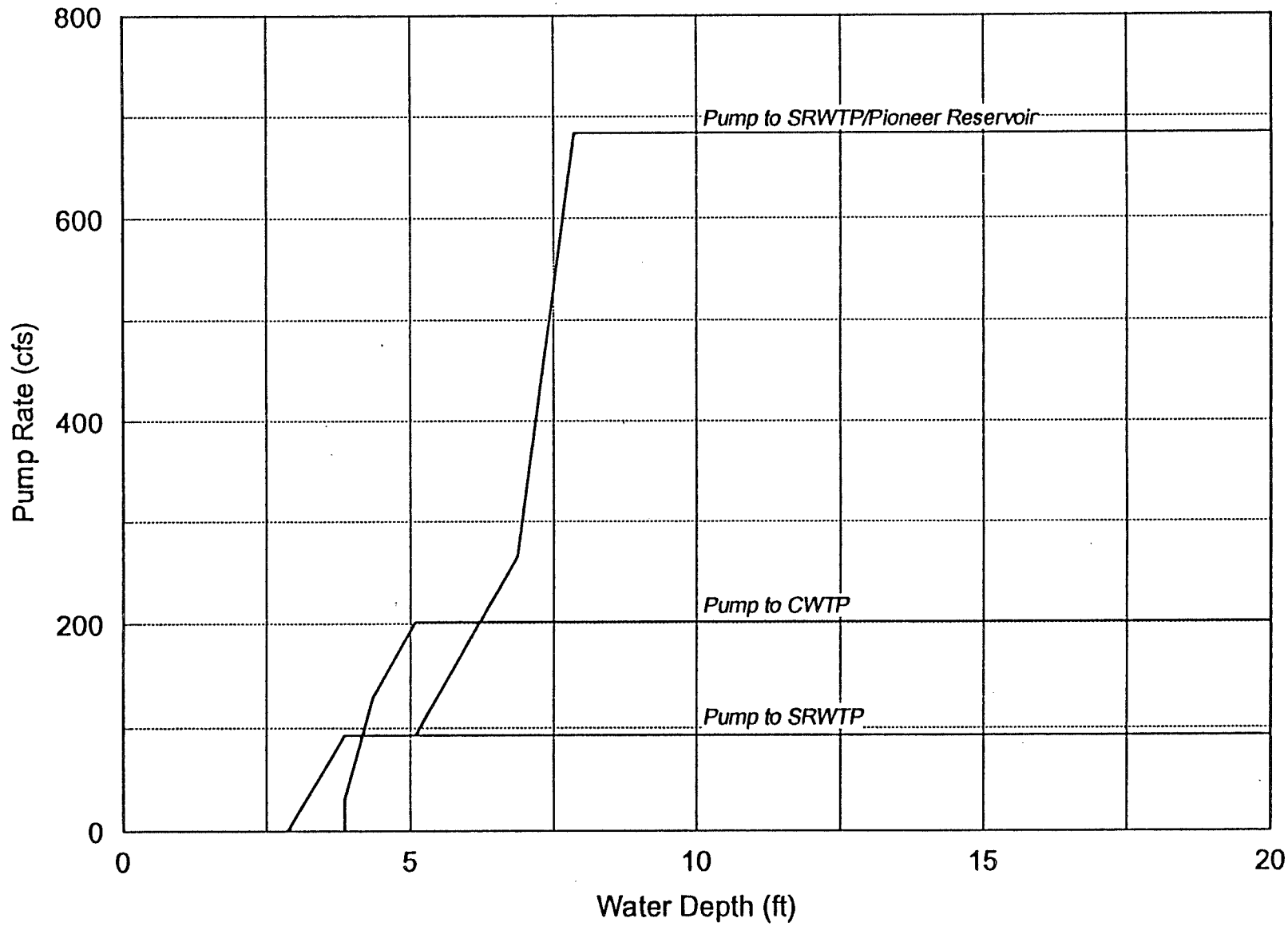
Figure 3

Inflow into Junction 20 is split between two pumps: 1) Pump 26-27 (pumping between Junctions 26 and 27), operating up to a peak output of 130 mgd to simulate delivery to CWTP; and 2) Pump 20-21, delivering flow to Junction 21. Pump 22-23 delivers flow from Junction 21 to the SRWTP up to a peak of 60 mgd, and simulates the diversion of flow from the Flow Control Structure to the SRWTP.

The relationship between water depth in Junction 20 and discharge from these pumps is summarized in Figure 4. This pump discharge curve simulates the overall Sump 2 operation recommended in the original Plan of Operations. Pump 22-23 conveys only the amount of flow diverted from the Sump 2 Flow Control Structure to the SRWTP.

Junction 21, as part of the Flow Control Structure, is also connected to Pioneer Reservoir. Conduit 1221 represents the 120-inch Pioneer Interceptor, modeled with as-built upstream and downstream invert elevations taken from the Sump 2 Plan of Operations.

During periods of low flows to Sump 2 (less than 60 mgd), inflow to Junction 20 is pumped only by Pump 20-21 for transfer to Junction 21. From Junction 21, Pump 22-23 transfers wastewater to SRWTP. As the wastewater inflow to Junction 20 increases, the water level in that junction rises to elevation 4.5 feet and Pump 26-27 starts up to pump wastewater to the CWTP. The pumping rate of Pump 20-21 remains at 60 mgd. When inflow exceeds 190 mgd, Pump 26-27 pumps at the maximum rate of 130 mgd and Pump 20-21 pumps at a rate of between 60 mgd to maximum capacity. With inflow exceeding the maximum capacity of Pump 22-23 (60 mgd), the water elevation in Junction 21 rises and enters Conduit 1221, the Pioneer Interceptor. Continued flow into Junction 21 is conveyed through Conduit 1221 and into Junction 11, the Pioneer Reservoir. Once Pioneer Reservoir is full, it will spill over its weir to the river outlet at Junction 13. When the water elevation in Junction 21 exceeds an elevation of 36.0 feet, wastewater spills to the river from the Flow Control Structure through Conduit 1225.



PUMP CURVES USED IN THE MODELING OF THE SUMP 2/PIONEER RESERVOIR SYSTEM

HDR

Figure 4

As indicated in Figure 4, this only occurs once Pioneer Reservoir is full and spilling from Conduit 1213, and the calculated flow rate being transferred from the Flow Control Structure to Pioneer Reservoir exceeds the design capacity of the Pioneer Interceptor.

G.2. Modeling Operations at the Pioneer Reservoir and Sump 1A

As with Sump 2, the previous versions of SSWMM91 EXTRAN setup provided only for a pumped discharge to a free outfall from Junction 10, Sump 1A. For the Phase 3 study, Pump 10-11 in Figure 4 simulates the operation of the pumps at Sump 1A, discharging directly into the Pioneer Reservoir. Water levels in Junction 10 which initiated the operation of Sump 1A were set according to the original Plan of Operations.

Pioneer Reservoir was simulated by a storage junction, Junction 11. The SSWMM91 requires input data on the incremental volume of available storage within the storage facility per foot of water depth. For model stability, the conduits connecting the storage junction had to be rectangular and a minimum of 500 feet long. The extra volume afforded by these conduits is not available in the actual storage structure so the modeled volume of the storage junction was adjusted accordingly.

The modeled configuration allows wastewater stored in Pioneer Reservoir to drain back to Junction 21, the Flow Control Structure, after the peak flows into Sump 2 have subsided below a rate of 60 mgd. Flows draining back to Junction 21 from Pioneer Reservoir are pumped only to SRWTP for secondary treatment. Pumping to the CWTP stops once the total inflow into Junction 20 drops below the SRWTP treatment rate allowance of 60 mgd.

G.3. Use of Model Modifications

The modified model incorporating the Sump 2 and Pioneer Reservoir/Interceptor infrastructure provided an analytical tool used in Phase 3 to evaluate several aspects of the study. First, it allows SSWMM91 to report hydrographs for overflows to the river individually from Pioneer Reservoir and Sump 2 for a given

rainstorm event. These hydrographs represent overflows that would occur, given the full utilization of all existing primary and secondary treatment capacity and Pioneer Reservoir.

For the treatment and storage alternatives at Pioneer Reservoir, it provided a means of identifying the total inflow hydrograph for the site essential for sizing the required facilities. The revised model setup was also useful in determining the period required for emptying the existing reservoir or a proposed storage reservoir or tunnel system.

APPENDIX A

TECHNICAL MEMORANDUM

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN

PHASE 3, TASK 3

PREPARED BY: Richard Dalrymple, City of Sacramento
Ken Fischer, City of Sacramento

DATE: December 20, 1991

SUBJECT: SWMM Parameter Refinements to Infiltration Rates, Impervious Percentages, and Sanitary Flows Within the Combined Sewer Area

A. INTRODUCTION

This Technical Memorandum (TM) is in response to the Phase 3, Task 3 TM entitled *Recommended Guidelines for Determining Land Use, Impervious Percents and Sanitary Flows within the Combined Sewer Area*. To be consistent, this TM will follow and incorporate the format and verbiage of the above mentioned Recommended Guidelines TM.

The purpose of this memorandum is to provide a detailed description of the methodology used to revise the impervious percent, infiltration rate, and sanitary flow parameters used in the Combined Sewer SWMM input files.

During Phase 2 of the improvement planning study for the Sacramento Combined Sewer System, the existing system set-up and input files for the Sacramento SWMM were expanded and improved. Modifications focused primarily on the EXTRAN model inputs rather than the RUNOFF block inputs, and are described in detail in the TM entitled *SWMM Expansion, Calibration, and Baseline Flooding Results for the 10-Year, 6-Hour Storm* (Appendix E of the Phase 2 Detailed Technical Report).

Work completed during Phase 3 includes additional refinements to the Sacramento SWMM input files for the existing Combined Sewer area (CSA). At the conclusion of Phase 2, it was determined that certain input

parameters to the model warranted examination. These parameters were discussed in Section 6 of the *Phase 2 Detailed Technical Report*. The Sacramento SWMM is being used in Phase 3 to size and analyze a variety of alternative improvement projects for the CSA.

The majority of the work on input data refinement was completed by the City of Sacramento's staff. The City's assessment included:

- The existing land use was determined for each basin in the CSA and for separated areas located east of the downtown area that contribute directly to the CSA. This land use assessment was completed using recent aerial photographs.

- The percent impervious values in the RUNOFF block input file were revised to reflect current land use in the CSA. Impervious percents for each basin in the SWMM RUNOFF block input file were based on the mixture of land use as determined during the land use assessment task.

- The average sanitary flow (in cfs) was determined for each basin in the CSA. This flow was computed based on the land use assessment and the City's existing *Design and Procedure Manual* for sanitary sewer design. Sanitary flows were imputed into the EXTRAN block input files as constant flows.

- Infiltration rates were revised based on the Soil Conservation Service's Soil Group maps. The soil group present in a particular basin determined the infiltration rate for that basin.

B. LAND USE ASSESSMENT

Sanitary flow rates and impervious percentages for a given sanitary/hydrologic basin are based on the mixture of land use within each basin. Once land use in the basin was determined, unit flows or impervious percents could be applied to each of the land use components in the basin to determine a combined, weighted value. For instance, a

basin that is comprised of residential and commercial land uses would exhibit runoff and sanitary flow characteristics in accordance with that particular mixture of land use.

In order to simultaneously determine sanitary flows and impervious percents a single set of land use classifications was used. All available sources for the determination of suitable land use classifications were reviewed. These sources included:

City of Sacramento *Design and Procedures Manual*
City of Sacramento Zoning Maps
City of Sacramento *General Plan*
Central Sacramento Community Plan
Draft of the *Sacramento City/County Drainage Manual*

None of the above sources could be used solely for the determination of land use which are pertinent to the hydrologic parameters and sewage flow rates. After reviewing the available sources, land use categories selected for application in this assessment were based upon the types of land use actually present in the CSA. The land use classifications used were readily discerned from aerial photographs, and were areas which have considerably different sewage flow rates and/or impervious percentages.

The land use classifications used in this assessment are presented in Table 1, along with unit values for sanitary flows and impervious percents. Unit rates for sanitary flows were obtained from the City of Sacramento *Design and Procedure Manual* (dated September 1, 1990). Unit values for impervious percentage were obtained from the Draft of the *Sacramento City/County Drainage Manual*. (Excerpts from these documents are appended.) These values were used as guidelines and judgement was used as required for specific cases.

The task was completed using a 1"=600′ aerial photograph of the entire CSA and several clear overlays of the shed boundaries and soil group boundaries. Since the overlays were 2 to 4% larger than the aerial

photograph, caution was used to compensate for the error. The overlays were adjusted and spot checked with the 1"=500′ as often as possible so that the basin boundaries lined up a close as possible.

The land use assessment consisted of three steps. The first step was determining the basins containing only one land use type. These were easily assessed from the 1"=600′ aerial photos. The second and third steps were determining the basins with multiple land uses. These were often difficult to assess at the 1′=600′ scale, so the City used a set of 1"=200′ aerial photos and delineated the basin boundaries on these 1"=200′ maps. The second step measured the different land uses in a basin using a scale to obtain approximate area of each land use. The third step was for basins that could not be measured with a scale. For this step, visual judgement was used to determine the break down of different land uses in percentage.

C. SANITARY FLOW RATES

Table 1 identifies average unit flow rates for each land use. Most of these flow rates were extracted from the City's *Design and Procedures Manual*, Section 9.1.2, "Infill and ESD". Two land uses in the design manual, Hospitals and Colleges, were based on average monthly water use. Since average water use is not readily available, alternative methods were used for estimating the sanitary flows from these land uses. The attendance of the colleges and secondary schools were combined to estimate their sanitary flows. For hospitals, a method was used from Metcalf & Eddy's *Wastewater Engineering Treatment/ Disposal/ Reuse* second edition, table 2-9. The table gave the average wastewater flows from institutional sources, which determined a flow rate from the number of beds and employees at a hospital.

The flow rates in the City's *Design and Procedures Manual* are based on Equivalent Single Family Dwellings (ESDs). One ESD is equal to 400 gallons per day (gpd). The City's ESD average sanitary flow calculation method parallels the County of Sacramento's method. (Note: The County is continuously revising their ESD estimate based on observed flow

monitoring. The City's current estimated ESD flow rate is conservative, compared to the County's current ESD flow rate of 335 gpd.)

The average sanitary flows were calculated for each basin in the CSA. From the Land Use Assessment, the amount of each land use in a basin was known and flow was determined for that land use based on the flow rates in Table 1. The flows from each land use within a basin were then combined into the total sanitary flow for the basin. The flow rates were finally converted to cubic feet per second (cfs) for input into the CSA SWMM model.

D. IMPERVIOUS PERCENTS

Table 1 identifies impervious percentages for each type of land use. These percentages were taken from the draft *Sacramento City/County Drainage Manual*. Determining impervious percentages required the same three step as were used on determining land uses. The first land use, residential, was straightforward in that the impervious percentage was easily discerned based on density of the dwelling units. The various types of residential land uses are list in Table 1 with just an impervious percentage. Commercial and industrial land uses were moderately site specific because the land use could be either paved, gravel, or dirt. These land uses required judgement to determine the impervious percentage of the particular area. These types were listed in Table 1 with an impervious percentage and the word "varies". The listed impervious percentage was an average value which was either raised or lowered based on how the particular area compared to the average land use. Land uses such as schools and hospitals were highly site specific and are listed in Table 1 with only the word "varies". These land uses required an assessment based on the amount of pavement and roof tops to determine the impervious percentage.

An impervious percentage was determined for each type of land use within each basin. A combined weighted average value was calculated for each basin based on the percentage and imperviousness of each land use type within the basin.

E. INFILTRATION RATES

The RUNOFF block of SWMM simulates infiltration of rain into pervious areas based on Horton's infiltration equation. The equation requires three parameters: the initial abstraction rate, the final rate, and the decay rate (i.e. the rate of change between the initial and final rates). The final rate is the parameter which effects RUNOFF most significantly, since this parameter is the amount of rainfall that is "lost" in pervious areas over the entire duration of the storm.

The infiltration capacity (final rate) of the soil is related to its hydrologic classification. A review of the 1990 Soil Conservation Service (SCS) soil survey for Sacramento County revealed that the CSA is covered by a combination of all soil classifications, but primarily covered by "D" class soils and "Urban fill" material, which are identified in Table 2.

Each of the SCS soil classifications were each assessed a final rate of infiltration based on the Draft of the *Sacramento City/County Drainage Manual*, Table 2-7. Table 2-7 provides a very detailed list of infiltration rates based on land use and soil classification. Because of time constraints, one infiltration rate was used per soil classification, which are identified in Table 3. The rate coincides with "Residential: 6-8 units/acre" in Table 2-5 of the Drainage Manual which corresponds to an impervious area of 50%. "Residential: 6-8 units/acre" is the "best fit" to the average impervious area of the entire CSA.

Table 2-7 does not give a rate for class A type soils, stating that the County does not contain significant amounts of class A type soils. HDR provided a rate based on the same methodology used in developing the Drainage Manual.

The manual also does not cover "Urban fill" material. The SCS soil survey defines these soils as consisting of large areas covered by impervious surfaces such as roads, driveways, buildings, and parking

lots. These areas are mostly 90% impervious. The soil type under these surfaces are similar to the soils that occur near the "Urban fill" materials. For this analysis the "Urban fill" materials were combined with the class D type soils. This is a conservative estimate, but the impacts should be minimal since only pervious surfaces affect infiltration.

Infiltration final rates for each basin within the CSA were determined from the SCS soil type present in each basin. The SCS soil type maps were digitized and reproduced at 1"=600′ on clear mylar to create a clear overlay that would match the shed boundary maps and the 1"=600′ aerial photo used in the land use assessment. During the land use assessment, an assessment of the amount of each soil type present within each basin was made. A weighted average final rate of infiltration was then calculated for each basin.

F. EFFECTS OF REFINEMENTS

The refinements to the CSA SWMM parameters, final rate, imperviousness, and base sewage flow, affected the model's output. The parameters were analyzed individually and combined to determine the overall effect on the SWMM model. Table 4 indicates the previous existing parameters and the revised values. Table 5 indicates the parameters individual and combined overall effect on the CSA SWMM model.

The refinements to these SWMM input parameters made a significant impact on the CSA SWMM model. Table 5 shows that each parameter, especially the final rate of infiltration, increased the total inflow and flooding. These refinements were made using the best available information. The validity of these refinements will be determined during the calibration of the CSA SWMM model as part of Phase 3, task 3.2 of the *Sacramento Combined Sewer System Improvement and Implementation Plan*.

TABLE 1. UNIT IMPERVIOUS PERCENTAGES AND SANITARY FLOW RATES		
Land Use	**Sanitary Flow[1]**	**Impervious %[2]**
Commercial and Offices	Commercial Curve, Plate 9-1 of the *Design Manual*	95% Varies
Industrial	Industrial Curve, Plate 9-1 of the *Design Manual*	70% Varies
Multi-Residential (Condominiums & Apartments)	Multi-Residential Curve, Plate 9-1 of the *Design Manual*	70% Varies
Residential 3-4 DU/acre	400 gal/Dwelling Unit(DU)/day	30%
Residential 4-6 DU/acre	400 gal/Dwelling Unit(DU)/day	40%
Residential 6-8 DU/acre	400 gal/Dwelling Unit(DU)/day	50%
Residential 8-10 DU/acre	400 gal/Dwelling Unit(DU)/day	60%
Freeway & Parking Lots	N.A.	95%
Parks, Cemeteries & Open Space	N.A.	5% Varies
Primary School Secondary School and Colleges	560 gal/day per 100 average daily attendance. 720 gal/day per 100 average daily attendance	Varies
Hospitals	approx. 172 gal/bed/day & 11 gal/employee/day[3]	Varies

1) From the City of Sacramento's *Design and Procedures Manual*
2) From the *City/County Drainage Manual*
3) From Metcalf & Eddy's *Wastewater Engineering: Treatment/Disposal/Reuse* Second Edition

TABLE 2. SOIL GROUPS IN THE COMBINED SEWER AREA		
Map Symbol	**Soil Name**	**Hydrologic Group**
124	Columbia	B
130	Consumnes	C
141	Egbert	C
166	Kimball	D
168	Lang	B
186	Orthents	none
205	Rossmoor	B
209	Sailboat	C
219	San Joaquin	D
226	Tinnin	A
227	Urban Land	none
230	Valpac	C
240	Xerarents	D

SOURCE: Soil Conservation Service's *Soil Survey of Sacramento County California* 6/91

TABLE 3. FINAL INFILTRATION RATES FOR SOIL CLASSIFICATIONS IN THE COMBINED SEWER AREA				
Soil Classification	**A**	**B**	**C**	**D**
Infiltration Rates (in/hr)	0.35	0.19	0.11	0.08

SOURCE: Draft of the *Sacramento City/County Drainage Manual*

TABLE 4. RESULTS OF REFINED PARAMETERS			
Parameter	**Existing**	**Revised**	**% Increase**
Impervious Percentage	52.04%	53.69%	+3.2%
Total Infiltration	0.150"	0.102"	-32.0%
Base Sewage Flow	22.23 cfs	50.47 cfs	+127.0%

TABLE 5. EFFECTS OF REFINEMENTS ON THE CSA SWMM MODEL				
Parameter	**Inflow**	**% Inc.**	**Flooding**	**% Inc.**
Existing	29,771,680	n/a	21,812,370	n/a
Revised Sanitary Flows[1]	30,788,310	+3.41%	22,715,160	+4.14%
Revised Impervious %[1]	30,248,590	+1.60%	23,197,000	+6.35%
Revised Infiltration[1]	33,379,150	+12.12%	30,491,220	+39.79%
Revised[2]	34,773,760	+16.80%	32,821,370	+50.47%

Units for inflow and flooding are cubic feet (cf)

1) Includes existing conditions.
2) Includes existing conditions plus revised sanitary flows, revised impervious percents, and revised infiltration.

The impervious area in small urban areas can be estimated by direct measurements of land use from aerial photography. Table 2-5 lists suggested values for effective percent impervious based on land use. The table provides general land use descriptions and Sacramento County Planning Zoning Types. The Sacramento County Projected 2010 Land Use map is provided in the Appendix to this manual for estimation of future basin imperviousness.

Effective percent impervious values may be reduced by 20 percent for events with recurrence intervals that are less than 10 years.

TABLE 2-5

Land Use vs. Effective Percent Impervious

Land Use From Aerial Photography	Effective Percent Impervious	Sacramento Co. Land Use Description
Commercial/Highways/Parking	99	Core Area Commercial
	95	Commercial/Offices
Apartments/Offices/Trailers	90	Urban Transit Development
	80	High Density Residential
	75	Neighborhood Transit Dev.
Condominiums/Schools/Industry	70	Limited Industrial
Residential: 8-10 units/acre	60	Medium Density Residential
Residential: 6-8 units/acre	50	General Industrial
Residential: 4-6 units/acre	40	Low Density Residential
Residential: 3-4 units/acre	30	
Residential: 2-3 units/acre	25	
Residential: 1-2 units/acre	20	
Residential: .5-1 units/acre	15	
Residential: .2-.5 units/acre	10	Agriculture/Residential
Residential: <.2 units/acre	5	Recreational
Open Space/Grassland/Cropland	2	Agriculture/Urban Reserve
	2	Agriculture/Cropland
	2	Agriculture/General 80
	2	Agriculture/General 20
Open Space/Woodland	1	Urban Study Area
	1	Natural Preserve

2.3.5 Initial Losses

Initial losses are highly dependent on the soil condition. Initial losses for pervious areas can be quite high under natural conditions when the soil is dry. Dry soil in the fall infiltrates a significantly higher amount of precipitation than the moist soil in the late winter. The nature of the storm also affects the initial loss. Calibration modelling using HEC-1 in the Sacramento area has shown that higher initial losses were appropriate for the more frequent events. Recommended initial losses for Sacramento County are shown in Table 2-6.

TABLE 2-6

Initial Losses

Recurrence Interval	Loss (inches)
2	.40
5	.25
10	.20
25	.15
50	.12
100	.10
200	.08
500	.06

2.3.6 Uniform Losses

Pervious areas exhibit varying runoff potential based on the soil types and vegetation present. The U.S. Soil Conservation Service (SCS), has classified soils into four hydrologic categories based on infiltration rates after prolonged wetting. Type A soils exhibit low runoff potential while type D soils exhibit high runoff potential. These groups are further subdivided by soil-cover. The SCS hydrologic soil types are shown on the maps included in the Appendix of this manual. The soil maps provided also show type E and F soils, these areas are either urban land with significant imperviousness or a mixture of soil types due to fill material, the infiltration capacity of these soils was not determined by the SCS. Table 2-7 provides suggested infiltration rates by hydrologic soil-cover types.

TABLE 2-7

Infiltration Rates by Hydrologic Soil-Cover Groups
(inches per hour)

Cover	Soil Group B	C	D
Commercial/Highways/Parking	.14	.07	.04
Apartments/Offices/Trailers	.16	.08	.05
Condominiums/Schools/Industry	.17	.09	.06
Residential: 8-10 units/acre	.18	.10	.07
Residential: 6-8 units/acre	.19	.11	.075
Residential: 4-6 units/acre	.20	.12	.08
Residential: 3-4 units/acre	.21	.125	.085
Residential: 2-3 units/acre	.22	.13	.09
Residential: 1-2 units/acre	.225	.135	.095
Residential: .5-1 units/acre	.23	.14	.10
Residential: .2-.5 units/acre	.235	.145	.105
Residential: <.2 units/acre	.24	.15	.11
Bare/Rockland/Newly Graded	.02	.01	.01
Pasture/Mowed Grass	.16	.09	.07
Crops	.17	.10	.07
Orchards	.12	.07	.05
Natural Grassland	.16	.09	.07
Dense Oak/Pine Woodland/Vines	.25	.16	.12
Open Oak/Pine Woodland/Grass	.18	.11	.08
Chaparral/Shrubs/Weeds	.20	.13	.10

* Sacramento County does not contain significant areas of Type "A" soils.

6. **Industrial** - Every attempt should be made to base flows on specific, known industrial development. In the absence of specific knowledge of development, flow shall be determined from the curves on Plate 10 shown in the Appendix to this Section.

9.1.2 Infill and ESD

For those areas where the sanitary sewer system is existing, the following categories, with Equivalent Single Family dwelling units (ESD), should be used for computing average flow (1 ESD = 400 gallons/day).

Category	Equivalent ESD
Auto Dealerships	0.2 per 1000 sq. ft. of gross floor area
Bakeries	0.5 per 1000 sq. ft. of gross floor area
Banks/Financial Institutions	0.3 per 1000 sq. ft. of gross floor area
Barber/Beauty Shops	0.1 for each barber or beautician chair
Bars	0.7 per 1000 sq. ft. of gross floor area
Bowling Alleys (including eating facilities)	0.4 for each bowling lane
Car Washes	
a. Automatic	1.0 per 12,000 gallons water used each month
b. Self-service	0.7 for each washing stall
Condominiums & duplex units	0.75 per unit
Dry Cleaners	1.7 per 1000 sq. ft of gross floor area
Fire Stations	1.0 per station
Garages	0.1 for each service bay
Halls, Lodges & Auditoriums	0.3 per 1000 sq. ft. of gross floor area
Health Studios/Gymnasiums	0.3 per 1000 sq. ft. of gross floor area

Category	Equivalent ESD
Hospitals	1.0 per 12,000 gallons water used each month
Hotels and Motels	0.3 for each sleeping room
Laundries	
a. Self-service	0.5 for each laundry machine
b. Commercial/Industrial	1.0 per 12,000 gallons water used each month
Markets	
a. W/ Garbage Disposal	0.6 per 1000 sq. ft. of gross floor area
b. W/out Garbage Disposal	0.2 per 1000 sq. ft. of gross floor area
Medical/Dental Offices (including eating facilities)	0.4 per 1000 sq. ft. of gross floor area
Mortuaries	0.8 per each slumber room
Office Buildings (including eating facilities)	0.2 per 1000 sq. ft. of gross floor area
Parks	1.0 per comfort station
Places of Worship (including residences)	0.2 per 1000 sq. ft. of gross floor area
Public Agencies (including eating facilities)	0.6 per 1000 sq. ft. of gross floor area
Rest and Convalescent Homes, Boarding Houses, Fraternities, Sororities, Convents, Dormitories, etc.	0.3 for each bed
Restaurants	
a. Dine-in	2.0 per 1000 sq. ft. of gross floor area

Category	Equivalent ESD
b. Take-out	1.7 per 1000 sq. ft. of gross floor area
c. Dine-in & Take-Out	1.9 per 1000 sq. ft. of gross floor area
Retail Stores	0.2 per 1000 sq. ft. of gross floor area
Schools	
a. Primary	1.4 per 100 average daily attendance
b. Secondary	1.8 per 100 average daily attendance
c. College & Universities	1.0 per 12,000 gallons of water used each month
Service Stations	0.1 for each gas pump
Theaters	0.3 per 100 seats
Used Car Lots	0.2 per 10 fixture units
Warehouses	0.1 per 1000 sq. ft. of gross floor area
Non-defined Commercial	1.0 per 12,000 gallons of water used each month

9.2 DESIGN FLOW

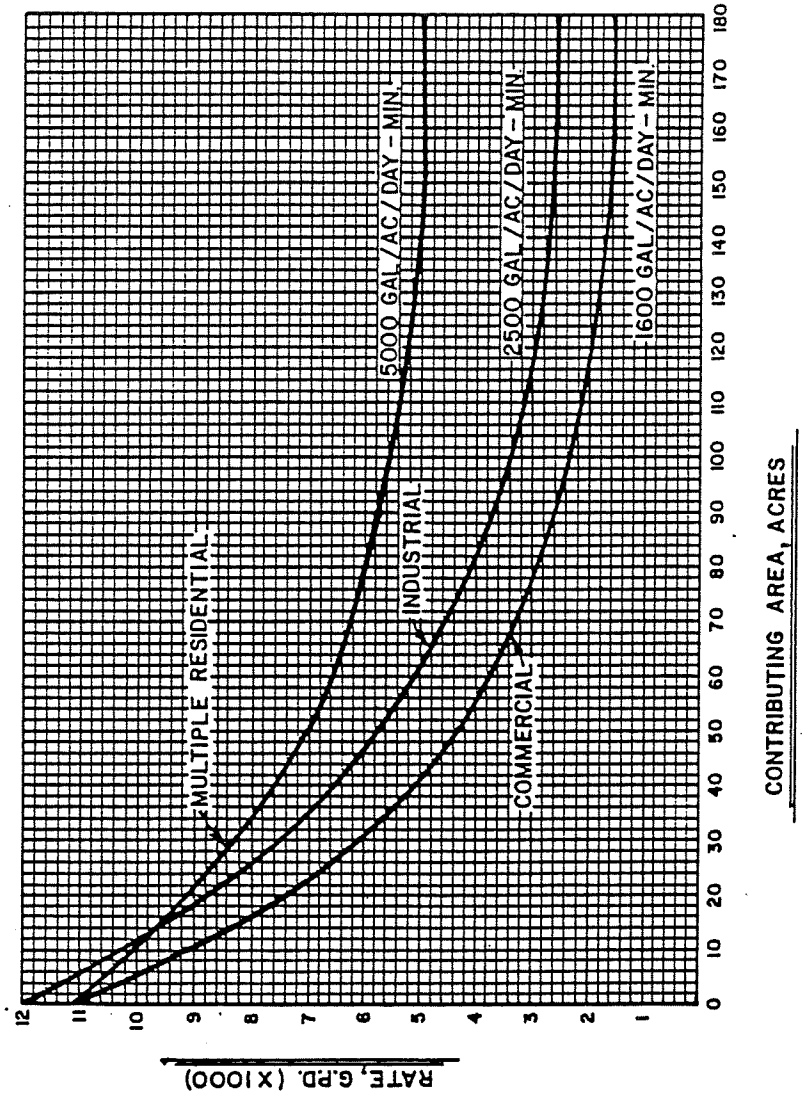
Average flow, as determined above, shall be multiplied by the peaking factor obtained from the curve on Plate 11 shown in the Appendix to this Section. The average ground water infiltration shall be 500 gallons per day per inch diameter per mile and added to this value to obtain the design flow. Areas deemed by the Engineer to have high ground water may require a larger value of infiltration assigned which will be determined by the Engineer.

9.3 PIPE SLOPE, VELOCITY AND SIZE

9.3.1 Laterals

Minimum size of laterals which serve single family shall be 4" diameter and duplex development shall be six inches (6") in diameter. Schools,

Average Sewer Flow (by area) Curves



CITY OF SACRAMENTO

ESTIMATED AVERAGE SEWAGE FLOW
COMMERCIAL, INDUSTRIAL, AND
MULTIPLE RESIDENTIAL ZONED AREAS

Plate 9-1
September 1, 1990

APPENDIX L

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM
IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 6

UNIT COST REFINEMENTS

Greg Jones
HDR Engineering, Inc.

Brierley & Lyman, Inc.

August 13, 1992

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM

IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 6

UNIT COST REFINEMENTS

Prepared by

Greg Jones
HDR Engineering, Inc.

Brierley & Lyman, Inc.

July 1, 1992

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SECTION 1
INTRODUCTION

Cost estimates for alternative improvement plans for the Sacramento Combined Wastewater Collection and Treatment System (CWCS) were prepared during the Phase 2 study. These costs were based on unit costs for various facilities or work items included in an improvement plan. As used in this study, a "unit cost" was considered to be the construction cost of a unit of work. When a unit cost is multiplied by the quantity of units needed to complete a work item, a total construction cost for that work item is derived. As an example of a unit cost and its use, a unit cost for gravity sewer installation would be the construction cost per foot of new sewer in place. The quantity of units for a particular work item would be the total length of new sewer to be installed. The estimate of construction cost would be the unit cost multiplied by the length of new sewer.

This approach, the analyses used to generate the unit costs and other cost estimating procedures are described in the *Phase 2 Detailed Technical Report* (dated July 1, 1991). Additional documentation is provided in Appendix H and Appendix I of that report.

As a part of the Phase 3 study effort, the unit costs developed during Phase 2 were reviewed and refined. This memorandum describes this review process and the revisions to the unit costs used in the Phase 2 study. The costs affected by this effort included the following:

- Unit costs for new sewer and storm drain construction. These were revised to reflect additional information obtained from local sewer construction contractors and from Brierley and Lyman, Inc., geotechnical engineers. The revisions include changes in assumed trench shoring, construction dewatering, new manholes and junction boxes, and costs associated with maintaining existing utilities within the vicinity of the work.

- The cost of constructing new sanitary sewer service connections. These were used to evaluate the sewer separation alternative plan being considered during the Phase 3 study. They were also used to evaluate repair and improvement costs for alternative plans in which sewers remained combined.

- The cost of constructing new storm drain drop inlets. These were used in the overall collection system upgrade cost estimates documented in the Phase 3 memorandum entitled *Combined Sewer Collection System Hydraulic Evaluation Improvements and Costs for the 2-, 5-, 10- and 25-Year Storms.*

- Pumping station costs. These were revised to reflect current price indices and have been compared to construction costs for recently bid and/or constructed pumping station projects in northern California.

- Storage reservoir costs. These were compared with the cost of the Pioneer Reservoir, updated to current dollars with the ENR Construction Cost Index.

- Overflow weir junction structures and tunnel drop shaft costs. Estimates of construction costs were prepared for side overflow weir junction structures and tunnel drop shafts.

In addition to these costs, efforts were made to assess the operation and maintenance costs associated with combined sewer storage reservoirs and tunnel systems. A telephone survey was conducted to collect this information.

The cost information presented in this memorandum was used in preparing total project costs for the Phase 3 study. This memorandum supplements the unit cost information prepared and presented in the Phase 2 technical memorandum entitled *Unit Construction Costs for Specific Alternative Projects.* It is important to note that all unit costs and non-construction cost factors used in this study are planning level values. It is anticipated that all project cost estimates will be refined during any subsequent predesign efforts.

SECTION 2
UNIT COSTS FOR NEW SEWER AND STORM DRAIN CONSTRUCTION

A. INTRODUCTION

Each of the alternative plans to be considered for upgrading the combined sewer system include the construction of new sewers. These new sewers are predominantly gravity sewers, although in a few instances, new force mains would be required.

For the purposes of this study, gravity sewers included two general components: new "collection" system sewers, generally 16 inches in diameter and smaller, and new "laterals, mains, and interceptors" for sewers over 16 inches in diameter. The 16-inch sewer diameter chosen to differentiate between collection system and lateral/main piping based on pipe diameter, was based on modeling convenience. The objective of identifying these two classes was to separate the existing sewers included in the SSWMM91 setup from those that would be evaluated using representative areas or basins.

New gravity sewers were used to either replace existing sewers, parallel existing sewers, or comprise a new network of consolidation conduits conveying excess flows diverted from the existing combined system to new flood or CSO control infrastructure. New force main sewers were used to convey pumped wastewater to either treatment, or storage facilities, or to the Sacramento River.

For the Phase 2 study, estimates of unit costs for new sewers were presented in terms of cost per lineal foot. These costs were developed with limited information from local contractors. The components of cost that warranted further investigation included those associated with shoring and bracing techniques, demolition and removal of existing sewers, maintaining existing utility service while new sewer work is in progress, and construction dewatering.

This section reviews the unit costs developed for new sewers. Revisions to the Phase 2 study costs reflect the opinions of local pipeline contractors with recent project experience in the City and County of Sacramento and surrounding

area. Revised construction dewatering costs were prepared by Brierley and Lyman, Inc., geotechnical engineers, from a preliminary geotechnical evaluation of the local soils and groundwater conditions using available reports.

The costs presented in this section and in the remainder of this memorandum are planning-level costs based on the assumptions stated, without the benefit of pre-design evaluation efforts. Actual construction costs are likely to differ from these estimated costs, depending upon actual construction locations within the study area, seasonal groundwater elevation, and restrictions placed on the progress of a particular project related to vehicle traffic, environmental concerns, right-of-way, or other factors not yet identified.

B. ASSUMPTIONS FOR NEW PIPE CONSTRUCTION

B.1. Phase 2 Assumptions

For purposes of establishing a unit construction cost per linear foot of new sewer, the Phase 2 study used the following assumptions.

- New sewers up to 144-inch diameter would be installed with conventional trenching and backfilling techniques. Larger conduits would be installed as tunnels.

- Pipeline construction costs include maintaining utility service to the residences and businesses within the study area throughout the course of the construction project. Limited service interruptions will be necessary, however, during reconnection of service laterals.

- New sewer pipe would be reinforced concrete pipe meeting ASTM C76, Class III, with rubber gasketed joints. The unit costs assumed do not include provision for cast-in-place PVC lining systems. PVC lining was not included because neither the City nor the County currently include this in their standard specifications.

- Traffic control restrictions would be imposed on the contractor, in order to limit periods of blocked or restricted roadway travel.

- Dewatering efforts would include sedimentation basins or treatment for sediment removal prior to discharge to the storm sewer system. Local landscaping and trees would be protected during dewatering efforts.

- New pipelines sized 84 inches and larger would include curb and gutter removal and replacement.

- The installed depth of the new sewers would vary according to nominal diameter, with larger sewers laid deeper than smaller sewers.

- New manholes and junction boxes would be provided at a spacing interval of 300 feet.

B.2. Phase 3 Unit Costs

The tables at the end of this section present the revised Phase 3 unit cost data for new gravity and force main sewers. The Phase 3 assumptions used and their comparison to the Phase 2 assumptions are discussed below.

B.3. Unit Cost Differences Between Phase 2 and Phase 3

The following summarizes the assumptions used in estimating the unit costs presented in Tables 2-1 and 2-2. The differences between the Phase 2 and Phase 3 unit costs are discussed according to the cost components presented in Tables 2-1 and Table 2-2.

B.3.a. Shoring, Cribbing and Bracing--

Unit costs for trench excavation reinforcement in the Phase 2 report were broken down into two categories: "Sheeting" and "Cribbing and Walers". This division was intended to make use of the readily available costs for both items of work. For Phase 2, the trench excavation reinforcement cost item represented a mixture of reinforcement techniques including using a trench box, wood sheeting, and soldier

piles with wood or steel sheeting.

For Phase 3, trench reinforcement costs were represented as a single item: "Shoring, Cribbing, and Bracing". The trench reinforcement system assumed for the Phase 3 unit cost spreadsheet was a trench box for excavations up to a maximum of 15 feet deep. For deeper trenches, the 15-foot high trench box would be supplemented by either excavating the trench above the shield back at a 1:1 slope or, where the work area is limited, steel sheeting would be used above the shield.

The costs for trench reinforcement for Phase 3 are generally lower than for the Phase 2 study, reflecting the limited use of steel sheeting and the economy of using a trench shield system where possible.

B.3.b. Manholes, Junction Boxes--

Manhole and utility relocation costs were separated for the revised Phase 3 unit costs. For Phase 2, unit costs for manholes were based on an average 300-foot manhole spacing interval. This cost was lumped in with an estimate for relocating existing utilities.

For Phase 3, manhole costs were based on the following assumptions:

1) Manhole spacing for sewers ranging from 12 inches to 72 inches in diameter at 400 feet on center, and spacing for larger sewers at 1,000 feet. The 400-foot spacing interval is a common limitation for the hydraulic jetting equipment used for flushing out smaller gravity sewer systems. For larger sewers (72 inches and larger), manhole spacing can be increased because maintenance personnel can physically walk inside of the pipe to clean out debris and flush out sediment with hand held hoses.

2) For sewers up to 21 inches nominal diameter, 48-inch diameter manholes were used. For sewer sizes in the range of 21 to 42 inches, a 60-inch diameter manhole was used. For 48-inch and larger sewers, a saddle manhole configuration with a top-mounted 48-inch manhole for personnel access was used.

3) Unit costs for 48-inch manholes were $180/vertical-foot, plus $500; 60-inch manholes were $210/vertical-foot plus $800, and saddle manholes were a combination of the cost for a 48-inch access manhole and a cost for the cast-in-place work for the lower part of the structure. This configuration conforms with City design standards.

B.3.c. Maintain, Relocate Utilities--

For Phase 3, a separate cost item was identified for "Maintain, Relocate Utilities". This cost was separated from "Manhole/Junction Structure" costs. The rationale for this separation was that costs associated with relocation of existing utilities were not a function of any of the variables listed in the Phase 2 unit cost spreadsheet.

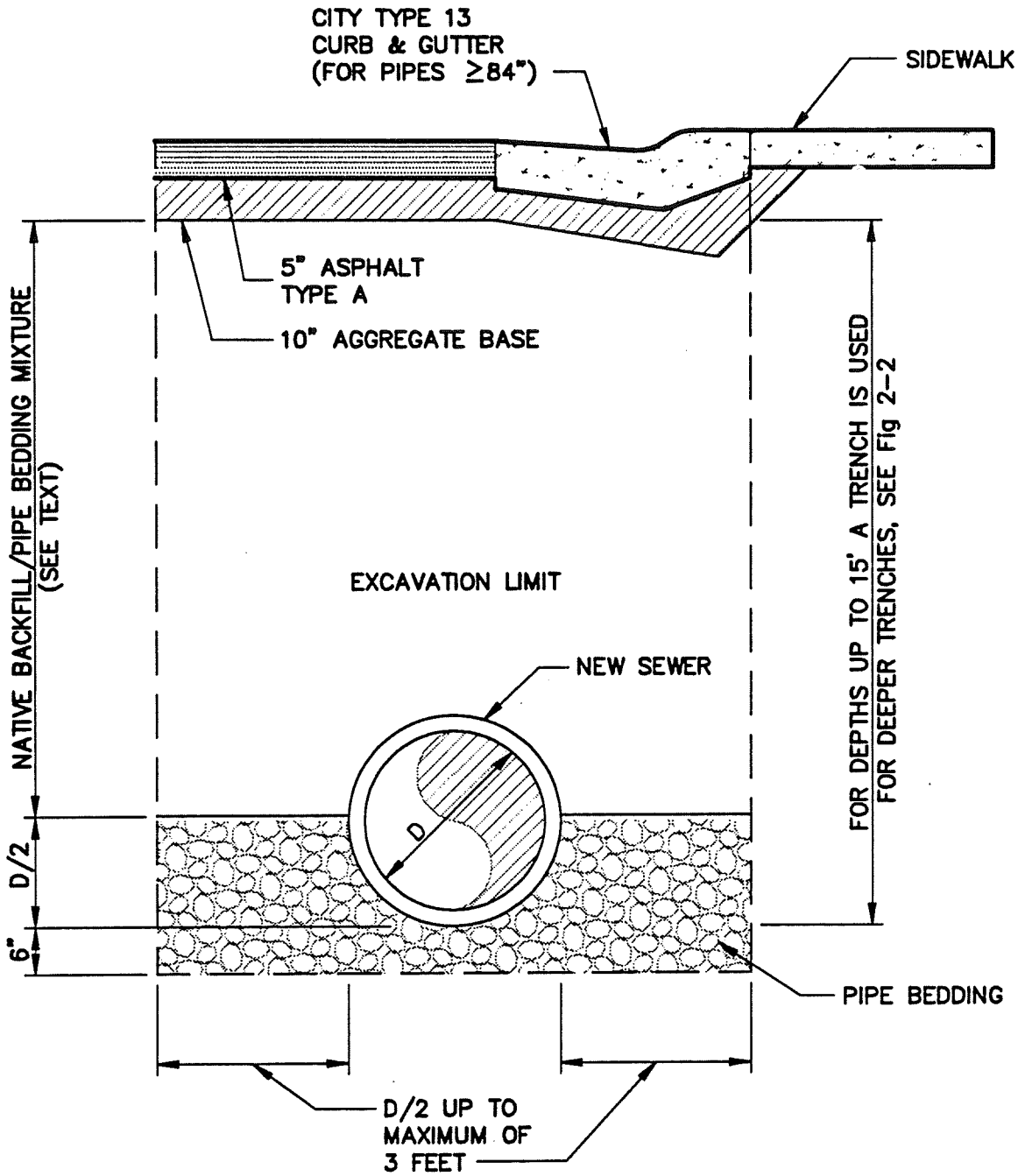
This component of pipeline construction cost is a function of the proximity and interference of existing utilities within the work area. To assign an average cost per unit length of sewer for this item of work is much more speculative than estimating the cost of new sewer manholes/junction structures. Consequently, the two cost items have been separated.

B.3.d. Excavation--

Excavation quantities for Phase 2 were based on a different trench configuration than Phase 3. The excavated trench configuration for Phase 2 was a vertical wall trench with shoring and reinforcement as required to make the work area safe.

The trench configuration assumed for Phase 3 for trench depths between 0 and 15 feet is illustrated in Figure 2-1. It assumed the trench reinforcement system to be a trench box, also referred to as a trench shield. The actual depth limit where trench boxes are useful depends upon the character of the soil. Where soils have been previously disturbed by buried utility construction, or where the soils are low in cohesion and tend to slough, the friction on the box becomes too great to move it, and supplemental reinforcement or excavation is required.

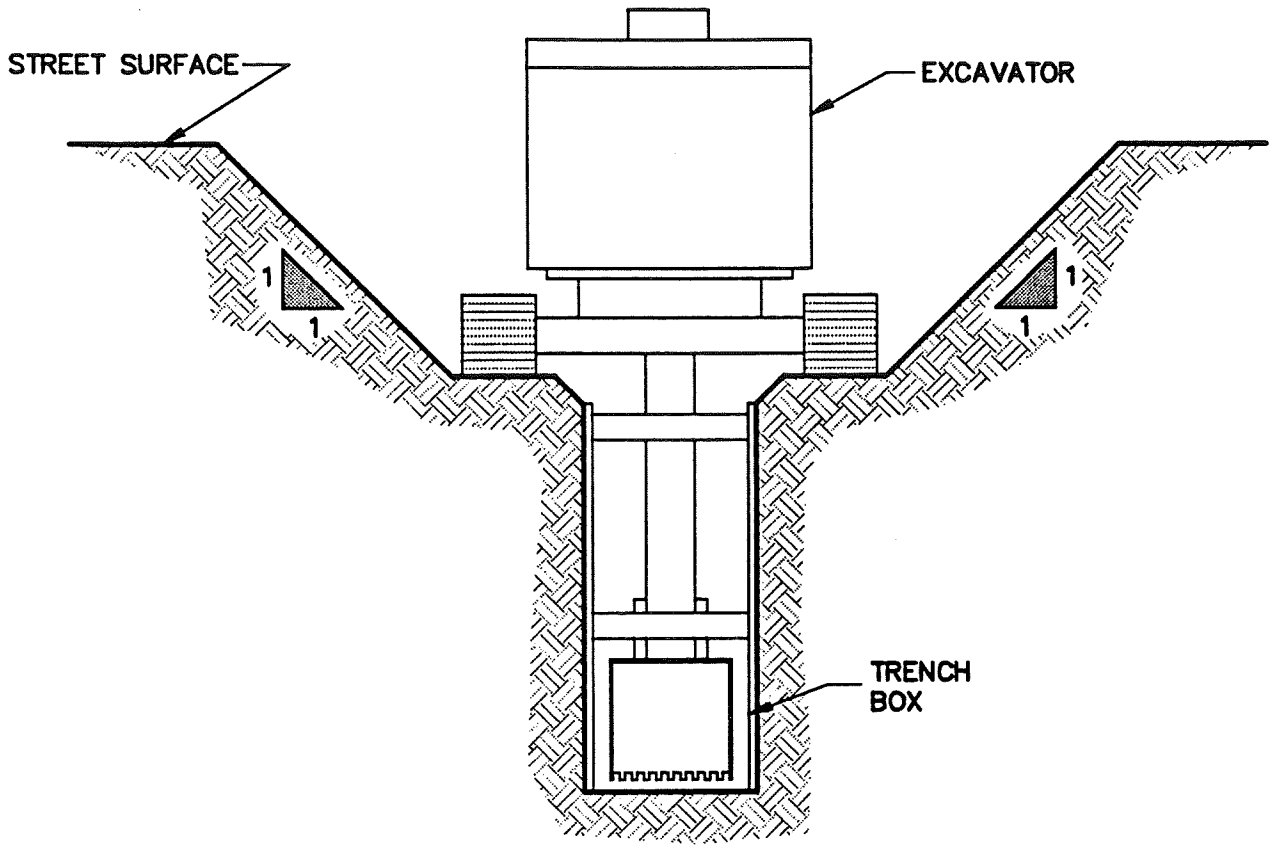
For deeper trenches, the assumed excavation configuration is illustrated in Figure 2-2. Two options were considered: one was to slope the upper portion of the trench back at a 1:1 slope above a 15-foot high trench shield. The other was

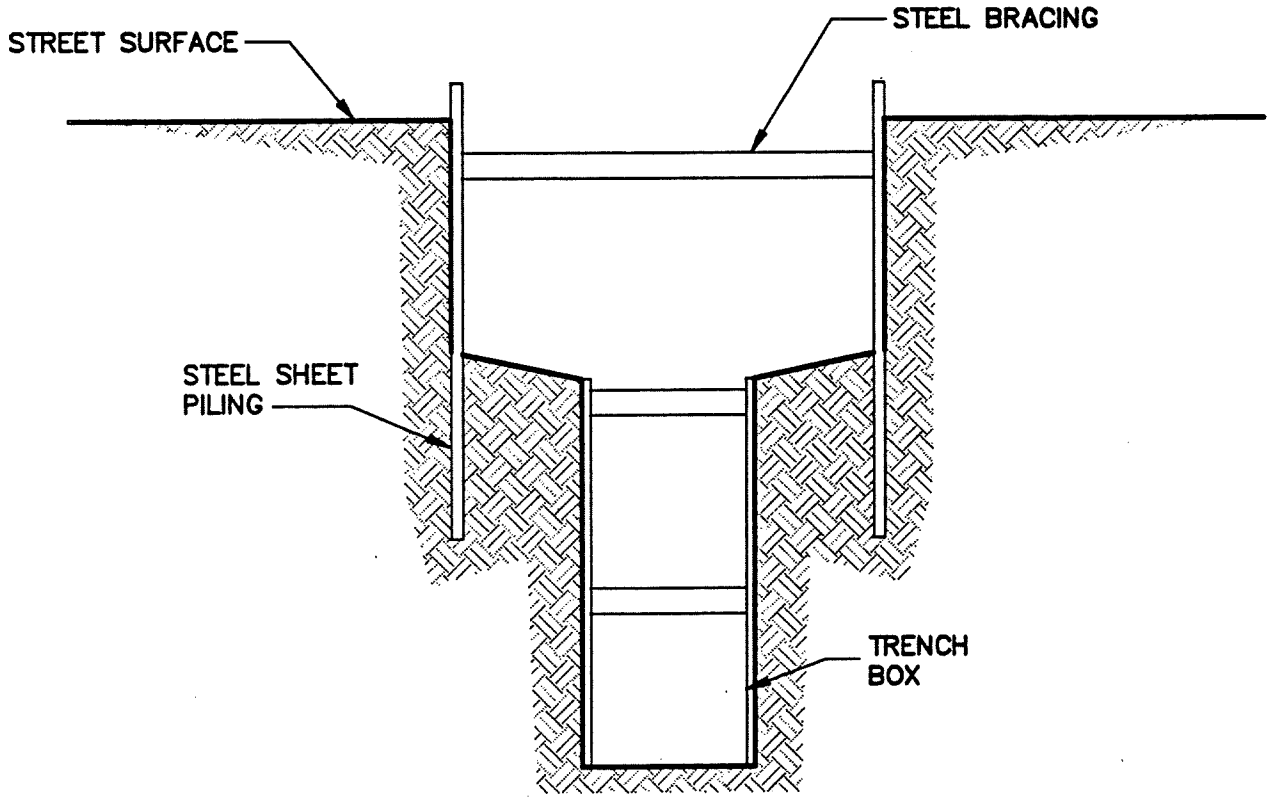


TYPICAL TRENCH DETAIL

HDR

Figure 2-1





METHODS OF DEEP TRENCH REINFORCEMENT

HDR

Figure 2–2

to use steel sheeting above the shield. These revised trench configurations were based on discussions with local contractors on current construction practices.

Existing sewer removal, where required, was included in the revised excavation costs. In some cases, removal of existing sewers has been difficult for contractors for two reasons. One is that the existing pipe may not come out of the excavation easily with conventional backhoe excavators, and consequently, there is additional cost involved. The other reason is that soils excavated from the trench are normally removed and stored off the construction site while pipe laying is in progress. This native excavated material, if determined to be suitable, would normally be returned to the site and used as backfill for the excavated trench. However, experience has shown that there is a good chance that the existing pipe will crumble into rubble and become mixed into the excavated material, especially the older brick sewers. This not only makes the excavated material unsuitable for trench backfill but also requires that it be classified as a contaminated soil waste requiring special disposal procedures.

The Phase 2 unit costs used for excavation were in the range of $15 to $20 per cubic yard (cy). Based on the experience of local contractors, these estimates were high. More accurate costs (in two components) are on the order of a base cost of $3/cy, plus trucking for excavated material which is suitable as backfill. Total unit costs for excavation were therefore revised to $13/cy which is an average of $6/cy for material which can be used as backfill, and $20/cy for material requiring special disposal procedures as a contaminated soil.

B.3.e. Dewatering--

The dewatering costs for Phase 2 ranged from $5 to $36 per lineal foot of new sewer pipe. This was based on one contractor's experience with pipe installed at shallower depths. The Phase 3 costs for dewatering range from $50 to $75 per lineal foot. These costs are the result of a geotechnical evaluation of construction dewatering requirements and costs by Brierley and Lyman Inc. Their evaluation and recommendations include wells drilled 100 feet on-center, in a line parallel to proposed sewer alignments.

Each well would deliver 75 gpm and be operational for a total of one month. No well points would be used in the trench, as was assumed for Phase 2. Water pumped from the wells would be treated for sediment removal prior to placement into the combined sewer system. The water table was assumed to be an average of 15 feet below the surface. Local contractor experience indicates that water table elevations are higher nearer the river, and lower further away.

For the combined sewer service area, the water table is more influenced by river stage than proximity to the rivers, except for the corridor within roughly 3 blocks of the American and Sacramento Rivers. Given this information, an average water table elevation for the entire study area was used rather than subdividing the area into sectors, each with a different dewatering cost. Shallow excavations no deeper than 15 feet did not include dewatering costs. Details of the analysis of dewatering construction costs provided by Brierley and Lyman, Inc. are provided in Appendix A to this memorandum.

B.3.f. C76 Pipe--

Phase 2 pipe prices were quoted directly from a local concrete pipe manufacturer for ASTM C76 Class III pipe. For the Phase 3 report, Class IV pipe was considered for invert depths over 19 feet. Class IV pipe may also be required when there is limited cover over new sewers. However, for the smallest pipe sizes the assumed depth of installation has sufficient cover to allow use of Class III pipe. Pipe costs were increased by 15 percent for those pipes over 30 inches in diameter to account for the Class IV designation.

B.3.g. Pipe Bedding--

The quantity of pipe bedding is different for Phase 2 and Phase 3. The trench configuration for Phase 2 assumed a total pipe bedding depth consisting of over-excavation equivalent to 33 percent of the pipe diameter, and backfill with bedding up over the pipe invert to a level equal to 15 percent of the pipe diameter. By comparison, the trench configuration used for Phase 3 included over-excavation of only 6 inches and bedding brought up to the pipe spring line. This bedding configuration is representative of Type B granular foundation bedding and is considered compatible with local area soil conditions within the study area.

Unit costs for pipe bedding were revised from a Phase 2 estimate of $12/cy in place to $15/cy for Phase 3. This revision is based on the recommendations of local contractors.

B.3.h. Pipe Laying--

The costs for pipe laying presented for Phase 2 were based on updating estimates of project costs developed by a consultant to the City for the Pioneer Interceptor project in 1977 using the ENR Construction Cost Index. These costs were revised for Phase 3 based on the recommendations of local sewer contractors.

B.3.i. Leak and Miscellaneous Testing--

For the Phase 2 effort, estimates for leak detection were based on hydrostatic testing, whereby the pipe would be filled with water and the standing water level checked after a period of time for a change in elevation related to exfiltration. The Phase 3 costs for this item were revised downward at the suggestion of local sewer contractors who said individual pipe joint testing was considered an acceptable substitute to hydrostatic testing by local City and County construction contract administrators.

B.3.j. Backfill--

Estimates of backfill quantities changed from Phase 2 to Phase 3 due to the change in assumed trench configuration.

Unit costs for backfill for Phase 2 were taken to be $1.50/cy on the assumption that native material could be used as fill. For Phase 3, unit backfill costs are $10/cy (which is an average of $5/cy for suitable native fill and $15/cy for placing imported fill from a distant source). Phase 3 backfill costs assumed importation of select material to replace rubble and contaminated soil resulting from the removal of existing sewers.

B.3.k. Asphalt Base--

A new unit cost item was created for roadway asphalt base for Phase 3. Phase 2 neglected to account for the replacement of the road base beneath the asphalt paving. For Phase 3, a 10-inch thick aggregate base beneath a 5-inch thick Type A asphalt surfacing was used. This structural section complies with State and

local road design requirements for urban roads and traffic loads.

B.3.l. Asphalt--

Quantities of asphalt varied with trench configuration between the Phase 2 and Phase 3 studies. Both assumed a 5-inch paving thickness, but the Phase 3 trench required wider trenches for deeper excavations where the upper portion of the trench was sloped back at a 1:1 slope.

Unit costs for asphalt used in Phase 2 were $66/ton. For Phase 3, a graduated unit cost was used to account for higher in place costs for small quantities of asphalt. The Phase 3 costs ranged from $42 to $116/ton.

B.3.m. Curb and Gutter, Demobilization, Site Clean-Up--

Unit construction costs for "Curb and gutter" were revised from Phase 2 to Phase 3 based on discussions with concrete contractors. Phase 2 assumed a per lineal foot cost of $33/lf and Phase 3 used $18/lf.

Similarly, unit costs for de-mobilization and site cleanup were revised downward from Phase 2 to Phase 3 after review and discussion with local contractors.

C. ESTIMATES OF UNIT COSTS FOR NEW SEWERS

Table 2-1 presents the unit pipe cost used in the Phase 3 study, and is a revision of the spreadsheet used to develop the unit costs for sewers for the Phase 2 report. These costs reflect the sewer installation configurations shown in Figure 2-1 and 2-2 and were used in the preparation of estimates of construction cost for the alternative improvement plans considered in Phase 3. Table 2-2 is similar to Table 2-1 except it applies to new force main sewers.

TABLE 2-1
UNIT COSTS FOR INSTALLED NEW GRAVITY SEWER PIPE

DIAMETER (inches)	WALL THICKNESS (inches)	PIPE WEIGHT (lbs/ft)	AVERAGE INVERT ELEVATION (MSL)	SEWER INVERT DEPTH (FT)	SHORING/ CRIBBING/ BRACING ($/lf)	MANHOLES/ JUNCTION BOXES ($/LF)	MAINTAIN/ RELOCATE UTILITIES ($/lf)	EXCAVATION (cy/lf)	EXCAVATION ($/lf)
8	2.50	37.82	16.0	6	16.68	2.70	9.00	0.32	4.17
10	2.75	51.16	14.0	8	18.37	3.60	13.00	0.52	6.82
12	3.00	66.23	12.0	10	22.00	4.50	15.00	0.78	10.11
15	3.00	80.95	10.0	12	23.57	5.40	15.68	1.16	15.05
18	3.00	95.67	8.0	14	25.13	6.30	16.36	1.61	20.94
21	3.00	110.39	7.0	15	25.91	7.88	17.05	2.01	26.12
24	3.00	125.11	6.0	16	20.46	8.40	17.73	2.46	32.02
27	3.25	152.14	5.0	17	27.96	8.93	18.41	2.99	38.88
30	3.50	181.74	4.0	18	35.46	9.45	19.09	3.59	46.70
33	3.75	213.88	3.0	19	42.96	9.98	19.77	4.27	55.49
36	4.00	248.58	2.0	20	50.46	10.50	20.46	5.02	65.24
39	4.25	285.84	1.0	21	57.96	11.03	21.14	5.84	75.95
42	4.50	325.65	0.0	22	65.46	6.45	21.82	6.74	87.63
45	4.75	368.02	−1.0	23	72.96	7.11	22.50	7.71	100.27
48	5.00	412.94	−2.0	24	80.46	7.81	23.18	8.76	113.87
51	5.25	460.42	−2.5	25	84.21	8.55	23.86	9.54	124.04
54	5.50	510.45	−3.0	25	87.96	9.33	24.55	10.35	134.57
57	5.75	563.04	−3.5	26	91.71	10.16	25.23	11.19	145.47
60	6.00	618.19	−4.0	26	95.46	11.05	25.91	12.06	156.72
63	6.25	675.89	−4.5	27	99.21	11.99	26.59	12.95	168.34
66	6.50	736.14	−5.0	27	102.96	12.98	27.27	13.87	180.31
69	6.75	798.95	−5.5	28	106.71	14.03	27.96	14.82	192.65
72	7.00	864.32	−6.0	28	110.46	6.05	28.64	15.80	205.35
78	7.50	1002.71	−6.5	29	114.21	7.00	30.00	16.80	218.41
84	8.00	1151.33	−7.0	29	117.96	8.04	31.37	17.83	231.83
90	8.00	1229.83	−7.5	30	121.71	9.18	32.73	18.89	245.62
96	8.50	1393.58	−8.0	30	125.46	10.43	34.09	19.98	259.76
102	8.50	1476.99	−8.5	31	129.21	11.78	35.46	21.10	274.26
108	9.00	1655.86	−9.0	31	132.96	13.24	36.82	22.24	289.13
114	9.00	1744.17	−9.5	32	136.71	14.81	38.18	23.41	304.36
120	10.00	2044.27	−10.0	32	140.46	16.51	39.55	24.61	319.94
132	10.00	2240.52	−10.5	33	144.21	20.25	42.28	26.45	343.84
144	10.50	2562.90	−11.0	33	147.96	24.50	45.00	28.33	368.33

TABLE 2-1
UNIT COSTS FOR INSTALLED NEW GRAVITY SEWER PIPE

DIAMETER (inches)	DEWATERING ($/lf)	C76 PIPE FOB FACTORY ($/lf)	PIPE SHIPPING ($/lf)	PIPE BEDDING (cy/lf)	PIPE BEDDING ($/lf)	PIPE LAYING ($/lf)	LEAK, MISC. TESTING ($/lf)	BACKFILL (cy/lf)	BACKFILL ($/lf)
8	0.00	3.25	0.38	0.03	0.52	10.70	1.94	0.27	2.68
10	0.00	4.45	0.51	0.05	0.70	11.35	2.05	0.45	4.52
12	0.00	5.75	0.66	0.06	0.89	12.00	2.16	0.68	6.81
15	0.00	8.63	0.81	0.08	1.22	12.98	2.33	1.02	10.21
18	0.00	11.50	0.96	0.11	1.59	13.95	2.49	1.43	14.28
21	0.00	15.00	1.10	0.13	2.01	14.93	2.66	1.77	17.73
24	50.00	19.00	1.25	0.16	2.46	15.91	2.82	1.95	19.51
27	51.47	23.00	1.52	0.20	2.96	16.89	2.99	2.35	23.45
30	52.94	28.75	1.82	0.23	3.50	17.86	3.15	2.81	28.07
33	54.41	33.35	2.14	0.27	4.08	18.84	3.32	3.34	33.37
36	55.88	43.70	2.49	0.31	4.70	19.82	3.48	3.93	39.34
39	57.35	50.95	2.86	0.36	5.37	20.80	3.65	4.60	45.99
42	58.82	58.08	3.26	0.41	6.08	21.77	3.81	5.33	53.31
45	60.29	63.48	3.68	0.46	6.83	22.75	3.98	6.13	61.31
48	61.76	68.77	4.13	0.51	7.62	23.73	4.14	7.00	69.99
51	62.50	77.40	4.60	0.56	8.46	24.71	4.31	7.62	76.19
54	63.23	85.96	5.10	0.62	9.33	25.68	4.47	8.26	82.61
57	63.97	97.18	5.63	0.68	10.25	26.66	4.64	8.92	89.24
60	64.70	109.14	6.18	0.75	11.22	27.64	4.80	9.61	96.09
63	65.44	119.03	6.76	0.81	12.22	28.62	4.97	10.31	103.15
66	66.17	129.61	7.36	0.88	13.26	29.59	5.13	11.04	110.42
69	66.91	140.88	7.99	0.96	14.35	30.57	5.30	11.79	117.90
72	67.64	152.09	8.64	1.03	15.48	31.55	5.46	12.56	125.60
78	68.38	178.54	10.03	1.12	16.83	33.50	5.79	13.25	132.47
84	69.11	204.99	11.51	1.21	18.20	35.46	6.12	13.95	139.45
90	69.85	297.56	12.30	1.31	19.61	37.41	6.45	14.65	146.55
96	70.58	317.40	13.94	1.40	21.04	39.37	6.78	15.38	153.75
102	71.32	374.33	14.77	1.50	22.51	41.32	7.11	16.11	161.07
108	72.05	405.26	16.56	1.60	24.00	43.28	7.44	16.85	168.50
114	72.79	436.20	17.44	1.70	25.53	45.23	7.77	17.60	176.05
120	73.52	467.13	20.44	1.81	27.08	47.19	8.10	18.37	183.70
132	74.26	529.00	22.41	2.02	30.28	51.10	8.76	19.32	193.15
144	74.99	590.87	25.63	2.24	33.60	55.01	9.42	20.24	202.40

TABLE 2-1
UNIT COSTS FOR INSTALLED NEW GRAVITY SEWER PIPE

DIAMETER (inches)	ASPHALT BASE (cy/lf)	ASPHALT BASE ($/lf)	ASPHALT (ton/lf)	ASPHALT ($/lf)	CURB/ GUTTER (ft/lf)	CURB/ GUTTER ($/lf)	ACCESS & TRAFFIC CONTROL ($/lf)	DEMOB AND SITE CLEANUP ($/lf)	TOTAL CONSTRUCTION COST, PHASE 3 ($/lf)	TOTAL CONSTRUCTION COST, PHASE 2 ($/lf)
8	0.04	0.79	0.04	7.00	0	0	3	3.30	66.11	N/A
10	0.05	0.99	0.05	7.00	0	0	3	3.50	79.85	N/A
12	0.06	1.18	0.06	7.00	0	0	3	3.70	94.77	144.32
15	0.08	1.48	0.07	7.00	0	0	3	4.00	111.35	167.01
18	0.09	1.78	0.09	7.00	0	0	3	4.30	129.59	181.06
21	0.11	2.07	0.10	7.00	0	0	3	4.60	147.05	197.15
24	0.15	2.96	0.15	9.00	0	0	3	4.90	209.41	209.07
27	0.20	3.85	0.19	9.00	0	0	3	5.20	237.49	226.55
30	0.25	4.73	0.24	9.00	0	0	3	5.50	269.03	241.05
33	0.29	5.62	0.28	10.00	0	0	3	5.80	302.12	260.17
36	0.34	6.51	0.33	10.00	0	0	3	6.10	341.67	342.11
39	0.38	7.40	0.37	10.00	0	0	3	6.40	379.83	383.48
42	0.43	8.28	0.42	12.00	0	0	3	6.70	416.47	407.44
45	0.48	9.17	0.46	12.00	0	0	3	7.00	456.33	435.30
48	0.52	10.06	0.51	13.00	0	0	5	7.30	500.82	460.47
51	0.55	10.65	0.54	13.00	0	0	5	7.60	535.07	484.15
54	0.58	11.24	0.57	14.00	0	0	5	7.90	570.94	508.50
57	0.61	11.84	0.60	14.00	0	0	5	8.20	609.17	539.88
60	0.65	12.43	0.63	15.00	0	0	5	8.50	649.83	567.13
63	0.68	13.02	0.66	15.00	0	0	5	8.80	688.11	592.80
66	0.71	13.61	0.69	16.00	0	0	10	9.10	733.78	629.30
69	0.74	14.20	0.72	17.00	0	0	10	9.40	775.84	622.12
72	0.77	14.79	0.75	18.00	0	0	10	9.70	809.46	694.11
78	0.80	15.39	0.78	20.00	0	0	10	10.30	870.84	749.19
84	0.83	15.98	0.81	21.00	1	18	10	10.90	949.92	842.29
90	0.86	16.57	0.84	23.00	1	18	10	11.50	1078.02	953.06
96	0.89	17.16	0.87	25.00	1	18	10	12.10	1134.86	1003.91
102	0.92	17.75	0.90	27.00	1	18	15	12.70	1233.58	1185.11
108	0.95	18.34	0.93	30.00	1	18	15	13.30	1303.88	1260.95
114	0.98	18.94	0.96	33.00	1	18	15	13.90	1373.90	1335.31
120	1.01	19.53	0.99	36.00	1	18	15	14.50	1446.65	1421.60
132	1.06	20.42	1.03	39.00	1	18	15	15.70	1567.63	1544.93
144	1.11	21.30	1.08	42.00	1	18	15	16.90	1690.91	1724.39

TABLE 2-2
UNIT COSTS FOR INSTALLED NEW FORCE MAIN SEWER PIPE

DIAMETER (inches)	WALL THICKNESS (inches)	PIPE WEIGHT (lbs/ft)	AVERAGE INVERT ELEVATION (MSL)	SEWER INVERT DEPTH (FT)	SHORING/ CRIBBING/ BRACING ($/lf)	MANHOLES/ JUNCTION BOXES ($/LF)	MAINTAIN/ RELOCATE UTILITIES ($/lf)	EXCAVATION (cy/lf)	EXCAVATION ($/lf)
12	3.00	66.23	17.0	5	22.00	2.25	15.00	0.41	5.30
15	3.00	80.95	16.8	5	22.23	2.36	15.68	0.53	6.92
18	3.00	95.67	16.5	6	22.45	2.48	16.36	0.67	8.67
21	3.00	110.39	16.3	6	22.68	3.02	17.05	0.81	10.53
24	3.00	125.11	16.0	6	22.91	3.15	17.73	0.96	12.52
27	3.25	152.14	15.8	6	23.14	3.28	18.41	1.13	14.63
30	3.50	181.74	15.5	7	23.36	3.41	19.09	1.30	16.85
33	3.75	213.88	15.3	7	23.59	3.54	19.77	1.48	19.20
36	4.00	248.58	15.0	7	23.82	3.68	20.46	1.67	21.67
39	4.25	285.84	14.8	7	24.05	3.81	21.14	1.87	24.25
42	4.50	325.65	14.5	8	24.27	5.91	21.82	2.07	26.96
45	4.75	368.02	14.3	8	24.50	6.54	22.50	2.29	29.79
48	5.00	412.94	14.0	8	24.73	7.21	23.18	2.52	32.74
51	5.25	460.42	13.8	8	24.95	7.94	23.86	2.75	35.81
54	5.50	510.45	13.5	9	25.18	8.71	24.55	3.00	39.00
57	5.75	563.04	13.3	9	25.41	9.53	25.23	3.25	42.31
60	6.00	618.19	13.0	9	25.64	10.41	25.91	3.52	45.74
63	6.25	675.89	12.8	9	25.86	11.34	26.59	3.79	49.29
66	6.50	736.14	12.5	10	26.09	12.32	27.27	4.07	52.96
69	6.75	798.95	12.3	10	26.32	13.36	27.96	4.37	56.75
72	7.00	864.32	12.0	10	26.55	5.78	28.64	4.67	60.67
78	7.50	1002.71	11.5	11	27.00	6.73	30.00	5.09	66.20
84	8.00	1151.33	11.0	11	27.45	7.77	31.37	5.54	71.98
90	8.00	1229.83	10.5	12	27.91	8.91	32.73	6.00	78.00
96	8.50	1393.58	10.0	12	28.36	10.16	34.09	6.48	84.26
102	8.50	1476.99	9.5	13	28.82	11.51	35.46	6.98	90.76
108	9.00	1655.86	9.0	13	29.27	12.97	36.82	7.50	97.50
114	9.00	1744.17	8.5	14	29.73	14.54	38.18	8.04	104.48
120	10.00	2044.27	8.0	14	30.18	16.24	39.55	8.59	111.70
132	10.00	2240.52	7.0	15	31.09	19.99	42.28	9.76	126.87
144	10.50	2562.90	6.0	16	32.00	24.25	45.00	11.00	143.00

TABLE 2-2
UNIT COSTS FOR INSTALLED NEW FORCE MAIN SEWER PIPE

DIAMETER (inches)	DEWATERING ($/lf)	C76 PIPE FOB FACTORY ($/lf)	PIPE SHIPPING ($/lf)	PIPE BEDDING (cy/lf)	PIPE BEDDING ($/lf)	PIPE LAYING ($/lf)	LEAK, MISC. TESTING ($/lf)	BACKFILL (cy/lf)	BACKFILL ($/lf)	ASPHALT BASE (cy/lf)
12	0.00	5.75	0.66	0.06	0.89	12.00	2.16	0.31	3.11	0.06
15	0.00	8.63	0.81	0.08	1.22	12.98	2.33	0.40	3.96	0.08
18	0.00	11.50	0.96	0.11	1.59	13.95	2.49	0.48	4.83	0.09
21	0.00	15.00	1.10	0.13	2.01	14.93	2.66	0.57	5.74	0.11
24	0.00	19.00	1.25	0.16	2.46	15.91	2.82	0.67	6.67	0.12
27	0.00	23.00	1.52	0.20	2.96	16.89	2.99	0.76	7.63	0.14
30	0.00	28.75	1.82	0.23	3.50	17.86	3.15	0.86	8.62	0.15
33	0.00	33.35	2.14	0.27	4.08	18.84	3.32	0.96	9.64	0.17
36	0.00	43.70	2.49	0.31	4.70	19.82	3.48	1.07	10.68	0.18
39	0.00	50.95	2.86	0.36	5.37	20.80	3.65	1.18	11.76	0.20
42	0.00	58.08	3.26	0.41	6.08	21.77	3.81	1.29	12.86	0.22
45	0.00	63.48	3.68	0.46	6.83	22.75	3.98	1.40	13.99	0.23
48	0.00	68.77	4.13	0.51	7.62	23.73	4.14	1.51	15.14	0.25
51	0.00	77.40	4.60	0.56	8.46	24.71	4.31	1.63	16.33	0.26
54	0.00	85.96	5.10	0.62	9.33	25.68	4.47	1.75	17.54	0.28
57	0.00	97.18	5.63	0.68	10.25	26.66	4.64	1.88	18.78	0.29
60	0.00	109.14	6.18	0.75	11.22	27.64	4.80	2.01	20.05	0.31
63	0.00	119.03	6.76	0.81	12.22	28.62	4.97	2.14	21.35	0.32
66	0.00	129.61	7.36	0.88	13.26	29.59	5.13	2.27	22.68	0.34
69	0.00	140.88	7.99	0.96	14.35	30.57	5.30	2.40	24.03	0.35
72	0.00	152.09	8.64	1.03	15.48	31.55	5.46	2.54	25.41	0.37
78	0.00	178.54	10.03	1.12	16.83	33.50	5.79	2.37	23.72	0.38
84	0.00	204.99	11.51	1.21	18.20	35.46	6.12	2.48	24.82	0.40
90	0.00	297.56	12.30	1.31	19.61	37.41	6.45	2.59	25.94	0.42
96	0.00	317.40	13.94	1.40	21.04	39.37	6.78	2.71	27.09	0.43
102	0.00	374.33	14.77	1.50	22.51	41.32	7.11	2.82	28.25	0.45
108	0.00	405.26	16.56	1.60	24.00	43.28	7.44	2.94	29.43	0.46
114	0.00	436.20	17.44	1.70	25.53	45.23	7.77	3.06	30.63	0.48
120	0.00	467.13	20.44	1.81	27.08	47.19	8.10	3.19	31.85	0.49
132	0.00	529.00	22.41	2.02	30.28	51.10	8.76	3.44	34.35	0.52
144	45.00	590.87	25.63	2.24	33.60	55.01	9.42	3.69	36.94	0.55

T, : 2-2

UNIT COSTS FOR INSTALLED NEW FORCE MAIN SEWER PIPE

DIAMETER (inches)	ASPHALT BASE ($/lf)	ASPHALT (ton/lf)	ASPHALT ($/lf)	CURB/ GUTTER (ft/lf)	CURB/ GUTTER ($/lf)	ACCESS & TRAFFIC CONTROL ($/lf)	DEMOB AND SITE CLEANUP ($/lf)	TOTAL CONSTRUCTION COST, PHASE 3 ($/lf)	TOTAL CONSTRUCTION COST, PHASE 2 ($/lf)
12	1.18	0.06	7.00	0	0	3	3.70	84.01	144.32
15	1.48	0.07	7.00	0	0	3	4.00	92.59	167.01
18	1.78	0.09	7.00	0	0	3	4.30	101.36	181.06
21	2.07	0.10	7.00	0	0	3	4.60	111.39	197.15
24	2.37	0.12	9.00	0	0	3	4.90	123.69	209.07
27	2.66	0.13	9.00	0	0	3	5.20	134.30	226.55
30	2.96	0.15	9.00	0	0	3	5.50	146.88	241.05
33	3.25	0.16	10.00	0	0	3	5.80	159.52	260.17
36	3.55	0.18	10.00	0	0	3	6.10	177.14	342.11
39	3.85	0.19	10.00	0	0	3	6.40	191.87	383.48
42	4.14	0.21	12.00	0	0	3	6.70	210.66	407.44
45	4.44	0.22	12.00	0	0	3	7.00	224.47	435.30
48	4.73	0.24	13.00	0	0	5	7.30	241.43	460.47
51	5.03	0.25	13.00	0	0	5	7.60	259.00	484.15
54	5.33	0.27	14.00	0	0	5	7.90	277.76	508.50
57	5.62	0.28	14.00	0	0	5	8.20	298.45	539.88
60	5.92	0.30	15.00	0	0	5	8.50	321.15	567.13
63	6.21	0.31	15.00	0	0	5	8.80	341.04	592.80
66	6.51	0.33	16.00	0	0	10	9.10	367.90	629.30
69	6.81	0.34	17.00	0	0	10	9.40	390.72	662.12
72	7.10	0.36	18.00	0	0	10	9.70	405.07	694.11
78	7.40	0.37	20.00	0	0	10	10.30	446.04	749.19
84	7.69	0.39	21.00	0	0	10	10.90	489.27	842.29
90	7.99	0.40	23.00	0	0	10	11.50	599.31	953.06
96	8.28	0.42	25.00	0	0	10	12.10	637.87	1003.91
102	8.58	0.43	27.00	0	0	15	12.70	718.11	1185.11
108	8.88	0.45	30.00	0	0	15	13.30	769.71	1260.95
114	9.17	0.46	33.00	0	0	15	13.90	820.81	1335.31
120	9.47	0.48	36.00	0	0	15	14.50	874.43	1421.60
132	10.06	0.51	39.00	0	0	15	15.70	975.88	1544.93
144	10.65	0.54	42.00	0	0	15	16.90	1125.27	1724.39

SECTION 3
UNIT COST FOR NEW SEWER SERVICES

A. INTRODUCTION

One component common to all improvement plans was the collection system upgrade. This work includes the installation of replacement sewers and sewer service connections for existing sewers sized 16-inches or smaller. These new sewers would be sized accordingly for the 2-, 5-, 10- or 25-year storm event to provide flood protection, or would be sized to handle sanitary flows only in the case of Separate Sanitary Sewer Alternative.

A portion of the overall cost of the collection system upgrade is that associated with the removal and replacement of the existing sewer service laterals that connect the individual customer's drain/sanitary piping to the City's conveyance system. In instances where the existing combined sewers in the street are upgraded or separated (as in the case of the Separate Sanitary Sewer Alternative), it is anticipated that the majority of these sewer service connections will be disturbed or destroyed and will need to be replaced. Under the current City sewer ordinances, the City is responsible for these services from the street (connection to the sewer) to the back side of the sidewalks. This is the segment of service lateral which would have to be replaced as part of the overall sewer upgrade program.

This section presents the analysis of costs associated with new sewer service connections for the Phase 3 study. The unit costs developed in this section will be referenced in the preparation and presentation of collection system improvements during Task 5, and in the preparation of overall improvement plan cost estimates during Task 6.

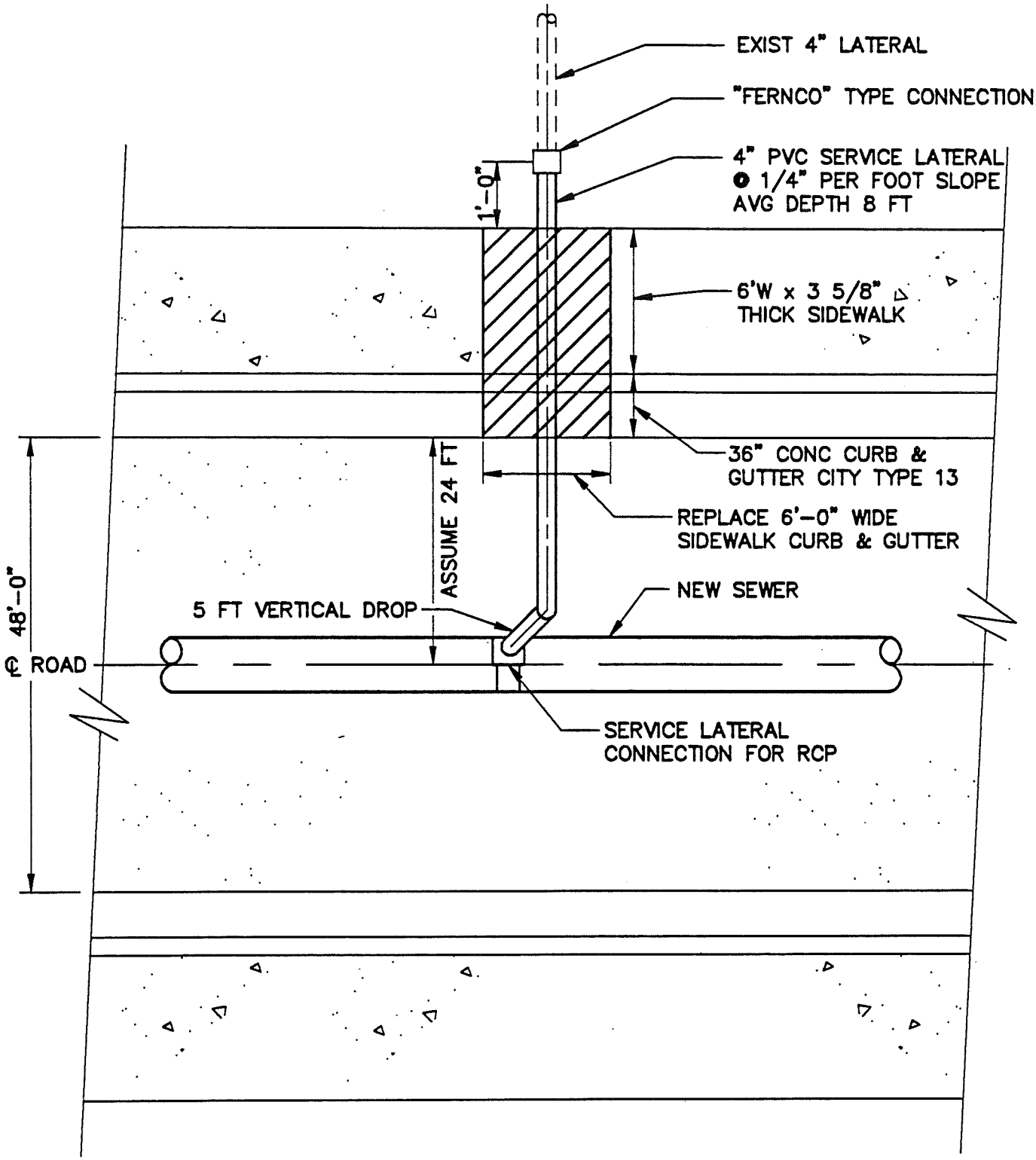
B. ASSUMPTIONS USED

For the purpose of preparing a cost estimate for a representative, new service connection, a typical sewer layout was defined. The configuration used for a

service lateral in relation to a typical street, is presented in Figure 3-1. As indicated by the figure, the paved street width is 48 feet, with the sewer aligned with the street center line. Laterals are assumed to be 4-inch PVC at an average depth of 8 feet below grade and run at a minimum slope of 0.02 ft/ft. It is further assumed that the lateral connection includes a vertical drop of 5 feet at the point of with the mainline sewer, connection and that the connection includes a 1/4 bend connected to a service lateral saddle connection on the sewer pipe. The street sewer used in this analysis was 15 inches in diameter. The removal and replacement of the curb/gutter, sidewalk, asphalt paving and pavement base is also a part of the cost of the lateral replacement.

C. ESTIMATED COSTS

Estimated costs for the removal and replacement of service laterals in accordance with the assumptions stated above and shown in Figure 3-1 are presented in Table 3-1. The total estimated cost for sewer service replacement is $2,450 per customer.



NEW SEWER SERVICE LATERAL DETAIL

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Figure 3-1

TABLE 3-1. UNIT COST FOR SEWER SERVICE LATERALS				
Description	Quantity	Units	Unit Cost ($)	Total ($)
Concrete Cutting and Removal	All Req'd	LS	$110	$110
Excavation	20	CY	$6.00	$120
Pipe Bedding	1.2	CY	$15.00	$18
New Sewer Lateral Materials	All Req'd	LS	$125	$125
Pipe Laying, Labor and Equipment	All Req'd	LS	$850	$850
Backfill	17	CY	$5.00	$85
Asphalt Base	1.3	CY	$19.25	$25
Asphalt Paving	0.75	CY	$150.00	$113
New Curb and Gutter	54	SF	$6.25	$338
Supervision	All Req'd	LS	$178.0	$178
Subtotal				$1,962
Overhead and Profit				$ 491
Total Estimated Cost				$2,453

SECTION 4
UNIT COST FOR NEW STORM DRAINAGE DROP INLETS

A. INTRODUCTION

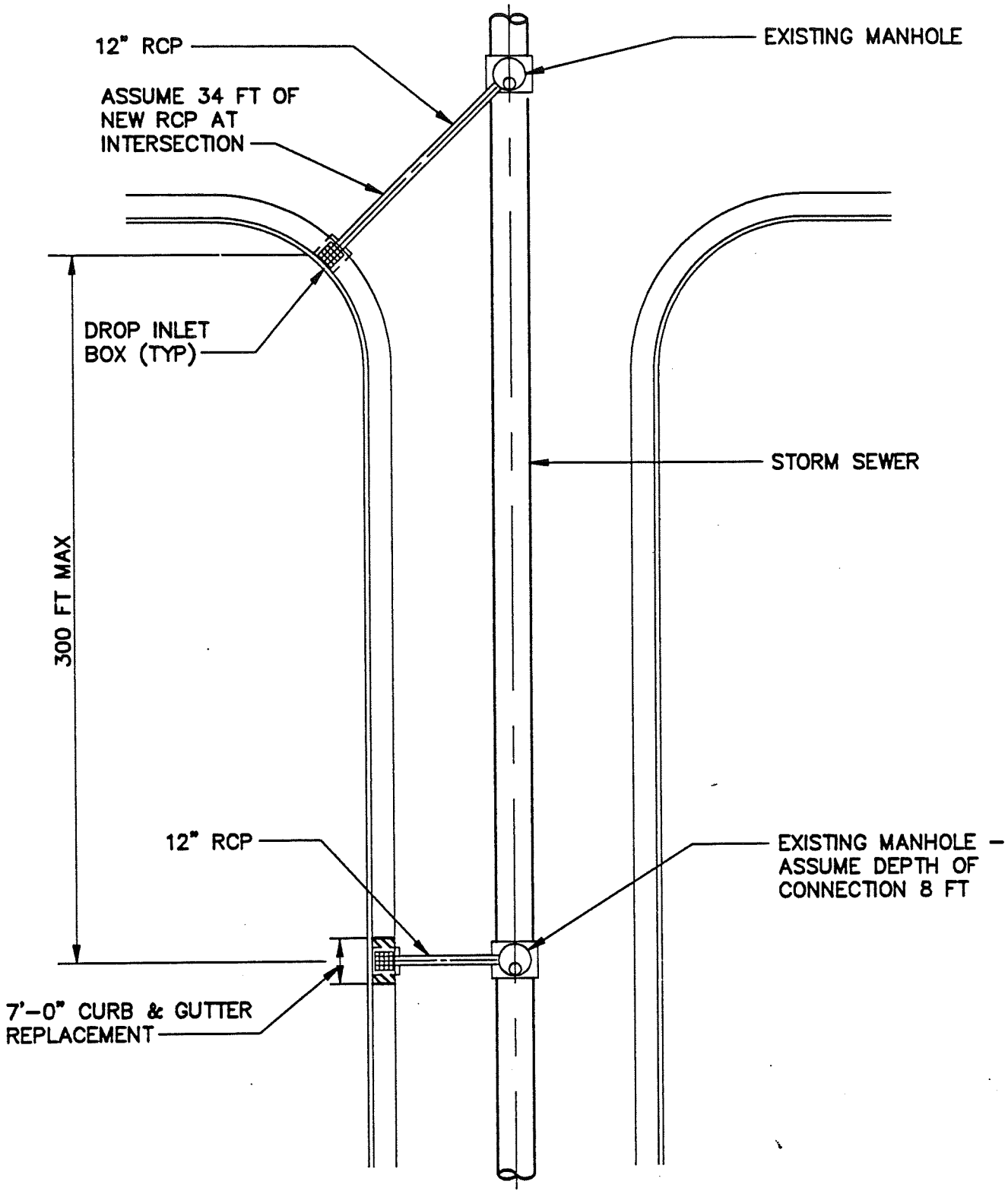
With the exception of the separation alternative involving the construction of a totally new sanitary sewage collection system, all the proposed improvement projects would involve the construction of new storm drainage inlets (DI's) and connecting drain lines. These new facilities would be necessary to replace existing infrastructure that is no longer functional or to provide additional relief for surface flooding during major storm events.

This section presents the unit costs associated with the installation of replacement drop inlets and drop inlet drain lines.

B. ASSUMPTIONS

The configuration of drop inlets and street drainage used in developing estimates of construction costs are presented in Figure 4-1. Drop inlet boxes are assumed to be City of Sacramento Type B drop inlets. Drop inlet drain lines are all assumed to be nominal 12 inches in diameter, reinforced concrete pipe. Connection to the street sewer is via a manhole. The curb and gutter is assumed to be City of Sacramento #13. Asphalt paving is 5 inches thick with a 10-inch base. Excavated native material is assumed to be suitable as backfill.

The number of new drop inlets and connecting drain lines required as a part of an overall improvement plan will depend on both the alternative layout and the design storm considered. Controlling flooding and outflows for a 25-year event will obviously require the addition of many more DI's than a 5-year storm event. To provide a unit cost per drop inlet added, two types of drop inlet are reviewed, one at an intersection and one at the gutter between street intersections. New drop inlets are assumed to be constructed in pairs. Figure 4-1 shows both types of drop inlets.



TYPICAL DI CONNECTION DETAIL

HDR

Figure 4-1

C. ESTIMATED COSTS

Costs for the drop inlets shown in Figure 4-1 are presented in Tables 4-1 and 4-2. For purposes of establishing a unit cost for inclusion in the collection system upgrade cost, it is recommended that the average of the two costs, $2,600, be used to estimate the costs for new drop inlets.

TABLE 4-1. UNIT COSTS FOR DROP INLETS - MID STREET

Description	Quantity	Units	Unit Cost ($)	Total ($)
Cut and Remove Existing Curb and Gutter	All Req'd	LS	$50.00	$50
Cut and Remove Existing Paving	All Req'd	LS	$60.00	$60
Excavation	11	CY	$6.00	$66
Pipe Bedding	1	CY	$15.00	$15
New RCP, Related Materials	25	LF	$5.75	$144
Pipelaying Labor and Equip	All Req'd	LS	$850.00	$850
Backfill	8	CY	$5.00	$40
Asphalt Base	1.67	CY	$19.25	$32
Asphalt Paving	0.74	CY	$150.00	$111
New Curb and Gutter Section	9	SF	$6.00	$54
New Drop Inlet With Curb Hood and Grate	1	EA	$350.00	$350
Supervision	All Req'd	LS	$177.00	$177
Subtotal				$1949
Overhead and Profit				$487
Total Estimated Cost				$2436

TABLE 4-2. UNIT COSTS FOR DROP INLETS - STREET INTERSECTION				
Description	Quantity	Units	Unit Cost ($)	Total($)
Cut and Remove Existing Curb and Gutter	All Req'd	LS	$50.00	$50
Cut and Remove Existing Paving	All Req'd	LS	$85.00	$85
Excavation	16	CY	$6.00	$96
Pipe Bedding	1.5	CY	$15.00	$23
New RCP, Related Materials	35	LF	$5.75	$201
Pipelaying Labor and Equip	All Req'd	LS	$850.00	$850
Backfill	11	CY	$5.00	$55
Asphalt Base	2.3	CY	$19.25	$44
Asphalt Paving	1.05	CY	$150.00	$158
New Curb and Gutter Section	9	SF	$6.00	$54
New Drop Inlet With Curb Hood and Grate	1	EA	$350.00	$350
Supervision	All Req'd	LS	$197.00	$197
Subtotal				$2163
Overhead and Profit				$540
Total Estimated Cost				$2703

SECTION 5
UNIT COSTS FOR PUMPING STATIONS

A. INTRODUCTION

Unit costs for pumping stations used for the Phase 2 study were generated using the process cost estimating program entitled "WATER CO$T" developed by Culp Wesner Culp (HDR Engineering, Inc.) in 1986. As described in the Phase 2 report, this program provides estimates of construction cost for water and wastewater processes. Input required for the program includes current labor and materials cost indices published by the US Bureau of Labor Statistics and the *Engineering News Record*. These indices have been updated for this Phase 3 report using October 1991 data and are listed in Table 5-1.

TABLE 5-1. COST INDICES USED IN WATER CO$T INPUT, OCTOBER 1991

Index	Value	Source
General Purpose Mach.	389.1	BLS 114[1]
Concrete	366.8	BLS 132
Steel	376.9	BLS 1017
Skilled Labor	452.5	ENR[2]
Pipe/Valve	446.6	BLS 1149
Electrical	279.5	BLS 117
Housing	412.2	ENR
Housing	150.0	Local Unit Cost
Producer	343.2	Price Index

1 - US Bureau of Labor Statistics
2 - Engineering News Record

The WATER CO$T cost estimating program provides a "base" construction cost for a particular process. The base construction cost for pumping stations computed by the program assumes a pumping station configured with screening, a wet well separated from the dry well, pumping equipment, piping within the structure

limits, electrical power distribution, lighting, equipment controls, heating and ventilating, and other auxiliary equipment including hoists and elevators.

For purposes of this study, the pumping station construction costs also include the following items:

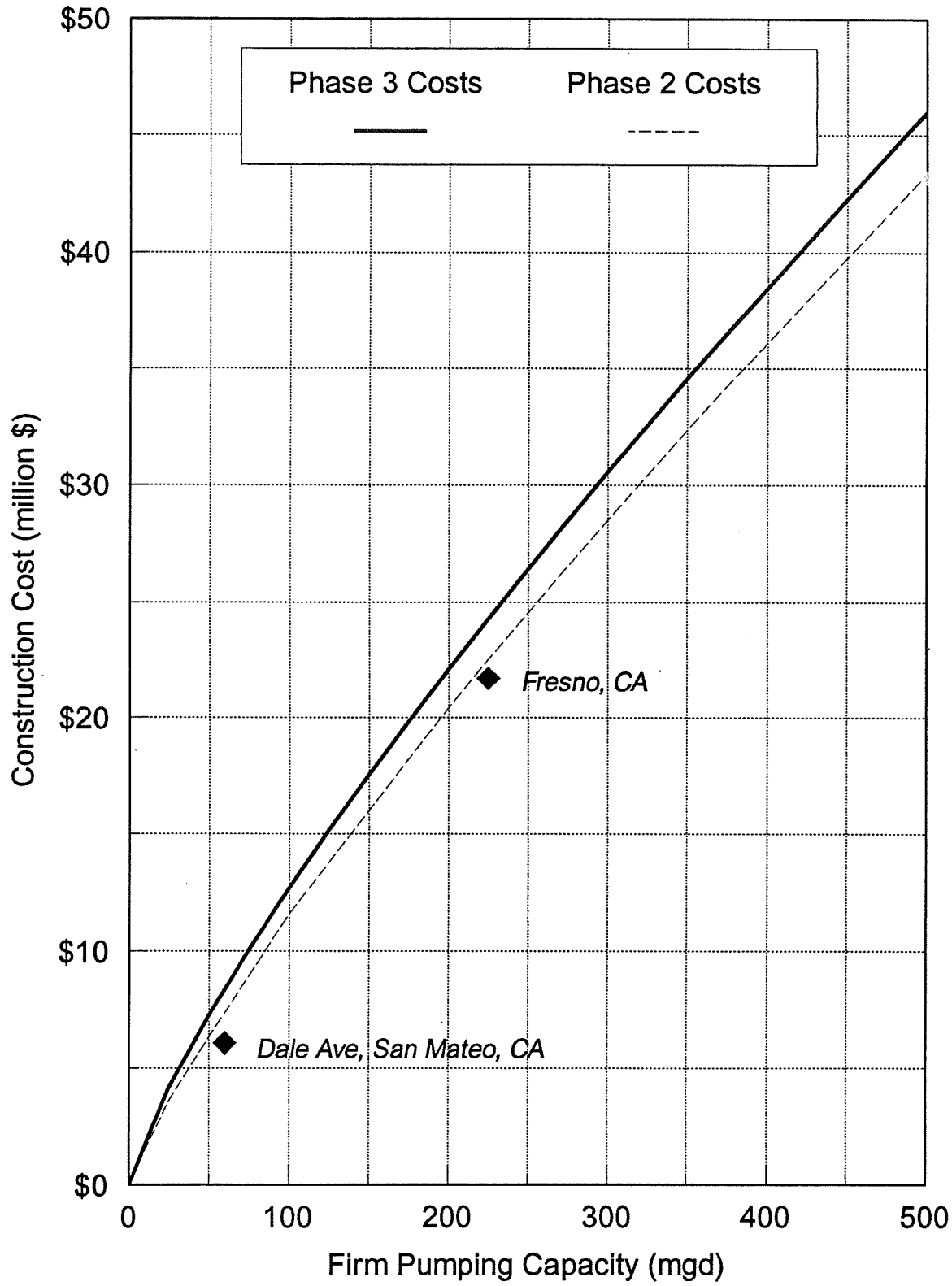
- Special site work and interface piping with the sewer system and storage reservoirs. This is estimated to represent 15 percent of the base construction cost.

- Special subsurface costs related to the pumping station sites including excavation, shoring and sheeting, dewatering, and piling foundation requirements for the pumping station wetwell. These costs are estimated to represent 10 percent of the base construction costs.

- Standby power costs are assumed to include the cost of providing independent electrical power supplies for the pumping stations. This would include either a standby diesel engine generation station or an additional power feeder from the SMUD distribution grid, or a combination of these alternatives. The cost for standby power is estimated to be 10 percent of the base construction cost.

Odor control costs are assumed to be part of the storage reservoir costs and are not included as part of the pumping station construction costs.

The output of the WATER CO$T estimating model for the aforementioned assumptions and input is shown in Figure 5-1 for both the Phase 2 and Phase 3 studies. The Phase 3 data have been recomputed using updated Bureau of Labor statistics and ENR indices. Firm pumping capacity as presented in the figure is defined as the rated output of the pumping station with one of the largest pumps out of service. Estimated construction costs are based on October 1991 dollars.



PUMP STATION CONSTRUCTION COST USED IN THE PHASE 3 PROJECT COST ESTIMATES

HDR

Figure 5-1

B. COMPARISON OF ESTIMATED UNIT COSTS WITH PROJECT COST DATA

The construction costs of two pumping station projects were reviewed and compared with the costs predicted by the curve presented in Figure 5-1. These projects are described below.

B.1. Fresno Headworks Project

An expansion of the Fresno/Clovis, California Regional Wastewater Treatment Plant is being designed and includes a new headworks. The headworks will include mechanically cleaned screens, a wet well/dry well configuration, pumping equipment with a firm pumping capacity of 225 mgd, grit removal, flow monitoring, and odor control. The estimated construction cost is $21.7 million in 1991 dollars, which excludes grit removal, odor control, and flow monitoring. This point is plotted on Figure 5-1.

B.2. Dale Avenue Pumping Station, San Mateo, California

The Dale Avenue Pumping Station in San Mateo, California was bid in 1990 with construction completed in the fall of 1991. The station, which has a firm pumping capacity of 60 mgd, includes a wet well/dry well configuration, and an access elevator, but no odor control or screening facilities. Excluding odor control but including screening facilities, the cost of the station in 1991 dollars is estimated to be $6.0 million.

B.3. Comparison

Comparing the costs of these pumping facilities against the cost curve presented in Figure 5-1, it appears that the estimated costs predicted by the WATER CO$T model are conservative.

SECTION 6
UNIT COSTS FOR STORAGE RESERVOIRS

A. INTRODUCTION

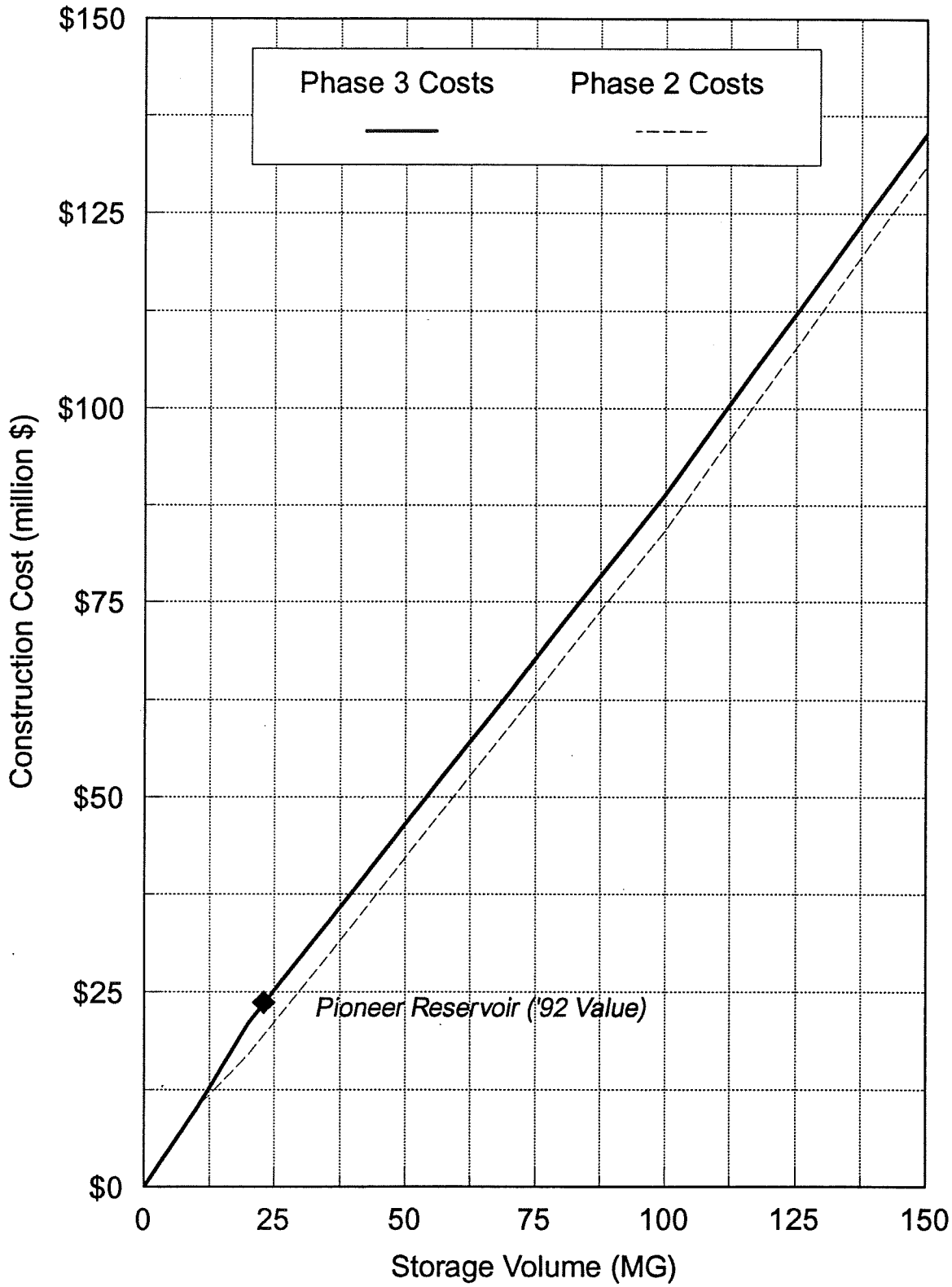
In addition to developing the estimates of pumping station costs presented in Section 5, the Phase 2 report also provided estimates of construction costs for above-ground storage facilities. As with the pumping stations, costs for above-ground storage facilities were generated for the Phase 2 report using the WATER CO$T estimating program with the cost indices presented in Table 5-1.

The "base" construction cost data for a storage structure presented in the Phase 2 report were generated by using the costs for square covered aeration basins, 15-foot side water depth, with the aeration system deleted. Adjustment of the cost data from the cost estimating program was provided for an additional 5 feet of side water depth, odor control, pumps, piping and appurtenances required for the wash down system, level monitoring system, telemetry, construction dewatering, internal partition walls, and foundation support piling. Not included were costs associated with pumping wastewater into or out of the reservoir.

The output of the WATER CO$T program for storage reservoirs is shown in Figure 6-1 for both the Phase 2 and Phase 3 reports. The Phase 3 data have been recomputed using updated Bureau of Labor statistics and ENR indices. It has also been adjusted in accordance with the discussion below. Storage reservoir capacity is assumed to be the maximum storage capacity of the structure without an overflow. Estimated construction costs are based on October 1991 dollars.

B. COMPARISON OF ESTIMATED UNIT COSTS WITH PROJECT COST DATA

The 1978 construction bid for the 23-million gallon Pioneer Reservoir facility was $13.47 million, which included minor upgrade work to Sump 1A. Using the ENR construction cost index to update this cost to 1991 dollars, the resulting $23.6 million project is plotted on Figure 6-1. The Phase 2 data were adjusted to coincide with this point and the revised curve plotted as indicated in Figure 6-1. The revised cost data were used in preparing construction costs for storage reservoirs in the Phase 3 study.



STORAGE RESERVOIR UNIT COST USED IN THE PHASE 3 PROJECT COST ESTIMATES

HDR

Figure 6-1

C. CONSTRUCTION DEWATERING COSTS AND THE UCD MEDICAL CENTER SITE

The interim storage reservoir under consideration for the UCD Medical Center site may be constructed as a below ground structure. It is assumed that the proposed structure will be below the local water table.

The dewatering cost for a large storage facility would be substantial. Based on an assumed structure area of 20,000 square feet and an average depth of 20 feet below ground surface, the construction dewatering costs could be as much as $250,000.

Of greater concern is the issue of how to protect the structure from the buoyant forces caused by the presence of a shallow water table. If the storage reservoir is to be kept empty and available for receiving wastewater flows, the buoyant uplift forces must either be countered with extra dead loading or the area must be continually dewatered and the reservoir fitted with pressure relief valves in the floor slab to provide structural protection in the event of a pumping failure. Alternatively, tie-down piles could be used to hold the structure in place.

The recommended approach to the problem is to avoid construction of the reservoir below the maximum expected area groundwater table. This could either be done through shortening the side water depth of the reservoir or raising the entire structure to an elevation where the influence of the water table is no longer an issue.

SECTION 7
UNIT COSTS FOR SIDE OVERFLOW WEIR JUNCTION STRUCTURES

A. INTRODUCTION

The storage and tunnel alternatives require weir diversion structures. These structures are intended to contain the dry weather sanitary flows within the existing collection system but relieve excess combined wastewater flows into a new network of consolidation conduits. These consolidation conduits convey wastewater to either storage or the tunnel system.

This section reviews estimated construction costs for side overflow weir junction structures developed for the Phase 3 study.

B. ASSUMPTIONS

The assumptions used in preparing unit costs for the overflow weir structures are listed below.

- The depth of the existing sewer on which the new overflow weir junction structure is constructed is the same as the depth of the new sewers assumed for Table 2-1. This range begins with a 24-inch sewer at an assumed depth of 16 feet and ends with a 114-inch sewer at a depth of 32 feet.

- Two configurations for overflow weirs were evaluated. One was for a single overflow weir and the other was for a double-sided overflow weir. The double-sided configuration, illustrated in Figure 7-1, was more cost effective, and therefore chosen over the single-sided weirs as a model for estimating construction costs.

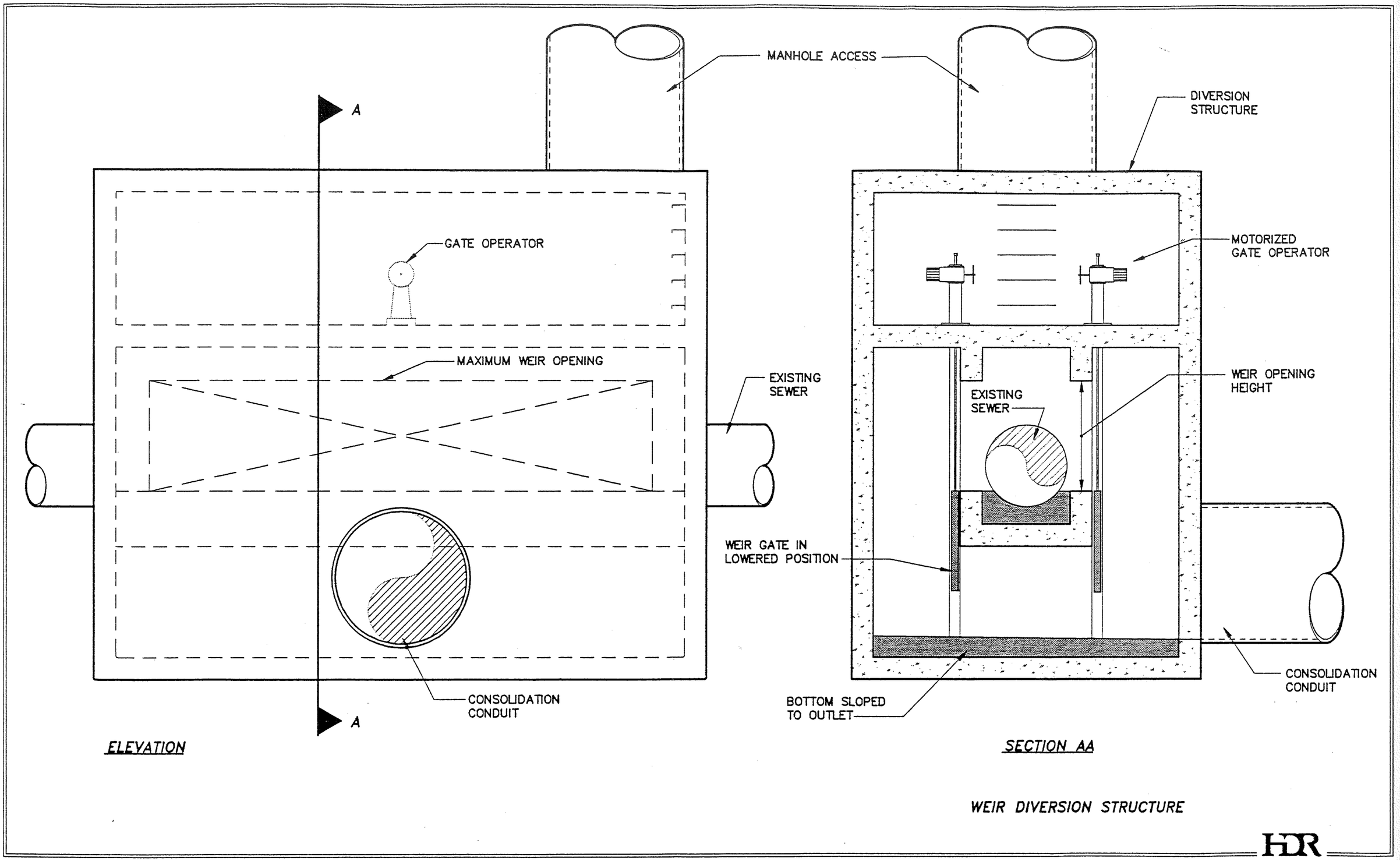

MANHOLE ACCESS
A
DIVERSION STRUCTURE
GATE OPERATOR
MOTORIZED GATE OPERATOR
MAXIMUM WEIR OPENING
EXISTING SEWER
WEIR OPENING HEIGHT
EXISTING SEWER
WEIR GATE IN LOWERED POSITION
CONSOLIDATION CONDUIT
A
CONSOLIDATION CONDUIT
BOTTOM SLOPED TO OUTLET
ELEVATION
SECTION AA
WEIR DIVERSION STRUCTURE
HDR

Figure 7-1

- The walls, roof and floor of the weir junction structure are 12 inches thick, reinforced cast-in-place concrete with an average unit cost of $500/cy. Access to the structure is via a conventional 48-inch diameter manhole.

- The elevation of the weir crest is equal to the invert elevation of the existing sewer plus 1/3 the diameter of the existing sewer.

- The nominal diameter of the new consolidation conduit is identical to the diameter of the existing sewer except where the SSWMM91 indicated a larger conduit was needed for model stability at the weir.

- The cost of dewatering the site of a side overflow weir junction box ranges from $9,000 to $25,000, depending upon the depth of the structure and one month of dewatering operations cost.

- Equipment costs for excavation, backfill, trucking, compaction, and other miscellaneous equipment will vary from $3400 to $4500 depending upon the depth of excavation.

C. ESTIMATED CONSTRUCTION COSTS

Estimated total construction costs for the double-sided weir configuration are presented in Table 7-1.

TABLE 7-1
SIDE OVERFLOW DIVERSION WEIR UNIT COSTS

EXISTING PIPE DIAMETER (feet)	NEW PIPE DIAMETER (feet)	TOTAL WEIR LENGTH L (feet)	JUNCTION BOX WIDTH W (feet)	JUNCTION BOX HEIGHT (feet)	JUNCTION BOX TOTAL LENGTH (feet)	BOX END WALLS VOLUME (cu yds)	LONGITUDINAL WALLS VOLUME (cu yds)	TOP BOX VOLUME (cu yds)	BOTTOM BOX VOLUME (cu yds)	WEIR WALL VOLUME (cu yds)
2.00	2.00	15.00	6.17	12.58	9.17	4.60	6.41	1.74	1.74	0.35
2.50	2.50	15.33	7.17	13.08	9.33	5.49	6.78	2.06	2.06	0.44
3.00	3.00	15.67	8.17	13.58	9.50	6.41	7.17	2.39	2.39	0.53
3.50	3.50	16.00	9.17	14.08	9.67	7.38	7.56	2.73	2.73	0.63
4.00	4.00	16.33	10.17	14.58	9.83	8.38	7.97	3.09	3.09	0.72
4.50	4.50	16.67	11.17	15.08	10.00	9.42	8.38	3.45	3.45	0.82
5.00	5.00	17.00	12.17	15.58	10.17	10.49	8.80	3.82	3.82	0.93
5.50	5.50	17.33	13.17	16.08	10.33	11.61	9.23	4.20	4.20	1.03
6.00	6.00	17.67	14.17	16.58	10.50	12.76	9.67	4.59	4.59	1.15
6.50	6.50	18.00	15.17	17.08	10.67	13.95	10.12	4.99	4.99	1.26
7.00	7.00	18.33	16.17	17.58	10.83	15.17	10.58	5.41	5.41	1.38
7.50	7.50	18.67	17.17	18.08	11.00	16.44	11.05	5.83	5.83	1.50
8.00	8.00	19.00	18.17	18.58	11.17	17.74	11.53	6.26	6.26	1.62
8.50	8.50	19.33	19.17	19.08	11.33	19.08	12.02	6.70	6.70	1.75
9.00	9.00	19.67	20.17	19.58	11.50	20.45	12.51	7.16	7.16	1.88
9.50	9.50	20.00	21.17	20.08	11.67	21.87	13.02	7.62	7.62	2.02
10.00	10.00	20.33	22.17	20.58	11.83	23.32	13.53	8.10	8.10	2.15
11.00	11.00	21.00	24.17	21.58	12.17	26.33	14.59	9.07	9.07	2.44
12.00	12.00	21.67	26.17	22.58	12.50	29.50	15.68	10.10	10.10	2.74

TABLE 7-1
SIDE OVERFLOW DIVERSION WEIR UNIT COSTS

EXISTING PIPE DIAMETER (feet)	AVERAGE SEWER INVERT DEPTH (feet)	DEPTH OF 48" MANHOLE RISER (feet)	MANHOLE RISER COST ($)	TOTAL CONCRETE VOLUME (cu yds)	COST OF CONCRETE ($)	GROUT VOLUME (cu yds)	GROUT COST ($)	PIPE BELLS ($)	MECHANICAL WEIR GATES ($)	TOTAL MATERIALS ($)
2.00	16	8.67	1560	14.85	8909.17	0.52	25.83	100	8000	18595
2.50	18	10.50	1890	16.84	10103.78	0.74	36.82	145	9000	21176
3.00	20	12.33	2220	18.90	11340.85	1.00	49.83	190	10000	23801
3.50	22	14.17	2550	21.03	12620.39	1.30	64.94	250	11000	26485
4.00	24	16.00	2880	23.24	13942.38	1.65	82.27	300	12000	29205
4.50	25	16.83	3030	25.51	15306.83	2.04	101.91	375	13000	31814
5.00	26	17.67	3180	27.86	16713.73	2.48	123.96	475	14000	34493
5.50	27	18.50	3330	30.27	18163.10	2.97	148.52	560	15000	37202
6.00	28	19.33	3480	32.76	19654.93	3.51	175.69	665	16000	39976
6.50	29	20.17	3630	35.32	21189.21	4.11	205.56	775	17000	42800
7.00	29	20.00	3600	37.94	22765.96	4.76	238.25	890	18000	45494
7.50	29	19.83	3570	40.64	24385.16	5.48	273.84	1290	19000	48519
8.00	30	20.67	3720	43.41	26046.82	6.25	312.44	1380	20000	51459
8.50	31	21.50	3870	46.25	27750.94	7.08	354.15	1625	21000	54600
9.00	31	21.33	3840	49.16	29497.52	7.98	399.07	1760	22000	57497
9.50	32	22.17	3990	52.14	31286.56	8.95	447.29	1900	23000	60624
10.00	32	22.00	3960	55.20	33118.06	9.98	498.92	2100	24000	63677
11.00	33	22.67	4080	61.51	36908.43	12.26	612.79	2300	26000	69901
12.00	33	22.33	4020	68.11	40868.63	14.83	741.48	2550	28000	76180

TABLE 7-1
SIDE OVERFLOW DIVERSION WEIR UNIT COSTS

EXISTING PIPE DIAMETER (feet)	EXCAVATION SHEETING (sf)	EXCAVATION SHEETING ($)	MOBILIZATION DEMOBILIZATION CLEAN UP ($)	LABOR & SUPERVISION COST ($)	EQUIPMENT COSTS ($)	CONSTRUCTION DEWATERING COSTS ($)	OVERHEAD AND PROFIT ($)	TOTAL CONSTRUCTION COST ($)
2.00	335.00	5025.00	700	52250	24000	9000	27393	136963
2.50	433.94	6509.17	725	53412	24533	11000	29339	146693
3.00	542.28	8134.17	750	54573	25067	13000	31331	156656
3.50	660.00	9900.00	775	55735	25600	15000	33374	166869
4.00	787.11	11806.67	800	56896	26133	17000	35460	177301
4.50	879.28	13189.17	825	58058	26667	18000	37138	185691
5.00	976.50	14647.50	850	59220	27200	19000	38852	194262
5.50	1078.78	16181.67	875	60381	27733	20000	40593	202966
6.00	1186.11	17791.67	900	61543	28267	21000	42369	211846
6.50	1298.50	19477.50	925	62704	28800	22000	44177	220883
7.00	1360.78	20411.67	950	63866	29333	22000	45514	227569
7.50	1423.78	21356.67	975	65028	29867	22000	46936	234681
8.00	1547.00	23205.00	1000	66189	30400	23000	48813	244067
8.50	1675.28	25129.17	1025	67351	30933	24000	50760	253798
9.00	1744.78	26171.67	1050	68512	31467	24000	52174	260872
9.50	1881.00	28215.00	1075	69674	32000	25000	54147	270735
10.00	1954.11	29311.67	1100	70836	32533	25000	55614	278072
11.00	2175.00	32625.00	1150	73159	33600	26000	59109	295544
12.00	2330.61	34959.17	1200	75482	34667	26000	62122	310610

SECTION 8
UNIT COSTS FOR TUNNELS AND DROP SHAFTS

A. INTRODUCTION

For the improvement plans involving tunnels, combined wastewater delivered to the tunnel system must pass through a drop shaft structure. These structures must be as deep as the tunnel system, and include substantial amounts of excavation, shoring, and dewatering. This section reviews the construction costs for drop shaft structures. It also presents tunnel unit costs for a larger range of tunnel diameters than presented in the Phase 2 study. Costs developed in this section were included in the overall total project cost for the tunnel alternative.

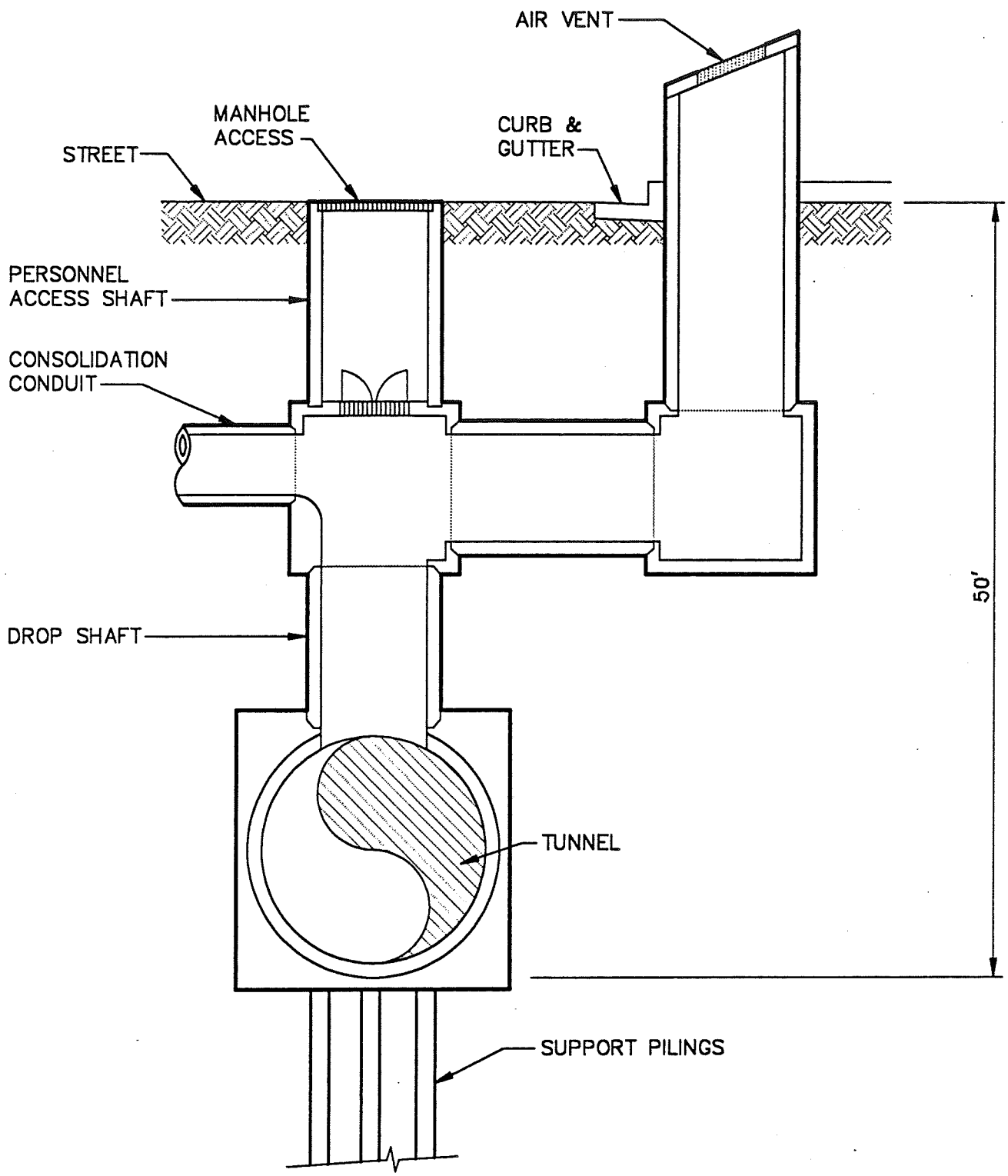
B. ASSUMPTIONS FOR DROP SHAFT CONSTRUCTION

The configuration of the drop shaft structure used for cost estimating purposes is shown in Figure 8-1. The structure is comprised of a vertical shaft with a personnel access shaft at the top, and an air vent shaft passing through the sidewalk. The air vent admits air into the shaft as flow cascades into the tunnel and prevents excessive air pressure from accumulating within the tunnels. Piling supports are provided at the base of the vertical assembly. The shaft is connected to the crown of the tunnel, as opposed to the side, for ease of constructibility. A plunge pool is not considered necessary for the proposed depth of the Sacramento tunnel system.

Other assumptions used in the development of the cost estimate are listed below.

- The excavation for the shaft would be advanced using a circular steel caisson. The cost of the caisson was estimated at $2.00/pound in place.

- Construction dewatering was estimated by Brierley and Lyman, Inc., to be roughly $35,000 for four wells drilled to 65 feet. An additional $7,000 was assumed for providing pumps, power and labor to operate the wells for a 6- to 7-month period while the structure was under construction.



TUNNEL DROP SHAFT SCHEMATIC

HDR

Figure 8-1

- Excavation would be accomplished with a crane operated clamshell. Unit costs for excavation were estimated to be $8.50/cy. This assumes that the excavated material could be stored and used as fill elsewhere.

- The piling at the base of the drop shaft would be driven 25 feet into native soils at a 4.5-foot spacing interval, at a unit cost of $20/lf of piling.

- Cast in place concrete was estimated at $600/cy.

- Unit costs for asphalt paving, asphalt base, and curb and gutter are the same as discussed in Sections 2 and 3.

C. ESTIMATED CONSTRUCTION COSTS

Table 8-1 presents the detailed estimate of construction costs for the tunnel drop shafts.

D. TUNNEL UNIT COSTS

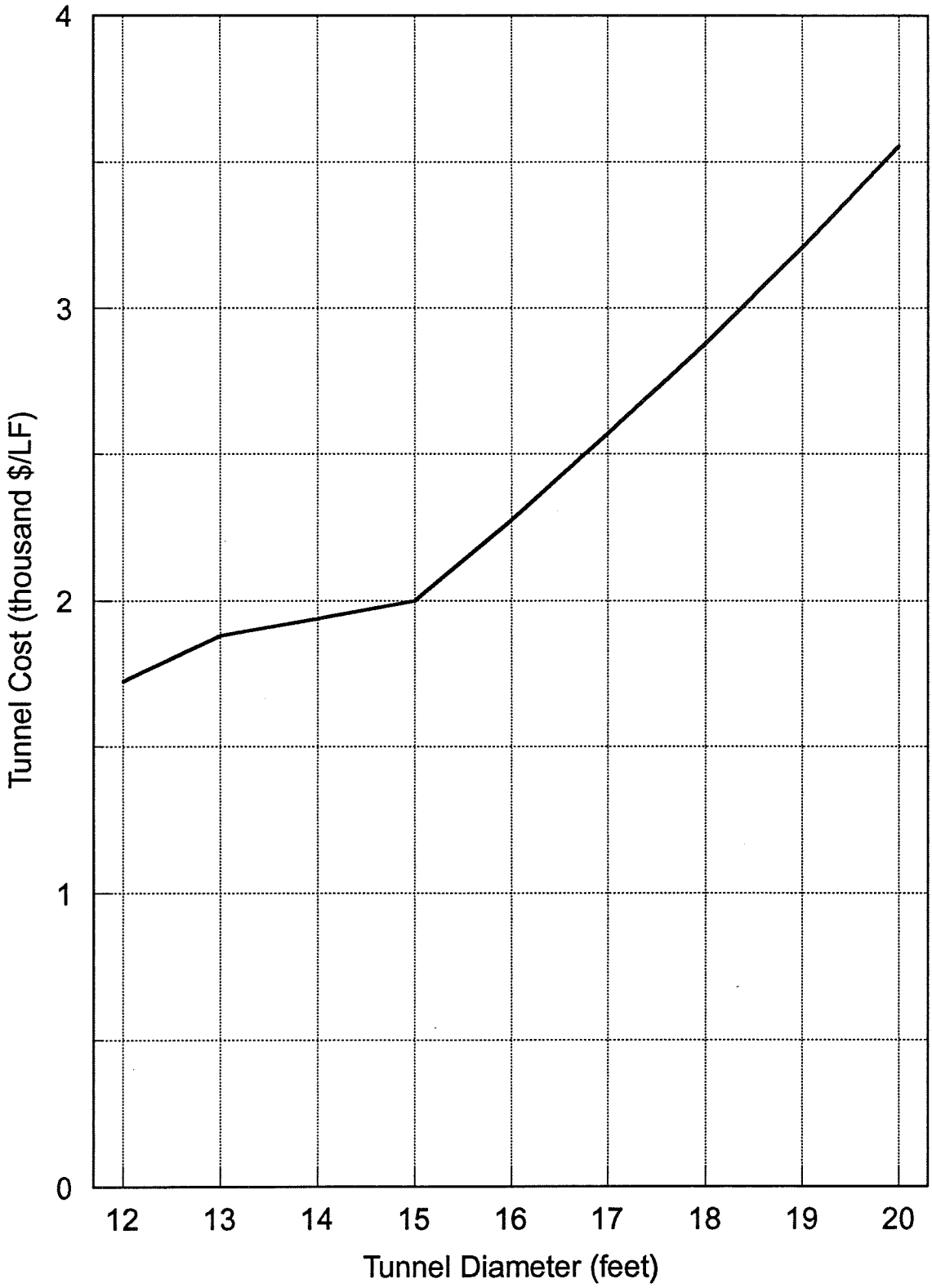
The tunnel diameters considered for the Phase 3 study varied from 13 to 20 feet in diameter. Larger diameter tunnels were not recommended for the soil conditions underlying the study area. Tunnel unit costs used in the Phase 3 study were developed as follows. For tunnel diameters ranging from 13 to 15 feet, the unit costs presented in the Phase 2 report were used. For diameters between 16 and 20 feet, the unit cost was taken to be proportional to the cross-sectional area of the tunnel, beginning with a unit cost of $2000/lf for a 15-foot tunnel. Conduits 12 feet in diameter and smaller were assumed to be installed as gravity sewers at costs presented in Section 2. The tunnel unit cost recommendations were provided by Brierley and Lyman, geotechnical engineers. Figure 8-2 illustrates these tunnel unit costs.

TABLE 8-1
TUNNEL DROP SHAFT COSTS

SHAFT DIAMETER (feet)	CASSION STEEL (pounds)	CASSION COST ($)	DEWATERING COST ($)	EXCAVATION (cu yds)	EXCAVATION COST ($)	DROP SHAFT CONCRETE (cu yds)	PILING COST ($)	CONCRETE COST ($)	BACKFILL (cu yds)
2.00	95971	191942	42000	1308.33	11121	318.52	32000	159260	790.34
2.50	95971	191942	42000	1308.33	11121	320.34	32000	160168	786.96
3.00	95971	191942	42000	1308.33	11121	322.34	32000	161168	782.84
3.50	95971	191942	42000	1308.33	11121	324.33	32000	162167	777.97
4.00	95971	191942	42000	1308.33	11121	326.33	32000	163166	772.35
4.50	95971	191942	42000	1308.33	11121	328.33	32000	164166	765.98
5.00	95971	191942	42000	1308.33	11121	330.33	32000	165165	758.86
5.50	95971	191942	42000	1308.33	11121	332.33	32000	166165	750.98
6.00	95971	191942	42000	1308.33	11121	334.33	32000	167164	742.36
6.50	95971	191942	42000	1308.33	11121	336.33	32000	168164	733.00
7.00	95971	191942	42000	1308.33	11121	338.33	32000	169163	722.88
7.50	95971	191942	42000	1308.33	11121	340.32	32000	170162	712.01
8.00	95971	191942	42000	1308.33	11121	342.32	32000	171162	700.39
8.50	95971	191942	42000	1308.33	11121	344.32	32000	172161	688.02
9.00	95971	191942	42000	1308.33	11121	346.32	32000	173161	674.90
9.50	95971	191942	42000	1308.33	11121	348.32	32000	174160	661.04
10.00	95971	191942	42000	1308.33	11121	350.32	32000	175160	646.42
10.50	95971	191942	42000	1308.33	11121	352.32	32000	176159	631.05
11.00	95971	191942	42000	1308.33	11121	354.32	32000	177158	614.94
12.00	95971	191942	42000	1308.33	11121	358.31	32000	179157	580.46
13.00	95971	191942	42000	1308.33	11121	362.31	32000	181156	542.98
14.00	95971	191942	42000	1308.33	11121	366.31	32000	183155	502.50
15.00	95971	191942	42000	1308.33	11121	370.31	32000	185154	459.03
16.00	95971	191942	42000	1308.33	11121	374.31	32000	187153	412.56
17.00	95971	191942	42000	1308.33	11121	378.30	32000	189151	363.08
18.00	95971	191942	42000	1308.33	11121	397.02	32000	198510	310.61

TABLE 8-1
TUNNEL DROP SHAFT COSTS

SHAFT DIAMETER (feet)	BACKFILL COST ($)	SHAFT LENGTH (feet)	SHAFT COST ($)	ASPHALT BASE (cu yds)	ASPHALT BASE ($)	ASPHALT PAVING (cu yds)	ASPHALT PAVING ($)	CURB/ GUTTER (lf)	CURB/ GUTTER ($)	DEMOB/ CLEANUP ($)	TOTAL COST ($)
2.00	3952	35.00	8565	49.19	947	24.69	3704	80.00	1440	700	455630
2.50	3935	35.00	9922	49.19	947	24.69	3704	80.00	1440	725	457903
3.00	3914	35.00	11320	49.19	947	24.69	3704	80.00	1440	750	460305
3.50	3890	35.00	12986	49.19	947	24.69	3704	80.00	1440	775	462971
4.00	3862	35.00	14648	49.19	947	24.69	3704	80.00	1440	800	465630
4.50	3830	35.00	16173	49.19	947	24.69	3704	80.00	1440	825	468147
5.00	3794	35.00	17930	49.19	947	24.69	3704	80.00	1440	850	470893
5.50	3755	35.00	19831	49.19	947	24.69	3704	80.00	1440	875	473779
6.00	3712	35.00	21670	49.19	947	24.69	3704	80.00	1440	900	476600
6.50	3665	35.00	23484	49.19	947	24.69	3704	80.00	1440	925	479391
7.00	3614	35.00	25970	49.19	947	24.69	3704	80.00	1440	950	482850
7.50	3560	35.00	29852	49.19	947	24.69	3704	80.00	1440	975	487703
8.00	3502	35.00	31592	49.19	947	24.69	3704	80.00	1440	1000	490409
8.50	3440	35.00	34649	49.19	947	24.69	3704	80.00	1440	1025	494429
9.00	3375	35.00	36818	49.19	947	24.69	3704	80.00	1440	1050	497557
9.50	3305	35.00	39002	49.19	947	24.69	3704	80.00	1440	1075	500695
10.00	3232	35.00	41307	49.19	947	24.69	3704	80.00	1440	1100	503951
10.50	3155	35.00	41307	49.19	947	24.69	3704	80.00	1440	1125	504899
11.00	3075	35.00	45655	49.19	947	24.69	3704	80.00	1440	1150	510191
12.00	2902	35.00	50227	49.19	947	24.69	3704	80.00	1440	1200	516640
13.00	2715	35.00	50227	49.19	947	24.69	3704	80.00	1440	1250	518501
14.00	2513	35.00	50227	49.19	947	24.69	3704	80.00	1440	1300	520348
15.00	2295	35.00	50227	49.19	947	24.69	3704	80.00	1440	1350	522179
16.00	2063	35.00	50227	49.19	947	24.69	3704	80.00	1440	1400	523996
17.00	1815	35.00	50227	49.19	947	24.69	3704	80.00	1440	1450	525797
18.00	1553	35.00	50227	49.19	947	24.69	3704	80.00	1440	1500	534943



UNIT COST FOR THE TUNNELS AS USED IN THE PHASE 3 COST ESTIMATES

HDR

Figure 8-2

SECTION 9
OPERATION AND MAINTENANCE COSTS FOR STORAGE AND TUNNEL FACILITIES

A. INTRODUCTION

As with the Phase 2 report, an analysis of revenue requirements for the least cost alternative improvement plan will be addressed in the Phase 3 report, including a discussion of the increase in user fees needed to pay for the CWCS improvements. Key elements for the user fee evaluation are the capital and operating (O&M) costs of the least cost alternative. Capital costs for all alternatives are identified in the Phase 3 technical memorandum entitled *Improvement Plans*.

To collect background information on the O&M costs incurred by other municipalities, a telephone survey was conducted to gather information on O&M procedures and costs. This section summarizes the response to that survey.

B. OPERATION AND MAINTENANCE SURVEY

The names and phone numbers of municipal authorities responsible for CSO control structures were taken from the Phase 2 *Survey of 21 Cities*, and from the current membership listing of the CSO Partnership. In all, a total of 24 contacts were made. The individuals surveyed were asked what their operations and maintenance program included, what manpower staffing requirements have been, and if available, what they use for an annual O&M budget. The survey was limited to those municipalities using either above-ground storage reservoirs or tunnels.

Routine O&M procedures varied among the survey respondents. Those with interim storage reservoirs all required cleaning and flushing. Some systems were set up with automated washdown and flushing systems, such as used at Sacramento's Pioneer Reservoir. These systems have limited labor requirements other than maintenance of the washdown system itself. Others require manual operation of hoses and water cannons for flushing and sediment removal, requiring anywhere from several hours to several mandays per storm event.

Some systems incorporate prescreening. Where screens are used, labor was required to clean and maintain them. There were insufficient data to correlate whether prescreening actually reduced overall O&M costs.

Routine O&M procedures reported for tunnel systems varied. As with the above-ground storage facilities, prescreening is used at some but not all tunnel facilities. Where screens are used, there is a labor requirement associated with their periodic cleaning. Different prescreening configurations were described. One utility reported using a central screening station upstream of its tunnel system, while another used screens at individual drop shaft structures.

The tunnels themselves are subject to accumulation of sediment. Some utilities occasionally cleaned out the sediment while others claimed their systems were self cleaning and needed no manual removal of sediment.

Definitive annual labor, energy, equipment, and chemical costs were provided only by Monroe County, New York, and a copy of that response is attached as Appendix B to this memorandum. Annual expenditures for any year are influenced by the size and frequency of the storms occurring and by the size and configuration of the individual CSO control infrastructure. Many respondents provided data for anticipated labor requirements on a per storm basis, but little data beyond that was available. Specific estimates for the Sacramento system were not developed due to the unknown nature of the system's use.

A summary of the data collected in the survey is presented in Table 9-1.

TABLE 9 - 1

SUMMARY OF O&M SURVEY

Agency/Location	Name of Facility	System Type/ Operation	System Size and Slope	Year Built	Maintenance Required	Equipment Required	Other Comments	Contact Person/ Phone No.
Department of Water and Pollution Control Atlanta, GA	CSO Tunnel	Tunnel Store CSOs in tunnel, pump to storm-weather plant.	34 MG 11,000' x 22'-8" diam. Slope = 0.005	1984	Empty screening's dumpster and clean sediment out of tunnel after each storm. During storms, operator comes over from nearby plant and watches screens.		Clean screening room, 8 man-hours per storm. Occasionally have to clean out sediment in tunnel too.	Roger Turner (404) 241-0116
Bay City Utilities Department Bay City, MI		Five separate basins, different locations. Store CSOs and drain back to sewer, or overflow.	5-6 MG each, 27.5 MG total	Mid 1980's	High pressure pumps flush out basins automatically with river water (5,000 gpm). Pneumatic valves are serviced. No screening of inflow.	Occasionally bring in additional lighting. See note 5.	Wastewater is chlorinated at basins. This causes problems at plant with biological treatment.	Otto Green (517) 893-5121
Massachusetts Water Resources Authority, Boston, MA	Cottage Farm & Prison Point	Six chamber basins at each facility	Cottage Farm 215k gal., Prison Point 200k gal. Pump out to WWTP or chlorinate and overflow.	Early 1970's, Early 1980's	Basins are emptied into storm wetwell after each storm. Wet well is then pumped down and manually hosed down. Sometimes takes two days. Inflow is screened.	None	Sometimes takes 2 to 3 days to pump tanks out to WWTP.	Greg Alessandro (617) 367-8432
City of Chattanooga, Chattanooga, TN		Tunnel. Drain back to sewer, or overflow to river.	3.5 MG - two - 9' x 12' box culverts with 7' high weir wall between. Slope of low-flow channel = 0.00055.	Under construction now.	Anticipate cleaning with skid-loader once per year. System is designed with automatic washdown system (pumped river water and nozzles mounted in ceiling).	None	Tunnel system under construction. No screening of inflow.	Alice Cannolle (615) 757-5026

Agency/Location	Name of Facility	System Type/ Operation	System Size and Slope	Year Built	Maintenance Required	Equipment Required	Other Comments	Contact Person/ Phone No.
Water Reclamation District of Greater Chicago, Chicago, IL	TARP	Deep, large tunnel. Capture 1st "flush", pump back to WWTP, balance overflows to lake.	1.0 billion gal., Diameter 15 to 33', slope = 0.0005	Mid 1980s	Two operators at pumping station 24 hours/day. $3.6 annual maintenance budget. See note 4.	Odor controls for 7-mile forcemain include liquid nitrogen and hydrogen peroxide.	Drop shafts receive alot wear and tear. 300' deep electrical power for pumping system out is major operating cost.	Alan Crowther (708) 780-4003
Storm Water Management Utility, Metropolitan Sewer District of Greater C., Cincinnati, OH		None					Looking at a tunnel system. (Predesign)	Marty Umberg (513) 244-1380
Division of Sewerage and Drainage, Columbus, OH	Whittier St. Storage	Storage Tank. Effluent drains to river, or interceptor.	3.8 MG, 3 tanks. Each tank 180' x 105' x 9'.	1940s	Wash down tank after use, takes approx 16 manhours. Wash down with river water (only one pump and hose,slow).	None	Traveling bridge for sludge collection. Can drain to river or sewer. See note 3.	Dick Morris (614) 645-7757
Decatur Sanitary District, Decatur, IL	McKinley & Seventh Ward	First flush storage tanks, and two swirl concentrators to remove solids before discharging CSOs to creek.	McKinley is 600k gal. Seventh Ward is 1.5 MG.	McKinley 1986. Seventh Ward June 1990.	Tanks are pumped down and manually flushed out with ground water at McKinley, and well water at Seventh Ward using fire hoses and water cannons. Swirl concentrators must be cleaned, also. Cleaning takes 3-8 manhours.	None	Spray headers did not work so they were removed. Water cannons work great. (City recommends lots of hose bibs.)	Charlie Boland (217) 422-6931
City of Grand Rapids, Grand Rapids, MI		Holding basin after primaries at WWTP, chlorinate overflow.	11 MG	1974	Manually flush basin 2-3 times/year. Preventative maintenance required on screw pumps.	None	Basin accepts overflow from treatment plant after receiving primary treatment. See note 6.	Darrell Smith (616) 456-3628
Metropolitan Sewage District, Milwaukee, WI		No CSO facilities currently.					Deep tunnel system under construction.	Steve Graef (414) 225-2088

Agency/Location	Name of Facility	System Type/ Operation	System Size and Slope	Year Built	Maintenance Required	Equipment Required	Other Comments	Contact Person/ Phone No.
Department of Water and Sewerage Services Nashville, TN		Equalization Basin at WWTP.	10 MG		Wash down equalization basin after use.		Currently designing deep tunnel plus storage chambers	Jim Tarpy (615) 862-4503
East Bay Municipal Utility District, Oakland, CA	Oak Port Facility Treatment Plant	Storage tank, at grade. Pump to tank, drain back to plant.	3.0 MG (another 11-MG tank under construction.)		Hose down after each storm, takes approx. 16 manhours.	None	Pumping station lifts CSOs to tanks. Pumping station has a coarse bar screen manually cleaned.	Marilyn Miller (415) 287-1661
NY City Dept. of Environmental Protection, Queens, NY		Storage basin (gravity return to plant) and in-line storage.	Basin is 10 MG. In line storage is 20 MG. Slope = 0.0025	1972	Last 3 MG in basin is pumped out and the basin cleaned about once a week. Cleaning takes approx. 8 hours.	None	Problem with H_2S. An open building might be better. Turning vanes in the lines may prevent settling at high flows.	Joe Silverstein (718) 235-8080
City of Richmond, Richmond, VA	Shockoe	Retention basin (covered tank). Drains by gravity back to plant.	35 MG + 10 MG in sewer	1987	2-3 operators at basin 24 hours/day. Also have ozone equip. for odor control, but not used. Clean screening room.		Considering adding swirl concentrators at treatment plant. Need better washdown system. Inflow to basin has auto screening.	David Van Gelder (804) 780-5363
Rochester Pure Waters District, Rochester, NY		Deep Tunnel System. 10' to 16' diameter, stores CSOs then pumped to enlarged WWTP.	176 MG. Slope = 0.0001 (Additional info. will be mailed.)	1988	Inspect tunnels 4 times per year. One tunnel has washdown nozzles built in (River Water) but system is not needed.	Special "inspection vehicles" used to tour tunnel.	Partially complete, one phase in use for 3 years. See note 7.	Sean Murphy, John Graham (716) 274-7724

Agency/Location	Name of Facility	System Type/ Operation	System Size and Slope	Year Built	Maintenance Required	Equipment Required	Other Comments	Contact Person/ Phone No.
City of Saginaw Wastewater Treatment Div., Saginaw, MI		Pump to storage tanks. 4 bays, two parallel trains, primary, secondary. Overflow to river.	3.5 MG	1978	Washdown with river water, use spray nozzles and water cannons. Wash solids to WWTP. See note 8. Considering adding tipping trough for cleaning tank bays.	None	Currently designing and building 6 additional tanks with 25 MG storage. No screens on inflow, use over/under weirs. Baffles on exit trap floatables.	Jim Anderson (517) 759-1630
Bureau of Water Pollution Control, San Francisco, CA	5 different systems.	Tunnel and box storage system. Store CSOs, pump out or overflow to deep ocean outfall.	West System: 50 MG. East System: 50 MG. Up to 17' x 24' Slope = .002	1986	No regular maintenance program. Rains flush out storage structures. (Self cleaning)		Annual power cost for pumping system out is approx. $4.5 million. Tunnels built with low flow channels. See Note 1.	Lou Vagadori (415) 648-6882
City of Seattle Drainage and Wastewater, Seattle, WA		Two tanks and several oversized sewers. Store CSO, drain to sewer.	Tanks are 1 MG each.	Early 1980's	Hose down tanks and wash bottom (takes approx. 240 manhours per cleaning). Hose down in-line storage pipes once a year. Vortex valves checked during storm and cleaned after storm. Solids settle when flows held by vortex valves. See Note 2.	Vacuum truck, self-contained air supply (SCA), lights, blowers, & gas monitors.	Newer in-line storage pipes have built-in bypass to drain. Sprinkler/washdown system in storage tanks has been removed (did not work).	Kevin Clark & Mel Labay (206) 684-7571 (206) 386-1230
Municipality of Metropolitan Seattle, Seattle, WA	Logboom Park Regulator Station	In-line storage.	4 MG (2 - 11' pipes) Slope = 0.001	1988	None required; system has automatic flushing system (high velocity when draining).			Bill Berry (206) 684-1800
Department of Public Works (Wayne Cnty), Southgate, MI	Southgate Wyandotte Basin	Storage tank. Store CSOs, pump back to plant.	35 MG	Early 1980's	Basin is flushed with automatic spray system. Preventative maintenance required on aeration system and screens. Screenings are hauled to the plant.			Tony Pitoniak (313) 285-5500

Agency/Location	Name of Facility	System Type/ Operation	System Size and Slope	Year Built	Maintenance Required	Equipment Required	Other Comments	Contact Person/ Phone No.
Metropolitan St. Louis Sewer District, St. Louis, MO		Small detention basin acts as in-line storage.	Negligible		None	None	CSOs are not controlled.	Jim Schmid (314) 768-6200
City of Toledo Dept. of Public Utilities, Toledo, OH		Tunnel. 60' deep. Store 1st flush only, pump this volume to plant, balance overflows.	5.5 MG (13.5' dia. x 1 mile long) Slope = 0.001	July 1990	Clean intake screens takes approx. 8 manhours after each storm. City has 3 full time (8 hrs/day) operators to cover 34 regulators and 3 CSO pumping stations.	Vacuum Truck.	Drop shaft screens clog with debris when it rains after long dry spell. Screens have 4" square openings. Elec pumping cost is approx. $10k/year.	Don Romes (419) 724-3861
Department of Environmental Facilities, Westchester County, NY	North Yonkers Pump Station	Three swirl concentrators	27 MG each	1991	Concentrators are hosed down after each storm. (Note: solids are pumped to plant)	None	Overflow from pumping station is concentrated, chlorinated and discharged. Solids are sent to plant.	Al Tramelli (914) 285-2477

NOTES

1. San Francisco - Portions of the Westside Boxes were constructed with decanting walls in the middle of the box shaped tunnels. This allows solids to settle out in the first chamber, as well as retention of floatables. These solids are flushed out after storms and pumped to the treatment plant. The decanted flow is stored and treated, or allowed to overflow to the SW Ocean Outfall (4.5 miles long). Lincoln Way Storage Box: Decanting wall is 1,100 feet long, Vicende Way Storage Box: decanting wall is 920 feet long.

2. Seattle - City of Seattle budgets 0.25% of capital costs when estimating annual maintenance costs. City stated that the in-line storage pipelines are much easier to clean than their storage tanks. Another advantage with in-line storage pipes is that they are mostly built in streets (as opposed to tanks, which are usually built on a lot with private neighbors) and thus are less likely to have sensitive neighbors.

3. Columbus - Recommends covered storage tanks to minimize odor problems. The City is considering adding "hydraulic flushers" at one end of each tank to automatically flush out solids.

4. Chicago - Operators try to pump down stored CSOs only after 10:00 pm to save demand charges and utilize cheaper power rates. The 1 billion gal. storage capacity is to store the first flush only, this system provides no flood control. Second phase, storage reservoirs, will be constructed later.

   A detailed annual maintenance budget will be sent to us.

5. Bay City - Chlorine is sometimes used to control odors from one of the storage tanks that was constructed under a park.

6. Grand Rapids - Overflow basins also work well to isolate industrial spills.

7. Rochester - The tunnel had screens for the inflow, but these were a real problem to keep clean, and to dispose of the screenings, so the Utility removed them. For additional information regarding the county's budgets, see attached.

8. Saginaw - Existing storage tank is divided into 4 bays. Two bays fill in parallel, with one inlet slightly higher than the first. Both of these bays then overflow into second set of bays, via a transfer channel. This provides some settling of solids. Baffles on outlet weir traps floatables.

APPENDIX A

APPENDIX A

MEMORANDUM FROM BRIERLEY AND LYMAN

REGARDING DEWATERING COSTS

MEMORANDUM

TO: Greg Jones,
HDR Engineering, Inc.

FROM: Alan Howard and Gary Brierley,
The Brierley & Lyman Division of Haley & Aldrich, Inc.

SUBJECT: Sacramento CSO Control Phase 3 Study-Construction Dewatering Analysis and Cost Estimate

DATE: November 21, 1991

COPIES: File

INTRODUCTION

The purpose of this memorandum is to provide you with planning/study level conceptual cost estimates for construction dewatering for the referenced project. Based on your letter request of November 12, 1991, the Brierley & Lyman Division of Haley & Aldrich, Inc. (Brierley & Lyman) has evaluated conceptual construction dewatering requirements for three construction alternatives and developed associated conceptual level cost estimates for dewatering. These analyses were based on existing information from the CSO Control Phase 2 study and additional information provided by you in your recent letter. Generalizations were made about project subsurface conditions and anticipated construction methodology, so our analyses are not site-specific.

We understand that updated conceptual cost estimates for construction dewatering are required for three construction alternatives consisting of: 1) common sewer trench excavations to depths up to approximately 35 ft below ground surface using trench box construction methods; 2) an interim storage reservoir to be constructed near the UC Davis Medical Center site which would consist of an approximately 400 x 500 ft reservoir constructed to depths up to 25 ft below ground surface; and 3) drop shafts for CSO diversions into deep storage tunnels.

The remainder of this memorandum includes a more detailed description of the three construction alternatives that require dewatering and a discussion of generalized subsurface soil and ground water conditions assumed for project construction. Also provided are a summary of conceptual level dewatering requirements such as anticipated steady state ground water withdrawal requirements and anticipated well spacing and staging to achieve dewatered construction conditions, and cost analyses associated with these alternative dewatering scenarios.

PROJECT DESCRIPTION

Specific alternative projects for flood and CSO control were previously evaluated for the City of Sacramento. Comparisons among these alternatives were made based on estimates of their respective project costs. The City of Sacramento has requested an update of the previously developed construction methods and unit costs and Brierley & Lyman has been asked by HDR to develop updated costs for construction dewatering associated with three viable alternatives still under consideration for flood and CSO control for the City.

One type of control infrastructure consists of constructing parallel sewers. For present purposes a system of parallel sewers varying from 12 inches to 144 inches in diameter, at corresponding depths of 10 to approximately 35 ft below existing ground surface, are envisioned to supplement existing sewers within the City of Sacramento. Construction below the water table is assumed to be accomplished by construction dewatering plus trenching using a trench box or trench shield, with trench widths from three feet up to 16 ft to allow two feet of working space either side of the installed pipes. Construction dewatering for this alternative would be accomplished by installing a series of continuous ground water dewatering wells or extraction well points and by phasing construction such that construction dewatering would advance ahead of construction such that several days' to one week's trenching production would be dewatered in advance of the trench excavation. Construction dewatering would thus proceed ahead of trench excavation and some dewatering equipment could be leap-frogged ahead of completed sections of trench. For this analysis we have evaluated conceptual construction dewatering requirements and cost per thousand linear feet (KLF) of trench and assumed that each KLF of trench would require construction dewatering for a period of approximately one month.

The second construction project requiring dewatering is the proposed interim storage reservoir being considered for the UC Davis Medical Center site. The UC Davis storage reservoir has a proposed capacity of approximately 26 million gallons and consists of an approximately 3.5 million cubic foot semi-rectangular storage facility as shown in figures provided to us. The structure is approximately 400 x 500 ft in plan view and varies in depth from approximately 13 to 25 ft. The average depth of the structure as shown is approximately 18 ft deep. No adjacent structures occur within approximately 180 ft of the proposed storage reservoir.

Construction of the reservoir may require construction dewatering that must be maintained throughout excavation and final construction activities for a period up to approximately 18 months. Therefore, construction dewatering is considered by Brierley & Lyman to be crucial for this alternative. The implications for a failure in the construction dewatering system are severe due to uplift potential of the semi-completed or completed and not yet filled storage reservoir due to hydraulic pressure of an elevated water table over a large area. For our discussion below we have assumed a simplified 400 x 500 ft structure approximately 20 ft deep that would require construction dewatering to a depth of approximately 25 ft below existing ground surface.

The third flood and CSO control infrastructure alternative consists of deep storage tunnels. Based on our previous technical memorandum on the deep tunnel alternative we envision slurry shield or earth pressure

balance tunneling procedures given the subsurface conditions beneath the City of Sacramento. Tunnel access shafts, however, will require construction dewatering which comprises the third dewatering alternative we have evaluated for this memorandum.

For the purpose of this analysis we have assumed maximum 40 ft diameter circular access shafts to be constructed by dewatering to approximately five feet below an invert depth of 50 ft followed by excavation and support using conventional sheeting or ribs and lagging. Alignment maintenance hole shafts would likely be 15 to 20 ft in diameter. Construction dewatering for the access shafts must be maintained throughout construction to provide continuous safe access to the tunnels and we have assumed a construction period of 18 months to two years plus perhaps four to six months of initial dewatering in advance of construction. Maintenance shafts would likely require dewatering for only about five months. Our analysis below is on a per shaft basis for each type of shaft since the tunnel configuration and number of shafts is unknown.

SUBSURFACE CONDITIONS

Subsurface conditions assumed for construction of the three flood and CSO control measures outlined in this memorandum are based on those developed in our previous study for deep tunnels for the Sacramento CSO project. Subsurface soil conditions consist of approximately 15 ft of fine-grained overbank silts and clays overlaying interlayered sand, silty-sand, silt and clay to a depth of approximately 100 ft below which finer grained clayey soils occur. Nearer to river locations gravelly layers sometimes occur at depths between approximately 20 and 40 ft below ground surface. The water table in the vicinity of the project varies from approximately 10 to 45 ft below ground surface and has been assumed to exist uniformly at about 15 ft below ground surface for the purpose of the analyses provided herein. Since all of the construction alternatives require construction at depths less than 100 ft we have assumed that the interlayered sands silty-sands, silts and clays from 15 ft below ground surface to approximately 100 ft below ground surface have a uniform hydraulic conductivity of a fine sand to silty sand of approximately 1×10^{-3} cm/sec. We have further assumed for the purpose of hydrogeologic analyses that the alluvial deposits behave as an unconfined aquifer the base of which is at a depth of approximately 100 ft where more clayey soils are encountered.

The following hydrologic parameters were assumed for the dewatering calculations performed to make our analyses:

- Hydraulic conductivity (K) = 1×10^{-3} cm/sec $\approx$ 2.8 ft/day
- Aquifer thickness (b) $\approx$ 85 ft
- Specific yield (S_y) $\approx$ 0.2 (Fetter, 1988)

DEWATERING REQUIREMENTS

Assuming a uniform depth to water of 15 ft precludes dewatering concerns for trenching for parallel sewers to depths less than 15 ft. Trench dewatering requirements will not vary significantly with depth for the proposed deeper sewer trenches for the Sacramento CSO project. Drilling and installation requirements on a per well basis will be about twice as much for well materials and pump capacity for dewatering to a depth of 40 ft for a 35 ft deep trench versus dewatering to 25 ft for a 20 ft deep trench, but overall mobilization, labor, equipment and operation costs will be similar. Since the layout of parallel sewer dimensions, depths and length are unknown, we have provided an analysis of the worst case for a 35 ft deep sewer line on a unit basis for 1,000 LF of trenching.

Steady state inflow to sewer trenches, given all of the assumptions outlined above, are estimated to range from approximately 250 to 500 gpm per KLF of trench for trenches ranging from 20 to 35 ft deep. To accomplish trench dewatering, we estimate the use of line dewatering wells at 100 ft spacings along the trench. Allowing for well inefficiencies and interfering cones of depression, we estimated the use of 50 to 100 gpm capacity wells operated for an approximately one month long period per KLF of trench. Drawdown to a depth of 40 ft at the dewatering wells would accomplish dewatering to approximately 35 ft at the midpoints between the wells.

Dewatering of the proposed UC Davis storage facility will require long term dewatering of an approximately 200,000 square foot area. We have assumed that dewatering of this area will require lowering the water table approximately 10 ft; from 15 ft to 25 ft below existing ground surface. Given our hydrogeologic assumptions, steady state inflow to the dewatered excavation will be approximately 50 to 100 gpm which inflow could be handled by one of two alternative dewatering configurations consisting of either a series of 10 to 15 shallow dewatering wells spaced evenly around the excavation or approximately four deep higher capacity wells centered within each quadrant of the excavation. Given well inefficiencies and interference, we anticipate the need for either ten to fifteen 10 to 20 gpm wells or four 50 gpm wells, respectively.

Dewatering of a 40 ft diameter tunnel shaft to a depth of 55 ft (five feet below an invert at 50 ft) with the water table at a depth of 15 ft would result in steady state inflow of about 75 gpm to the shaft excavation. Deep dewatering wells would be used to dewater the shaft and maintain a depressed water table during tunnel construction for site access at the main shaft(s). Allowing for well inefficiencies and interference we estimate the need for approximately four 50 gpm deep wells spaced evenly around the shaft.

DEWATERING COSTS

We have made some generalized assumptions about dewatering costs to provide you with unit costs for dewatering the three construction alternatives. Dewatering costs include drilling, installation, equipment, labor, supervision and operation. Costs are incurred by both the dewatering subcontractor and the general contractor during both the installation and operation phases. Furthermore, contingencies must

be made for site access, utilities, inflation and trade union requirements. Based on available information from dewatering subcontractors and our experience, we have combined these considerations to develop unit costs as discussed below.

For sewer trench dewatering as outlined above, we estimate $50.00 to $75.00/LF of trench for trench section dewatered for a one month period. Deeper trench contracts will be closer to $75.00/LF and the shallowest trenches requiring dewatering may be constructable at dewatering costs at or slightly less than $50.00/LF.

Dewatering of the underground storage facility will require continuous monitoring during the construction period to preclude serious problems for the project caused by dewatering system failure. Hence, operation costs for our assumed 18 month construction schedule are high. We estimate approximately $50,000.00 for the installation of a dewatering system plus an additional $10,000.00 per month for operation and labor costs for a continuously monitored system. Budgeting for approximately $250,000.00 for dewatering should be allowed for this alternative. Considerable construction cost, and perhaps underdrain design and long term permanent dewatering costs, could be saved by siting such a facility above the water table, gaining additional storage through increased area versus increased depth. Conversely design of a superstructure such as a parking facility on top of the reservoir could provide needed dead load for a better design at depths below the water table.

Tunnel shaft dewatering on a per shaft basis is estimated to be approximately $50,000.00 for access shafts and $35,000.00 for maintenance shafts for the configuration of four deep wells outlined above. Since tunneling is a more or less continuous operation, monitoring of the dewatering system would likely be accomplished by tunnel contractor personnel already on-site. We estimate an additional $1,000.00 per month operation and equipment cost to the contractor once the installation is complete. Hence, access shafts open for 24 months would cost approximately $75,000.00, and maintenance shafts open for five months would cost approximately $40,000.00 to dewater.

M05454.017

APPENDIX B

APPENDIX B

TUNNEL COST INFORMATION FROM MONROE COUNTY, NEW YORK

MONROE COUNTY PURE WATERS

COMBINED SEWER OVERFLOW
ABATEMENT PROGRAM
(CSOAP)

The CSOAP Tunnels are considered a part of the Rochester Pure Water District's collection system. This surface collection system consists of 575 miles of combined sewers, 65 miles of sanitary sewers, and 65 miles of storm sewers. Incorporating the tunnels as part of the collection system has two (2) advantages. First, they act as storage vessels to "equalize" flows into the treatment plant. Secondly, they provide unequalled flexibility for operating and maintaining the surface sewers, including trunk sewers, pump stations, and inverted siphons.

In addition to the 34 miles of deep rock tunnel, an Additional Treatment Facility (ATF) was constructed at the Frank E. VanLare Wastewater Treatment Facility (FEV). Average dry-weather flow at FEV is 100 mgd. Prior to the construction of ATF peak wet-weather flows at FEV reached into the low 300 mgd range. Now, peak wet-weather flows of 600 mgd are possible, with rates in the mid 250 mgd range receiving secondary treatment. The balance of flows receive preliminary treatment and chlorination, and are dischsrged approximately three (3) miles into Lake Ontario. Of course, plant conditions, including equipment serviceability, are major factors in determining the degree of treatment at the various rates.

A portion of the CSOAP system has been in operation since 1985; the Culver-Goodman Tunnel (Officially designated as CSOAP Phase I, but known within the operations group as the Eastside Tunnel System). This system has "captured" an average, annual storm-related volume of approximately 2.5 billion gallons. The destinction of "storm-related" flows versus CSOs is that storm-related flows include in-line relief, which have the potential of causing basement and street flooding.

The second phase of CSOAP, or the Westside Tunnel System, has been in partial operation since mid-1989. Four (4) major overflow discharge points have been "connected" to completed segments of tunnel and have diverted approximately 1 billion gallons of CSOs in 1989 and 2 billion gallons of CSOs in 1990. In this instance, those flows that were diverted were strictly CSOs destined for discharge into the river. Of the approximately 3.5 and 4.5 billion gallons of storm-related flow collected and conveyed to FEV in 1989 and 1990, respectively, approximately 50% received secondary treatment. Keep in mind that we only have approximately 34% of available storage capacity on the Westside Tunnel System; the remainder becoming available by January 1992 as the last segment of tunnel comes on-line. Along with this segment of tunnel, however, will come seven (7) additional overflow discharge points. While we are anticipating the percentage of CSOs receiving secondary treatment to increase, only time will reveal the full extent as varying rain events hit the area.
With this brief background of the system, the following is a discussion about staffing and costs.

The TUNNELS AND TECHNICAL SERVICES unit (TTS) consists of approximately twenty-five (25) people. A manager (me), a Senior Engineer, an Assistant Engineer, two (2) field supervisors, a CAD

group, a specifications writer, five (5) dispatchers, and ten (10) field technicians. One (1) supervisor and eight (8) technicians are assigned in the Technical Services section, which performs closed-circuit video inspections and flow studies (e.g., I/I). The operations and maintenance of the tunnels is based around the Tunnel section consisting of three (3) people. These include one (1) field supervisor and two (2) operators. The Assitant Engineer splits his time between the two (2) sections, with the majority of time spent with Technical Services. The Senior Engineer spends his time with specification and electrical issues. I attend to all issues, leaning towards the newer (i.e., more recently established) Tunnel section and CAD group. [For clarification of terms, the hierarchy is: Department (Public Works), Division (Pure Waters), Unit (Tunnels and Technical Services, Frank C. VanLare WWTF), Section (Technical Services, Tunnels), Group (CAD, Dispatchers,

The Tunnels section receives assistance for labor intensive tunnel entrances (for internal inspections) from the Tech. Services section. Mechanical maintenance support is provided by a section of mechanics in another operating unit. Finally, operations assistance is provided by the dispatchers and FEV operators who monitor the tunnel system with an extensive instrumentation and control network.

The budget for the Tunnel section over the past two (2) years is as follows:

DESCRIPTION	1991	1992
Salaries (including overtime)	$289,704	$380,555
Major Equipment (e.g., motors)	$7,500	$43,000
Contracted Services, incl. utilities	$200,000	$216,600
Supplies, Minor Equipment, tools	$55,670	$66,320
Employee Benefits	$52,818	$83,425
Vehicle Purchase, Telephone, Insurance	$46,805	$46,888
SECTION TOTALS	$652,497	$836,788

Approximately half of the Contracted Services represents utility costs for several control structures ($132,000 budgeted in 1992) associated with the Westside Tunnel System. Flows from this system are conveyed to FEV via a 7' inverted siphon tunnel. The electric costs represent power necessary to energize hydraulic units for gate control/movement, lighting, and heating costs. Leased telephone line costs are budgeted at $23,236 in 1992. We have undertaken steps to reduce these costs. Leased lines will be replaced by District owned fiber optic lines. Heating requirements at structures are being evaluated.
Flows from the Eastside Tunnel System are conveyed to FEV via a large pump station. These pumping costs have been established at $40/million gallons. Therefore, approximate annual electrical costs for the 2.5 billion gallons of wet-weather flow is $100,000. These expenses are included in the budget for FEV and are not reflected in the above table.

Major pieces of equipment were procured along with the capital construction expenses. Examples of equipment include: several pickup trucks and service vans, dump truck equipped with a hydraulic clam for cleaning grit chambers, two (2) diesel-powered inspection vehicles for use in the tunnel (confined space), and one

(1) large, trailer-mounted and two (2) small, dolly-mounted portable hydraulic power packs.

Generally, our major equipment purchases out of operating funds have been minimal; hydraulic attachments for tunnel vehicles, computers, etc. This year, however, we are including funds for the fiber optic project we plan on undertaking.

Contracted services consist mainly of crane rentals for tunnel inspections ($15,000), auxilliary electrical generator preventive maintenance ($7,000), electrical conduit installations for monitoring sites ($15,000), professional engineering services ($20,000), and contracted maintenance on small equipment and fixed gas detectors ($3,500).

All preventive maintenance and corrective maintenance on mechanical and electrical equipment is performed with in-house staff. Solids depositions within the tunnel are removed through operational means (e.g., inducing scouring velocities) or with skid loaders (with our staff).

Minor equipment and tools encompass frames and covers, locking devices, heaters, and dehumidifiers for underground vaults, trunk mounted and portable two-way radios, meters, parts for conveyors and bar screens, small motors (aggregate value of about $45,000), safety equipment (ropes, harnesses, clothing, gloves, boots, hard hats, etc. valued at $13,000), fuel, oil, grease, misc. chemicals, de-icer, cleaning fluids, etc. ($6,000), landscaping supplies and other items for visual upkeep of structures (e.g., paint/grafitti removal) ($2,000).

C:\WP50\TUNNELS\CSOAP.HDR

MONROE COUNTY

DEPARTMENT OF PUBLIC WORKS
DIVISION OF PURE WATERS

THOMAS R. FREY
COUNTY EXECUTIVE

THOMAS A. LOW
DIRECTOR OF PUBLIC WORKS

JOHN E. GRAHAM, P.E.
ADMINISTRATOR
(716) 274-7700
FAX (716) 274-7734

November 26, 1991

Mr. Arthur Fleming
HDR Engineering
Suite 200 C
11225 Southeast 6th Street
Belvien, Washington 98004

Dear Mr. Fleming:

I am sorry Sean was not able to personally respond to your technical request concerning our tunnel system and its current operation, but I will attempt to address your questions from an Administrator's viewpoint.

I am enclosing an engineering data sheet that was developed to address tunnel diameters, volumes and construction costs in which your client would surely be interested. These dollar numbers reflect construction activity that occurred through the 1980's and will have to be adjusted for your construction schedule.

I am also enclosing a budget work sheet that was taken out of Monroe County's 1991 annual budget report. I highlighted the tunnel section which represents the staff and dollar amounts currently budgeted for the entire tunnel system maintenance and operation. If it isn't evident, the tunnel system is a gravity operation which means there will be no large utility expenditures or mechanical maintenance problems. Flows are controlled with hydraulic gate systems that were supplied by Rodney Hunt.

Also enclosed are two public relations articles that were written by Rodney Hunt and Rotork which describe and overview of our tunnel system and its impact to the City of Rochester.

When your firm and client are closer to developing an operational program, please feel free to call or visit us which I think would be well worth your while since we are one of the few agencies that actually "operate" a tunnel system.

Sincerely,

John E. Graham

John E. Graham, P.E.
Administrator

JEG:sh
Enclosures

COMBINED SEWER OVERFLOW ABATEMENT PROGRAM

TOTAL

TUNNEL	DIAMETER (FT.)	LENGTH (L.F.)	VOLUME ($X10^6$ $FT.^3$)	CONSTRUCTION COST
Cross-Irondequoit	16	32,000	6.4	$ 37,400,000
Genesee River Interceptor Southwest	16	8,300	1.7	19,800,000
Culver-Goodman	16	30,500	6.1	95,900,000
			14.2	$153,100,000
Lyell Avenue Contract No. 1	12	10,800	1.2	$ 15,600,000
Saxton-Colvin and Jay-Arnett Contracts Nos. 2 and 3	10	18,500	1.5	34,200,000
Lake Avenue Contract No. 4	14	12,100	1.9	41,500,000
St. Paul Boulevard Contracts Nos. 5 and 9	7	16,100	.6	51,000,000
Tiger-Carlisle System Contracts Nos. 6, 7 and 7A	14	14,500	2.3	39,600,000
State - Mt. Hope Contract 8	14	8,600	1.4	37,900,000
Seneca-Norton Contract No. 12A	12	4,500	.5	12,000,000
			9.4	$231,800,000

+10%

+10%

ADD 10-12%

FOR INSPECT

DESIGN SLOPE .01%

1 SET OF SCREENS DESIGNED + LATER REMOVED (SEE PHOTOS)

DEPARTMENT: Public Works (84)
DIVISION: Pure Waters (85)
SECTION: Tunnels and Technical Services (30)

SECTION DESCRIPTION

This section inspects, improves, operates and maintains the County tunnel and overflow systems. These systems improve the water quality of the Genesee River, Irondequoit Bay and Lake Ontario, and relieve localized backups in basements in various areas of the City.

Engineering support is also offered by this division to the Rochester and Gates-Chili-Ogden Collection Systems. Expenses are charged to each system on the basis of actual use as determined from payroll and equipment records.

BUDGET SUMMARY	Amended Budget 1990	Proposed Budget 1991	Adopted Budget 1991
APPROPRIATIONS			
Personal Services	490,055	634,016	634,016
Equipment	29,500	12,000	12,000
Expenses	221,560	238,560	238,560
Supplies and Materials	76,870	90,370	90,370
Employee Benefits	96,379	115,529	115,529
Interfund Transfers	260,093	205,325	205,325
TOTAL	1,174,457	1,295,800	1,295,800

REVENUE

Tunnels is supported by revenue from the Rochester Pure Waters District. Technical Services charges each district where work is performed.

APPENDIX M

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 6

IMPROVEMENT PLANS

Greg Jones
Judy A. Zavadil
Koos Prins
Paula M. Arsenault
HDR Engineering, Inc.

August 13, 1992

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM

IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 6

IMPROVEMENT PLANS

Prepared by

Greg Jones
Judy Zavadil
Koos Prins
Paula Arsenault

HDR Engineering, Inc.
El Dorado Hills, CA

July 1, 1992

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EXECUTIVE SUMMARY

A. INTRODUCTION

Improvement plans were developed for the City of Sacramento's Combined Wastewater Collection and Treatment System (CWCS). These plans were designed to improve the existing level of flood and outflow protection and to provide two levels of combined sewer overflow (CSO) control.

The major facilities and improvements included in these alternative plans are tunnels, conveyance improvements, storage reservoirs, primary treatment plants, separation of combined sewers, rehabilitation of existing facilities and outflow control by inflow regulation. For the sewer separation alternatives, one involves constructing a new sanitary system (with the existing combined system piping serving as storm drains). The other involves installing a new storm drain system.

Except for the sewer separation alternatives, all other alternatives assume the existing combined sewer system will continue to be operated as a combined system. The Inflow Control Alternative and the Baseline Rehabilitation Alternative, as developed in this report, are not complete improvement plans in that they do not provide for a level of flood and CSO control commensurate with the other plans. The Inflow Control Alternative provides for control of outflows by retention of floodwater in the streets. The Baseline Rehabilitation Alternative simply rehabilitates the existing CWCS infrastructure to assure continued reliability in the future with no provisions for improved flood, outflow or CSO control.

A series of alternative plans was developed to provide flood control for three levels of design storm recurrence intervals: 2, 5, and 10 years. A 25-year storm frequency was also considered for the tunnel alternative only. CSO protection was provided to control design storms with frequencies of 1/year and 5/year. For the purposes of this study, design storm frequencies were assumed to correspond to CSO frequency, as per the City's direction for this study.

Proposed tunnel, storage, treatment and separation alternative improvement plans were checked for compliance with these design criteria via computer modeling

using the City's urban runoff model, the 1991 version of the Sacramento Storm Water Management Model (SSWMM91). Estimates of total project costs were prepared for each alternative. Least-cost alternatives were identified for each of the 2-, 5-, and 10-year flood control design storm events.

B. ALTERNATIVE DESCRIPTIONS

Flood protection was provided by either upsizing the existing combined sewer conveyance system or by using weirs to divert the peak flows from the existing sewers into a new network of consolidation conduits. These conduits would deliver combined wastewater to tunnels, storage, or treatment facilities, depending upon the alternative being considered.

The analysis assumed full use of the existing flood and CSO control facilities including Pioneer Reservoir, Pioneer Interceptor, the CWTP and 60-mgd of City Interceptor and Sacramento Regional Wastewater Treatment Plant (SRWTP) capacity allocated for the combined system. The SWMM91 model includes modifications to simulate the operation of these facilities according to the CWCS original plan of operations.

For the tunnel alternatives, the tunnel system alignment considered for this study is similar to that described for the Phase 2 study. The total tunnel length is 49,200 feet, with diameters considered ranging from 12 to 20 feet, depending on the design storm and conveyance requirements. For the 2-year storm, tunnels were not cost effective. Two tunnel outlet pumping structures were required for all tunnel alternatives, one near Sump 1A and one near Sump 2. Analysis of the tunnels assumed that excess wastewater, above the minimum containment of the CSO design storm volumes, could be discharged to the Sacramento River during the storm event.

Least-cost tunnel alternatives were selected based on the lower capital cost for one of two conditions: 1) retention of maximum storage in the tunnels (i.e. pumping only the minimum amount of water out of the tunnels to the river during the storm event while still providing flood control); and 2) pumping all flow entering the tunnels to the river except that volume required for CSO control.

The recommended tunnel systems for controlling the 5-year flood control design storm, for both the 1/year and 5/year CSO control design storms, are configured for retention of maximum storage. The recommended 10-year tunnel systems for both 1/year and 5/year CSO control are configured for retention of only the volume required for CSO control. The 25-year tunnel is configured for maximum storage retention.

Storage facilities considered were similar in design to the existing Pioneer Reservoir. Three different storage sites were considered: the Pioneer Reservoir site, the Western Pacific Railroad (WPRR) site and the UC Davis Medical Center site. Combinations of these sites were used for both the storage and treatment alternatives. As with the tunnel alternatives, flood control conveyance improvements consisted of upsized sewers and consolidation conduits. For the Pioneer Reservoir and WPRR sites, storage facilities were sized for CSO control, with flood control provided by conveyance directly to the river. The UC Davis Medical Center site was only used for flood control storage. A cost comparison was completed for storage of excess flows (i.e. flood control versus minimum CSO control requirements) versus constructing pumping/conveyance facilities to the river. It was determined that pumping to the river from the WPRR and Pioneer Reservoir sites would be less costly than construction of storage-only for excess flows.

In the treatment alternatives, primary treatment and disinfection was provided for CSO control, with flood protection provided by stormwater conveyance to the river from either the WPRR or Pioneer Reservoir sites. Construction of primary treatment facilities was evaluated for four sites: WPRR, Pioneer Reservoir, Meadowview and SRWTP. The latter two sites were not cost effective. The analysis of the remaining two sites included consideration of storage, with equalization basins used for attenuation of treatment plant peak flows. An optimization procedure was used to identify the least-cost combinations of storage and treatment to be used for the cost analysis.

Two sewer separation alternatives were analyzed. One was the construction of new storm drainage and the other a new sanitary sewer system. The storm drainage system was sized for a 10-year storm event.

In response to a request from the Regional Water Quality Control Board, the feasibility of outflow control via inflow regulation was reviewed. In this alternative, flow regulators are installed in drop inlets or manhole catch basins. These regulators limit flows into the system to its capacity in a gravity/open channel flow condition, thereby reducing the potential for outflows. Stormwater that does not enter the sewer system however, will add to ponding in the streets. A complete outflow control alternative therefore must also include street modifications to provide controlled detention and conveyance of stormwater. The Village of Skokie, Illinois is presently implementing this type of alternative. A combined sewer improvement program incorporating these features may potentially provide desired levels of CSO and outflow control, while allowing controlled flooding, at significantly less cost that conventional storage and tunnel alternatives. Because the design criteria for this type of system are relatively unknown, however, further study of flow regulation for outflow control and street detention and conveyance is recommended.

C. RESULTS OF ANALYSIS

The significant findings for the technical memorandum are the results of the analysis of alternative improvement plans, including the estimates of total project costs.

Table 1 summarizes the most technically viable improvement plans for the separation alternatives, the Baseline Rehabilitation Project, and the 2-, 5-, 10- and 25-year flood control design storms. The 5/year CSO control alternatives for the storage and treatment alternatives did not require any new storage or treatment facilities for CSO control. For these alternatives, flood control is provided by conveyance, including upsized sewers and new pumping facilities to discharge combined wastewater to the river.

The table also identifies the least-cost alternatives for each level of CSO and flood control design storm, and for the two separation alternatives. These are indicated by the shadow boxes in the table.

TABLE 1
SUMMARY OF ALTERNATIVE IMPROVEMENT PLAN ESTIMATED COSTS ($ MILLION) (4)

ALTERNATIVE DESCRIPTION	OTHER	DESIGN STORM CRITERIA						
		2-YEAR FLOOD CONTROL		5-YEAR FLOOD CONTROL		10-YEAR FLOOD CONTROL		25-YEAR (1)
		1/YR CSO	5/YR CSO	1/YR CSO	5/YR CSO	1/YR CSO	5/YR CSO	FLOOD CONTROL
TUNNEL ALTERNATIVES:								
TUNNEL SYSTEM				$528	$525	$738	$734	$978
STORAGE ONLY ALTERNATIVES:								
WPRR		$416				$850		
WPRR AND PIONEER		$335						
WPRR AND UCD						$911		
TREATMENT ONLY ALTERNATIVES:								
UPSIZED CWCS WITH TREATMENT AT PIONEER (6)		$300						
STORAGE/TREATMENT ALTERNATIVES:								
WPRR		$416		$647				
WPRR AND PIONEER		$335		$616		$827		
WPRR AND UCD				$672				
UCD AND PIONEER				$653		$886		
WPRR, UCD AND PIONEER				$668		$863		
UPSIZED CWCS WITH STORE/TREAT AT WPRR (6)		$418						
CONVEYANCE-ONLY ALTERNATIVES:								
WPRR			$270		$573		$760	
WPRR AND PIONEER			$270		$551		$757	
UPSIZED CWCS CONVEYANCE SYSTEM (6)			$250					
CONVEYANCE/STORAGE ALTERNATIVES:								
WPRR AND UCD					$657		$882	
UCD AND PIONEER					$663		$896	
WPRR, UCD AND PIONEER					$652		$865	
SEPARATION ALTERNATIVES:								
NEW STORM DRAIN SYSTEM (2)	$936							
NEW SANITARY SEWER SYSTEM (3)	$293							
BASELINE REHABILITATION:	(7) $88							

BOLD SHADING INDICATES LEAST-COST PLAN.

NOTES:

(1) THE 25-YEAR TUNNEL ALTERNATIVE ALSO CONTROLS THE 1/YEAR AND 5/YEAR CSO DESIGN STORMS.
(2) THE NEW STORM DRAINAGE SYSTEM IS SIZED FOR CONTROLLING THE 10-YEAR FLOOD CONTROL DESIGN STORM.
(3) THIS ALTERNATIVE PROVIDES MINIMAL FLOOD CONTROL BEYOND THE CAPACITY OF THE EXISTING COMBINED SYSTEM.
(4) COSTS FOR REHABILITATION OF THE EXISTING CONVEYANCE SYSTEM ARE NOT INCLUDED IN THIS TABLE, EXCEPT FOR BASELINE REHABILITATION PROJECT.
(5) THESE ALTERNATIVES DO NOT INCLUDE ANY NEW STORAGE OR TREATMENT FACILITIES FOR FLOOD OR CSO CONTROL IMPROVEMENTS.
(6) THESE ALTERNATIVES USE UPSIZING OF EXISTING PIPES RATHER THAN A CONSOLIDATION CONDUIT NETWORK TO ACHIEVE FLOOD CONTROL CRITERIA.
(7) THIS PROJECT INCLUDES REHABILITATION WORK SPECIFICALLY RELATED TO THE SEPARATE SANITARY SEWER ALTERNATIVE.

SECTION 1
INTRODUCTION

A. BACKGROUND

The City of Sacramento's existing combined sewer service area experiences wide-spread flooding for storms exceeding approximately a 2-year return period. Flooding occurs as a result of several factors: inadequate drainage inlet (DI) capacity, inadequate collection system capacity (local, small-diameter piping), and inadequate conveyance system capacity (pumping stations, laterals/mains, interceptors). When the rainfall exceeds the capacity of the sewers, surcharging occurs in the system. In certain low-lying areas, outflows of combined wastewater have occurred from the collection system onto the streets and into basements during larger rainstorm events. In addition, based on City records kept between January 1986 and October 1989, the City estimates that CSOs to the river occur an average of 4 to 5 times a year. These periodic discharges are referred to as combined sewage overflows or CSOs.

To address the problems of flooding, outflows and CSOs, the City initiated a multi-phased planning study in 1990 to identify a comprehensive long-term upgrade to the existing Combined Wastewater Collection and Treatment System (CWCS). The current study is Phase 3, which was completed with the submittal on July 1, 1992 of the *Phase 3 Detailed Technical Report*.

Phase 3 is an extension of the Phase 2 study, in that a substantial number of additional alternative improvement plans and design sizing conditions are being analyzed. A critical aspect of the Phase 3 analysis has been the improvements made to the Sacramento SWMM source code, which has allowed for the simulation of a variety of complex alternative facilities and operational scenarios. A working version of the EXTRAN block for modeling such facilities was not available during the Phase 2 study. The revised version of the Sacramento SWMM is referred to as the SSWMM91.

The Phase 2 study report, entitled *Phase 2 Detailed Technical Report* and dated July 1, 1991, presented the analysis of improvements to meet the "10/1 Criteria",

i.e. flood control for a 10-year 6-hour storm event and control of CSOs to an average of one event per year. Included in the Phase 2 report was the technical memorandum, *Alternative Improvement Projects and Plan Combinations* (dated July 1, 1991), which provided detailed description of the potential upgrades to the City's CWCS for the 10/1 Criteria. A brief review of that document is included in the following subsection.

B. PHASE 2 IMPROVEMENT PLAN TECHNICAL MEMORANDA

The Phase 1 *Technical Overview Report on the City of Sacramento Combined Sewer System*, dated October 1, 1990, described four generic types of CWCS upgrade programs: storage, treatment, conveyance, and separation. In addition, it listed potential project sites suitable for supporting new CWCS infrastructure. The Phase 2 improvement plan analysis effort started with this listing of potential project sites and applied a screening process. This screening evaluation eliminated several sites on the basis of socio-economic issues and preliminary system hydraulic information.

The project sites remaining after screening were incorporated into comprehensive improvement plans which assumed the candidate project sites would be developed as storage reservoirs, treatment plants and/or pumping stations. These main facilities were also supplemented by a network of conveyance improvements. Each of these comprehensive improvement plans had to be configured to provide a minimum level of flood control for a 6-hour synthetic design storm with a 10-year recurrence interval.

New sewer laterals, mains, interceptors and/or tunnels were identified to connect the new storage reservoirs, pumping stations and treatment plants with the existing CWCS. Improvements common to all alternatives included rehabilitation of existing laterals and mains, an upgrade of the collection system (i.e. sewers smaller than approximately 15 inches in diameter), improvements to Pioneer Reservoir and rehabilitation of Sumps 1, 1A and 2.

Analysis of the performance of each of the proposed improvement plans was originally intended to be completed with the assistance of the Sacramento SWMM

computer model. However, difficulties in simulating the operation of diversion weirs and other facilities in the model limited the use of the model to analyzing the alternatives without diversion weirs. Consequently, a spreadsheet hydrographic analysis method was devised and used as a substitute means of evaluating the performance of each alternative improvement plan.

Unit construction costs were prepared and used to estimate the total project costs of the alternative improvement plans. These were presented in a technical memorandum entitled *Unit Construction Costs for Specific Alternative Projects* (dated July 1, 1991). Total improvement plan costs were compared and a least cost alternative improvement plan was identified for controlling flooding for the 10-year design storm event. The least-cost project identified for the Phase 2 study was the Tunnel Alternative (Alternative "B", according to the Phase 2 identifier system). An affordability analysis was completed for this least-cost plan, the results for which were presented in the *Phase 2 Report*. The estimated improvement plan cost of approximately $575 million was determined to be unaffordable.

C. OBJECTIVE OF THE PHASE 3 IMPROVEMENT PLAN EVALUATION

The Phase 3 study includes in-depth analyses of a wider variety and larger number of alternative improvement projects and plan combinations than the Phase 2 study, and responds to the Regional Water Quality Control Board's comments on the *Phase 2 Detailed Technical Report* and corresponding analyses. Improvement plans have been developed for four levels of flood control, represented by design storms for 2-, 5-, 10- and 25-year return periods, and two levels of CSO control, represented by 1/year and 5/year frequency design storms. This range was used in order to develop relationships between benefits and costs for alternative plans. The Regional Board requested that improvement plan cost estimates be prepared for a storm 30-percent higher in peak intensity than the 10-year storm event. The 25-year storm event is intended to satisfy this request. The Board also requested that alternative improvements directed specifically towards controlling outflows be evaluated, and that collection system improvements costs be re-examined because of the impact of these costs on the overall improvement plan costs.

Both the Phase 2 and 3 studies analyze alternative project configurations that

employ storage reservoirs and pumping stations, tunnels, and treatment facilities. Both consider the performance of these alternatives under the influence of a 10-year synthetic design storm event. Both reviewed unit cost information and estimates of total project costs for each alternative.

However, the Phase 3 study provides more detailed analysis of the development of alternative configurations, based on application of the SSWMM91 model. Simulation of the diversion weirs proposed as part of the conveyance system upgrade is now possible as a result of modifications to the program code. As mentioned, CSO control criteria have been expanded from one to two design storm events. Each alternative plan is reviewed for its flood control performance under at least three different design storms. The least-cost alternative for each design storm, where storage facilities are specified, will be analyzed for its response to long-duration storm events.

In addition, the Phase 3 effort addresses in more detail an independent analysis of the separation alternatives, outflow control and a baseline rehabilitation alternative. The two separation alternatives considered include construction of: 1) new sanitary sewers with use of the existing combined system as a storm drainage system; and 2) construction of new storm sewers and use of the existing combined system as a sanitary sewer system. A separate section of this memorandum (Section 9) addresses the feasibility of an outflow control alternative by means of inflow regulation. And the Baseline Rehabilitation Alternative defined in Section 3 identifies the rehabilitation requirements of the existing system to sustain its reliability.

As with Phase 2, comparisons of the total project costs for these alternatives are presented and the least cost alternatives identified. These total project costs are used in the Phase 3 technical memoranda on *Cost Benefit Analysis of Alternatives* and the *Rate Assessment and Revenue Program.*

D. DEFINITIONS

The following definitions are provided for terms and acronyms used frequently throughout this memorandum.

cfs: An abbreviation for cubic feet per second.

Collection System: The network of sewers less than 16 inches in diameter.

Combined Sewage: Wastewater which is a combination of both stormwater and sanitary sewage.

Combined Sewer Service Area: The area of the City drained by the CWCS.

Combined Sewer System: The entire collection system, plus laterals and mains, and interceptors in the combined sewer service area.

Consolidation Conduits: Proposed new sewers which would receive combined wastewater from diversion weir structures constructed on the existing CWCS.

CSOs: Combined sewer overflows, which are intentional or unintentional overflows of untreated wastewater from the combined sewer system to a receiving water located "outside" of the system.

CWCS: An abbreviation for the City of Sacramento Combined Wastewater Collection and Treatment System. This includes the existing combined sewer system, pumping stations at Sumps 1, 1A and 2, the Flow Control Structure at Sump 2, the Pioneer Interceptor, Pioneer Reservoir, the Combined Wastewater Treatment Plant (CWTP) and the force main interceptors connecting these facilities.

CWTP: An abbreviation for the Combined Wastewater Treatment Plant. This facility provides only primary treatment of combined wastewater.

DIs Drop inlets.

ENR: *Engineering News Record.*

Flooding: Flooding is defined as the inundation of streets, landscape and structures with floodwater.

Floodwater: Floodwater is composed of an uncontained accumulation of ponded stormwater and/or outflows from the combined system.

Interceptors: Sewers which are greater than 60 inches in diameter.

Laterals/Mains: Sewers which are between approximately 15 and 60 inches in diameter.

MG: An abbreviation for million gallons.

mgd: An abbreviation for million gallons per day.

Outflow: As defined in the draft technical memorandum entitled *Analytical Approach to Outflow Analysis*, dated February 13, 1992, outflows are defined as combined sewage, containing raw sanitary sewage and stormwater, which backs up and flows out of the combined system into low-lying areas, streets, or below-grade structures and basements.

Ponding: Ponding is stormwater runoff that stands in the streets either because of inadequate drainage inlet capacity or because the sewer system is too full to accept the rate of runoff entering through the DIs.

Regional Board: The State of California Central Valley Regional Water Quality Control Board.

River: References to "the river" refer to the Sacramento River and are typically in the context of the receiving water for CSO discharges.

Sanitary Sewage: Domestic wastewater, caused by human activity, which is generated from residential, commercial or industrial sources and includes human wastes.

SSWMM91: The version of the Sacramento Stormwater Management Model including all modifications used in this Phase 3 study.

Stormwater: Runoff water from rainstorm events. Its origin is from precipitation only and it is not mixed with any wastewater constituents.

SRWTP: An abbreviation for the County of Sacramento's Regional Wastewater Treatment Plant which provides secondary wastewater treatment.

Surcharge: Surcharging occurs when more water enters the sewers than can be conveyed by the pipes under gravity flow (open channel) conditions. When inflow into the sewers exceeds the pipeline carrying capacity, the sewers pressurize and water levels (hydraulic grade line) in manholes rise above the crowns of the incoming and outgoing pipes.

SECTION 2
COMBINED WASTEWATER COLLECTION SYSTEM INFRASTRUCTURE AND OPERATIONS

A. EXISTING SYSTEM

The City's existing CWCS is described in detail in the *Technical Overview Report on the City of Sacramento Combined Sewer System*, which was submitted to the Regional Board on October 1, 1990, during the Phase 1 study. That document includes descriptions of the major components of the existing CWCS, such as the combined sewer system, the pumping stations at Sump 1A and Sump 2, Pioneer Reservoir, the Combined Wastewater Treatment Plant (CWTP) and the Pioneer Interceptor. Additional description was presented in the *Phase 2 Detailed Technical Report*, and the appended technical memorandum entitled *Evaluation of Reliability and Remaining Life at Existing CWCS Pumping Station Facilities*, both dated July 1, 1991.

A map of the existing combined sewer area and adjacent contributing sanitary sewer basins is presented in Figure 2-1. The limits of the combined sewer system indicated in this figure are coincident with the limits of the modeled study area used in the Phase 3 study.

B. CWCS PLAN OF OPERATIONS

Upon completion of the Pioneer Reservoir and Pioneer Interceptor projects in 1978, a Plan of Operation for the CWCS was prepared for the City. This original Plan of Operation documented the operational strategy for the entire system. Simply stated, the operational strategy provided either treatment or storage of all combined wastewater conveyed by the existing collection system up to the peak capacity of those storage and treatment facilities. Beyond that capacity, any additional inflow of combined wastewater was discharged to the river without treatment.

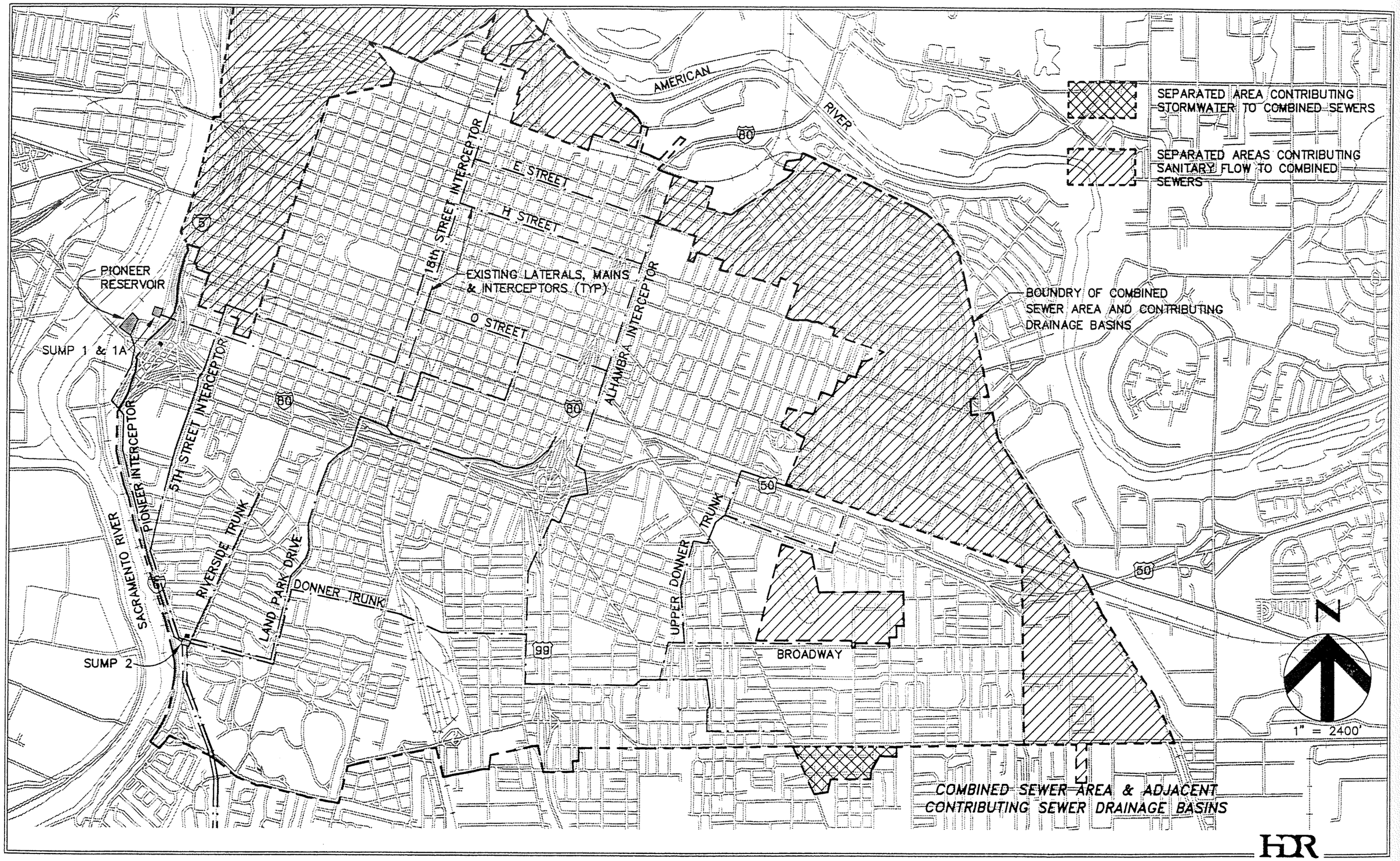

SEPARATED AREA CONTRIBUTING STORMWATER TO COMBINED SEWERS
SEPARATED AREAS CONTRIBUTING SANITARY FLOW TO COMBINED SEWERS
AMERICAN RIVER
E STREET
H STREET
O STREET
19th STREET INTERCEPTOR
ALHAMBRA INTERCEPTOR
EXISTING LATERALS, MAINS & INTERCEPTORS (TYP)
BOUNDRY OF COMBINED SEWER AREA AND CONTRIBUTING DRAINAGE BASINS
PIONEER RESERVOIR
SUMP 1 & 1A
SUMP 2
SACRAMENTO RIVER
PIONEER INTERCEPTOR
5TH STREET INTERCEPTOR
RIVERSIDE TRUNK
LAND PARK DRIVE
DONNER TRUNK
UPPER DONNER TRUNK
BROADWAY
80
50
99
N
1" = 2400'
COMBINED SEWER AREA & ADJACENT CONTRIBUTING SEWER DRAINAGE BASINS
HDR

Figure 2-1

Under the original CWCS Plan of Operation, the sequence of CWCS facilities brought on line during dry weather months was to: 1) send the first 60 mgd of flow entering Sump 2 (and Flow Control Structure) to the Sacramento Regional Wastewater Treatment Plant (SRWTP); 2) send flows over 60 mgd to Pioneer Reservoir; and 3) when Pioneer Reservoir was full, send additional flows to the CWTP. For wet weather months, the original plan of operation deployment sequence was to: 1) send the first 60 mgd of flow to SRWTP; 2) send flows over 60 mgd to the CWTP; and 3) send flows to the Pioneer Reservoir from the downtown area only when the water level at Sump 1A was high enough to trigger operation of the pumps at Sump 1A. Under this scenario, the third pump (of three pumps) at Sump 1A was rarely used, because of its high setpoint for operation.

Operation of the CWCS under the original Plan of Operation did not convey larger rainfall events without causing flooding in, and outflows from, the combined sewer system. The potential for public health hazard posed by outflows caused the Regional Board to initiate regulatory action against the City to upgrade the CWCS. The Cease and Desist Order issued by the Regional Board also included requirements for complying with State and National CSO Strategies.

Key components of the Phase 1 *Technical Overview Report* were the recommended changes to the original CWCS Plan of Operation. These changes were documented in the City's *Interim Plan of Operations for NPDES Permit No. CA0079111*, which was adopted and dated on November 2, 1990. Implementation began on December 1, 1990.

This operational plan will remain in effect until the City can implement a comprehensive improvement plan for the entire CWCS or until the Regional Board modifies the City's current NPDES permit. Under the new Interim Plan of Operations, the goal is to improve flood protection and sanitation for the City. Actions to control surcharging and flooding in the system will also aid in controlling outflows. This is accomplished at the potential expense of increasing CSOs to the Sacramento River, relative to estimated historical frequencies for the existing system. Increased CSOs would occur because CWCS pumping stations and other facilities are being operated, per the adopted interim plan, to draw down the hydraulic grade line in the system as much as possible. An important feature of this strategy is the dedication of Pioneer Reservoir to Sump 1A, which means

that flows are not currently being sent from Sump 2 to the Reservoir. This minimizes hydraulic head on the Sump 1A pumps, which in turn maximizes pumping capacity, thereby also maximizing flood control in the downtown area.

Features of the Interim Plan of Operation are discussed in the Phase 1 *Technical Overview Report* and are summarized below.

1. Pump operation set points in Sump 1A and Sump 2 wet wells were lowered to draw down the hydraulic grade line in the collection system. This reduces the frequency and incidence of flooding in the combined sewer collection system service area.

2. Pioneer Reservoir is filled only from Sump 1A. Flows from Sump 2 are directed only to SRWTP, CWTP and the river.

3. Initial flows from Sump 1A are allowed to pass through the Pioneer Interceptor to Sump 2 to prime the Stage 1 pumps. Once the Stage 1 pumps at Sump 2 are primed, the Sump 1A pumps fill the Pioneer Interceptor. All sections of Pioneer Reservoir are filled evenly once the Pioneer Interceptor is full. The elevation of the overflow gate for spilling to the river from Pioneer Reservoir is lowered, when necessary, to keep the head on the Sump 1A pumps at a minimum and maximize flood control in the downtown area.

4. The necessity for lowering the overflow gates at Pioneer Reservoir during a storm event is determined from water levels measured at key manhole monitoring locations by Rain Patrol personnel. If water levels are rising and reach a pre-determined level, then Pioneer Reservoir water level is lowered to an elevation such that the Sump 1A pumping capacity will be maximized.

C. FUTURE CWCS OPERATIONAL STRATEGY

The future operational strategy for the upgraded CWCS would depend upon whether the existing system will be used as a combined sewer system. If sewer separation

is not implemented, the strategy for future CWCS operations will once again be to minimize the discharges of untreated combined wastewater to the Sacramento River, as long as the design level of flood control is met by the upgraded system. Improvements will be designed to provide the combined sewer service area with a higher level of flood control protection, and will allow the City staff more flexibility to control discharges to the river. Existing storage at Pioneer Reservoir and treatment facilities at the CWTP will be fully utilized without compromising the flood protection of the combined sewer service area.

Conversely, if complete sewer separation is implemented as part of the overall improvement plan, future operations will continue to focus on maximizing the amount of stormwater runoff pumped to the river. With a separated system, this pumped discharge would only be storm drainage.

D. COMBINED WASTEWATER MANAGEMENT

Alternatives for upgrading the existing CWCS will be based on one or more of the general methods of combined wastewater management described below. Sections 5 through 9 present the details of alternatives that correspond to these general management approaches.

D.1. Provide Additional Storage

One direct means of providing control of combined wastewater is the addition of storage facilities. Larger rainstorm events result in flows in the collection system which are very high for a relatively short period of time. Storage alternatives divert these flows from the collection system and convey them to storage. This relieves the collection system of excessive flows and thereby controls outflows. Other benefits can include smaller sized downstream conveyance and treatment facilities. Stored wastewater would be pumped back into the combined sewer system at a controlled rate during off-peak flow conditions following subsidence of storm flows.

Storage can be provided by constructing more reservoirs similar to Pioneer Reservoir. These structures can be configured as above grade or below grade

facilities. Larger sized conveyance conduits and tunnels can also be used as storage facilities as well as for conveyance.

D.2. Improve Conveyance and Treatment Capacity

A second approach to combined wastewater management is to increase the capacity of existing conveyance and treatment systems in order to handle higher wet weather flow rates within the CWCS. This approach improves the ability of the system to handle peak wastewater flows by eliminating restrictions which result in outflows and flooding within the combined sewer service area. The most obvious upgrade in this program would be the replacement of existing sewers with new, larger capacity conduits.

Upsizing pumping stations receiving combined wastewater flows would also be considered if it lowered the upstream hydraulic grade line in the combined sewer conveyance system low enough to avoid outflows and flooding within the service area. Upgrading existing treatment facility capacity or constructing new treatment facilities may be necessary as a consequence of improving the ability of the conveyance system to deliver wastewater flows at higher peak flow rates. Without including facilities to store or treat the additional flow delivered to the downstream points in the system, CSOs would increase.

D.3. Separate Stormwater and Sanitary Flows

Complete or partial separation of the sanitary and storm drainage systems is a direct approach to outflow and CSO control, because it completely eliminates the formation of combined wastewater. This can be accomplished by either building a new sanitary sewer system or a new storm drainage system within the combined sewer service areas. If a new storm drainage system is constructed, the existing combined sewers are reserved for sanitary flow. If new sanitary sewers are constructed, the existing combined sewers are reserved for storm drainage.

This type of program can be extremely disruptive to pedestrian and vehicle traffic due to the lengthy construction period needed to install the new sewer system. New sewers are required to parallel most every sewer in the existing

sewer system. Sewer work would be required in almost every street in the combined sewer service area.

D.4. Inflow Control Program

Inflow control programs employ methods to restrict the flow of stormwater runoff into the combined sewer system. The result is that the capacity of the combined sewers is reserved for sanitary flows plus a limited amount of stormwater, compatible with the hydraulic capacity of the system. Since the amount of combined flow is limited, surcharging is prevented, which prevents the occurrence of outflows from the sewer system.

Stormwater that is not allowed into the combined sewer collection system is either detained in surface impoundments, routed through streets modified to convey increased flows, or routed through separated stormwater conveyance systems. Inflow control programs increase the number of street flooding locations, but minimize the volume and depth of flooding at each location to acceptable levels.

SECTION 3
IMPROVEMENT PLAN IDENTIFICATION AND DESIGN AND ANALYSIS ASSUMPTIONS

A. INTRODUCTION

This section identifies the alternative improvement plans analyzed in the Phase 3 study and summarizes the design and analysis assumptions applied to all of the alternatives evaluated. Assumptions or analytical procedures specific to a particular alternative improvement plan are described in the section dedicated to that improvement plan analysis (Sections 5 through 9). Unit costs used in the estimates of total project costs are discussed in Section 4 of this memorandum.

B. THE SACRAMENTO SWMM PROGRAM

The City of Sacramento's combined sewer system has been modeled with a modified version of the EPA's Storm Water Management Model, SWMM, a computer model used to simulate hydrologic/hydraulic systems, including conveyance, storage and treatment facilities.

The Sacramento SWMM computer program, SSWMM91, consists of two sub-programs that are executed separately. The first program is called the RUNOFF block. The RUNOFF block has the capability of simulating runoff from defined drainage basins, given a precipitation event and the physical parameters of the drainage basins such as the basin area, basin width, impervious percentage, and infiltration capacity.

The second program is called the EXTRAN block. This program has the ability to simulate the entry of runoff into a piped collection system and the gravity or pressure flow of water in that collection system. EXTRAN input consists of the output hydrographs generated by the RUNOFF block, sanitary flow rates, and data on the lengths, diameters, slopes and Manning's roughness coefficients for the conduits. In addition, a surface "overflow channel" system can also be modeled that represents the routing of floodwater through the streets. The model provides a dynamic simulation; flow rates, velocities and water surface elevations are reported for the collection system in a time history format over the defined duration of the rainstorm event.

A more detailed review of SSWMM91 is provided in the Phase 2 technical memorandum entitled *SWMM Expansion, Calibration, and Baseline Flooding Results for the 10-Year 6-Hour Storm*, dated July 1, 1991. That document is Appendix E to the *Phase 2 Detailed Technical Report*.

The EPA program (unmodified version) is described in detail in the current versions of the EPA's *Users Manual*, including appendices available from the EPA Office of Municipal Environmental Research Center, Office of Research and Development, Cincinnati, Ohio. The following discussion assumes a limited knowledge of the model. Details of the model configuration, program code and limitations can be reviewed in the EPA *Users Manual*.

C. FLOOD AND CSO CONTROL DESIGN STORMS

Design storms are a hypothetical distribution of rainfall depths or intensities over a time increment for a given storm duration and frequency. Synthetic design storms were used to determine the performance of proposed improvement plans measured in terms of flood and CSO control. For this planning study, the flood control design storms were also used as a measure of the effectiveness of the alternatives to control street outflows.

Two sets of design storms were used to analyze alternative improvement plans. The first involves design storms for flood/outflow control project sizing. Design storms for flood control are six hours in duration with return periods of 2, 5, 10 and 25 years. The second involves design storms for sizing CSO control facilities, such as new storage facilities and treatment plants. Storm frequencies of 1/year and 5/year were used for CSO control project sizing.

The use of design storms in designing conveyance and storage facilities is widespread. Design storms are valuable in that they provide a relative measure by which several alternatives can be compared. The use of design storms, however, does require some assumptions that may not apply in all cases.

The most critical of these assumptions is that the frequency of the precipitation will produce runoff and facility response of the same frequency. This assumption

may not always be true due to varying conditions in the drainage basin and conveyance system. For example, the same precipitation may be applied to a basin and the timing and volume of the runoff may differ as a result of the initial moisture in the soil from previous events. The soil will infiltrate much more precipitation and produce less runoff in a fall storm with little previous rainfall than in a spring storm with previous rainfall. The assumption that a storm event of a particular frequency will produce a conveyance system response of the same frequency can also be invalid in the case of a conveyance system that has a significant amount of storage.

These types of problems associated with use of synthetic design storms in analyzing a system as hydraulically complex as the CWCS can be solved through the use of continuous simulation. Continuous simulation involves modeling a system using a long-term rainfall record (usually hourly) or a selected representative period (e.g. 10 years) of that record. Statistics, such a CSO or flooding frequency, can then be derived directly from the simulation under actual, historical rainfall conditions rather than extrapolated from the use of single design events. It was determined by City staff, that this approach would be a refinement to the current study, and would be completed during future evaluations.

C.1. Flood Control Design Storms

Improvement plans were configured so as to eliminate the occurrence of street floodwater when subjected to a synthetic design storm. The level of flood control afforded by a particular improvement plan was defined by the recurrence interval of the design storm used to configure that improvement plan.

The flood control design storm recurrence intervals of 2, 5, 10, and 25 years were established by the City based on the results of the Phase 2 analysis, and based on comments received from the Regional Board on the *Phase 2 Detailed Technical Report*. The design storms were developed and input to the RUNOFF block for generating hydrographs for the combined service area basins. Conveyance and storage improvements to control flooding were then sized and configured to convey the resulting flows in the system.

C.2. CSO Control Design Storms

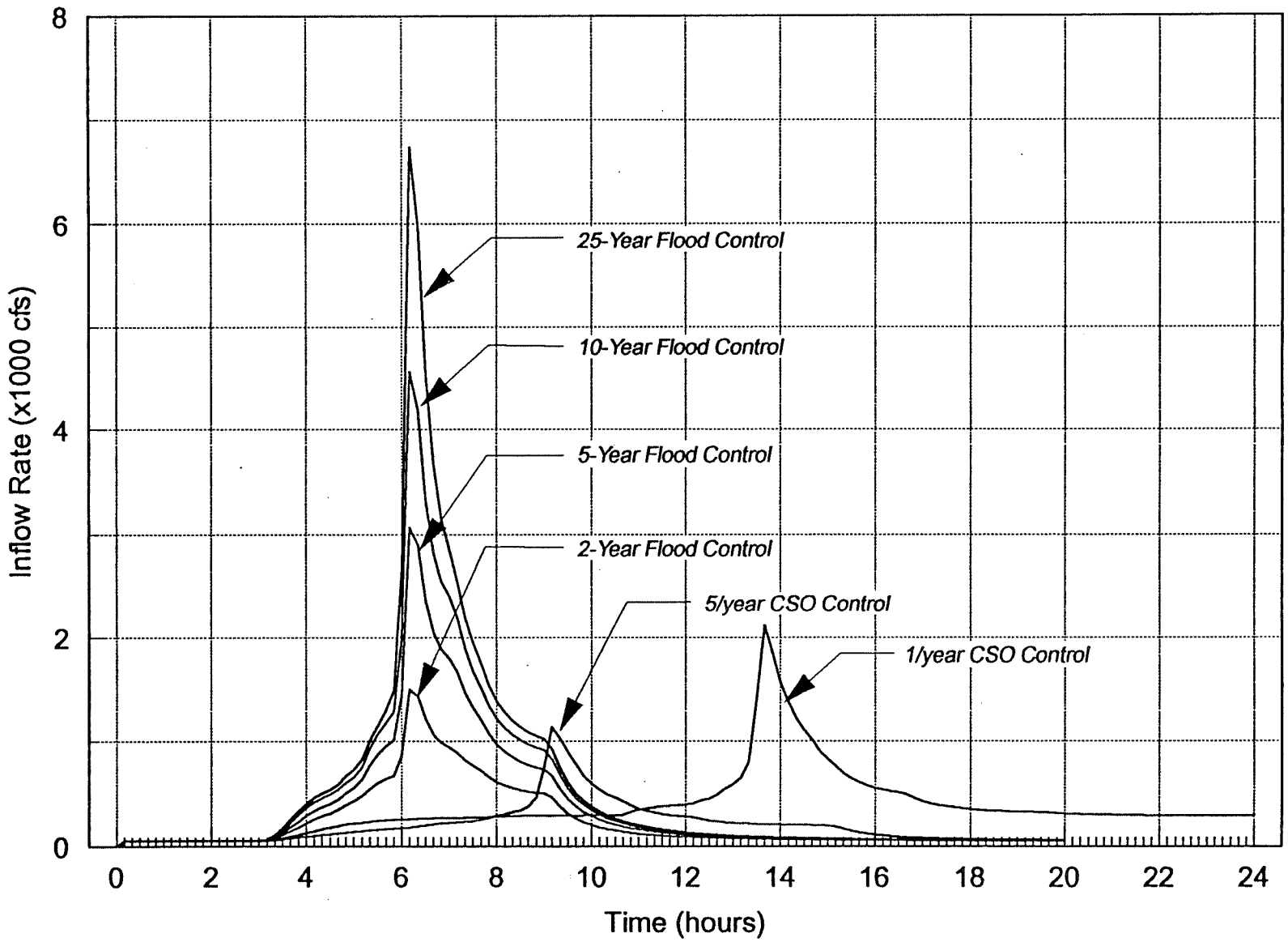
In addition to flood control design storms, alternative improvement plans were evaluated for the control of design events representing frequencies of 1/year and 5/year storms. The underlying assumption made by the City was that the design storm frequency for the system would correspond to the CSO frequency for the existing system. As previously discussed, this underlying assumption may not necessarily be correct for the combined sewer system.

The use of the 1/year design storm is based on a Phase 2 technical memorandum *Survey of Cities Developing or Implementing CSO Control Plans*, dated July 1, 1991. This study reported that a number of states impose a limitation of one CSO event per year, or control of CSOs for the 1/year storm event. One of the objectives of the Phase 3 study was to identify the costs of providing CSO control to a similar level, i.e. the 1/year storm, for Sacramento.

The objective of the 5/year design storm analysis was intended to assess the CSO upgrade costs, if any, to provide a CSO control protection level similar to the existing CWCS. Based on 4 years of recent historical data the existing CWCS spills approximately 4 to 5 times per year to the Sacramento River. The use of the 5/year storm was intended to replicate existing CSO conditions, again based on the City's assumption of equating rainfall and CSO frequency.

The flood control and CSO design storms were derived in accordance with the procedures outlined in the Phase 3, Task 3 technical memorandum *Design Storms, Rainfall Analysis and Flood Volume Maps for the Phase 3 Analysis.*

The runoff hydrographs into the CWCS resulting from the above discussed design storms are shown in Figure 3-1



SYSTEM INFLOW HYDROGRAPHS FOR THE CSO AND FLOOD CONTROL DESIGN STORMS

HDR

Figure 3-1

C.3. Discussion of Outflow Control

The Cease and Desist Order levied on the City by the Regional Water Quality Board specifically addressed the control of potential health threats posed by combined sewage outflows. Quantifying existing and future reduced outflows associated with alternative plans would provide a criterion in addition to flood and CSO criterium for selecting an optimum plan. One of the initial efforts in the Phase 3 study was to determine whether outflows from the system could be quantified, given the analytical tools available. Following the submittal of the *Phase 2 Detailed Technical Report*, the Regional Board also requested that the City determine the amount of outflows, and to differentiate floodwater caused by stormwater incapable of entering the system from floodwater caused by outflows from the surcharged system. The City was asked to develop costs and improvements associated only with the control of system outflows. Based on an extensive review of available analytical tools and the technical aspects of modeling outflows, it was determined that the requested analysis could not be completed within the time constraints of the Phase 3 study. Furthermore, it was estimated that the time and expense necessary to develop a theoretical computer model and cost allocation procedures would be unreasonable. Finally, the modeling effort could prove impossible to carry out satisfactorily to satisfy the Regional Board's request.

For these reasons, it was determined that outflow control benefits could only be measured relative to flood control benefits. That is, if an alternative improvement plan was developed to control street flooding for a given design storm, then it was assumed that outflows to the street would also be controlled for that event. The development of improvement plans and the preparation of the benefit cost analysis was based on this assumption.

D. OVERVIEW OF ALTERNATIVE IMPROVEMENT PLANS EVALUATED FOR PHASE 3

Improvement plan alternatives can be classified according to the type of facilities incorporated into the proposed plan. The types of improvement plans evaluated include:

- Tunnel Alternatives.
- Storage Alternatives.
- Treatment Alternatives.
- Separate Sanitary Sewer Alternative.
- Separate Storm Drain Alternative.
- Inflow Control Alternative.
- Localized Flooding Improvement Alternative.
- Baseline Rehabilitation Alternative.

The Phase 3 improvement plan alternatives were configured to meet both flood and CSO control design storm criteria. The performance of proposed improvements for providing flood and CSO control was verified using the SSWMM91.

For the storage, treatment and tunnel alternatives, flood control was achieved by upsizing the existing conveyance system, and by relieving excess flows from the existing system using weirs to divert flow into a network of consolidation conduits. The network of consolidation conduits drain to designated storage and/or treatment sites, or into a tunnel system. The conduits were sized to eliminate flooding in the combined sewer service area for a given design storm.

To configure alternatives to also provide CSO control, the 1/year and 5/year CSO design storms were applied to the individual improvement plans configured for the 2-, 5-, and 10-year flood control design storm events. The combined wastewater flows and volumes resulting from the application of the CSO storms were used in determining the sizes of storage, treatment, and tunnel storage needed for CSO control.

It was determined from model runs that CSO control for the 5/year design storm is provided by the existing CWCS facilities, operating under the original plan of operations. Therefore, all alternative improvement plans configured for the 5/year CSO design storm consisted of only the facilities that would be needed for flood control. CSO control for the 1/year design storm required storage and/or treatment improvements in addition to the flood control improvements.

Five of the alternatives evaluated during the Phase 3 study were not developed to control both a predefined CSO and flood design storm, e.g. the 5-year flood control and 1/year CSO control design storms. Furthermore, some of these alternatives were evaluated only from a general feasibility standpoint. These include:

- Separate Sanitary Sewer Alternative.
- Separate Storm Drain Alternative.
- Inflow Control Alternative.
- Localized Flooding Improvement Alternative.
- Baseline Rehabilitation Alternative.

Descriptions of the alternative improvement plans and design considerations unique to each are summarized in the sections below.

D.1. Tunnels

The tunnel alternatives consist of three components used to convey or store combined sewage. The first component consists of replaced service laterals and collectors. These replaced conduits convey flow to the mains in the existing combined sewer system. Excess flow that cannot be handled by these mains is drawn off by diversion weirs and directed into the second main component, a network of new consolidation conduits. The consolidation conduits receive excess flows at one or more locations and convey the flow to tunnel drop shafts. These drop shafts empty into the tunnel system, the third component of the system.

Flows in the tunnel are directed to terminal points near Sumps 1A and 2. A portion of the total inflow into the tunnels was retained within the tunnels by discharge control structures located at the ends of the tunnels at Sumps 1A and 2. These discharge control structures were set to retain the design CSO control volumes within the tunnels. Provisions were also included to allow diversion of Pioneer Reservoir overflows into the tunnel network for CSO control. Tunnel pump stations discharge all flows in excess of the CSO volume directly to the river without treatment up to the point where the combined sewer service area no longer contributes significant runoff to the tunnel system. At that point, pumped

discharge to the river ends and the contents of the tunnel system are stored for secondary treatment at SRWTP during off-peak flow conditions.

The analysis of the tunnel alternatives included variations in tunnel diameters, storage, and discharge control. These parameters were varied to configure the alternatives for the flood control 5-, 10-, and 25-year and the CSO control 1/year and 5/year design storm events. The sensitivity of these design parameters to cost was also considered. For the 2-year design storm, tunnels were not cost effective, since most of the existing piping system can convey the 2-year storm without flooding.

Tunnel system configurations were influenced by constructibility limitations. One of these limitations was that the tunnels should be a minimum of one mile long if they are to be cost competitive with other improvement plan alternatives. A major portion of the tunnel construction cost is in contractor mobilization and construction of the tunnel access shaft. Minimum constant-diameter tunnel sections of one mile are needed to distribute these front-end costs over the entire project.

Another construction limitation involved tunnel alignment. Tunnel boring machines cannot construct bends with a centerline radius smaller than roughly 200 feet. Consequently, the right-of-way for tunnel projects had to be relatively straight, using only long radius bends where required.

The Phase 2 recommendation for a tunnel depth of approximately 50 feet was used in configuring the tunnel networks considered for the Phase 3 study.

D.2. Storage

As with the tunnel alternatives, the storage alternatives were configured assuming new diversion weirs installed on the existing collection system. Instead of conveyance to a tunnel system, diverted flows were conveyed to storage sites by consolidation conduits.

The analysis of storage alternatives involved the use of both flood control and CSO control design storms run in the SSWMM91, similar to the analysis of tunnel alternatives. Storage volumes and pumping capacities required for both flood and CSO control were sized from the model output file data on peak flows and inflow volumes to storage sites.

Three sites were considered for proposed storage reservoirs: Pioneer Reservoir, Western Pacific Railroad and UC Davis Medical Center. Combinations of these sites were considered in improvement plan alternatives. These combinations are listed in Table 6-1.

In addition to the alternatives listed in Table 6-1, two other conveyance alternatives were configured for the 2-year design storm. These consisted of using the Pioneer Reservoir and WPRR sites for locating storage facilities, with flow delivered by upsizing the existing CWCS without using side overflow diversion weirs and consolidation conduits. These two alternatives are referred to as the "2-Year Upsized CWCS" alternatives.

The WPRR and Pioneer Reservoir sites are close enough to the Sacramento River that combined wastewater exceeding the volume produced by the CSO design storm could be discharged to the river. River discharge configurations have potentially lower costs than storing the entire design storm volume at these sites. Flows directed to the UCD Medical Center would be to far from the river for discharge. The UCD Medical Center requires flow to be stored at the site and discharged to the existing collection system during off-peak flow conditions for secondary treatment.

Overflow from the existing Pioneer Reservoir was diverted back into the network of consolidation conduits for the 5- and 10-year alternatives, in order to meet CSO control objectives. The exception was for alternatives that include a facility at Pioneer Reservoir. For the 2-year alternatives, CSO control was provided either by storage at the Pioneer Reservoir site and/or by storage at the WPRR site. A diversion structure was located on the existing Donner trunk at the WPRR site to divert stormwater flow into the facilities.

Storage reservoirs used in improvement plans were assumed to be similar in construction to the existing Pioneer Reservoir. The elevations of the proposed storage reservoirs were assumed to be high enough to avoid construction below the local area water table. This assumption necessitated the use of an influent pumping station to lift wastewater from the consolidation conduits up into the storage reservoirs.

D.3. Treatment

The treatment alternatives utilized the same combinations of sites as the storage alternatives as listed in Table 6-1. Flood control for the 2-, 5- and 10-year design storms was provided by upgrading the existing CWCS and/or providing networks of weirs and consolidation conduits to divert flows to the sites. Three of the alternatives also considered storage at the UCD Medical Center site for flood control. The treatment facilities were sized to control the CSO design storms. Flows in excess of that required to control the CSO storms were pumped to the river without treatment.

The analysis of treatment alternatives considered treatment at four sites: Pioneer Reservoir, WPRR, Meadowview and SRWTP. Conveyance to the Meadowview and SRWTP sites was provided by a pumping station at the WPRR site and a long force main connecting the pumping station to the candidate treatment sites.

WPRR and Pioneer sites were evaluated to determine if the addition of flow equalization storage would lower the overall cost of the alternative. Inflow hydrographs to the facilities generated from the CSO design storms were used to perform a cost optimization of the treatment facility flow rate versus storage facility volume. From this optimization, a rated treatment capacity and storage volume was determined for each site. All flows less than or equal to the rated capacity of the treatment facility were treated. All flows in excess of the treatment rate were stored until the available storage is filled. Inflow rates and volumes in excess of the primary treatment and storage facilities respectively are discharged to the river without treatment.

The use of available flow equalization storage was maximized throughout the storm, and all water remaining in storage after the storm would receive primary treatment. Primary treatment consists of chlorination, sedimentation with a minimum of 1.5 hours of detention time for the peak operational flow rate, and dechlorination prior to discharge to the river. This assumed mode of operation for flow equalization basins differs slightly from the "storage-only" alternatives. For the latter type of alternative plan, water remaining in the storage facilities was discharged to the existing conveyance system for treatment at the SRWTP.

D.4. Separation

Two improvement plan alternatives were configured based on the concept of separating the combined wastewater conveyance system. One assumed construction of a new sanitary sewer system, the other a new storm drainage system. The area served by these new sewer systems would be coincident with the existing combined system.

D.4.a. New Storm Drain System--

The Separate Storm Drain Alternative was configured by assuming the existing combined sewer system would remain in service as a sanitary sewer system. The new system provided drainage for the entire combined sewer service area, excluding the existing separated sewer service area. Sizing of new sewers was based on conveying the 10-year design storm. Analysis using CSO design storms was not needed, since this alternative eliminates CSOs. New storm sewers were configured to drain to two new stormwater pumping stations located near the existing pumping facilities at Sump 1A and Sump 2. Collected stormwater would be pumped to the Sacramento River. Routing of the new storm sewers followed the existing combined sewer conduits modeled in the SSWMM91.

D.4.b. New Sanitary Sewer System--

The new sanitary sewer system assumed the existing combined sewer collection system would remain in service as a storm drainage system. New sanitary sewers modeled in the SSWMM91 were sized using sanitary hydrographs. These were developed using a diurnal curve approach that included the peaking factors

recommended in the City's *Design and Procedures Manual and Improvement Standards*. This approach was conservative, in that the resulting predicted peak flows included a design factor of safety. It is anticipated that actual peak flows would be lower. Individual drainage basin sanitary hydrographs were derived by imposing the diurnal curve in the average sanitary flow estimates prepared by the City. Added to this was an allowance of 200 gpd/acre to account for groundwater infiltration.

The network of new sanitary sewers needed to serve the basins modeled in the SSWMM91 was assumed to parallel the existing modeled combined sewers. New sewer sizes were estimated by reducing the size of the existing system conduits to convey the estimated peak flows generated by the routing of the sanitary hydrographs. New sewers were configured to drain to Sump 2 for conveyance to SRWTP for secondary treatment.

By contrast, separation of the existing collection system sewers (pipes less than or equal to 16 inches in diameter) was assumed to be a network of uniformly sized 8-inch conduits paralleling the existing collection system. This estimate excluded the existing backyard easement sewers that convey only sanitary flows.

The Separate Sanitary Sewer Alternative includes only a minor flood control upgrade beyond the capacity of the existing system. However, the existing system provides flood control to a 2-year event in most areas. CSOs are eliminated, and flood control is slightly improved, by removing the sewage portion of flow from the conveyance system. This alternative also eliminates outflows.

D.5. Inflow Control Alternative

The Phase 3 study included a review of the feasibility of inflow control as an alternative to upsizing the existing conveyance system to control street outflows. Outflows occur when combined wastewater backs out of the conveyance system onto streets or into building structures.

The Inflow Control Alternative involves restricting stormwater inflow into the combined sewer system such that gravity flow conditions are maintained. This

controls surcharging and subsequent outflows during storm events. Inflow control is accomplished by installing flow restriction devices at individual drop inlets in the collection system.

Only a feasibility-level evaluation was completed for this alternative, since sufficient data were not available for a complete analysis. The ability of this alternative to meet flood, outflow, or CSO control was not accessed.

D.6. Localized Flooding Improvement Alternative

The City Utilities Department staff developed an alternative that provides limited flood control for six key problem areas in the combined system. This alternative uses a diversion and consolidation conduit system, similar to the other alternatives, to draw off excess flows from the existing sewer system. These conduits lead to underground storage facilities that consist of parallel 12-foot diameter pipes. Because of the time constraints involved in analyzing this alternative, the six areas were isolated from the system for modeling and analysis. Therefore, flows that back up into these six areas (several of which are low in elevation) from adjacent areas were not considered.

D.7. Baseline Rehabilitation Alternative

Minimum improvements to the CWCS are represented in the Baseline Rehabilitation Alternative, which includes structural and other reliability upgrades to the existing pumping stations and conveyance system. No outflow, flood or CSO control improvements are included in this alternative, and no SWMM analysis was performed. Additional description of the components of this alternative is provided in Section 4.

E. SSWMM91 INPUT PARAMETERS

E.1 SSWMM91 Model of the Existing System

All of the alternative improvement plan model networks developed in the Phase 3 study used the existing system model set-up as a foundation. The extent of the

modeled study area includes the entire combined sewer service area and adjacent drainage basins that contribute either stormwater or sanitary sewage to the CWCS. All interceptors and most laterals/mains 16 inches and greater in diameter have been included in the EXTRAN input files. Most recently for the Phase 3 study, the operation of Pioneer Reservoir and its outfall, Pioneer Interceptor, and the Flow Control Structure at Sump 2 and its two outfalls, have been incorporated into the EXTRAN input files.

E.2. Revised SSWMM91 Model Parameters

Individual drainage basin characteristics influence the routing and runoff of rainfall incident upon those basins. The Phase 2 study recommended the re-evaluation of hydrologic parameters input to the RUNOFF block and sanitary flows input to the EXTRAN block setup.

For this Phase 3 study, the City Utilities Department staff developed the technical memorandum entitled *SWMM Parameter Refinements to Infiltration Rates, Impervious Percentages, and Sanitary Flows Within the Combined Sewer Area*. Aerial photos and Soil Conservation Service (SCS) soil classification mapping were examined for each basin in SSWMM91. Aerial photos were used to define land use, from which impervious percentages could be determined. By applying typical infiltration rates for each soil type within a basin, a total weighted infiltration rate was also developed for each basin in the RUNOFF block setup files.

This analysis also provided an opportunity to review the amount of sanitary base flow generated by each basin. During the examination of the aerial photos, the land use classification for each basin was established and correlated with the sewage generation rates listed in the City's *Design and Procedures Manual*. Sanitary flow data were input as constant flows to junctions in the EXTRAN block input file. The total average sanitary base flow rate for the combined sewer service area was estimated by City staff to be approximately 32 mgd.

Additional changes to the EXTRAN file to improve the numerical stability of the model are described in Appendix C.

E.3. Infiltration and Inflow Contribution from the Separated Sewer Area

Junction flow data in SSWMM91 for the separated sewer area consists of both sanitary base flow and rainfall-dependent infiltration and inflow (RDI/I). RDI/I is water that enters the sewer system in direct response to rainfall. RDI/I enters the system directly through cross-connections between the storm drain and combined sewer system or through area drains and downspouts illegally connected to the sanitary system. RDI/I also enters the sewer system indirectly through sewer defects, particularly in shallow pipes such as private building laterals.

Rainfall-dependent infiltration for the separated sewer area was selected to be 2,400 gallons per day per acre of drainage basin, for the 10-year design storm event. These rates agree with the draft technical memorandum *Design Flow Criteria*, prepared for the County of Sacramento Sanitation District Sewerage Expansion Study by James M. Montgomery Engineers, dated December 2, 1991. The value of 2,400 gal/acre/day was adjusted according to the storm event considered with the multipliers listed in Table 3-1. These multipliers were also adopted for the Sacramento County interceptor master planning documents.

TABLE 3-1. RAINFALL INDUCED I/I MULTIPLIERS

Flood Control Design Storm	Multiplier
2-Year	0.85
5-Year	0.85
10-Year	1.0
25-Year	1.2

F. DESIGN AND ANALYSIS ASSUMPTIONS

Modeling assumptions common to several alternative improvement plans analyzed using SSWMM91 are presented below. Modeling assumptions that are specific to a particular alternative are described in the sections dedicated to that improvement plan analysis.

F.1. General Design Criteria

All new sewers, whether they were part of the collection system, laterals, mains, interceptors or tunnels, were assumed to have a Manning's roughness coefficient of 0.014, which is conservative for large diameter pipes. Commonly used values range from 0.010 for very smooth pipe to 0.016 for older, rough, unfinished pipe. The differences in modeled flow rates resulting from variation of this parameter are within the accuracy limits of any field flow monitoring and model calibration techniques.

Peak velocities for new sewers were limited to between 2 and 10 feet per second for the applied design storm event.

New sewers were modeled to provide a minimum cover of 2.5 feet. This allows for standard urban street construction practices of placing an aggregate base and pavement over the pipe in thicknesses typical for urban streets.

New sewers were almost exclusively modeled as circular conduits. Exceptions to this were for discharge from the tunnel system and modeling the operation of the Pioneer Reservoir and Flow Control Structure at Sump 2.

All new conduits with the exception of the replacement conduits were designed for approximately 80 percent of the total conduit capacity at the peak of the design storm. In SSWMM91, conduits are joined together at junctions. For new conduits, the crown elevations of the sewers entering and exiting a junction were modeled with the same crown elevations. This assures minimal restrictions in hydraulic transitions from one diameter sewer to another. In sewer design, this would be preferable where adequate grade is available. However, it is not uncommon practice to maintain a constant flow line invert elevation at straight-flow-through junctions where existing surface grades are relatively flat.

F.2. Replacement Pipes

Several modeling options exist for upgrading the existing collection system for flood control exist. The options selected for alleviating existing flooding

conditions are replacement pipes, parallel pipes, and diversion weirs to a consolidation conduit. Factors that influenced the option used were the size of the conduit, the age and condition of the conduit, location of the nearest consolidation conduit and locations of nearby weirs and drop shafts.

Flooding was first alleviated where possible by the upsizing existing conduits. This provided the City with a new replacement sewer with extended new life for that section of the sewer system. If a single new sewer could not be used to replace the existing sewer due to cover limitations, a new parallel sewer or a diversion weir was considered. If replacing the existing sewer with a larger sewer would not solve the flooding or if the undersized sewer was a major interceptor, then a diversion weir was used to relieve flow from the existing system.

Much of the flooding in the existing combined sewer system is due to undersized laterals and collectors. To draw off excess flows from smaller conduits would not be cost effective. Therefore, these undersized conduits were upsized in order to covey flow to more central locations were the flow was diverted by means of diversion weirs.

The replacement conduits were designed to take advantage of higher conveyance due to surcharged conditions. This was done for several reasons. First, pipes designed for open channel flow conditions would end up being considerably larger than the mains to which they connect. Second, the larger diameters would have difficulty meeting cover requirements. New replacement conduits were consequently designed to be surcharged. The existing conduits remaining in place were also allowed to surcharge. However the pipes were sized to control the surcharge levels and to maintain the hydraulic grade line between 2 and 3 feet below the manhole lid elevation. Exceptions exist where manholes are shallow or where the cost of meeting the criteria outweighs the benefit.

F.3. Diversion Weirs

Diversion weirs were used to relieve the existing CWCS sewer system of excessive flows. These excess flows were collected in a new system of gravity sewers

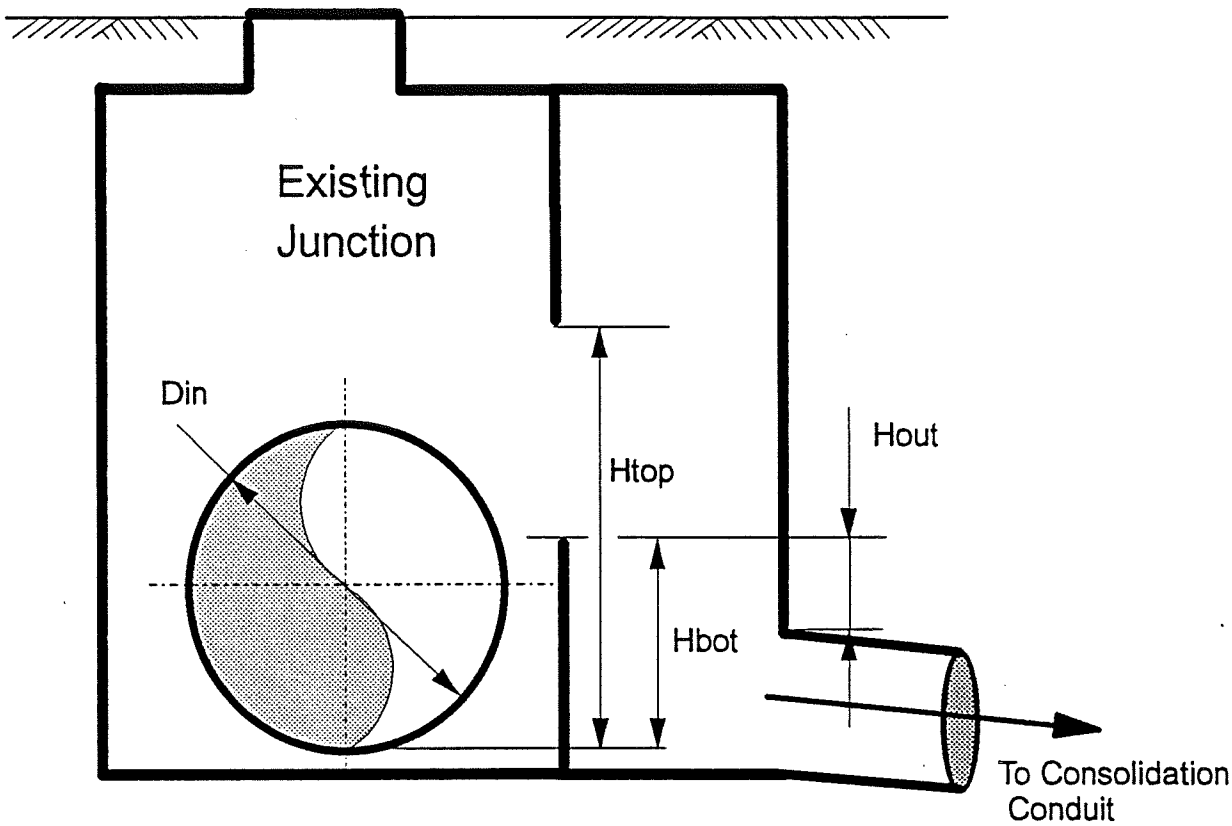
referred to in this study as "consolidation conduits". An illustration of a diversion weir as modeled in the SSWMM91 is provided in Figure 3-2.

Input data required for modeling the weir includes: the weir length, the weir coefficient, and the height of the bottom and top of the weir with respect to the invert of the junction (as shown in Figure 3-2). Weir lengths were set at a nominal 10 to 30 feet, with the majority of the weir lengths being 20 feet. The weirs were assumed to be double sided, meaning that a overflow would exist on each side of the conduit (see Figure 4-7). Therefore, most weirs would be 10 feet long on each side of the structure. The weir coefficient was set at 2.8, as a conservative value to allow for varying hydraulic conditions. This value was also recommended by the modeling consultant.

The height of the weirs were set to be a function of the design storm at peak flow, in order to divert excess flows while utilizing the existing combined sewer system as much as possible. The weir height was chosen as a percentage of the diameter of the existing sewer from which flows were directed. For the larger storm events, lower weir heights were necessary to lower the hydraulic grade line for flow in the existing sewers. For the smaller storm events, the weirs were raised to maximize utilization of the existing collection system. As a general guide, the weir heights indicated in Table 3-2 were used to model the weirs. In some locations, these weir heights had to be lowered in order to control local flooding.

TABLE 3-2. DIVERSION WEIR CREST DEPTH SETTINGS

Design Storm Recurrence Interval	Weir Crest Depth As A Percentage of Existing Sewer Diameter
2	80
5	80
10	33
25	33



Weir Settings:

Hbot, from 0.33(Din) to 0.80(Din)

Htop - Hbot $\geq$ 5.0 ft

Hout > 0.0 ft

SCHEMATIC ILLUSTRATION OF THE WEIR SETTINGS AS MODELED FOR ALL IMPROVEMENT PLANS UTILIZING DIVERSION WEIRS

HDR

Figure 3-2

It must be noted that these weir heights are designed for the peak flow of an event, and that during the leading and trailing portions of the design storm, more water may be diverted to the consolidation conduits than necessary, especially with the 10 and 25 year weir settings.

In practice, these weirs would be constructed as weir gates with motor-driven operators and a telephone-signal telemetry control system. This would allow the operations staff to control the amount of water relieved from the existing collection system. However, only fixed weirs can be modeled by SSWMM91 at this time. Further description of these weir structures is provided in Section 4, and Appendix C.

There was concern that by allowing the weirs to be so low in the model during the less frequent design storms, the storage capacity of the existing sewer system would be underutilized. However, the flow into the combined sewer collection system at the peak of the storm exceeds the capacity of the existing system to such an extent, that full flow conditions were reached at one or two junctions downstream of a diversion weir. For the smaller design storm events, weir depths could be set higher and still make full use of the storage capacity of the existing collection system. Again, it is important to remember that the model only allows for simulation of static weir operation, rather than the dynamic operation conditions that would exist in reality.

F.4. Consolidation Conduits

The diversion weirs spill excess flows from the existing system into a network of consolidation conduits which convey the flow to tunnels, storage facilities, or treatment facilities. The consolidation conduits were designed in accordance with the design criteria as outlined in Subsection F.1. The conduits were also designed so that the top of the consolidation conduit would always lie below the edge of the overflow weir to avoid any flow from transferring back across the weir in open channel flow conditions. See Figure 3-2. These conduits were not surcharged for reasons described in Appendix C.

The trade-offs between increasing the slope of the pipe in order to decrease the needed diameter was examined to determine the most important consideration with respect to cost. It was determined that to attempt to place the conduits shallow in order to avoid reduced trenching costs resulted in pipe diameters which required the same or even more excavation. The slopes and diameters used for the conduit design are considered reasonable for the design flows.

The storage and treatment alternatives collect and convey large flows which require consolidation conduit diameters in excess of 12 feet. It was assumed that large diameter conduits, up to 12 feet, would be constructed using open trench construction techniques. Above that, the conduits were assumed to be constructed by tunneling. As described further in Section 5, tunnels are subject to the construction limitation of using only a single diameter for an entire tunnel segment. To be cost effective, the length of tunnel must be a minimum of a mile long. Consequently where tunneling was required for consolidation conduit diameters greater than 12 feet, a long run of tunnel of constant diameter was used.

G. MODELING CSO OPERATIONAL CRITERIA

The criteria for all new improvement plans used to determine whether combined wastewater would require treatment (either primary or secondary) versus direct discharge to the river approximates the City's permitted operations within the recent past. Below is a summary of the City of Sacramento current discharge prohibitions.

1) The discharge to the Sacramento River is prohibited at the following discharge points unless the specified conditions are met.

    a) CWTP. No discharge unless a flow of 60 mgd is being sent to the SRWTP.

    b) Sump 2 and Pioneer Reservoir. No discharge unless a flow of 130 mgd is being sent to the CWTP, and 60 mgd is being sent to SRWTP.

2) Other than as a result of wet weather, it is prohibited to discharge from the CWTP, Pioneer Reservoir or any sumps to surface waters or surface water drainage courses.

In the development of alternative improvements, the above criteria were adhered to as well as the following additional criteria for storage, treatment, and discharge of combined wastewater during and after a storm event.

1) During a storm event, all tunnel, storage, and treatment alternatives must store the CSO design storm volume, but may discharge to the river any volume in excess of the CSO design storm volume that reaches these facilities

2) After a storm event has ended, any combined wastewater remaining in the tunnel facilities, storage only facilities, and in Pioneer Reservoir must receive secondary treatment at the SRWTP. Any combined sewage remaining in the storage/treatment facilities after a storm event must receive primary treatment. In this context, a storage/treatment facility is used to describe a primary treatment facility with upstream storage for flow equalization.

3) Untreated combined wastewater discharged to the river is considered a CSO. Combined wastewater treated either at a primary treatment facility or at SRWTP and subsequently discharged to the river is not considered a CSO.

4) No new combined wastewater or stormwater outfalls are allowed to discharge into the American River. All new primary effluent, combined wastewater and stormwater discharges would be directed into the Sacramento River.

5) Volumes and flow rates for minimum sizing of storage and primary treatment facilities were estimated using the CSO control design storms (i.e. either 1/year or 5/year design storm frequencies).

H. MODELING EXISTING FACILITIES

The EXTRAN block setup was modified for the Phase 3 study to simulate the operation of major existing facilities in the CWCS. This included Pioneer Reservoir, Pioneer Interceptor, the Flow Control Structure at Sump 2, the interceptors carrying flows south from the Flow Control Structure to the CWTP, and the outfalls to the river at both Pioneer Reservoir and the Sump 2 Flow Control Structure. The capacity of the existing CWCS infrastructure is summarized in Table 3-3.

These data were used as input to the models simulating the performance and contribution of the existing system in controlling flooding and CSOs.

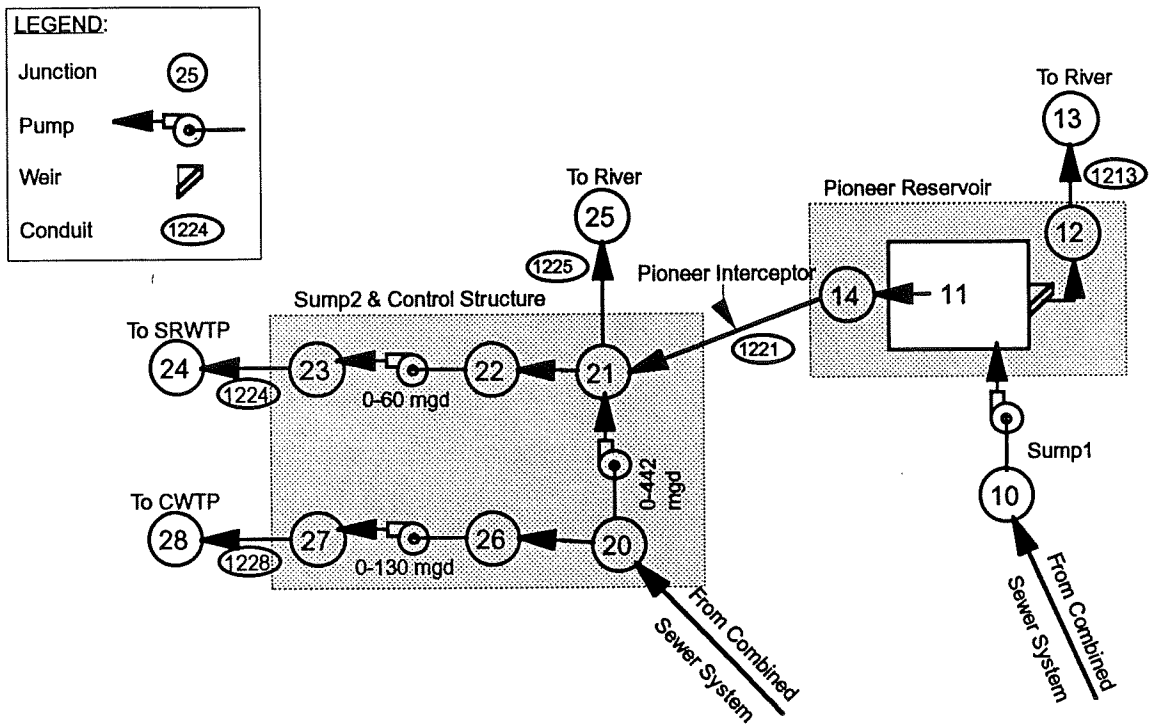
The modeling of these facilities was designed to reflect the original Plan of Operations, which minimized the volume of wastewater spilled to the river. The model network used to simulate this is shown schematically in Figure 3-3 and explained further in the following subsections.

TABLE 3-3. CAPACITIES AND PERFORMANCE RATINGS FOR EXISTING CWCS INFRASTRUCTURE

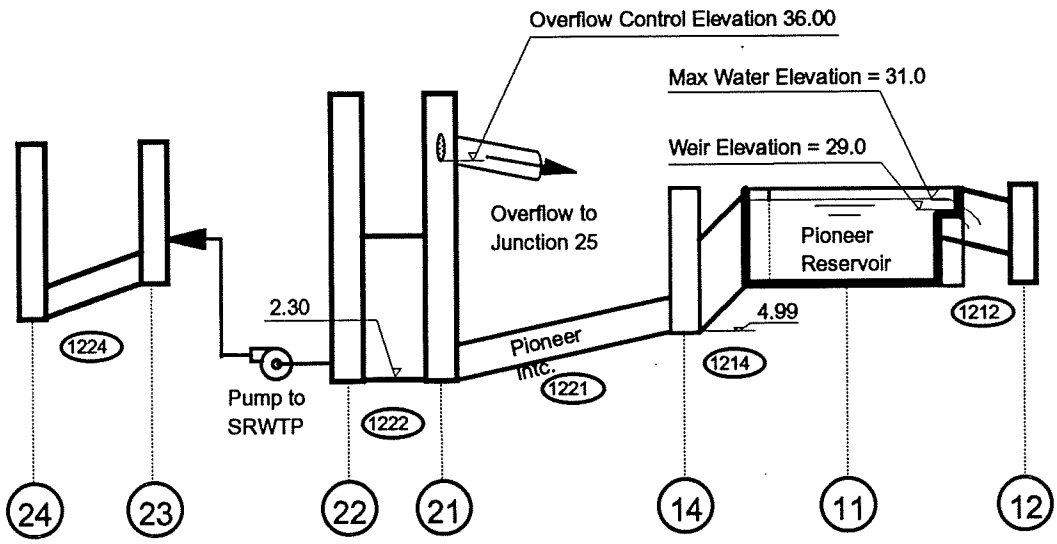
Description	Capacity Rating
Pioneer Reservoir	23 MG
Pioneer Interceptor	120-inch, 9100 lf
CWTP	130 mgd
City Allowance at SRWTP	60 mgd
Sump 1A	84 to 124 mgd Depending upon Pioneer Reservoir Level
Sump 2	572 mgd

H.1. Modeling the Operation of Sump 2

Previous versions of EXTRAN setup files for the CWCS included a pumping station (Sump 2) at Junction 20, which pumped flow to a free outfall. The rate of pumping



SCHEMATIC LAYOUT OF THE SUMP 2 AND PIONEER RESERVOIR SYSTEM MODEL



SCHEMATIC SECTIONAL VIEW OF THE SUMP 2 AND PIONEER RESERVOIR MODEL SETUP

HDR

Figure 3-3

was controlled by the water elevation at Junction 20, as recommended by the original Plan of Operations for Sump 2. During the Phase 2 study, setpoints for Sumps 1A and 2 were revised from previous work to be in accordance with the Interim Plan of Operations (adopted November 2, 1990). Previous modeling efforts did not provide for any simulation of Pioneer Interceptor conveyance, storage in Pioneer Reservoir, conveyance to treatment facilities, or overflows to the Sacramento River.

To simulate pumping operations at Sump 2 and the Flow Control Structure, and provide diversion of pumped flows to treatment, storage and overflows to the river, a total of 3 pumps were used as indicated in Figure 3-3. Inflow into Junction 20 is split between two pumps: 1) Pump 26-27 (pumping between Junctions 26 and 27), operating up to a peak output of 130 mgd to simulate delivery to CWTP; and 2) Pump 20-21, delivering flow to Junction 21. Pump 22-23 delivers flow from Junction 21 to the SRWTP up to a peak of 60 mgd, and simulates the diversion of flow from the Flow Control Structure to the SRWTP.

The relationship between water depth in Junction 20 and discharge from these pumps is summarized in Figure 3-4. This pump discharge curve simulates the overall Sump 2 operation recommended in the original Plan of Operations. Pump 22-23 conveys only the amount of flow diverted from the Sump 2 Flow Control Structure to the SRWTP.

Junction 21, as part of the Flow Control Structure, is also connected to Pioneer Reservoir. Conduit 1221 represents the 120-inch Pioneer Interceptor, modeled with as-built upstream and downstream invert elevations taken from the Sump 2 Plan of Operations.

During periods of low flows to Sump 2 (less than 60 mgd), inflow to Junction 20 is pumped only by Pump 20-21 for transfer to Junction 21. From Junction 21, Pump 22-23 transfers wastewater to SRWTP. As the wastewater inflow to Junction 20 increases, the water level in that junction rises to elevation 4.5 feet and Pump 26-27 starts up to pump wastewater to the CWTP. The pumping rate of Pump 20-21 remains at 60 mgd. When inflow exceeds 190 mgd, Pump 26-27 pumps at the maximum rate of 130 mgd and Pump 20-21 pumps at a rate of between 60 mgd to maximum

capacity. With inflow exceeding the maximum capacity of Pump 22-23 (60 mgd), the water elevation in Junction 21 rises and enters Conduit 1221, the Pioneer Interceptor. Continued flow into Junction 21 is conveyed through Conduit 1221 and into Junction 11, the Pioneer Reservoir. Once Pioneer Reservoir is full, it will spill over its weir to the river outlet at Junction 13. When the water elevation in Junction 21 exceeds an elevation of 36.0 feet, wastewater spills to the river from the Flow Control Structure through Conduit 1225.

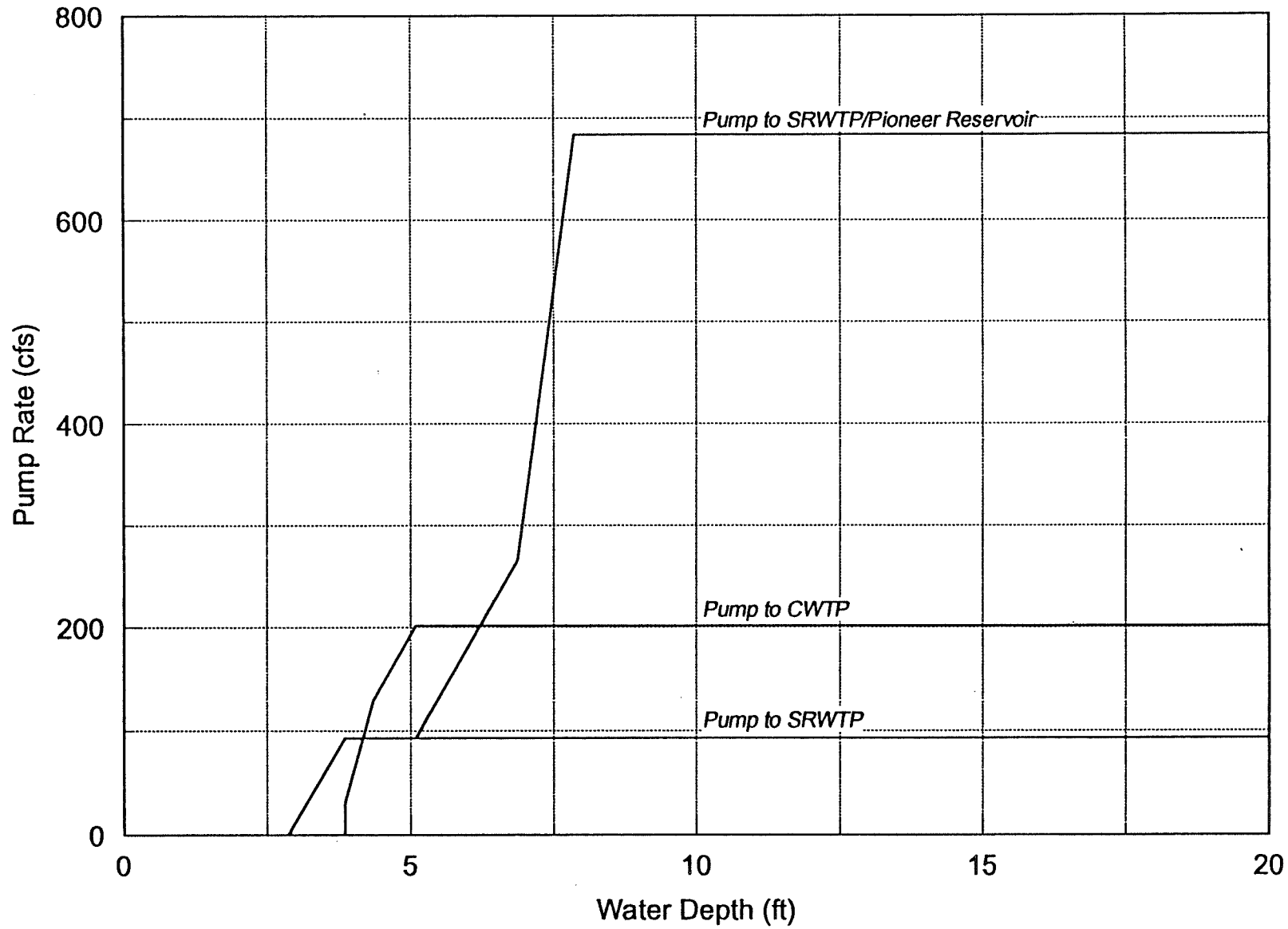
As indicated in Figure 3-4, this only occurs once Pioneer Reservoir is full and spilling from Conduit 1213, and the calculated flow rate being transferred from the Flow Control Structure to Pioneer Reservoir exceeds the design capacity of the Pioneer Interceptor.

H.2. Modeling Operations at the Pioneer Reservoir and Sump 1A

As with Sump 2, the previous versions of SSWMM91 EXTRAN setup provided only for a pumped discharge to a free outfall from Junction 10, Sump 1A. For the Phase 3 study, Pump 10-11 in Figure 3-4 simulates the operation of the pumps at Sump 1A, discharging directly into the Pioneer Reservoir. Water levels in Junction 10 which initiated the operation of Sump 1A were set according to the original Plan of Operations.

Pioneer Reservoir was simulated by a storage junction, Junction 11. The SSWMM91 requires input data on the incremental volume of available storage within the storage facility per foot of water depth. For model stability, the conduits connecting the storage junction had to be rectangular and a minimum of 500 feet long. The extra volume afforded by these conduits is not available in the actual storage structure so the modeled volume of the storage junction was adjusted accordingly.

The modeled configuration allows wastewater stored in Pioneer Reservoir to drain back to Junction 21, the Flow Control Structure, after the peak flows into Sump 2 have subsided below a rate of 60 mgd. Flows draining back to Junction 21 from Pioneer Reservoir are pumped only to SRWTP for secondary treatment. Pumping to



PUMP CURVES USED IN THE MODELING OF THE SUMP 2/PIONEER RESERVOIR SYSTEM

HDR

Figure 3-4

the CWTP stops once the total inflow into Junction 20 drops below the SRWTP treatment rate allowance of 60 mgd.

H.3. Use of Model Modifications

The modified model incorporating the Sump 2 and Pioneer Reservoir/Interceptor infrastructure provided an analytical tool used in Phase 3 to evaluate several aspects of the study. First, it allows SSWMM91 to report hydrographs for overflows to the river individually from Pioneer Reservoir and Sump 2 for a given rainstorm event. These hydrographs represent overflows that would occur, given the full utilization of all existing primary and secondary treatment capacity and Pioneer Reservoir.

For the treatment and storage alternatives at Pioneer Reservoir, it provided a means of identifying the total inflow hydrograph for the site essential for sizing the required facilities. The revised model setup was also useful in determining the period required for emptying the existing reservoir or a proposed storage reservoir or tunnel system.

SECTION 4
UNIT COSTS USED IN ESTIMATING TOTAL PROJECT COSTS

A. INTRODUCTION

The estimates of total project costs for the alternative improvement plans prepared for the Phase 2 study report were based on unit costs for various components of new infrastructure. These unit costs and the supporting assumptions for their use were presented in detail in Appendix H of the Phase 2 report entitled *Unit Construction Costs for Specific Alternative Projects*, dated July 1, 1991. As part of the Phase 3 study effort, some of the unit costs used in the Phase 2 study were revised. These revised unit costs and those which remained unaltered from Phase 2 were used in the preparation of the estimates of total project cost for the Phase 3 alternative improvement plans.

This section presents a review of the unit costs prepared for the Phase 2 study, identifies the recommended revisions to those costs and discusses why revisions to the Phase 2 unit costs were considered necessary. These unit costs are construction costs only and do not include project contingency costs, engineering design and construction administration costs. These other project costs are factored into the individual project cost estimate spreadsheets presented in Appendix E of this memorandum.

The costs presented in this section and in the remainder of this memorandum are planning-level costs based on the assumptions stated, without the benefit of site specific information normally collected in a pre-design effort. Actual construction costs are likely to differ from these estimated costs depending upon actual construction locations within the study area, seasonal groundwater elevation and restrictions placed on the progress of the work related to vehicle traffic control, environmental concerns, right-of-way, or other factors not yet identified.

B. UNIT COSTS FOR NEW SEWER CONSTRUCTION

Each of the alternative improvement plans considered for upgrading the CWCS requires the construction of new sewers. These sewers are either sanitary sewers, storm drain sewers or combined sewers. They include both gravity conveyance and force main systems. For this study, sewers 16 inches and smaller were classified as the collection system, and were not included in the Sacramento SWMM model. Sewers larger than 16 inches but less than 60 inches in diameter were classified as laterals and mains. Sewers over 60 inches were classified as interceptors. Laterals, mains and interceptors are all included in the SSWMM91 model.

Upgrades to the collection system required for specific design storms were determined from evaluations of representative basins, as described in detail in Appendix J of the Phase 3 report entitled *Combined Sewer Collection System Hydraulic Evaluation Improvements and Costs for the 2-, 5-, 10-, and 25-year Storms*, dated July 1, 1992. The estimates in that draft memorandum were revised, based on a re-evaluation of the total acreage to which per-acre unit costs would be applied. Upgrades to specific laterals, mains and interceptors were based on analysis using the SSWMM91.

Estimates of construction costs for all required sewer improvements were determined by applying unit costs for sewer construction to the total footage of new sewers.

The *Phase 2 Detailed Technical Report* presented a table of unit costs for new sewers based on published information from construction cost estimating guides, project experience and limited input from local pipeline construction contractors. In reviewing the Phase 2 report, the Regional Board had indicated that these unit costs may be too high. To improve on these costs for the Phase 3 report, an effort was made to solicit more input on construction costs from local pipeline contractors. Brierley and Lyman, Inc., geotechnical consultants to the Phase 2 study, were also requested to prepare construction cost estimates for trench dewatering. The revised unit construction costs were generally lower than the Phase 2 costs, most notably in the smaller diameters.

B.1. Phase 2 Assumptions

For purposes of establishing a unit construction cost per linear foot of new sewer, the Phase 2 study used the following assumptions.

- New sewers up to 144-inch diameter will be installed with conventional trenching and backfilling techniques. Larger conduits will be installed as tunnels.

- Pipeline construction costs include maintaining utility service to the residences and businesses within the study area throughout the course of the construction project. Limited service interruptions will be necessary, however, during reconnection of service laterals.

- New sewer pipe will be reinforced concrete pipe meeting ASTM C76, Class III, with rubber gasketed joints. The unit costs assumed do not include provision for cast-in-place PVC lining systems. PVC lining was not included because neither the City nor the County currently include this in their standard specifications.

- Traffic control restrictions would be imposed on the contractor, in order to limit periods of blocked or restricted roadway travel.

- Dewatering efforts include sedimentation basins or treatment for sediment removal prior to discharge to the storm sewer system. Local landscaping and trees would be protected during dewatering efforts.

- New pipelines sized 84 inches and larger include curb and gutter removal and replacement.

- The installed depth of the new sewers varied according to nominal diameter, with larger sewers laid deeper than smaller sewers.

- New manholes and junction boxes will be provided at a spacing interval of 300 feet.

B.2. Phase 3 Estimates of Unit Construction Costs for New Sewers

Table 4-1 presents the revised estimates of unit costs for new gravity sewer construction used for the Phase 3 report. Table 4-2 presents similar unit cost data for force main sewers. These unit cost data were used to estimate new sewer construction costs for all of the alternative improvement projects identified in Section 3.

B.3. Unit Cost Differences Between Phase 2 and Phase 3

The following summarizes the assumptions used in estimating the unit costs presented in Table 4-1 and 4-2. The differences between the Phase 2 and Phase 3 unit costs are discussed according to the cost items presented in Table 4-1 and Table 4-2.

B.3.a. Shoring, Cribbing and Bracing--

Unit costs for trench excavation reinforcement in the Phase 2 report were broken down into two categories: "Sheeting" and "Cribbing and Walers". This division was intended to make use of the readily available costs for both items of work. For Phase 2, the trench excavation reinforcement cost item represented a mixture of reinforcement techniques including using a trench box, wood sheeting, and soldier piles with wood or steel sheeting.

For Phase 3, trench reinforcement costs were represented as a single item: "Shoring, Cribbing, and Bracing". The trench reinforcement system assumed for the Phase 3 unit cost spreadsheet was a trench box for excavations up to a maximum of 15 feet deep. For deeper trenches, the 15-foot high trench box would be supplemented by either excavating the trench above the shield back at a 1:1 slope or, where the work area is limited, steel sheeting would be used above the shield.

TABLE 4-1
UNIT COSTS FOR INSTALLED NEW GRAVITY SEWER PIPE

DIAMETER (inches)	WALL THICKNESS (inches)	PIPE WEIGHT (lbs/ft)	AVERAGE INVERT ELEVATION (MSL)	SEWER INVERT DEPTH (FT)	SHORING/ CRIBBING/ BRACING ($/lf)	MANHOLES/ JUNCTION BOXES ($/LF)	MAINTAIN/ RELOCATE UTILITIES ($/lf)	EXCAVATION (cy/lf)	EXCAVATION ($/lf)
8	2.50	37.82	16.0	6	16.68	2.70	9.00	0.32	4.17
10	2.75	51.16	14.0	8	18.37	3.60	13.00	0.52	6.82
12	3.00	66.23	12.0	10	22.00	4.50	15.00	0.78	10.11
15	3.00	80.95	10.0	12	23.57	5.40	15.68	1.16	15.05
18	3.00	95.67	8.0	14	25.13	6.30	16.36	1.61	20.94
21	3.00	110.39	7.0	15	25.91	7.88	17.05	2.01	26.12
24	3.00	125.11	6.0	16	20.46	8.40	17.73	2.46	32.02
27	3.25	152.14	5.0	17	27.96	8.93	18.41	2.99	38.88
30	3.50	181.74	4.0	18	35.46	9.45	19.09	3.59	46.70
33	3.75	213.88	3.0	19	42.96	9.98	19.77	4.27	55.49
36	4.00	248.58	2.0	20	50.46	10.50	20.46	5.02	65.24
39	4.25	285.84	1.0	21	57.96	11.03	21.14	5.84	75.95
42	4.50	325.65	0.0	22	65.46	6.45	21.82	6.74	87.63
45	4.75	368.02	−1.0	23	72.96	7.11	22.50	7.71	100.27
48	5.00	412.94	−2.0	24	80.46	7.81	23.18	8.76	113.87
51	5.25	460.42	−2.5	25	84.21	8.55	23.86	9.54	124.04
54	5.50	510.45	−3.0	25	87.96	9.33	24.55	10.35	134.57
57	5.75	563.04	−3.5	26	91.71	10.16	25.23	11.19	145.47
60	6.00	618.19	−4.0	26	95.46	11.05	25.91	12.06	156.72
63	6.25	675.89	−4.5	27	99.21	11.99	26.59	12.95	168.34
66	6.50	736.14	−5.0	27	102.96	12.98	27.27	13.87	180.31
69	6.75	798.95	−5.5	28	106.71	14.03	27.96	14.82	192.65
72	7.00	864.32	−6.0	28	110.46	6.05	28.64	15.80	205.35
78	7.50	1002.71	−6.5	29	114.21	7.00	30.00	16.80	218.41
84	8.00	1151.33	−7.0	29	117.96	8.04	31.37	17.83	231.83
90	8.00	1229.83	−7.5	30	121.71	9.18	32.73	18.89	245.62
96	8.50	1393.58	−8.0	30	125.46	10.43	34.09	19.98	259.76
102	8.50	1476.99	−8.5	31	129.21	11.78	35.46	21.10	274.26
108	9.00	1655.86	−9.0	31	132.96	13.24	36.82	22.24	289.13
114	9.00	1744.17	−9.5	32	136.71	14.81	38.18	23.41	304.36
120	10.00	2044.27	−10.0	32	140.46	16.51	39.55	24.61	319.94
132	10.00	2240.52	−10.5	33	144.21	20.25	42.28	26.45	343.84
144	10.50	2562.90	−11.0	33	147.96	24.50	45.00	28.33	368.33

TABLE 4-1
UNIT COSTS FOR INSTALLED NEW GRAVITY SEWER PIPE

DIAMETER (inches)	DEWATERING ($/lf)	C76 PIPE FOB FACTORY ($/lf)	PIPE SHIPPING ($/lf)	PIPE BEDDING (cy/lf)	PIPE BEDDING ($/lf)	PIPE LAYING ($/lf)	LEAK, MISC. TESTING ($/lf)	BACKFILL (cy/lf)	BACKFILL ($/lf)
8	0.00	3.25	0.38	0.03	0.52	10.70	1.94	0.27	2.68
10	0.00	4.45	0.51	0.05	0.70	11.35	2.05	0.45	4.52
12	0.00	5.75	0.66	0.06	0.89	12.00	2.16	0.68	6.81
15	0.00	8.63	0.81	0.08	1.22	12.98	2.33	1.02	10.21
18	0.00	11.50	0.96	0.11	1.59	13.95	2.49	1.43	14.28
21	0.00	15.00	1.10	0.13	2.01	14.93	2.66	1.77	17.73
24	50.00	19.00	1.25	0.16	2.46	15.91	2.82	1.95	19.51
27	51.47	23.00	1.52	0.20	2.96	16.89	2.99	2.35	23.45
30	52.94	28.75	1.82	0.23	3.50	17.86	3.15	2.81	28.07
33	54.41	33.35	2.14	0.27	4.08	18.84	3.32	3.34	33.37
36	55.88	43.70	2.49	0.31	4.70	19.82	3.48	3.93	39.34
39	57.35	50.95	2.86	0.36	5.37	20.80	3.65	4.60	45.99
42	58.82	58.08	3.26	0.41	6.08	21.77	3.81	5.33	53.31
45	60.29	63.48	3.68	0.46	6.83	22.75	3.98	6.13	61.31
48	61.76	68.77	4.13	0.51	7.62	23.73	4.14	7.00	69.99
51	62.50	77.40	4.60	0.56	8.46	24.71	4.31	7.62	76.19
54	63.23	85.96	5.10	0.62	9.33	25.68	4.47	8.26	82.61
57	63.97	97.18	5.63	0.68	10.25	26.66	4.64	8.92	89.24
60	64.70	109.14	6.18	0.75	11.22	27.64	4.80	9.61	96.09
63	65.44	119.03	6.76	0.81	12.22	28.62	4.97	10.31	103.15
66	66.17	129.61	7.36	0.88	13.26	29.59	5.13	11.04	110.42
69	66.91	140.88	7.99	0.96	14.35	30.57	5.30	11.79	117.90
72	67.64	152.09	8.64	1.03	15.48	31.55	5.46	12.56	125.60
78	68.38	178.54	10.03	1.12	16.83	33.50	5.79	13.25	132.47
84	69.11	204.99	11.51	1.21	18.20	35.46	6.12	13.95	139.45
90	69.85	297.56	12.30	1.31	19.61	37.41	6.45	14.65	146.55
96	70.58	317.40	13.94	1.40	21.04	39.37	6.78	15.38	153.75
102	71.32	374.33	14.77	1.50	22.51	41.32	7.11	16.11	161.07
108	72.05	405.26	16.56	1.60	24.00	43.28	7.44	16.85	168.50
114	72.79	436.20	17.44	1.70	25.53	45.23	7.77	17.60	176.05
120	73.52	467.13	20.44	1.81	27.08	47.19	8.10	18.37	183.70
132	74.26	529.00	22.41	2.02	30.28	51.10	8.76	19.32	193.15
144	74.99	590.87	25.63	2.24	33.60	55.01	9.42	20.24	202.40

TABLE 4-1
UNIT COSTS FOR INSTALLED NEW GRAVITY SEWER PIPE

DIAMETER (inches)	ASPHALT BASE (cy/lf)	ASPHALT BASE ($/lf)	ASPHALT (ton/lf)	ASPHALT ($/lf)	CURB/ GUTTER (ft/lf)	CURB/ GUTTER ($/lf)	ACCESS & TRAFFIC CONTROL ($/lf)	DEMOB AND SITE CLEANUP ($/lf)	TOTAL CONSTRUCTION COST, PHASE 3 ($/lf)	TOTAL CONSTRUCTION COST, PHASE 2 ($/lf)
8	0.04	0.79	0.04	7.00	0	0	3	3.30	66.11	N/A
10	0.05	0.99	0.05	7.00	0	0	3	3.50	79.85	N/A
12	0.06	1.18	0.06	7.00	0	0	3	3.70	94.77	144.32
15	0.08	1.48	0.07	7.00	0	0	3	4.00	111.35	167.01
18	0.09	1.78	0.09	7.00	0	0	3	4.30	129.59	181.06
21	0.11	2.07	0.10	7.00	0	0	3	4.60	147.05	197.15
24	0.15	2.96	0.15	9.00	0	0	3	4.90	209.41	209.07
27	0.20	3.85	0.19	9.00	0	0	3	5.20	237.49	226.55
30	0.25	4.73	0.24	9.00	0	0	3	5.50	269.03	241.05
33	0.29	5.62	0.28	10.00	0	0	3	5.80	302.12	260.17
36	0.34	6.51	0.33	10.00	0	0	3	6.10	341.67	342.11
39	0.38	7.40	0.37	10.00	0	0	3	6.40	379.83	383.48
42	0.43	8.28	0.42	12.00	0	0	3	6.70	416.47	407.44
45	0.48	9.17	0.46	12.00	0	0	3	7.00	456.33	435.30
48	0.52	10.06	0.51	13.00	0	0	5	7.30	500.82	460.47
51	0.55	10.65	0.54	13.00	0	0	5	7.60	535.07	484.15
54	0.58	11.24	0.57	14.00	0	0	5	7.90	570.94	508.50
57	0.61	11.84	0.60	14.00	0	0	5	8.20	609.17	539.88
60	0.65	12.43	0.63	15.00	0	0	5	8.50	649.83	567.13
63	0.68	13.02	0.66	15.00	0	0	5	8.80	688.11	592.80
66	0.71	13.61	0.69	16.00	0	0	10	9.10	733.78	629.30
69	0.74	14.20	0.72	17.00	0	0	10	9.40	775.84	622.12
72	0.77	14.79	0.75	18.00	0	0	10	9.70	809.46	694.11
78	0.80	15.39	0.78	20.00	0	0	10	10.30	870.84	749.19
84	0.83	15.98	0.81	21.00	1	18	10	10.90	949.92	842.29
90	0.86	16.57	0.84	23.00	1	18	10	11.50	1078.02	953.06
96	0.89	17.16	0.87	25.00	1	18	10	12.10	1134.86	1003.91
102	0.92	17.75	0.90	27.00	1	18	15	12.70	1233.58	1185.11
108	0.95	18.34	0.93	30.00	1	18	15	13.30	1303.88	1260.95
114	0.98	18.94	0.96	33.00	1	18	15	13.90	1373.90	1335.31
120	1.01	19.53	0.99	36.00	1	18	15	14.50	1446.65	1421.60
132	1.06	20.42	1.03	39.00	1	18	15	15.70	1567.63	1544.93
144	1.11	21.30	1.08	42.00	1	18	15	16.90	1690.91	1724.39

TABLE 4-2

UNIT COSTS FOR INSTALLED NEW FORCE MAIN SEWER PIPE

DIAMETER (inches)	WALL THICKNESS (inches)	PIPE WEIGHT (lbs/ft)	AVERAGE INVERT ELEVATION (MSL)	SEWER INVERT DEPTH (FT)	SHORING/ CRIBBING/ BRACING ($/lf)	MANHOLES/ JUNCTION BOXES ($/LF)	MAINTAIN/ RELOCATE UTILITIES ($/lf)	EXCAVATION (cy/lf)	EXCAVATION ($/lf)
12	3.00	66.23	17.0	5	22.00	2.25	15.00	0.41	5.30
15	3.00	80.95	16.8	5	22.23	2.36	15.68	0.53	6.92
18	3.00	95.67	16.5	6	22.45	2.48	16.36	0.67	8.67
21	3.00	110.39	16.3	6	22.68	3.02	17.05	0.81	10.53
24	3.00	125.11	16.0	6	22.91	3.15	17.73	0.96	12.52
27	3.25	152.14	15.8	6	23.14	3.28	18.41	1.13	14.63
30	3.50	181.74	15.5	7	23.36	3.41	19.09	1.30	16.85
33	3.75	213.88	15.3	7	23.59	3.54	19.77	1.48	19.20
36	4.00	248.58	15.0	7	23.82	3.68	20.46	1.67	21.67
39	4.25	285.84	14.8	7	24.05	3.81	21.14	1.87	24.25
42	4.50	325.65	14.5	8	24.27	5.91	21.82	2.07	26.96
45	4.75	368.02	14.3	8	24.50	6.54	22.50	2.29	29.79
48	5.00	412.94	14.0	8	24.73	7.21	23.18	2.52	32.74
51	5.25	460.42	13.8	8	24.95	7.94	23.86	2.75	35.81
54	5.50	510.45	13.5	9	25.18	8.71	24.55	3.00	39.00
57	5.75	563.04	13.3	9	25.41	9.53	25.23	3.25	42.31
60	6.00	618.19	13.0	9	25.64	10.41	25.91	3.52	45.74
63	6.25	675.89	12.8	9	25.86	11.34	26.59	3.79	49.29
66	6.50	736.14	12.5	10	26.09	12.32	27.27	4.07	52.96
69	6.75	798.95	12.3	10	26.32	13.36	27.96	4.37	56.75
72	7.00	864.32	12.0	10	26.55	5.78	28.64	4.67	60.67
78	7.50	1002.71	11.5	11	27.00	6.73	30.00	5.09	66.20
84	8.00	1151.33	11.0	11	27.45	7.77	31.37	5.54	71.98
90	8.00	1229.83	10.5	12	27.91	8.91	32.73	6.00	78.00
96	8.50	1393.58	10.0	12	28.36	10.16	34.09	6.48	84.26
102	8.50	1476.99	9.5	13	28.82	11.51	35.46	6.98	90.76
108	9.00	1655.86	9.0	13	29.27	12.97	36.82	7.50	97.50
114	9.00	1744.17	8.5	14	29.73	14.54	38.18	8.04	104.48
120	10.00	2044.27	8.0	14	30.18	16.24	39.55	8.59	111.70
132	10.00	2240.52	7.0	15	31.09	19.99	42.28	9.76	126.87
144	10.50	2562.90	6.0	16	32.00	24.25	45.00	11.00	143.00

BLE 4-2

UNIT COSTS FOR INSTALLED NEW FORCE MAIN SEWER PIPE

DIAMETER (inches)	DEWATERING ($/lf)	C76 PIPE FOB FACTORY ($/lf)	PIPE SHIPPING ($/lf)	PIPE BEDDING (cy/lf)	PIPE BEDDING ($/lf)	PIPE LAYING ($/lf)	LEAK, MISC. TESTING ($/lf)	BACKFILL (cy/lf)	BACKFILL ($/lf)	ASPHALT BASE (cy/lf)
12	0.00	5.75	0.66	0.06	0.89	12.00	2.16	0.31	3.11	0.06
15	0.00	8.63	0.81	0.08	1.22	12.98	2.33	0.40	3.96	0.08
18	0.00	11.50	0.96	0.11	1.59	13.95	2.49	0.48	4.83	0.09
21	0.00	15.00	1.10	0.13	2.01	14.93	2.66	0.57	5.74	0.11
24	0.00	19.00	1.25	0.16	2.46	15.91	2.82	0.67	6.67	0.12
27	0.00	23.00	1.52	0.20	2.96	16.89	2.99	0.76	7.63	0.14
30	0.00	28.75	1.82	0.23	3.50	17.86	3.15	0.86	8.62	0.15
33	0.00	33.35	2.14	0.27	4.08	18.84	3.32	0.96	9.64	0.17
36	0.00	43.70	2.49	0.31	4.70	19.82	3.48	1.07	10.68	0.18
39	0.00	50.95	2.86	0.36	5.37	20.80	3.65	1.18	11.76	0.20
42	0.00	58.08	3.26	0.41	6.08	21.77	3.81	1.29	12.86	0.22
45	0.00	63.48	3.68	0.46	6.83	22.75	3.98	1.40	13.99	0.23
48	0.00	68.77	4.13	0.51	7.62	23.73	4.14	1.51	15.14	0.25
51	0.00	77.40	4.60	0.56	8.46	24.71	4.31	1.63	16.33	0.26
54	0.00	85.96	5.10	0.62	9.33	25.68	4.47	1.75	17.54	0.28
57	0.00	97.18	5.63	0.68	10.25	26.66	4.64	1.88	18.78	0.29
60	0.00	109.14	6.18	0.75	11.22	27.64	4.80	2.01	20.05	0.31
63	0.00	119.03	6.76	0.81	12.22	28.62	4.97	2.14	21.35	0.32
66	0.00	129.61	7.36	0.88	13.26	29.59	5.13	2.27	22.68	0.34
69	0.00	140.88	7.99	0.96	14.35	30.57	5.30	2.40	24.03	0.35
72	0.00	152.09	8.64	1.03	15.48	31.55	5.46	2.54	25.41	0.37
78	0.00	178.54	10.03	1.12	16.83	33.50	5.79	2.37	23.72	0.38
84	0.00	204.99	11.51	1.21	18.20	35.46	6.12	2.48	24.82	0.40
90	0.00	297.56	12.30	1.31	19.61	37.41	6.45	2.59	25.94	0.42
96	0.00	317.40	13.94	1.40	21.04	39.37	6.78	2.71	27.09	0.43
102	0.00	374.33	14.77	1.50	22.51	41.32	7.11	2.82	28.25	0.45
108	0.00	405.26	16.56	1.60	24.00	43.28	7.44	2.94	29.43	0.46
114	0.00	436.20	17.44	1.70	25.53	45.23	7.77	3.06	30.63	0.48
120	0.00	467.13	20.44	1.81	27.08	47.19	8.10	3.19	31.85	0.49
132	0.00	529.00	22.41	2.02	30.28	51.10	8.76	3.44	34.35	0.52
144	45.00	590.87	25.63	2.24	33.60	55.01	9.42	3.69	36.94	0.55

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UNIT COSTS FOR INSTALLED NEW FORCE MAIN SEWER PIPE

DIAMETER (inches)	ASPHALT BASE ($/lf)	ASPHALT (ton/lf)	ASPHALT ($/lf)	CURB/ GUTTER (ft/lf)	CURB/ GUTTER ($/lf)	ACCESS & TRAFFIC CONTROL ($/lf)	DEMOB AND SITE CLEANUP ($/lf)	TOTAL CONSTRUCTION COST, PHASE 3 ($/lf)	TOTAL CONSTRUCTION COST, PHASE 2 ($/lf)
12	1.18	0.06	7.00	0	0	3	3.70	84.01	144.32
15	1.48	0.07	7.00	0	0	3	4.00	92.59	167.01
18	1.78	0.09	7.00	0	0	3	4.30	101.36	181.06
21	2.07	0.10	7.00	0	0	3	4.60	111.39	197.15
24	2.37	0.12	9.00	0	0	3	4.90	123.69	209.07
27	2.66	0.13	9.00	0	0	3	5.20	134.30	226.55
30	2.96	0.15	9.00	0	0	3	5.50	146.88	241.05
33	3.25	0.16	10.00	0	0	3	5.80	159.52	260.17
36	3.55	0.18	10.00	0	0	3	6.10	177.14	342.11
39	3.85	0.19	10.00	0	0	3	6.40	191.87	383.48
42	4.14	0.21	12.00	0	0	3	6.70	210.66	407.44
45	4.44	0.22	12.00	0	0	3	7.00	224.47	435.30
48	4.73	0.24	13.00	0	0	5	7.30	241.43	460.47
51	5.03	0.25	13.00	0	0	5	7.60	259.00	484.15
54	5.33	0.27	14.00	0	0	5	7.90	277.76	508.50
57	5.62	0.28	14.00	0	0	5	8.20	298.45	539.88
60	5.92	0.30	15.00	0	0	5	8.50	321.15	567.13
63	6.21	0.31	15.00	0	0	5	8.80	341.04	592.80
66	6.51	0.33	16.00	0	0	10	9.10	367.90	629.30
69	6.81	0.34	17.00	0	0	10	9.40	390.72	662.12
72	7.10	0.36	18.00	0	0	10	9.70	405.07	694.11
78	7.40	0.37	20.00	0	0	10	10.30	446.04	749.19
84	7.69	0.39	21.00	0	0	10	10.90	489.27	842.29
90	7.99	0.40	23.00	0	0	10	11.50	599.31	953.06
96	8.28	0.42	25.00	0	0	10	12.10	637.87	1003.91
102	8.58	0.43	27.00	0	0	15	12.70	718.11	1185.11
108	8.88	0.45	30.00	0	0	15	13.30	769.71	1260.95
114	9.17	0.46	33.00	0	0	15	13.90	820.81	1335.31
120	9.47	0.48	36.00	0	0	15	14.50	874.43	1421.60
132	10.06	0.51	39.00	0	0	15	15.70	975.88	1544.93
144	10.65	0.54	42.00	0	0	15	16.90	1125.27	1724.39

The costs for trench reinforcement for Phase 3 are generally lower than for the Phase 2 study, reflecting the limited use of steel sheeting and the economy of using a trench shield system where possible.

B.3.b. Manholes, Junction Boxes--

Manhole and utility relocation costs were separated for the revised Phase 3 unit costs. For Phase 2, unit costs for manholes were based on an average 300-foot manhole spacing interval. This cost was lumped in with an estimate for relocating existing utilities.

For Phase 3, manhole costs were based on the following assumptions:

1) Manhole spacing for sewers ranging from 12 inches to 72 inches in diameter at 400 feet on center, and spacing for larger sewers at 1,000 feet. The 400-foot spacing interval is a common limitation for the hydraulic jetting equipment used for flushing out smaller gravity sewer systems. For larger sewers (72-inch and larger), manhole spacing can be increased because maintenance personnel can physically walk inside of the pipe to clean out debris and flush out sediment with hand-held hoses.

2) 48-inch diameter manholes were used for sewers up to 21 inches nominal diameter. For sewer sizes in the range of 21 to 42 inches, a 60-inch diameter manhole was used. For 48-inch and larger sewers, a saddle manhole configuration with a top-mounted 48-inch manhole for personnel access was used.

3) Unit costs for 48-inch manholes were $180/vertical foot, plus $500. 60-inch manholes were $210/vertical-foot plus $800, and saddle manholes were a combination of the cost for a 48-inch access manhole and a cost for the cast-in-place work for the lower part of the structure. This configuration conforms with City design standards.

B.3.c. Maintain, Relocate Utilities--

For Phase 3, a separate cost item was identified for "maintain, relocate utilities". This cost was separated from "manhole/junction structure" costs. The

rationale for this separation was that costs associated with relocation of existing utilities were not a function of any of the variables listed in the Phase 2 unit cost spreadsheet.

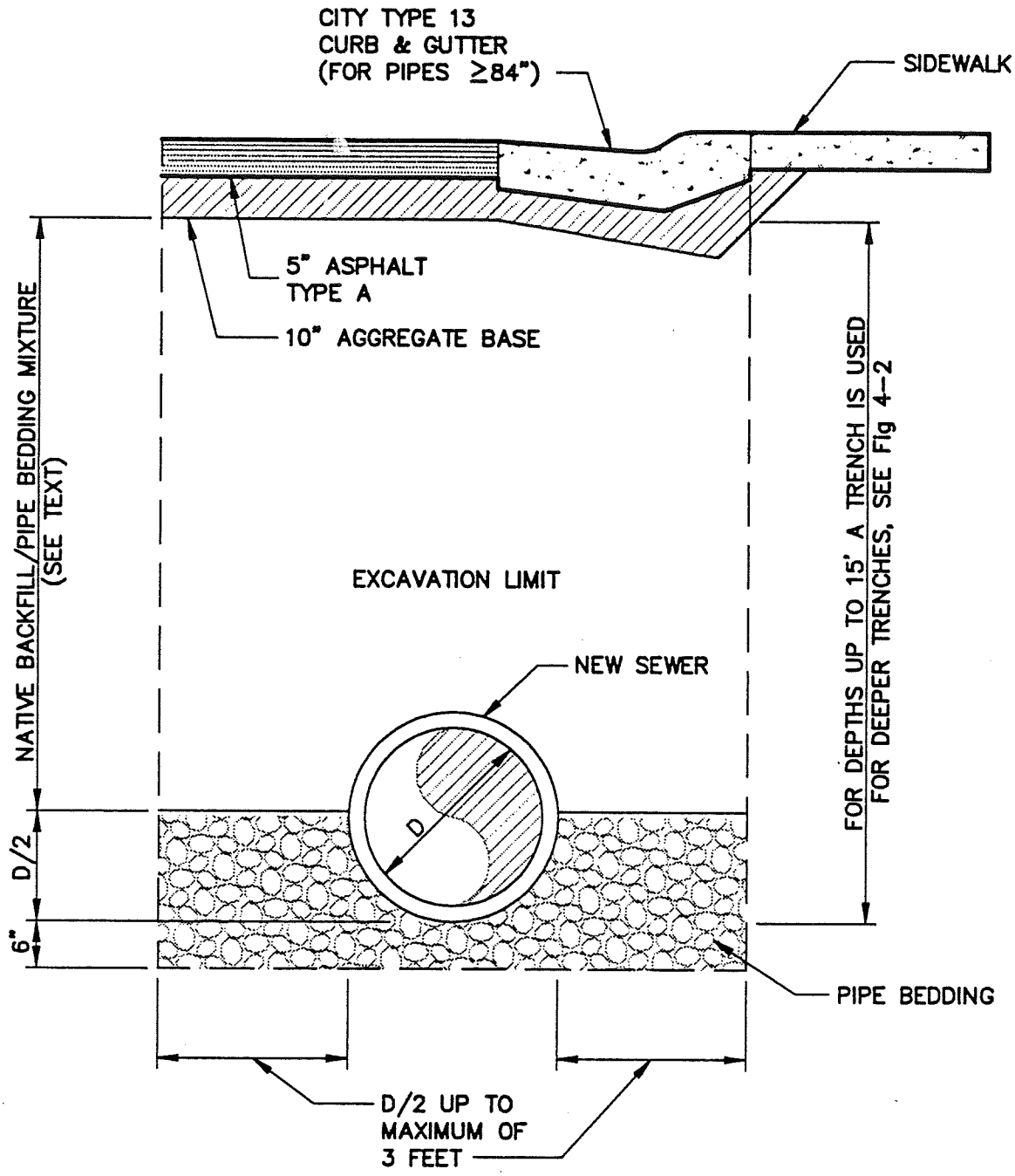
This component of pipeline construction cost is a function of the proximity and interference of existing utilities within the work area. To assign an average cost per unit length of sewer for this item of work is much more speculative than estimating the cost of new sewer manholes/junction structures. Consequently, the two cost items have been separated.

B.3.d. Excavation--

Excavation quantities for Phase 2 were based on a different trench configuration than Phase 3. The excavated trench configuration for Phase 2 was a vertical wall trench with shoring and reinforcement as required to make the work area safe.

The trench configuration assumed for Phase 3 for trench depths between 0 and 15 feet is illustrated in Figure 4-1. It assumed the trench reinforcement system to be a trench box, also referred to as a trench shield. The actual depth limit where trench boxes are useful depends upon the character of the soil. Where soils have been previously disturbed by buried utility construction, or where the soils are low in cohesion and tend to slough, the friction on the box becomes too great to move it, and supplemental reinforcement or excavation is required.

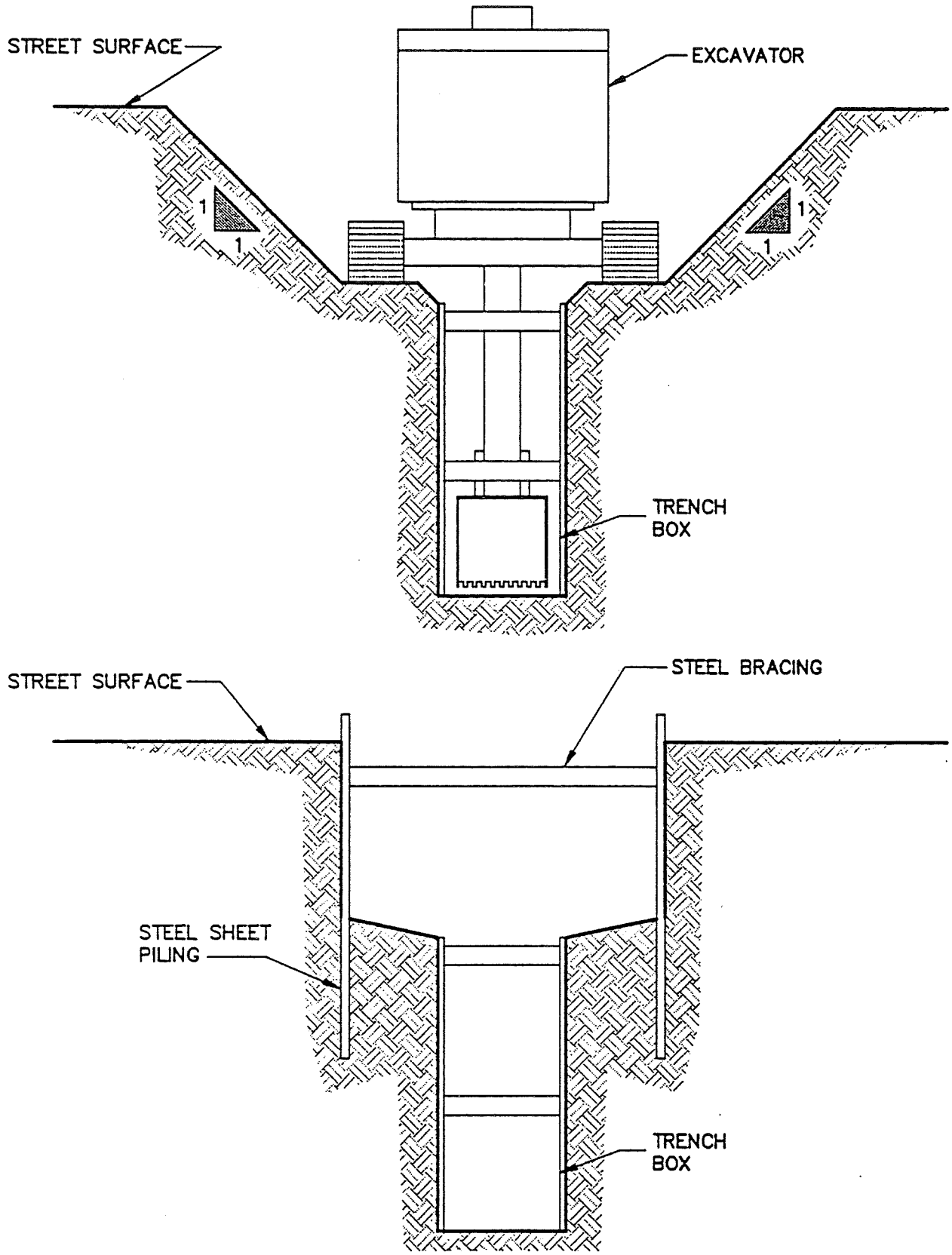
For deeper trenches, the assumed excavation configuration is illustrated in Figure 4-2. Two options were considered: one was to slope the upper portion of the trench back at a 1:1 slope above a 15-foot high trench shield. The other was to use steel sheeting above the shield. These revised trench configurations were based on discussions with local contractors on current construction practices.



TYPICAL TRENCH DETAIL

HDR

Figure 4-1



METHODS OF DEEP TRENCH REINFORCEMENT

HDR

Figure 4-2

Existing sewer removal, where required, was included in the revised excavation costs. In some cases, removal of existing sewers has been difficult for contractors for two reasons. One is that the existing pipe may not come out of the excavation easily with conventional backhoe excavators, and consequently, there is additional cost involved. The other reason is that soils excavated from the trench are normally removed and stored off the construction site while pipe laying is in progress. This native excavated material, if determined to be suitable, would normally be returned to the site and used as backfill for the excavated trench. However, experience has shown that there is a good chance that the existing pipe will crumble into rubble and become mixed into the excavated material, especially the older brick sewers. This not only makes the excavated material unsuitable for trench backfill but also requires that it be classified as a contaminated soil waste requiring special disposal procedures.

The Phase 2 unit costs used for excavation were in the range of $15 to $20 per cubic yard (cy). Based on the experience of local contractors, these estimates were high. More accurate costs (in two components) are on the order of a base cost of $3/cy, plus trucking for excavated material which is suitable as backfill. Total unit costs for excavation were therefore revised to $13/cy which is an average of $6/cy for material which can be used as backfill, and $20/cy for material requiring special disposal procedures as a contaminated soil.

B.3.e. Dewatering--

The dewatering costs for Phase 2 ranged from $5 to $36 per lineal foot of new sewer pipe. This was based on one contractor's experience with pipe installed at shallower depths. The Phase 3 costs for dewatering range from $50 to $75 per lineal foot. These costs are the result of a geotechnical evaluation of construction dewatering requirements and costs by Brierley and Lyman Inc. Their evaluation and recommendations include wells drilled 100 feet on-center, in a line parallel to proposed sewer alignments.

Each well would deliver 75 gpm and be operational for a total of one month. No well points would be used in the trench, as was assumed for Phase 2. Water pumped from the wells would be treated for sediment removal prior to placement into the combined sewer system. The water table was assumed to be an average of 15 feet

below the surface. Local contractor experience indicates that water table elevations are higher nearer the river, and lower further away.

For the combined sewer service area, the water table is more influenced by river stage than proximity to the rivers, except for the corridor within roughly 3 blocks of the American and Sacramento Rivers. Given this information, we used an average water table elevation for the entire study area rather than subdividing the area into sub-areas with different dewatering characteristics. Shallow excavations no deeper than 15 feet did not include dewatering costs. Details of the analysis of dewatering construction costs provided by Brierley and Lyman, Inc. are provided in Appendix A of this memorandum.

B.3.f. C76 Pipe--

Phase 2 pipe prices were quoted directly from a local concrete pipe manufacturer for ASTM C76 Class III pipe. For the Phase 3 report, Class IV pipe was considered for invert depths over 19 feet. Class IV pipe may also be required when there is limited cover over new sewers. However, for the smallest pipe sizes the assumed depth of installation has sufficient cover to allow use of Class III pipe. Pipe costs were increased by 15 percent for those pipes over 30 inches in diameter to account for the Class IV designation.

B.3.g. Pipe Bedding--

The quantity of pipe bedding is different for Phase 2 and Phase 3. The trench configuration for Phase 2 assumed a total pipe bedding depth consisting of over-excavation equivalent to 33 percent of the pipe diameter and backfill with bedding up over the pipe invert to a level equal to 15 percent of the pipe diameter. By comparison, the trench configuration used for Phase 3 included over-excavation of only 6 inches and bedding brought up to the pipe spring line. This bedding configuration is representative of Type B granular foundation bedding and is considered compatible with local area soil conditions within the study area.

Unit costs for pipe bedding were revised from a Phase 2 estimate of $12 per cubic yard in place to $15/cy for Phase 3. This revision is based on the recommendations of local contractors.

B.3.h. Pipe Laying--

The costs for pipe laying presented for Phase 2 were based on updating estimates of project costs developed by a consultant to the City for the Pioneer Interceptor project in 1977 using the ENR Construction Cost Index. These costs were revised for Phase 3 on the recommendations of local sewer contractors.

B.3.i. Leak and Miscellaneous Testing--

For the Phase 2 effort, estimates for leak detection were based on hydrostatic testing whereby the pipe would be filled with water and the standing water level checked after a period of time for a change in elevation related to exfiltration. The Phase 3 costs for this item were revised downward at the suggestion of local sewer contractors who said individual pipe joint testing was considered an acceptable substitute to hydrostatic testing by local City and County construction contract administrators.

B.3.j. Backfill--

Estimates of backfill quantities changed from Phase 2 to Phase 3 due to the change in assumed trench configuration.

Unit costs for backfill for Phase 2 were taken to be $1.50/cy on the assumption that native material could be used as fill. For Phase 3, unit backfill costs are $10/cy (which is an average of $5/cy for suitable native fill and $15/cy for placing imported fill from a distant source). Phase 3 backfill costs assumed importation of select material to replace rubble and contaminated soil resulting from the removal of existing sewers.

B.3.k. Asphalt Base--

A new unit cost item was created for roadway asphalt base for Phase 3. Phase 2 neglected to account for the replacement of the road base beneath the asphalt paving. For Phase 3, a 10-inch thick aggregate base beneath a 5-inch thick Type A asphalt surfacing was used. This structural section complies with state and local road design requirements for urban roads and traffic loads.

B.3.l. Asphalt--

Quantities of asphalt varied with trench configuration between the Phase 2 and

Phase 3 studies. Both assumed a 5-inch paving thickness but the Phase 3 trench required wider trenches for deeper excavations where the upper portion of the trench was sloped back at a 1:1 slope.

Unit costs for asphalt used in Phase 2 were $66/ton. For Phase 3 a graduated unit cost was used to account for higher in place costs for small quantities of asphalt. The Phase 3 costs ranged from $42 to $116/ton.

B.3.m. Curb and Gutter, Demobilization, Site Clean-Up--

Unit construction costs for "curb an gutter" were revised from Phase 2 to Phase 3 based on discussions with concrete contractors. Phase 2 assumed a per lineal foot cost of $33/lf and Phase 3 used $18/lf.

Similarly, unit costs for de-mobilization and site cleanup were revised downward from Phase 2 to Phase 3 after review and discussion with local contractors.

C. UNIT COSTS FOR NEW SEWER SERVICES

In order to prepare estimates of cost for systemwide improvement plans, it was necessary to address the required upgrades for the collection system (i.e. sewers 15 inches and smaller in diameter). A portion of the overall cost of the collection system upgrade was that associated with the removal and replacement of the existing sewer service laterals which connect the individual customer's drain piping to the City's conveyance system.

In instances where the existing combined sewers in the street were replaced or paralleled, the majority of these sewer service connections would be disturbed or destroyed and would require replacement. Under the current City sewer ordinances, the City is responsible for these services from the street to the back side of the sidewalks. This is the segment of service lateral which would have to be replaced as part of the overall sewer upgrade program.

The unit construction costs presented in this section were used in the preparation of collection system improvement costs presented in the technical memorandum entitled *Combined Sewer Collection System Hydraulic Evaluation,*

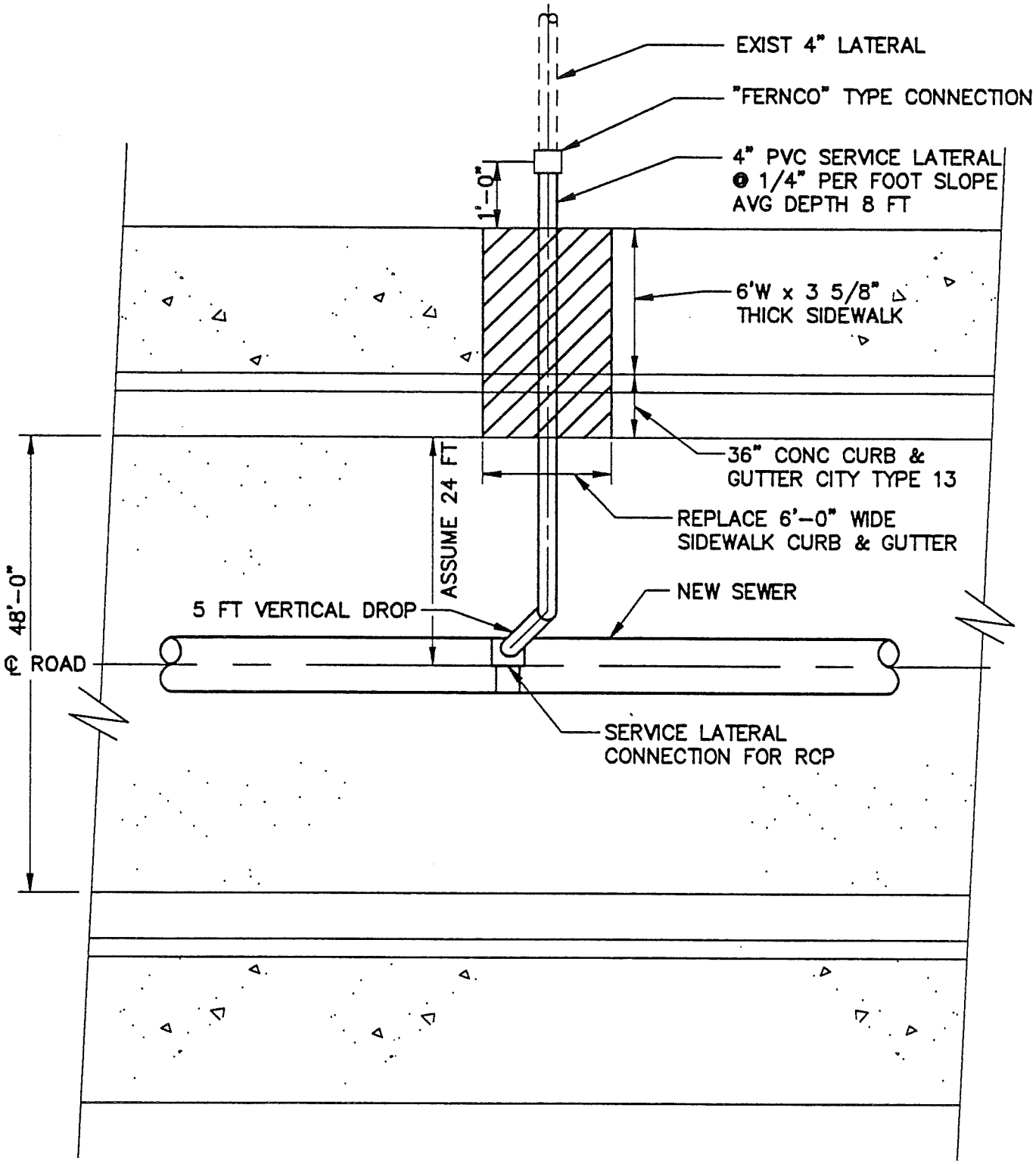
Improvements and Costs for the 2-, 5-, 10-, and 25-Year Storms, dated July 1, 1992.

C.1. Assumptions

For the purpose of preparing a cost estimate for a representative, new service connection, a typical sewer layout had to be defined. The configuration used for the cost estimate is presented in Figure 4-3. As indicated by the figure, the paved street width is 48 feet with the sewer coincident with the street center line. Laterals were assumed to be 4-inch PVC at an average depth of 8 feet below grade and run at a minimum slope of 0.02 ft/ft. It was further assumed that the lateral connection included a vertical drop of 5 feet at the point of connection to the street sewer and that the connection included a 1/4 bend and saddle connection on the sewer pipe. The street sewer was assumed to be 15 inches in diameter. The removal and replacement of the curb/gutter, sidewalk, asphalt paving and pavement base was assumed to be part of the cost of the lateral replacement.

C.2. Estimated Costs

Estimated costs for the removal and replacement of service laterals in accordance with the assumptions stated above and shown in Figure 4-3 are presented in Table 4-3. The total estimated cost for sewer service replacement is $2,450 per customer.



NEW SEWER SERVICE LATERAL DETAIL

HDR

Figure 4-3

TABLE 4-3. UNIT COSTS FOR SEWER SERVICE LATERALS				
Description	Quantity	Units	Unit Cost ($)	Total ($)
Concrete Cutting and Removal	All Req'd	LS	$110	$110
Excavation	20	CY	$6.00	$120
Pipe Bedding	1.2	CY	$15.00	$18
New Sewer Lateral Materials	All Req'd	LS	$125	$125
Pipe Laying, Labor and Equipment	All Req'd	LS	$850	$850
Backfill	17	CY	$5.00	$85
Asphalt Base	1.3	CY	$19.25	$25
Asphalt Paving	0.75	CY	$150.00	$113
New Curb and Gutter	54	SF	$6.25	$338
Supervision	All Req'd	LS	$178.0	$178
Subtotal				$1,962
Overhead and Profit				$ 491
Total Estimated Cost				$2,453

D. UNIT COSTS FOR NEW STORM DRAINAGE DROP INLETS

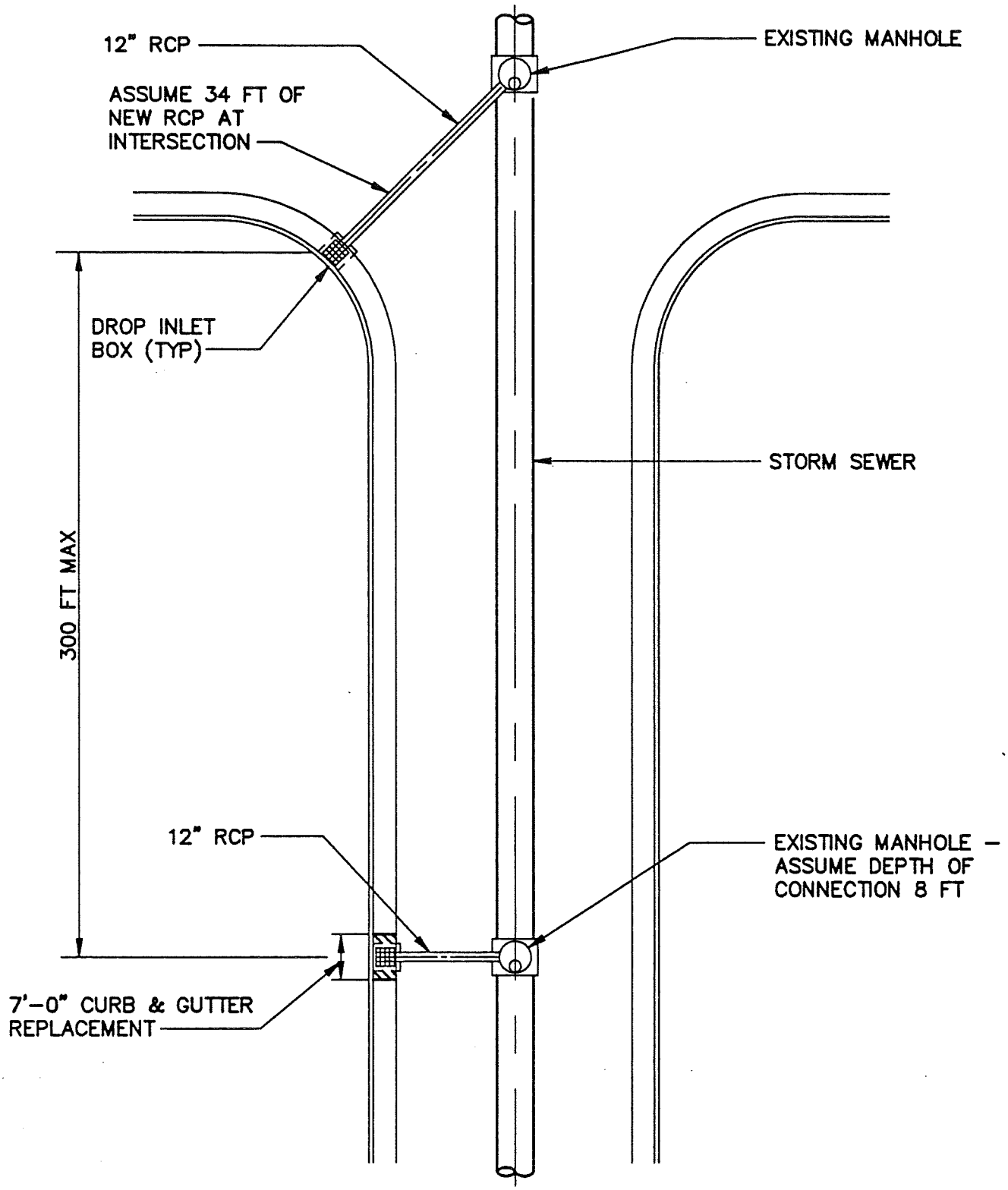
With the exception of the separation alternative involving the construction of a totally new sanitary sewage collection system, alternative improvement projects would involve the construction of new storm drainage inlets and connecting drain lines. These new facilities would be necessary to replace the existing infrastructure which is no longer functional or to provide additional relief for surface flooding during major storm events.

The following paragraphs present estimates of unit construction costs associated with the installation of replacement drop inlets and drop inlet drain lines. As with the unit costs for new sewer laterals, these costs are used in the development of collection system upgrade costs presented in the technical memorandum entitled *Combined Sewer Collection System Hydraulic Evaluation, Improvements and costs for the 2-, 5-, 10- and 25-Year Storms*, dated July 1, 1992.

D.1. Assumptions

The configuration of drop inlets and street drainage used in developing estimates of construction costs is presented in Figure 4-4. Drop inlet boxes are assumed to be City of Sacramento Type B drop inlets. Drop inlet drain lines are all assumed to be nominal 12 inches in diameter, reinforced concrete pipe. Connection to the street sewer is via a manhole. The curb and gutter is assumed to be City of Sacramento #13. Asphalt paving is 5 inches thick with a 10-inch base. Excavated native material is assumed to be suitable as backfill.

To provide a unit cost per drop inlet added, two types of drop inlet are reviewed, one at an intersection and one at the gutter between street intersections. New drop inlets are assumed to be constructed in pairs. Figure 4-4 shows both types of drop inlets.



TYPICAL DI CONNECTION DETAIL

HDR

Figure 4-4

D.2. Estimated Costs

Costs for the drop inlets shown in Figure 4-4 are presented in Table 4-4 and Table 4-5. For purposes of establishing a drop inlet installation unit construction cost, the average of the two costs, $2,600, was used to estimate the costs for new drop inlets.

TABLE 4-4. UNIT COSTS FOR DROP INLETS - MID STREET

Description	Quantity	Units	Unit Cost ($)	Total ($)
Cut and Remove Existing Curb and Gutter	All Req'd	LS	$50.00	$50
Cut and Remove Existing Paving	All Req'd	LS	$60.00	$60
Excavation	11	CY	$6.00	$66
Pipe Bedding	1	CY	$15.00	$15
New RCP, Related Materials	25	LF	$5.75	$144
Pipelaying Labor and Equip	All Req'd	LS	$850.00	$850
Backfill	8	CY	$5.00	$40
Asphalt Base	1.67	CY	$19.25	$32
Asphalt Paving	0.74	CY	$150.00	$111
New Curb and Gutter Section	9	SF	$6.00	$54
New Drop Inlet With Curb Hood and Grate	1	EA	$350.00	$350
Supervision	All Req'd	LS	$177.00	$177
Subtotal				$1949
Overhead and Profit				$487
Total Estimated Cost				$2436

TABLE 4-5. UNIT COSTS FOR DROP INLETS - STREET INTERSECTION				
Description	Quantity	Units	Unit Cost ($)	Total($)
Cut and Remove Existing Curb and Gutter	All Req'd	LS	$50.00	$50
Cut and Remove Existing Paving	All Req'd	LS	$85.00	$85
Excavation	16	CY	$6.00	$96
Pipe Bedding	1.5	CY	$15.00	$23
New RCP, Related Materials	35	LF	$5.75	$201
Pipelaying Labor and Equip	All Req'd	LS	$850.00	$850
Backfill	11	CY	$5.00	$55
Asphalt Base	2.3	CY	$19.25	$44
Asphalt Paving	1.05	CY	$150.00	$158
New Curb and Gutter Section	9	SF	$6.00	$54
New Drop Inlet With Curb Hood and Grate	1	EA	$350.00	$350
Supervision	All Req'd	LS	$197.00	$197
Subtotal				$2163
Overhead and Profit				$540
Total Estimated Cost				$2703

E. UNIT COSTS FOR PUMPING STATIONS

Unit costs for pumping stations used for the Phase 2 report were generated using the process cost estimating program entitled "WATER CO$T" developed by Culp-Wesner-Culp (HDR Engineering, Inc.) in 1986. These cost data are reviewed below for use in the Phase 3 study.

E.1. Assumptions

As described in the Phase 2 report, the WATER CO$T program provides estimates of construction cost for water and wastewater processes. Input required for the program includes current labor and materials cost indices published by the US Bureau of Labor Statistics and the ENR. These indices have been updated for this Phase 3 report using October 1991 data and are listed in Table 4-6.

TABLE 4-6. COST INDICES USED IN WATER CO$T INPUT, OCTOBER 1991

Index	Value	Source
General Purpose Mach.	389.1	BLS 114[1]
Concrete	366.8	BLS 132
Steel	376.9	BLS 1017
Skilled Labor	452.5	ENR[2]
Pipe/Valve	446.6	BLS 1149
Electrical	279.5	BLS 117
Housing	412.2	ENR
Housing	150.0	Local Unit Cost
Producer	343.2	Price Index

1 US Bureau of Labor Statistics
2 Engineering News Record

The cost estimating program provides a "base" construction cost for a particular process. The base construction cost for pumping stations computed by the program assumes a pumping station configured with screening, a wet well separated from

the dry well, pumping equipment, piping within the structure limits, electrical power distribution, lighting, equipment controls, heating and ventilating, and other auxiliary equipment including hoists and elevators.

For purposes of this study, the pumping station construction costs also included the following items:

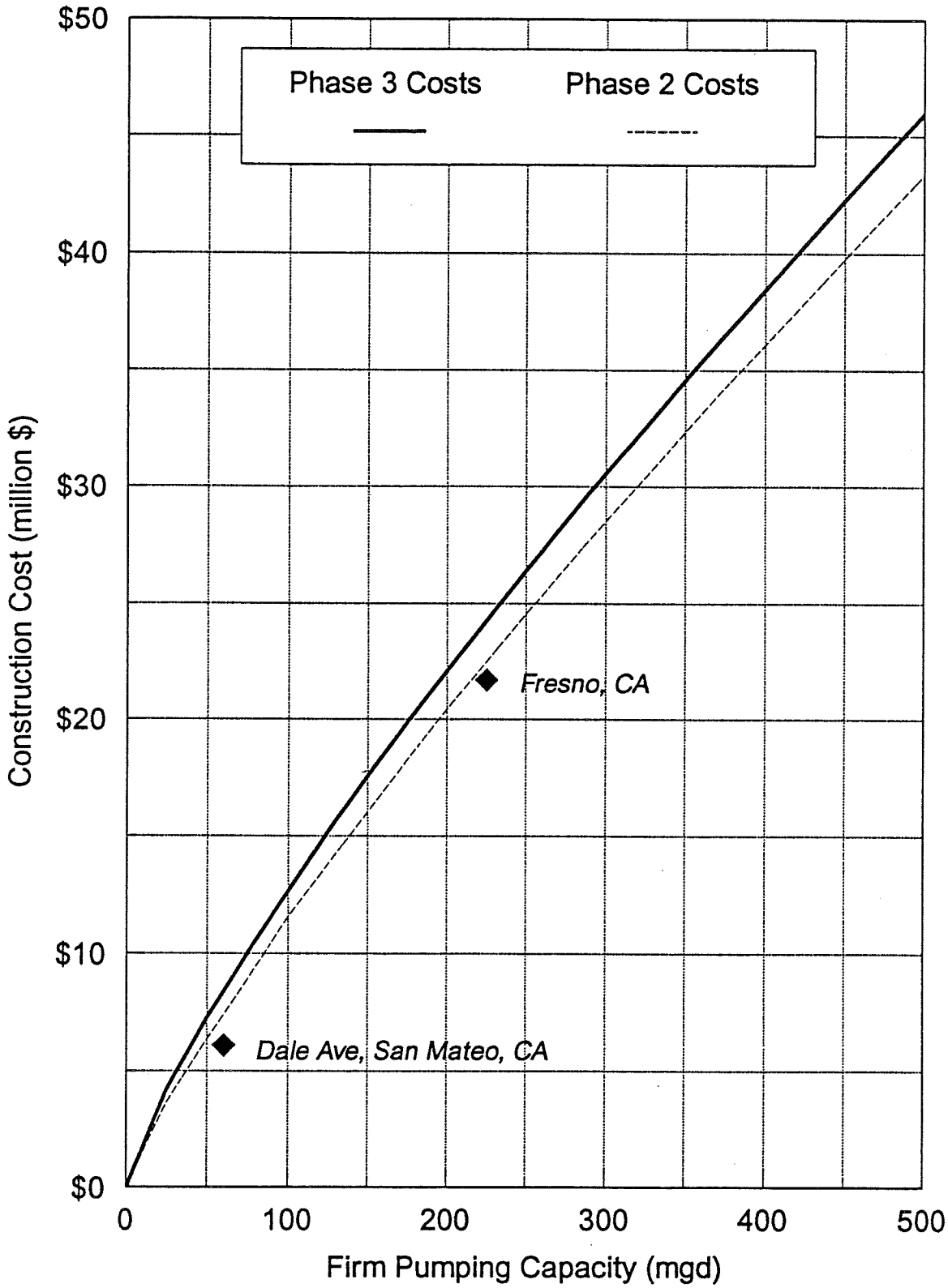
- Special site work and interface piping with the sewer system and storage reservoirs. This was estimated to represent 15 percent of the base construction cost.

- Special subsurface costs related to the pumping station sites including excavation, shoring and sheeting, dewatering, and piling foundation requirements for the pumping station wetwell. These costs were estimated to represent 10 percent of the base construction costs.

- Standby power costs. These are assumed to include the cost of providing independent electrical power supplies for the pumping stations. This would include either a standby diesel engine generation station or an additional power feeder from the SMUD distribution grid, or a combination of these alternatives. The cost for standby power was estimated to be 10 percent of the base construction cost.

Odor control costs were assumed to be part of the storage reservoir costs and are not included as part of the pumping station construction costs.

The output of the WATER CO$T estimating model for the aforementioned assumptions and input is shown in Figure 4-5. Firm pumping capacity as presented in the figure is defined as the rated output of the pumping station with one of the largest pumps out of service. Estimated construction costs were based on October 1991 dollars.



PUMP STATION CONSTRUCTION COST USED IN THE PHASE 3 PROJECT COST ESTIMATES

HDR

Figure 4-5

E.2. Comparison of Estimated Unit Costs With Project Cost Data

The construction costs of two pumping station projects were reviewed and compared with the costs predicted by the curve presented in Figure 4-5. These projects are described below.

E.2.a. Fresno Headworks Project--

An expansion of the Fresno/Clovis, California Regional Wastewater Treatment Plant is presently being designed, including a new headworks. The headworks will include mechanically cleaned screens, a wet well/dry well configuration, pumping equipment with a firm pumping capacity of 225 mgd, grit removal, flow monitoring, and odor control. Estimates of construction costs in 1991 dollars which exclude grit removal, odor control, and flow monitoring are $21.7 million. This point is plotted on Figure 4-5.

E.2.b. Dale Avenue Pumping Station, San Mateo, California--

The Dale Avenue Pumping Station in San Mateo, California was bid in 1990 with construction completed in the Fall of 1991. The station, which has a firm pumping capacity of 60 mgd, includes a wet well/dry well configuration, and an access elevator, but no odor control or screening facilities. Excluding odor control but including screening facilities, the cost of the station in 1991 dollars is $6.0 million.

E.2.c. Comparison--

Comparing the costs of these pumping facilities against the cost curve presented in Figure 4-5, the estimated costs predicted by the WATER CO$T model are similar but somewhat higher than the representative project cost data.

F. UNIT COSTS FOR STORAGE RESERVOIRS

In addition to developing the estimates of pumping station costs, the Phase 2 report also provided estimates of construction costs for above-ground interim storage facilities. These estimates are evaluated below for use in the Phase 3 report.

F.1. Assumptions

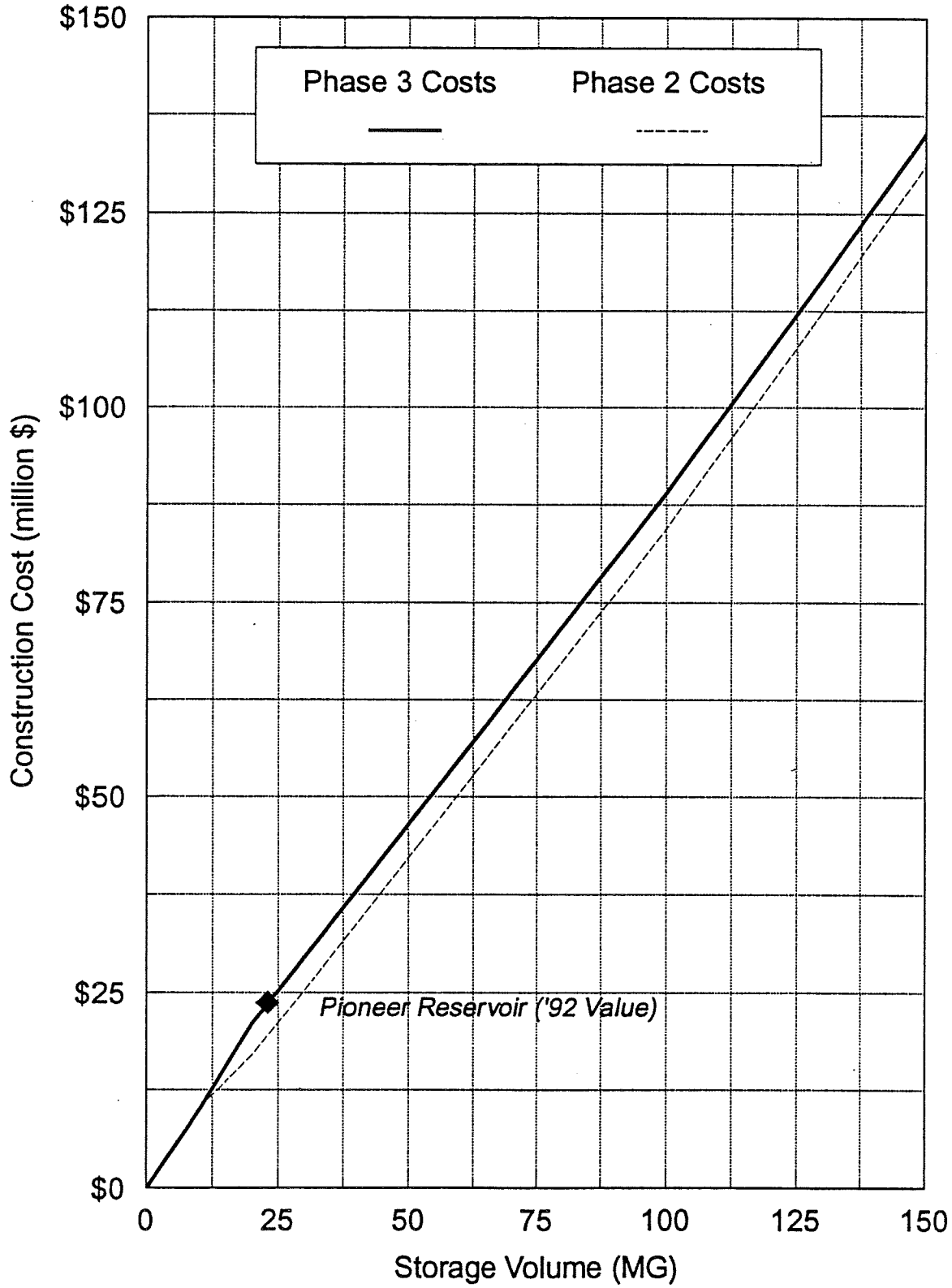
As with the pumping stations, costs for above-ground storage facilities were generated for the Phase 2 report using the WATER CO$T estimating program with the cost indices presented in Table 4-6.

The "base" construction cost data for a storage structure presented in the Phase 2 report were generated by using the costs for square covered aeration basins, 15-foot side water depth, with the aeration system deleted. Adjustment of the cost data from the cost estimating program was provided for an additional 5 feet of side water depth, odor control, pumps, piping and appurtenances required for the wash down system, level monitoring system, telemetry, construction dewatering, internal partition walls, and foundation support piling. Not included were costs associated with pumping wastewater into or out of the reservoir.

The output of the WATER CO$T program for storage reservoirs is shown in Figure 4-6 for both the Phase 2 and Phase 3 reports. The Phase 3 data has been recomputed using updated Bureau of Labor statistics and ENR indices. It has also been adjusted in accordance with the discussion below. Storage reservoir capacity is assumed to be the maximum storage capacity of the structure without an overflow. Estimated construction costs are based on October 1991 dollars.

F.2. Comparison of Estimated Unit Costs With Project Cost Data

The 1978 construction bid for the 23 million gallon Pioneer Reservoir facility was $13.47 million, which included minor upgrade work to Sump 1A. Using the ENR construction cost index to update this cost to 1991 dollars, the resulting $23.6-million project is plotted on Figure 4-6. The Phase 2 data was adjusted to coincide with this point and the revised curve plotted as a solid line in Figure 4-6. The revised cost data will be used in the preparation of construction costs for storage reservoirs in the Phase 3 report.



STORAGE RESERVOIR UNIT COST USED IN THE PHASE 3 PROJECT COST ESTIMATES

HDR

Figure 4-6

F.3. Construction Dewatering Costs and the UCD Medical Center Site

The storage reservoir under consideration for the UCD Medical Center site may be constructed as a below ground structure. It was initially assumed that the proposed structure would be below the local water table.

The dewatering cost for a large storage facility would be substantial. Based on an overall structure area of 20,000 square feet and an average depth of 20 feet below ground surface, the construction dewatering costs could come to as much as $250,000 based on a limited geotechnical analysis of construction dewatering costs.

Of greater concern is the issue of how to protect the structure from the buoyant forces caused by the presence of a shallow water table. If the storage reservoir is to be kept empty and available for receiving wastewater flows, the buoyant uplift forces must either be countered with extra dead loading or the area must be continually dewatered and the reservoir fitted with pressure relief valves in the floor slab to provide structural protection in the event of a pumping failure. Alternatively, tie-down piles could be used to hold the structure in place.

The most direct approach to the problem is to avoid construction of the reservoir below the maximum expected area groundwater table. This could either be done through shortening the side water depth of the reservoir or raising the entire structure to an elevation where the influence of the water table is no longer an issue. For purposes of evaluating storage facilities at this site, the final assumption was that the structure will be located high enough to be out of the local water table.

G. UNIT COSTS FOR SIDE OVERFLOW WEIR DIVERSION STRUCTURES

The storage, tunnel and treatment alternatives require weir diversion structures. These structures are intended to contain the dry weather sanitary flows within the existing collection system but relieve excess combined wastewater flows into a new network of consolidation conduits.

This section reviews estimated construction costs for side overflow weir junction structures developed for the Phase 3 report. These costs were used in the preparation of construction cost estimates for the alternative improvement plans.

G.1. Assumptions

Assumptions used in the preparation of unit costs for the overflow weir structures are listed below.

- The depth of the existing sewer on which the new overflow weir junction structure is constructed is the same as the depth of the new sewers assumed for Table 4-1. This range begins with a 24-inch sewer at an assumed depth of 16 feet and ends with a 114-inch sewer at a depth of 32 feet.

- Two configurations for overflow weirs were evaluated. One was for a single overflow weir and the other was for a double-sided overflow weir. The double-sided configuration, illustrated in Figure 4-7, was more cost effective and chosen over the single-sided weirs as a model for estimating construction costs.

- The walls, roof and floor of the weir junction structure were 12 inches thick, reinforced cast-in-place concrete with an average unit cost of $500/cy. Access to the structure was via a conventional 48-inch diameter manhole.

- The elevation of the weir crest was equal to the invert elevation of the existing sewer plus 1/3 the diameter of the existing sewer.

- The nominal diameter of the new consolidation conduit was identical to the diameter of the existing sewer except where the SSWMM91 indicated a larger conduit was needed for model stability at the weir.

- The cost of dewatering the site of a side overflow weir junction box ranged from $9,000 to $25,000 depending upon the depth of the structure and one month of dewatering operations cost.

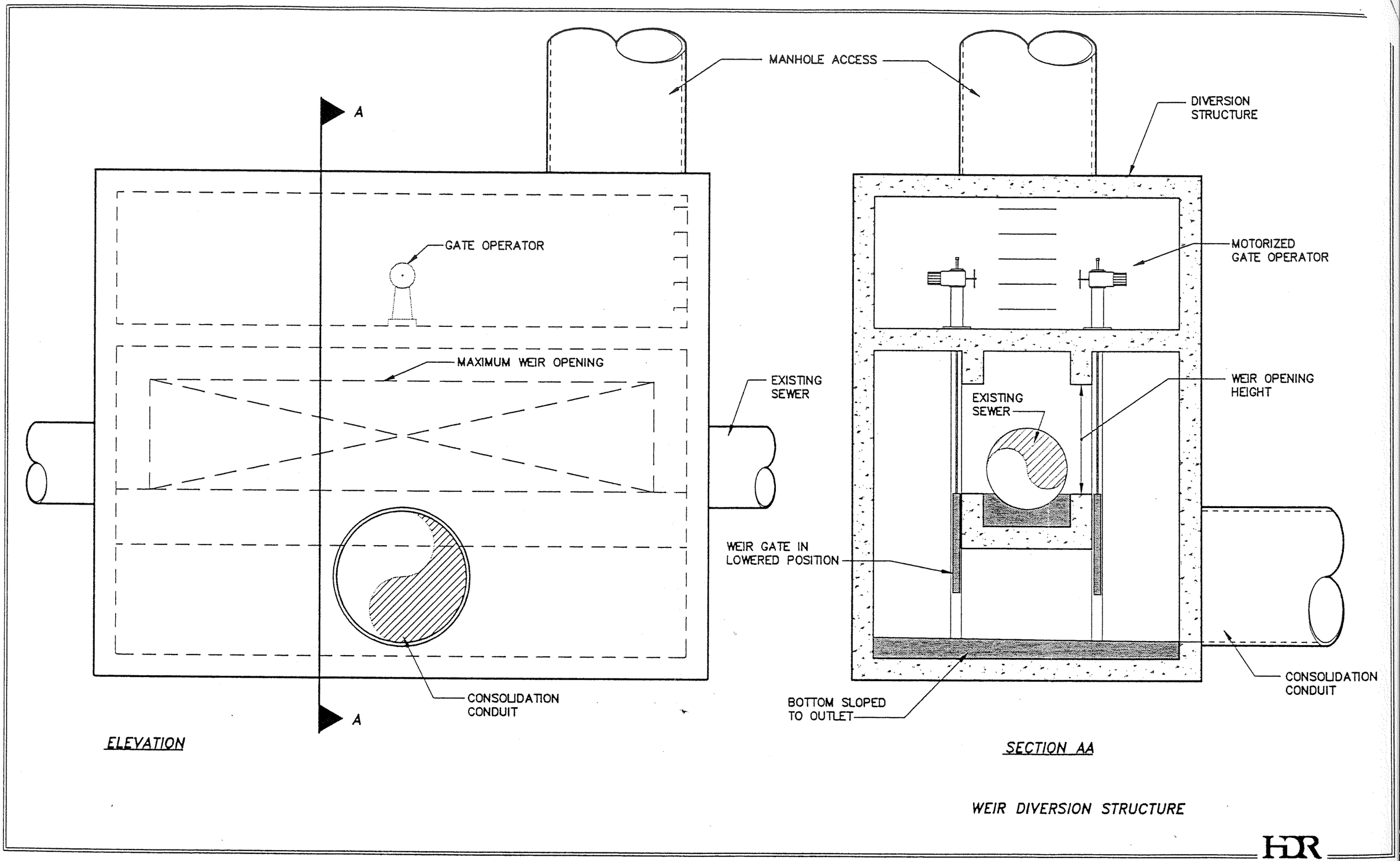

MANHOLE ACCESS
A
DIVERSION STRUCTURE
GATE OPERATOR
MOTORIZED GATE OPERATOR
MAXIMUM WEIR OPENING
EXISTING SEWER
WEIR OPENING HEIGHT
EXISTING SEWER
WEIR GATE IN LOWERED POSITION
CONSOLIDATION CONDUIT
A
CONSOLIDATION CONDUIT
BOTTOM SLOPED TO OUTLET
ELEVATION
SECTION AA
WEIR DIVERSION STRUCTURE
HDR

Figure 4-7

- Equipment costs for excavation, backfill, trucking, compaction, and other miscellaneous equipment would vary from $3400 to $4500 depending upon the depth of excavation.

G.2. Estimated Construction Costs

Estimated total construction costs for the double-sided weir configuration are presented in Table 4-7.

H. UNIT COSTS FOR TUNNELS AND DROP SHAFTS

For the tunnel alternative, combined wastewater delivered to the tunnel system must pass through a drop shaft structure. These structures must be as deep as the tunnel system, and include substantial amounts of excavation, shoring, and dewatering. Unit costs for tunnels are also presented here, including a wider range of tunnel diameters than was used in Phase 2. Costs described here are included in the overall total project cost for the tunnel alternative.

H.1. Assumptions for Drop Shaft Costs

The configuration of the drop shaft structure used for cost estimating purposes is shown in Figure 4-8. As shown, the structure comprises a vertical shaft dropping directly into the tunnel. A covered personnel access shaft with manhole entry is provided at the top of the shaft. An air vent is provided off to the side of the shaft to admit air as flow is spilling into the tunnel, and to also vent the tunnel to prevent excessive air pressure from developing. Piling supports are provided at the base of the shaft to avoid settlement of the structure.

TABLE 4-7
SIDE OVERFLOW DIVERSION WEIR UNIT COSTS

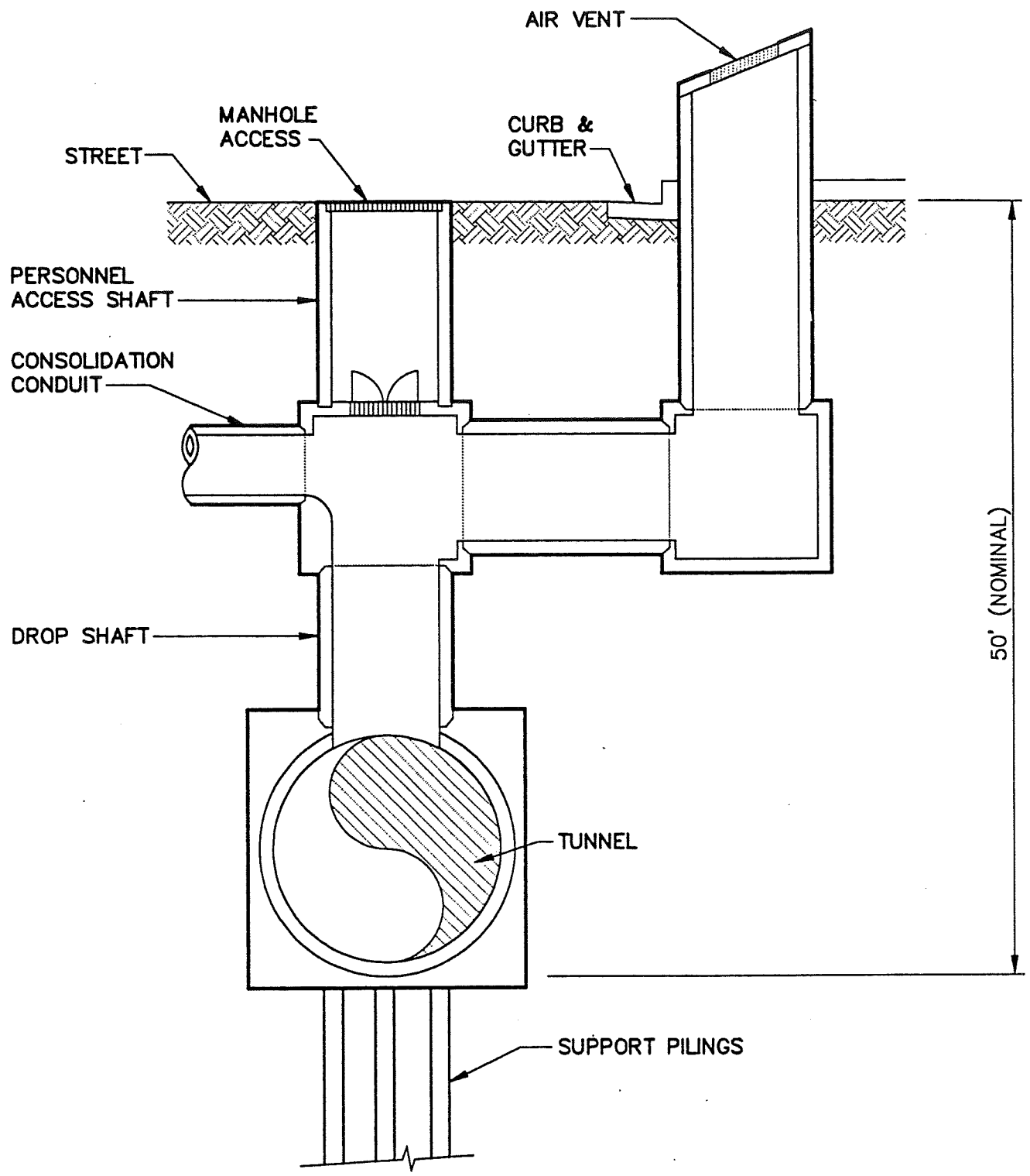
EXISTING PIPE DIAMETER (feet)	NEW PIPE DIAMETER (feet)	TOTAL WEIR LENGTH L (feet)	JUNCTION BOX WIDTH W (feet)	JUNCTION BOX HEIGHT (feet)	JUNCTION BOX TOTAL LENGTH (feet)	BOX END WALLS VOLUME (cu yds)	LONGITUDINAL WALLS VOLUME (cu yds)	TOP BOX VOLUME (cu yds)	BOTTOM BOX VOLUME (cu yds)	WEIR WALL VOLUME (cu yds)
2.00	2.00	15.00	6.17	12.58	9.17	4.60	6.41	1.74	1.74	0.35
2.50	2.50	15.33	7.17	13.08	9.33	5.49	6.78	2.06	2.06	0.44
3.00	3.00	15.67	8.17	13.58	9.50	6.41	7.17	2.39	2.39	0.53
3.50	3.50	16.00	9.17	14.08	9.67	7.38	7.56	2.73	2.73	0.63
4.00	4.00	16.33	10.17	14.58	9.83	8.38	7.97	3.09	3.09	0.72
4.50	4.50	16.67	11.17	15.08	10.00	9.42	8.38	3.45	3.45	0.82
5.00	5.00	17.00	12.17	15.58	10.17	10.49	8.80	3.82	3.82	0.93
5.50	5.50	17.33	13.17	16.08	10.33	11.61	9.23	4.20	4.20	1.03
6.00	6.00	17.67	14.17	16.58	10.50	12.76	9.67	4.59	4.59	1.15
6.50	6.50	18.00	15.17	17.08	10.67	13.95	10.12	4.99	4.99	1.26
7.00	7.00	18.33	16.17	17.58	10.83	15.17	10.58	5.41	5.41	1.38
7.50	7.50	18.67	17.17	18.08	11.00	16.44	11.05	5.83	5.83	1.50
8.00	8.00	19.00	18.17	18.58	11.17	17.74	11.53	6.26	6.26	1.62
8.50	8.50	19.33	19.17	19.08	11.33	19.08	12.02	6.70	6.70	1.75
9.00	9.00	19.67	20.17	19.58	11.50	20.45	12.51	7.16	7.16	1.88
9.50	9.50	20.00	21.17	20.08	11.67	21.87	13.02	7.62	7.62	2.02
10.00	10.00	20.33	22.17	20.58	11.83	23.32	13.53	8.10	8.10	2.15
11.00	11.00	21.00	24.17	21.58	12.17	26.33	14.59	9.07	9.07	2.44
12.00	12.00	21.67	26.17	22.58	12.50	29.50	15.68	10.10	10.10	2.74

TABLE 4-7
SIDE OVERFLOW DIVERSION WEIR UNIT COSTS

EXISTING PIPE DIAMETER (feet)	AVERAGE SEWER INVERT DEPTH (feet)	DEPTH OF 48" MANHOLE RISER (feet)	MANHOLE RISER COST ($)	TOTAL CONCRETE VOLUME (cu yds)	COST OF CONCRETE ($)	GROUT VOLUME (cu yds)	GROUT COST ($)	PIPE BELLS ($)	MECHANICAL WEIR GATES ($)	TOTAL MATERIALS ($)
2.00	16	8.67	1560	14.85	8909.17	0.52	25.83	100	8000	18595
2.50	18	10.50	1890	16.84	10103.78	0.74	36.82	145	9000	21176
3.00	20	12.33	2220	18.90	11340.85	1.00	49.83	190	10000	23801
3.50	22	14.17	2550	21.03	12620.39	1.30	64.94	250	11000	26485
4.00	24	16.00	2880	23.24	13942.38	1.65	82.27	300	12000	29205
4.50	25	16.83	3030	25.51	15306.83	2.04	101.91	375	13000	31814
5.00	26	17.67	3180	27.86	16713.73	2.48	123.96	475	14000	34493
5.50	27	18.50	3330	30.27	18163.10	2.97	148.52	560	15000	37202
6.00	28	19.33	3480	32.76	19654.93	3.51	175.69	665	16000	39976
6.50	29	20.17	3630	35.32	21189.21	4.11	205.56	775	17000	42800
7.00	29	20.00	3600	37.94	22765.96	4.76	238.25	890	18000	45494
7.50	29	19.83	3570	40.64	24385.16	5.48	273.84	1290	19000	48519
8.00	30	20.67	3720	43.41	26046.82	6.25	312.44	1380	20000	51459
8.50	31	21.50	3870	46.25	27750.94	7.08	354.15	1625	21000	54600
9.00	31	21.33	3840	49.16	29497.52	7.98	399.07	1760	22000	57497
9.50	32	22.17	3990	52.14	31286.56	8.95	447.29	1900	23000	60624
10.00	32	22.00	3960	55.20	33118.06	9.98	498.92	2100	24000	63677
11.00	33	22.67	4080	61.51	36908.43	12.26	612.79	2300	26000	69901
12.00	33	22.33	4020	68.11	40868.63	14.83	741.48	2550	28000	76180

TABLE 4-7
SIDE OVERFLOW DIVERSION WEIR UNIT COSTS

EXISTING PIPE DIAMETER (feet)	EXCAVATION SHEETING (sf)	EXCAVATION SHEETING ($)	MOBILIZATION DEMOBILIZATION CLEAN UP ($)	LABOR & SUPERVISION COST ($)	EQUIPMENT COSTS ($)	CONSTRUCTION DEWATERING COSTS ($)	OVERHEAD AND PROFIT ($)	TOTAL CONSTRUCTION COST ($)
2.00	335.00	5025.00	700	52250	24000	9000	27393	136963
2.50	433.94	6509.17	725	53412	24533	11000	29339	146693
3.00	542.28	8134.17	750	54573	25067	13000	31331	156656
3.50	660.00	9900.00	775	55735	25600	15000	33374	166869
4.00	787.11	11806.67	800	56896	26133	17000	35460	177301
4.50	879.28	13189.17	825	58058	26667	18000	37138	185691
5.00	976.50	14647.50	850	59220	27200	19000	38852	194262
5.50	1078.78	16181.67	875	60381	27733	20000	40593	202966
6.00	1186.11	17791.67	900	61543	28267	21000	42369	211846
6.50	1298.50	19477.50	925	62704	28800	22000	44177	220883
7.00	1360.78	20411.67	950	63866	29333	22000	45514	227569
7.50	1423.78	21356.67	975	65028	29867	22000	46936	234681
8.00	1547.00	23205.00	1000	66189	30400	23000	48813	244067
8.50	1675.28	25129.17	1025	67351	30933	24000	50760	253798
9.00	1744.78	26171.67	1050	68512	31467	24000	52174	260872
9.50	1881.00	28215.00	1075	69674	32000	25000	54147	270735
10.00	1954.11	29311.67	1100	70836	32533	25000	55614	278072
11.00	2175.00	32625.00	1150	73159	33600	26000	59109	295544
12.00	2330.61	34959.17	1200	75482	34667	26000	62122	310610



TUNNEL DROP SHAFT SCHEMATIC

HDR

Figure 4–8

Other assumptions used in the development of the cost estimate are listed below.

- The excavation for the shaft would be advanced using a circular steel caisson. The cost of the caisson was estimated at $2.00/pound in place.

- Construction dewatering was estimated by Brierley and Lyman to be roughly $35,000 for four wells drilled to 65 feet. An additional $7,000 was assumed for providing pumps, power and labor to operate the wells for a 6 to 7 month period while the structure was under construction.

- Excavation would be accomplished with a crane-operated clamshell. Unit costs for excavation were estimated to be $8.50/cy. This assumes that the excavated material could be stored and used as fill elsewhere.

- The floor slab at the base of the drop shaft would be supported by piling driven 25 feet into native soils at a 4.5-foot spacing interval, at a unit cost of $20/lf of piling.

- Cast in place concrete was estimated at $600/cy.

- Unit costs for asphalt paving, asphalt base, and curb and gutter are the same as discussed for gravity sewer construction costs.

H.2. Estimated Construction Costs

Table 4-8 presents the detailed estimate of construction costs for the tunnel drop shafts.

H.3. Tunnel Unit Costs

The tunnel diameters considered for the Phase 3 study varied from 13 to 20 feet in diameter. Larger diameter tunnels were not recommended for the soil conditions underlying the study area. Tunnel unit costs used in the Phase 3 study were developed as follows. For tunnel diameters ranging from 13 to 15 feet, the unit costs presented in the Phase 2 report were used. For diameters between 16 and 20 feet, the unit cost was taken to be proportional to the cross-sectional area of

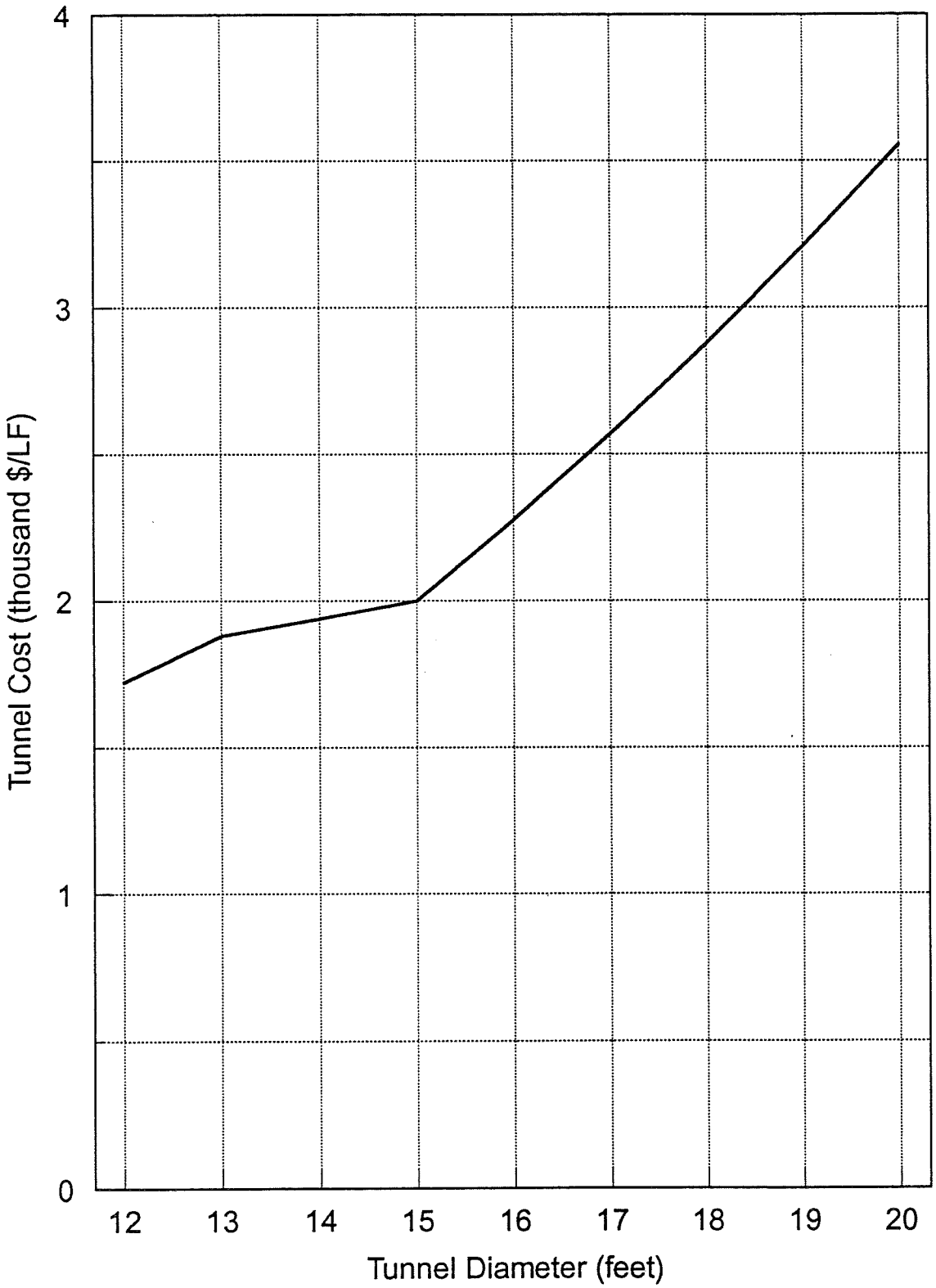
the tunnel, beginning with unit cost of $2000/lf for a 15-foot tunnel. Conduits 12 feet in diameter and smaller were assumed to be installed as gravity sewers. The tunnel unit cost recommendations were provided by Brierley and Lyman, geotechnical engineers. Figure 4-9 illustrates these tunnel unit costs.

TABLE 4-8
TUNNEL DROP SHAFT COSTS

SHAFT DIAMETER (feet)	CASSION STEEL (pounds)	CASSION COST ($)	DEWATERING COST ($)	EXCAVATION (cu yds)	EXCAVATION COST ($)	DROP SHAFT CONCRETE (cu yds)	PILING COST ($)	CONCRETE COST ($)	BACKFILL (cu yds)
2.00	95971	191942	42000	1308.33	11121	318.52	32000	159260	790.34
2.50	95971	191942	42000	1308.33	11121	320.34	32000	160168	786.96
3.00	95971	191942	42000	1308.33	11121	322.34	32000	161168	782.84
3.50	95971	191942	42000	1308.33	11121	324.33	32000	162167	777.97
4.00	95971	191942	42000	1308.33	11121	326.33	32000	163166	772.35
4.50	95971	191942	42000	1308.33	11121	328.33	32000	164166	765.98
5.00	95971	191942	42000	1308.33	11121	330.33	32000	165165	758.86
5.50	95971	191942	42000	1308.33	11121	332.33	32000	166165	750.98
6.00	95971	191942	42000	1308.33	11121	334.33	32000	167164	742.36
6.50	95971	191942	42000	1308.33	11121	336.33	32000	168164	733.00
7.00	95971	191942	42000	1308.33	11121	338.33	32000	169163	722.88
7.50	95971	191942	42000	1308.33	11121	340.32	32000	170162	712.01
8.00	95971	191942	42000	1308.33	11121	342.32	32000	171162	700.39
8.50	95971	191942	42000	1308.33	11121	344.32	32000	172161	688.02
9.00	95971	191942	42000	1308.33	11121	346.32	32000	173161	674.90
9.50	95971	191942	42000	1308.33	11121	348.32	32000	174160	661.04
10.00	95971	191942	42000	1308.33	11121	350.32	32000	175160	646.42
10.50	95971	191942	42000	1308.33	11121	352.32	32000	176159	631.05
11.00	95971	191942	42000	1308.33	11121	354.32	32000	177158	614.94
12.00	95971	191942	42000	1308.33	11121	358.31	32000	179157	580.46
13.00	95971	191942	42000	1308.33	11121	362.31	32000	181156	542.98
14.00	95971	191942	42000	1308.33	11121	366.31	32000	183155	502.50
15.00	95971	191942	42000	1308.33	11121	370.31	32000	185154	459.03
16.00	95971	191942	42000	1308.33	11121	374.31	32000	187153	412.56
17.00	95971	191942	42000	1308.33	11121	378.30	32000	189151	363.08
18.00	95971	191942	42000	1308.33	11121	397.02	32000	198510	310.61

TABLE 4-8
TUNNEL DROP SHAFT COSTS

SHAFT DIAMETER (feet)	BACKFILL COST ($)	SHAFT LENGTH (feet)	SHAFT COST ($)	ASPHALT BASE (cu yds)	ASPHALT BASE ($)	ASPHALT PAVING (cu yds)	ASPHALT PAVING ($)	CURB/ GUTTER (lf)	CURB/ GUTTER ($)	DEMOB/ CLEANUP ($)	TOTAL COST ($)
2.00	3952	35.00	8565	49.19	947	24.69	3704	80.00	1440	700	455630
2.50	3935	35.00	9922	49.19	947	24.69	3704	80.00	1440	725	457903
3.00	3914	35.00	11320	49.19	947	24.69	3704	80.00	1440	750	460305
3.50	3890	35.00	12986	49.19	947	24.69	3704	80.00	1440	775	462971
4.00	3862	35.00	14648	49.19	947	24.69	3704	80.00	1440	800	465630
4.50	3830	35.00	16173	49.19	947	24.69	3704	80.00	1440	825	468147
5.00	3794	35.00	17930	49.19	947	24.69	3704	80.00	1440	850	470893
5.50	3755	35.00	19831	49.19	947	24.69	3704	80.00	1440	875	473779
6.00	3712	35.00	21670	49.19	947	24.69	3704	80.00	1440	900	476600
6.50	3665	35.00	23484	49.19	947	24.69	3704	80.00	1440	925	479391
7.00	3614	35.00	25970	49.19	947	24.69	3704	80.00	1440	950	482850
7.50	3560	35.00	29852	49.19	947	24.69	3704	80.00	1440	975	487703
8.00	3502	35.00	31592	49.19	947	24.69	3704	80.00	1440	1000	490409
8.50	3440	35.00	34649	49.19	947	24.69	3704	80.00	1440	1025	494429
9.00	3375	35.00	36818	49.19	947	24.69	3704	80.00	1440	1050	497557
9.50	3305	35.00	39002	49.19	947	24.69	3704	80.00	1440	1075	500695
10.00	3232	35.00	41307	49.19	947	24.69	3704	80.00	1440	1100	503951
10.50	3155	35.00	41307	49.19	947	24.69	3704	80.00	1440	1125	504899
11.00	3075	35.00	45655	49.19	947	24.69	3704	80.00	1440	1150	510191
12.00	2902	35.00	50227	49.19	947	24.69	3704	80.00	1440	1200	516640
13.00	2715	35.00	50227	49.19	947	24.69	3704	80.00	1440	1250	518501
14.00	2513	35.00	50227	49.19	947	24.69	3704	80.00	1440	1300	520348
15.00	2295	35.00	50227	49.19	947	24.69	3704	80.00	1440	1350	522179
16.00	2063	35.00	50227	49.19	947	24.69	3704	80.00	1440	1400	523996
17.00	1815	35.00	50227	49.19	947	24.69	3704	80.00	1440	1450	525797
18.00	1553	35.00	50227	49.19	947	24.69	3704	80.00	1440	1500	534943



UNIT COST FOR THE TUNNELS AS USED IN THE PHASE 3 COST ESTIMATES

HDR

Figure 4-9

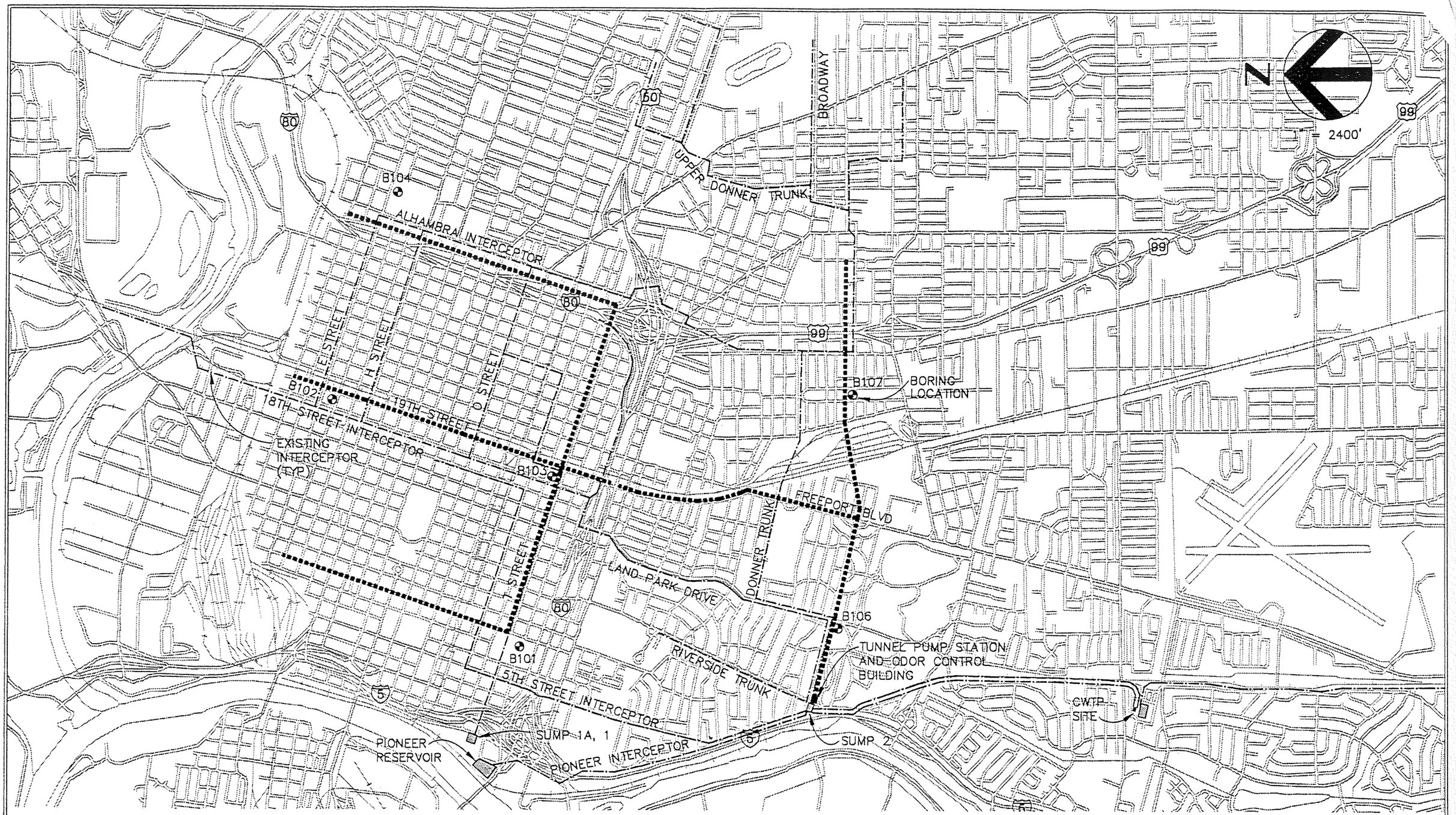
SECTION 5
TUNNEL ALTERNATIVES

A. INTRODUCTION

This section describes the analysis of tunnel system alternatives. Tunnels were intensively reviewed for both the Phase 2 and Phase 3 studies. They were of particular interest in Phase 2 because the tunnel system alternative was identified as the least-cost alternative which would satisfy both flood and CSO control requirements for the 10-year design storm event. In Phase 3, tunnel alternatives were the focus of a detailed hydraulic analysis because of their complexity and unusual dynamic response behavior to different design storms and outlet pumping configurations.

The tunnels were recognized in Phase 2 as having the advantage of minimal disruption to vehicle and pedestrian traffic within the study area, relative to construction activity. They also have the advantage of providing both storage and conveyance of combined wastewater in a single type of infrastructure. This latter characteristic is a primary reason for their lower overall project cost in comparison to other alternatives, as discussed later in Section 11.

The Phase 2 study included a limited borehole and geotechnical evaluation of the combined sewer study area. The objective of that evaluation was to identify the compatibility of the soils underlying the area with tunnel construction. Although there were only six boreholes advanced for the entire effort, the conclusions were that for planning purposes, tunnels should be considered a viable alternative. Balanced earth pressure tunneling machines were identified as capable of boring the tunnels at depths of roughly 50 feet below the ground surface. Details of this boring effort and the recommendations of the geotechnical engineer were provided in the Phase 2 technical memorandum entitled *Unit Construction Costs for Specific Project Alternative Projects*, dated July 1, 1991. The Phase 2 tunnel alignment, borehole locations, tunnel pumping station/odor control building are presented in Figure 5-1. Odor control facilities are also part of the Phase 3 tunnel alternatives and are included in the construction of the tunnel pumping stations.



PHASE TWO TUNNEL ALIGNMENTS
& BOREHOLE LOCATIONS

HDR

Figure 5–1

Tunnel sizing during Phase 2 was based on the volumes of combined wastewater relieved from the existing collection system using the hydrographic spreadsheet analysis outlined in the technical memorandum entitled *Alternative Improvement Projects and Plan Combinations*, dated July 1, 1991. Alternative improvement plans, including the tunnel system, could not be modeled during Phase 2 with the Sacramento SWMM; therefore, the adequacy of the tunnels to convey the design storm was not verified.

The objective of the Phase 3 tunnel alternative analysis was to review the proposed tunnel configurations using the SSWMM91, and to arrive at improvement plans that would comply with the design and analysis assumptions described in Section 3. Tunnel configurations were developed for control of the 2-, 5-, 10- and 25-year design storms. These tunnel systems were then checked to verify they would control both the 1/year and 5/year CSO design storms.

B. TUNNEL ALTERNATIVE OVERVIEW

The tunnel alternative consists of three components used to convey or store combined sewage. The first component consists of replaced service laterals and collectors. These replaced conduits convey flow to the mains in the existing combined sewer system. Excess flow that cannot be handled by these mains is drawn off by diversion weirs and directed into a network of new consolidation conduits. The consolidation conduits receive excess flows at one or more locations and convey the flow to tunnel drop shafts. These dropshafts empty into the tunnel system. The tunnel system provides both conveyance and storage. Flows in the tunnel are directed towards outlet pumping stations and storage is provided as a result of tunnel volume in excess of the conveyance needs of the system.

C. TUNNEL ALTERNATIVE DESIGN CONSIDERATIONS

There were many considerations involved in the design of replacement pipes, weirs, consolidation conduits, orifices and tunnels. The following is a summary of the design criteria used as guidelines in modeling tunnel alternatives. For further discussion on the development of these guidelines and on the special hydraulic and model considerations involved, refer to Appendices C and D at the

end of this memorandum.

C.1. Conveyance Capacity

C.1.a. Replacement Pipes--

Most of the flooding in the existing combined sewer system is due to undersized upstream laterals and collectors. To draw off excess flows from these smaller conduits would not be cost effective. Therefore, these undersized conduits are upsized in order to direct flow to more central locations were the flow can be diverted. The replacement conduits were designed to take advantage of higher conveyance due to surcharged conditions. This was done for two reasons: 1) there are a large number of pipes that require upsizing, and the diameters necessary for open channel flow conditions in the pipes would be considerably larger and consequently extremely expensive; and 2) pipes designed for open channel flow conditions would also be considerably larger than the mains to which they connect. The new conduits were consequently designed to be surcharged. However, the pipes were sized to control the surcharge levels and to maintain the hydraulic gradeline at least 2 feet below the manhole lid elevation. Exceptions exist where manholes are shallow or where the cost of the criteria outweighs the benefit. The velocity in the replaced conduits was controlled to range from 2 to 10 fps.

C.1.b. Consolidation Conduits--

For sizing consolidation conduits, the primary consideration was to maintain the ratio of the maximum flow rate to the full pipe flow rate at 80 percent. The actual ratios for new consolidation conduits ranged between 60 and 90 percent, depending on the conduit diameter used. Surcharged conditions were not allowed in the new conduits, and the design velocity was maintained within the 2 to 10 fps range as outlined in Section 3.

C.1.c. Tunnels--

The tunnels were modeled as conduits draining to outlet locations at slopes of 0.5 to 1.0 foot per mile. Tunnel conveyance capacity limitations are mainly dictated by requirements for hydraulic stability in the tunnels. Municipalities with similar tunnel systems have reported significant hydraulic surges due to

hydraulic instability and water hammer effects in the tunnels. These hydraulic effects can be controlled by minimizing pressure buildups by air trapped in the tunnel, and by limiting the flow velocities in the tunnel. Modeling experience revealed that maintaining the 80-percent design to maximum capacity flow ratio as a conveyance limitation for the tunnels provides stable hydraulic flow in the tunnels. It was therefore applied in sizing the tunnels in Phase 3.

As outlined in the Phase 2 study report, the recommendations of Brierley and Lyman, geotechnical engineers, was that for the tunnels to be cost effective, they should be constructed of a constant diameter for at least a length of a mile, and preferably longer. In order to conform to this constructibility criteria, tunnels were modeled with constant diameter for long sections. Since tunnel sizing was based on the 80 percent full flow conveyance criterion for critical sections, the velocity in tunnel sections with lower maximum flow rates dropped below the 2 fps minimum velocity criterion specified for designing storm drain conduits.

C.2. Diversion Weirs

Diversion weirs were used in the SSWMM91 to relieve flows from the existing conveyance system. Figure 5-2 is a schematic illustration for weirs. Weirs were used only where upsizing existing sewers would not eliminate flooding for a given design storm. In the selection of weir locations, the number of weirs was minimized, and weirs were grouped in order to limit the number of tunnel drop shafts. The height of the weirs (relative to the existing system pipe) was set to divert only excess flows and to utilize the conveyance capacity of the existing combined sewer system as much as possible. These weir height settings are different for each flood control design storm. The diversion weirs are discussed in more detail in Section 3 and Appendix C of this memorandum.

C.3. Tunnel Drop Shafts

The networks of new consolidation conduits were configured to convey excess flow diverted by weirs from the existing sewer system to the tunnel drop shafts and into the tunnel. The drop shafts were modeled in SSWMM91 as consolidation conduit

junctions with side outlet orifices. (See Figure 5-2.) These orifice connections joined the consolidation conduit junctions with the tunnel conduit junctions. The cross-sectional area of the orifice used for each drop shaft was twice the collective cross-sectional area of the consolidation conduits contributing flow into the drop shaft. This cross-sectional area was chosen in order to assure that the headloss through the orifice would be minimal and consequently more closely represent the transfer of water to the tunnel as a drop shaft. Bottom drop orifices within SSWMM91 would best model these structures, however, this model option was not functional.

Figure 5-2 also presents an illustration of a portion of a tunnel alternative model network with both diversion weirs and orifices simulating tunnel drop shafts.

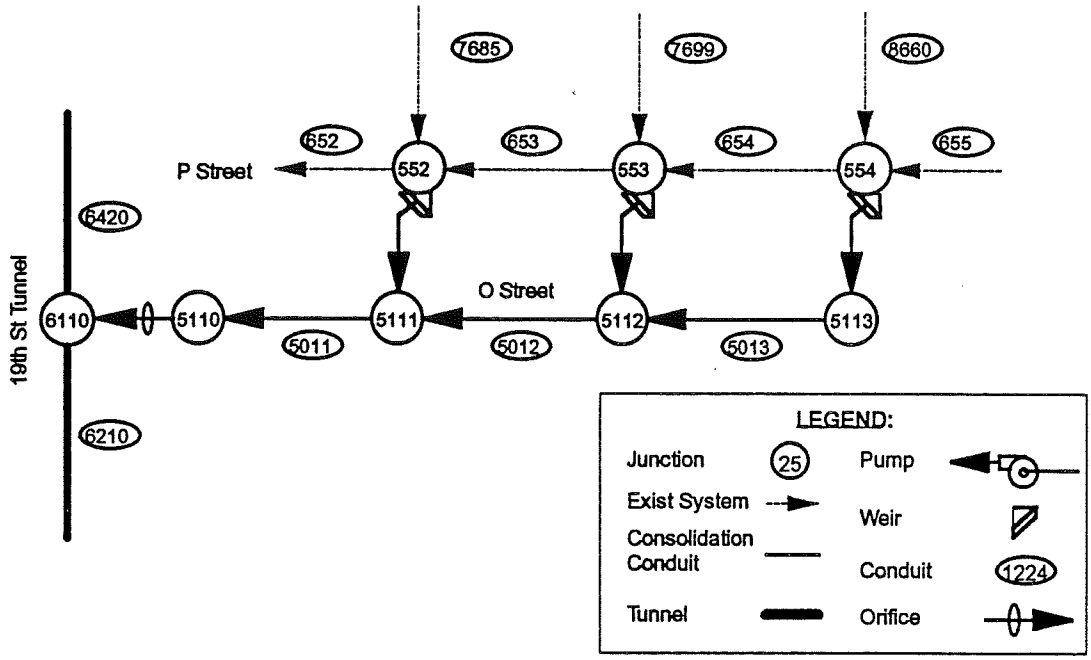
C.4. Tunnel Alignments

In the Phase 2 study, tentative alignments for the tunnels were routed through the combined sewer service area. Selection of these alignments was based on identification of right-of-way compatible with the tunneling process and the locations where floodwater occurred most frequently. Boreholes drilled for Phase 2 were located on these preliminary tunnel alignments. The Phase 2 tunnel alignment and borehole locations are shown in Figure 5-1.

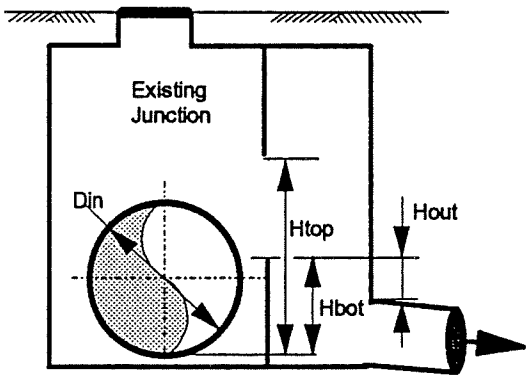
The Phase 3 analysis of tunnel alternatives was initiated using the Phase 2 tunnel alignment. However, this alignment was altered to improve and optimize the storage utilization and hydraulic stability in the tunnels.

Based on the recommendations of the geotechnical consultants for the Phase 2 effort, tunnels considered for the Phase 3 study conformed to the following criteria:

1) As previously mentioned, tunnels had to be of constant diameter for lengths of at least one mile, and preferably longer, to make construction economically viable. Mobilization costs, including the boring machine, shaft lining forms, and on-site setup efforts, are so expensive that



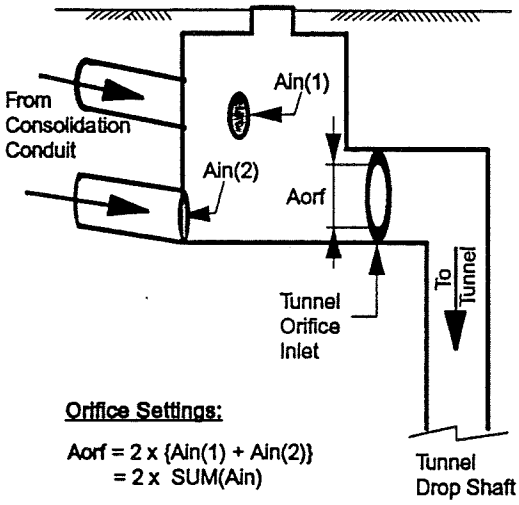
SCHEMATIC OF CONFIGURATION FOR DIVERSION WEIRS, CONSOLIDATION CONDUIT AND TUNNEL MODEL LAYOUT.



Weir Settings:

Hbot = 0.33 x Din to 0.8 x Din
Htop - Hbot ≥ 5.0 ft
Hout > 0.0 ft

Weir Schematics



Orifice Settings:

Aorf = 2 x {Ain(1) + Ain(2)}
= 2 x SUM(Ain)

Tunnel Orifice Inlet Schematics

SCHEMATICS OF DIVERSION WEIR AND TUNNEL INLET ORIFICE SETTINGS USED FOR TUNNEL MODEL CONFIGURATIONS.

HDR

Figure 5-2

tunneling as a construction technique is competitive only if these costs can be distributed over a project at least one mile long.

2) Tunnels must be aligned to avoid changes to center line alignment with anything smaller than a 200-foot radius. The implication of this criterion was that right-of-way for the tunnel alignments had to be long and relatively straight without any sharp bends. In some cases, right-of-way acquisitions may be necessary to conform to this criterion.

Figure 5-3 illustrates the alignments used in the Phase 3 study, for each of the flood control design storms.

C.5. Tunnel Elevations

Based on the geotechnical investigations conducted during the Phase 2 study, a tunnel depth of approximately 50 feet below ground level was recommended. With possible tunnel diameter variation between 12 and 20 feet and ground elevations of approximately 10 to 30 feet MSL, it was decided that for purposes of this study, the uppermost crown elevation of the tunnel system would be set at -15.00 feet MSL, independent of the tunnel diameter. This means that the most upstream invert elevations for each tunnel system are based on a crown elevation of -15.00 feet MSL. Tunnels were then sloped to provide gravity flow to the two tunnel pumping stations.

C.6. Tunnel Maintenance Requirements

In an effort to determine whether the overall cost of the tunnel alternatives should include an estimated cost for a flushwater system, several municipalities operating combined wastewater tunnels were contacted. The majority of respondents indicated that their tunnel system did not require any tunnel maintenance procedures, such as flushing after a storm event or periodic removal of accumulated debris. Some respondents suggested that the larger storm events created flow velocities sufficient to scour the sediment within the tunnels and move it downstream. This indicates that all debris would be flushed towards the

TUNNEL ALIGNMENTS FOR 5-10-& 25-YEAR DESIGN STORMS

HDR

Figure 5-3

tunnel pumping stations and be lifted out of the tunnels either by the pumps or manually through periodic maintenance operations.

For purposes of preparing an estimate of total project cost, no tunnel flushwater system was included. However, it was assumed that periodic removal of larger debris would be required either by automatic screening equipment or manual cleaning.

D. MODELING APPROACH

This subsection describes the approach followed to arrive at the final consolidation conduit and tunnel configuration recommended for each design storm.

The initial modeling effort for the tunnel alternatives was to establish tunnel configurations in the SSWMM91 which would meet the conveyance needs for a particular flood control design storm (i.e. the 2-, 5-, 10- and 25-year design storms). This required selection of replacement conduits for the existing conveyance system, locating weirs and setting their heights (crest elevations), choosing a consolidation conduit network and tunnel configuration for each flood control design storm which eliminate flooding within the study area.

After the aforementioned weirs, replacement conduits and consolidation conduits were set for a design storm, numerous tunnel configurations were analyzed with the SSWMM91 which varied only in the diameter of the tunnel segments which made up the overall tunnel system. For each of these tunnel configurations for a given design storm, the tunnel overflow elevation (to the Sacramento River) was set in order to maximize storage within the tunnel while still providing flood control within the modeled study area. From this, a least-cost configuration was identified. For purposes of this analysis, cost comparisons were based upon unit construction costs for the tunnel pump stations and tunnel conduit construction using data presented in Section 4. This analysis was carried out to determine the tradeoff in cost between larger tunnel systems with small pumping stations versus larger pumping stations and smaller pumping stations. Flood control design storms of 6-hour duration were used in all comparative tunnel analyses.

After a least-cost tunnel configuration for each design storm was chosen, the outlet control elevation affecting the storage utilization, and consequently peak pumping rates, was varied in order to analyze the effect of tunnel outflow elevation on the total cost of the alternative. For these analyses, the tunnel outflow elevations at both tunnel pumping stations shown in Figure 5-3 were varied but always set identical to one another in all alternatives modeled.

The last steps in the approach were (1) to apply the 1/year and 5/year CSO control design storms to the various configurations in order to set the outlet control elevation in the tunnel to an elevation which would contain the 1/year and 5/year CSO storm volumes, and (2) to choose two each least-cost tunnel configurations for each flood control design storm; one which would a CSO control level of 1/year and one which would provide a CSO control level of 5/year. For example, a least cost 5-year tunnel configuration was identified which would provide flood protection for the 5-year flood control design storm <u>and</u> CSO control for the 1/year CSO design storm. A least cost 5-year tunnel configuration was also identified which would provide flood protection for the 5-year flood control design storm <u>and</u> CSO control for the 5/year CSO design storm.

D.1. Consolidation Conduit Network

The following approach was developed for the selecting the configuration of the consolidation conduit network used in modeling the tunnel alternatives:

1) The possible locations of diversion weirs were selected so as to limit their total number. This was done by upsizing the existing conduits, initially using the 10-year design storm.

2) Consolidation conduits were routed to convey flows removed from the existing sewers by the diversion weirs to tunnel drop shafts. These new sewers were laid out to minimize the number of tunnel drop shafts.

3) In order to comply with the flood control criteria, the weir overflow height for the 10-year consolidation conduit system was set at 33 percent

of the maximum pipe diameter at the diversion junction. (See Section 3 for an outline on the diversion weir design criteria.)

4) Once the 10-year consolidation conduit network was established, the diameters of the individual conduits comprising the overall network were down-sized to accommodate the 5-year and 2-year design storms. Diversion weirs were either eliminated or set to a weir height of 80 percent of the maximum conduit diameter at the diversion junction.

5) Upsizing the 10-year system to handle the 25-year design storm involved additional upsizing of the existing system, the addition of extra diversion weirs, and the extension of the consolidation network.

The result of this approach was that for the 5-year consolidation conduit system, only a few of the weirs were eliminated, but the weir heights of the remaining weirs were increased to 80 percent of the maximum conduit diameter at the diversion junction.

For the 2-year storm, all the weirs were eliminated, with the exception of 5 weirs in the Land Park area. The total volume of flow diverted to the consolidation conduits and tunnel system for the 2-year storm was relatively small: a volume equivalent to approximately 0.2 miles of 15-foot diameter tunnel. Due to the minimum 1 mile tunnel length limitations previously mentioned, it was concluded that a 2-year tunnel alternative is not a viable option. The 2-year tunnel alternative was therefore not evaluated further.

D.2. Tunnel Network

Evaluating the 10-year design storm inflow with the Phase 2 tunnel alignment revealed the following limitations in that tunnel system:

1) A large fraction of the tunnel inflow enters the system in the Broadway area. This inflow is directed to the Land Park area; however, most of the available tunnel storage was in the downtown area. This results in

inefficient tunnel storage utilization, i.e. the storage in the tunnel network is not where the flows are generated within the study area.

2) The flow from the Broadway area caused the tunnel to back up to the downtown area, via the Freeport Boulevard tunnel section (see Figure 5-1) to the T Street tunnel, where it joined with the inflow from the downtown area. The flow of water north the Freeport tunnel was opposite to the flow of water moving south down 19th Avenue, and flow from the Alhambra tunnel, resulting in hydraulic surges and waves in the tunnel system.

3) All flow was diverted to the Sump 2 site from where it would be pumped to the river. This resulted in extremely high conveyance requirements in the tunnel section from Freeport Boulevard to the Sump 2 site.

These limitations were overcome by realigning the tunnel as shown in Figure 5-3 and incorporating the following changes to the Phase 2 tunnel alignment:

1) Tunnel inflow in the Broadway area was directed to the downtown area by means of a new Broadway tunnel approximately following the alignment of Stockton Boulevard. This directed the flow to available downtown storage, resulting in more optimum storage utilization. It also reduced the possibility of hydraulic instability due to opposing flow directions in the same tunnel segment.

2) A second tunnel outlet was provided at the current Sump 1A site. This resulted in a more equal distribution of flow within the tunnel system.

This revised tunnel alignment showed none of the hydraulic instability exhibited in the Phase 2 tunnel system. This alignment was therefore used as the Phase 3 tunnel alignment.

D.3. Tunnel Hydraulic Stability

In many of the tunnel configurations modeled, flow instability was exhibited. This was manifest by rapid changes in water level within the tunnel conduits and

is especially difficult for the SSWMM91 Extran block to accommodate. These conditions can cause the programmed calculations to have a large error in continuity, or to terminate a model run prematurely. Factors affecting the hydraulic stability in the tunnels were identified as:

1) The conveyance capacity of the tunnel.
2) Alignment of the tunnels.
3) Inflows to the tunnels exceeding the available storage capacity.
4) The outlet control elevation of the tunnel.

Several approaches were used to address the above conditions and assure the stability of the tunnel alternatives. A further discussion on the dynamics of the tunnel modeling and design and the approaches taken to solve the above limitations can be found in Appendix D of this memorandum.

D.4. Identifying Tunnel Configurations for Flood Control Design Storm Events

Alternative tunnel configurations for the 5-, 10-, and 25-year design storm events were identified for different combinations of tunnel diameters and outflow control elevations.

Tunnel diameters for the various design storms, as shown in Figures 5-4 to 5-6 and summarized in Tables 5-2, 5-3, and 5-4, were based on the conveyance limitations and alignment criteria previously discussed. The process for selecting the tunnel configurations can be summarized as follows:

1) Initially, six configurations were modeled for the 10-year event and four for the 5 year event. In each of these configurations, all tunnels sections were of uniform diameter. For the 10-year event, tunnel configuration diameters ranged from 15 to 20 feet. For the 5-year event the diameters ranged from 12 to 15 feet.

   The SSWMM91 output from these configurations indicated that certain tunnel sections did not comply with the 80-percent of full pipe flow conveyance criteria. Sections with conveyance criteria limitations were the 19th

Street, Freeport, Broadway, and T Street sections. The minimum diameter needed to meet the criteria in these sections is listed in Table 5-1.

TABLE 5-1. MINIMUM DIAMETER FOR CONVEYANCE		
Section	5-Year	10-Year
19th Street	13	17
Freeport	13	18
Broadway	13	17
T Street	13	19

2) In view of the above limitations, it was decided that the proposed tunnel configurations should be modified to have tunnel segments of varying diameters. Each segment's diameter was selected in accordance with the most critical conveyance requirements in that tunnel segment or the diameter of the tunnel alternative, whichever was the larger. As an example, for the 17-foot 10-year tunnel, the diameters of the T Street and Freeport tunnel segments were set at 19 and 18 feet, respectively, to comply with the conveyance capacity criteria. Whereas the diameters of rest of the tunnel system were set at 17 feet. For reference purposes, this configuration is referred to as the 17-foot 10-year tunnel, although the diameter of certain segments of the tunnel are larger than 17 feet.

3) In the configurations described above, the 8th Street and Alhambra segments of the tunnel configurations are not designed for conveyance limitations but rather are given the diameter of the configuration. These segments serve primarily as storage reservoirs.

4) Due to constructibility considerations, the maximum tunnel diameter was limited to 20 feet. In order to provide conveyance capacity larger than that of the 20-foot tunnel, parallel tunnels would be necessary. Parallel tunnels were only considered for the T Street tunnel segment of the 25-year tunnel system.

D.5. Tunnel Outflow and Storage Criteria

According to the City's guidelines with regard to tunnel discharge control, as discussed in Section 3, inflow to the tunnel may be discharged to the river but should be terminated once the tunnels are no longer needed for flood control, i.e. when the inflow from the consolidation conduits into the tunnel stops. A second requirement for tunnel discharge control is that the tunnel system must provide storage without spilling for the CSO design storm volumes (the 1/year and 5/year CSO storms). Due to limitations in the current version of the SSWMM91, these guidelines could not be built into the tunnel model and had to be approximated following two approaches, i.e. maximizing tunnel storage and varying the outlet control elevation, as discussed below.

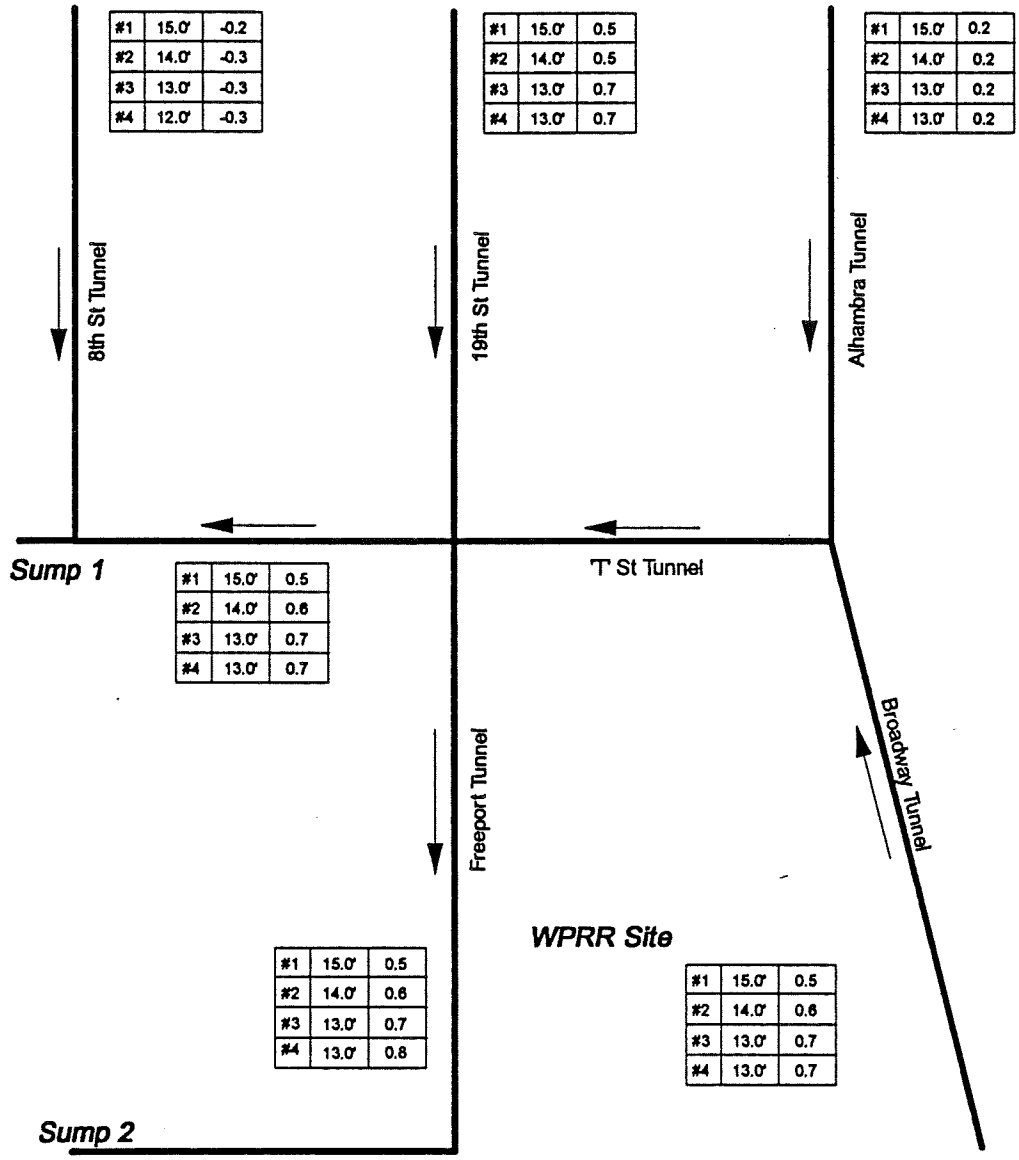
TABLE 5-2. TUNNELING REQUIREMENTS FOR 5-YEAR TUNNEL ALTERNATIVES

Tunnel Diameters	Tunnel Lengths (feet)			
	12-Foot Tunnel Alternative	13-Foot Tunnel Alternative	14-Foot Tunnel Alternative	15-Foot Tunnel Alternative
12-foot tunnels	6,000	0	0	0
13-foot tunnels	43,200	49,200	0	0
14-foot tunnels	0	0	49,200	0
15-foot tunnels	0	0	0	49,200

TABLE 5-3. TUNNELING REQUIREMENTS FOR THE 10-YEAR TUNNEL ALTERNATIVES

Tunnel Diameters	Tunnel Lengths (feet)			
	17-Foot Tunnel Alternative	18-Foot Tunnel Alternative	19-Foot Tunnel Alternative	20-Foot Tunnel Alternative
17-foot tunnels	26,100	0	0	0
18-foot tunnels	13,800	39,000	0	0
19-foot tunnels	9,300	9,300	49,200	0
20-foot tunnels	0	0	0	49,200

TABLE 5-4. TUNNELING REQUIREMENTS FOR THE 25-YEAR TUNNEL ALTERNATIVES		
Tunnel Diameters	Tunnel Lengths (feet)	
	20-Foot Tunnel Alternative	2*20-Foot Tunnel Alternative
20-foot tunnels	49,200	40,300
Two Parallel 20-foot tunnels	0	8,900



#1	15.0'	-0.2
#2	14.0'	-0.3
#3	13.0'	-0.3
#4	12.0'	-0.3

#1	15.0'	0.5
#2	14.0'	0.5
#3	13.0'	0.7
#4	13.0'	0.7

#1	15.0'	0.2
#2	14.0'	0.2
#3	13.0'	0.2
#4	13.0'	0.2

#1	15.0'	0.5
#2	14.0'	0.6
#3	13.0'	0.7
#4	13.0'	0.7

#1	15.0'	0.5
#2	14.0'	0.6
#3	13.0'	0.7
#4	13.0'	0.8

#1	15.0'	0.5
#2	14.0'	0.6
#3	13.0'	0.7
#4	13.0'	0.7

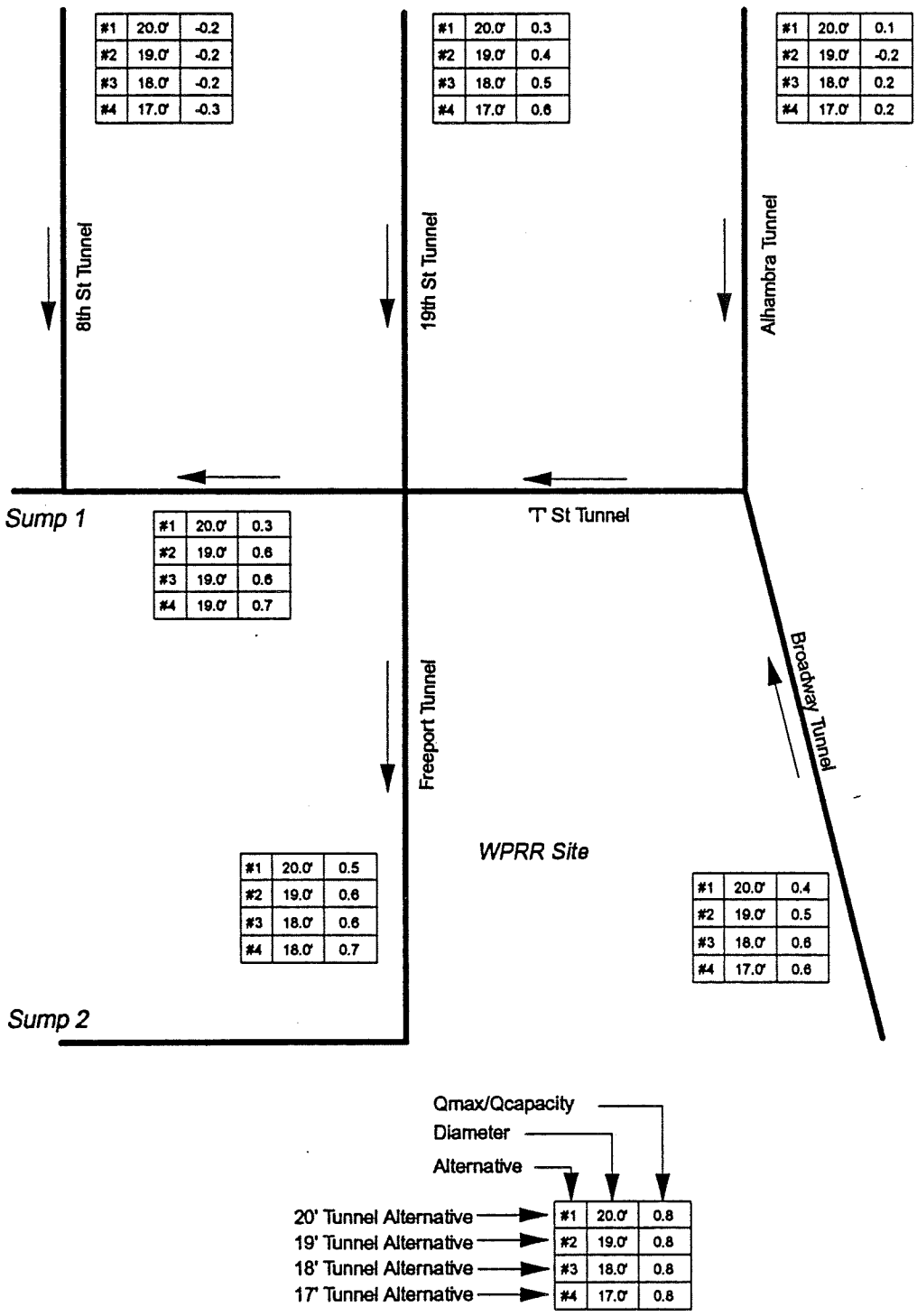
Qmax/Qcapacity
Diameter
Alternative

15' Tunnel Alternative	#1	15.0'	0.8
14' Tunnel Alternative	#2	14.0'	0.8
13' Tunnel Alternative	#3	13.0'	0.8
12' Tunnel Alternative	#4	12.0'	0.8

SCHEMATIC LAYOUT OF THE 5-YEAR TUNNEL ALTERNATIVE CONFIGURATIONS CONSIDERED IN THE TUNNEL OPTIMIZATION EVALUATIONS.

HDR

Figure 5-4



#1	20.0'	-0.2
#2	19.0'	-0.2
#3	18.0'	-0.2
#4	17.0'	-0.3

#1	20.0'	0.3
#2	19.0'	0.4
#3	18.0'	0.5
#4	17.0'	0.6

#1	20.0'	0.1
#2	19.0'	-0.2
#3	18.0'	0.2
#4	17.0'	0.2

#1	20.0'	0.3
#2	19.0'	0.6
#3	19.0'	0.6
#4	19.0'	0.7

#1	20.0'	0.5
#2	19.0'	0.6
#3	18.0'	0.6
#4	18.0'	0.7

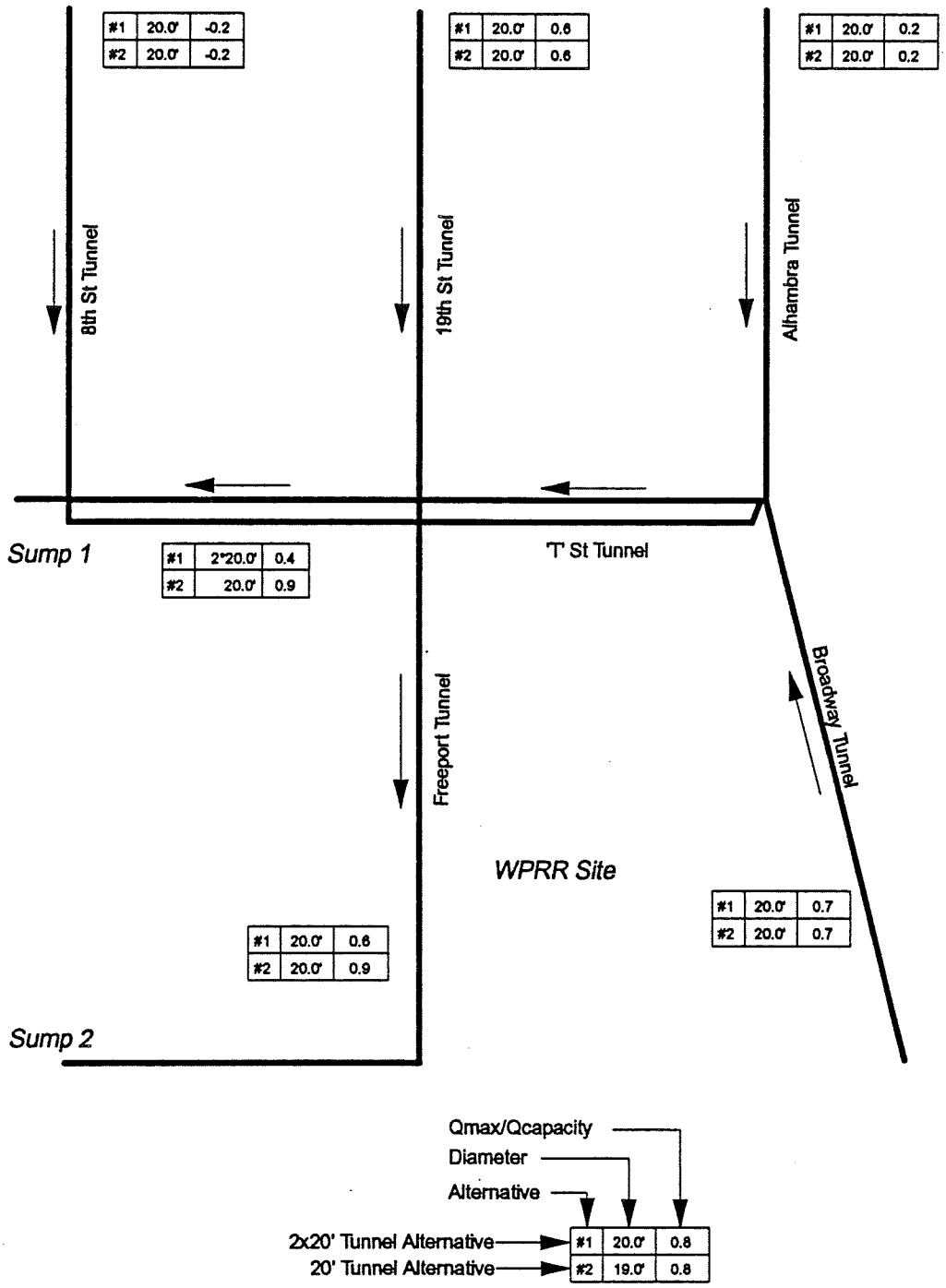
#1	20.0'	0.4
#2	19.0'	0.5
#3	18.0'	0.6
#4	17.0'	0.6

#1	20.0'	0.8
#2	19.0'	0.8
#3	18.0'	0.8
#4	17.0'	0.8

SCHEMATIC LAYOUT FOR THE 10-YEAR TUNNEL ALTERNATIVE CONFIGURATIONS CONSIDERED IN THE TUNNEL OPTIMIZATION EVALUATION

HDR

Figure 5-5



SCHEMATIC LAYOUT FOR THE 25-YEAR TUNNEL ALTERNATIVE CONFIGURATIONS CONSIDERED IN THE TUNNEL OPTIMIZATION EVALUATION

HDR

Figure 5-6

D.5.a. Maximizing Tunnel Storage--

In maximizing tunnel storage, tunnel discharge will only occur once the tunnel is full and will terminate once the inflow to the tunnel stops. Tunnel outlet control for maximizing tunnel storage utilization, as shown in Figure 5-7, was modelled as a tunnel overflow set at a control elevation of -18.00 feet MSL. The overflow was modelled as a culvert type conduit (conduit 6443 in Figure 5-7) 100 feet wide and 3 feet deep. This overflow conduit simulates either an overflow weir or an outlet pump system.

In setting up the model, it proved necessary to provide a buffer storage volume above the maximum discharge control elevation to equalize the effect of peak inflows in the tunnel. Based on a sensitivity analysis it was concluded that a control depth of 3 feet below the uppermost crown elevation of the tunnel system (-15.00 feet MSL) provides sufficient buffer storage capacity, and storage utilization of more than 90% for tunnel systems varying between 12 and 20 feet in diameter.

The above model set-up complies with the City's requirements in that it maximizes the storage of the tunnel while minimizing the discharges to the river after inflow to the tunnel has stopped. It also provides storage of combined sewage for CSO control.

The disadvantage of this approach is that the tunnel system might provide a maximized storage volume in excess of the CSO storage requirements. According to guidelines set for this project, all inflow stored after the storm event must be treated at the Sacramento Regional Wastewater Treatment Plant. The excess stored volume would need to be treated, increasing treatment costs and limiting the rate and the time it will take to empty the storage facilities. The estimated drainage time for the tunnel system, for tunnel diameters between 15 and 20 feet, ranges between approximately 2 and 4 days[1] respectively.

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[1] Based on an emptying rate of approximately 28 mgd, i.e. the difference between the maximum secondary treatment rate of 60 mgd and the constant sanitary inflow rate of approximately 32 mgd.

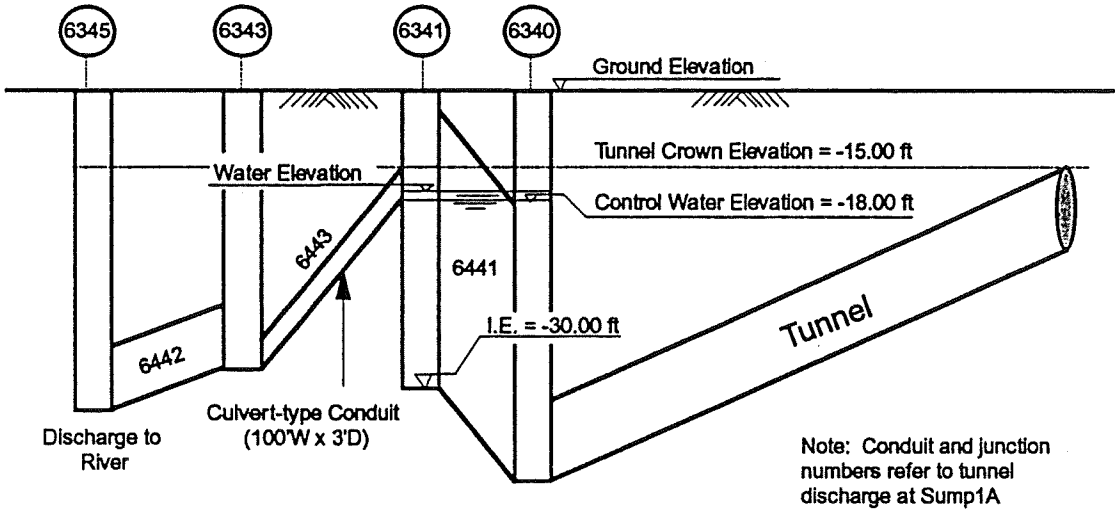
D.5.b. Varying Outlet Control Elevations--

As an alternative to the above procedure, the outlet control elevations can be varied between the maximum of -18.00 feet MSL, as defined to maximize tunnel storage, and the minimum outlet control elevation that will provide sufficient storage to satisfy the CSO control requirements.

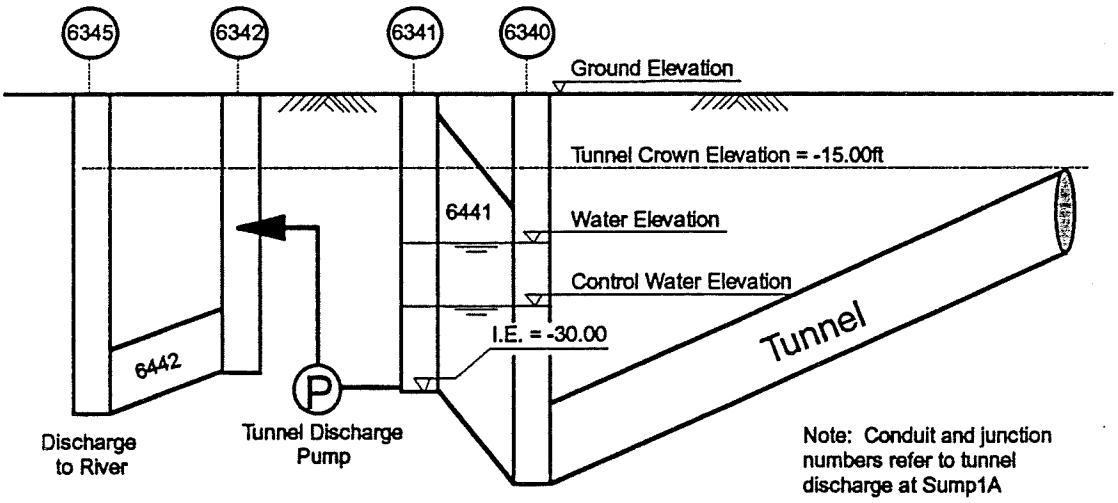
The tunnel outlet control was modeled, as shown in Figure 5-7, by replacing the tunnel overflow conduit in the previous case with an outlet pump system. The selected pump system was modeled to pump at a variable rate, starting up the pump at a given control level and increasing the pump rate with the rising water level. The maximum pumping rate occurs when the water level reaches the uppermost crown elevations of the tunnel system (-15.00 feet MSL). A typical set of pump curves for modeling the outlet control pumps at various outlet control elevations are shown are shown in Figure 5-8. For each configuration, the pump rate at the given control elevation is zero and increases to the maximum overflow rate occurring for the tunnel outlet configured for maximum storage with the control elevation at 18.00 feet MSL.

This approach to modeling the outlet control elevation meets the City requirement to store the CSO volume. It also pumps inflow in excess of the CSO storage volume to the river, minimizing the volume pumped to secondary treatment as well as emptying time for the tunnel system.

The SSWMM91, which only allows pump rates to vary with the water elevation, limits the ability to simulate the requirement that outflow from the tunnel be terminated once the tunnel is no longer required for flood control purposes. The model set-up used approximates but can not entirely meet the given criteria. Instead the pumping from the tunnel will continue until the water level in the tunnel has dropped to the outlet control elevation. The volume discharged after inflow to the tunnel has stopped depends on the discharge control elevation setting. At the maximum discharge control level this volume becomes insignificant. Refer to Appendix D of this memorandum and Figure D-6 in that appendix for further discussion.



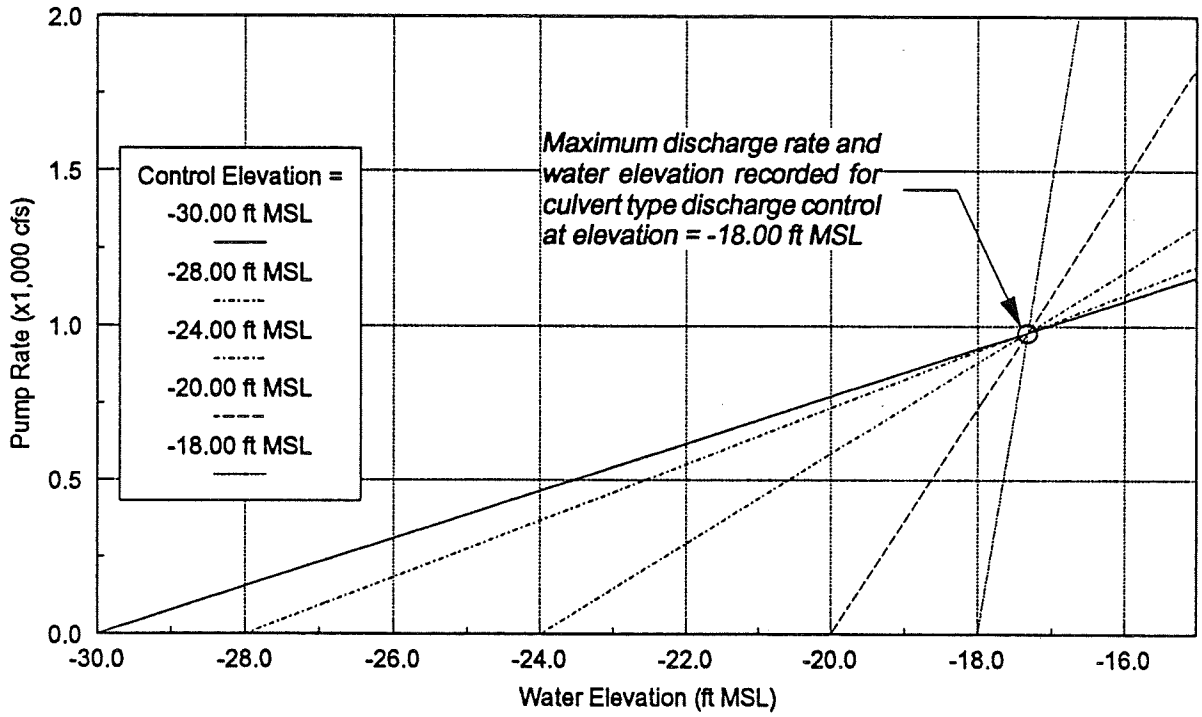
SCHEMATIC ILLUSTRATION OF THE TUNNEL DISCHARGE CONTROL AS MODELED FOR THE MAXIMIZED STORAGE ALTERNATIVES



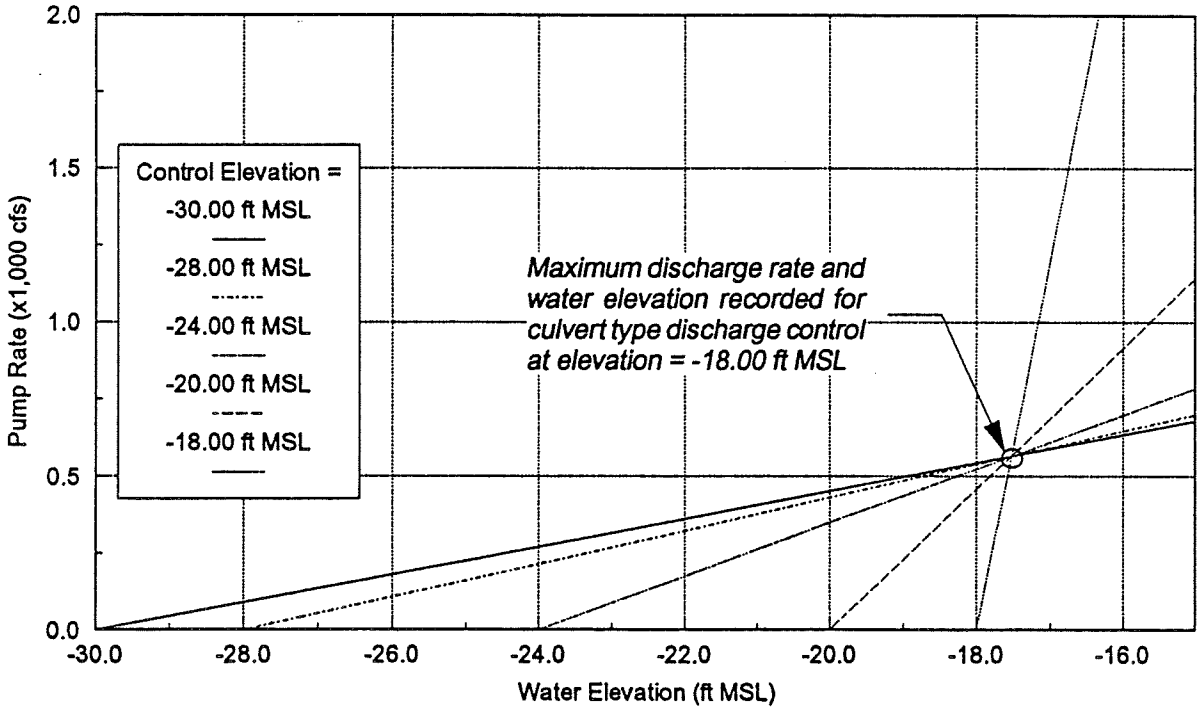
SCHEMATIC ILLUSTRATION OF THE TUNNEL DISCHARGE CONTROL AS MODELED FOR THE CONTROL ELEVATION EVALUATION ALTERNATIVES

HDR

Figure 5-7



Pump Curves Selected for the Sump1a Discharge Pumps of the 17 foot 10-Year Tunnel Configurations.



Pump Curves Selected for the Sump2 Discharge Pumps of the 17 foot 10-Year Tunnel Configurations.

HDR

Figure 5-8

D.6. Operation of Tunnels for CSO Control

The tunnel alternatives considered in the CSO control analysis were limited to the 5- and 10-year alternatives. In order to determine the storage volume which would contain the 1/year and 5/year CSO design storms for the different flood control alternatives, operational changes needed to be made to the model.

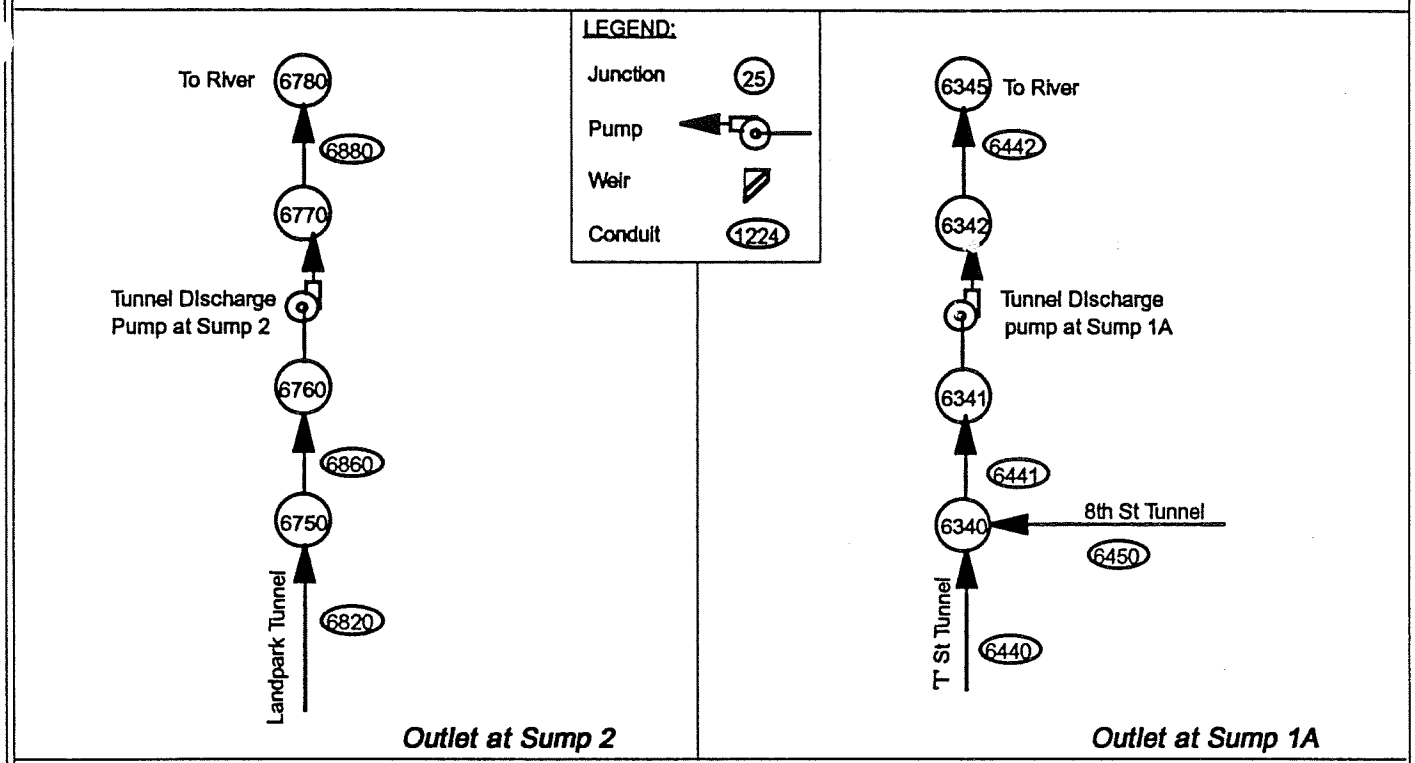
For each flood control alternative, the heights of the diversion weirs were set to maximize use of the existing combined sewer system, while diverting enough flow to mitigate flooding. Consequently the weirs were set lower for the 10-year design storm and higher or removed for the more frequent storms. In order to evaluate the 10-year tunnel alternative with the more frequent CSO storms, the weirs were set to the 2-year weir configuration, (the 5 weirs in the Land Park area) maximizing the use of the existing CWCS.

Another model change made for CSO control analysis was the operation of Pioneer Reservoir. For flood control purposes it was assumed that overflow from the existing Pioneer Reservoir will be discharged to the river, regardless of whether storage was available in the tunnel. However, for CSO control, the overflow from Pioneer Reservoir was diverted to the tunnel at Sump 1A. The model configurations for the flood control and the CSO control models at the tunnel outlets are illustrated in Figure 5-9.

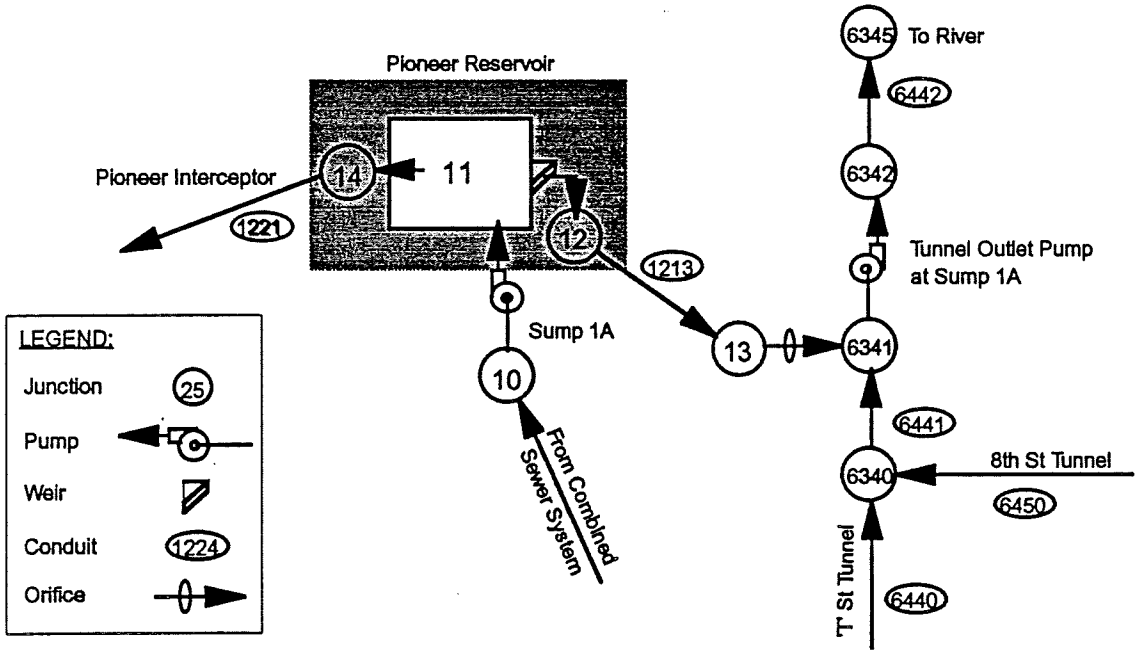
After the above model modifications were made, the CSO design storms were then applied to the flood control alternatives to determine the amount of tunnel storage volume required for CSO control.

E. TUNNEL ANALYSIS RESULTS

The approach to determine the least-cost alternatives for the 5-, 10-, and 25-year flood control, and 5- and 10- year alternatives using the 1/year and 5/year CSO events can be outlined as follows:



SCHEMATIC OF THE TUNNEL DISCHARGE CONTROL AS MODELED FOR FLOOD CONTROL ALTERNATIVES



SCHEMATIC OF THE TUNNEL DISCHARGE CONTROL CONFIGURATION AT SUMP 1A AS MODELED FOR THE CSO CONTROL ALTERNATIVES

HDR

Figure 5-9

1) The least-cost tunnel alternative for maximized storage was determined by comparing the total tunnel cost of the alternatives. This total tunnel cost includes the construction cost of the tunnels and the tunnel outlet pumping stations.

2) For the least-cost maximized storage alternative(s) the effect of lowering the outlet control elevation below the identified maximum of -18.00 feet MSL was investigated to determine the most cost effective outflow control elevation.

3) By routing the identified CSO events through the 5- and 10-year tunnel models, the required CSO storage requirements were estimated. Based on this information and the above tunnel optimization information, the least-cost CSO tunnel alternatives were selected.

The results of this analysis are reviewed in the subsequent discussion. An evaluation of the tunnel dynamics is attached as Appendix D to this memorandum.

E.1. Maximum Storage Analysis

E.1.a. 5-Year Tunnel Alternative--

The model output showed that, when routing a 5-year, 6-hour storm through the tunnel model, both the 14- and 15-foot tunnel configurations provide sufficient storage for all inflow. The smaller 14-foot tunnels, however, required pumps at both tunnel outlets to pump the tunnel inflow in excess of the available storage capacity to the river.

The comparison of the tunnel costs, as shown in Figure 5-10, indicates that the total cost of the 14-foot configuration (tunnel conduits and pumping stations) is lower than that of the 13 foot configuration, although the tunnel cost for the 13-foot tunnel conduits is lower than that of the 14-foot tunnel conduits.

E.1.b. 10-Year Tunnel Alternative--

The analysis of the 10-year tunnel configurations indicated that the storage of even the largest tunnel configuration (the 20 foot tunnel) can only capture

approximately 80% of the total tunnel inflow. All 10-year tunnel alternatives, therefore, required outlet pumping stations. The cost comparison between the 10-year tunnel alternatives (see Figure 5-10) showed a cost reduction with a reduction in tunnel diameter. The 17-foot tunnel configuration was the smallest 10-year tunnel configuration to comply with the selected design requirements.

Based on this cost analysis, the 17-foot tunnel proved to be the least-cost maximized storage configuration for the 10-year tunnel system.

E.1.c. 25-Year Tunnel Alternative--

Only two 25-year tunnel configurations were evaluated, i.e. a 20-foot tunnel and a system with two parallel 20-foot tunnels along T Street. Of these two configurations, the single 20-foot tunnel system with outlet pumps proved to be less expensive.

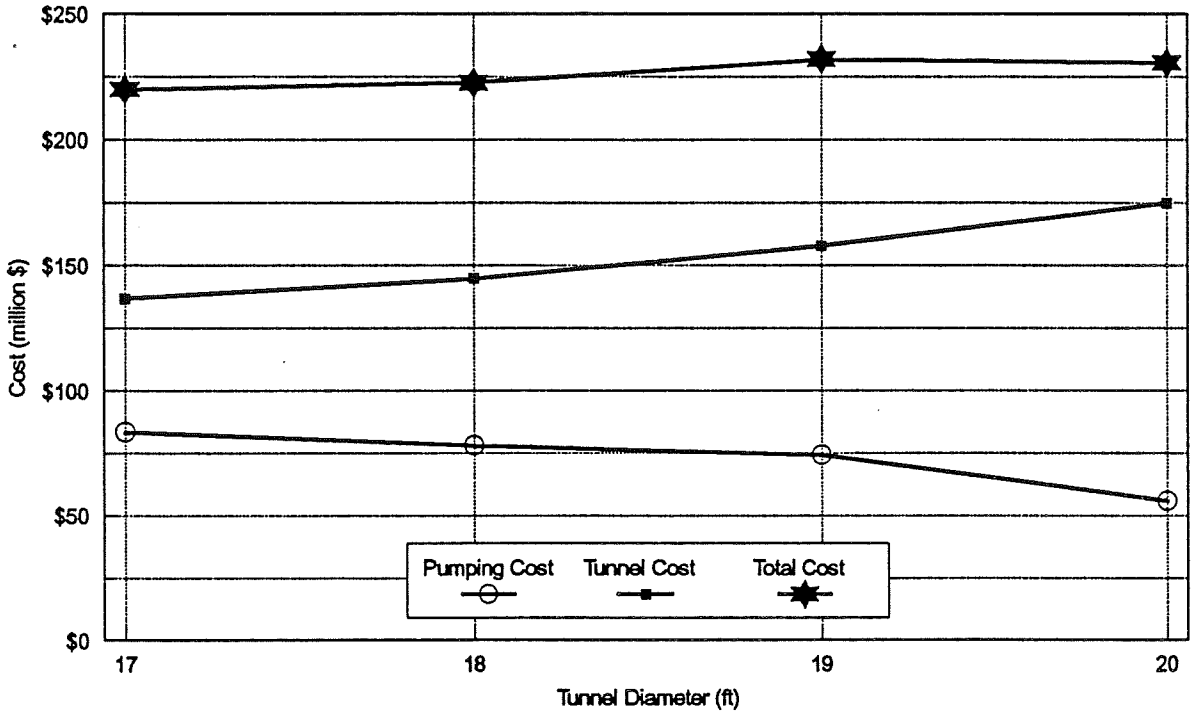
E.2 Outlet Control Analysis

E.2.a. 5-Year Tunnel Alternative--

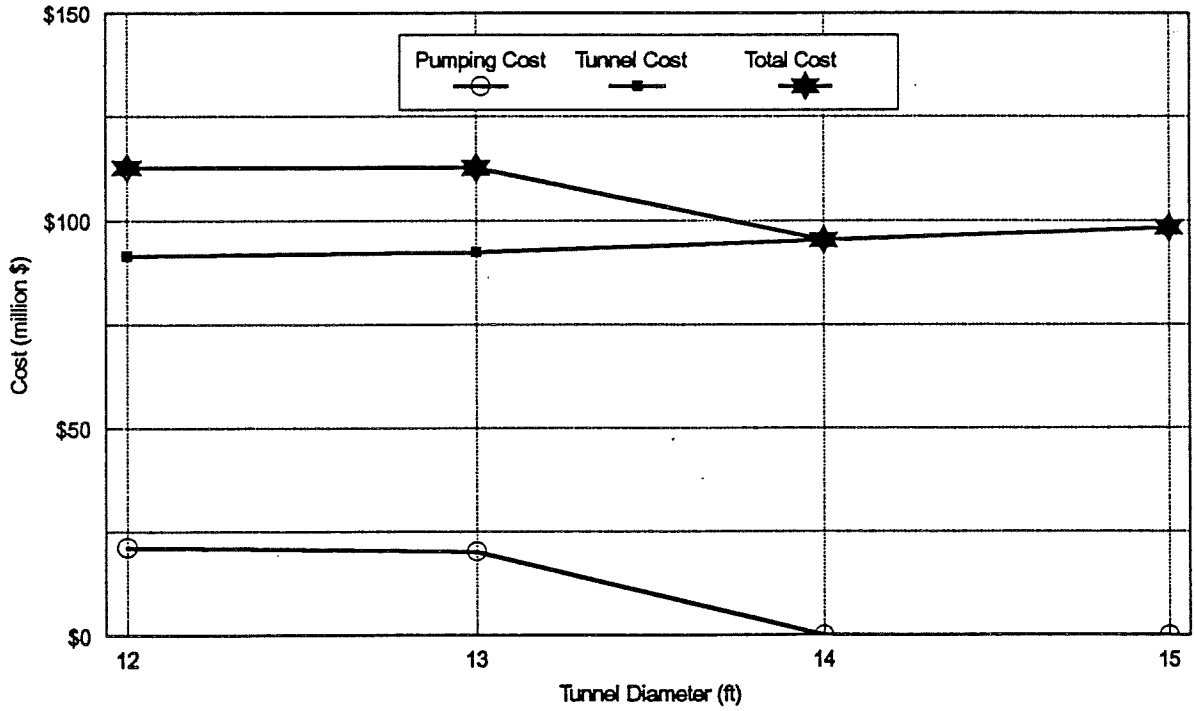
In the above evaluation of the least-cost tunnel with maximized storage, it was apparent that the cost of the 14-foot tunnel was lower than that of the 13-foot mainly because the 14-foot tunnel system required no outlet pumps. In lowering the outlet control elevation of the 14-foot tunnel, the available storage was reduced to the point where tunnel inflow will exceed the available storage volume, resulting in a tunnel discharge. Such a system will require additional cost for a pump station over and above the tunnel cost, and might therefore be more expensive than the 13-foot tunnel system. The evaluation for the least-cost tunnel configuration for control elevations below the maximum elevation of -18.00 ft MSL was therefore extended to incorporate the 13-foot tunnel system.

The results of the cost comparison between the 13- and 14-foot tunnel configurations for control elevations ranging from -18.00 to -30.00 feet MSL are shown in Figure 5-11. The following was determined from the analysis:

1) For control elevations below -18.00 feet MSL the 14-foot tunnel becomes more expensive than the maximized storage alternative; and



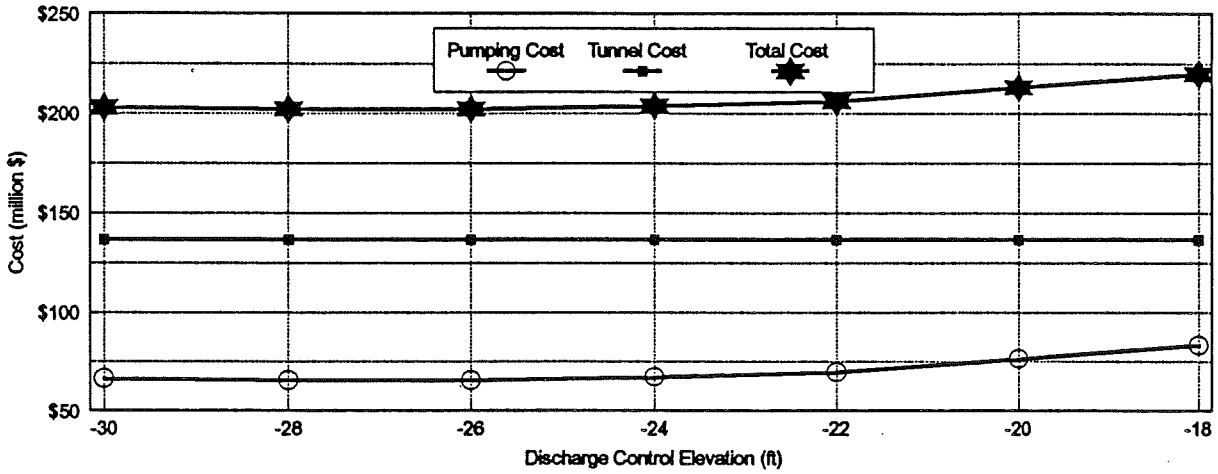
COST COMPARISON BETWEEN 10-YEAR TUNNEL CONFIGURATIONS WITH VARIATIONS IN THE TUNNEL DIAMETER



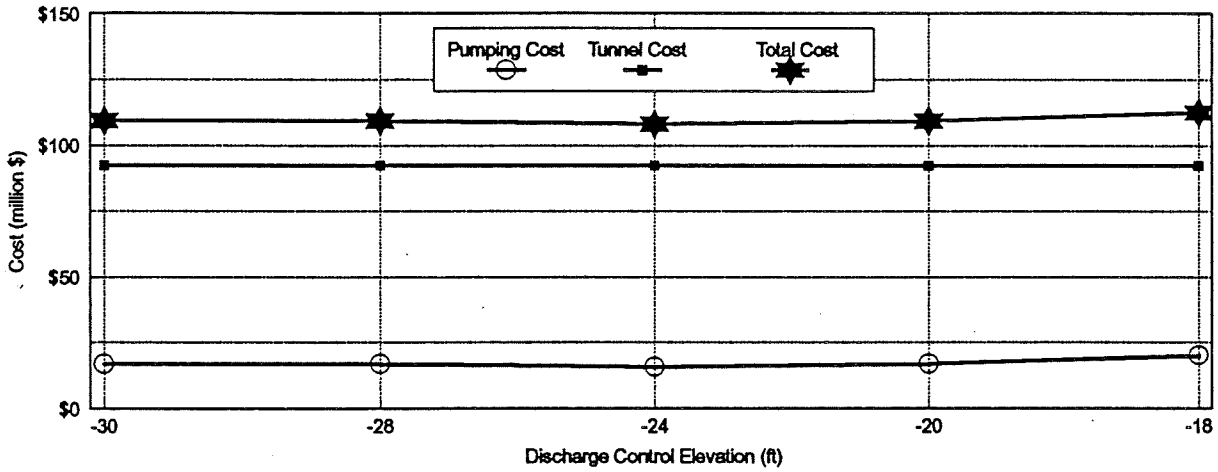
COST COMPARISON BETWEEN 5-YEAR TUNNEL CONFIGURATIONS WITH VARIATIONS IN THE TUNNEL DIAMETER

HDR

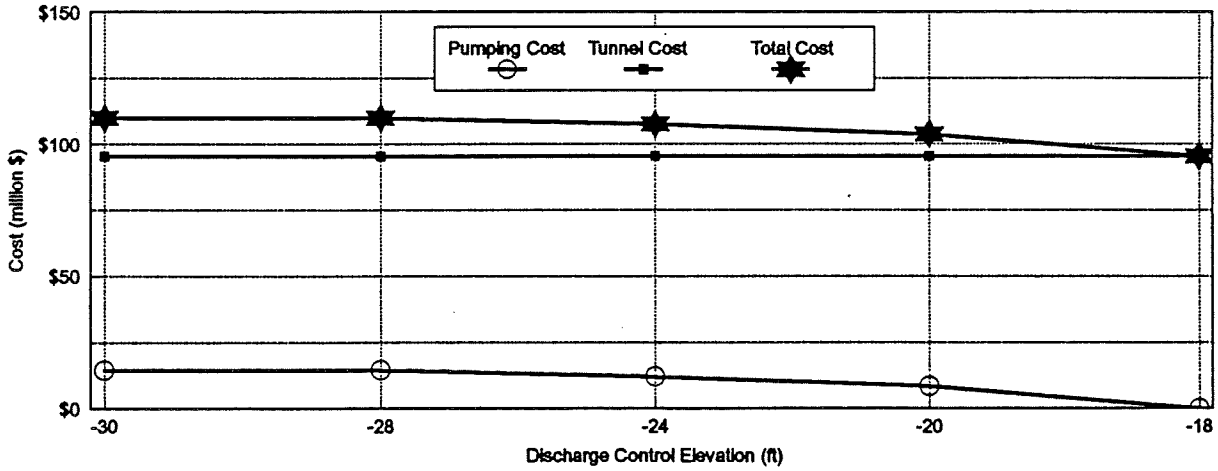
Figure 5-10



COST COMPARISON BETWEEN 17 FOOT 10-YEAR TUNNEL CONFIGURATIONS WITH VARIATIONS IN THE TUNNEL DISCHARGE CONTROL ELEVATION



COST COMPARISON BETWEEN 13 FOOT 5-YEAR TUNNEL CONFIGURATIONS WITH VARIATIONS IN THE TUNNEL DISCHARGE CONTROL ELEVATION



COST COMPARISON BETWEEN 14 FOOT 5-YEAR TUNNEL CONFIGURATIONS WITH VARIATIONS IN THE TUNNEL DISCHARGE CONTROL ELEVATION

HDR

Figure 5-11

2) For control elevations less than -24.00 feet MSL the cost difference between the 13 and 14 foot tunnels becomes insignificant.

It can therefore be concluded that the 14-foot tunnel at outlet control elevation -18.00 MSL is the least-cost 5-year tunnel alternative for flood control.

E.2.b. 10-Year Tunnel Alternative--

Only the 17-foot, 10-year tunnel was considered in the evaluation of cost effects due to a reduction in outlet control elevation. As indicated in Figure 5-11, the total cost decreased for a decrease in the control elevation from the maximum of -18.00 feet MSL to -24.00 feet MSL. Below the elevation of -24.00 feet MSL there is no significant cost reduction with further reduction in the control elevation.

A 17-foot, 10-year tunnel with a control elevation of less than -24.00 feet MSL is thus regarded as the least-cost alternative for flood control.

E.2.c. 25-Year Tunnel Alternative--

The 25-year tunnel was not evaluated for cost effects related to variations in the outlet control elevation.

E.3. CSO Analysis

E.3.a. 5-Year Tunnel Alternative--

The total tunnel inflow when routing the 1/year CSO event through the 5-year CSO tunnel configuration, as outlined previously, exceeds the maximum available storage capacity of the 14-foot tunnel. The 15-foot tunnel, however, provided sufficient storage for CSO control. This tunnel configuration was therefore selected as the 1/year CSO, 5-year tunnel alternative.

The 5/year CSO event can be handled by the existing system and therefore does not require tunnel storage. For cost comparison purposes, the 5/year CSO, 5-year tunnel alternative was assumed to be the 5-year least-cost tunnel, which is the 14-foot tunnel without tunnel discharge pumps.

E.3.b. 10-Year Tunnel Alternative--

The tunnel inflow for the 1/year CSO event filled the 17-foot, 10-year tunnel to an elevation of approximately -23.00 feet MSL. The 1/year CSO, 10-year tunnel was therefore taken as a 17-foot tunnel with an outlet control elevation of -23.00 feet MSL.

As with the 5-year tunnels, the 5/year storm did not spill into the 10-year tunnel configuration. The 5/year CSO, 10-year tunnel was thus selected to be equivalent to the least-cost 10-year flood control alternative.

F. TUNNEL ANALYSIS SUMMARY

This section has reviewed the applied design criteria, modeling approach and the alternative analysis method used to determine the least-cost alternatives for the 5-, 10-, and 25-year flood control tunnel systems, and for the 1/year and 5/year CSO tunnels for the 5- and 10-year tunnel configurations. The recommended alternatives are summarized in Table 5-5. In conclusion the following aspects of the recommended alternatives should be highlighted:

1) The CSO inflow into the 5-year tunnel configuration exceeds the available storage of the 14-foot flood control tunnels, and therefore requires the larger 15-foot tunnel configuration to provide CSO control storage.

2) The 5/year CSO event did not spill into the tunnel system. Consequently this alternative did not require any specific storage requirements for CSO control, and was therefore selected to be equivalent to the flood control alternatives.

3) Since 25-year flood control alternative storage capacity (14.9×10^6 cubic feet), is considerably greater than the required CSO storage volume for the 1/year CSO event (8×10^6 cubic feet), the 25-year, 1/year and 5/year CSO alternatives were taken to be the same as the 25-year flood control alternative.

The total project costs for the tunnel alternatives are presented and evaluated in Section 11.

TABLE 5-5. SUMMARY OF RECOMMENDED TUNNEL ALTERNATIVES				
Tunnel Configuration	Alternative Description			
	1/year CSO		5/Year CSO & Flood Control	
	Diameter (feet)	Control Elevation (ft MSL)	Diameter (feet)	Control Elevation (ft MSL)
5-Year Tunnel	15	-18.00	14	-18.00
10-Year Tunnel	17	-23.00	17	< -24.00
25-Year Tunnel	-	-	20	-18.00

SECTION 6
STORAGE/PUMPING ALTERNATIVES

A. INTRODUCTION

This section describes the analysis of alternatives which provide for interim storage of combined wastewater. The storage facilities described in this section are similar in construction and operation to the existing Pioneer Reservoir. As with the tunnel alternatives (see Section 5), the storage reservoirs considered would provide the CWCS with a means of storing excess wastewater flows diverted from the existing combined sewer system, thereby providing a level of control for flooding and CSOs.

In this study, storage reservoirs were also considered for use in attenuating peak pumping requirements in alternatives where pumping over long distances was involved. For all sites considered, the storage facilities would be emptied back into the collection system for treatment at the SRWTP, after a storm event passed and flows in the existing collection system subsided. This is in accordance with the assumptions identified in Section 3.

As listed in Table 3-1, there were a total of five storage alternatives evaluated for each of the 2-, 5- and 10-year design storm events. Two additional 2-year storage alternatives were added to this for a total of 17 storage alternatives evaluated for each of the two CSO storm events.

Three sites were considered as candidate storage sites for this study. These sites are:

- The Pioneer Reservoir Site (P)
- The Western Pacific Railroad (WPRR or WP) Site
- The UC Davis Medical Center Site (UCD)

The five original alternatives evaluated for each flood control design storm were identified according to which storage site(s) were used for that alternative. These storage alternatives are listed in Table 6-1.

TABLE 6-1. STORAGE ALTERNATIVE NOMENCLATURE	
Alternative Abbreviation	**Storage Sites Used in Alternative**
UCDP	UC Davis and Pioneer Sites
WPRR	Western Pacific Site
WPP	Western Pacific and Pioneer Sites
WPUCD	Western Pacific and UC Davis Sites
WPUCDP	Western Pacific, UC Davis and Pioneer Sites

For each of the alternatives listed in Table 6-1, a network of diversion weirs and consolidation conduits similar to those used for the tunnel alternatives were included to collect and convey combined sewage to the storage reservoirs.

In addition to the above alternatives, two other 2-year alternatives were evaluated which did not include any diversion weirs. Instead, the improvements for these alternatives upsized the existing collection system as necessary to control flooding and convey all inflow to the existing Sumps 1A and 2. These alternatives are discussed later in this section. Upsized CWCS conveyance systems included storage at the Pioneer Reservoir site and storage at the WPRR site, and were evaluated for the 2-year design storm only.

As with the tunnel alternatives, the objective of the Phase 3 storage alternative analysis was to devise improvements to the existing CWCS which would control the occurrence of flooding and CSOs for given design storms. These design storms are discussed in the Phase 3, Appendix H technical memorandum *Design Storms, Rainfall Analysis and Flood Volume Maps for the Phase 3 Analysis*, dated July 1, 1992.

Unit costs for the components of these facilities were taken from Section 4, and used to estimate total alternative costs. These total project costs are presented for all alternatives in Section 11.

B. STORAGE ALTERNATIVE MODELING ASSUMPTIONS

The modeling and analysis assumptions which were applied to all the alternatives analyzed with the SWMM are discussed in Section 3. The analysis assumptions used specifically for the storage alternatives are discussed below.

B.1. Storage Facility Configurations

The type of storage facilities considered are similar to the City's existing Pioneer Reservoir. These facilities are constructed as reinforced concrete reservoirs with internal baffles and weirs to allow inflow to fill one section before spilling over into the adjacent cell. With this arrangement, smaller storm events would not fill the entire reservoir. Consequently, the effort required to clean up after draining the reservoir is minimized.

Inflow to the storage reservoir sites would be delivered by gravity flow consolidation conduits. The invert elevations of these conduits were necessarily deep at the storage sites. The actual depth is dependent upon the length of gravity sewer between the storage site and the upstream weirs served by the sewers, and upon the required diameter of the sewer at the storage site. The invert elevations of these consolidation conduits at the storage sites would be lower than the floor elevation of the proposed storage reservoir. As a consequence, a pumping station would be required for each of the storage sites to lift the influent wastewater delivered by new consolidation conduits up into the storage reservoirs. This is illustrated schematically in Figure 6-1.

B.2. Total Containment Versus Discharge to the River

The Pioneer Reservoir site and, to a lesser extent, the WPRR site are located close enough to the Sacramento River to consider discharging wastewater diverted to these sites directly to the river. By contrast, the UC Davis Medical Center site is too far east to consider any direct discharge from this site to the river. Discharge of untreated combined wastewater from the Pioneer Reservoir and WPRR sites was considered acceptable if the volume of wastewater diverted to the

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TE-FIG40.DWG
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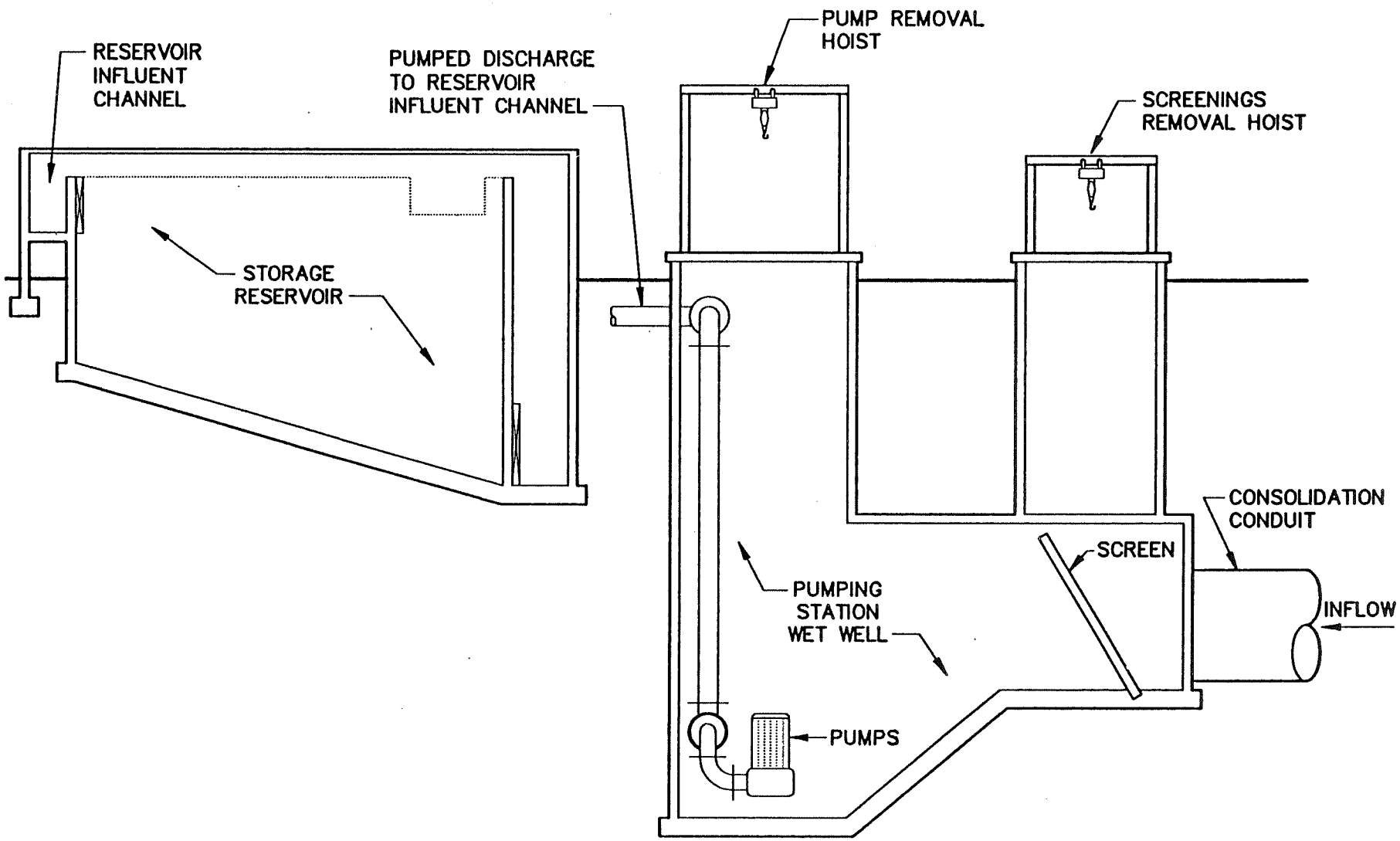


STORAGE RESERVOIR AND PUMPING STATION SCHEMATIC

HDR

Figure 6-1

site for the 2-, 5- or 10-year design storms exceeded the volume diverted to the site for either the 5/year or the 1/year CSO storm events.

A cost comparison between storage at the WPRR site and the direct discharge from the WPRR site to the river are summarized in Table 6-2. The cost estimates do not include any common cost such as the consolidation outflow pumping cost. The estimate for the storage includes the cost of the storage facility and the related site work. The cost estimated for discharge directly to the river includes the cost of the outfall sewer and the outfall structure at the river. The results summarized in Table 6-2 indicate that direct discharge to the river is more cost effective than storage. For the Pioneer Reservoir site, direct discharge should be less expensive and, therefore, even more cost effective, because this site is located next to the river and thus will require less piping cost.

Based on the results of this analysis, it was concluded that for all storage alternatives, inflow to the Pioneer Reservoir and WPRR sites in excess of the CSO volume (i.e. the inflow produced by the CSO design storm) will be pumped directly to the river.

C. STORAGE FACILITY DESCRIPTIONS

C.1. Storage at the Pioneer Reservoir Site

The storage reservoir proposed for the Pioneer Reservoir site would be located immediately northeast of the existing reservoir. Inflow would be delivered to the site from either the Pioneer Interceptor, Sump 1A or a new pumping station. The new pumping station would be constructed to deliver wastewater conveyed to the site by the new network of consolidation conduits. Flows could be directed to either the existing Pioneer Reservoir or the new storage reservoir, depending upon conditions during a particular storm event.

TABLE 6-2. COST COMPARISON BETWEEN STORAGE DIRECT DISCHARGE TO THE RIVER FOR WPRR ALTERNATIVES

Alternative	Storage Only		Direct Discharge to the River Only		
	Storage Volume (MG)	Storage Cost ($)	Pump Rate (cfs)	Outfall Diameter (inches)	Discharge Cost ($)
10-Year Flood Control:					
- WPRR	137	$124,725,000	2390	240	$30,617,000
- WPP	62	$58,200,000	995	156	$16,542,000
- WPUCD	104	$94,200,000	1870	216	$24,942,000
- WPUCDP	30	$31,000,000	485	108	$7,216,000
5-Year Flood Control:					
- WPRR	53	$50,550,000	1170	168	$17,046,000
- WPP	24	$25,900,000	495	108	$7,216,000
- WPUCD	42	$41,200,000	910	156	$16,542,000
- WPUCDP	14	$15,500,000	270	81	$4,497,000
2-Year Flood Control:					
- WPRR	6	$7,500,000	62	39	$2,362,000

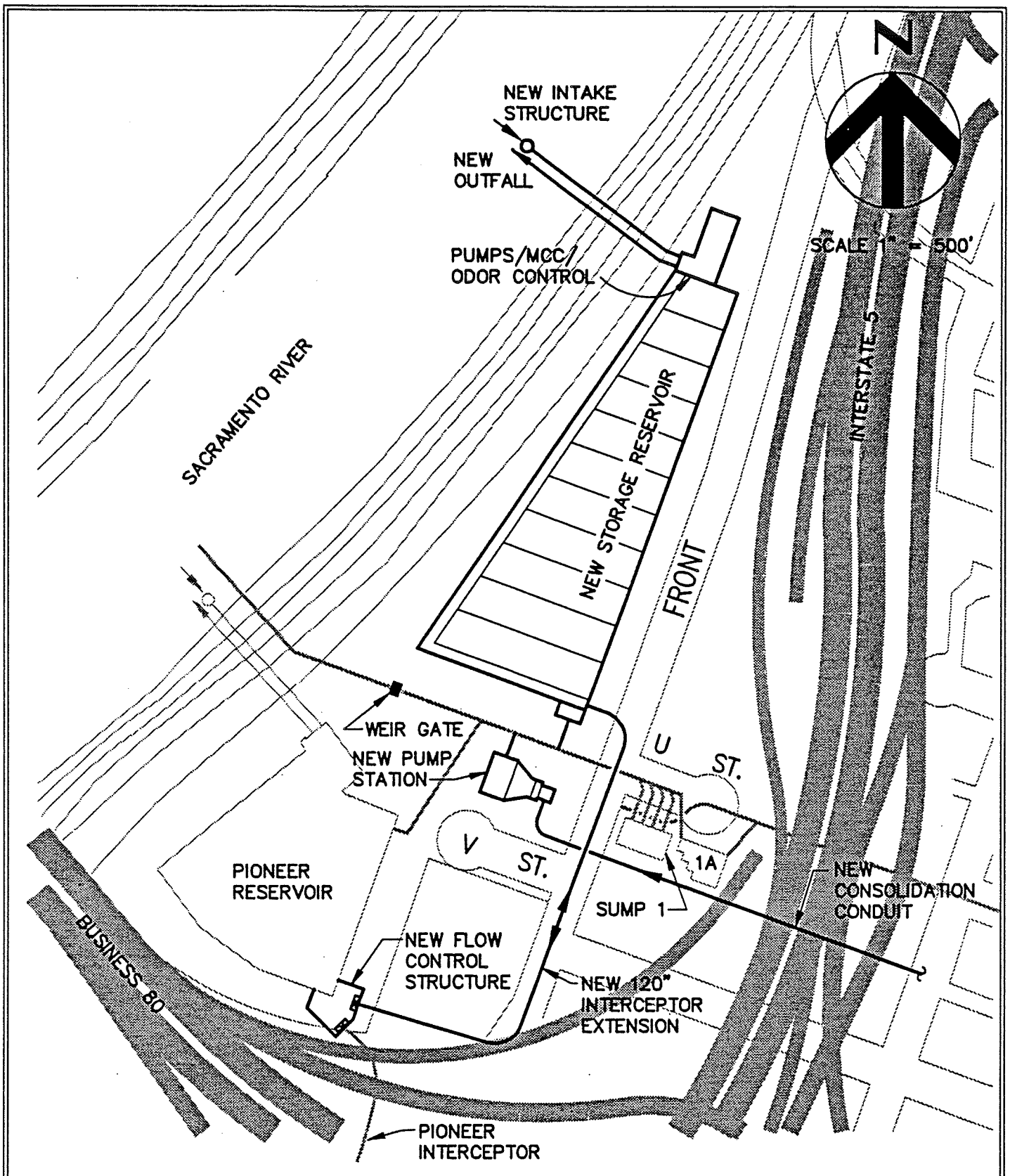
Both the existing and the new reservoirs would have a dedicated overflow discharge to the river and a flushwater intake from the river for washdown following a rainfall event. Each of the reservoirs would drain through the Pioneer Interceptor to the Sump 2 site for conveyance to SRWTP after flows in the CWCS had subsided. Pumps would be included for emptying the new reservoir and for its flushwater system. The motor control center and the new facility's odor control equipment would be located at the northern end of the reservoir structure, accessible by operations and maintenance vehicles from Front Street. A proposed arrangement for achieving this operational flexibility is shown in Figure 6-2.

As with the tunnel alternatives, the pumping station proposed for the Pioneer Reservoir site is intended to be separate from existing Sump 1A. The reason for this configuration is the invert elevations of the sewers entering Sump 1A are higher than the invert elevation of the new consolidation conduits entering the new pumping station at the Pioneer Reservoir site. If the inflow to Sump 1A were to be pumped by the new pumping station, there would be a higher lift and more energy unnecessarily expended.

C.2. Storage at the Western Pacific Site

Two options for development of the WPRR site are shown in Figures 6-3 and 6-4. In Figure 6-3, a new storage reservoir is located at the north end of the property. The network of consolidation conduits delivering wastewater to the site terminate at a new pumping station to the south of the reservoir. The new reservoir is located to the north of the existing 114-inch Donner Trunk. Sufficient separation distance between the reservoir and the sewer would be provided to ensure that the sewer could be excavated for repair or maintenance.

The new pumping station would deliver wastewater from the consolidation conduits to the reservoir. The influent station would include pumps within the reservoir structure that would empty the reservoir after the storm event into the Donner Trunk. Flushwater for the facility would be provided by a well field located on the WPRR site. Access to the facility for vehicle traffic would be off 24th Street.



PIONEER RESERVOIR SITE
DEVELOPED AS A STORAGE FACILITY

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16-FIG16.DWG
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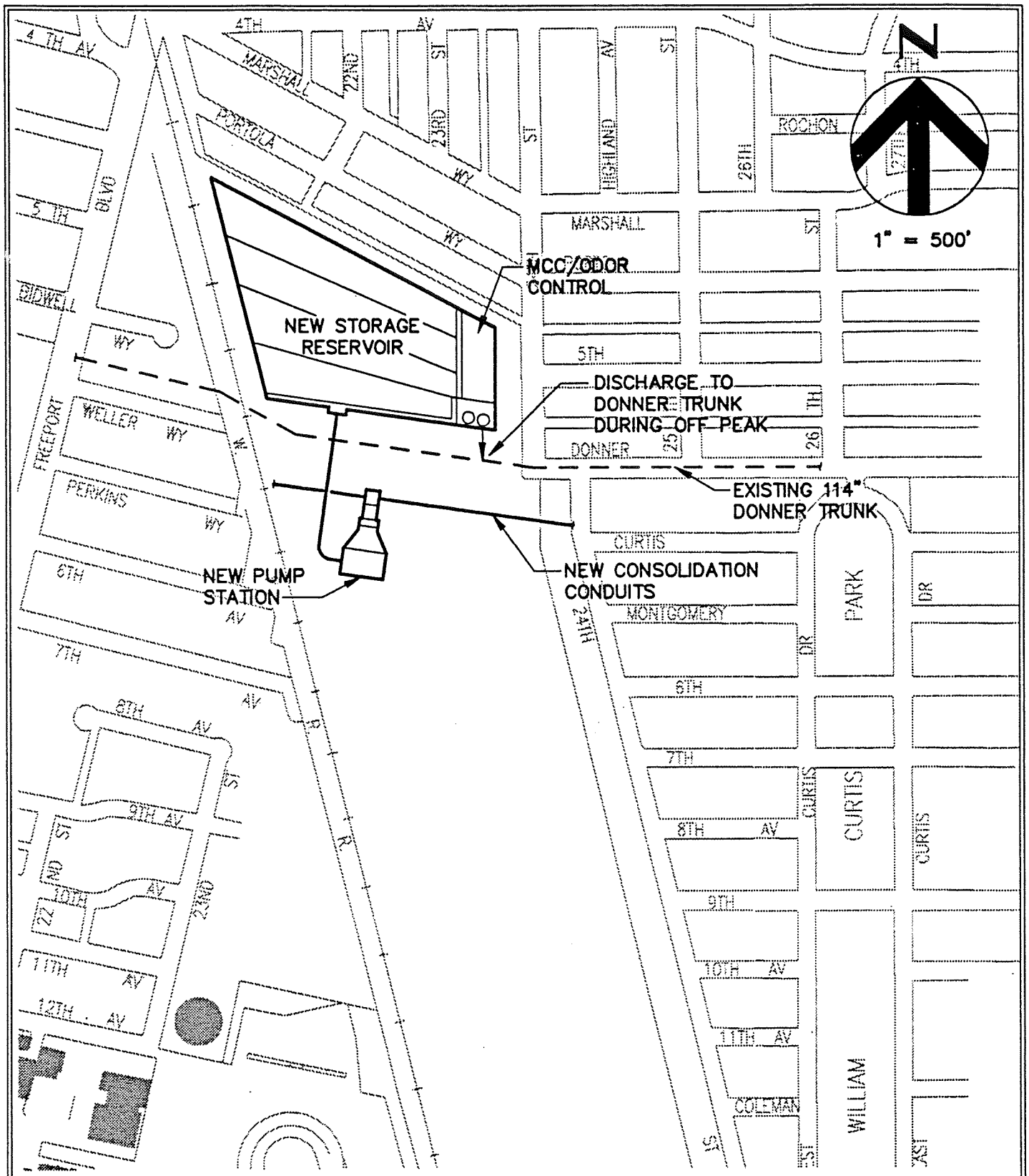
Figure 6-2



WESTERN PACIFIC RAILROAD SITE
DEVELOPED WITH STORAGE FACILITIES

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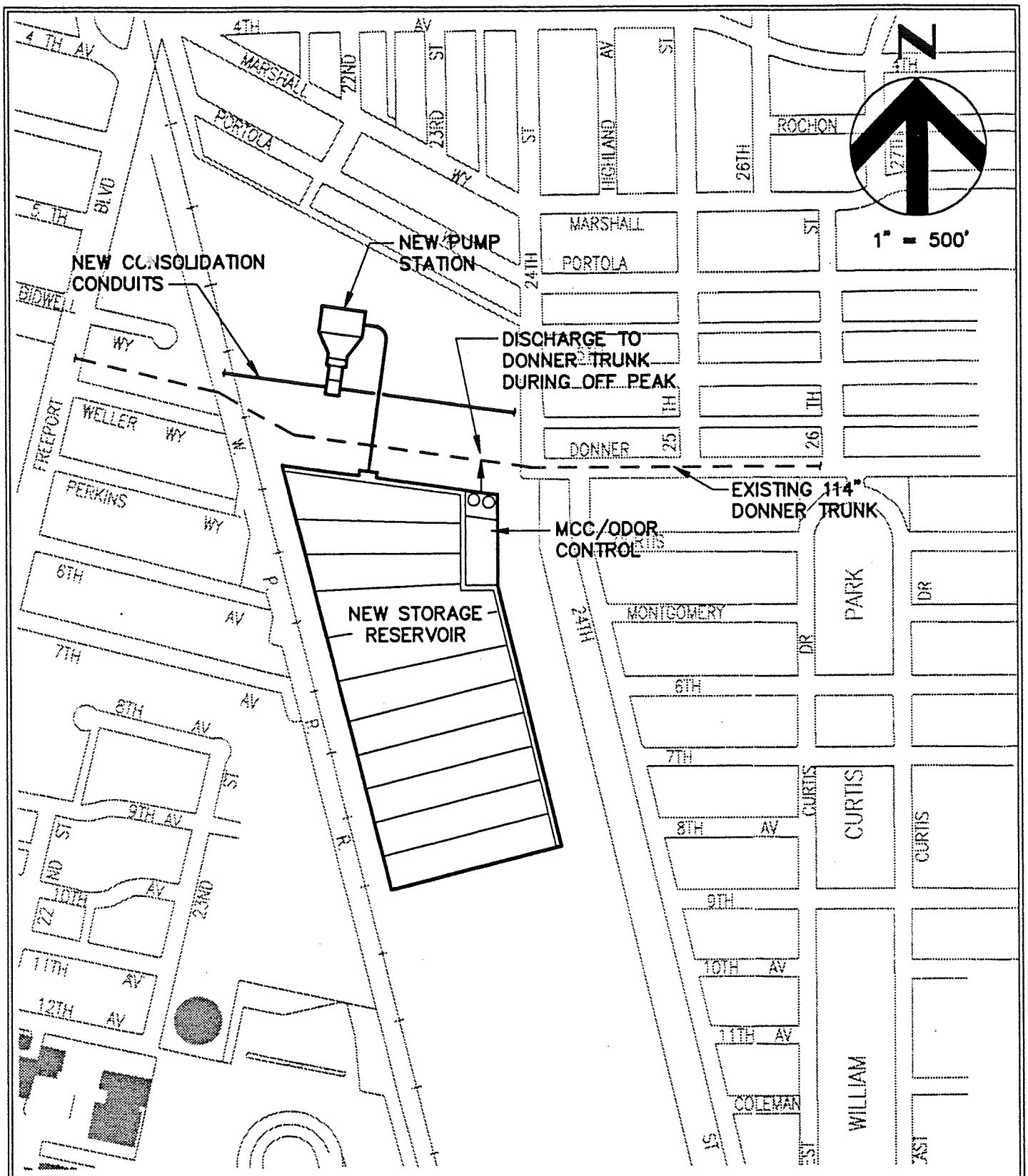
Figure 6–3



WESTERN PACIFIC RAILROAD SITE
DEVELOPED WITH ALTERNATE STORAGE FACILITIES

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Figure 6–4

A second alternative for developing the WPRR site with storage facilities is illustrated in Figure 6-4. In this case, the relative positions of the pumping station and the storage reservoir are reversed. This alternative still reserves an easement for access to the existing 114-inch sewer. It also allows a much larger storage reservoir to be constructed than shown in Figure 6-3, because the available area south of the 114-inch sewer is much larger than the area north of the 114-inch sewer.

Not shown in either Figures 6-3 or 6-4 are diversion structures downstream of the inflow pumping station. These structures allow for diversion of wastewater flows to the river for all volumes in excess of the CSO inflow volume to the site.

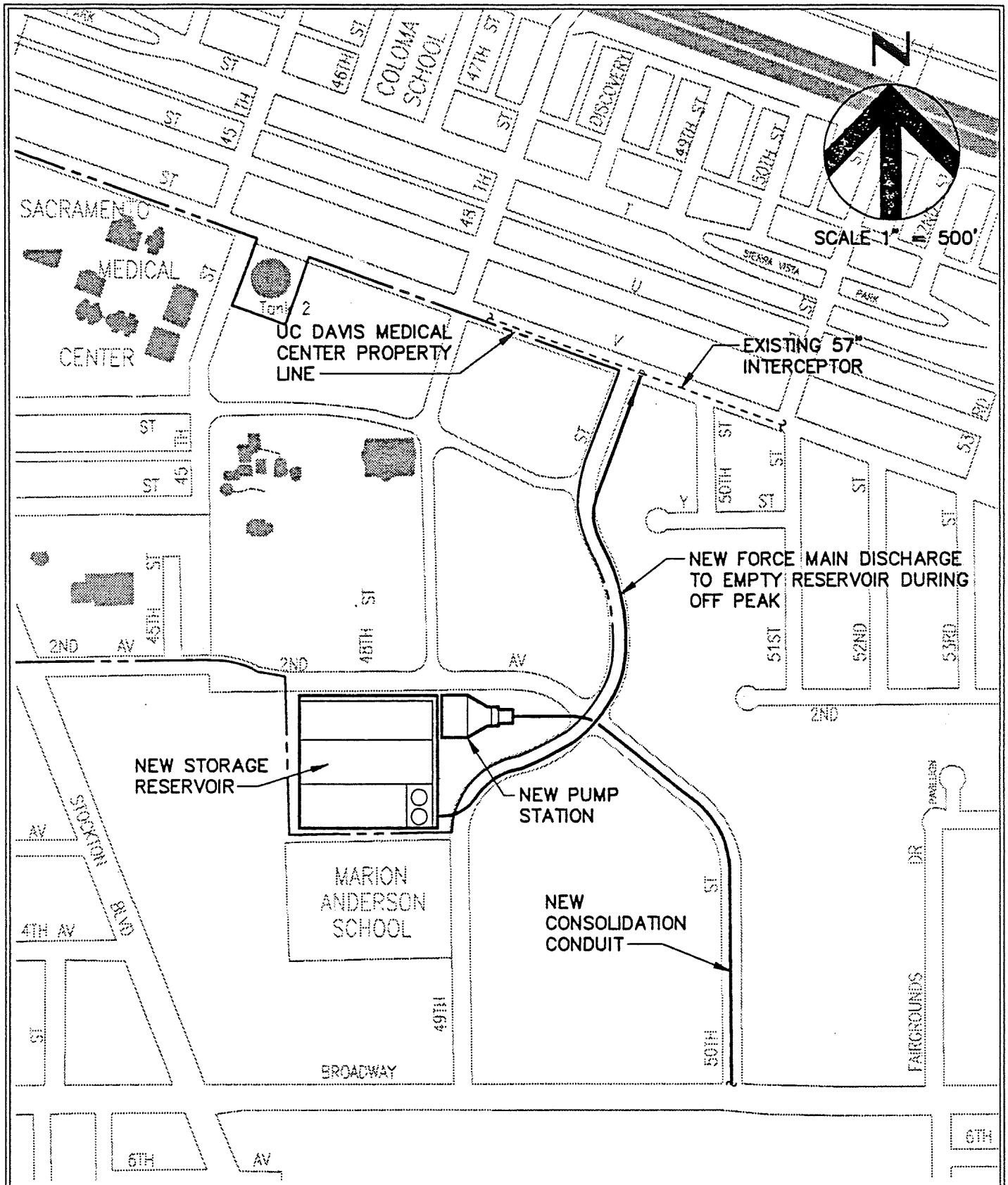
C.3. Storage at the UC Davis Medical Center Site

The UC Davis Medical Center site developed with storage facilities is illustrated in Figure 6-5. The consolidation conduit bringing inflow to the site would be routed up 50th Street to a pumping station at the intersection of 2nd Avenue and 48th Street. Pumps in the storage reservoir structure would empty the reservoir after the storm event into the existing 57-inch City Interceptor on V Street via a new force main in 49th Street.

Flushwater for the site would have to be developed from a well field in the vicinity of the site. No diversion structures for river discharge were proposed for the UC Davis site because the site is too far from the river to make a river discharge option cost effective.

C.4. WPRR, Pioneer Reservoir and UCD Site Considerations

Because this study is being conducted at the facility planning level, detailed investigation of the sites was beyond the scope of work. Therefore, it should be pointed out that the development of these sites has not been discussed or coordinated with the owners of these properties by the study team staff. The team's understanding is that ground shown in Figure 6-5 will possibly be developed as a maintenance center for the entire UC Davis Medical Center.



UC DAVIS MEDICAL CENTER
DEVELOPED AS A STORAGE FACILITY

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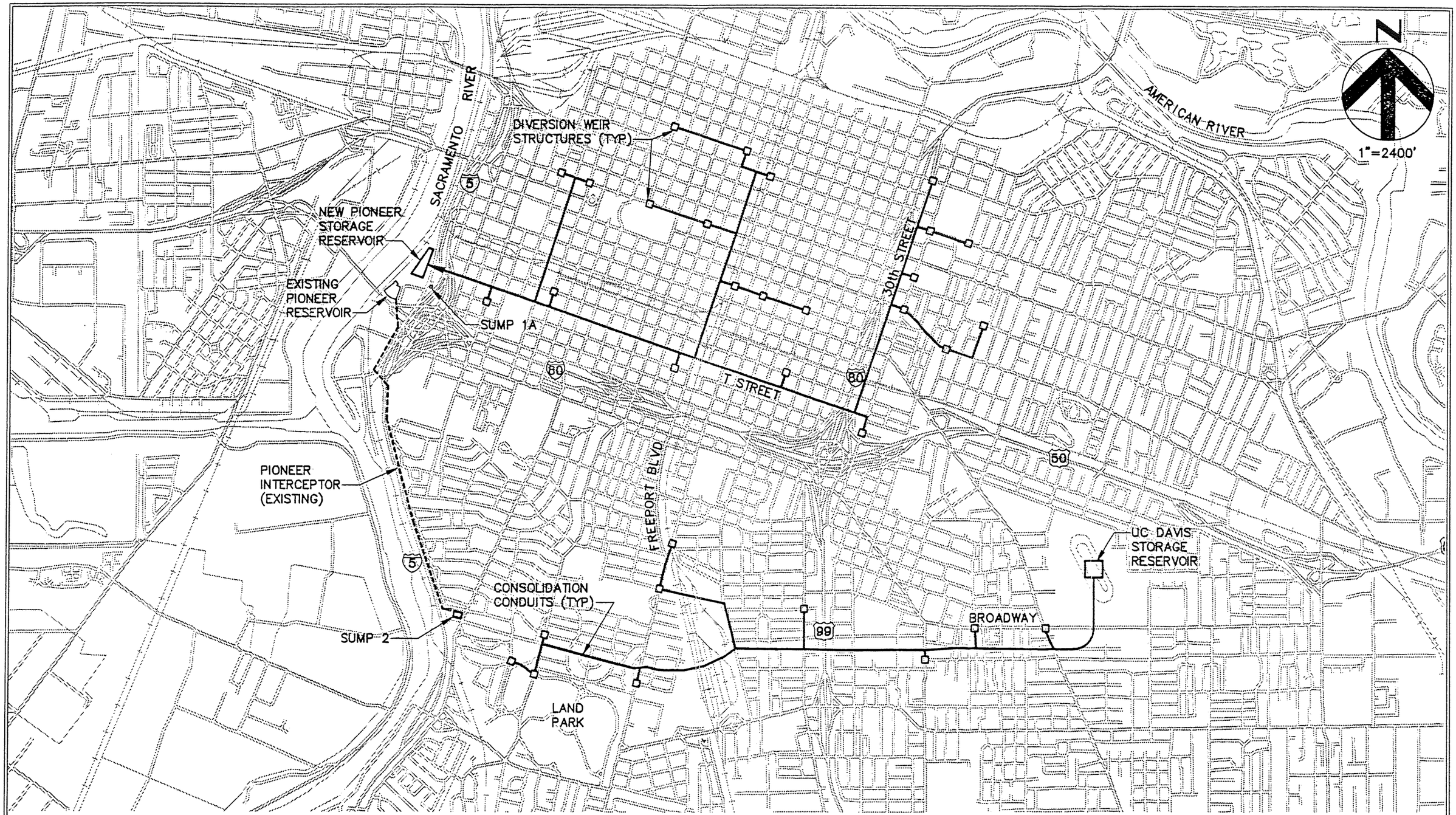
Figure 6–5



UC DAVIS/PIONEER STORAGE ALTERNATIVE WITH CONSOLIDATION CONDUIT NETWORK FOR 10-YEAR EVENT

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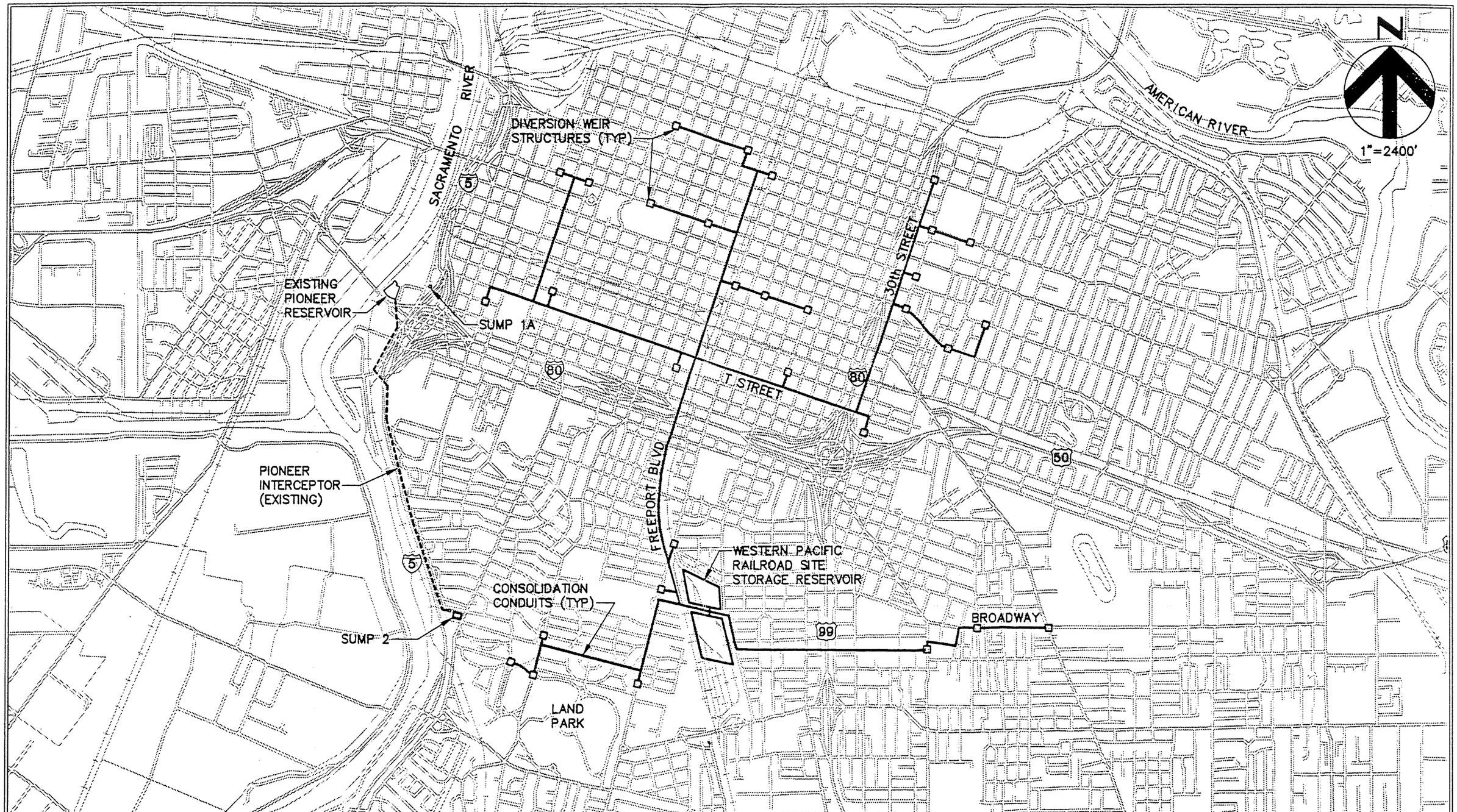
Figure 6-6



WESTERN PACIFIC RAILROAD SITE STORAGE ALTERNATIVE WITH CONSOLIDATION CONDUIT NETWORK FOR 10-YEAR EVENT

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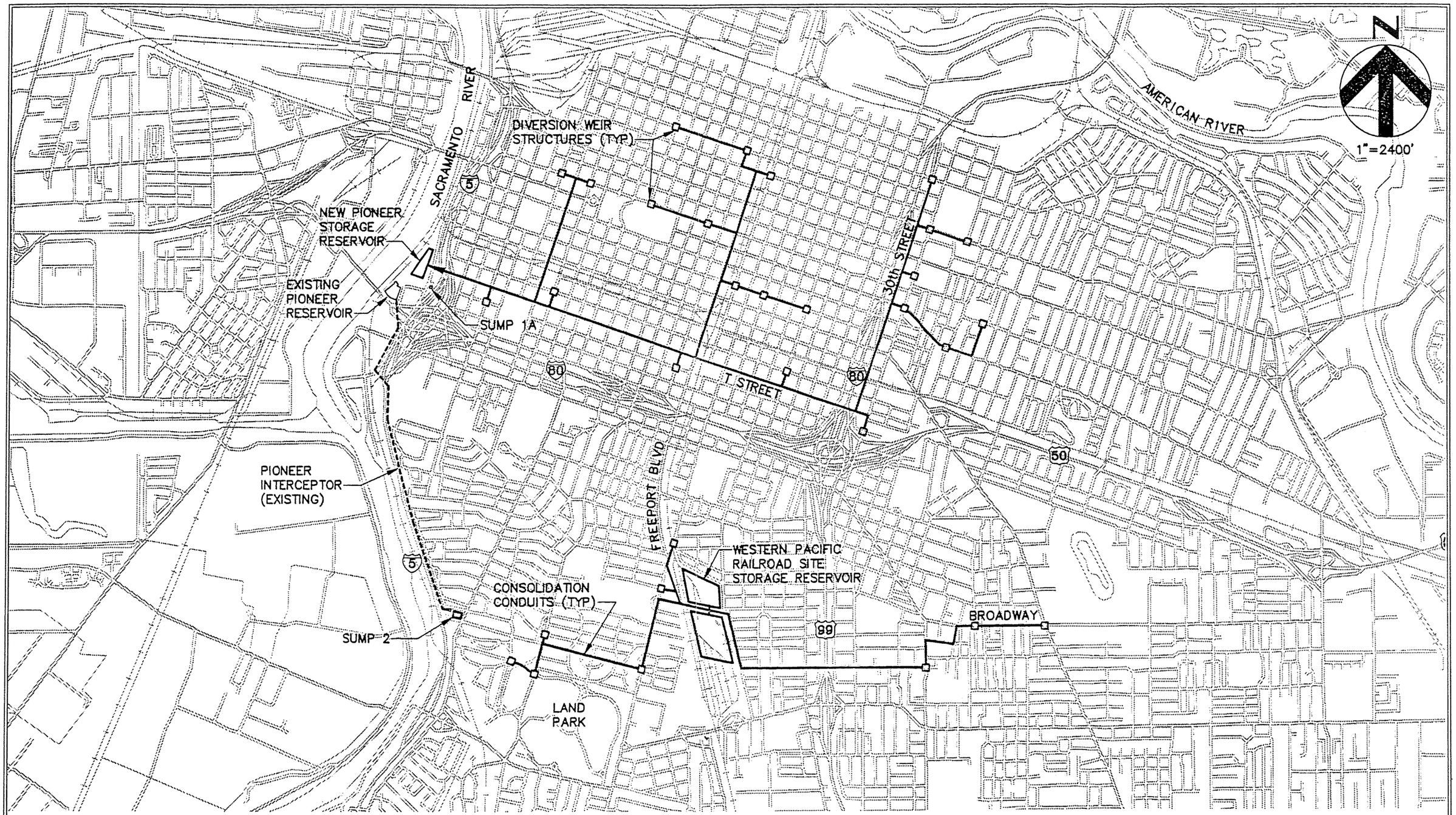
Figure 6-7



WESTERN PACIFIC/PIONEER STORAGE ALTERNATIVE WITH CONSOLIDATION CONDUIT NETWORK FOR 10-YEAR EVENT

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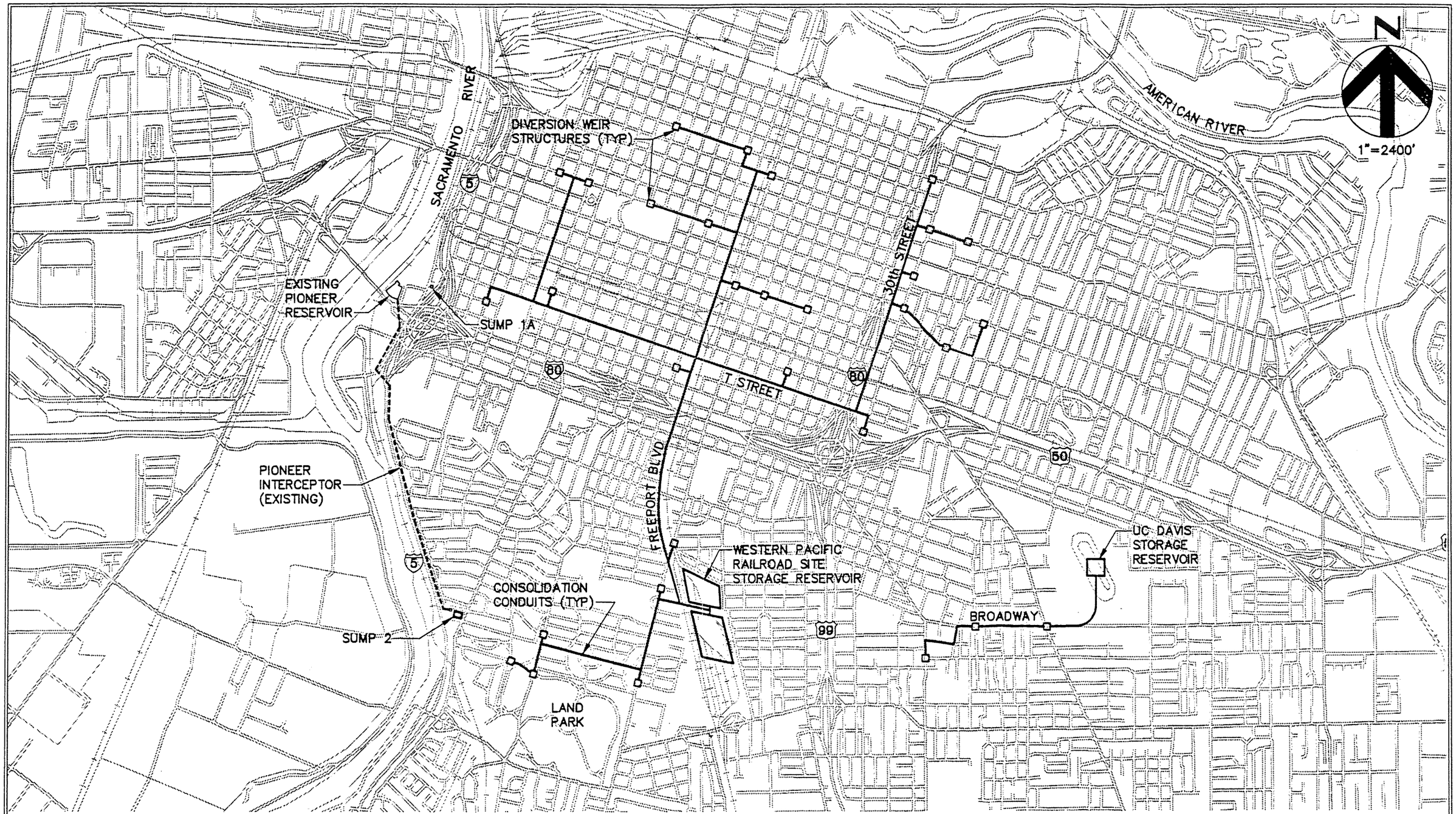
Figure 6-8



WESTERN PACIFIC/UC DAVIS STORAGE ALTERNATIVE WITH CONSOLIDATION CONDUIT NETWORK FOR 10-YEAR EVENT

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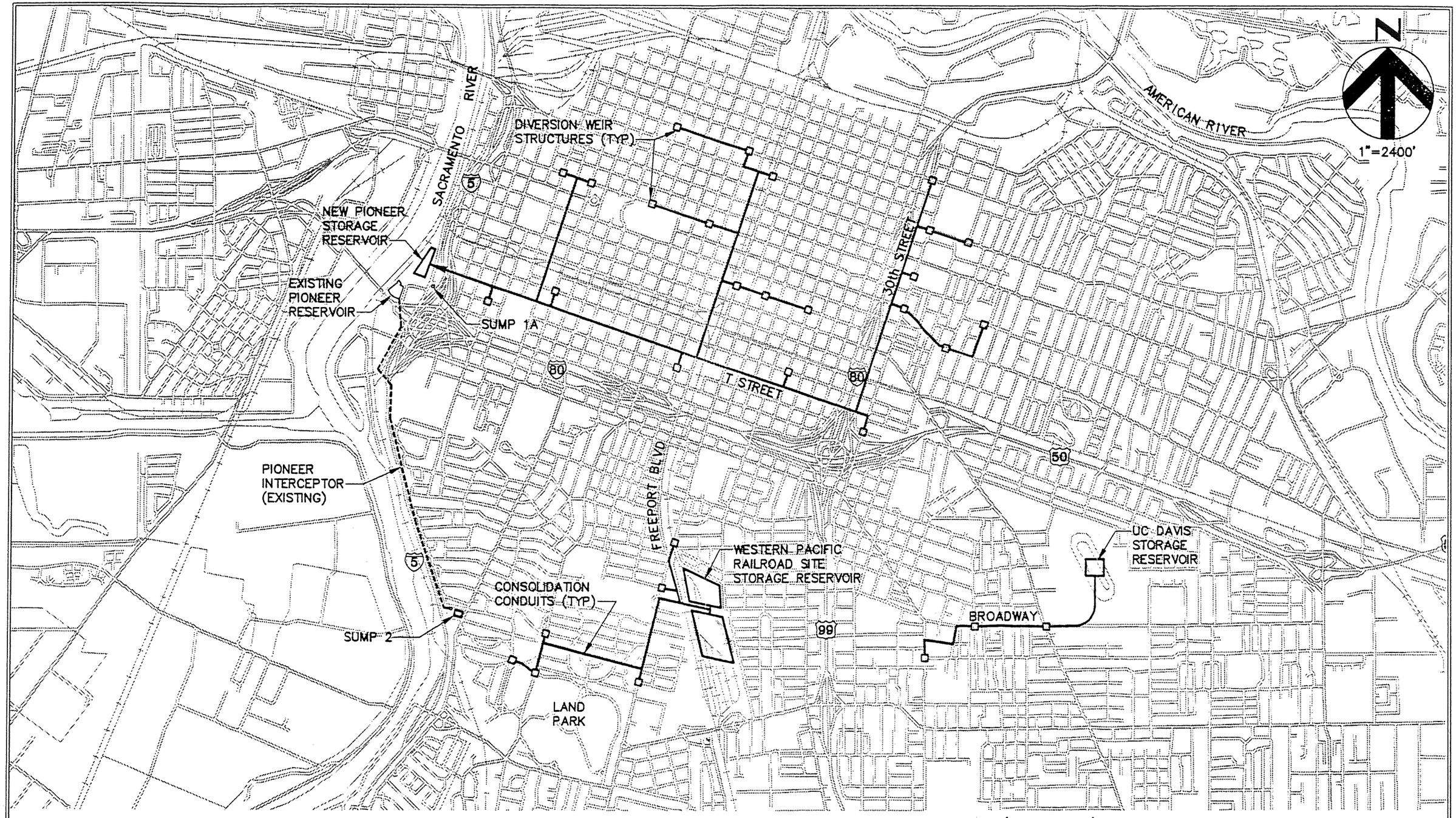
Figure 6-9



WESTERN PACIFIC/UC DAVIS/PIONEER STORAGE ALTERNATIVE WITH CONSOLIDATION CONDUIT NETWORK FOR 10-YEAR EVENT

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Figure 6-10

The WPRR site is a contaminated site presently in the preliminary phases of remediation. The Pioneer Reservoir site is also a contaminated site now in the final stages of remediation with near-term plans for development as a part of a larger riverfront enhancement plan. Should any of these sites be selected for further evaluation, contact with the owners and a detailed underground investigation must be made.

D. SIZING DESIGN CRITERIA

D.1. Storage Reservoirs

New storage reservoirs were sized initially on the basis of the volume required to control flooding for the 6-hour, 2-, 5- and 10-year events. The output data from the SSWMM91 includes a hydrograph (flow rate versus time) for the inflow of wastewater into a storage site. This hydrograph was integrated for each of the design storms to obtain the total volume of inflow to the storage site for the entire rainfall event.

This inflow was then checked against the volume required for the CSO control storm events. Where the total inflow volume to the storage site for the flood control design event exceeded the inflow volume for the CSO control design event, the difference was discharged to the river for the Pioneer Reservoir and WPRR sites. In this case, the storage reservoir was sized for the CSO event inflow volume. For the UC Davis Medical Center site, the larger of the two inflow volumes (i.e. the volume from the 2-, 5- or 10-year design storm, or the volume from the CSO event) was used to size the capacity of the storage reservoir.

D.2. Pumping Stations

New pumping stations required at the storage sites to lift wastewater into the storage reservoirs were sized with firm capacity equal to the peak flow rate of the total inflow to the storage site. The peak flow rate of the floodwater control design storm (i.e. the 2-, 5- or 10-year event) was compared to the peak flow rate for the CSO storm under consideration. The higher peak flow rate was then used to size the pumping station.

Force main outfalls to the river from the Pioneer and Western Pacific sites were sized based on a peak flow velocity of 8 feet per second.

D.3. Consolidation Conduit Networks for the Storage Facilities

The weir locations and consolidation conduit networks used for the storage alternatives were the same as the weirs and consolidation conduits used for the tunnel alternatives. Where the consolidation conduits terminated with dropshafts for the tunnel alternatives, the dropshafts were replaced with junctions. The tunnels were replaced with consolidation conduits with alignments altered so as to terminate at the storage sites.

Figures 6-6 through 6-10 are maps of the consolidation conduit networks for the five different storage alternatives configured for the 10-year event. The configuration of the consolidation networks for the 5-year design storm are very similar and consequently separate figures mapping these alternatives are not provided in this memorandum. The consolidation conduits for the 2-year design storm are illustrated in Figure 6-11.

E. ANALYTICAL PROCEDURES

E.1. 10-Year Alternatives

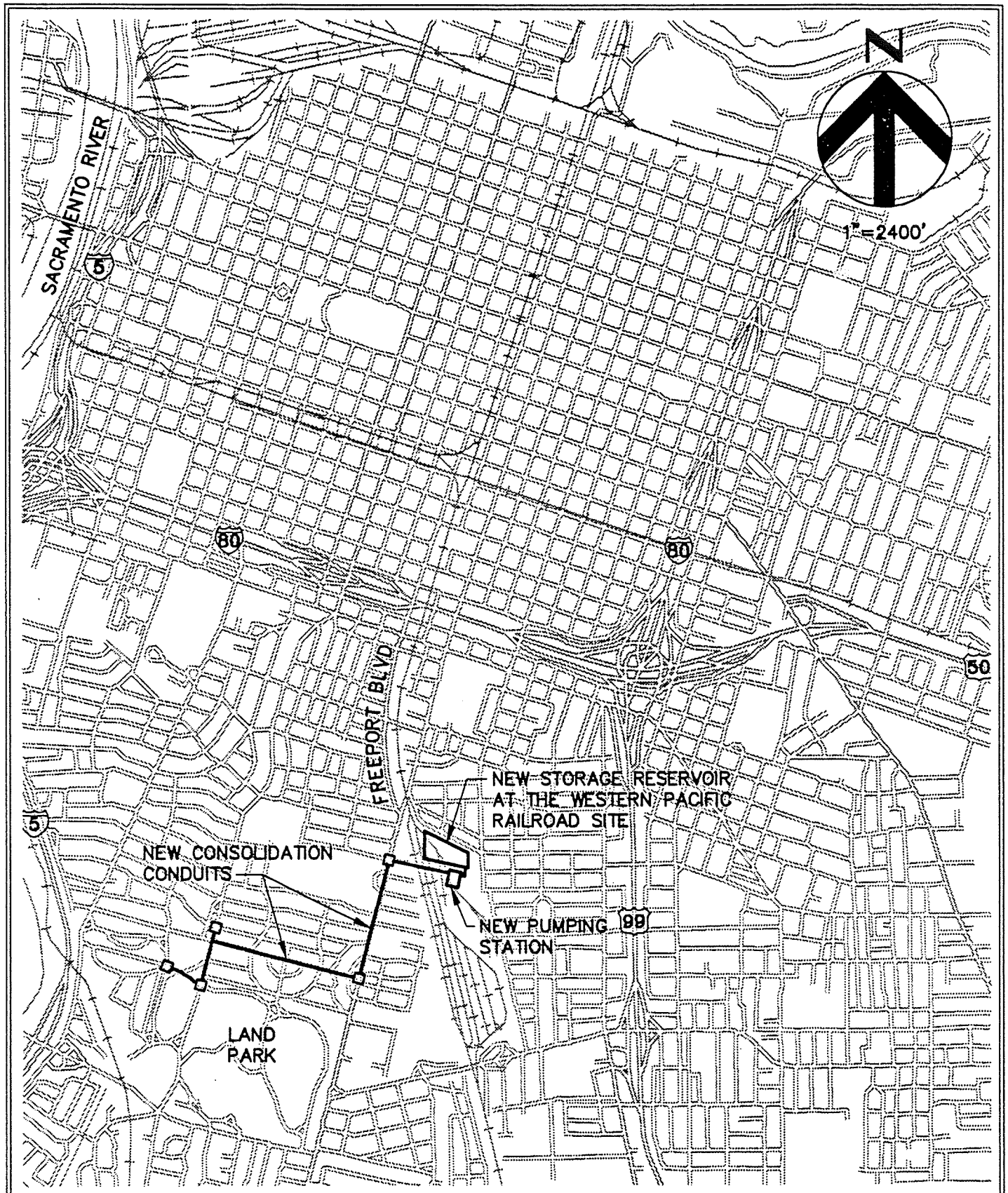
Modeling efforts for the storage alternatives began with the 10-year alternatives. The 10-year tunnel alternatives were modified to arrive at the consolidation conduit network configurations shown in Figures 6-6 through 6-10. The new consolidation conduits were modeled according to the assumptions outlined in Section 3, including roughness coefficients, crown-to-crown matching elevations, and diameters. Diameters were selected for peak flows roughly equal to 80 percent of full pipe flow rate capacity. Weir heights were set for the 10-year event at 33 percent of the existing sewer diameter.

Routing for the new, large-diameter consolidation conduits was confined as much as possible to the tunnel alignments. This worked out satisfactorily for the alternatives using the Pioneer Reservoir and WPRR sites, since the tunnel

Fig. 6-6

UC Davis/Pioneer Storage

6-15, 16, 17, 18 & 19



2-YEAR WESTERN PACIFIC RAILROAD STORAGE ALTERNATIVE USING DIVERSION WEIRS AT LAND PARK

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Figure 6-11

alignments intercepted these sites. For the UC Davis Medical Center site, consolidation conduits were routed through Curtis Park and across Highway 99 to the UC Davis Medical Center site using dedicated right-of-way as much as possible. However, in the southeast portion of the study area, public right-of-way does not consist of long straight corridors. Rather, the streets are offset at intersections making construction of large diameter sewers or tunnels difficult. In some locations, additional right-of-way would have to be purchased to accommodate the proposed sewers.

By starting with the tunnel input data files, the network of weirs and upsized pipes upstream of the weirs was already complete. By providing adequate conveyance downstream of the weirs in the consolidation conduits, control of flooding was achieved.

Problems experienced with the existing EXTRAN code prohibited the modeling of pumping stations and storage nodes at the storage site. Incorporating these facilities into the model created surcharged conditions in the consolidation network which, when reaching (backing up to) a diversion weir, created a numerical error at that particular weir. In order to overcome this modeling limitation the terminal points for the consolidation conduits at the storage reservoir sites were modeled as free outfalls. The storage reservoir volume required for each storage site was determined per the criteria discussed above. According to design and analysis assumptions in Section 3, any water entering the storage reservoirs could not be discharged to the river without treatment. These facilities would be drained back to the existing system, and pumped through Sump 2 for treatment at SRWTP.

Quality control checks were conducted on the output data generated by the SWMM for compliance with the design and analysis assumptions. The same procedures used for quality control on the tunnel model runs were also used in checking the output data for the storage alternatives.

E.2. 5-Year Alternatives

The development of consolidation conduit and weir networks for the 5-year alternatives began with the 10-year layouts. Through an iterative process of

model runs and output review, weirs were deleted, weir heights were raised, and conduit diameters reduced until the criteria described in Section 3 were achieved. Other analytical procedures were identical to that described for the 10-year alternatives.

E.3. 2-Year Alternatives

Once the 10- and 5-year storage alternatives were modeled for control of floodwater, the 2-year storage alternatives were considered. As discussed in Section 5, the procedure used to model the 2-year alternatives was to use the 5-year model configuration for the system upstream of the diversion weirs, and eliminate as many weirs as possible while still controlling the occurrence of flooding. Using this procedure, all but five of the weirs could be eliminated. The remaining weirs would be located in the Land Park area, a low-lying area with persistent flooding problems, as shown in Figure 6-11.

For flood control purposes, it made little sense to consider the UC Davis Medical Center site or the Pioneer Reservoir site as flood control alternatives for this 2-year configuration, since the all the remaining diversion weirs are located in the Land Park area. To use the UC Davis Medical Center site, the wastewater diverted from the Land Park weirs would have to be routed over 2 miles to the east. To use the Pioneer Reservoir site, the wastewater diverted from the Land Park weirs would have to be conveyed from Land Park to the Pioneer Reservoir site, most probably via a pumping station at the WPRR site and a force main routed up Freeport Boulevard and west along T Street. By contrast, the WPRR site afforded a storage site relatively close to the diversion weirs in Land Park. Consequently, for the 2-year flood control alternative using diversion weirs at Land Park, the WPRR site was the only storage site considered.

For CSO control, storage or treatment of the Pioneer Reservoir overflow had to be considered in addition to storing (or treating as discussed in Section 7) the flow from the Land Park diversion weirs. Providing facilities to handle the Pioneer Reservoir overflow at that site and the Land Park flows at the Western Pacific RR site, was identified as a CSO control alternative for this improvement plan. Another configuration was to divert a flow volume from the existing 114-inch Donner trunk sewer to the WPRR site for storage. This diverted volume would

be equivalent to the Pioneer Reservoir overflow volume for the event. By diverting this amount from the Donner Trunk, any spilling from the reservoir would be prevented. However, in practice, the operation of such a facility would be problematic, since one would have to predict the amount to divert at the WPRR site in order to avoid a spill to the river from Pioneer Reservoir.

As an option to using diversion weirs in the Land Park area, an input data file was compiled for a 2-year configuration in which all the weirs were eliminated and flooding was controlled only by upsizing the existing combined wastewater collection system. This alternative is referred to as the 2-year "Upsized CWCS" alternative and is similar to the Phase 2 10-year "Conveyance Flood Control Alternative". With this configuration, all combined wastewater would flow to either Sump 1A or Sump 2. The inflow to Sumps 1A and 2 would be either be treated at SRWTP and CWTP or diverted to the Pioneer Reservoir.

Flood control for the Upsized CWCS configuration requires that the Pioneer Reservoir overflow be discharged to directly the river. The 2-year Upsized CWCS alternative, therefore, requires no additional flood control storage or pumping facilities.

CSO control for the Upsized CWCS configuration requires the storage or treatment of the Pioneer Reservoir overflow. The storage sites retained for consideration CSO control for these Upsized CWCS alternatives were the Pioneer Reservoir site and the WPRR site. As in the case of the 2-year configuration with the diversion weirs, treatment/storage of the Pioneer Reservoir overflow at the WPRR site was assumed to be viable by diverting flow equivalent to the Pioneer Reservoir overflow from the Donner Trunk to the Western Pacific RR site.

As in the previous case, the UC Davis Medical Center site did not appear to be a practical CSO control alternative for the Upsized CWCS configuration, because all the water was conveyed in an upsized collection system to the west. Conveyance of wastewater from the western edge of the combined sewage service area to a storage site at the eastern side of the service area would not be cost effective. Given this reasoning, the UC Davis Medical Center site was eliminated from further consideration for this configuration.

The four 2-year CSO control storage alternatives considered, as schematically illustrated in Figure 6-12, can be summarized as follows:

Configuration with diversion weirs in the Land Park Area:

- Storage at the WPRR and the Pioneer Reservoir sites, referred to as the 2-year Western Pacific/Pioneer alternative (2-year WPP).
- Storage only at the WPRR site, with flow equivalent to Pioneer Reservoir overflow diverted from the Donner Trunk, referred to as the 2-year Western Pacific alternative (2-year WPRR).

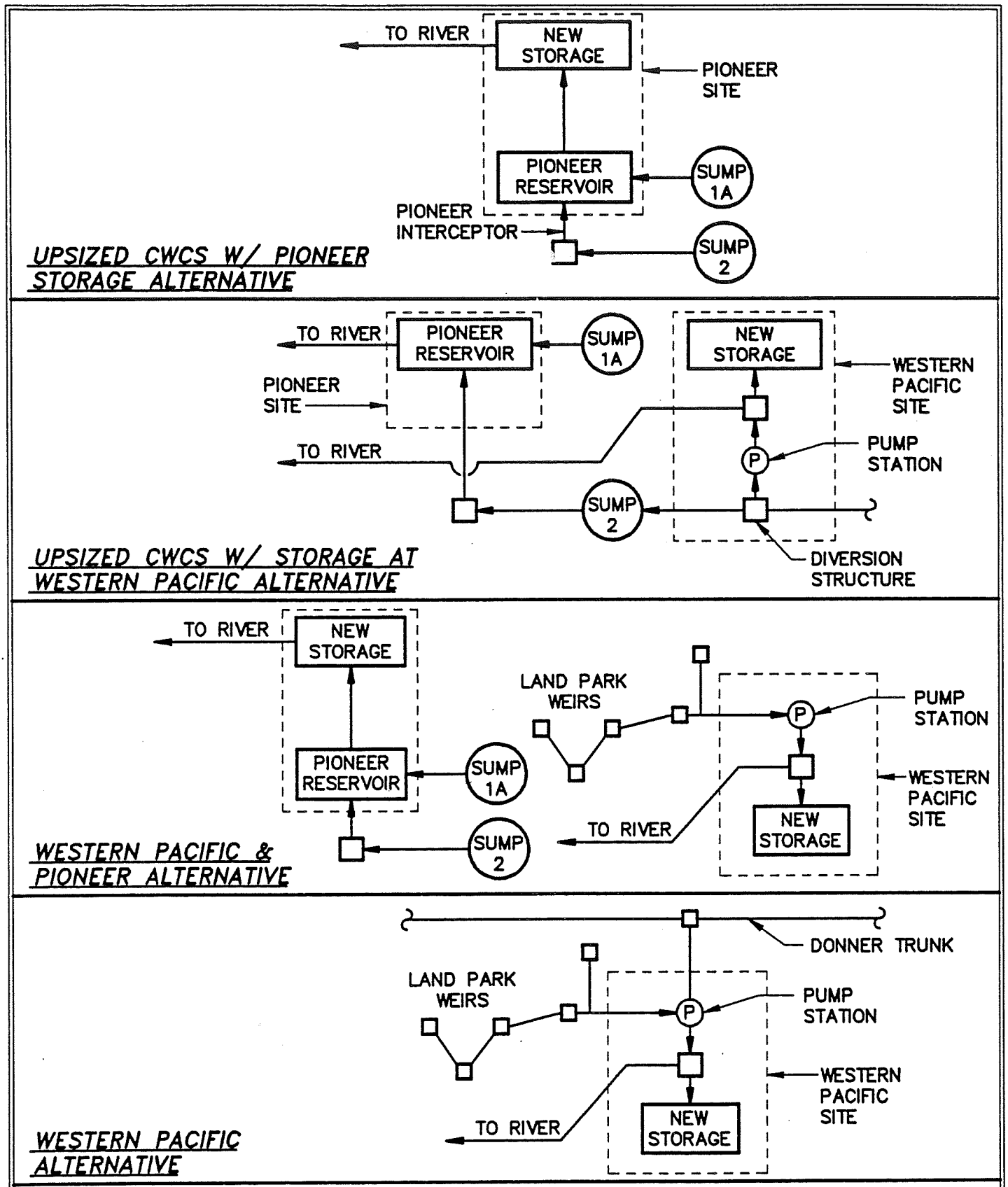
The Upsized CWCS configuration:

- Storage only at the Pioneer Reservoir site.
- Storage at the WPRR site, with flow equivalent to Pioneer Reservoir overflow diverted from the Donner Trunk.

F. EVALUATION OF STORAGE ALTERNATIVE MODEL OUTPUT DATA

The storage alternatives sized for flood control were run with the SSWMM91 using the 1/year and 5/year CSO design storms. For the 5-year and 10-year alternatives with no consolidation conduit outflow at the Pioneer Reservoir site, the overflow from the reservoir was diverted into the consolidation conduit network on T Street to be conveyed to the WPRR site. The conveyance capacity of the T Street and downstream consolidation conduits were upsized for this purpose and was modeled as such.

The CSO configurations used for the 2-year alternatives were as described above and illustrated in Figure 6-12.



SCHEMATICS OF THE 2-YEAR STORAGE ALTERNATIVES
1 PER YEAR CSO CONTROL

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Figure 6-12

F.1. Flood Control

The output data files from these runs were examined for the following information.

1. The total inflow volume to the storage sites and peak flow rates for the CSO storm events were compared to the total inflow volume and flow rates for the 2-, 5- and 10-year design storms. Final recommendations for storage reservoir and pump station sizing were prepared based on the criteria outlined earlier and on the assumption that the discharges to the river from the Western Pacific and Pioneer sites were acceptable for all but the CSO control inflow volumes to those sites.

2. For the 2-year conveyance storage alternatives, the model runs were used to define the storage requirements. Pumping capacity requirements were noted and used in the development of final project cost estimates.

F.2. CSO Control

Just as for the tunnel alternatives, for each flood control storage alternative, the heights of the diversion weirs for the flood control design storms were set to maximize the use of the existing combined sewer system, while diverting enough flow to mitigate flooding. Consequently, the weirs were set lower for the 10-year design storm and higher or removed for the more frequent storms. In order to evaluate the 5- and 10-year storage alternatives with the more frequent CSO storms, the weirs settings were adjusted. It was anticipated that, since the intensities of the 5/year and 1/year CSO storms are less than that of the 2-year flood control storm, the 2-year weir configuration (the 5 weirs in the Land Park area) would be sufficient to maximize the use of the existing CWCS and to prevent or minimize street flooding. The 2-year weir configuration was consequently used for all CSO control analysis.

Application of the 5/year CSO design storm to the 2-, 5- and 10-year storage alternatives configured with the 2-year weirs did not result in either the occurrence of floodwater or a spill to the river. As with the tunnel alternatives, the 5/year CSO storm was completely retained in Pioneer Reservoir

with the CWTP operational and full use made of the available 60-mgd capacity to SRWTP.

Therefore, for CSO control levels limited to as many as five spills per year, no storage is required for the WPRR site nor is any additional storage required for the Pioneer site. All inflow to the WPRR site and the Pioneer Reservoir site can be discharged to the river as long as Pioneer Reservoir is nearly full. For the UC Davis Medical Center site, the required storage is the larger of the CSO or flood control design storms.

Application of the 1/year CSO design storm provided some interesting results. It was expected that the 2-year weir settings would have controlled flooding during storms with smaller recurrence intervals (i.e the CSO control design storms) when applied to the 2-, 5- or 10-year storage alternatives. However, application of the 1/year CSO control design storm to the 2-, 5- and 10-year storage configurations with 2-year weirs required upsizing some of the storage reservoir volumes to maintain flood control.

For the 10-year alternatives, the 1/year CSO control storm storage volumes were smaller than that of the flood control design storms. However, for all the 5-year event alternatives and the 2-year WPRR alternative using the Land Park weirs, the 1/year CSO design storm resulted in larger wastewater storage requirements than the flood control design storms. Furthermore, the 2-year weir setting did not divert sufficient flow from the CWCS to eliminate all street flooding for the 2, 5- and 10-year storage alternatives. It should be noted that the resultant street flooding compared to that predicted by the existing CWCS model was significantly reduced.

These required increases in storage volumes for the 5- and 2-year storage alternatives subject to the 1/year CSO design storm is due to the differences in the synthetic design storms for flood and CSO control. These differences are discussed in Appendix H of the Phase 3 report entitled *Design Storms, Rainfall Analysis and Flood Volume Maps for the Phase 3 Analysis*, dated July 1, 1992.

G. SUMMARY

A summary of the storage reservoir volumes and pumping requirements for the storage alternatives based on flood and CSO control is presented in Table 6-3.

TABLE 6-3. SUMMARY OF MAJOR STORAGE FACILITIES FOR FLOOD AND CSO CONTROL				
Storage Alternative	Storage Reservoirs (MG)	New Pumping Stations (mgd)	Storage Reservoirs (MG)	New Pumping Stations (mgd)
	1/Year CSO Frequency		5/Year CSO Frequency	
UCDP, 10-year	63 @ UCD 50 @ Pioneer	590 @ UCD 890 @ Pioneer	63 @ UCD	590 @ UCD 890 @ Pioneer
WPRR, 10-year	60 @ WPRR	1550 @ WPRR		1550 @ WPRR
WPP, 10-year	10 @ WPRR 50 @ Pioneer	640 @ WPRR 890 @ Pioneer		640 @ WPRR 890 @ Pioneer
WPUCD, 10-year	60 @ WPRR 32 @ UCD	1210 @ WPRR 370 @ UCD	32 @ UCD	1210 @ WPRR 370 @ UCD
WPUCDP, 10-year	10 @ WPRR 32 @ UCD 50 @ Pioneer	310 @ WPRR 370 @ UCD 890 @ Pioneer	32 @ UCD	310 @ WPRR 370 @ UCD 890 @ Pioneer
UCDP, 5-year	24 @ UCD 51 @ Pioneer	310 @ UCD 440 @ Pioneer	24 @ UCD	310 @ UCD 440 @ Pioneer
WPRR, 5-year	60 @ WPRR	760 @ WPRR		760 @ WPRR
WPP, 5-year	10 @ WPRR 51 @ Pioneer	320 @ WPRR 440 @ Pioneer		320 @ WPRR 440 @ Pioneer
WPUCD, 5-year	42 @ WPRR 11 @ UCD	590 @ WPRR 190 @ UCD	11 @ UCD	590 @ WPRR 190 @ UCD
WPUCDP, 5-year	10 @ WPRR 11 @ UCD 51 @ Pioneer	170 @ WPRR 190 @ UCD 440 @ Pioneer	11 @ UCD	170 @ WPRR 190 @ UCD 440 @ Pioneer
WPP, 2-year	1.4 @ WPRR	50 @ WPRR		
WPRR, 2-Year	60 @ WPRR	500 @ WPRR		
Upsized CWCS w/ WPRR, 2-year	58 @ WPRR	500 @ WPRR		
Upsized CWCS w/ Pioneer, 2-year	58 @ Pioneer	0 @ Pioneer		

SECTION 7
TREATMENT ALTERNATIVES

A. INTRODUCTION

As explained in Section 3 there were five types of Combined Wastewater Collection System (CWCS) upgrades considered in the Phase 3 study. This section provides a detailed description of the analyses performed on the treatment alternatives.

Conveyance improvements for controlling flooding in the combined sewer service area will increase flows delivered to the downstream points in the system. These excess flows can be discharged to the Sacramento River during a large storm event. However a portion of this must be captured for immediate primary treatment or stored for treatment at SRWTP following the storm event. This latter portion is the wastewater which would cause CSOs at frequencies established for the study. For this study, the inflow to the CWCS for all events smaller than or equal to the CSO control design storms (either 1/year or 5/year) had to be captured for treatment and or storage.

Treatment facilities evaluated as additions to the existing CWCS were designed to prevent CSOs. Therefore, they provided the CSO control portion of an overall improvement plan. These facilities had to be combined with flood control improvements (i.e. conveyance), to meet a given set of flood and CSO control design storm criteria, such as the 5-year 6-hour and 1/year design storms, respectively.

As discussed in Section 6, it was more cost effective to pump flows reaching the WPRR and Pioneer sites directly to the river than to store them on site. It was concluded that for these two sites, all inflow above the CSO inflow volume should be pumped to the river. The UC Davis site however, proved to be too far from the river for pumping to be a viable option; consequently, storage was the only option considered for that site. The use of storage and/or treatment facilities at the WPRR and Pioneer sites were driven by CSO control requirements rather than flood control requirements. Since the same criteria (i.e. CSO control criteria) applied to both storage and treatment alternatives, it was concluded that storage

and treatment facilities should be evaluated together in order to develop least-cost project combinations. This approach was applied not only to the WPRR and Pioneer Sites, but to the other candidate treatment sites as well.

Treatment processes for combined wastewater were reviewed in the Phase 2 technical memorandum entitled *Alternative Improvement Projects and Plan Combinations*, dated July 1, 1991. That review considered process options such as drum screening, primary clarification, solids reduction using swirl concentrators, solids removal via combination treatment/storage facilities, dissolved air flotation and rapid sand filtration. Of these, the processes recommended for further consideration were primary clarification, similar to the facilities used at the CWTP, and combination treatment/storage facilities.

This section describes the analysis of treatment alternatives configured with either primary clarification or combination treatment/storage facilities recommended in the Phase 2 study.

B. ANALYSIS APPROACH

The treatment alternatives identified in the Phase 3 study Scope of Work were summarized in Table 3-1, Section 3. Those alternatives included three treatment sites: the Pioneer Reservoir site, the Meadowview Wastewater Treatment site and the SRWTP site. A fourth site, the Western Pacific Railroad (WPRR) site, was added to this list per the City's request, during the course of this evaluation.

The approach to evaluating these alternatives is outlined as follows:

1) The first step was to identify the construction costs associated with developing only treatment facilities at each of the four sites. These treatment facility costs were computed as a function of flow rates through the treatment facilities.

2) The results of (1) were used to select the least-cost treatment alternative sites among the alternatives considered. It was assumed that

wastewater was collected at the WPRR site for conveyance to the Meadowview or SRWTP treatment sites.

3) By combining a storage reservoir with the treatment facilities, the impact of using storage reservoirs on overall treatment costs was evaluated. The function of these storage facilities was to provide flow equalization for the influent flows to treatment. Because the cost associated with a treatment facility is directly proportional to the flow rate through the treatment plant, it is possible to reduce the required capacity and cost of a treatment plant using a storage facility to reduce peak influent flow rate.

4) Using this procedure, the least-cost treatment/storage combination was estimated for each storage site alternative identified in Section 6 (except for the UCD Medical Center site).

Storage and treatment combinations were considered only for a particular site where cost reduction benefits could be realized for the treatment facilities. For instance, additional storage at the Pioneer Reservoir site would not provide significant flow equalization, and therefore little or no cost reduction benefit to a treatment plant located at the WPRR site. This approach stems from the results provided in Section 6 regarding pumping versus storage for flood control. It is less costly to pump excess flows (i.e. inflows to a treatment site in excess of those created by CSO control design storms) to the river for discharge than it is to build flood control storage at the Pioneer Reservoir and WPRR sites. Therefore, the main purpose of the storage facilities is CSO control. Because treatment plants are sized to meet this identical objective (i.e. CSO control), then it is reasonable to conclude that the combination of these two types of facilities should be considered and the least cost combination identified.

The construction costs of storage reservoirs used as equalization basins were determined as a function of treatment plant flow rate for each of the treatment sites. These storage costs were then added to the treatment costs to arrive at a total cost for both treatment and storage as a function of treatment flow rate.

These total costs were then examined to determine the least-cost combination for storage and treatment facilities.

It is important to note that the storage reservoirs associated with treatment facilities, as presented in this section, would be operated as equalization basins in conjunction with a treatment plant at a particular site. All combined wastewater entering these facilities would receive treatment at the adjacent primary treatment facility, with the effluent discharged to the river. Unlike the flood-control-only storage, combined wastewater entering these combination treatment/storage facilities is not routed to SRWTP.

C. TREATMENT ALTERNATIVE MODELING AND ANALYSIS ASSUMPTIONS

The modeling assumptions common to all improvement plan alternatives investigated are presented in Section 3. The assumptions and analytical procedures specific to the treatment alternatives are discussed below.

C.1. Conveyance Improvements

The initial configuration of treatment and treatment/storage alternative plans, like all other alternatives, involved developing conveyance improvements for the 2-, 5- and 10-year, 6-hour flood control design storms. Treatment alternatives were not evaluated for the 25-year design storm event. For the 5- and 10-year design storms, a new network of consolidation conduits conveys flow to an alternative treatment site. For the 2-year design storm system, two conveyance upgrade networks were considered: one involving upsizing existing pipes and the other involving the installation of consolidation conduits at key problem flood areas. Treatment and treatment/storage facilities were then added to this improved conveyance network to create complete improvement plans. Consolidation conduit networks are identical to those described and illustrated in Section 6, for the corresponding storage-only locations.

C.2. Flow Rate for Treatment

The treatment flow rate required for CSO control was the criterion used to size

the treatment facilities. Two treatment process configurations were considered in this analysis: combined treatment/storage facilities (described in the Phase 2 report and illustrated in Figures 7-1 through 7-3) and a conventional primary treatment plant. The application and details of the two processes will be discussed later in the section. For combined treatment/storage facilities, the sizing criteria used a 1.5-hour hydraulic retention period based on the peak flow rate into the treatment facility. For conventional primary treatment facilities without storage, the primary clarifiers were sized for a surface overflow rate of 1200 gallons/day/square foot, as they were in the Phase 2 study, based on the peak flow rate delivered to the treatment facility.

As was seen from the tunnel and storage alternative analysis, the 5/year CSO control design storm required no additional upgrade to the existing CWCS for CSO control. Therefore, only the 1/year CSO design storm flows were considered in the sizing of treatment facilities.

C.3. River Discharge

In Section 6, the storage reservoirs for the WPRR and Pioneer sites, were sized for CSO control only. Any other inflow volume to these two sites over and above that delivered by the CSO design storms was diverted to the river without treatment.

For the treatment alternatives reviewed in this section, similar assumptions regarding river discharge were made. Specifically, for the alternatives using either the Pioneer Reservoir or WPRR sites, any additional inflow rates over and above the peak inflow rates delivered by the CSO design storms were diverted to the river without treatment.

C.4. Combined Storage/Treatment Facilities

There is an important distinction to be made between the CSO design storm inflow <u>volume</u> to be stored and the inflow <u>rate</u> to be treated. The storage reservoirs proposed in Section 6 were sized for a total inflow volume. This is compared with proposed treatment facilities in this section sized for an inflow rate. It was

assumed that once treatment operations were started, treatment would continue for the entire storm at the peak flow rate for the facility. It is possible that the total volume treated could exceed the CSO control volume, should the City choose to do so for an actual storm event.

C.5. Pioneer Overflow During the CSO Event

For the 1/year CSO event, the CWCS was assumed to be delivering wastewater to the SRWTP and CWTP and also to Pioneer Reservoir through the Pioneer Interceptor. Pioneer Reservoir also received wastewater from the operation of Sump 1A. For all the treatment alternatives (as well as the storage-only and tunnel alternatives), the Pioneer Reservoir (without improvements) overflowed during the 1/year CSO event.

For all the 5- and 10-year treatment alternatives with treatment at the WPRR site, the excess that would otherwise overflow was assumed to be diverted to the consolidation conduit network in T Street for conveyance to treatment at the WPRR site. For the 5- and 10-year UCDP treatment alternative, the overflow from Pioneer Reservoir was assumed to be directed to the new treatment facility at the Pioneer site.

For the 2-year alternatives, there were no consolidation conduits in T Street, so overflow from Pioneer Reservoir was diverted to treatment at the Pioneer site. For the 2-year alternatives with storage/treatment only at the WPRR site, the model assumed that excess flow was routed back into the existing system for conveyance to the WPRR site. In reality, the WPRR facilities would need to be operated such that enough flow would be diverted from the Donner Trunk to avoid a spill at Pioneer.

D. DESCRIPTION OF TREATMENT ALTERNATIVES

The treatment alternatives reviewed for this section consisted of treatment at the Pioneer Reservoir site and treatment alternatives using the WPRR site. This latter group of alternatives included treatment facilities located at the Western Pacific site, the Meadowview site and the SRWTP site. Each of these alternatives

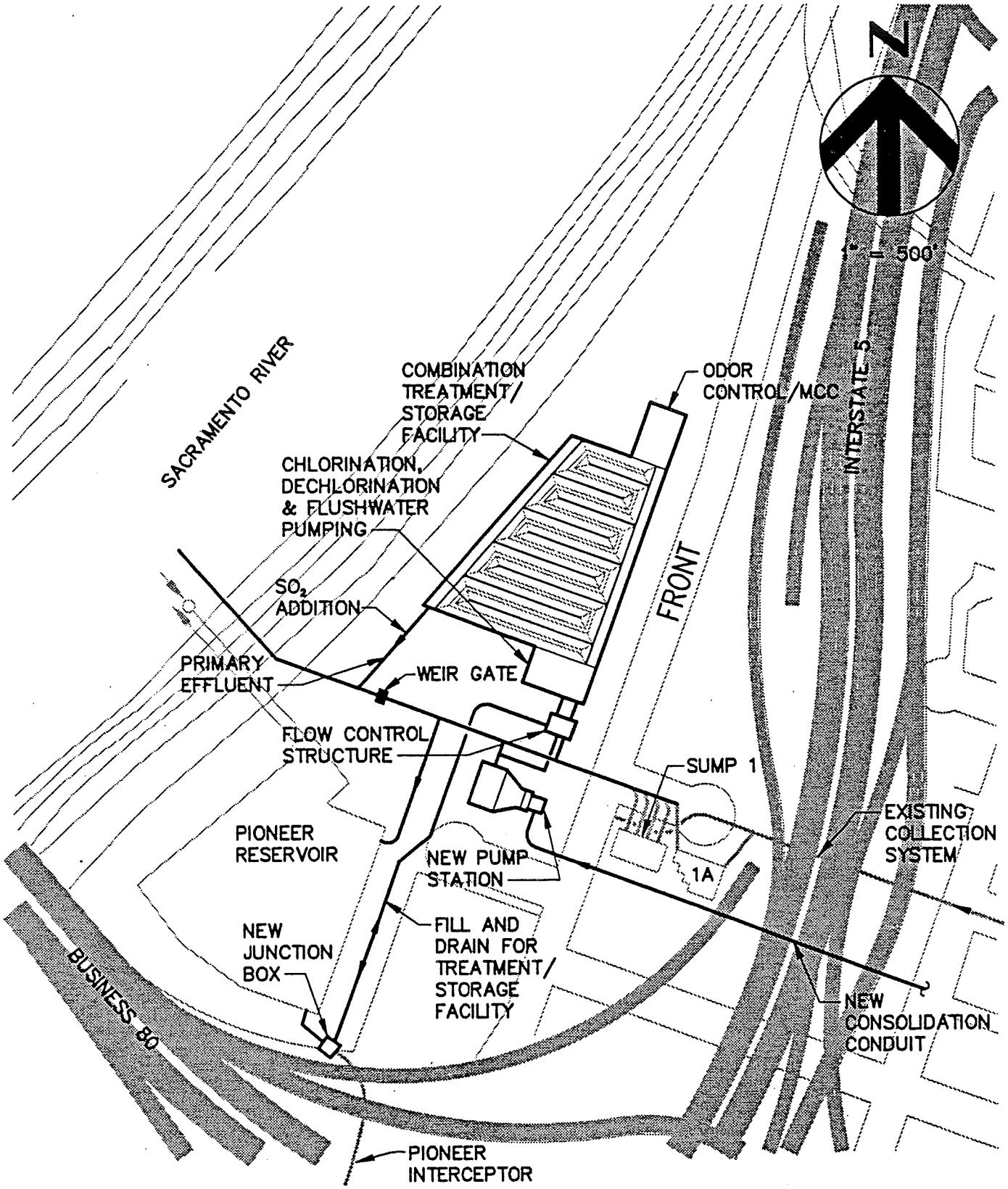
is described below.

D.1. Treatment at the Pioneer Site

The combined wastewater inflow to the Pioneer Reservoir site was delivered by: 1) the Pioneer Interceptor, 2) Sump 1A or 3) by a network of new consolidation conduits serving the diversion weirs needed for flood control in the portion of the study area north of Highway 50. The Pioneer Interceptor would deliver flows in excess of 190 mgd. Sump 1A would deliver the flows from the existing CWCS serving the downtown area.

The treatment facility for the Pioneer Reservoir site was a combination treatment/storage facility. Figure 7-1 illustrates how the site would be developed for the new treatment facility. Inflow from Pioneer Reservoir enters a new junction box at the existing Pioneer Reservoir which can either feed the existing reservoir or the new treatment/storage facility. Inflow to Sump 1A and from the consolidation conduits serving the area north of Highway 50 is discharged into a common header which can feed either the existing reservoir or the proposed treatment/storage facility.

Figures 7-2 and 7-3 illustrate the construction and operational details of the proposed treatment/storage facility at the Pioneer Reservoir site. The construction of the facility would include a flow control structure which would combine inflow from the common discharge header and the Pioneer Interceptor into a single influent conduit for the structure. Wastewater entering the structure would initially pass through a series of flow-through channels arranged in a serpentine configuration. All wastewater treated at the plant would receive chlorination. The residence time within the structure serves as the contact time required for disinfection. Once the structure is filled to capacity, it begins to overflow and discharge to the river. The flow pattern within the structure changes from serpentine channel flow to the pass through flow pattern shown in Figure 7-3. All effluent from the structure intended for discharge to the river would be dechlorinated. Flushing water for the facility would be withdrawn from



PIONEER RESERVOIR SITE DEVELOPED AS A COMBINATION TREATMENT/STORAGE FACILITY

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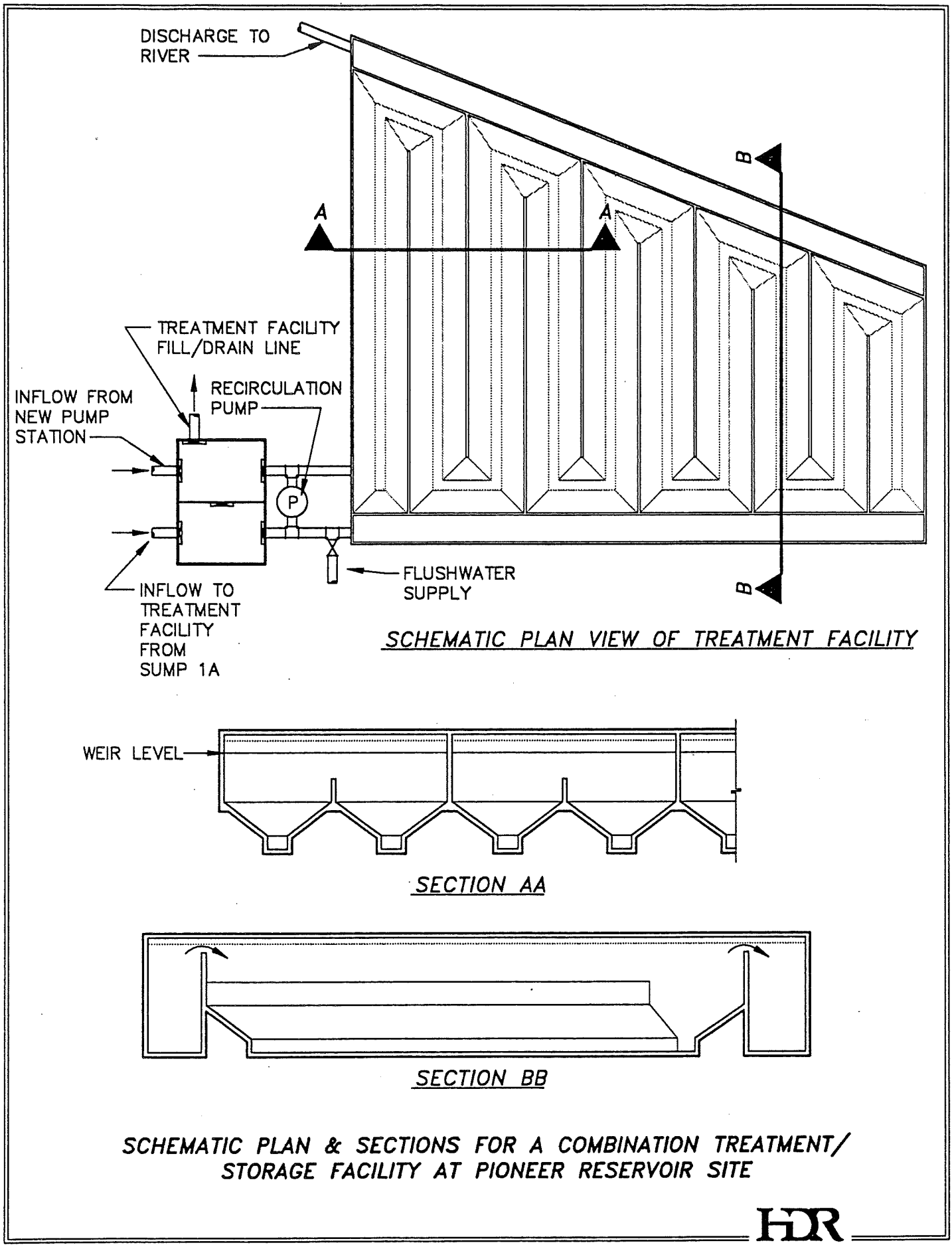
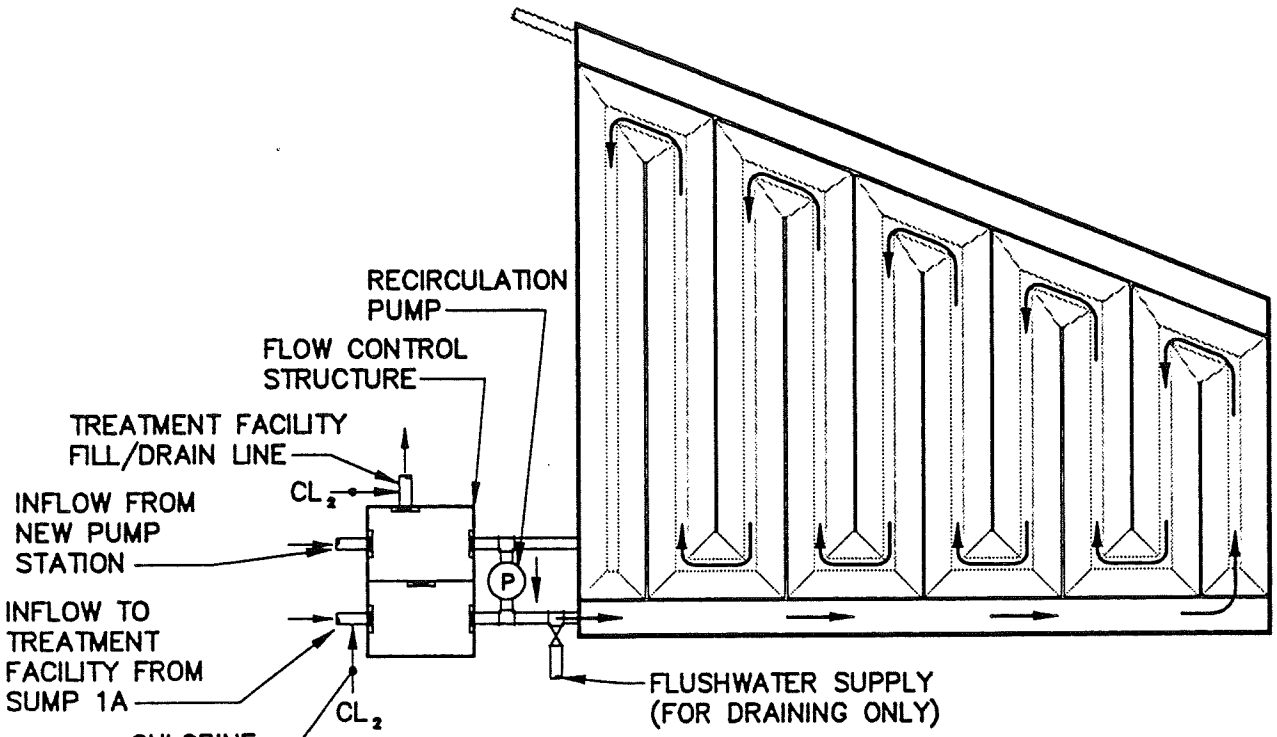
Figure 7–1



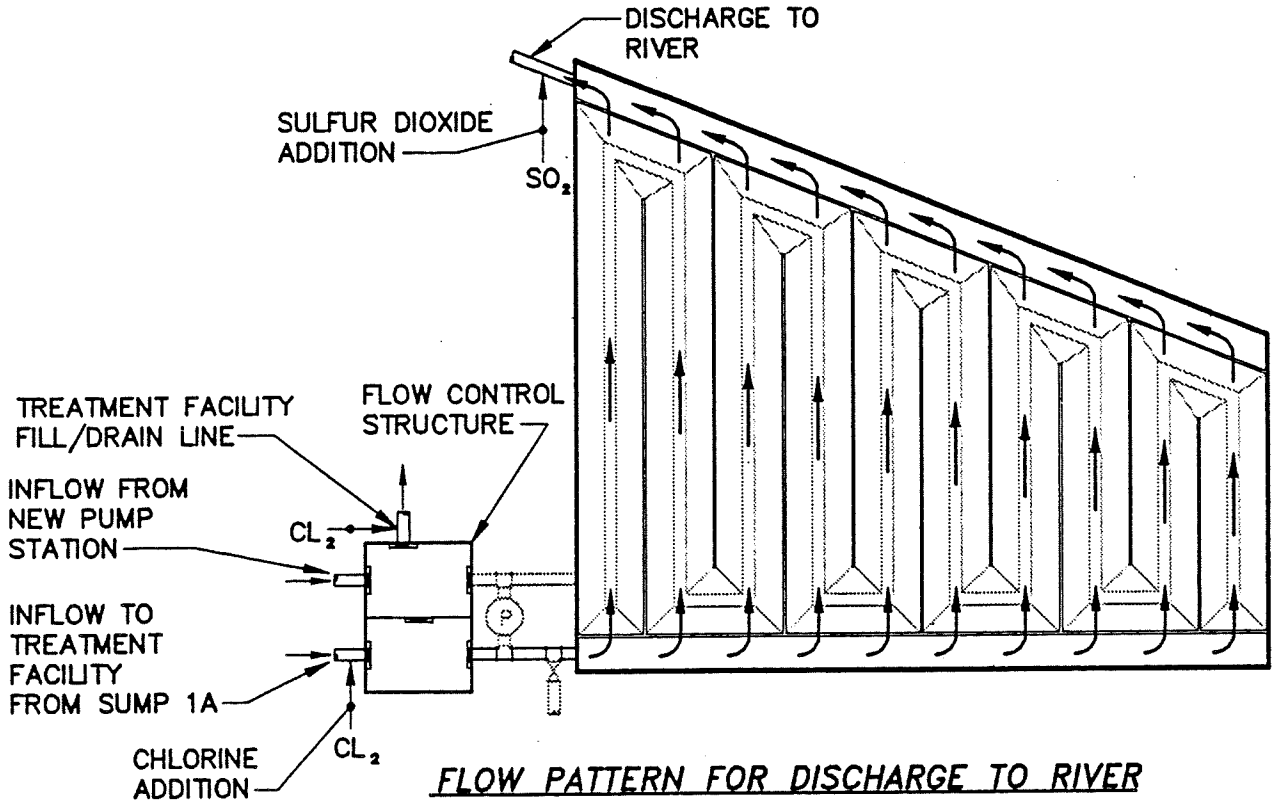
SCHEMATIC PLAN & SECTIONS FOR A COMBINATION TREATMENT/ STORAGE FACILITY AT PIONEER RESERVOIR SITE

HDR

Figure 7-2



FLOW PATTERN FOR FILLING/DRAINING



FLOW PATTERN FOR DISCHARGE TO RIVER

SCHEMATIC OPERATIONAL PLAN FOR A COMBINATION TREATMENT/ STORAGE FACILITY AT PIONEER RESERVOIR SITE

HDR

Figure 7–3

the river and recirculated through the serpentine channels to scour the solids deposited from the wastewater handled during the event. The waste from this process would be routed to the SRWTP.

D.2. Treatment Facilities Using the Western Pacific Site

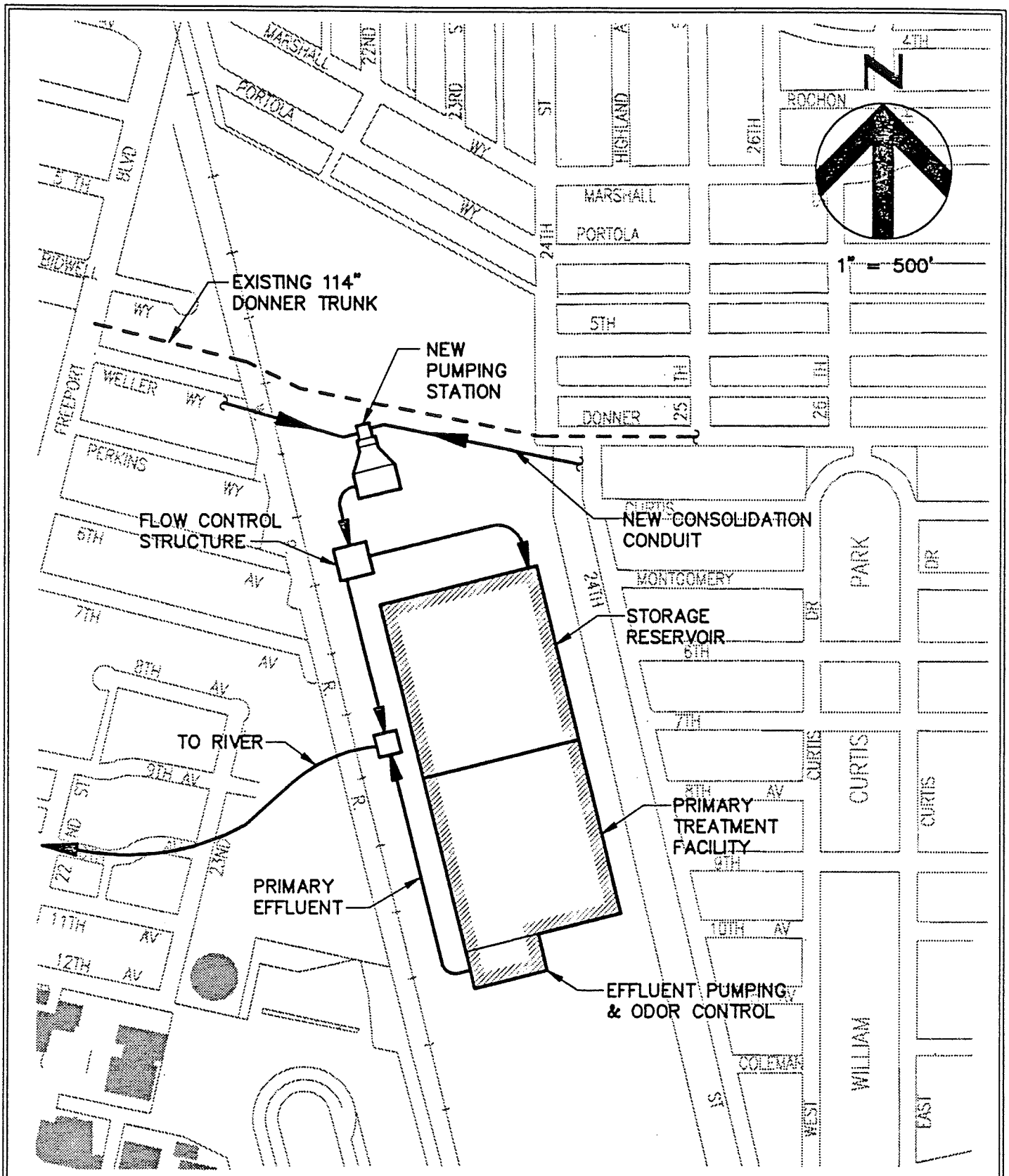
The Western Pacific site was used in the configuration of three different treatment alternatives. These included: 1) treatment at the WPRR site only, 2) treatment at the Meadowview site, and 3) treatment at the SRWTP site. All these use the WPRR site for receiving inflows of combined wastewater and pumping the CSO control portion of the total inflow to a possible storage reservoir for flow equalization.

D.2.a. Treatment at the Western Pacific Site--

Figure 7-4 illustrates the configuration of treatment and storage facilities for the WPRR site. Inflow to the site would be pumped to a flow control structure. The flow control structure would divert all flows exceeding the rate required for CSO control to the river. A storage reservoir would be constructed to receive the CSO storm flows. Gravity flow would transfer wastewater from the storage reservoir to the treatment facility. Treated effluent would be pumped to the river along with the excess (above CSO control requirements) untreated diverted wastewater from the flow control structure. A well field would be developed for supply of flushwater to clean out the storage and treatment facilities after each use.

D.2.b. Meadowview Treatment Alternative--

The Meadowview site is adjacent to the Sacramento River near the intersection of Interstate 5 and Freeport Boulevard. Figure 7-5 illustrates how wastewater collected at the Western Pacific site in accordance with the WPRR alternative could be pumped from that site to the Meadowview site via a new force main. The corridor selected for this force main is the Union Pacific Railroad right-of-way.



WESTERN PACIFIC RAILROAD SITE DEVELOPED WITH PUMPING STATIONS, STORAGE AND TREATMENT FACILITIES

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FIGURE 7–4

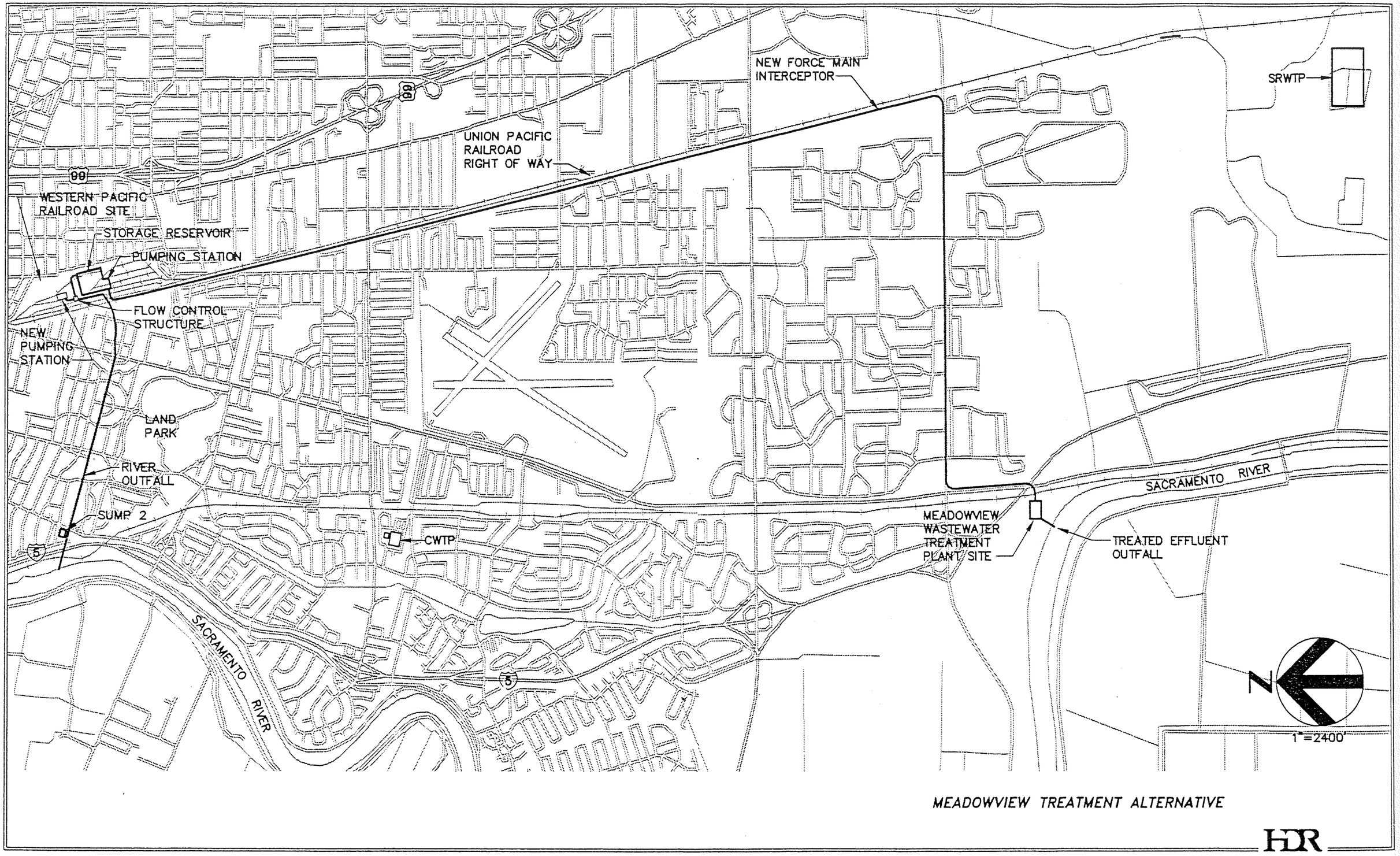

NEW FORCE MAIN INTERCEPTOR
SRWTP
UNION PACIFIC RAILROAD RIGHT OF WAY
WESTERN PACIFIC RAILROAD SITE
STORAGE RESERVOIR
PUMPING STATION
FLOW CONTROL STRUCTURE
NEW PUMPING STATION
LAND PARK
RIVER OUTFALL
SUMP 2
CWTP
SACRAMENTO RIVER
MEADOWVIEW WASTEWATER TREATMENT PLANT SITE
TREATED EFFLUENT OUTFALL
SACRAMENTO RIVER
N
1"=2400'
MEADOWVIEW TREATMENT ALTERNATIVE
HDR

Figure 7-5

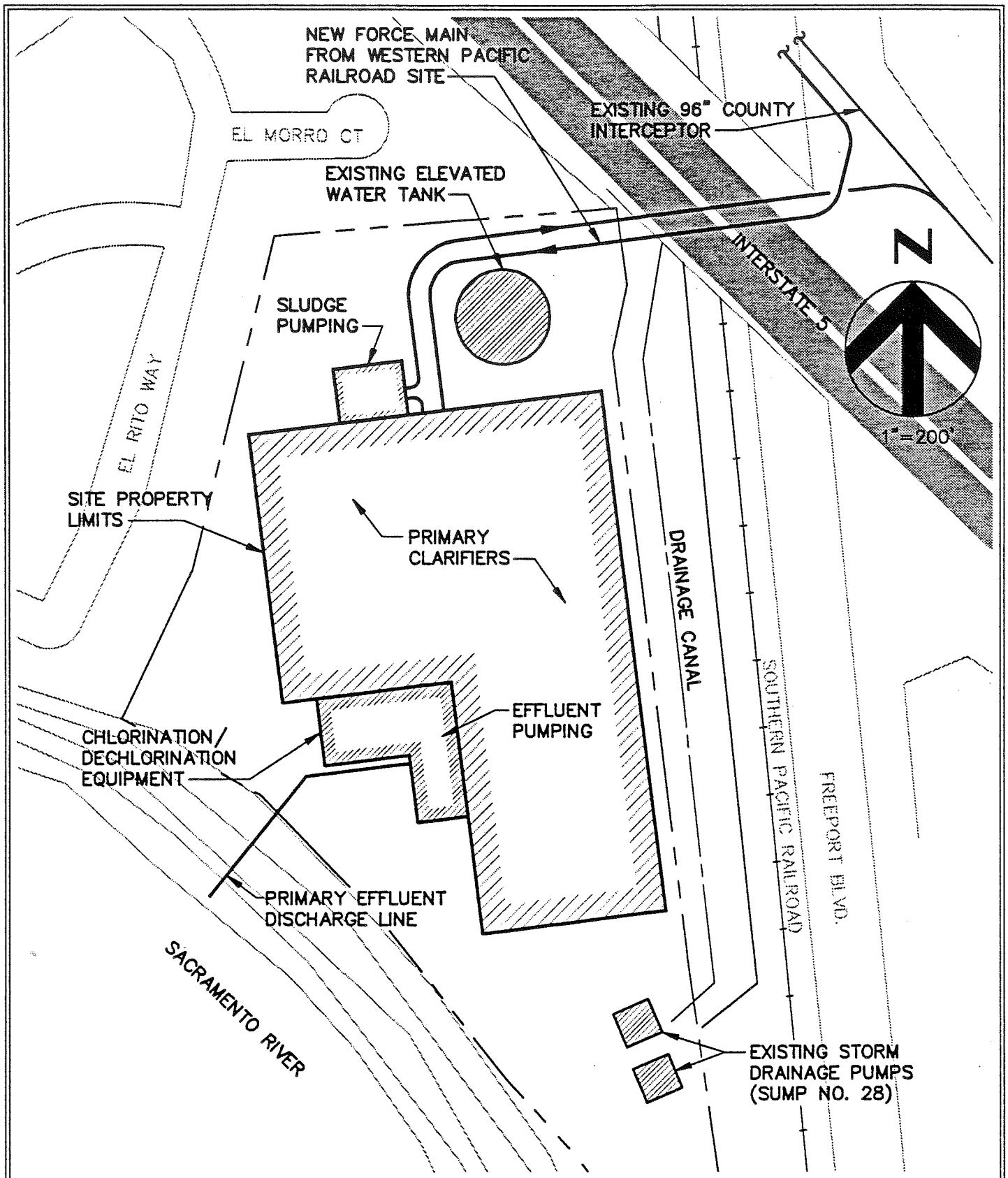
In a telephone conversation with the Real Estate Manager from the Union Pacific Railroad Company in Omaha, it was suggested the railroad right-of-way may be available for use as a utility easement. Figure 7-6 illustrates the Meadowview site developed with primary treatment facilities. Prechlorinated raw combined wastewater would enter the site from the north. Primary clarifiers would be fed from a central pipe gallery and collection structure. Primary sludge would be pumped into the County's 96-inch interceptor for conveyance to SRWTP. An effluent outfall line would terminate in the river and discharge dechlorinated effluent. A flushwater intake line would provide washdown water from the river for the clarifiers following a storm.

D.2.c. SRWTP Treatment Alternative--

The treatment alternative for SRWTP is similar to the Meadowview alternative in that it utilizes the WPRR alternative configuration to collect all inflow of diverted wastewater at the Western Pacific site. All inflow is pumped to a flow control structure with the treated portion conveyed to interim storage. Flow is pumped out of interim storage and through a force main to the northern edge of the SRWTP property just north of the proposed Consumnes River Boulevard alignment and west of the Union Pacific Railroad right-of-way. Treatment would be provided by primary clarifiers with effluent pumped to the river. Primary sludge would be removed and pumped to the SRWTP through a new force main. A well field would have to be developed to supply the site with flushwater to clean out the clarifiers following their use. The proposed improvement plan is illustrated in Figures 7-7 and 7-8.

E. LEAST-COST TREATMENT/STORAGE ALTERNATIVE FOR 1/YEAR CSO CONTROL

The analysis to identify the least-cost treatment alternative was carried out in accordance with the analytical approach described earlier. The objective of the analysis was to identify the least-cost (construction cost) treatment alternative. This would not only reveal the site with the lowest treatment cost but also the least-cost treatment/storage facility configuration for that site.



MEADOWVIEW SITE DEVELOPED AS A PRIMARY TREATMENT FACILITY

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Figure 7-6

SRWTP TREATMENT ALTERNATIVE

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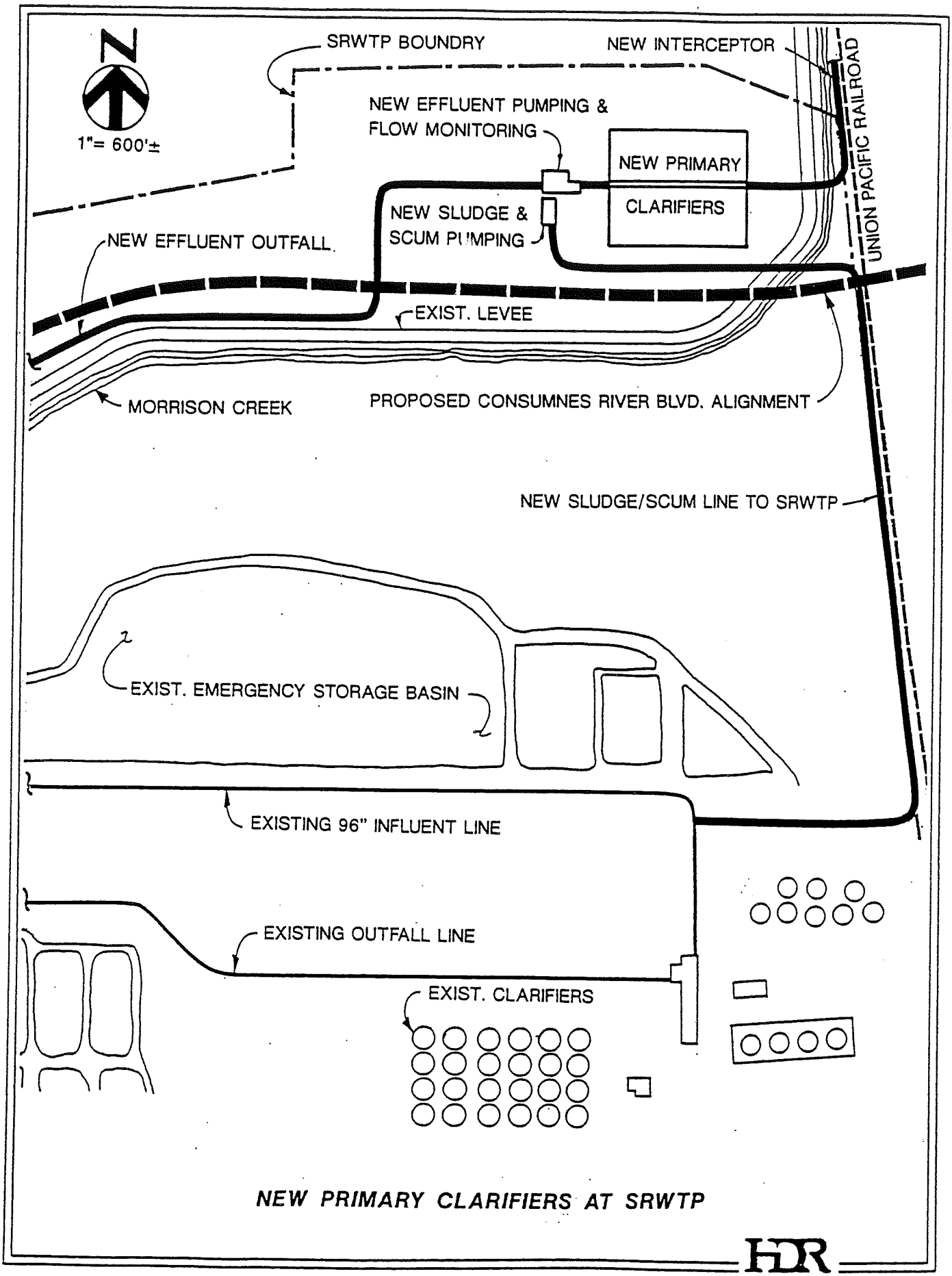
Figure 7–7



NEW PRIMARY CLARIFIERS AT SRWTP

Figure 7-8

E.1. Treatment Costs

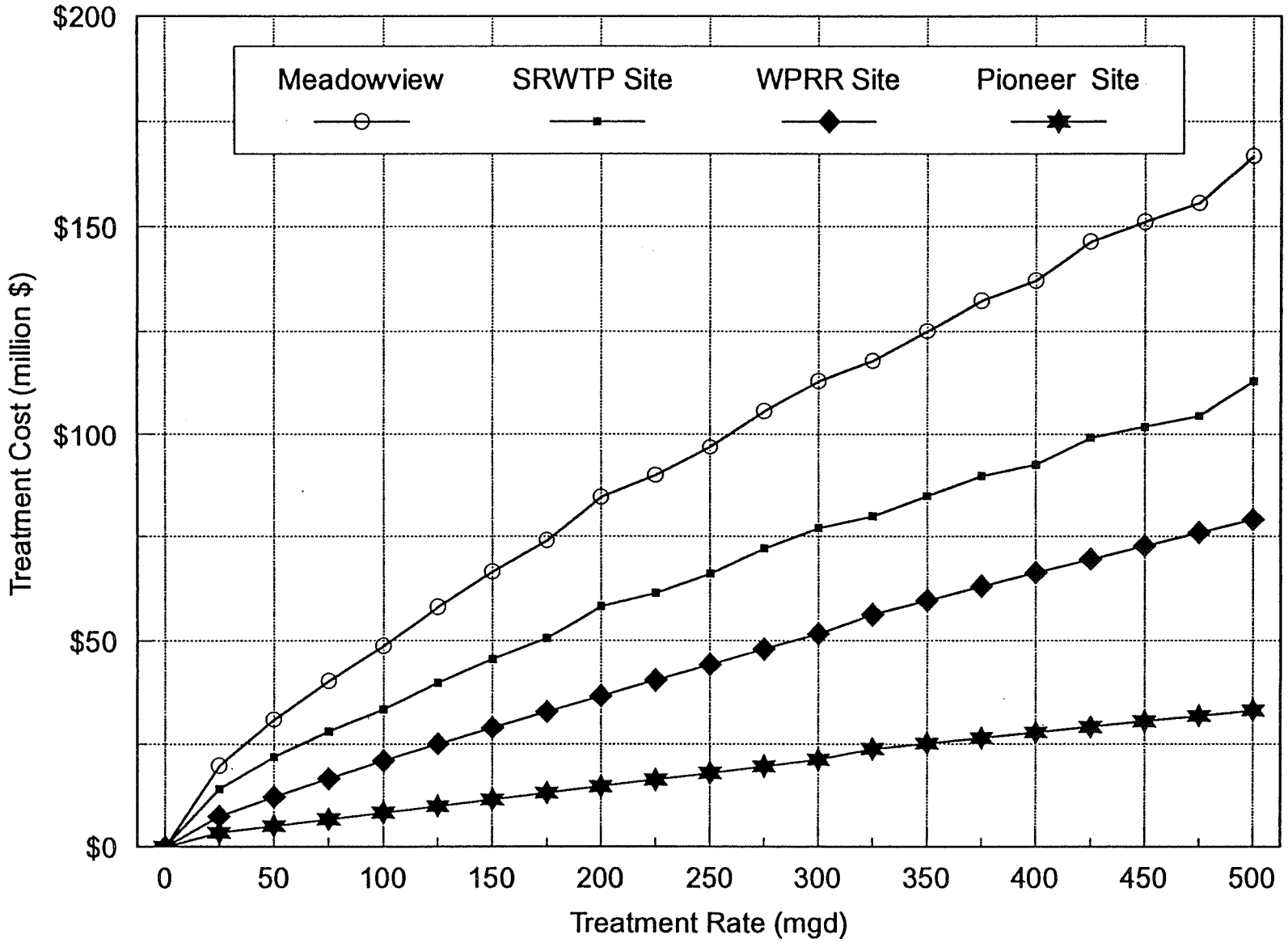
The costs of treatment were determined for each of the four treatment sites as a function of treatment flow rate. The flow rates selected for cost estimating were based on the range of CSO control inflow rates to the individual treatment sites. The cost components comprising the overall treatment costs were all in terms of construction costs. Itemized components included the treatment facilities, effluent pumping, treated effluent outfall lines, sludge pumping and well fields developed for a source of flushwater. For the Western Pacific alternatives, costs for pumping and a force main from the WPRR site to the treatment facilities were also included.

Figure 7-9 compares the treatment costs for the four treatment sites. As expected, the costs of the treatment facilities increase with increasing treatment flow rate. The treatment costs are the lowest for the Pioneer Reservoir site for all flow rates evaluated. For the alternatives using the WPRR site, the lowest treatment cost is for the treatment facility located at the Western Pacific site.

Based on this information, the treatment alternatives for the Meadowview and SRWTP sites were eliminated from further consideration. Treatment alternatives using the Pioneer Reservoir and WPRR sites were retained for further analysis.

E.2. Treatment and Storage Combined Costs

To evaluate the benefit of adding storage upstream of the treatment facilities for equalization of treatment plant influent flows, it was first necessary to identify the CSO inflow rates and inflow volumes to the WPRR and the Pioneer sites. The inflow rates were extracted from the SSWMM91 output files for the different storage alternative models reviewed in Section 6 using the 1/year CSO storm event.



COMPARISON OF THE TREATMENT FACILITY CONSTRUCTION COST FOR THE IDENTIFIED TREATMENT SITES

HDR

Figure 7-9

The storage reservoirs were sized to provide attenuation of the peak influent flow and to discharge to the adjacent treatment facilities at a constant rate for the majority of the design storm event. The total volume of wastewater delivered by the storage facility to the treatment facility would be equal to the CSO design storm inflow volume to the treatment site. To determine the required size of the equalization-storage reservoirs as a function of treatment flow rate, the following relationship was used.

$$V_{req'd} = \text{maximum value of}\{[(Q_3-Q_2)*(t_i-t_{i-1})]+V_{i-1}\}$$

Where $V_{req'd}$ = the required storage volume
Q_3 = combined wastewater inflow rate to the treatment site
Q_2 = treatment plant influent flow rate
t_i = time step i
t_{i-1} = time step i-1
V_{i-1} = volume of storage at time step i-1

The inflow hydrograph for Q_3 was represented by the total inflow to the treatment site resulting from either the flood control design storm or the 1/year CSO design storm, whichever resulted in the largest volume. The value of Q_2 was varied from 0 to the maximum CSO storm inflow rate, as an approximation of the range of values which bracket the peak cumulative inflow rate for the site. The storage volume required for the 1/year CSO design storage was used as the upper limit on the required storage volume. The time steps are identical to those reported in the SSWMM91 output data files for the 1/year CSO event. This procedure was used for each storage configuration presented in Section 6.

Once the storage volumes for equalization were computed as a function of treatment flow rate, the cost of these storage facilities was developed based on the unit costs reported in Section 4. These storage reservoir costs were added to the treatment plant costs to develop total combined treatment and storage cost data as a function of treatment flow rate.

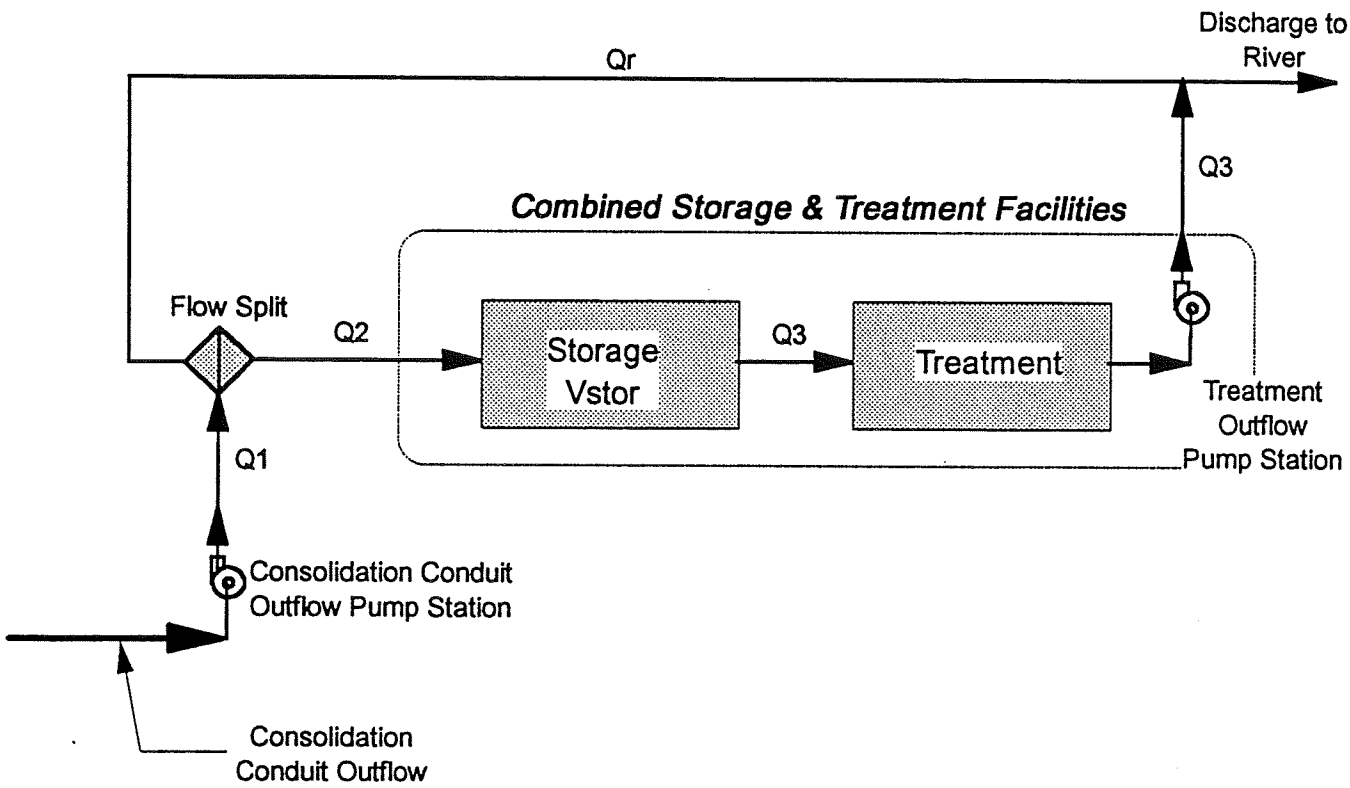
E.3. Evaluation of the Combined Storage and Treatment Cost Data

It had been identified that a combination of storage and treatment, with the latter providing equalization of the peak inflow rates, might be more cost efficient than to provide a stand alone storage or treatment facility. The combined storage and treatment system model considered in a spreadsheet analysis (which was to determine the least-cost combination) is schematically illustrated in Figure 7-10. This system was evaluated to satisfy the requirements for both the CSO and flood control design storms. The sizing of the storage and the treatment facilities was limited by the maximum CSO outflow rate and the total CSO volume. These constrains were built into the model by incorporating the following boundary conditions:

1) The flow rate diverted to treatment (Q_3) would not exceed the maximum 1/year CSO storm inflow rate at that particular site. If the consolidation inflow rate exceeded the CSO storm outflow rate, (i.e. if the flood control outflow rate was the largest) the excess flow would be discharge directly to the river.

2) The required equalization storage (V_{stor}) was assessed by varying the treatment rate from a maximum rate equal to the maximum CSO inflow rate (the treatment only condition), and a minimum of zero when the assessed storage requirements equaled or exceeded the total CSO volume (the storage only condition).

3) The storage limitations were set at a minimum of zero for the treatment only condition, and a maximum equal to the total CSO volume for the storage-only condition. For the storage-only condition, all inflow in excess of the total CSO volume would be discharge directly to the river. This was only applicable for the cases where the flood control outflow volumes exceeded the total CSO volume at the particular site.



NOMENCLATURE AND MASS BALANCE EQUATIONS:

Q1	=	Consolidation conduit outflow rate
Q2	=	Discharge rate to storage and treatment
	≤	Maximum CSO discharge rate (Qcso)
Q3	=	Treatment rate
	=	0 for Vstor = Vcso
	≤	Maximum CSO discharge rate (Qcso)
Qr	=	Discharge rate to river
	=	0 for Q1 ≤ Qcso
	=	(Q1 - Qcso) for Q > Qcso
dV/dt	=	Change of the volume in storage with time
	=	(Q2 - Q3)
Vi	=	Volume in storage at any time interval i
	=	Vi-1 + (Q2 - Q3) x dt
dt	=	Time interval
Vstor	=	Storage requirement
	=	0 for Q3 ≥ Qcso
	=	Maximum[Vi] for all i and Vstor < the total CSO volume (Vcso)
	=	Vcso for Maximum[Vi] ≥ Vcso

SCHEMATIC AND EQUATIONS FOR THE MODEL USED IN THE OPTIMIZATION ANALYSIS FOR THE COMBINATION OF STORAGE AND TREATMENT OF CONSOLIDATION CONDUIT OUTFLOW

HDR

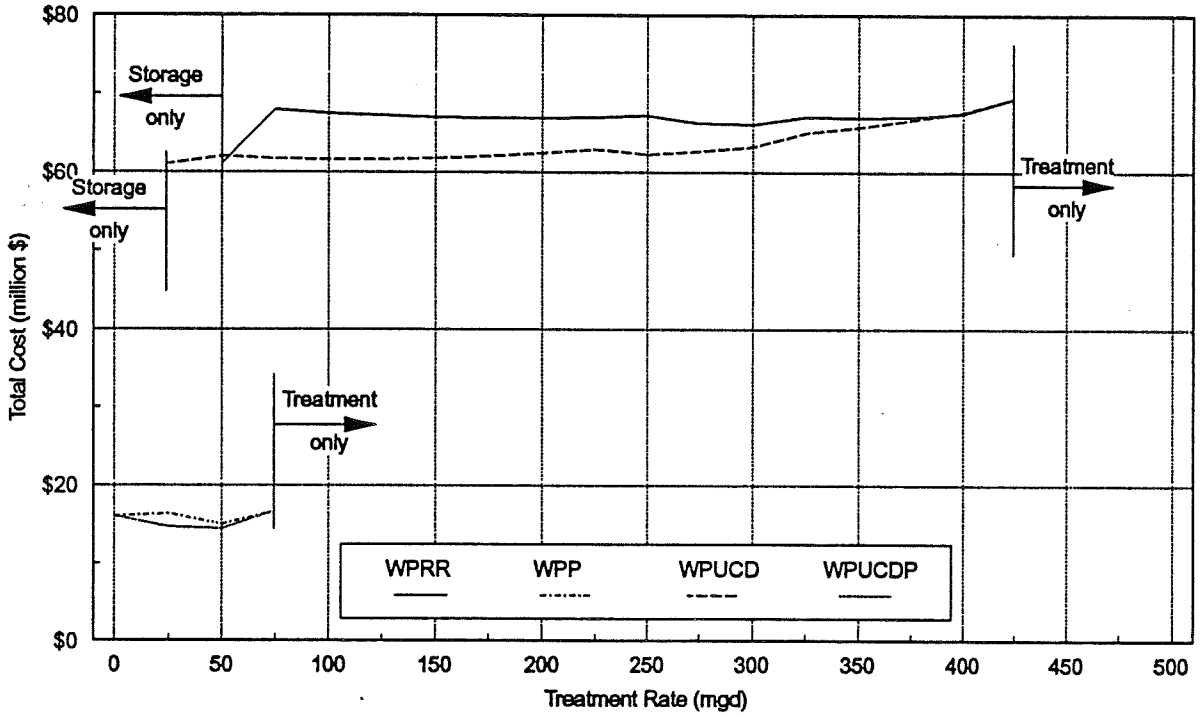
Figure 7-10

To evaluate the benefit of adding storage upstream of the treatment facilities for equalization of treatment plant influent flows, it was first necessary to identify the CSO inflow rates and CSO discharge volumes to the WPRR and the Pioneer Reservoir sites. These inflow rates were extracted from the SSWMM91 output files for the different storage alternative models reviewed in Section 6 using the 1/year CSO storm event.

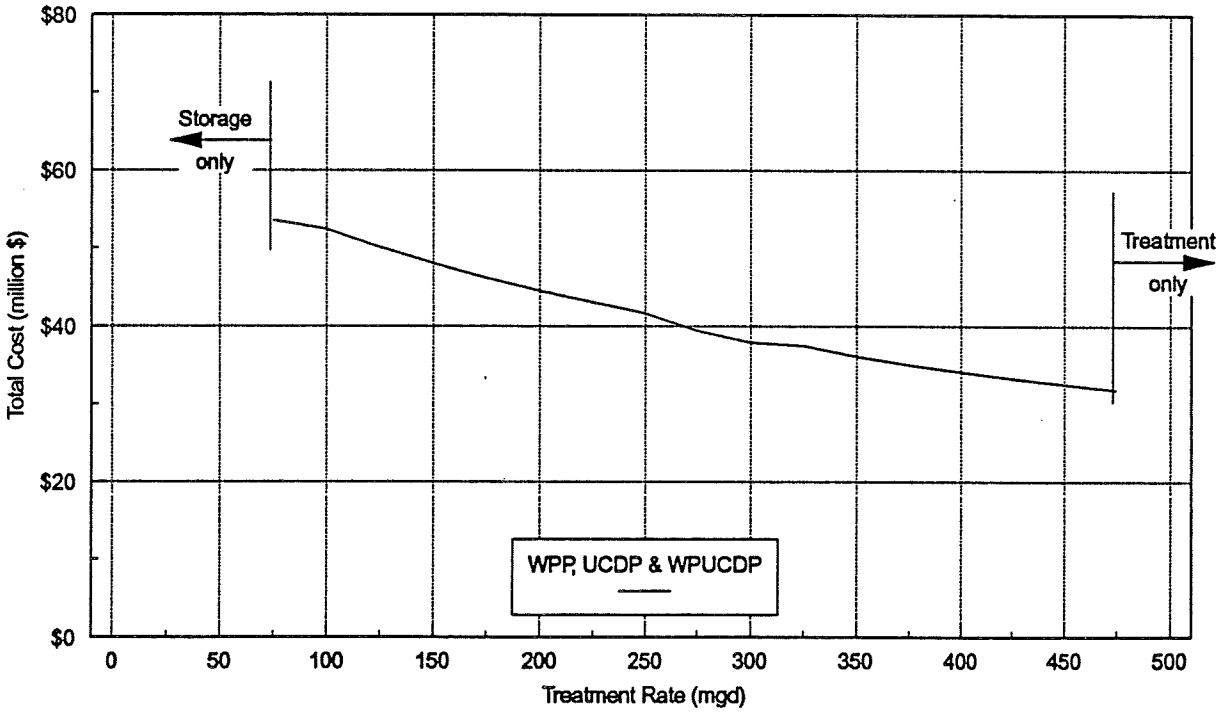
Once the storage volumes for equalization were computed as a function of treatment flow rate, the cost of these storage facilities was developed based on the unit costs reported in Section 4. These storage reservoir costs were added to the treatment plant costs to develop total combined treatment and storage cost data as a function of treatment flow rate.

E.3.a. Least-Cost Treatment/Storage Alternatives for the 10-year Event-- The treatment and storage alternatives using the Western Pacific site (ie. the WPRR, WPUCD, WPP, and WPUCD alternatives) are shown in the upper half of Figure 7-11. For those alternatives in which Pioneer Reservoir overflow is directed to the WPRR site, i.e. the WPRR and WPUCD curves, storage of the 1/year CSO volume (approximately 60 MG) is more cost effective than a storage/treatment combination. On the other hand, for those WPRR alternatives also including use of the Pioneer site, i.e. the WPP and WPUCDP curves, a combination of storage and treatment at WPRR was more cost effective. The estimated facility sizes for these alternatives are (1) a 50-mgd treatment plant with 2.8 MG of storage for the WPP alternative, and (2) a 50-mgd plant with 2.2 MG of storage for the WPUCDP alternative.

By contrast, for the 10-year event alternatives using the Pioneer Reservoir site (i.e. WPP, UCDP and WPUCDP), the construction costs are lower for a treatment facility with no storage than for storage only or a combination of storage and treatment. This treatment facility would have a peak treatment capacity of 475 mgd. Therefore based on the data presented in Figure 7-11, for the 10-year alternatives, storage only or storage and treatment should be provided at the WPRR site and treatment only should be provided at the Pioneer Reservoir site.



COMPARISON OF THE STORAGE/TREATMENT COST AT WPRR FOR THE 1/YEAR CSO CONTROL 10-YEAR ALTERNATIVES



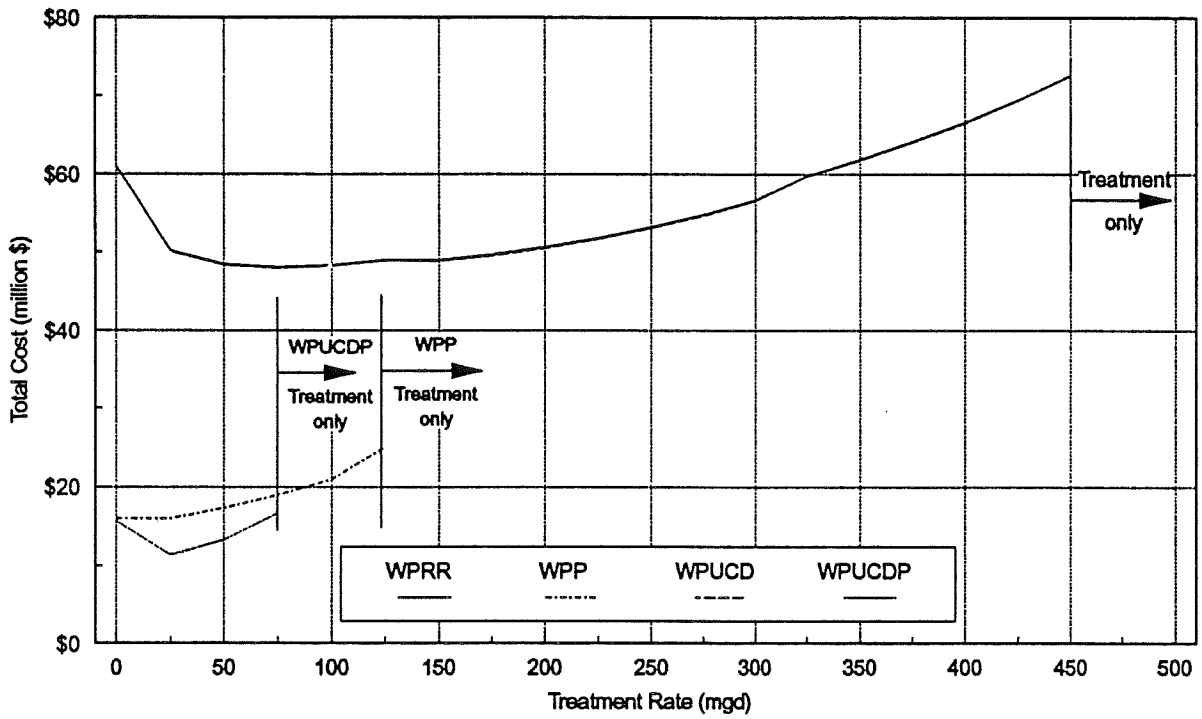
COMPARISON OF THE STORAGE/TREATMENT COST AT PIONEER FOR THE 1/YEAR CSO CONTROL 10-YEAR ALTERNATIVES

HDR

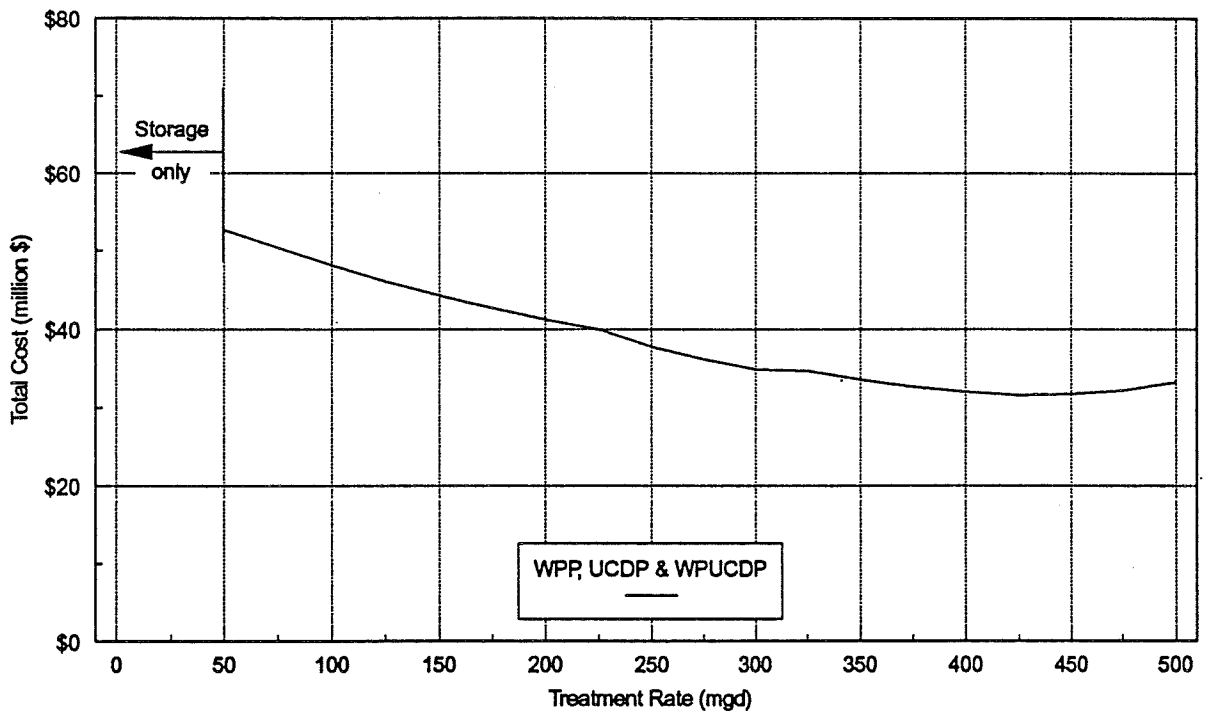
Figure 7-11

E.3.b. Least-Cost Treatment/Storage Alternatives for the 5-year Event-- The data in Figure 7-12 provide similar comparisons for the two sites for the 5-year design storm. For the Western Pacific site, the cost analyses showed that storage/treatment requirements for the WPRR and WPUCD alternatives are identical. For both of these the least-cost combination includes a treatment plant with a peak capacity of 75 mgd plus approximately 32 MG of storage. When facilities at the Pioneer site are combined with treatment and storage at Western Pacific, i.e. the WPP and WPUCDP alternatives, a treatment and storage combination also proved to be most cost effective in each case. In the case of the WPP alternative, the cost difference between storage only and storage plus treatment was insignificant, although the minimum cost was for 8.5 MG of storage with a 25-mgd treatment plant. The least-cost WPUCDP alternative also includes a 25-mgd treatment plant, but with 3.9 MG of storage.

For the Pioneer Reservoir site, the data indicate the least-cost alternative for the WPP, UCDP and WPUCD alternatives is to provide a combination of both treatment and storage. There is little difference in combined storage/treatment cost between providing combined storage/treatment at 425 mgd and providing only treatment at 485 mgd. However, the Pioneer Reservoir site is limited in area and cannot readily support separate treatment and storage facilities. The recommendation is to provide only a treatment facility with a peak flow rate of 485 mgd, the peak rate of inflow to the site for the 1/year CSO event.



COMPARISON OF THE STORAGE/TREATMENT COST AT WPRR FOR THE 1/YEAR CSO CONTROL 5-YEAR ALTERNATIVES



COMPARISON OF THE STORAGE/TREATMENT COST AT PIONEER FOR THE 1/YEAR CSO CONTROL 5-YEAR ALTERNATIVES

HDR

Figure 7-12

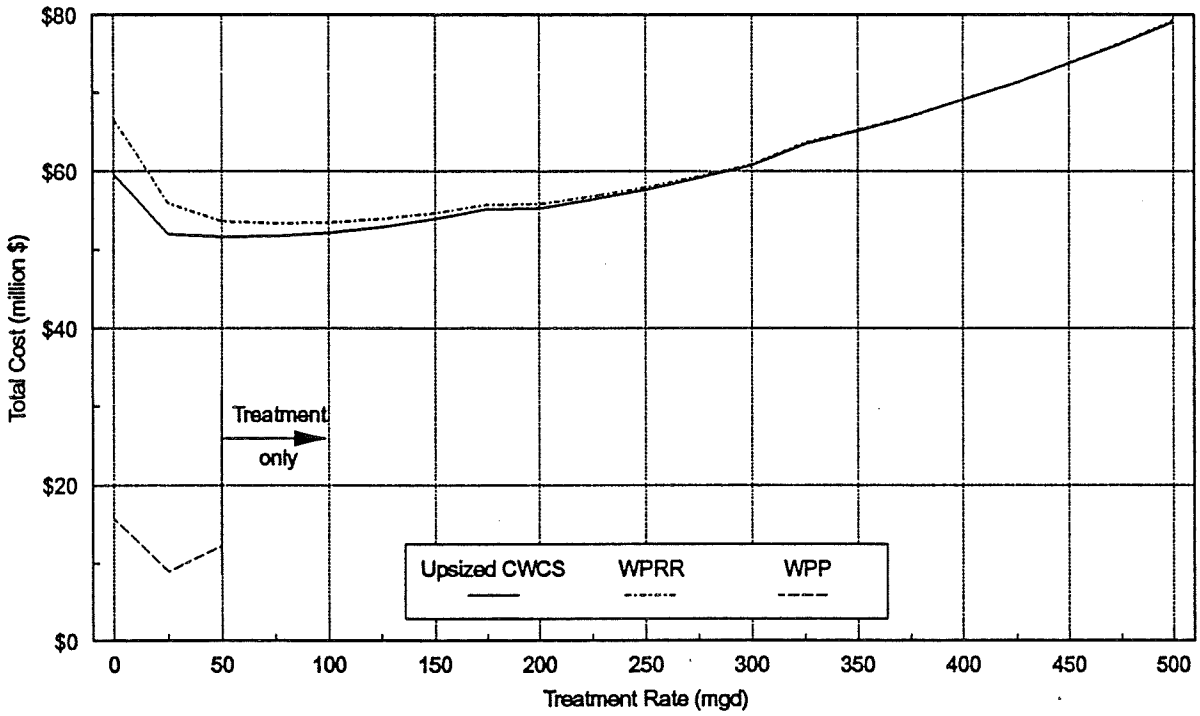
E.3.c. Least-Cost Treatment/Storage Alternatives for the 2-year Event-- For the 2-year event, four alternatives were analyzed: 1) the upsized CWCS alternative for the Pioneer Reservoir site; 2) the upsized CWCS alternative for the WPRR site; 3) the use of both WPRR and Pioneer Reservoir sites with Land Park diversion weirs; and 4) the use of the WPRR site with Land Park weirs and a diversion structure on the Donner Trunk.

The cost data in the lower half of Figure 7-13 represent the upsized CWCS and WPP alternatives for facilities at the Pioneer Reservoir site. The data indicate the least-cost combination is to provide both storage and treatment at the Pioneer Reservoir site. As previously indicated, this site cannot support both storage and treatment as separate facilities. Since the cost difference for providing only treatment versus a combination of treatment and storage at the least-cost flow rate is minor, the recommendation is to provide only treatment at the Pioneer site at a flow rate of 500 mgd.

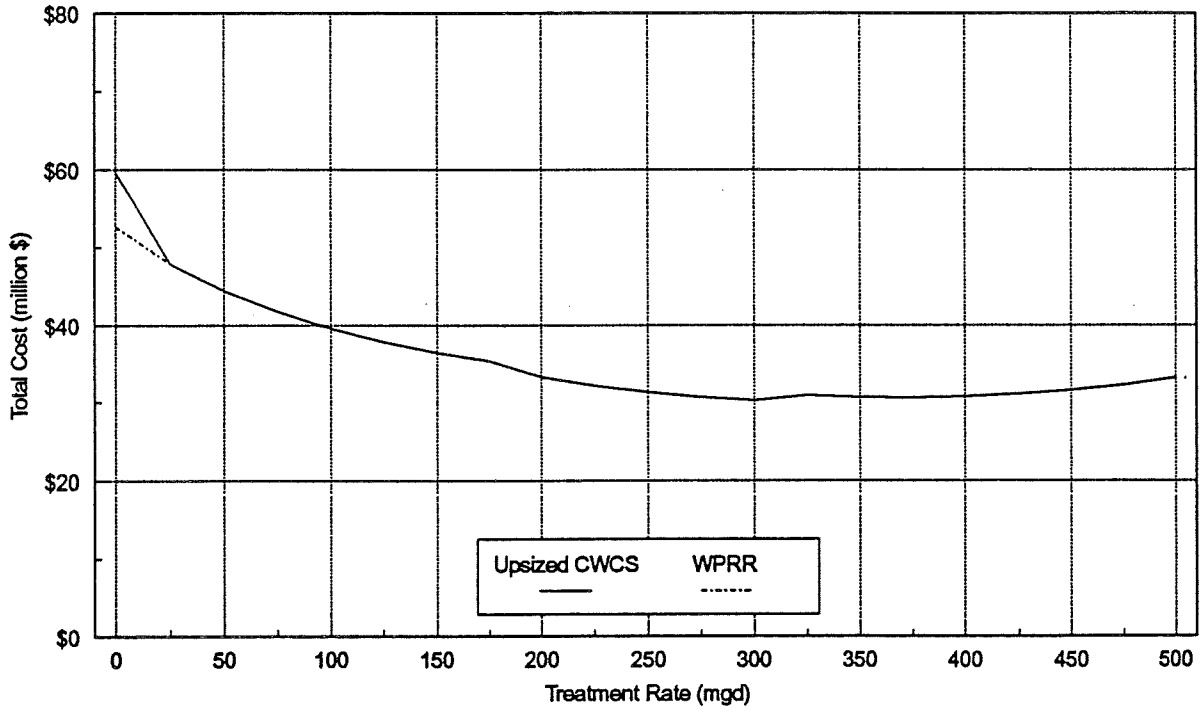
For the upsized CWCS alternative with flow diverted only to the WPRR site, the least-cost combination at the WPRR site is to provide both storage and treatment, as shown in the upper half of Figure 7-13. The optimum storage capacity would be a 41.6-MG reservoir and the corresponding treatment rate would be 50 mgd.

The cost data for the alternative using both the Pioneer Reservoir and WPRR sites, (with the latter receiving inflow from the Land Park weirs) show that the least-cost combination of storage and treatment is for a 25-mgd treatment facility with 1.4 MG of storage at the WPRR site. The least-cost combination for the Pioneer Reservoir site is very close to using treatment only. As with the other alternatives using the Pioneer Reservoir site, the recommendation is to use only treatment due to space limitations for that site. The treatment rate would be 500 mgd.

For the alternative using the WPRR site only with inflow delivered by both the Land Park weirs and a diversion structure on the Donner Trunk, the cost data indicate that the least-cost combination for storage and treatment is to develop both storage and treatment facilities at the WPRR site. The storage capacity would be 38.4 MG and the treatment facility would have a peak capacity of 75 mgd. Figure 7-14 schematically illustrates these 2-year treatment alternative configurations.



COMPARISON OF THE STORAGE/TREATMENT COST AT WPRR FOR THE 1/YEAR CSO CONTROL 2-YEAR ALTERNATIVES



COMPARISON OF THE STORAGE/TREATMENT COST AT PIONEER FOR THE 1/YEAR CSO CONTROL 2-YEAR ALTERNATIVES

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Figure 7-13

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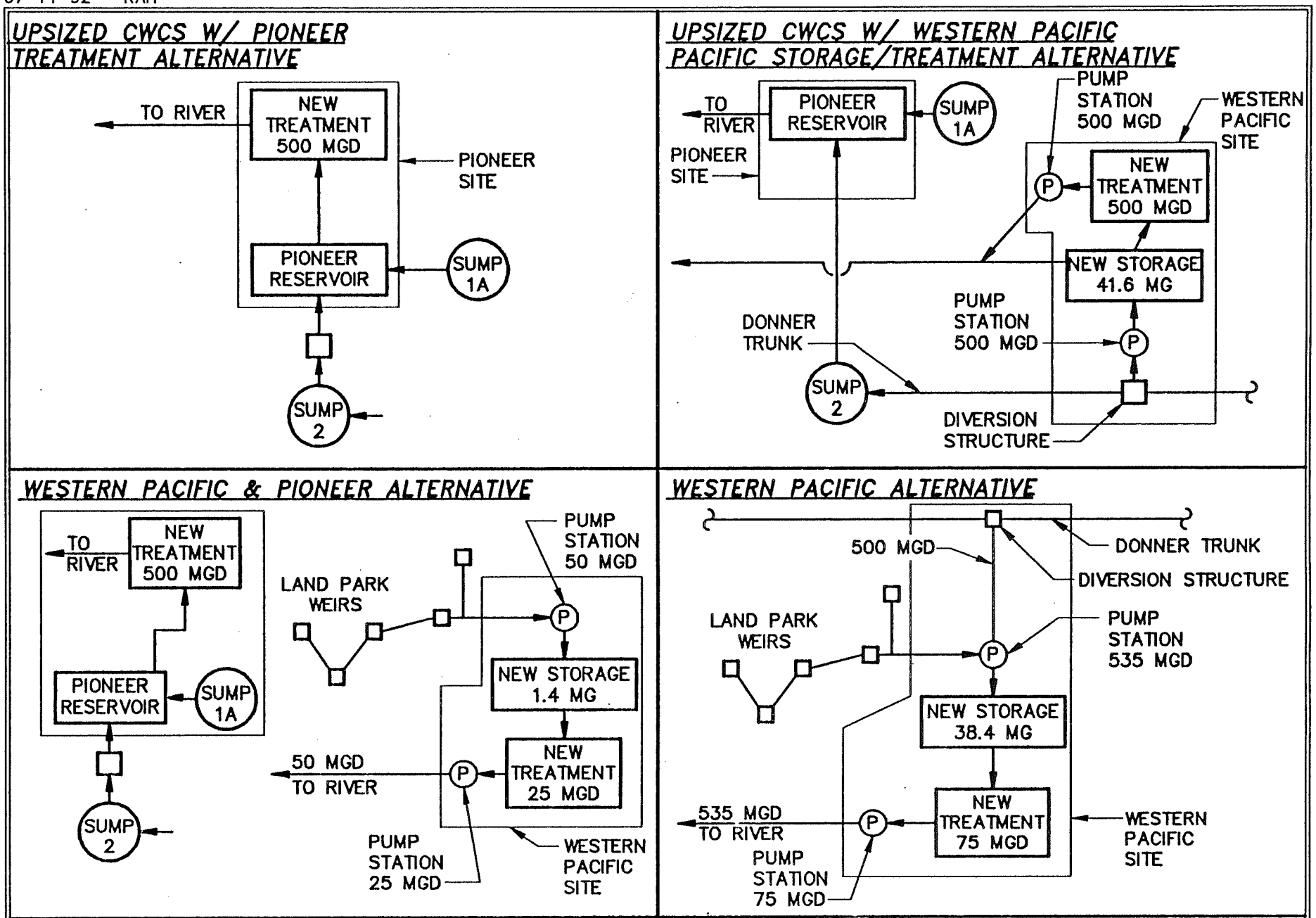


SCHEMATICS OF THE 2-YEAR TREATMENT ALTERNATIVES
1 PER YEAR CSO CONTROL

HDR

Figure 7-14

F. SUMMARY OF THE TREATMENT ALTERNATIVES ANALYSIS

The treatment alternatives were reviewed and compared in terms of their total construction costs. The costs of treatment facilities at four alternative sites were determined and the two least-cost alternative sites, the Pioneer Reservoir site and the WPRR site were retained for further analysis. These sites were evaluated with upstream storage facilities to provide flow equalization and reduce the size of downstream treatment facilities. Plots of the total costs for combined storage and treatment facilities were made and reviewed to identify the least-cost combinations of storage and treatment facilities.

Although the analysis was originally devised to evaluate the optimum treatment configurations for the various candidate treatment sites, it also indicated which alternatives should use only storage as opposed to treatment or a combination of storage and treatment. Consequently the analysis resulted in selecting the optimum configurations for both the storage and treatment alternatives.

Table 7-1 summarizes the analysis results. The combinations of storage and treatment facilities listed in the table were used to develop total project costs for the storage/treatment alternatives, which are presented in Section 11.

TABLE 7-1. LEAST-COST STORAGE AND TREATMENT COMBINATIONS FOR 1/YEAR CSO					
Alternative	UCD Med Center	Western Pacific Site		Pioneer Site	
	Storage Capacity (MG)	Storage Capacity (MG)	Treatment Capacity (mgd)	Storage Capacity (MG)	Treatment Capacity (mgd)
10-Year Flood Control Design Storm					
UCDP	63	-	-	-	475
WPRR	-	60	-	-	-
WPP	-	2.8	50	-	475
WPUCD	32	60	-	-	-
WPUCDP	32	2.2	50	-	475
5-Year Flood Control Design Storm					
UCDP	24	-	-	-	485
WPRR	-	32.2	75	-	-
WPP	-	8.5	25	-	485
WPUCD	11	32.2	75	-	-
WPUCDP	11	3.9	25	-	485
2-Year Flood Control Design Storm					
Upsized CWCS w/ Pioneer Treatment	-	-	-	-	500
Upsized CWCS w/ WPRR Treatment	-	41.6	50	-	-
WPRR and Pioneer	-	1.4	25	-	500
WPRR only	-	38.4	75	-	-

SECTION 8
NEW SEPARATED STORM DRAIN AND SANITARY SEWER SYSTEMS

A. BACKGROUND

This section describes the development and analysis of new separated storm drain and sanitary sewer systems. Either of these new systems would be operated in conjunction with the existing combined sewer network for handling either stormwater or sanitary inflows.

The new separated storm drain alternative would provide stormwater drainage for the entire study area, excluding the existing separated sewer systems. Stormwater in this system would be discharged to the Sacramento River without treatment, since it contains no sanitary sewage. In this analysis, the existing combined sewer system would be used only as a sanitary sewer system.

The new sanitary sewer alternative would serve the entire study area, including the sanitary flow from areas with existing separated sewer systems. All wastewater collected by this system would be pumped to the Sacramento Regional Wastewater Treatment Plant (SRWTP) for secondary treatment. In this analysis, the existing combined sewer system would be used only as a storm drain system.

B. MODELING ASSUMPTIONS

B.1. Separated Sewer Alignment

The new separated storm drain and sanitary sewer systems must be aligned to facilitate the connection of existing storm drain and sanitary services. The selected new separated system alignments are therefore, with the exception of a few deviations, a match of the existing combined sewer system. In order to comply with the depth requirements of service connections, the invert elevations of individual lines were set equal to or deeper than the existing combined sewer system.

Other assumptions regarding the physical alignment and location of the new separated sewers are noted below.

1) To avoid physical interference, the alignments of the main interceptors in the new sewers were selected to avoid the alignments of main interceptors in the combined sewer system.

2) The separated storm drain system for the region north of Highway 50 is separated from that south of the highway, with the northern sector draining to the site of Sump 1A, and the southern sector draining to the Sump 2 site.

3) The entire sanitary sewer system conveys wastewater to the existing Sump 2 site, from which it will be pumped to the SRWTP.

B.2. Design Criteria

The sizing of both systems is based on the design criteria presented in Section 3. For both alternatives, the slopes of individual pipes were adapted from the existing combined sewer system. However, the vertical alignments of the new systems were adjusted to match the crown elevations of the major lines entering and leaving a specific junction.

The new sewer design criteria include the following:

1) The maximum allowable flow capacity of the new sewers was limited to 80 percent of the full flow capacity of the line.

2) Surcharging of the system was not allowed at any time.

3) The minimum pipe coverage was set at 2.5 feet.

B.3. Design Inflow

B.3.a. Storm Drain System--

The 10-year design storm for flood control was applied to this system analysis. Since the separated storm drain system will only handle stormwater inflow, constant inflows at all nodes were ignored.

B.3.b. Sanitary Sewer System--

The City prepared estimates of average dry weather sanitary flow rates based on a land use assessment for each of the SWMM basins in the RUNOFF block setup for the existing combined sewer conditions. These estimates were described in and provided to HDR, in a memorandum dated November 25, 1991. The City's average flow rates were incorporated into all alternative improvement plan EXTRAN setup files in which the sewers remained combined (sanitary and stormwater contributions in a single network). Average sanitary flow rates were used in those configurations because stormwater inflows to the system were the driving factor in sizing combined system conveyance improvements. However, in order to size pipes for the separate sanitary conveyance system in this analysis, peak sanitary flow rates were needed.

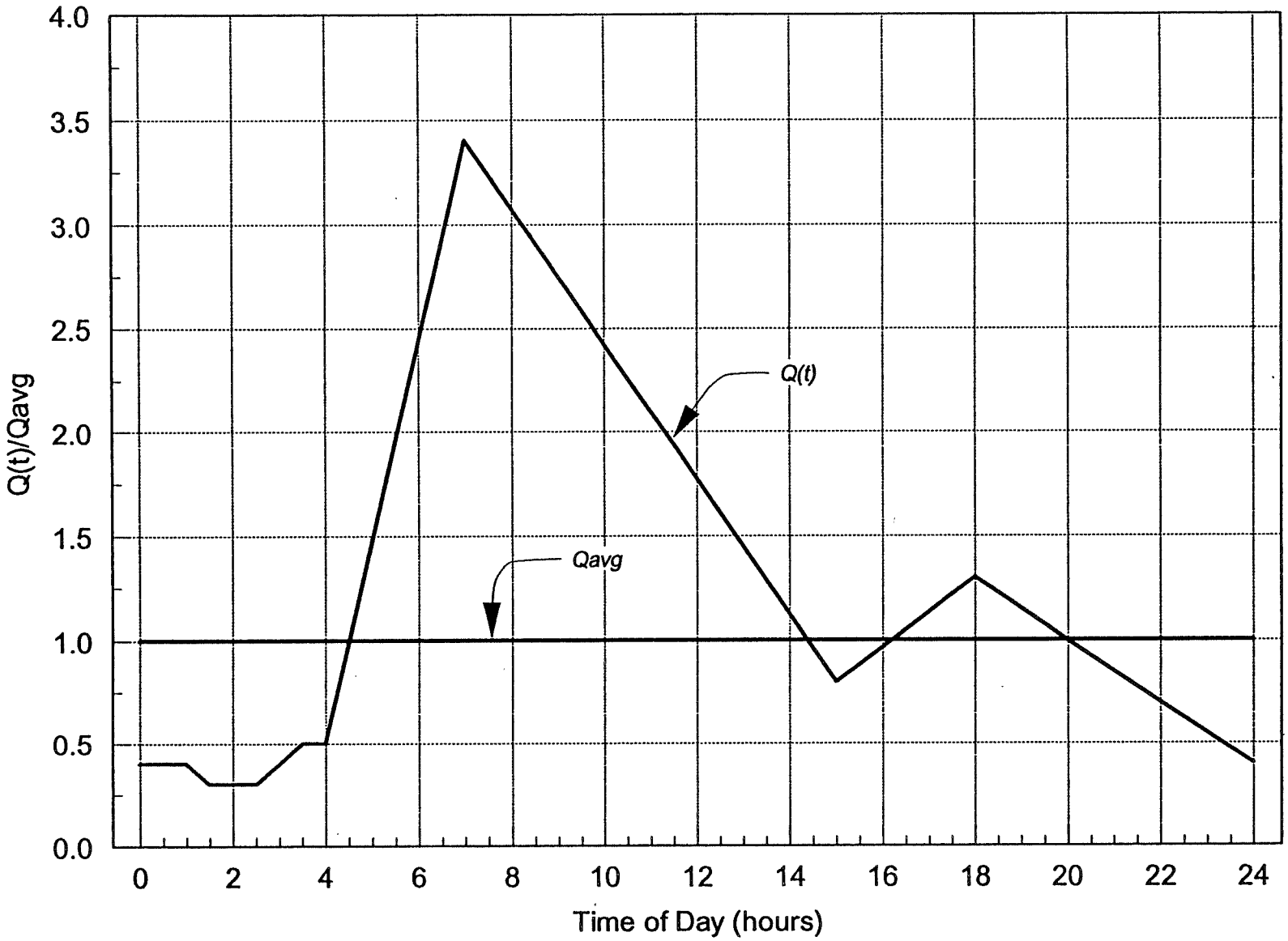
Average sanitary flow rates can be converted to peak flow through the use of either peaking factors or diurnal curves. Peaking factors, which relate peak flow to either average flow or contributing area, are most often used in static models of hydraulic systems. Peaking factors are inversely proportional to the size of the contributing area, for a given point of analysis in the sewer system. The City of Sacramento's sanitary flow peaking factors are presented on Plate 9-2 of the City's *Design and Procedures Manual and Improvement Standards* (September 1, 1990). In the City's case, the factors are represented as a relationship between peak and average flow.

Diurnal curves, on the other hand, are similar to a unit hydrograph approach for analyzing runoff in that they prescribe the "shape" or time variation of a sanitary "hydrograph" and are dimensionless. Imposing an average sanitary flow rate on a diurnal curve allows for the development of a time varying sanitary flow curve, which can then be used in a dynamic or routing type of model.

Normally, diurnal curves are developed from flow monitoring data collected for basins consistent in size to those represented in the hydraulic model. Measured peaking factors (in this case defined as the ratio of peak flow to average flow) are nearly always less than those specified as design criteria. Depending on the system and the model application, safety factors (in terms of a multiplier or percentage of full pipe flow) are applied to ensure that the designed pipe will convey the projected amount of flow.

In order to properly size a new, separate sewer system using the Sacramento SWMM, sanitary hydrographs were developed using a diurnal curve approach which includes the peaking factors contained in the City's *Design and Procedures Manual and Improvement Standards*. Since dry weather flow data are not available for small basins within the Sacramento combined system, the basic shape of the diurnal curve (e.g. the hour at which flow will peak) had to be estimated through an examination of curves developed for small basins in other municipalities, including Everett and Seattle, Washington and Sanger, California. In addition, the curve incorporates a peaking factor of 3.4 at peak hour flow, which was taken from Plate 9-2 of the City's *Manual*. This factor was selected because the average of the sanitary flow contributions from basins in the combined system is approximately 0.1 mgd, based on the City's analysis. The diurnal unit hydrograph as used in the new separated sanitary sewer modeling is shown in Figure 8-1.

A sanitary hydrograph is developed for each basin by imposing the diurnal curve on the average sanitary flows prepared by the City. Added to these is an allowance of 200 gpd/acre to account for groundwater infiltration. This allowance is consistent with the design criteria being used by the Regional Sanitation District on their Sacramento Sewerage Expansion Study. Basin sanitary hydrographs are combined to represent total sanitary contribution to each junction in the combined system. The resulting sanitary hydrographs are then entered into the EXTRAN block for sizing the new sanitary interceptor and lateral system, which would parallel the existing combined sewer system.



DIURNAL UNIT SANITARY INFLOW HYDROGRAPH USED IN THE SEPARATED SANITARY SEWER ANALYSIS

HDR

Figure 8-1

C. MODELING RESULTS

C.1. Storm Drain System

1) Due to the relatively small catchment area, the peak attenuation in the drainage system is relatively small. The maximum total flow for the system was approximately 3,500 cfs.

2) By dividing the basin into two sub-basins, i.e. the sector north of Highway 50 and the sector south of the highway, the flow rate at each of the two system discharge points (at Sump 1A and Sump 2) was approximately 50 percent of the total flow rate.

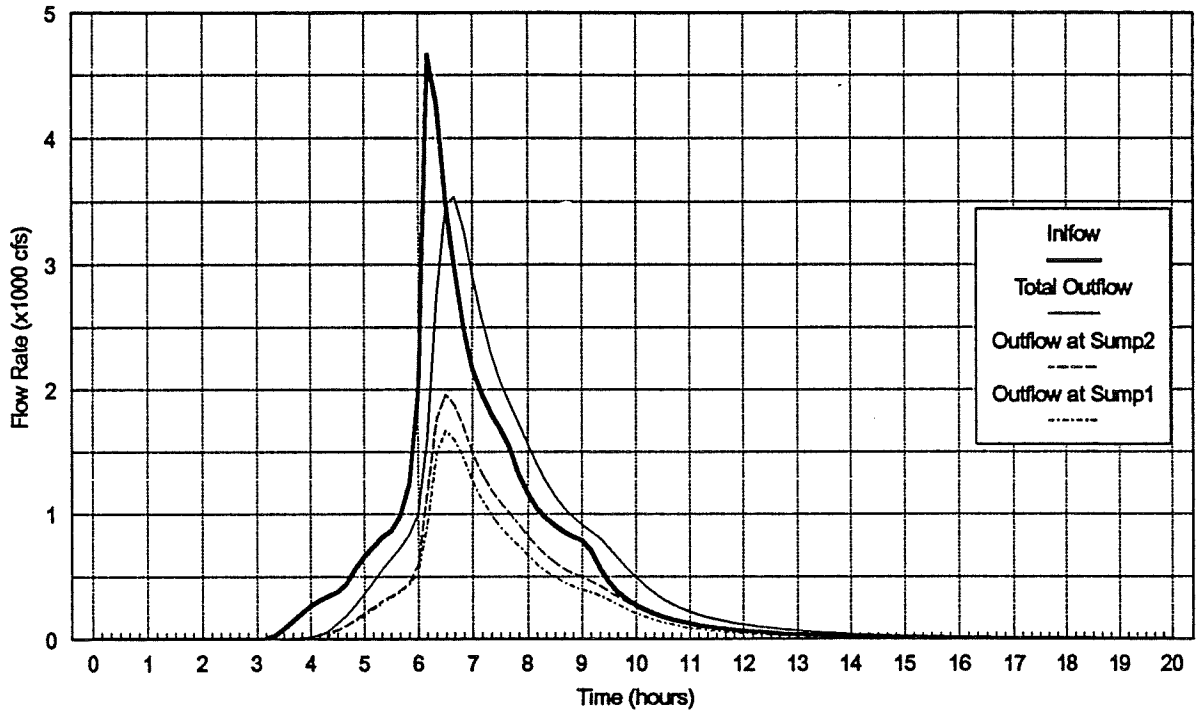
3) Construction limitations, as outlined in Section 3, require that all pipes with diameters larger than 12 feet be tunnels, and that to be cost-effective, a tunnel must be the same diameter for at least one mile in length. In order to comply with these design guidelines, the separated storm drain system required almost 15,000 feet of 19-foot and 11,000 feet of 20-foot tunnels.

C.2. Sanitary Sewer System

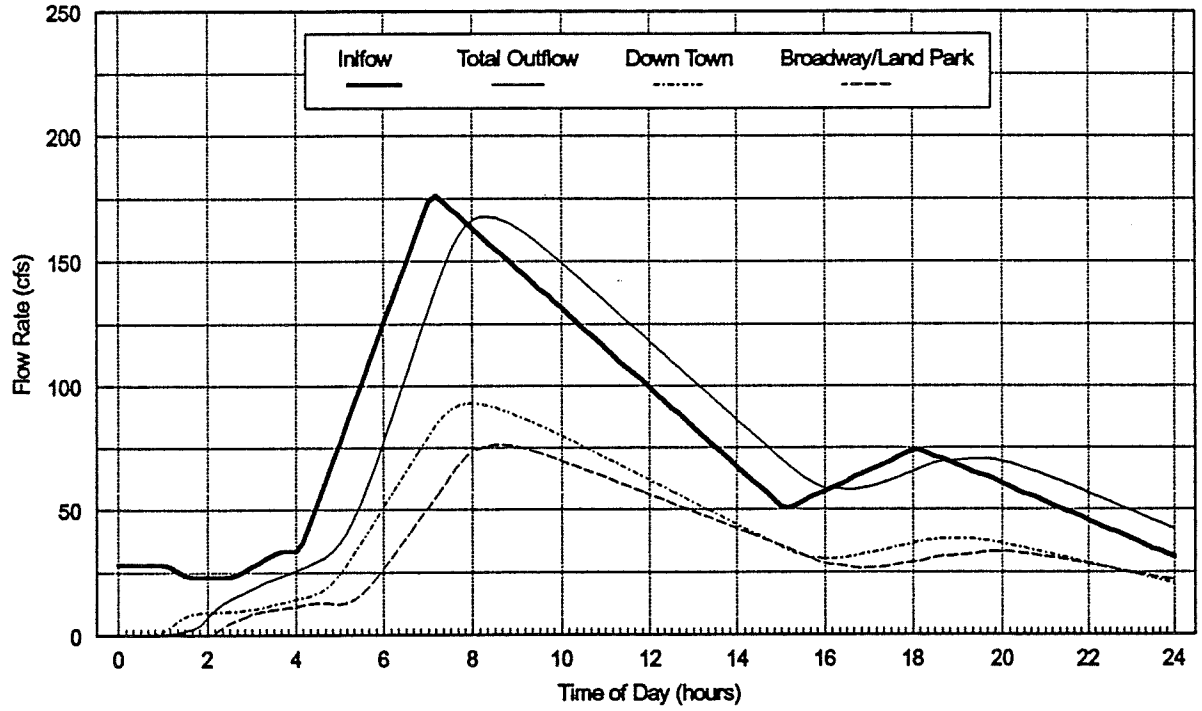
The model output is summarized in Table 8-1 and shown in Figure 8-2.

TABLE 8-1. SUMMARY OF SEPARATED SEWER MODELING OUTPUT

Site Description	Discharge Rate (cfs)	Discharge Volume (cu ft)
Storm Sewers:		
Sump 1A	1670	16,500,000
Sump 2	1960	14,250,000
Total		30,750,000
Sanitary Sewer:		
Sump 2	168	6,500,000



INFLOW AND OUTFLOW HYDROGRAPHS FOR THE SEPARATED STORMWATER DRAIN SYSTEM



INFLOW AND OUTFLOW HYDROGRAPHS FOR THE SEPARATED SANITARY SEWER SYSTEM

HDR

Figure 8-2

The analysis demonstrates the following:

1) The attenuation in the sanitary sewer system's peak flow rate was less than that of the stormwater drain system.

2) The maximum anticipated system discharge peak at Sump 2 of approximately 168 cfs (108 mgd) was more than the presently allocated maximum treatment capacity at the SRWTP of 93 cfs (60 mgd). The maximum total daily discharge, however, was estimated at approximately 50 mgd, which is within the allocated treatment capacity.

3) As in the case of the separated storm drain system, the sanitary flow contributions by the northern sector (north of Highway 50) and the southern sector (south of the highway) are of similar magnitude. Unlike the separated storm drain system, the entire separated sanitary system drained to the Sump 2 site from where it would be pumped through the existing force main to the SRWTP.

D. SUMMARY

The new separated storm drain system was sized to convey the runoff as a result of a 10-year 6-hour rainfall event. This alternative did not prevent separated stormwater discharge to the river. The conveyance capacity of the existing sewer system, which would become a dedicated sanitary sewer system, would exceed the sanitary conveyance requirements.

A new separated sanitary sewer system would, however, not provide flood control or prevent stormwater discharge to the river. It would only remove the contaminated fraction from the present combined sewer outflow. Although this would satisfy the CSO control criteria, it would not alleviate flooding within the service area.

SECTION 9
INFLOW CONTROL ALTERNATIVE

A. INTRODUCTION

Outflows occur due to the inadequate capacity of the conveyance system to handle large stormwater inflows. During heavy rainfall events, the sewers become surcharged, causing flow to back up onto streets in low-lying areas. The approach taken in the Phase 2 analysis with regard to outflows was that the presence of flooding approximately parallels the occurrence of outflows. Therefore, alternative improvements designed to control flooding would also control outflows for a given design storm. For instance, conveyance upgrades provide sufficient capacity to prevent flooding in the service area during design storm events, and were also assumed to provide control of outflows (limited surcharging, however, was allowed). It was estimated by the planning team, during the Phase 2 study, that the presence of outflows would be difficult to quantify.

A.1. Regional Board Questions and Comments

The Regional Board posed several questions and comments related to the evaluation of combined system outflows in its comments on the *Phase 2 Detailed Technical Report*. These were presented in an internal memorandum and letter to the City dated August 16, 1991, and are summarized below for reference.

1. "What real net reduction in outflows is realized by the complete containment [of flooding and CSOs] in the 10/1 scenario?" (The "10/1 scenario" refers to the Phase 2 study analysis criteria, which consisted of controlling flooding during the 10-year 6-hour storm, and reducing CSOs to 1 event per year on an average annual basis.)

2. "Out of this volume [i.e. the 10-year flood volume controlled by improvements] what is the reduction of outflow volume? The goal is to reduce the outflows from the system, and not to control general street flooding unrelated to outflows. The differentiation between the two should be made. It is understood that improving the system to reduce outflows will substantially improve the flooding conditions and that the 10/1 proposal would have no outflow or flooding occurring from a storm with a frequency of once every ten years."

3. "Similarly, the costs presented for the improvements should differentiate, if possible, between costs related to outflow prevention, and those that only help reduce general flooding. Improvements necessary to prevent outflows that also have a side benefit of flood prevention should be included in the cost of the outflow prevention."

4. "What is the occurrence of outflows out of the collection system that consists of 15-inch and less diameter pipes? Do the improvements for the collection system help only flooding, or do they substantially reduce outflows?"

5. "Many costs are applied to the alternatives that are costs that would have been accrued by the City even without an upgrade of the system. These costs should be broken out so that a true cost of what we are requiring the City to do can be ascertained."

6. "The focus and content of the [Phase 3] report..." and "improvements and their related costs" should be "primarily for outflow reductions. Benefits of flood control and CSO reduction should be secondary. Optimization of the criteria for maximum reduction should be made."

In these comments, the Regional Board requested that the City determine the quantity of outflows from the combined system, and differentiate between floodwater volumes caused by stormwater ponding due to hydraulic restrictions in the system from the floodwater volumes caused by outflows from a surcharged system. The Regional Board also requested that the City prepare cost estimates for improvements to control system outflows alone, separating these from the costs for the additional improvements necessary to control stormwater ponding and combined sewer overflows.

A response to the Regional Board's request and comments was prepared in the form of a technical memorandum entitled *Approach to Outflow Analysis* (dated February 13, 1992). That memorandum presented a feasibility evaluation of quantifying (volume, location) outflows in the combined sewer service area, and of differentiating between floodwater due to ponding versus floodwater due to system outflows. Given the analytical tools available to the City or through the development of new tools, it was concluded that such an evaluation is not possible.

A.2. Scope of Analysis

This section is a follow-up to the previously completed February 13, 1992 technical memorandum. It presents a review of the feasibility and cost of controlling outflows using inflow regulation devices in the combined sewer collection system. It also provides recommendations for further study. Although it is not feasible to determine the amount of outflows in the service area, it is possible to implement improvements specifically directed towards controlling them.

The Inflow Control Alternative, as evaluated in this section, is incomplete compared to other alternative plans, with regards to meeting flood and CSO control criteria. This section is a brief examination of the feasibility, including the potential advantages and disadvantages, and cost for only regulating stormwater contributions to the combined sewer system. It does not provide for the cost of mitigating the excess volume and duration of floodwater created by inflow regulation devices. In addition, the control of CSOs to the Sacramento River as a part of this alternative is not addressed. It is possible that, with construction of mini-detention ponds, sufficient street storage may be provided to control CSOs to a sufficient degree, however, this was not quantified as part of the Phase 3 analysis.

A.3. Detailed Description and Causes of Outflows

As stated in the list of definitions in Section 1 of this Technical Memorandum, outflow is combined sewage containing raw sanitary sewage, which backs up and flows out of the combined system into generally low-lying areas, streets, or below grade structures and basements. When outflows occur in low-lying streets, the combined sewage generally comes out through street drop inlets (DIs), which normally function as street drains to convey stormwater runoff into the combined sewers. Outflows into below-grade structures usually enter through the structures private sewer tap connection to the combined sewer system. Whether or not outflows to a low-lying structure or basement occur is highly site-specific and dependent upon the configuration and elevation of the individual private sewer tap connection to the combined sewer and the relative elevation of the combined

sewer pipes to the low lying structure or basement.

Outflows generally occur during periods of intense rainfall, and are caused by differences in water level throughout the system. Combined sewage is forced out of a surcharged sewer system in low-lying areas when neighboring higher-elevation areas are flooded with excessive amounts of stormwater. Figure 9-1 illustrates this phenomenon. Figure 9-2 indicates how flooding occurs when the hydraulic grade line (water level) in the system rises above a low-lying manhole. The relationships of hydraulic grade lines and structure in the figure show, for example, that if inflow continued to enter Manhole B during a 5-year or 10-year storm event, the water level would not only rise in Manhole B, but also cause outflow from Manhole A upstream.

B. APPROACH

B.1. Alternative Overview

One of the goals of the combined sewer system improvement program is to prevent outflows of combined sewage into streets and buildings during storm events. Tunnel, storage and conveyance alternatives control outflows by providing conventional conveyance and storage systems of sufficient capacity to prevent surcharging of the combined sewers during design storm events.

The inflow control alternative takes a different approach and establishes controlling outflows as its primary objective. Flooding would undoubtedly worsen. CSO control is considered a secondary goal. Controlling outflows from the combined sewer system is achieved by limiting stormwater flow into the system to that which can be conveyed by gravity, that is, inflow rates are matched to carrying capacity of the existing system. With such limitations on system inflow, the duration and volume of street flooding is increased. An overall improvement program incorporating outflow control would therefore include mitigating flooding impacts by incorporating controlled street detention. Theoretically, inflow regulation with street detention will attenuate the peak runoff flows that would normally be conveyed by the collection system, thus potentially reducing the cost of conveyance and storage improvements necessary to carry the excess flow. Figure 9-3 illustrates the principles behind the outflow control alternative.

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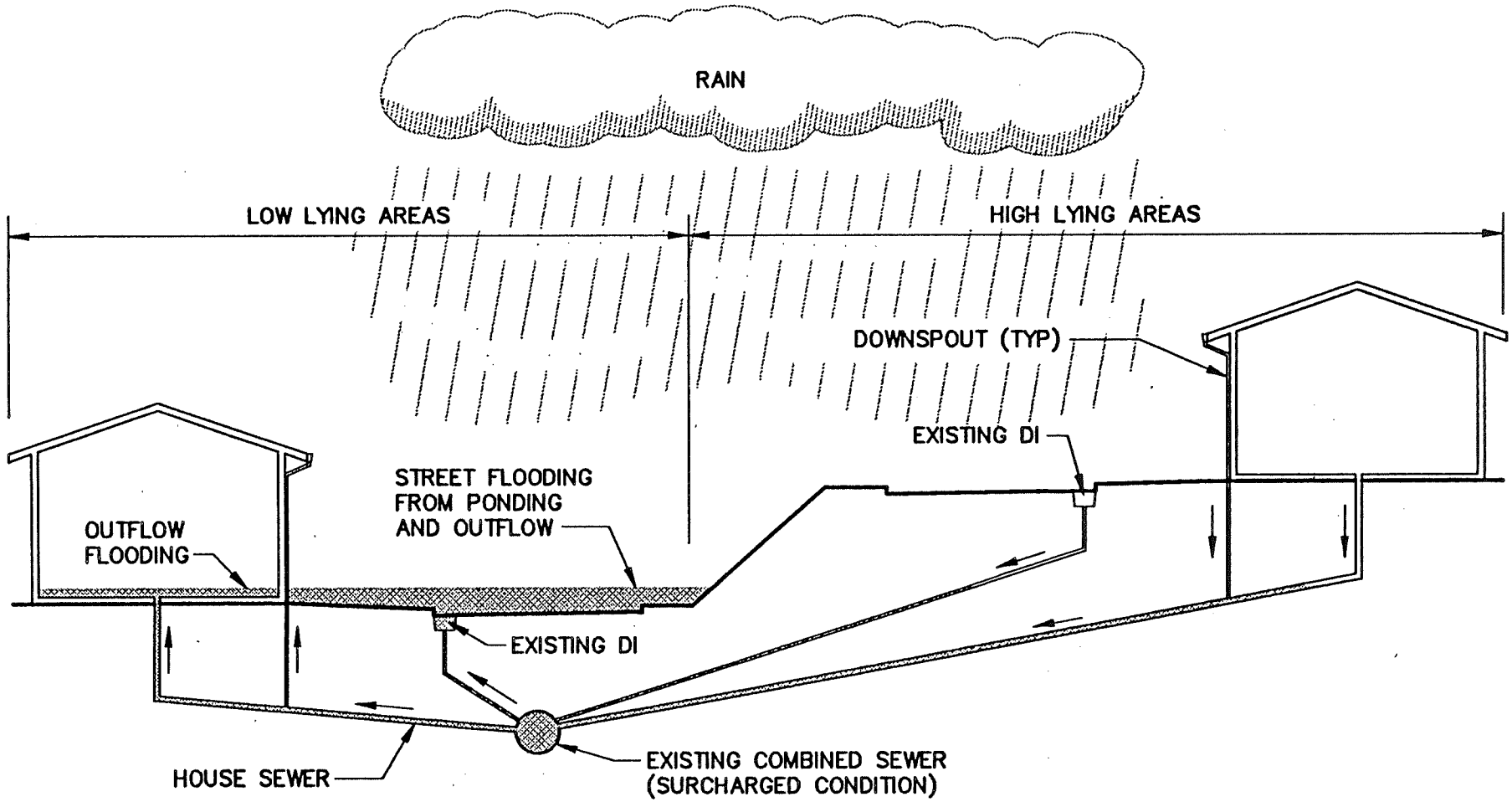

LEGEND

→ FLOW DIRECTION

EXISTING CONDITION – SURCHARGED SEWER WITH OUTFLOW AND STREET & BUILDING FLOODING

HDR

Figure 9-1

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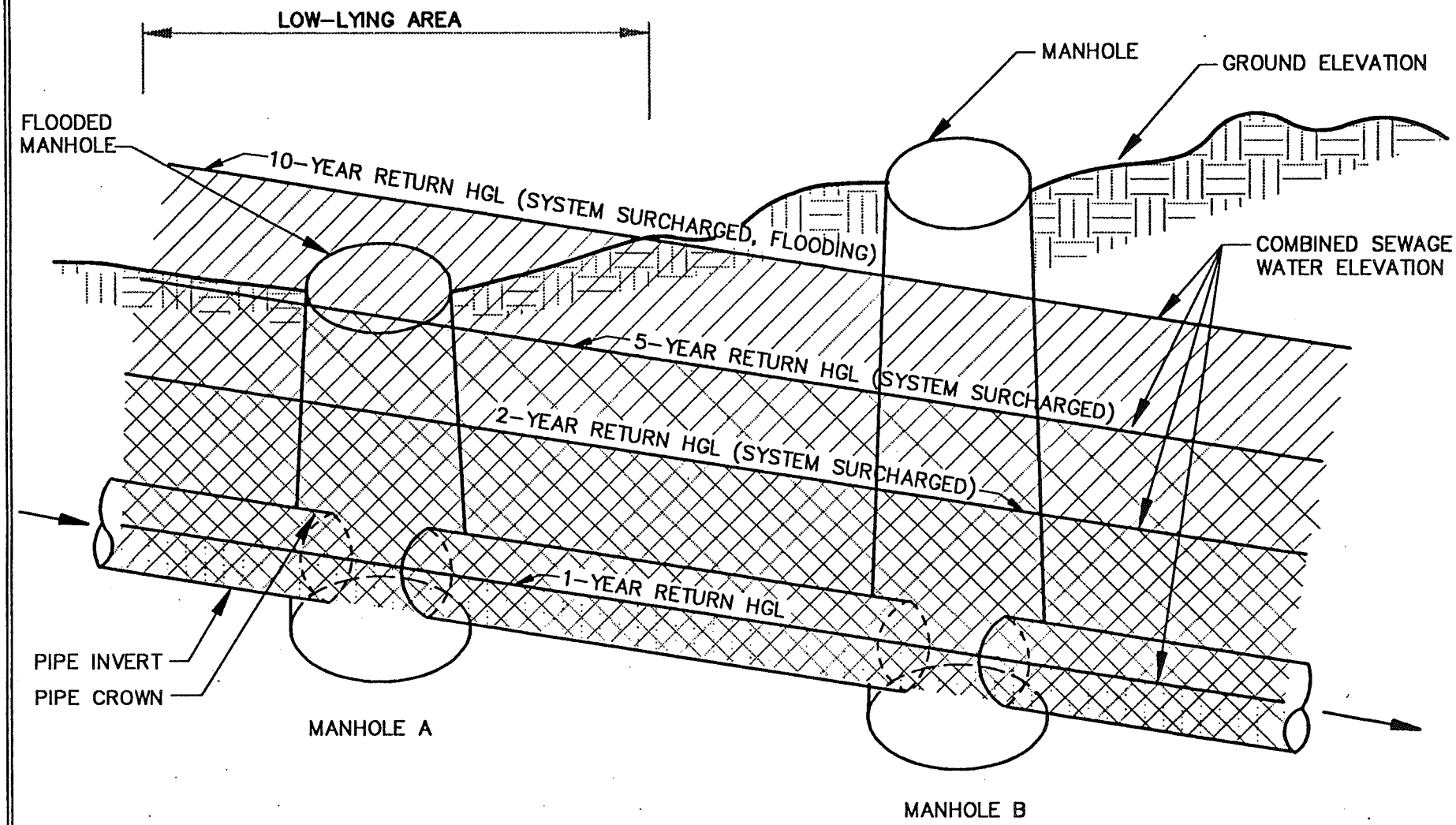
TB-FIG48.DWG
03-11-92 - RAH



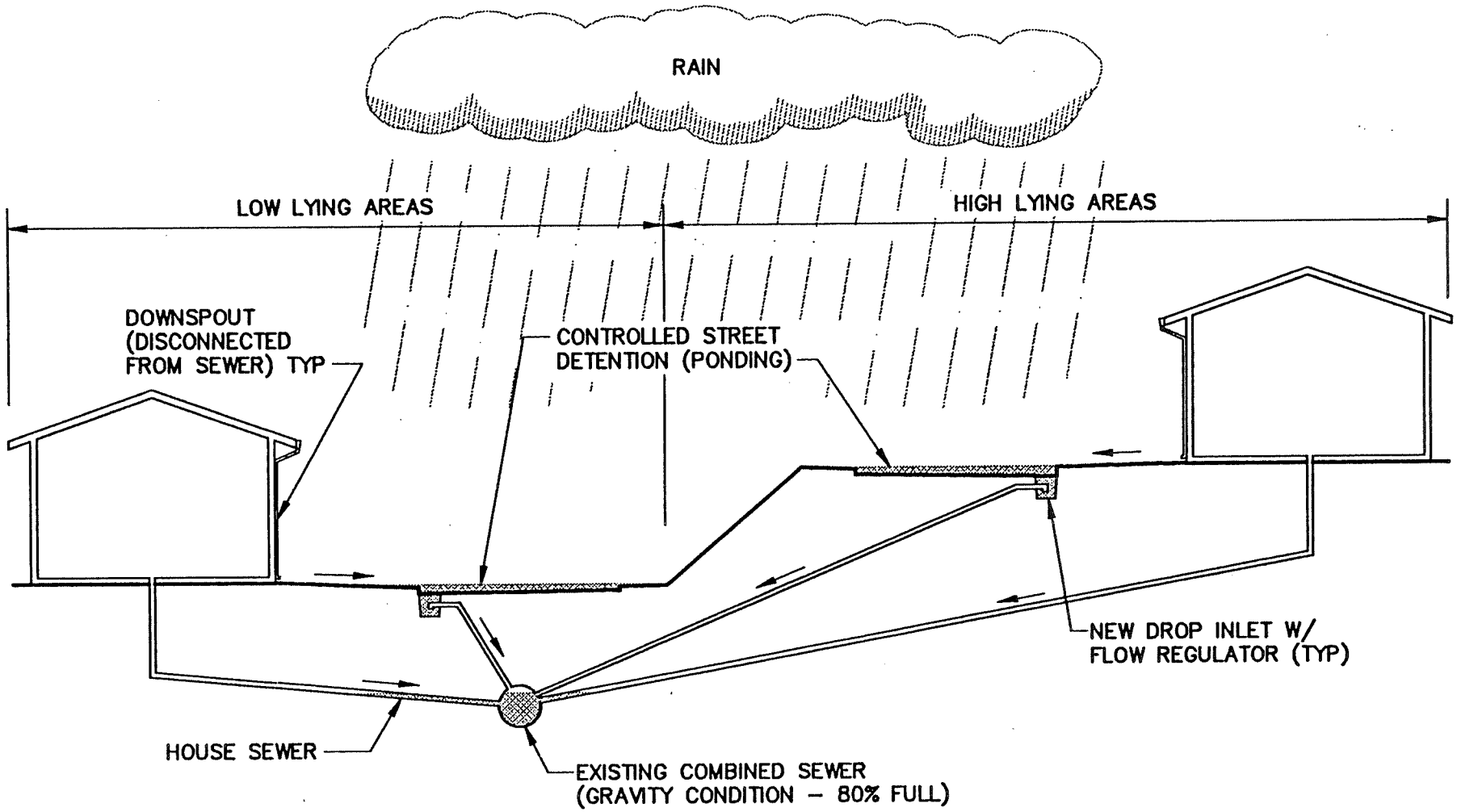
HGL = HYDRAULIC GRADE LINE OR WATER SURFACE ELEV FOR COMBINED SEWER

COMBINED SEWER DESIGN STORMS AND FLOODING DUE TO A SURCHARGED SYSTEM

HDR

Figure 9-2

07140-001-141-36
TB-FIG10.DWG
04-22-92 - RAH



LEGEND

→ FLOW DIRECTION

CONDITION WITH FLOW REGULATION AND CONTROLLED STREET DETENTION

HDR

Figure 9-3

B.2. Objectives

This analysis of the Inflow Control Alternative has the following objectives:

1) To determine the feasibility of providing a system to control outflows by using flow regulators to control stormwater flow into the system.

2) To estimate the cost of outflow control by inflow regulation alone without including costs for mitigating the resulting increase in stormwater ponding.

3) To determine the feasibility of improvements such as street detention and conveyance to mitigate the increased stormwater ponding caused by controlling stormwater inflows.

4) To outline the direction of further study needed to completely analyze an outflow control alternative in order to allow for comparisons between it and the other conveyance and storage alternatives.

C. FEASIBILITY AND DESCRIPTION OF OUTFLOW CONTROL BY INFLOW REGULATION

Installing flow regulators at the drainage inlets to the combined system will control outflows by limiting flows into the system to its hydraulic capacity in a gravity flow condition. Low-maintenance flow restrictors have been shown to work successfully in Skokie, Illinois; Seattle, Washington and other communities. However, outflow control by restricting inflow is not without impact, since the combined system will be able to carry less flow in a gravity (open channel) condition than it can when surcharged. Thus, during periods of peak rainfall, less water will enter the system and more will pond, flooding the streets. Mitigating increased flooding is therefore a significant part of implementing this alternative.

The following subsections present technical information and a feasibility discussion of outflow control by inflow regulation. Two types of flow regulator devices, hanging trap and vortex, are described. Mitigation measures for flooding increases are also presented.

Based on the technical information provided below, four alternative outflow control configurations were developed, which are described in a subsequent subsection. Cost estimates were prepared for one of these alternatives, involving flow regulator installation in drainage inlets.

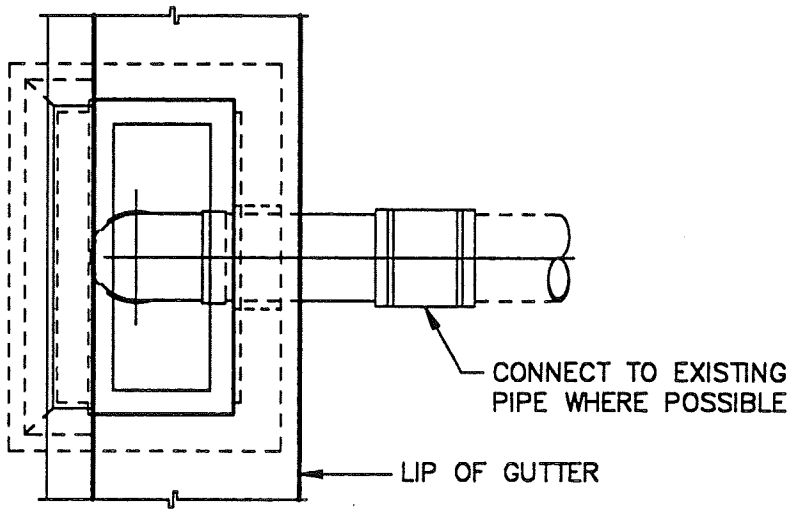
C.1. Flow Regulation Devices and Installations

Flow regulation devices are designed to limit inflows to the system, thereby "matching" inflow to the hydraulic capacity of the system. Since surcharging is controlled to a design storm/flow level, the occurrence of reverse flows is minimized. However, if system surcharging and reverse flows do occur, the devices described below do not provide backflow prevention.

Though many types of flow regulators exist, installations in Skokie and Seattle have shown two types to be relatively maintenance-free and cost-effective. A very simple device is the hanging trap. Figure 9-4 illustrates an installation in a drop inlet. Hanging traps are typically shop-built from a PVC pipe elbow and endcap, with the cap drilled to create a sharp-edged flow restricting orifice sized to pass the allowable design flow for its location.

For low flow rates, i.e. for orifices less than 3 inches, the small orifices of the hanging traps tend to clog. Thus, more expensive vortex flow regulators which have larger intakes for a given flow are recommended. Figure 9-5 illustrates a vortex regulator installed in a DI.

Vortex regulators are custom manufactured by several suppliers, including Hydro Brake Systems, H.I.L. Technology, Inc., and John Meunier, Inc. Appendix B of this memorandum contains catalog cuts for these manufacturers. These devices have no moving parts and operate by directing the flow to rotate inside a cylindrical vessel toward a central outlet creating a vortex. The resulting vortex flow has such high peripheral velocities and large centrifugal forces that a pressurized air filled core is created in the center of the chamber. The back pressure from the air filled core causes high hydraulic resistance. The higher the inflow pressure head, the faster the vortex rotates, which increases the back pressure and thus the resistance to flow.



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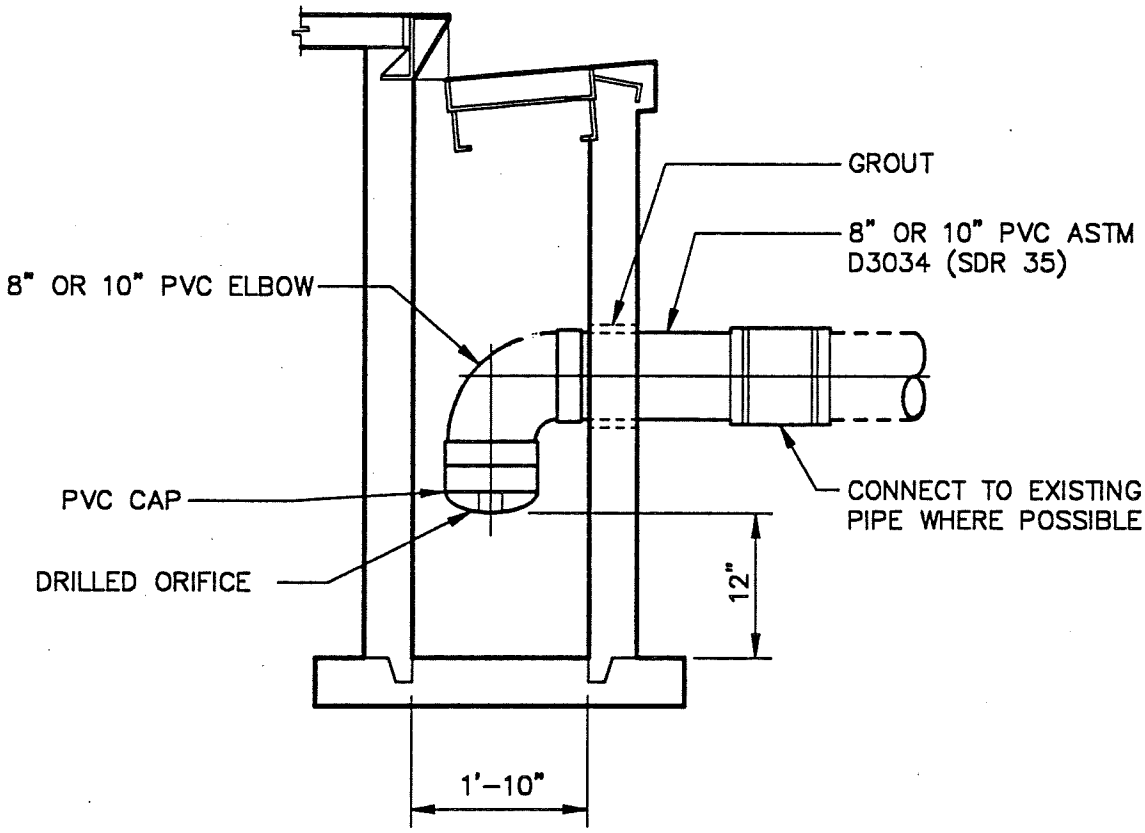
NTS



SECTION

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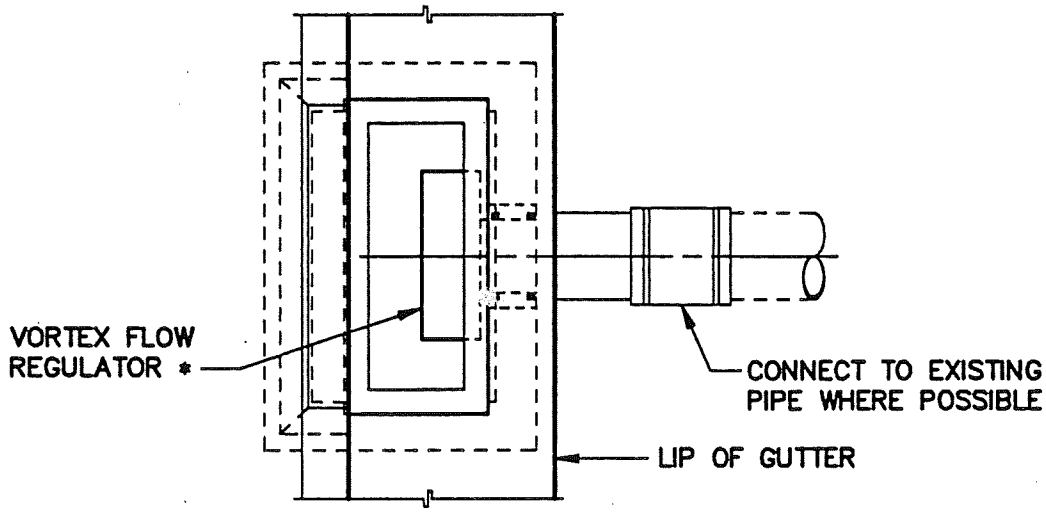
TYPE B DROP INLET WITH HANGING TRAP FLOW REGULATOR

HDR

Figure 9-4



PLAN

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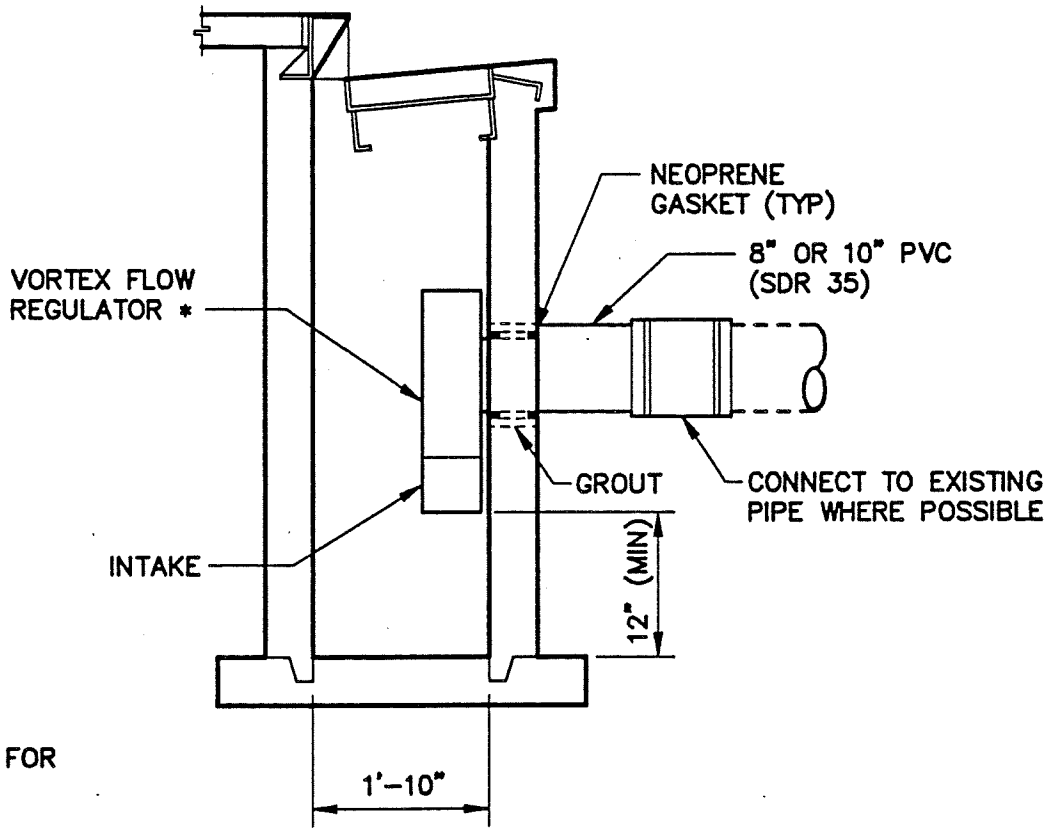


* SEE APPENDIX FOR CATALOG CUTS

SECTION

NTS

TYPE B DROP INLET WITH VORTEX FLOW REGULATOR

HDR

Figure 9-5

Tables 9-1 and 9-2 present data on sizes and flow rates for the two types of flow regulators previously described. As indicated by the table, the vortex type is applicable only to small flow rates.

Both hanging traps and vortex flow regulators are small in size and can be installed in conventional drop inlets, such as the Type B inlets used by the City. Figures 9-4 and 9-5 illustrate typical configurations.

Both types of regulators can also be installed in manhole type catch basins serving two or more inlets, as shown in Figures 9-6 and 9-7. Manholes suitable for installation must be off-line from (as opposed to flow-through) collection system piping. Ideally, manholes are centered in the intersections where water is detained in the streets, with multiple drainage inlet connections to each manhole.

In the City of Sacramento, most of the existing DIs in the combined sewer service area are small gutter drain type inlets, that do not have drop inlets (i.e. with a box structure). Gutter drains are unsuitable for retrofitting with flow regulators. Therefore, for the purposes of this assessment, it was assumed that all the inlets in the combined sewer service area will be replaced. Figure 9-8 shows the details of an installation that includes DI replacement. This configuration is consistent with the City of Sacramento's *Standard Specifications for Public Works Construction* (dated June 1989).

Direct drop inlet installation requires that a flow regulator be installed in each inlet, whereas manhole type installations would require fewer flow regulators but more structures throughout the system. This is because new off-line manholes would need to be installed at intersections where detention is provided. Even for alternatives involving flow regulation at the manhole, it is probable that all DIs would be replaced, since new manholes and connection piping to DIs would be required. Replacement of the connection piping to the existing DIs would probably destroy enough of the existing DIs to warrant replacement.

TABLE 9-1
HANGING TRAP ORIFICE CAPACITY

iameter (in)	Area (sq in)	Discharge Coeff, C	Head, h, (ft)	Capacity, Q, (cfs)	Comments
0.75	0.44	0.598	3.0	0.03	May clog, use Vortex instead.
1.0	0.79	0.598	3.0	0.05	May clog, use Vortex instead.
1.5	1.77	0.598	3.0	0.10	May clog, use Vortex instead.
2.0	3.14	0.598	3.0	0.18	May clog, use Vortex instead.
2.5	4.91	0.598	3.0	0.28	May clog, use Vortex instead.
3.0	7.07	0.599	3.0	0.41	Smallest recommended size
3.5	9.62	0.599	3.0	0.56	
4.0	12.57	0.599	3.0	0.73	
4.5	15.90	0.600	3.0	0.92	
5.0	19.63	0.600	3.0	1.14	
6.0	28.27	0.600	3.0	1.64	
7.0	38.48	0.601	3.0	2.23	
8.0	50.27	0.601	3.0	2.92	
9.0	63.62	0.602	3.0	3.70	
10.0	78.54	0.602	3.0	4.56	Open pipe, no restricting orifice

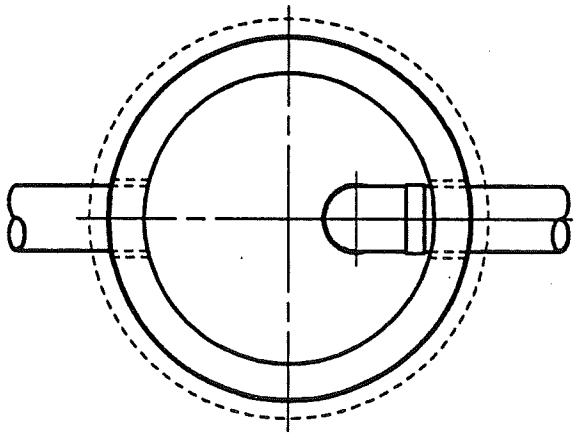
Orifice equation: $Q=(Ca(2gh)^{1/2})/144$

Q=Capacity (cfs)
C=coeff of discharge (interpolated from Kings & Brater, Handbook of Hydraulics,1976 table 4.6. Assumes round, sharp edged orifice)
a=area of orifice (sq. inches)
g=acceleration due to gravity=32.2 ft/s^2
h=head on centerline of orifice (ft)

TABLE 9-2
VORTEX FLOW REGULATOR CAPACITY
(HYDROBRAKE Standard Vortex Valves)

Model No.	Outlet Diameter (in)	Equivalent Hanging Trap Orifice	Capacity, Q (cfs)	Comments
38-S-9	1.5	0.75	0.029	May clog
51-S-9	2.0	1.00	0.053	May clog
64-S-9	2.5	1.75	0.083	May clog
76-S-9	3.0	1.50	0.117	Smallest recommended size
89-S-9	3.5	1.75	0.159	
102-S-9	4.0	2.25	0.211	
114-S-9	4.5	2.50	0.264	

Assume h=3.0 ft



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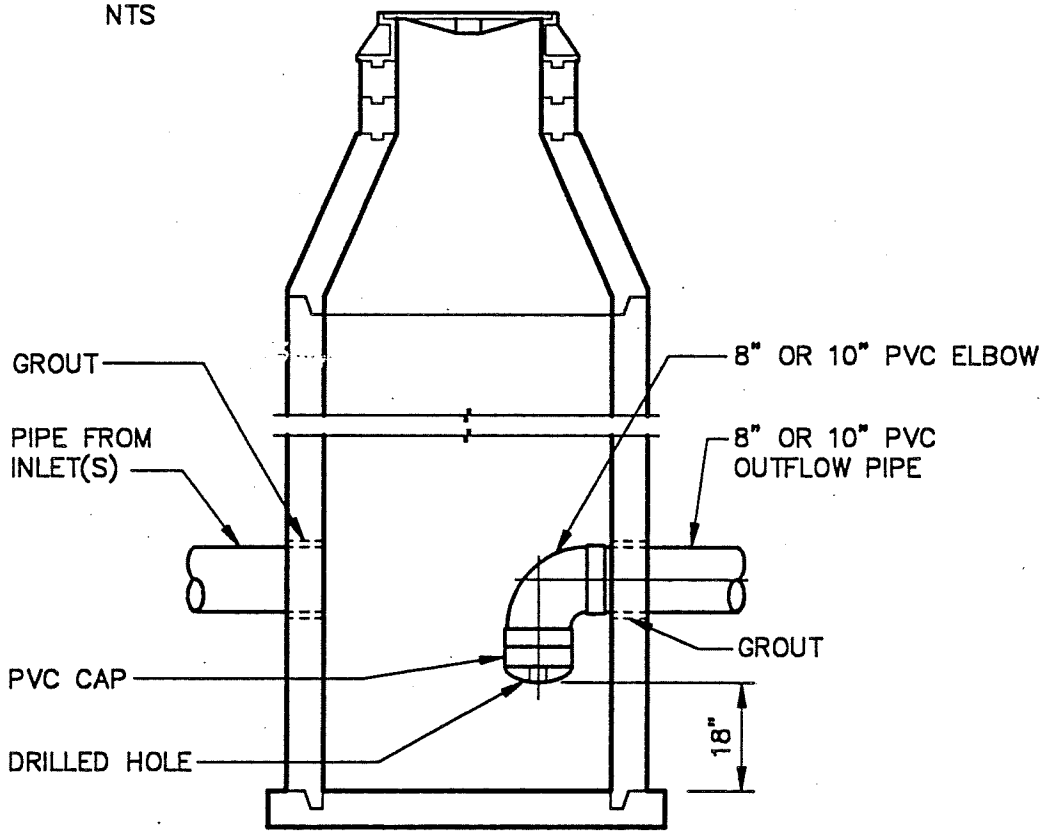

SECTION
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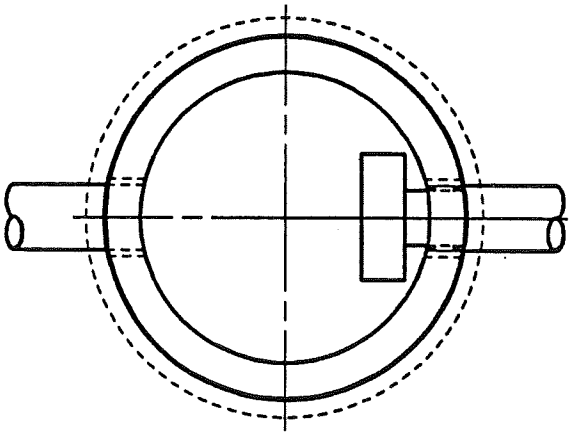
MANHOLE WITH HANGING TRAP FLOW REGULATOR

HDR

Figure 9–6



PLAN

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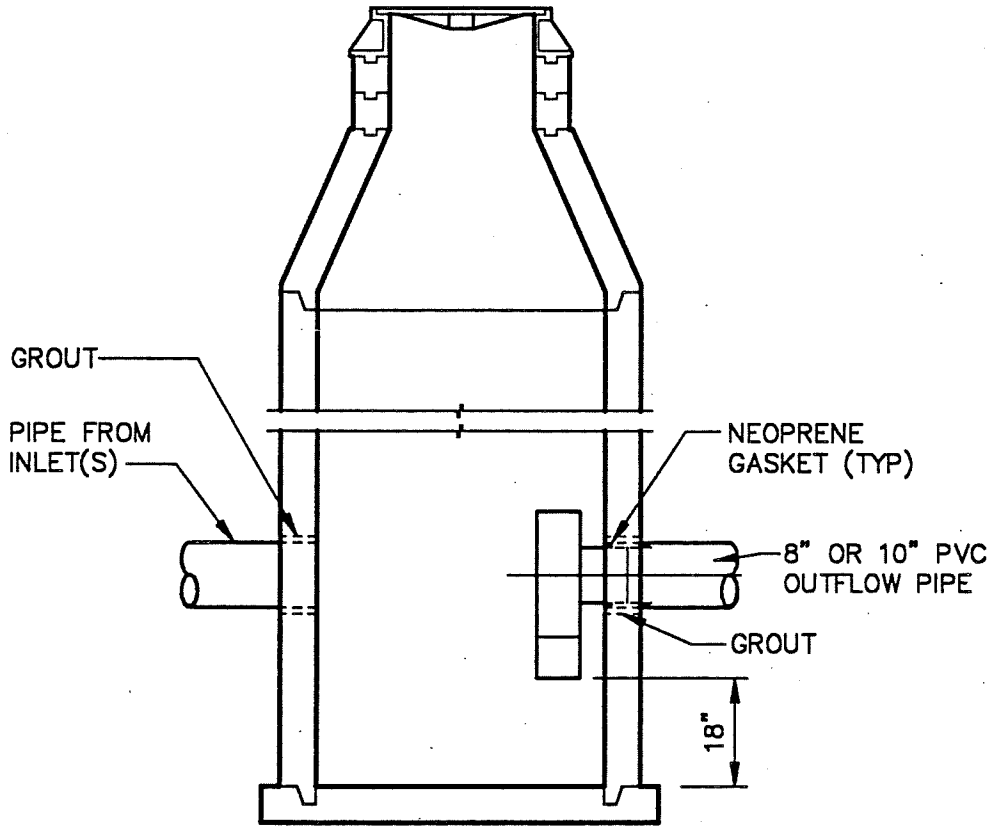

SECTION

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MANHOLE WITH VORTEX FLOW REGULATOR

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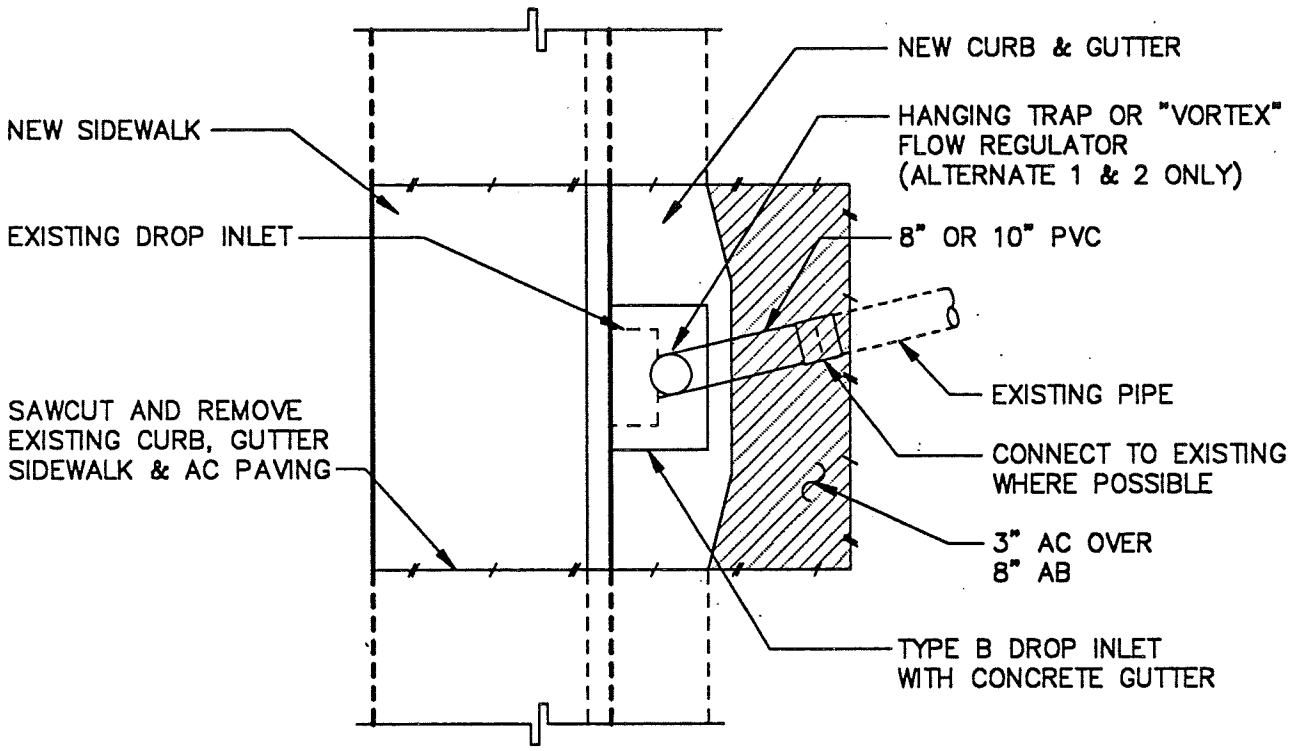
Figure 9-7



PLAN

1/4" = 1'-0"

DROP INLET REPLACEMENT WITH FLOW REGULATOR

HDR

Figure 9-8

C.2. Mitigation of Increased Flooding due to Flow Regulation

The increase in stormwater flooding caused by flow regulation for outflow control would require mitigation if the selected design storm (e.g. a 10-year event) could not be controlled by street detention. Upsizing or paralleling of the existing conveyance system, where necessary, could provide this mitigation by conveying excess peak runoff from the streets. This would eliminate stormwater ponding resulting from occurrence of the design storms. Using the detention basins and street conveyance prior to its entry into the underground collection system, the size and number of additional conveyance improvements could be reduced.

If enough upstream detention and street conveyance could be provided, the existing underground combined sewer system may prove adequate to handle larger storm events than its current, approximately 2-year design storm capacity for peak flow conveyance. In most cases, the pipeline sizes required to convey the excess design storm peak flows (which would be reduced by detention) could be downsized compared to that required without the street detention. Thus, it is possible that savings in sewer improvement costs may result.

It is important to note that an overall plan for controlling flooding as part of this alternative depends on the availability of street storage and conveyance in order to delay stormwater entry into the underground collection system, thereby resulting in cost savings compared to the "conventional" pipeline improvements. The capacity available to store and convey flow in the streets is dependent on street topography, which is highly site-specific. In general, the street grades in the City of Sacramento are quite flat. Detailed topographic information would be required to accurately assess this aspect of alternative feasibility.

Street and intersection detention berms similar to those shown in Figures 9-10, 9-11 and 9-12 could be constructed to detain stormwater at most inlets. With berm cross slopes (street longitudinal slopes) at the street 1/4-crown limited to 2.5 percent, the wide, low berms would offer minimal obstruction to traffic at typical design speeds for crossing a normal city intersection. The flow regulators required for outflow control would control the rate of discharge into the combined system from the numerous mini-detention basins. Figure 9-9

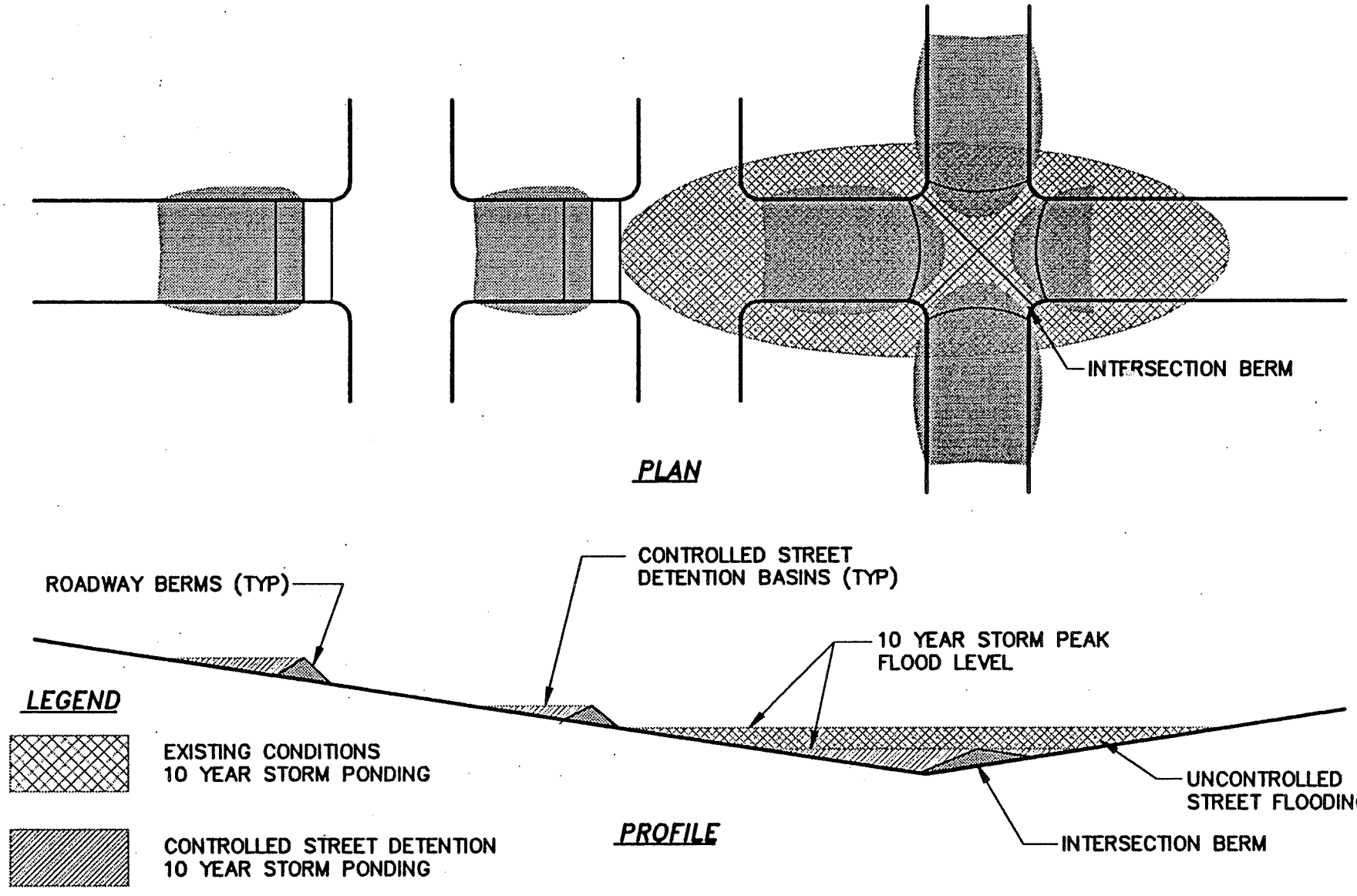
illustrates the difference between controlled and uncontrolled street flooding.

Detention using street berms would be limited to local streets and minor collectors. Arterial streets and major collectors carrying heavy traffic would not be considered for detention ponding. Street detention berms can be designed to limit ponding to the suggested allowable storm runoff encroachment, as identified in Table 9-3.

TABLE 9-3. SUGGESTED ALLOWABLE STORM RUNOFF ENCROACHMENT IN STREETS

Grade	Encroachment
Local	
Continuous Grade	One lane width is open to traffic.
Sag	Maximum depth in any traffic lane is 6".
Collector	
Continuous Grade	One lane width in each direction is open.
Sag	Maximum depth in any traffic lane is 6".
Arterial and Expressway	
Continuous Grade	No encroachment in the traffic lanes.
Sag	

07140-001-141-38
TS-FIG08.DWG
03-03-92 - RAH

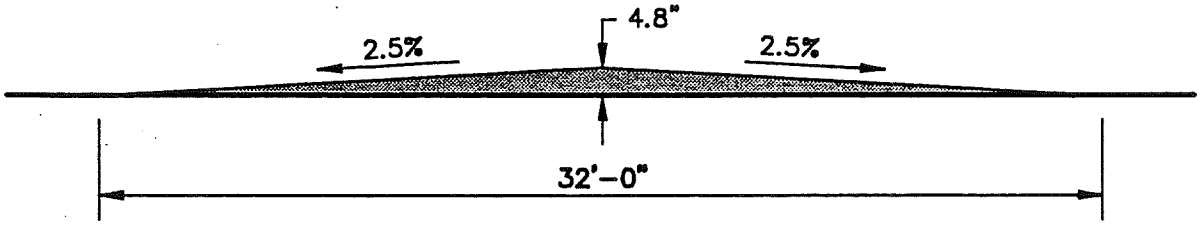


STREET DETENTION VS UNCONTROLLED STREET FLOODING

** IN PART FROM RAASCH, GARY E, URBAN RUNOFF
CONTROL SYSTEM CASE STUDY, SKOKIE, ILLINOIS, 1986

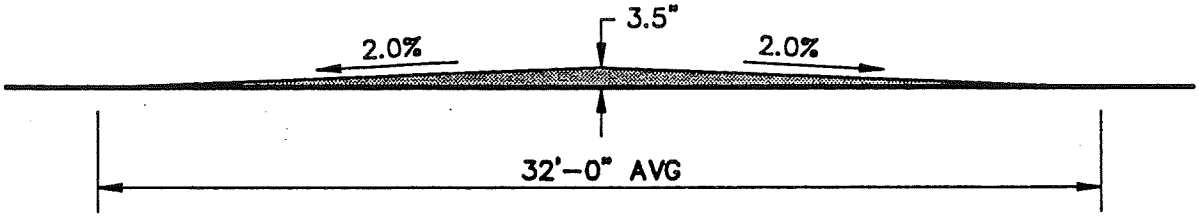
HDR

Figure 9-9

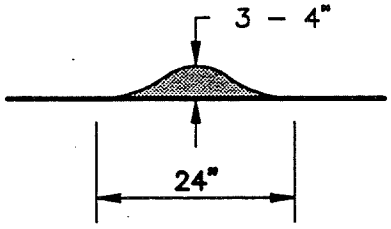


ROADWAY STORMWATER CONTROL BERM *

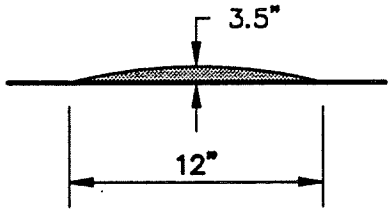
(SEE ALSO Figure 9-11 FOR INTERSECTION BERMS)



NORMAL INTERSECTION CROSS - SLOPE



LOW SPEED CONTROL BERM



UNDULATION-INTERMEDIATE SPEED CONTROL BERM

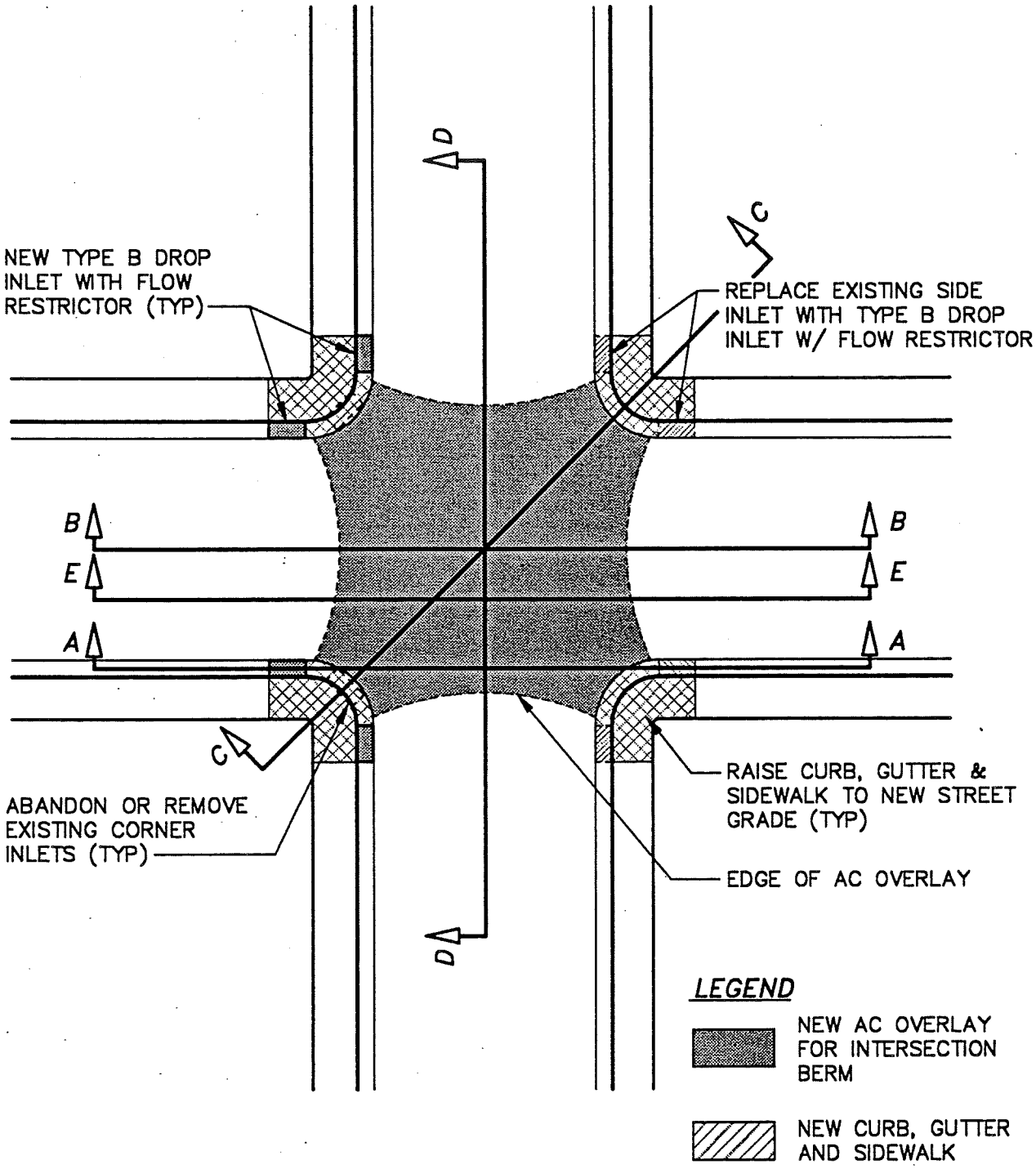
STORMWATER CONTROL BERM COMPARISON **

* CROSS SECTION @ 1/4 CROWN

** IN PART FROM RAASCH, GARY E, URBAN RUNOFF CONTROL SYSTEM CASE STUDY, SKOKIE, ILLINOIS, 1986

HDR

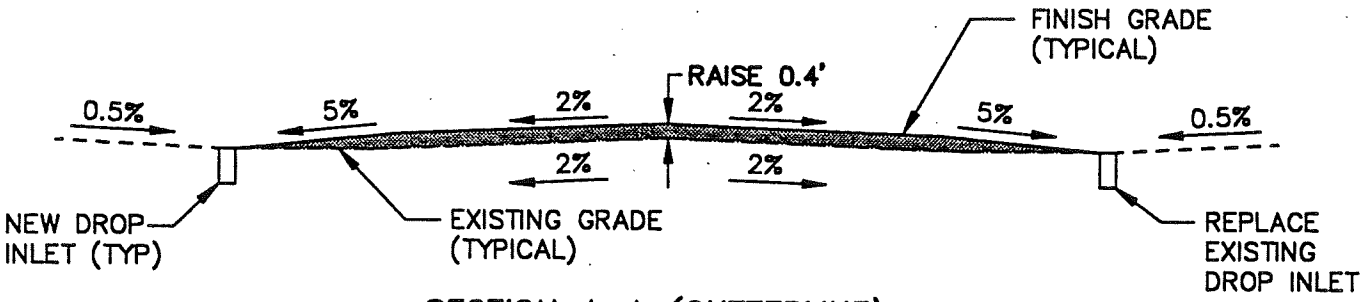
Figure 9-10



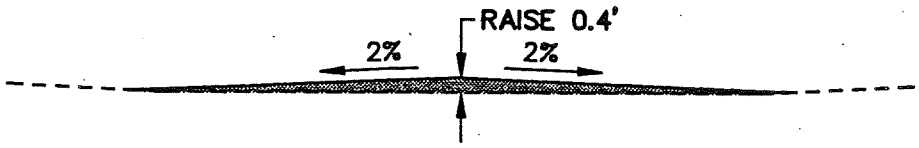
TYPICAL INTERSECTION BERM
FOR LOCAL STREET WITH 50-FT RIGHT-OF-WAY

HDR

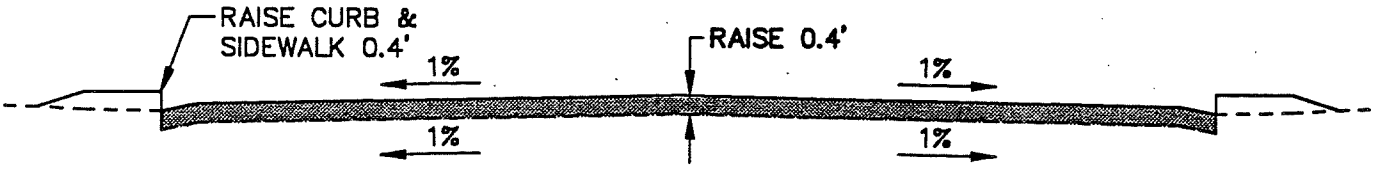
Figure 9-11



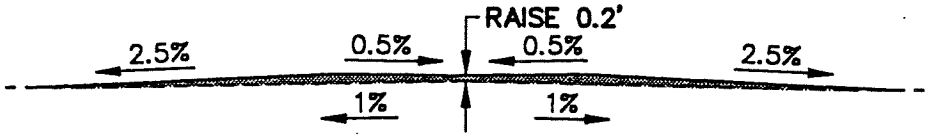
SECTION A–A (GUTTERLINE)



SECTION B–B & D–D (CENTERLINE PROFILES)



SECTION C–C (DIAGONAL CROSS SECTION)



SECTION E–E 1/4 CROWN PROFILE

TYPICAL INTERSECTION BERM CROSS SECTIONS
FOR LOCAL STREET WITH 50–FT RIGHT–OF–WAY

HDR

Figure 9–12

To maximize the collection capacity of the new mini-detention basins, existing downspouts connected to the sewer would need to be converted to surface discharge as shown in Figure 9-2. As can be seen from Figure 9-9, even without collection and conveyance system upsizing, construction of street detention berms will reduce flooding at the lowest lying intersections by detaining some water at higher inlets.

The objective of a street conveyance/detention scheme, for a desired level of outflow and flood protection, is to utilize the existing underground system where possible and to minimize the size and cost of upgrades while keeping ponding and flow in the streets at acceptable levels. With the additional stormwater flooding anticipated because of inflow regulation for outflow control, street conveyance and detention provide potentially cost-effective mitigation for the expected flow increases.

Street detention could potentially provide an improved overall level of flood protection compared to the existing system, especially if coupled with other conveyance improvements. Such a system might be engineered to reduce or eliminate the present frequent flooding of low-lying intersections and make ponding at the numerous new mini detention basins relatively infrequent. Essentially, stormwater ponding would be spread around the service area. Since the very occasional ponding would occur only on residential and minor collector streets and be limited to strict pavement encroachment guidelines, property damage and significant disruption of traffic would be minimized.

Additional evaluation will establish the complete costs of such a system for comparison with the other conveyance and storage alternatives. However, because it is not the conventional solution to combined sewer capacity problems, a strong public relations campaign would be a vital element in securing its successful implementation. In addition, a policy decision would have to be made by the City regarding levels of flooding that are acceptable.

D. SUMMARY OF INFLOW CONTROL ALTERNATIVES

Review of the previous information allowed for development of a set of specific alternatives for inflow control. Three alternatives are discussed and a fourth

comprehensive alternative is proposed for future study. Three of the alternatives include a plan for mitigating the increased flooding caused by inflow regulation through street detention and street conveyance. Costs are presented for one of the alternatives.

Due to the time and informational constraints of the Phase 3 study, cost estimates could only be developed for the flow regulation (at DIs) portion of a complete inflow control alternative, i.e Alternative 1 described below. Flood mitigation/reduction, CSO control and other elements of a complete plan are not included. Descriptions of other alternatives, however, are presented in the following subsections. Development of Alternatives 2, 3, and 4 listed below are the subject of further study.

D.1. Alternative 1 - Inflow Control with DI Regulators

By replacing all the existing inlets with new Type B drop inlets with flow regulators, this first alternative provides inflow control by inflow regulation alone. It does not consider mitigation for the increased flooding such inflow regulation will cause. It includes replacing existing inlets in place and thus utilizes existing piping as much possible. A combination of hanging trap and vortex flow regulators is assumed. To minimize clogging, vortex flow regulators with their larger inlets are proposed for all installations where the equivalent round orifice would be 3 inches or smaller. Hanging traps would be used for larger orifices. In order to provide maximum flexibility, City standard 10-inch pipe would be used to connect to the existing combined sewer system. The storm drain lines to the existing inlets would be used when possible.

D.2. Alternative 2 - Inflow/Flood Control with DI Regulators and Detention

This second alternative would include installing flow regulators in new drop inlets as in Alternative 1. In addition, it would include mitigation of the increased flooding that inflow regulation would cause by utilizing the streets for detention and conveyance of stormwater. This would be accomplished by installing stormwater control berms and intersection berms on minor streets and minor collectors to create numerous mini-detention basins.

As can be seen from Figure 9-13, existing inlets on the apex of the curb returns would need to be replaced with two inlets at the curb return points of tangency. Existing inlets at the points of tangency would be replaced at the same location.

D.3. Alternative 3 - Inflow/Flood Control with Manhole Regulators and Detention

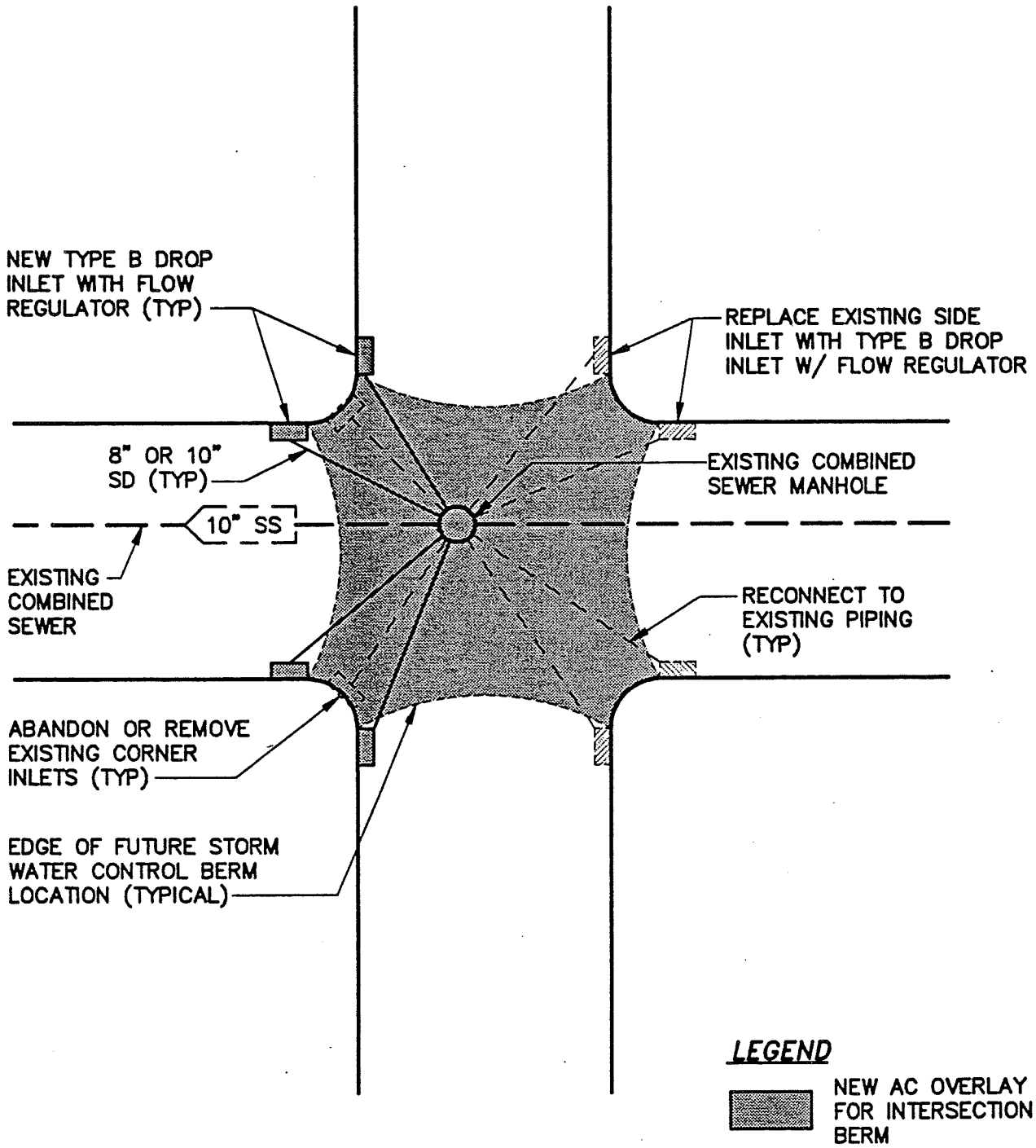
This third alternative is very similar to Alternative 2. However, instead of installing the flow regulators in drop inlets as in Alternatives 1 and 2, two or more drop inlets would feed an off-line manhole in which a single flow regulator is installed. This would greatly reduce the number of flow regulating devices required and has been the preferred method in some cities. Figure 9-14 illustrates the configuration. The drawback is that in addition to the new manhole structure, this alternative would also require new local storm drain lines connecting the drop inlets to the manhole and new piping from it to the existing combined sewer system. This alternative also includes detention basin structures and street conveyance.

D.4. Alternative 4 - Comprehensive Inflow Control Alternative

This comprehensive alternative would include inflow control measures such as those discussed for Alternatives 2 and 3 as well as additional conventional conveyance and storage facilities. This alternative would provide outflow, CSO and flood protection.

E. COST ESTIMATE FOR ALTERNATIVE 1

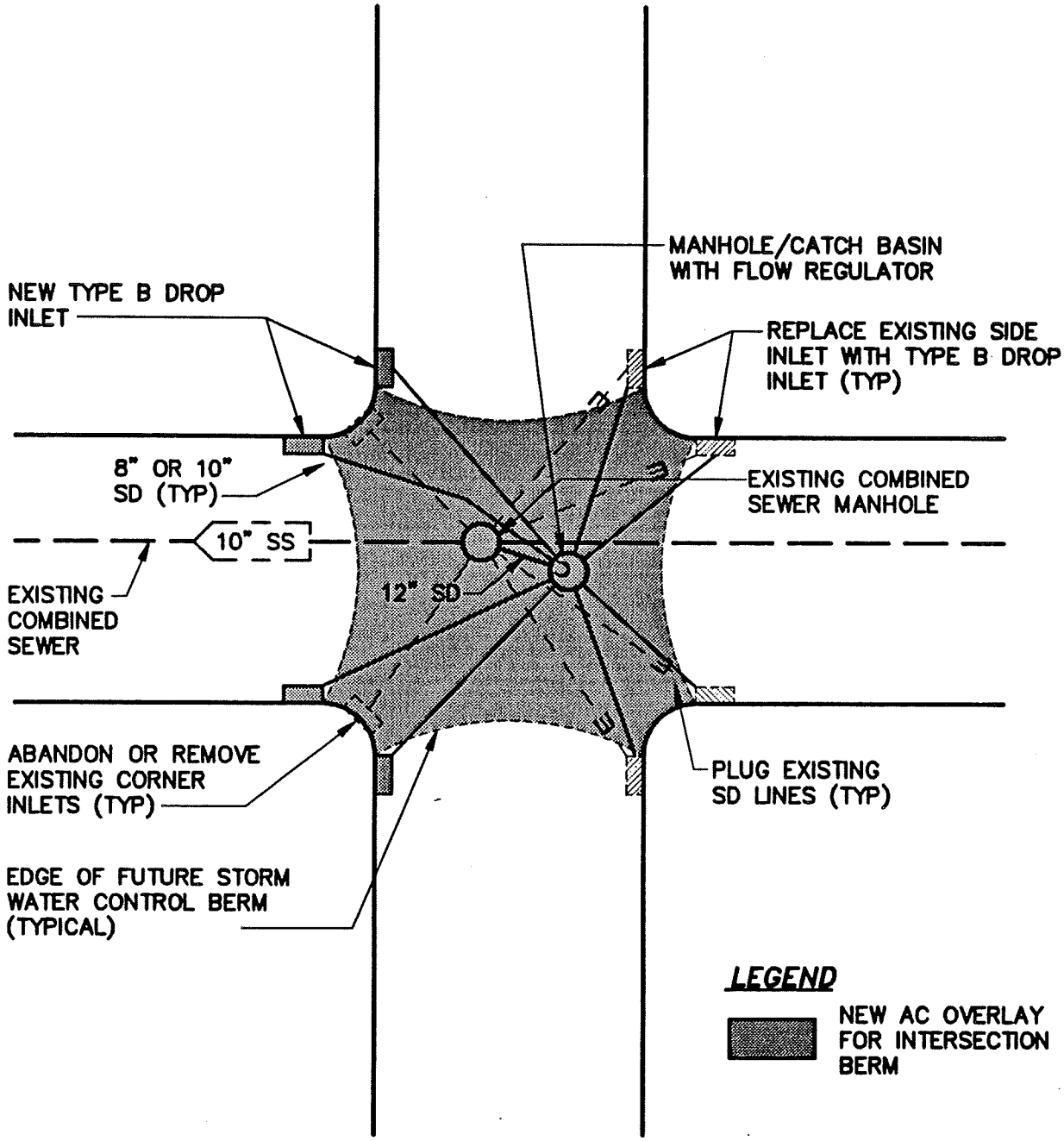
Using the limited information available on the details of the local collection system, a cost estimate was prepared for Alternative 1. The information used in this estimate draws substantially from the Phase 3 technical memorandum *Combined Sewer Collection System Hydraulic Evaluation, Improvements and Costs for the 2-, 5-, 10- and 25-year Storms*, dated July 1, 1992. In general, the procedure uses the six representative basins analyzed in the collection system evaluation report for extrapolation to the entire combined sewer system.



TYPICAL INTERSECTION RETROFIT
REPLACE INLETS WITH TYPE B INLETS WITH FLOW REGULATORS

HDR

Figure 9–13



TYPICAL INTERSECTION RETROFIT – REPLACE INLETS W/ TYPE B INLETS, INSTALL OFF-LINE MANHOLE W/ FLOW REGULATOR

HDR

Figure 9-14

The following is an estimate of the cost of controlling outflow by inflow regulation at the drop inlets without controlling any general stormwater street flooding arising from the proposed inflow regulation. A step-by-step procedure is presented.

First, the number of inlets to be replaced were counted within the six representative basins studied, a total of 221 acres. Then, an estimated number of inlets per acre was calculated as shown in Table 9-4. In Tables 9-5 and 9-6, a detailed estimate was prepared for each type of flow regulating drop inlet installation. Finally, in Table 9-7, an estimate of the number of inlets for the entire combined sewer service area was made by applying the factor from Table 9-4 to the applicable acreage for the combined sewer service area from the collection system technical memorandum. Then, individual installation costs were applied to the estimated number of inlets to give a total estimated cost.

In preparing this estimate the following assumptions were made:

1. Flow regulation of surface runoff alone would be sufficient to control outflow without adding costs to disconnect downspouts tied directly to the combined sewer system, although this would probably be required.

2. Inlet replacements would be configured as shown in Figure 9-6.

3. 50 percent of the flow regulators will be of the hanging trap type and 50 percent will be of the vortex type.

4. Costs are given in March 1992 dollars.

5. This is an approximate budget level estimate only. Costs and quantities may change significantly as further study and design better define the project.

TABLE 9-4. AVERAGE NO. EXISTING DROP INLETS PER ACRE IN REPRESENTATIVE ACRES			
Representative Basin	Area (acres)	No. Existing Inlets	No. Inlets Per Acre
1	36.7	58	1.58
2	34.1	19	0.56
3	43.7	31	0.71
4	33.7	37	1.10
5	44.0	71	1.61
6	28.5	35	1.23
Total	220.7	251	1.14

TABLE 9-5. DROP INLET REPLACEMENT WITH HANGING TRAP FLOW REGULATOR				
Item	Quantity	Unit	Unit Cost	Total Cost
Concrete Curb & Gutter Removal	All Req'd	LS	$50.00	$50
Asphalt Concrete Removal	All Req'd	LS	$60.00	$60
Excavation	11	CY	$6.00	$66
Backfill-Hand Compaction	11	CY	$5.00	$55
Drop Inlet	1	EA	$350.00	$350
10" PVC Pipe	5	LF	$5.00	$25
Caulder Coupler Connection to Existing Pipe	1	EA	$18.00	$18
Pipe Installation (Labor & Equipment)	All Req'd	LS	$850.00	$850
10" PVC Elbow for Hanging Trap	1	EA	$48.00	$48
10" PVC Drilled Cap (Hanging Trap Orifice)	1	EA	$12.00	$12
3.5" Sidewalk Replacement/Curb & Gutter	9	SF	$6.00	$54
Asphalt Base	2	CY	$19.25	$39
Asphalt Paving	0.7	CY	$150.00	$105
Supervision	All Req'd	LS	$175.00	$175
Subtotal				$1,907
Overhead and Profit				$476
Total				$2,383

TABLE 9-6. DROP INLET REPLACEMENT WITH VORTEX FLOW REGULATOR				
Item	Quantity	Unit	Unit Cost	Total Cost
Concrete Curb & Gutter Removal	All Req'd	LS	$50.00	$50
Asphalt Concrete Removal	All Req'd	LS	$60.00	$60
Excavation	11	CY	$6.00	$66
Backfill-Hand Compaction	11	CY	$5.00	$55
Drop Inlet	1	EA	$350.00	$350
10" PVC Pipe	5	LF	$5.00	$25
Caulder Coupler Connection to Existing Pipe	1	EA	$18.00	$18
Pipe Installation (Labor & Equipment)	All Req'd	LS	$850.00	$850
Vortex Flow Regulator	1	EA	$300.00	$300
Vortex Flow Regulator Installation	1	LS	$50.00	$50
3.5" Sidewalk Replacement/Curb & Gutter	9	SF	$6.00	$54
Asphalt Base	2	CY	$19.25	$39
Asphalt Paving	0.7	CY	$150.00	$105
Supervision	All Req'd	LS	$202.00	$202
Subtotal				$2,224
Overhead and Profit				$556
Total				$2,780

TABLE 9-7. ESTIMATED QUANTITIES & COSTS FOR FLOW REGULATOR INSTALLATIONS W/O MITIGATION FOR FLOODING		
Total Applicable Acreage*	5835	
Avg. No. of Inlets/Acre based on representative basins (Table 9-4)	1.14	
Estimated No. of Inlets in Combined Sewer Area	6652	
Estimated No. of Inlets with Flow Regulators Required	Hanging trap @ 50% 3326	Vortex @ 50% 3326
Estimated Cost of New Inlets with Flow Restrictor Installation (Tables 9-5, 6)	$2,383.00	$2,780.00
Subtotal Cost of New Inlets	$7,925,858	$9,246,280
Total Cost of Flow Regulation for Outflow Control, W/O Mitigation of Increased Flooding	$17,172,138	

* Average taken from Revised Task 5, Draft TM Combined Sewer Collection System Hydraulic Evaluation, Improvements and Costs for the 2-, 5-, 10-, and 25-Year Storms

F. COST SUMMARY AND RECOMMENDATIONS FOR FURTHER STUDY

F.1. Cost Summary

Outflow control can be achieved by installing flow regulation devices in new type B drop inlets throughout the combined sewer service area, as outlined in inflow control Alternative 1. The cost of these installations is estimated at $17,200,000 for the entire combined sewer service area if mitigation of increased flooding caused by flow regulation is disregarded.

F.2. Recommendations for Further Study

To completely study alternatives incorporating outflow control and other design criteria to the same degree as the other combined sewer improvement program alternatives (described in Sections 5 through 8), both topographic survey work and SSWMM91 analyses would be required. Two possible approaches are reviewed here for further development and feasibility screening of this type of alternative. The first would be to complete a detailed study of the six representative basins analyzed in the collection system evaluation, and extrapolate the results to the entire combined sewer service area. The second would be to collect detailed field and background information on the entire service area (i.e. topographic and street details), and then prepare a comprehensive assessment. The SSWMM91 (with certain source code modifications) would be used in both approaches.

F.2.a. Approach 1 - SWMM Representative Basin Analysis of Outflow Control-- This approach would use the six representative collection system basins as a "template" for a detailed analysis, with the results extrapolated to the entire system. First, the volume of flooding and the amount of outflow would be estimated for each SWMM basin in the SSWMM91 model for the 2-, 5-, 10- and 25-year design storms by comparing storm runoff volumes under the following scenarios: 1) existing unimproved combined sewer system capacity; and 2) with the enhanced capacities of the other (e.g. tunnel and storage) alternatives. This would provide information for extrapolating from the representative basins to the entire system and for cost comparisons.

The drainage subbasins to each drop inlet in the six representative basins would be surveyed and mapped in detail. Using the SSWMM91, the depth and area of flooding for the representative SWMM basins at each drop inlet would be determined for: 1) the existing condition; 2) with inflow regulation for outflow control alone; and 3) with street detention and street conveyance mitigation. These results would then be extrapolated to the entire combined sewer system and used to generate inputs for the system-wide SSWMM91 model. The system-wide SSWMM91 model would then be used to determine approximate excess flow rates and volumes above system capacity for the entire combined sewer system for the conditions 2 and 3 above.

For the six basins, cost estimates would be developed for flow regulation and for modifying the streets for detention and conveyance to mitigate the increased flooding arising from flow regulation. These study area costs would be extrapolated to the entire combined sewer system.

F.2.b. Comprehensive Improvement Program Using Outflow Control--
The analysis of this alternative must really examine the combined system in its entirety, and focus on the main low-lying and problem areas in the system, rather than on basins that were selected as representative for an entirely different purpose. The six basins are representative of the collection system layout, structure, type, and age rather than being representative of either low-lying areas where outflows and flooding are likely to occur or areas defined by available surface storage and street conveyance capacity. In addition, while the SSWMM91 provides information on the amount of flooding at modeled junctions in the system, it does not identify the exact location, extent, or depth of flooding. A more accurate view of flooding depths and locations could be developed by preparing accurate topographic information and then imposing the flooding information generated by SWMM onto a topography/street map. Developing this information is crucial for obtaining accurate results on which a feasibility judgment could be based.

The detailed steps in this comprehensive analysis would be as follows:

- Develop accurate topographic information for the entire combined system area.

- Delineate regional basins contributing to major sump locations and major interceptors.

- Generate design storm runoff hydrographs for all basins.

- Determine required volume of ponding in streets and other possible surface locations by subtracting the capacity of sewer system laterals, mains, and interceptors from the runoff hydrograph.

- Determine allowable ponding depths and choose street berm locations.

- Calculate surface storage volume from berm locations and topographic survey data; adjust numbers and locations of berms and storage area until required ponding volume is accounted for.

The above analysis would determine whether sufficient street storage exists within designated allowable ponding depths, and the number of structures needed to contain the excess storm volumes. A reliable total cost for each design storm could be calculated based on street and inlet modifications. It is assumed that all inlets within the combined system would be modified and the total number of inlets would need to be verified. The above method would also indicate the storm frequency at which additional storage and conveyance structures would be needed. The design of these additional structures would then require use of the modified SWMM version as discussed in the representative basin analysis to determine sizing and costs.

F.2.c. Comparison of Two Approaches--

The second, more comprehensive approach would yield more accurate results, more accurate costs, and better integration into an overall improvement program for the combined system. The primary reasons for this conclusion are:

- A complete outflow control alternative would include street drainage and conveyance which must consider "intra-basin" stormwater conveyance via the street system. This factor, which has significant cost implications, cannot be considered properly by examining only isolated, representative basins.

- In addition, flooding at one point is often the result of downstream conditions, i.e., in many areas the basins are highly interdependent and not disconnected areas.

- Finally, surface storage availability will vary greatly from basin to basin, as will the risks of flooding. Consequently, significant errors could result by simply extrapolating from six basins to the entire service area.

G. SUMMARY

As a possible alternative to major conveyance system upgrades, outflows can be prevented by installing flow regulators in drop inlets or manhole catch basins. These regulators limit flows into the collection system to the capacity of the system operated in a gravity/open channel flow condition, thereby controlling surcharging and the potential for corresponding outflows.

Based on experience in other cities, hanging trap flow regulators are recommended for higher flows and vortex regulators for lower flows. The cost of installing such flow regulators in new Type B drop inlets, one of the alternatives for controlling outflow, is estimated at approximately $17,200,000 for the entire combined sewer system. This cost does <u>not</u> include mitigation of the increased stormwater flooding that is expected once flow into the system is regulated. Modifying the streets with low, unobstructive berms to allow them to be used together with the flow regulators for controlled detention and conveyance of stormwater, is also not included in this cost estimate.

A combined sewer improvement program incorporating these features may potentially provide desired levels of CSO and outflow control, while allowing controlled flooding, at less cost than the conventional storage and tunnel alternatives. Further study of flow regulation for outflow control and street detention and conveyance is recommended.

SECTION 10
COORDINATION WITH COUNTY INTERCEPTOR PLAN

A. INTRODUCTION

The Sacramento County Regional Sanitation District (District) is currently engaged in a planning effort for the area-wide sewer interceptor system. The objective of the study is to develop a plan for new interceptor sewers which would serve the future needs of the County area. James M. Montgomery Consulting Engineers, Inc. is developing the plan for the District. One option for the development of future County interceptors is to combine the flows from the Natomas Interceptor and portions of the flows from the Dry Creek Interceptor (serving Rio Linda and Gibson Ranch areas) into a new interceptor routed across the American River and then south along Highway 160 to the Union Pacific Railroad right-of-way to the SRWTP.

With this alignment for a new large-diameter interceptor, the possibility of joint use of the interceptor for conveyance of both sanitary flow and combined wastewater was discussed with the District and their engineering consultant. This section presents the preliminary evaluations made to date for joint use of a new County interceptor. It should be noted these initial evaluations are preliminary in nature and do not represent comprehensive evaluations on behalf of the District or its engineering consultant, or by the City or HDR Engineering, Inc.

B. PRELIMINARY EVALUATION OF THE JOINT USE CONCEPT

The first step in evaluating this concept was to prepare a summary of peak combined wastewater flows generated in the vicinity of the 19th Street and Freeport Boulevard corridor for the 2-, 5- and 10-year storm events. This was prepared by HDR and was transmitted to the District's consultant. For the 2-year design storm, the peak flow rate of combined wastewater diverted from the existing conveyance system into consolidation conduits, within the vicinity of the proposed new interceptor, would be approximately 40 mgd. Similarly, for the 5- and 10-year events the peak of the diverted flow would be 720 and 1400 mgd, respectively.

HDR's information regarding peak flows, and the District consultant's response, were discussed at a joint meeting with City staff and District staff. In that meeting, the District's consultant suggested that, based on review of USGS topographic data, the maximum slope available for the proposed interceptor would be 0.0006. At this slope, the maximum pipeline size which would still provide minimum velocities under average dry weather flow conditions would be a 108-inch conduit. Using these limitations, the maximum total conveyance for the pipeline would be 200 mgd. For conveyance of projected design peak dry weather flows, the size of interceptor required for future estimated flows, without combined sewage contributions is either a 90- or 96-inch sewer, depending upon the location along the interceptor and the limits of the County service area. Therefore, construction of a 108-inch interceptor would be oversized relative to the needs of the upstream service area.

Of the 200 mgd conveyed by a 108-inch sewer, the portion contributed by peak wet weather flows for the service area upstream of the City's combined sewer service area is dependent upon the choice of upstream service area boundaries. As of February, 1992, the District and its consultant had defined two development alternatives for the upstream service area. One is the Urban Policy Area or the area defined by the County *General Plan*. The other is the Urban Services Area, which is development out to the limits of the County line. Table 10-1 summarizes the available capacity in the proposed County interceptor for the City's combined wastewater for the two development alternatives. These values assume a 108-inch conduit at a slope of 0.0006 with a peak capacity of 200 mgd.

TABLE 10-1. CAPACITY IN PROPOSED COUNTY INTERCEPTOR

Development Alternative Service Area	Peak Wet Weather Flow From Service Area (mgd)	Capacity Remaining for Combined Wastewater (mgd)
Urban Policy Area	71	129
Urban Services Area	135	65

C. OPTIONS FOR CITY USE OF THE COUNTY INTERCEPTOR

Several options are available to the City for use of the proposed County interceptor. One is to divert combined wastewater directly to the interceptor on peak, without any storage for attenuation of the peak flow. The data in Table 10-1 indicate the available capacity in a 108-inch interceptor would be sufficient to convey the excess peak flows diverted from the existing combined system for only the 2-year design storm event.

Another option would be to transfer combined wastewater to the interceptor off peak. This option would only be feasible when coupled with alternative improvement plans that include either a storage reservoir or tunnel system. In this case, oversizing the proposed County interceptor would probably not be necessary, since stored combined wastewater could be pumped into the interceptor after peak storm flows had passed .

D. SEQUENTIAL STORM ANALYSIS AND THE PROPOSED COUNTY INTERCEPTOR

The most feasible option for coordinating transfer of combined wastewater to SRWTP appears to be in conjunction with storage-only or tunnel alternatives. The benefit to the City's improved system would be in reducing emptying times for these facilities. However, an additional pumping station would be required to lift wastewater from the storage or tunnel system into the new County interceptor. A benefit would only be realized if a long-duration storm analysis revealed that use of only the City Interceptor (through Sump 2 and the Flow Control Structure) would be inadequate for achieving the necessary emptying rates. Then the cost of upsizing new storage facilities could be weighed against the cost of adding another pumping station to use a new County interceptor.

A long-duration storm analysis has not been completed for the least-cost alternatives identified in this technical memorandum. This sequential storm analysis would evaluate the impact of multiple storms and inter-storm periods on the response of the least-cost alternative improvement plans identified in this study. Of particular interest would be the ability of any tunnel or storage-only alternatives to empty fast enough to provide flood protection and CSO control for

subsequent storm events. The wastewater stored in these facilities must receive secondary treatment, and the available allotment to the City for conveyance to SRWTP is fixed at a maximum rate of 60 mgd. Consequently the sequential storm analysis would reveal the need to either make the required storage or tunnel facility larger or provide a new conveyance system to SRWTP. In this situation, use of the proposed County interceptor for conveyance of the City's combined wastewater may provide some cost savings for the tunnel and storage alternatives.

Further study of the benefits available to the City through use of the County interceptor system is warranted. This is best accomplished through the use of a long-duration storm analysis. As discussed in Appendix I of Phase 3 entitled *Approach to Long Duration Storm Development and Use for the Phase 3 Analysis*, the SSWMM91 model source code was not able to simulate multiple storms. Consequently, to conduct the analysis of benefits available through use of the County interceptor, the model source code will first have to be modified.

SECTION 11
TOTAL PROJECT COSTS FOR THE ALTERNATIVE IMPROVEMENT PLANS

A. INTRODUCTION

The efforts described in the previous sections were focused on identifying alternative improvement plans for the CWCS that would control flooding (and therefore outflows) and CSOs. These improvement plans incorporate tunnels, conveyance improvements, storage facilities, treatment plants, sewer separation and/or inflow control methods as a means to achieve the flood and CSO control objectives. With the exception of the separated sewer alternative, the localized flooding improvement alternative and the inflow control alternative, the remainder of the improvement plans satisfy both the flood and CSO control criteria described in Section 3. These criteria control flooding for the 2-, 5- and 10-year design storm events (and the 25-year event for tunnels) and provide CSO control for 1/year and 5/year design storm frequencies.

Summaries of total project costs are presented in this section for these alternative improvement plans. The least-cost improvement plans for the 2-, 5- and 10-year events are identified in Section 12.

B. DESCRIPTION OF ALTERNATIVES

For the tunnel, storage and treatment alternatives analyses, there were several assumptions regarding the operation of the existing CWCS which were common to all alternatives considered. First, the existing collection system was assumed to drain to Sumps 1A and 2 as it presently does. The pumping capacity of existing Sumps 1A and 2 were assumed to remain unchanged. Upgrades to the existing conveyance system with either weirs or replacement of existing sewers with larger sewers were configured to avoid the occurrence of flooding without increasing the design pumping capacity at either Sumps 1A or 2.

Second, the operation of Pioneer Reservoir and Pioneer Interceptor was such that all flow into Sump 2 which could not be conveyed to either the SRWTP or CWTP would be delivered to Pioneer Reservoir through the Pioneer Interceptor. This

method of operating the existing CWCS facilities maximizes the use of Pioneer Reservoir. Under this scenario, CSOs would occur first at Pioneer Reservoir before they would at the Sump 2 Flow Control Structure.

Third, the operation of the CWTP was modeled to ensure that it operates at peak flow capacity prior to overflow occurring at either the Sump 2 outfall or at Pioneer Reservoir. Once again, this assumed operating condition made maximum use of the existing system's capabilities.

B.1. Tunnel Alternatives

For the tunnel alternatives, the configurations recommended for cost analysis reviewed in Section 5 are summarized in Table 11-1.

TABLE 11-1. TUNNEL ALTERNATIVES FOR WHICH ESTIMATES OF TOTAL PROJECT COSTS WERE PREPARED

Flood Control Level	Alternative Configuration			
	1/Year CSO		5/Year CSO	
	Tunnel Diameter (feet)	Overflow Control Elevation (feet MSL)	Tunnel Diameter (feet)	Overflow Control Elevation (feet MSL)
5-Year Tunnel	15	-18.00	14	-18.00
10-Year Tunnel	17	-23.00	17	< -24.00
25-Year Tunnel	-	-	20	-18.00

As explained in Section 5, these configurations were the result of hydraulic analyses to identify the least-cost configuration which would still meet the design storm control criteria. Tunnels were initially examined for their ability to provide flood protection. An optimization process was undertaken to compare various combinations of tunnel storage volumes and outflow pumping rates to identify the least-cost configuration for the tunnels which would still satisfy the flood control design criteria. Once this was established, these configurations were checked for their ability to control the CSO design storms.

CSO flows spilling at Pioneer Reservoir were diverted back into the tunnels for storage. The 10-year and 25-year tunnel configurations were capable of controlling both the 5/year and the 1/year CSO storm events. For the 5-year tunnels, the size of the tunnels had to be increased to provide more storage to control the 1/year CSO storm.

B.2. Storage and Treatment Alternatives

The storage and treatment alternatives were analyzed on the same premise as the tunnels. Provisions for pumping directly to the Sacramento River from either the Pioneer Reservoir site or the Western Pacific site were determined to be a cost effective means of flood control. The UCD site was considered too far from the river to make pumping to the river practical. The required storage or treatment facilities were sized as the minimum necessary for CSO control.

The storage and treatment alternatives for which total project cost estimates were prepared are summarized in Tables 11-2 and 11-3. As seen in Table 11-2, the storage and treatment alternatives for the 1/year CSO design storm control included storage and treatment facilities for the Western Pacific site and treatment facilities for the Pioneer Reservoir site. New pumping stations are required for the Western Pacific site alternatives using storage-only, or storage and treatment, with the latter alternatives requiring pumping of treated effluent to the river. New pumping stations are also required for the Pioneer Reservoir site to deliver wastewater to the primary treatment facilities there for the 5- and 10-year events. For the 2-year event, wastewater is delivered to the new primary treatment facility at the Pioneer Reservoir site by the Pioneer Interceptor and Sump 1A.

As with the tunnel alternative, the application of the 1/year CSO storm event to one of the storage alternatives (UCDP) required upsizing to a larger storage volume for the CSO event than was required to store only the 5-year design storm.

For the 5/year CSO design storm control, the storage and treatment alternatives all become conveyance alternatives, since this design storm did not produce a spill from the existing system. This is illustrated in Table 11-3.

TABLE 11–2
STORAGE AND TREATMENT ALTERNATIVES FOR WHICH
ESTIMATES OF TOTAL PROJECT COST WERE PREPARED
1/YEAR CSO CONTROL

ALTERNATIVE	WESTERN PACIFIC RAILROAD SITE				PIONEER RESERVOIR SITE				UC DAVIS SITE	
	NEW INFLUENT PUMP STA (mgd)	NEW STORAGE RES'VOIR (MG)	NEW TREATMENT FACILITY (mgd)	NEW EFFLUENT PUMP STA (mgd)	NEW INFLUENT PUMP STA (mgd)	NEW STORAGE RES'VOIR (MG)	NEW TREATMENT FACILITY (mgd)	NEW EFFLUENT PUMP STA (mgd)	NEW INFLUENT PUMP STA (mgd)	NEW STORAGE RES'VOIR (MG)
10–YEAR										
– WPRR	1550	60	–	–	–	–	–	–	–	–
– WPP	640	2.8	50	50	890	–	475	–	–	–
– WPUCD	1210	60	–	–	–	–	–	–	370	32
– UCDP	–	–	–	–	890	–	475	–	590	63
– WPUCDP	310	2.2	50	50	890	–	475	–	370	32
5–YEAR										
– WPRR	760	32.2	75	75	–	–	–	–	–	–
– WPP	320	8.5	25	25	440	–	485	–	–	–
– WPUCD	590	32.2	75	75	–	–	–	–	190	11
– UCDP	–	–	–	–	440	–	485	–	310	24
– WPUCDP	170	3.9	25	25	440	–	485	–	190	11
2–YEAR										
– WPRR	535	38.4	75	75	–	–	–	–	–	–
– WPRR & PIONEER	50	1.4	25	25	–	–	500	–	–	–
– UPSIZED CWCS WITH PIONEER TREATMENT	–	–	–	–	–	–	500	–	–	–
– UPSIZED CWCS WITH WPRR TRMT/STORAGE	500	41.6	50	50	–	–	–	–	–	–

TABLE 11–3
CONVEYANCE ALTERNATIVES FOR WHICH
ESTIMATES OF TOTAL PROJECT COST WERE PREPARED
5/YEAR CSO CONTROL

ALTERNATIVE	WESTERN PACIFIC RAILROAD SITE				PIONEER RESERVOIR SITE				UC DAVIS SITE	
	NEW INFLUENT PUMP STA (mgd)	NEW STORAGE RES'VOIR (MG)	NEW TREATMENT FACILITY (mgd)	NEW EFFLUENT PUMP STA (mgd)	NEW INFLUENT PUMP STA (mgd)	NEW STORAGE RES'VOIR (MG)	NEW TREATMENT FACILITY (mgd)	NEW EFFLUENT PUMP STA (mgd)	NEW INFLUENT PUMP STA (mgd)	NEW STORAGE RES'VOIR (MG)
10–YEAR										
– WPRR	1550	–	–	–	–	–	–	–	–	–
– WPP	640	–	–	–	890	–	–	–	–	–
– WPUCD	1210	–	–	–	–	–	–	–	370	32
– UCDP	–	–	–	–	890	–	–	–	590	63
– WPUCDP	310	–	–	–	890	–	–	–	370	32
5–YEAR										
– WPRR	760	–	–	–	–	–	–	–	–	–
– WPP	320	–	–	–	440	–	–	–	–	–
– WPUCD	590	–	–	–	–	–	–	–	190	11
– UCDP	–	–	–	–	440	–	–	–	310	24
– WPUCDP	170	–	–	–	440	–	–	–	190	11
2–YEAR										
– WPRR	40	–	–	–	–	–	–	–	–	–
– WPP	40	–	–	–	–	–	–	–	–	–
– UPSIZED CWCS	–	–	–	–	–	–	–	–	–	–

There are no new storage or treatment facilities required for CSO control level of 5/year. All improvements satisfying the 5/year CSO design storm control criteria therefore centered on flood control, with excess storm flows discharged to the river without treatment or storage, and use of the existing CWCS maximized.

It also should be noted that the 2-year alternative involving upsizing the existing collection system and tapping the Donner Trunk at Western Pacific becomes impractical. The upsized system would convey all wastewater needed for flood control to Sumps 1A and 2 where it would spill into the river. Therefore, this alternative has been eliminated from Table 11-3.

B.3. Separation Alternatives

There were two separation alternatives considered. One was to provide a new sanitary sewer system for the entire combined sewer service area. The other was to provide a new storm drainage system for the combined sewer service area with capacity to provide flood protection for the 10-year design storm. The new storm sewer alternative assumes all stormwater is routed to either Sump 1A or Sump 2. New stormwater pump stations are provided at each of these sites. For the new sanitary sewer alternative, all flow is routed to Sump 2 and a new sanitary pumping station is provided for conveyance to SRWTP.

C. ESTIMATED TOTAL PROJECT COSTS

The detailed estimates of total project costs for the tunnel, storage/treatment, conveyance and separation alternatives are presented in detail in Appendix C. A summary of these total project costs is presented in Table 11-4.

C.1. Construction Costs

The estimates of total project cost comprise both construction and non-construction costs but exclude operation and maintenance costs. The construction cost components were calculated using the unit costs presented in Section 4. All new sewer and tunnel costs were estimated by adding the total lineal footage of conduit or tunnel of the same diameter and applying the unit cost to that total.

TABLE 11-4

SUMMARY OF ALTERNATIVE IMPROVEMENT PLAN ESTIMATED COSTS ($ MILLION) (4)

ALTERNATIVE DESCRIPTION	OTHER	DESIGN STORM CRITERIA						
		2-YEAR FLOOD CONTROL		5-YEAR FLOOD CONTROL		10-YEAR FLOOD CONTROL		25-YEAR (1)
		1/YR CSO	5/YR CSO	1/YR CSO	5/YR CSO	1/YR CSO	5/YR CSO	FLOOD CONTROL
TUNNEL ALTERNATIVES:								
TUNNEL SYSTEM				$528	$525	$736	$734	$976
STORAGE ONLY ALTERNATIVES:								
WPRR		$416				$850		
WPRR AND PIONEER		$335						
WPRR AND UCD						$911		
TREATMENT ONLY ALTERNATIVES:								
UPSIZED CWCS WITH TREATMENT AT PIONEER (6)		$300						
STORAGE/TREATMENT ALTERNATIVES:								
WPRR		$416		$647				
WPRR AND PIONEER		$335		$616		$827		
WPRR AND UCD				$672				
UCD AND PIONEER				$653		$886		
WPRR, UCD AND PIONEER				$668		$863		
UPSIZED CWCS WITH STORE/TREAT AT WPRR (6)		$418						
CONVEYANCE-ONLY ALTERNATIVES:								
WPRR			$270		$573		$760	
WPRR AND PIONEER			$270		$551		$757	
UPSIZED CWCS CONVEYANCE SYSTEM (6)			$250					
CONVEYANCE/STORAGE ALTERNATIVES:								
WPRR AND UCD					$657		$862	
UCD AND PIONEER					$663		$896	
WPRR, UCD AND PIONEER					$652		$865	
SEPARATION ALTERNATIVES:								
NEW STORM DRAIN SYSTEM (2)	$936							
NEW SANITARY SEWER SYSTEM (3)	$293							
BASELINE REHABILITATION:	(7) $88							

BOLD SHADING INDICATES LEAST-COST PLAN.

NOTES:

(1) THE 25-YEAR TUNNEL ALTERNATIVE ALSO CONTROLS THE 1/YEAR AND 5/YEAR CSO DESIGN STORMS.

(2) THE NEW STORM DRAINAGE SYSTEM IS SIZED FOR CONTROLLING THE 10-YEAR FLOOD CONTROL DESIGN STORM.

(3) THIS ALTERNATIVE PROVIDES MINIMAL FLOOD CONTROL BEYOND THE CAPACITY OF THE EXISTING COMBINED SYSTEM.

(4) COSTS FOR REHABILITATION OF THE EXISTING CONVEYANCE SYSTEM ARE NOT INCLUDED IN THIS TABLE, EXCEPT FOR BASELINE REHABILITATION PROJECT.

(5) THESE ALTERNATIVES DO NOT INCLUDE ANY NEW STORAGE OR TREATMENT FACILITIES FOR FLOOD OR CSO CONTROL IMPROVEMENTS.

(6) THESE ALTERNATIVES USE UPSIZING OF EXISTING PIPES RATHER THAN A CONSOLIDATION CONDUIT NETWORK TO ACHIEVE FLOOD CONTROL CRITERIA.

(7) THIS PROJECT INCLUDES REHABILITATION WORK SPECIFICALLY RELATED TO THE SEPARATE SANITARY SEWER ALTERNATIVE.

The costs for diversion weirs, pumping stations, and storage reservoirs were estimated from the unit cost data. Unit cost information for the combination storage/treatment facilities illustrated in Section 7, and associated sludge pumping was taken from the Phase 2 unit cost data for primary treatment. Other construction cost items included in the total project costs were miscellaneous pipe connections and crossings at the major pumping and storage or treatment sites, outfall structures to the river, and structures to connect Pioneer Reservoir to the new consolidation conduits or treatment facilities at that site.

C.2. Collection System Upgrade Costs

Costs for upgrade of the existing collection system were taken from the Phase 3 technical memorandum entitled *Combined Sewer Collection System Hydraulic Evaluation, Improvements and Costs for the 2-, 5-, 10- and 25-Year Storms.* These costs are summarized in Table 11-5.

TABLE 11-5. CONSTRUCTION COSTS FOR THE COLLECTION SYSTEM UPGRADE

Design Storm	Construction Cost
2-year	$150,650,000
5-year	$181,080,000
10-year	$195,580,000
25-year	$216,830,000

C.3. Rehabilitation Costs

In addition to the collection system upgrade costs identified in Table 11-5, other cost items common to all alternatives included: 1) the rehabilitation of Sumps 1, 1A and 2; 2) Pioneer Reservoir improvements; 3) rehabilitation of the collection system, laterals and mains; and 4) acquisition of equipment for inspection and monitoring of the CWCS.

The costs for rehabilitation of Sumps 1, 1A and 2 and the Pioneer Reservoir improvements were taken from the Phase 2 study. Estimates of rehabilitation costs for the existing laterals and mains and collection system to remain in

service for the 2-, 5-, 10- and 25-year alternatives and for purchase of the inspection and monitoring equipment were extracted from Appendix K of the *Phase 2 Detailed Technical Report*. These costs are summarized in Table 11-6.

TABLE 11-6. REHABILITATION COSTS

Item	2-year	5-year	10-year	25-year
Sumps 1, 1A and 2	$20,975,000	$20,975,000	$20,975,000	$20,975,000
Pioneer Reservoir	$762,600	$762,600	$762,600	$762,600
Rehabilitation of Collection System, Laterals and Mains (1)	$18,477,190	$16,652,000	$16,187,000	$15,216,000
Inspection and Monitoring Equipment	$1,500,000	$1,500,000	$1,500,000	$1,500,000
Total	$41,714,790	$39,889,600	$39,424,600	$38,453,600

1 Includes Manhole Repair and Rehabilitation

C.4. Non-Construction Costs

Non-construction costs included with each of the improvement plan total cost estimates were estimated as a percentage of the construction cost subtotal. These included a construction contingency estimated at 25 percent, project engineering design at 6 percent, construction inspection at 10 percent, administration and legal at 3 percent and bonds, insurance and miscellaneous costs at 5 percent of the construction cost subtotal.

C.5. Localized Flooding Improvement Alternative Costs

Table 11-7 summarizes the major elements included in this alternative and the estimated cost in 1992 dollars. The following elements were not included in the cost estimate: discharge pumping stations, odor control, flushwater systems, baffling and control systems for the storage facilities, and mitigation for residential neighborhoods where the facilities will be located.

TABLE 11-7. SUMMARY COST ESTIMATE FOR LOCALIZED FLOODING IMPROVEMENT[1]	
Item	**Cost ($ thousands)**
Conveyance and Storage Flood Control Improvements for Six Problem Areas[3]:	
42nd Street and UCD Medical Center	36,000
Alhambra Boulevard Corridor	26,000
Land Park Drive and Larkin	24,000
Downtown	46,200
Land Park Drive and 11th Avenue	20,500
18th Street and Broadway	19,300
Pumping Station, Pioneer Reservoir, Conveyance System Rehabilitation and Reliability Improvements	51,740
Subtotal Construction Cost	$ 223,740
Total Cost[2]	$ 333,373

1 Prepared by City Utilities Department.
2 Includes contingencies, design, inspection, administration, legal, bonds, insurance, and miscellaneous.
3 Does not include discharge pumping, odor control, flushwater, controls, etc.

D. RECOMMENDATIONS FOR FURTHER STUDY

The estimated total project costs identified in Table 11-4 are construction costs. No operation and maintenance costs are included in these costs. For the tunnel systems, the cost of tunnel maintenance and tunnel pumping station operations could be significant. These have not been addressed in this study and should be considered prior to adopting a final improvement plan.

These estimates also do not include any costs for major environmental clean up of soils contaminated by hazardous materials. The contingency included in the project costs is not intended to cover this type of project cost. Predesign efforts should include an environmental audit of the study area to identify possible problem areas.

Also it should be noted that these projects have not yet been analyzed with the long-duration storm analysis. It is possible that the tunnels or storage

facilities will not empty quickly enough to handle the occurrence of back-to-back storms. All existing and new facilities were configured in this study to empty to the existing conveyance system, through Sump 2 and Flow Control Structure to the SRWTP. This assumes the imposition of the current 60-mgd capacity allowance in the City Interceptor for combined system flows. Evaluating the issue of emptying times and the occurrence of sequential storms is the next step in the analysis.

In addition, there has been no in-depth analysis considering the use of possible future County interceptors for conveyance of combined wastewater to the SRWTP. This could have an impact on the overall cost of combined system plans that include storage facilities, especially if the long-duration storm analysis results in facility upsizing to handle long-duration storms. These issues should be reviewed prior to final selection of an improvement plan.

Finally, as noted earlier, further investigation of the inflow control alternative is also recommended. A program consisting of topographic surveys and SWMM modeling, beyond the scope of this Phase 3 study, could provide data detailed enough for accurate judgements regarding the feasibility and cost of this alternative.

SECTION 12
IDENTIFICATION OF LEAST-COST IMPROVEMENT PLANS

A. INTRODUCTION

Section 11 presented the costs of alternatives reviewed for the Phase 3 study. This section identifies the least-cost alternatives both for the separated sewer alternatives and for those alternatives providing flood and CSO control for the 2-, 5-, 10- and 25-year design storms.

B. LEAST-COST ALTERNATIVES

Of the sewer separation alternatives, the least-cost option is the Separate Sanitary Sewer Alternative. This alternative essentially parallels the existing combined sewers with new, sanitary sewers.

Table 12-1 summarizes the estimated total project costs for all the least-cost alternatives. Table 12-2 summarizes the facilities included in the least-cost alternatives wherein the sewers remain combined.

For the 2-year flood control design storm with a 1/year CSO storm control, the least-cost alternative is to provide a combined treatment/storage process facility at the Pioneer Reservoir site and upsize the existing combined sewer collection system to convey wastewater to the site for flood control. For the 2-year design storm with a 5/year CSO storm control, the existing Pioneer Reservoir is capable of storing the design CSO storm event without a spill. For flood control, the existing collection system is upsized.

For the 5-year design storm with either a 1/year or 5/year CSO storm control, the least-cost alternatives were tunnels. For a 1/year CSO storm control, the tunnels consisted of a system of 15-foot diameter conduits. For a 5/year CSO storm control, a 14-foot diameter tunnel system was used.

As with the 5-year alternatives, the 10-year least cost alternatives were also tunnels. For the 1/year and the 5/year CSO storm control levels, the 17-foot

tunnel system was identified.

For the 25-year design storm, an estimate of total project cost was prepared for only one alternative, a network of 20-foot diameter tunnels.

TABLE 12-1. SUMMARY OF COST ESTIMATES FOR LEAST-COST ALTERNATIVE IMPROVEMENT PLANS

Flood Control Design Storm	Least-Cost Alternative	Total Improvement Plan Cost ($ millions)[4]	
		1/Year CSO	5/Year CSO
Alternatives Wherein Sewers Remained Combined			
2-year	Upsize Existing CWCS Conveyance w/ Treatment at Pioneer Reservoir Site	$ 363	
5-year	15-foot Tunnel System	$ 587	
10-year	17-foot Tunnel System	$ 797	
2-year	Upsize Existing CWCS Conveyance		$ 312
5-year	14-foot Tunnel System		$ 584
10-year	17-foot Tunnel System		$ 793
25-year	20-foot Tunnel System	$ 1,035[1]	$ 1,035[1]
Sewer Separation Alternatives			
N.A.[2]	New Sanitary System[3]	$ 381	

1 This alternative controls both the 1/year and 5/year CSO control design storms.
2 The flood control capability of the existing system would remain unchanged from the existing system, which approximately controls flooding for the 1.5- to 2-year storm.
3 The existing piping would serve as the storm drain system, with minimal flood control improvements.
4 Includes all rehabilitation/repair and reliability.

TABLE 12-2. SUMMARY OF MAJOR FACILITIES INCLUDED IN LEAST-COST COMBINED SEWER ALTERNATIVE IMPROVEMENT PLANS

Description	CSO Design Storm	2-Year Flood Control Design Storm	5-Year Flood Control Design Storm	10-Year Flood Control Design Storm	25-Year Flood Control
Tunnels w/ Consolidation Conduit and Weir System	1/Year		consolidation conduit/weir system; 15-foot tunnel; CSO elevation at -18.00 feet MSL; 14-mgd emptying stations at Sump 2 and 1A to SRWTP	consolidation conduit/weir system; 17-, 18- and 19-foot tunnel segments); CSO elevation at -23.00 feet MSL; 430-mgd and 265-mgd CSO stations at Sump 1A and 2, respectively; 14-mgd emptying stations at Sump 2 and 1A to SRWTP	consolidation conduit/weir system; 20-foot tunnel; CSO elevation at -18.00 feet MSL; 720-mgd and 350-mgd CSO stations at Sump 1A and 2, respectively; 14-mgd emptying stations at Sump 2 and 1A to SRWTP
	5/Year		consolidation conduit/weir system; 14-foot tunnel; CSO elevation at -18.00 feet MSL; 14-mgd emptying stations at Sump 2 and 1A to SRWTP	consolidation conduit/weir system; 17-, 18- and 19-foot tunnel segments); CSO elevation at <-24.00 feet MSL; 420-mgd and 250-mgd CSO stations at Sump 1A and 2, respectively; 14-mgd emptying stations at Sump 2 and 1A to SRWTP	
Upsized CWCS Conveyance w/ Primary Treatment Plant at Pioneer Reservoir Site	1/Year	upsize existing laterals and mains; 500-mgd primary treatment plant; gravity effluent discharge			
Upsized CWCS Conveyance	5/Year	upsize existing laterals and mains			

APPENDIX A
Brierley & Lyman Memorandum on Dewatering Costs

MEMORANDUM

TO: Greg Jones,
HDR Engineering, Inc.

FROM: Alan Howard and Gary Brierley,
The Brierley & Lyman Division of Haley & Aldrich, Inc.

SUBJECT: Sacramento CSO Control Phase 3 Study-Construction Dewatering Analysis and Cost Estimate

DATE: November 21, 1991

COPIES: File

INTRODUCTION

The purpose of this memorandum is to provide you with planning/study level conceptual cost estimates for construction dewatering for the referenced project. Based on your letter request of November 12, 1991, the Brierley & Lyman Division of Haley & Aldrich, Inc. (Brierley & Lyman) has evaluated conceptual construction dewatering requirements for three construction alternatives and developed associated conceptual level cost estimates for dewatering. These analyses were based on existing information from the CSO Control Phase 2 study and additional information provided by you in your recent letter. Generalizations were made about project subsurface conditions and anticipated construction methodology, so our analyses are not site-specific.

We understand that updated conceptual cost estimates for construction dewatering are required for three construction alternatives consisting of: 1) common sewer trench excavations to depths up to approximately 35 ft below ground surface using trench box construction methods; 2) an interim storage reservoir to be constructed near the UC Davis Medical Center site which would consist of an approximately 400 x 500 ft reservoir constructed to depths up to 25 ft below ground surface; and 3) drop shafts for CSO diversions into deep storage tunnels.

The remainder of this memorandum includes a more detailed description of the three construction alternatives that require dewatering and a discussion of generalized subsurface soil and ground water conditions assumed for project construction. Also provided are a summary of conceptual level dewatering requirements such as anticipated steady state ground water withdrawal requirements and anticipated well spacing and staging to achieve dewatered construction conditions, and cost analyses associated with these alternative dewatering scenarios.

PROJECT DESCRIPTION

Specific alternative projects for flood and CSO control were previously evaluated for the City of Sacramento. Comparisons among these alternatives were made based on estimates of their respective project costs. The City of Sacramento has requested an update of the previously developed construction methods and unit costs and Brierley & Lyman has been asked by HDR to develop updated costs for construction dewatering associated with three viable alternatives still under consideration for flood and CSO control for the City.

One type of control infrastructure consists of constructing parallel sewers. For present purposes a system of parallel sewers varying from 12 inches to 144 inches in diameter, at corresponding depths of 10 to approximately 35 ft below existing ground surface, are envisioned to supplement existing sewers within the City of Sacramento. Construction below the water table is assumed to be accomplished by construction dewatering plus trenching using a trench box or trench shield, with trench widths from three feet up to 16 ft to allow two feet of working space either side of the installed pipes. Construction dewatering for this alternative would be accomplished by installing a series of continuous ground water dewatering wells or extraction well points and by phasing construction such that construction dewatering would advance ahead of construction such that several days' to one week's trenching production would be dewatered in advance of the trench excavation. Construction dewatering would thus proceed ahead of trench excavation and some dewatering equipment could be leap-frogged ahead of completed sections of trench. For this analysis we have evaluated conceptual construction dewatering requirements and cost per thousand linear feet (KLF) of trench and assumed that each KLF of trench would require construction dewatering for a period of approximately one month.

The second construction project requiring dewatering is the proposed interim storage reservoir being considered for the UC Davis Medical Center site. The UC Davis storage reservoir has a proposed capacity of approximately 26 million gallons and consists of an approximately 3.5 million cubic foot semi-rectangular storage facility as shown in figures provided to us. The structure is approximately 400 x 500 ft in plan view and varies in depth from approximately 13 to 25 ft. The average depth of the structure as shown is approximately 18 ft deep. No adjacent structures occur within approximately 180 ft of the proposed storage reservoir.

Construction of the reservoir may require construction dewatering that must be maintained throughout excavation and final construction activities for a period up to approximately 18 months. Therefore, construction dewatering is considered by Brierley & Lyman to be crucial for this alternative. The implications for a failure in the construction dewatering system are severe due to uplift potential of the semi-completed or completed and not yet filled storage reservoir due to hydraulic pressure of an elevated water table over a large area. For our discussion below we have assumed a simplified 400 x 500 ft structure approximately 20 ft deep that would require construction dewatering to a depth of approximately 25 ft below existing ground surface.

The third flood and CSO control infrastructure alternative consists of deep storage tunnels. Based on our previous technical memorandum on the deep tunnel alternative we envision slurry shield or earth pressure

balance tunneling procedures given the subsurface conditions beneath the City of Sacramento. Tunnel access shafts, however, will require construction dewatering which comprises the third dewatering alternative we have evaluated for this memorandum.

For the purpose of this analysis we have assumed maximum 40 ft diameter circular access shafts to be constructed by dewatering to approximately five feet below an invert depth of 50 ft followed by excavation and support using conventional sheeting or ribs and lagging. Alignment maintenance hole shafts would likely be 15 to 20 ft in diameter. Construction dewatering for the access shafts must be maintained throughout construction to provide continuous safe access to the tunnels and we have assumed a construction period of 18 months to two years plus perhaps four to six months of initial dewatering in advance of construction. Maintenance shafts would likely require dewatering for only about five months. Our analysis below is on a per shaft basis for each type of shaft since the tunnel configuration and number of shafts is unknown.

SUBSURFACE CONDITIONS

Subsurface conditions assumed for construction of the three flood and CSO control measures outlined in this memorandum are based on those developed in our previous study for deep tunnels for the Sacramento CSO project. Subsurface soil conditions consist of approximately 15 ft of fine-grained overbank silts and clays overlaying interlayered sand, silty-sand, silt and clay to a depth of approximately 100 ft below which finer grained clayey soils occur. Nearer to river locations gravelly layers sometimes occur at depths between approximately 20 and 40 ft below ground surface. The water table in the vicinity of the project varies from approximately 10 to 45 ft below ground surface and has been assumed to exist uniformly at about 15 ft below ground surface for the purpose of the analyses provided herein. Since all of the construction alternatives require construction at depths less than 100 ft we have assumed that the interlayered sands silty-sands, silts and clays from 15 ft below ground surface to approximately 100 ft below ground surface have a uniform hydraulic conductivity of a fine sand to silty sand of approximately 1×10^{-3} cm/sec. We have further assumed for the purpose of hydrogeologic analyses that the alluvial deposits behave as an unconfined aquifer the base of which is at a depth of approximately 100 ft where more clayey soils are encountered.

The following hydrologic parameters were assumed for the dewatering calculations performed to make our analyses:

- Hydraulic conductivity (K) = 1×10^{-3} cm/sec $\approx$ 2.8 ft/day
- Aquifer thickness (b) $\approx$ 85 ft
- Specific yield (S_y) $\approx$ 0.2 (Fetter, 1988)

Dewatering Requirements

Assuming a uniform depth to water of 15 ft precludes dewatering concerns for trenching for parallel sewers to depths less than 15 ft. Trench dewatering requirements will not vary significantly with depth for the proposed deeper sewer trenches for the Sacramento CSO project. Drilling and installation requirements on a per well basis will be about twice as much for well materials and pump capacity for dewatering to a depth of 40 ft for a 35 ft deep trench versus dewatering to 25 ft for a 20 ft deep trench, but overall mobilization, labor, equipment and operation costs will be similar. Since the layout of parallel sewer dimensions, depths and length are unknown, we have provided an analysis of the worst case for a 35 ft deep sewer line on a unit basis for 1,000 LF of trenching.

Steady state inflow to sewer trenches, given all of the assumptions outlined above, are estimated to range from approximately 250 to 500 gpm per KLF of trench for trenches ranging from 20 to 35 ft deep. To accomplish trench dewatering, we estimate the use of line dewatering wells at 100 ft spacings along the trench. Allowing for well inefficiencies and interfering cones of depression, we estimated the use of 50 to 100 gpm capacity wells operated for an approximately one month long period per KLF of trench. Drawdown to a depth of 40 ft at the dewatering wells would accomplish dewatering to approximately 35 ft at the midpoints between the wells.

Dewatering of the proposed UC Davis storage facility will require long term dewatering of an approximately 200,000 square foot area. We have assumed that dewatering of this area will require lowering the water table approximately 10 ft; from 15 ft to 25 ft below existing ground surface. Given our hydrogeologic assumptions, steady state inflow to the dewatered excavation will be approximately 50 to 100 gpm which inflow could be handled by one of two alternative dewatering configurations consisting of either a series of 10 to 15 shallow dewatering wells spaced evenly around the excavation or approximately four deep higher capacity wells centered within each quadrant of the excavation. Given well inefficiencies and interference, we anticipate the need for either ten to fifteen 10 to 20 gpm wells or four 50 gpm wells, respectively.

Dewatering of a 40 ft diameter tunnel shaft to a depth of 55 ft (five feet below an invert at 50 ft) with the water table at a depth of 15 ft would result in steady state inflow of about 75 gpm to the shaft excavation. Deep dewatering wells would be used to dewater the shaft and maintain a depressed water table during tunnel construction for site access at the main shaft(s). Allowing for well inefficiencies and interference we estimate the need for approximately four 50 gpm deep wells spaced evenly around the shaft.

Dewatering Costs

We have made some generalized assumptions about dewatering costs to provide you with unit costs for dewatering the three construction alternatives. Dewatering costs include drilling, installation, equipment, labor, supervision and operation. Costs are incurred by both the dewatering subcontractor and the general contractor during both the installation and operation phases. Furthermore, contingencies must

be made for site access, utilities, inflation and trade union requirements. Based on available information from dewatering subcontractors and our experience, we have combined these considerations to develop unit costs as discussed below.

For sewer trench dewatering as outlined above, we estimate $50.00 to $75.00/LF of trench for trench section dewatered for a one month period. Deeper trench contracts will be closer to $75.00/LF and the shallowest trenches requiring dewatering may be constructable at dewatering costs at or slightly less than $50.00/LF.

Dewatering of the underground storage facility will require continuous monitoring during the construction period to preclude serious problems for the project caused by dewatering system failure. Hence, operation costs for our assumed 18 month construction schedule are high. We estimate approximately $50,000.00 for the installation of a dewatering system plus an additional $10,000.00 per month for operation and labor costs for a continuously monitored system. Budgeting for approximately $250,000.00 for dewatering should be allowed for this alternative. Considerable construction cost, and perhaps underdrain design and long term permanent dewatering costs, could be saved by siting such a facility above the water table, gaining additional storage through increased area versus increased depth. Conversely design of a superstructure such as a parking facility on top of the reservoir could provide needed dead load for a better design at depths below the water table.

Tunnel shaft dewatering on a per shaft basis is estimated to be approximately $50,000.00 for access shafts and $35,000.00 for maintenance shafts for the configuration of four deep wells outlined above. Since tunneling is a more or less continuous operation, monitoring of the dewatering system would likely be accomplished by tunnel contractor personnel already on-site. We estimate an additional $1,000.00 per month operation and equipment cost to the contractor once the installation is complete. Hence, access shafts open for 24 months would cost approximately $75,000.00, and maintenance shafts open for five months would cost approximately $40,000.00 to dewater.

APPENDIX B
Catalog Details on Inflow Regulating Devices

Hydro-Brake

liquid flow controls

February 1982

CONTEXT

The control of flow of liquids, particularly where there are intermittent surges, is a frequent need both in the public sector and in industry.

Ideally, a flow control device while being reliable and accurate, should be self-activating with rapid response and need neither instrumentation nor external power. It is important that a free passage should always be maintained and the possibility of blockage must be minimal; further, there should be no risk of downstream damage, for example from jet flow. A flow control should be easy to install, be robust enough to withstand misuse, and require minimal maintenance.

The Hydro-Brake has been developed to the point where it meets the foregoing requirements, and its success and reliability have been amply demonstrated by well over one thousand installations and many years of experience, in several countries, in a variety of situations.

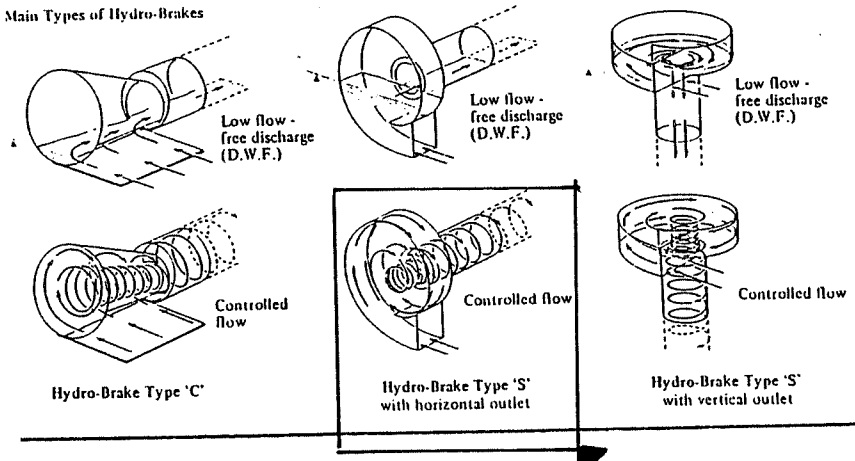
DESCRIPTION

The Hydro-Brake is a precision engineered component made in stainless steel, which is push fitted, using compressible rings for seals, into the system it is designed to control. It has no moving parts and operates by directing the flow to rotate inside a cylindrical vessel towards a central outlet. A vortex flow results with high peripheral velocities and large centrifugal forces near the exit. These forces become so great that an air filled core is generated in the centre of the chamber. This means that the openings can be relatively large and explains why a Hydro-Brake is not easily blocked. The vortex creates a back pressure opposing flow, so high hydraulic resistances can be achieved. The higher the inflow pressure head, the faster the vortex rotates which in turn, automatically increases resistance to the flow.

The potential applications of Hydro-Brakes are numerous and since each application has a unique combination of conditions and performance specifications, there are no 'standard' models. Aqua Group design each installation to the given requirements and where these are not known, facilities for investigation and research are offered.

Hydro-Brakes range in size from small units that can be held in the hand, for example for flow control in a laboratory, up to units several metres in diameter to control flow from a dam. Hydro-Brakes are frequently manufactured and transported in sections for site assembly. For pressure applications flanged connections can be specified.

Main Types of Hydro-Brakes



VALUE RECOMMENDED

TYPICAL USES

There are many situations where a reliable flow control device may be needed. These include :

Storm Water Management	Low cost, high reliability inlet control systems.
New Developments	To attain zero increase in the rate of run off, resulting from new development.
Storm Overflows	Controlling the outflow in the sewer and regulating overflow.
Storage Facilities	Controlling outflows from balancing ponds, reservoirs and underground storage tanks.
Treatment Plants	Control of hydraulic load.
Flow Balancing	Attenuation of surge flows and suppression of pump surges.
Control of Dams	Controlling dams with accurate regulation of compensation water and overflow outlets.
Flow Dividers	For multi-distribution of flow from a single source.
Drop Shaft Control	Energy dissipation and flow control in drop shafts.
Land Drainage Control	For brooks and ditches to control both run off rates and downstream flooding.
Roof Downspout Control	Regulation and control of downspouts and mobilisation of roof water storage.

STORM WATER MANAGEMENT

Publication ref AP50 quoting case histories on this subject will be supplied on request.

Amongst the examples listed above, one of the widest uses of Hydro-Brakes is in storm water management.

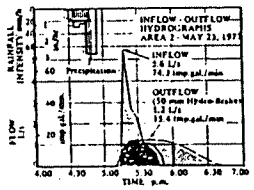
Flood control can now be achieved for relatively little cost to avoid expensive consequent damage to property and risk to health and life arising from surcharge and overflow of sewers.

The Hydro-Brake is the key invention which has opened up an entirely fresh technique for economical storm water management. The concept of Aqua Group's storm water management is to retain storm water until the drainage system has time to cope. Then to ensure that the maximum flow entering the sewers never exceeds their capacity. Storm flow is attenuated to allow the main to drain at its designed rate, by mobilising various places in the system for temporary storage of water; strategically located, carefully designed Hydro-Brakes provide the means for the necessary inlet control to the sewer.

1. The diagram illustrates the attenuation achieved in the outflow by Hydro-Brake control. (This example taken from the Canada Mortgage and Housing Association Report SCAT 7).

2a. Without control water flows rapidly into the sewer which can quickly become overloaded and pressurised.

2b. With Hydro-Brake control water is fed at a controlled rate into the system so that the design capacity of the sewer is never exceeded.



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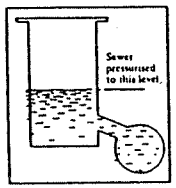
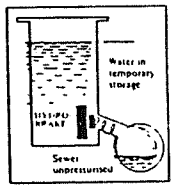
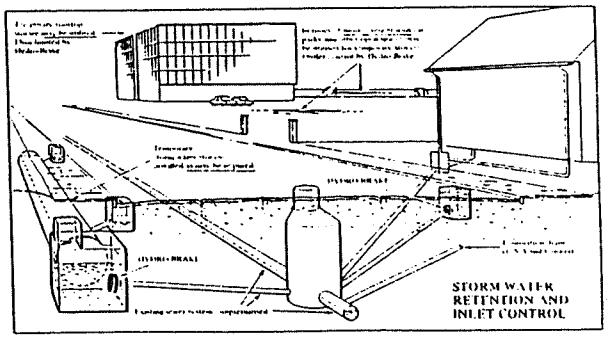


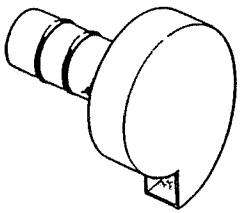
2a



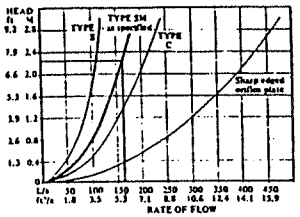
2b

Hydro-Brake Type S being installed with compressible sealing rings.



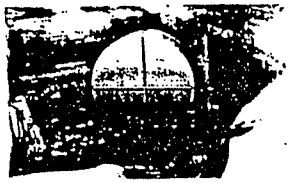


Hydro-Brakes Type S or SM
Generally used to control storm water in the flow to sewers. They are well suited for gullies where a gas trap is needed. They can also be used in foul sewers with a downward vertical outlet.

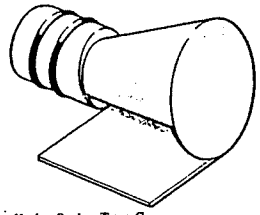


Head Discharge characteristics for Model 355 SM 455.

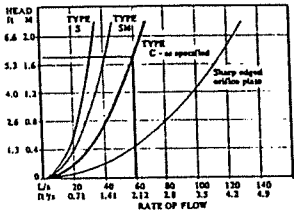
Specified duty : 158 l/s (5.6 c f s) at a maximum head of 2.25 m (7' 4").



BOROUGH OF YORK, TORONTO CANADA
Model 355 SM was installed to control flow into a main sewer whilst mobilising additional underground storage in tanks to the left and right of this catchbasin.

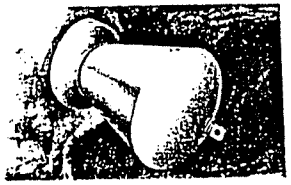


Hydro-Brakes Type C
In their simplest form provide suitable controls for storm water flow conditions, yet allowing dry weather flow to pass without restriction. Requires benching to direct flow at correct angle of entry.

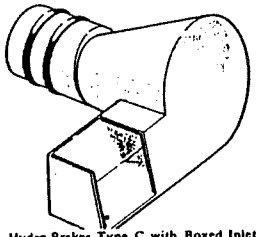


Head Discharge characteristics for Model 200 C 355

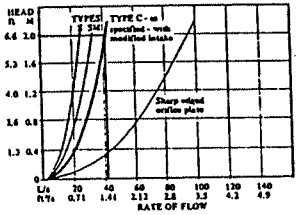
Specified duty : 58 l/s (1.75 c f s) at 1.75 m (5' 8") design head.



PORTLAND, MAINE, U S A
Model 200 C 355 installed in a combined sewer as a system control.

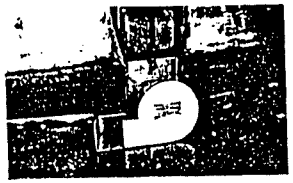


Hydro-Brakes Type C with Boxed Inlet
Provide free discharge of dry weather flow and control for storm water flow where benching is not available, also type used for smaller units.

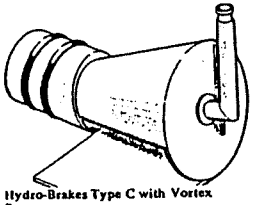


Head Discharge characteristics for Model 172 C

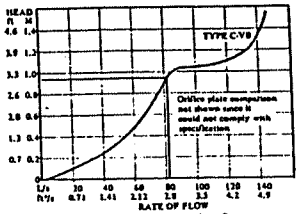
Specified duty : 40 l/s (1.4 c f s) at 2.0 m (6' 8") design head.



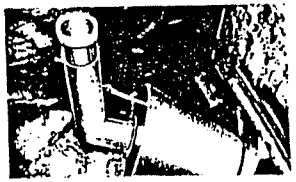
THAMES WATER AUTHORITY, ENGLAND
Model 172 C was fitted to a balancing tank in Appleton Sewage Works (plate mounted).



Hydro-Brakes Type C with Vortex Suppressor
Provide free discharge of dry weather flow and control for storm water flow, plus automatic suppression of the vortex when the head of water exceeds a given limit, which reinitiates when water level drops below suppressor pipe.



Head Discharge characteristics for Model 271 C 425.
Specified duty : 80 l/s (2.82 c f s) 1.1 m (3' 7"), with 120 l/s (4.24 c f s) at 1.2 m (3' 11") for maximum head.

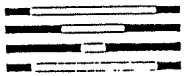


OXFORD CITY COUNCIL, ENGLAND - as agents for Thames Water Authority. M 271 C 425 was fitted to each of two foul sewers at St. Clements. Shown here under test at the University of Bristol.

COMPARISON WITH ALTERNATIVES

Hydro-Brakes have significant advantages over conventional flow control devices. For example where the requirements are to restrict a flow to 35 litres/sec (1.25 cf/s) at a maximum static head of 3 metres (10 ft), the comparative outlet sizes for each are as shown below :

For Outlet Sizes - see text



Hydro-Brake 200 mm (8 in)
Sharp edged Orifice Plate 97.92 mm (3.8 in)
Gate Valve Screwed down to 36 mm (1.4 in) clearway height of gate above invert.
Throttle Pipe 300 metres* (1000 ft) level pipework 200 mm (8 in) diameter.

PERFORMANCE

Hydro-Brakes are versatile and can be designed to the requirements of any application by modifying its component parts; in addition, they can normally be tailored to fit space limitations and existing site conditions.

Pressure Head Normally, the physical conditions of the site will dictate the head.

Outflow Rates Each unit is unique, individually designed to pass the required flow at the operating head. The head discharge curve is parabolic within the vortex range and this may be seen from the examples on page 3.

Size Aqua Group will advise on the size of unit needed for any application. The only limitation is the minimum clear pipe diameter to suit the liquid, typically 100 mm (4 in) minimum for crude sewage and 65 mm (2.5 in) for storm water.

Types of Liquid Hydro-Brakes will control all reasonable free flowing liquids such as sewage. For industry, more viscous liquids can be controlled but some research and development may be needed.

VARIATIONS

Vortex Suppressor A Hydro-Brake may be designed so that its vortex is automatically suppressed to allow free full pipe flow when the pressure head exceeds a pre-determined limit. When the pressure head falls below this upper limit, the vortex re-initiates and the flow control facility is resumed. (See example on page 3).

Pressure Applications Flanged connections can be specified.

Gas Traps The Hydro-Brake Series S, used with a horizontal outlet, functions as a gas trap.

Restricted Access Hydro-Brakes may be delivered in sections for bolting together in situ, when access to their location is limited.

HYDRO-BRAKE INSTALLATIONS

Authorities to whom Hydro-Brakes have been supplied *include* those listed on the right.

CANADA The Cities of Toronto and Metropolitan Toronto, Ottawa, Windsor, the Town of Markham and the Boroughs of Scarborough and York - Ontario. The Cities of Dartmouth - Nova Scotia, Regina - Saskatchewan, Winnipeg - Manitoba, Edmonton and Calgary - Alberta, St. John - New Brunswick, St. John's - Newfoundland. DENMARK The City of Fredericia. MEXICO The City of Mexico City, D.F. NORWAY The City of Oslo. UNITED KINGDOM South West Water Authority, Severn Trent Water Authority - Lower Trent & Avon Divisions, Thames Water Authority - Vales Division, Stratford-on-Avon District Council, Oxford City Council, Tortridge District Council. UNITED STATES The Cities of Portland - Maine, Euclid, Cleveland, North Royalton - Ohio, New York and Rochester - New York State, Seattle and Washington - Washington State, Salt Lake City - Utah, Casper - Wyoming, Pensicola - Florida. SWEDEN The City of Gothenburgh.

HYDRO GROUP SERVICE

Aqua Group welcome enquiries for applications of their products and there is no obligation for an initial enquiry and consultation.

Hydro-Dynamic Separator publication ref TP2 will be supplied on request.

Simple Applications Where all the design data is available, a free quote for the appropriate Hydro-Brake installation will be provided; essential design data includes:
i) Operating head.
ii) Rate of flow required at this head.
iii) Description of application.
iv) Installation details.

Investigations Where adequate design data is not available, Aqua Group will provide a free quote for the cost of investigation to establish agreed design data.

Urban Flooding Control Aqua Group have a sophisticated computerised technique of hydrological analysis for temporary flood storage requirements to ensure that a sewer system never becomes overloaded, according to its design parameters causing flooding. A free quote for such a service will be provided. Alternatively, the appropriate programme and necessary micro computer can be supplied as a complete package for the client's use.

Hydro-Brake Fabrication Design, fabrication and supply of Hydro-Brakes is always carried out by Aqua Group.

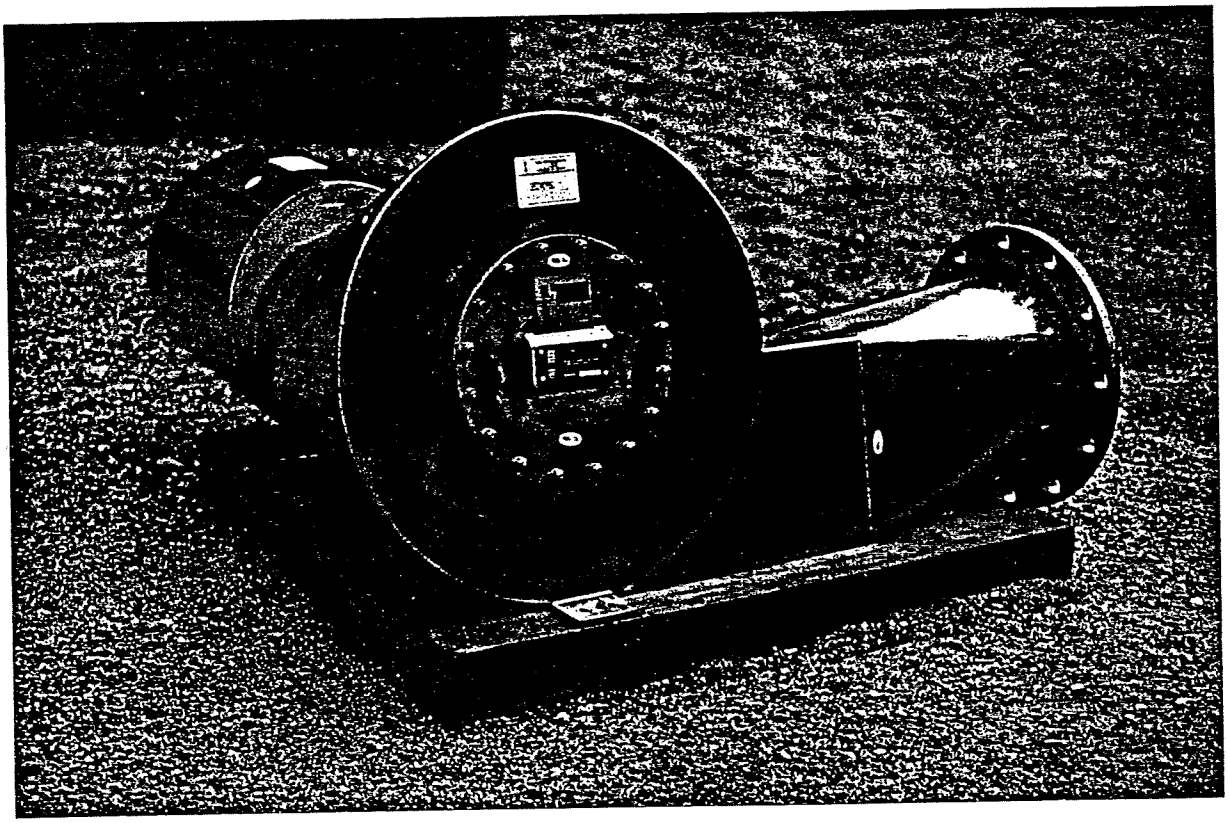
Hydro-Brake Installation Normally, this is carried out by the client. Alternatively, Aqua Group will quote for installation.

Hydro-Brake Systems
One Union St. Suite 401
Portland, Maine 04101
USA
A Division of Aqua Group, Inc.

Tel. # (207)774-0747
Fax. # (207)772-5035

H.I.L.
TECHNOLOGY
INC.

REG-U-FLO®
VORTEX VALVE

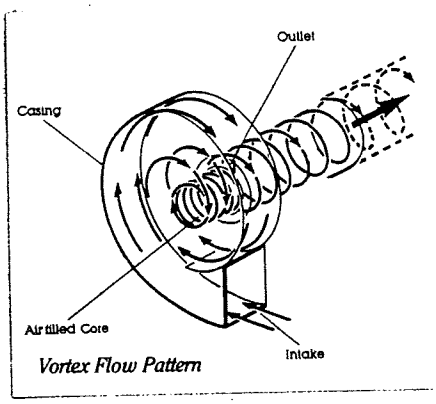


Keeping Water In Its Place
Without Costing The Earth

THE REG-U-FLO® VORTEX

"Keeping water in its place without costing the Earth" is sometimes easier said than done. This is especially true in urban areas where new buildings, wider roads, and larger parking lots have reduced natural drainage, causing rapid runoff of stormwater in increased volume which cannot be handled by existing sewers and watercourses. The resultant flooding or sewage overflow is often devastating to property, dangerous to public health, and ultimately a drain on the public purse. Installing additional relief sewers and interceptors is expensive, disruptive to the community, and in most cases merely transfers the problem downstream.

Clearly a more rational approach to stormwater management is needed, and innovative engineers have focused on the concept of stormwater detention. The idea is simple: reduce flow to the downstream network by installing flow control devices at key points within the system which mobilize the storage volume inherent in upstream pipes, manholes, and surface terrain. Or to put it another way, ensure that maximum flow entering the sewer system does not exceed its capacity. For such a system to work, a self activating, economical, reliable flow control device which is easily installed into the existing drainage network to regulate the flow is required. Such a device exists — the Reg-U-Flo Vortex Valve from H.I.L. Technology, Inc.



Vortex Flow Pattern

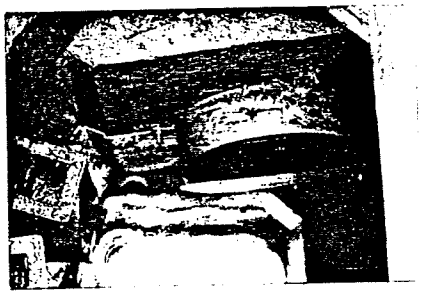
The design is simple but ingenious. It consists of an intake, a volute, and an outlet. The flow is directed tangentially into the volute, generating high peripheral velocities that induce an air filled core with a resulting back pressure

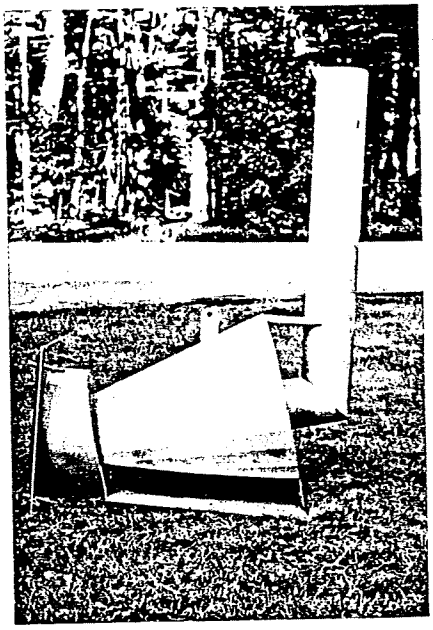
The first use of vortex flow control was in Rome in 97 A.D. in the distribution of water from aqueducts. Forgotten during the Middle Ages, vortex flow principles were rediscovered during the Renaissance by Leonardo da Vinci who sketched a bathplug vortex. Further research in the 17th and 18th centuries by the likes of Torricelli, Huygens, Newton, and Coriolis laid the scientific and mathematical framework for H.I.L.'s 20th century adaptation of vortex flow theory to stormwater management techniques.

that reduces the discharge. Units are custom designed for optimum performance, varying in size from a few inches to many feet in diameter. Typical flow rates range from as low as 0.05 cfs for an individual catchbasin control to 150 cfs for flow control at dams and rivers.

The Reg-U-Flo Vortex Valve is clearly superior to other flow control devices. Robustly fabricated of corrosion resistant stainless steel, it is totally self activating, has no moving parts, uses no external power, requires little or no maintenance, and lasts indefinitely. It has clear openings up to 400% larger than conventional flow regulators based on the same design criteria, greatly reducing the likelihood of blockage. Because of its unique three phase head discharge characteristic, it can reduce overall storage requirements by up to 30%, significantly reducing project costs. When used on CSO abatement programs, a greater percentage of the polluted first flush is retained in the system, reducing the volume of overflow.

Performance can be enhanced with a variety of accessories. The variable intake gate allows for modification of the unit's flow rate by up to 300% after installation, and the pivoting bypass door allows easy system draindown for maintenance purposes. Vortex suppressor pipes are also available to increase the flow by up to 50% with a very small increase in head.


Type SH Reg-U-Flo Vortex Valve with design discharge of 0.5 cfs (14 l/s) under 5 ft. (1.5 m) head. Here the spigot of the valve is located in the outlet pipe and the valve itself is held in position by three anchor bolts. The SH type unit has a submerged inlet which acts as a gas trap and will also intercept oil and floatables. The geometry of the casing may be modified and reduced in size, resulting in a Type SMH. As with all Reg-U-Flo Vortex Valves, the intake is rectangular or trapezoidal in shape because independent surveys have shown that circular orifices provide a significantly greater chance for blockage to occur.


An 18" (457 mm) Type C Reg-U-Flo Vortex Valve with a vortex suppressor pipe. When the water level rises above the mouth of the vortex suppressor pipe it passes down the pipe and suppresses the air filled core within the unit. In this way temporary increases in discharge of up to 50% can be effected with a minimal increase in head. The regular flow pattern is resumed once the water level falls below the mouth of the vortex suppressor pipe. The pipe may be telescopic or flexible so that onsite adjustments can be made.

SIMPLE EFFICIENT EFFECT

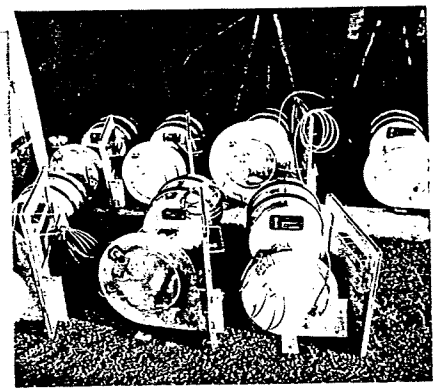
LVE

The Vortex Valve's effectiveness has been proven in over 7500 installations worldwide. Its unique capabilities can be applied to any situation where flow control is required, in particular to CSO abatement and stormwater management on new developments in order to maintain flow rates at those of the predeveloped site. Other applications include storage facilities to control outflow, treatment plants to control the hydraulic load, flow balancing, inlet control, catchbasin control, downspout control, irrigation, flow splitting, drop shaft control, land drainage control, and control of compensation water and outlets from dams.

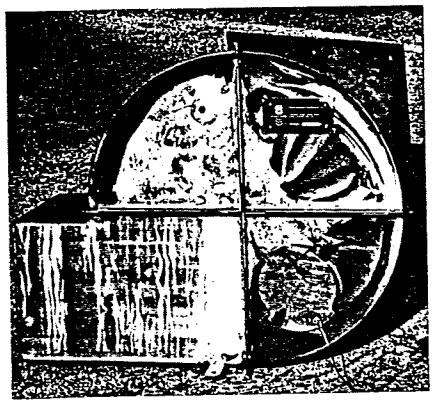
H.I.L. Technology, Inc.'s engineering staff can advise you free of charge and without obligation on how the Reg-U-Flo Vortex Valve can solve your particular problem. Flow characteristics, installation drawings, and technical specifications are readily available upon request. Demonstration units can also be provided. As with all of H.I.L. Technology's equipment, the Reg-U-Flo Vortex Valve carries an excellent product warranty.

SIMPLE EFFICIENT EFFECTIVE
ECONOMICAL INGENIOUS

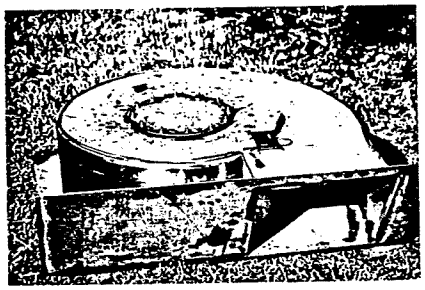
The Reg-U-Flo Vortex Valve from H.I.L. Technology, Inc. Keeping water in its place without costing the Earth.



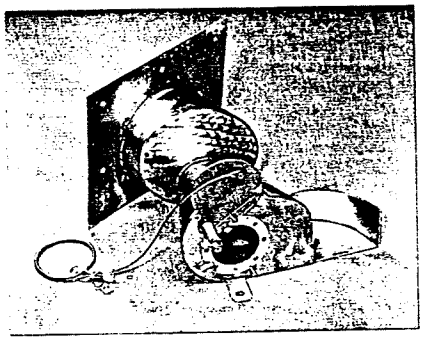
A number of CH Vortex Valves supplied to the City of Dearborn, Michigan. The valves incorporate a push fit spigot, spring loaded pivoting bypass door with pull wire, a box intake and a sliding intake gate. The sliding intake gate adjusts the point of vortex initiation allowing the engineer to modify the flow characteristics of the unit by up to 300% following installation, thus facilitating fine tuning and staged implementation.



If the Reg-U-Flo Vortex Valve is to be fitted into an existing manhole with limited access through the cover, as part of a CSO abatement program for example, the valve can be supplied in bolt together sections.



Primarily Type S series units are designed for use on stormwater systems. Type S units can, however, be used on combined and sanitary systems if the orientation of the unit is such that the flow enters the unit and rotates in a horizontal plane before discharging vertically out of the unit. When mounted in this orientation they are termed SV units and are particularly useful as velocity controls and energy dissipators. The photograph shows a plate mounted (offseating) Type SV unit which was used as a velocity control on a steep sewer line. An inspection hatch was provided in the top of the unit.



A plate mount 6" (150 mm) Type CH unit with a swept box intake and pivoting bypass door. The CH Type unit is similar to the C and is generally used for low sanitary and combined flows. The photograph shows the door in the open position, allowing rapid draindown of the system for inspection purposes and it doubles as a rodding/jetting facility mitigating costly maintenance. The pivoting bypass door is recommended for all Reg-U-Flo Vortex Valves but particularly those at lower flow regimes.

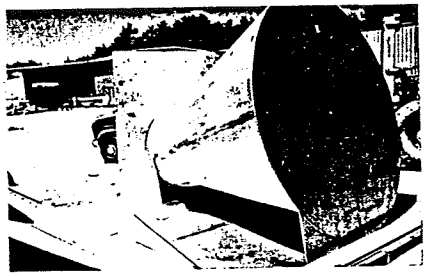
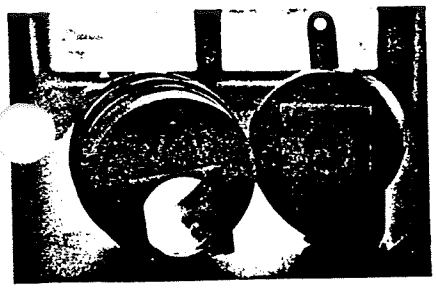


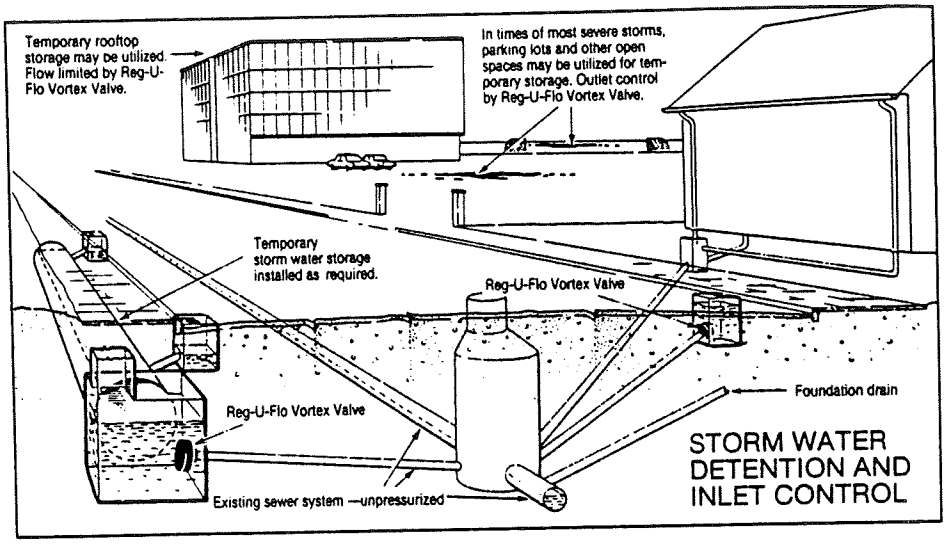
Plate mounted 34" (864 m) Type C supplied to the City of St. Paul, Minnesota. One of the largest units in North America. Flow 52 cfs (1.5 m^3/s) under a head of 10 ft. (3.0 m). The unit stands about 6 ft. (1.8 m) high and 7 ft. (2.1 m) long. The intake plate of the Type C is installed flush with the invert of the control chamber. Where possible the unit is offset up to 30° to maintain a near straight flow path through the unit so that sanitary solids under dry weather flow conditions may pass unimpeded, minimizing the possibilities of clogging. The Type C unit is highly versatile and may be used for stormwater, sanitary and combined sewer applications.

The craftsman at work. Final adjustments being made to the seating of the stainless steel pivoting bypass door. On completion, each item is stringently inspected to ensure high quality assurance to the client.

VE ECONOMICAL INGENIOUS



The Type SH has the lowest coefficient of discharge and is therefore able to offe the largest area of flow for a given head and discharge. For this reason it is most suitable for surface water applications particularly with flows as low as 0.05 cfs (1.4 l/s) such as catchbasin controls. The photograph shows a catchbasin control in its two component parts with the nylon seating/seal. The spigot on the left is simply push fit into the outlet pipe of the control chamber and the valve itself locates on the slides. A hook facilitates removal of the valve from the spigot for periodic inspection.



STORM WATER DETENTION AND INLET CONTROL

WHY CHOOSE THE REG-U-FLO VORTEX VALVE?

The advantages of the Reg-U-Flo Vortex Valve from both a technical and economic standpoint are clear. For example, if the requirement is to restrict the flow to 1.35 cfs (38 l/s) under a head of 9 ft (2.75 m) the solutions could be:

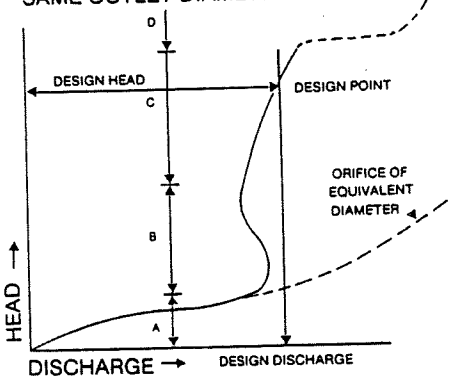
1. 1 No. 8" (200 mm) Type SH Reg-U-Flo Vortex Valve.
2. 1000 ft (305 m) of 8" (200 mm) diameter throttle pipe laid level.
3. 1 No. Gate Valve screwed down to 1.4" (36 mm) above invert.
4. 1 No. 4" (100 mm) diameter orifice plate.

As the figures show, the area of flow on the Reg-U-Flo Vortex Valve is four times larger than the equivalent orifice plate. The Vortex Valve not only offers a large area of flow, but is easily maintained when compared with conventional devices.

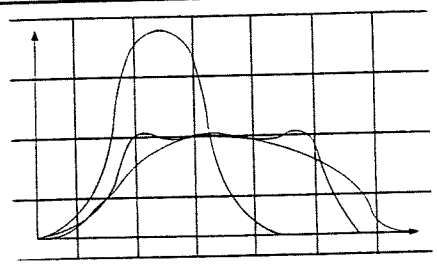
OTHER ADVANTAGES OF THE REG-U-FLO VORTEX VALVE

1. No external power requirement.
2. No moving parts.
3. Totally self activating.
4. Robust stainless steel construction.
5. Outlet openings up to 400% greater than conventional controls.
6. Virtually maintenance free.
7. Simple to install.
8. Reduced capital costs.
9. Individually designed.
10. Versatile.
11. Short delivery time.
12. Proven track record.

COMPARATIVE HEAD/DISCHARGE CHARACTERISTICS OF REG-U-FLO VORTEX VALVE AND ORIFICE PLATE OF SAME OUTLET DIAMETER.



A = Lower Limb — Orifice Flow
B = Transition Zone
C = Fully initiated Vortex Flow Pattern
D = Curve Modified with Vortex Suppressor Pipe (optional)
Zone A = Orifice discharge under dry weather flow conditions with negligible backwatering effect.
Zone B & C = Shows almost vertical curve, therefore little increase in discharge in spite of large increase in head compared with conventional orifice plate.



STORMPHASE

H.I.L. Technology, Inc. has a suite of computer programs which enable the engineer to model hydrologically both the undeveloped and the developed site in order to assess the benefits of the inlet control system on a particular catchment. Demonstration copies of the program can be made available.

H.I.L. TECHNOLOGY INC.

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TOLL FREE — 1-800-848-2706

H.I.L. Technology, Inc. welcomes inquiries for applications of its products and there is no obligation for initial inquiries or consultation.

H.I.L. Technology, Inc. has a comprehensive network of agents and representatives throughout the USA and Canada.

H.I.L. RVV 2/91

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HYDROVEX

HYDROVEX PRODUCT LINE

FLOW LIMITORS

- HHV Horizontal HYDROVEX Valve
- IHV Inclined HYDROVEX Valve
- VHV Vertical HYDROVEX Valve
- DCH Digitally Controlled HYDROVEX Valve
- TTT Thistle Tube Throttle

SEDIMENTATION CONTROL

- SFT Sediment Flushing Tank

WATER LEVEL CONTROLLER

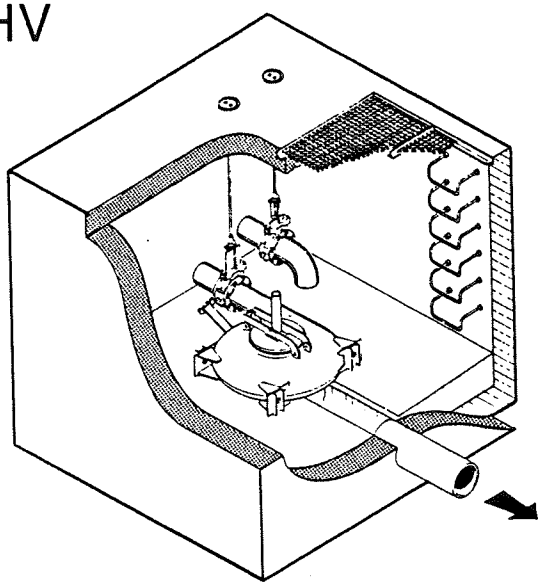
- ARS Air Regulated Siphon

FOR USE IN

Combined Sewer Regulation and Control
Sanitary Flow Interception
Storm Water Management
Flood Control

JOHN MEUNIER INC.

Horizontal HYDROVEX Valve
HHV



Description:

Discharge throttling device to limit outflow from storage and overflow tanks or reservoirs.

Principle:

The discharge is controlled by creating an air-filled vortex which reduces the effective water passage area without physically reducing the orifice size.

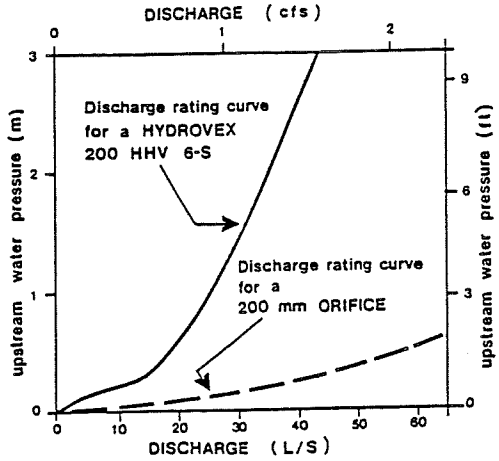
Advantages:

- Long-lasting stainless steel construction.
- No moving parts.
- No external energy requirements.
- No reduction in flow area to obtain required discharge curves.
- Easy to install in existing or new chambers.
- Low maintenance costs.
- Interchangeable outlet sizes yielding from 0.80 to 1.85 the nominal flow to accomodate future changes in flow regime.

Options:

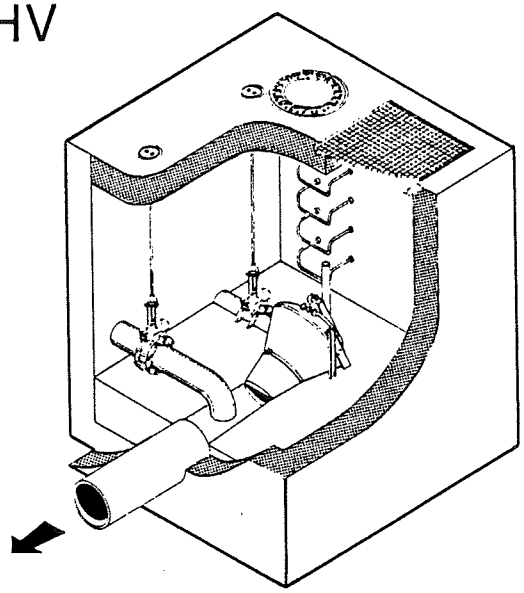
- Remote control
- May be used as a flowmeter with the addition of a pressure transducer.

Accuracy: ± 5 % at design flow.



Comparison of a typical 200 HHV 6S "HYDROVEX" Valve with an equivalent orifice

Inclined HYDROVEX Valve
IHV



Description:

Discharge throttling device to limit outflow from storage and overflow tanks or reservoirs.

Principle:

The discharge operates in two stages. Up to a predetermined upstream water level (UWL), the valve operates as a simple orifice, with a capacity approximately equal to an orifice with an equivalent diameter. When the UWL exceeds a certain value, an air-filled vortex forms at the center of the outlet which effectively reduces the water passage area without physically reducing the outlet size.

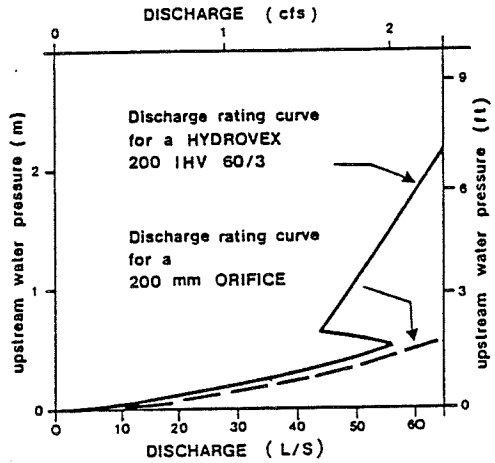
Advantages:

- Long-lasting stainless steel construction.
- No moving parts.
- No external energy requirements.
- No reduction in flow area to obtain required discharge curve.
- Easy to install in existing or new chambers.
- Low maintenance costs.
- Interchangeable outlet sizes yielding from 0.80 to 1.85 the nominal flow to accomodate future changes in flow regime.

Options:

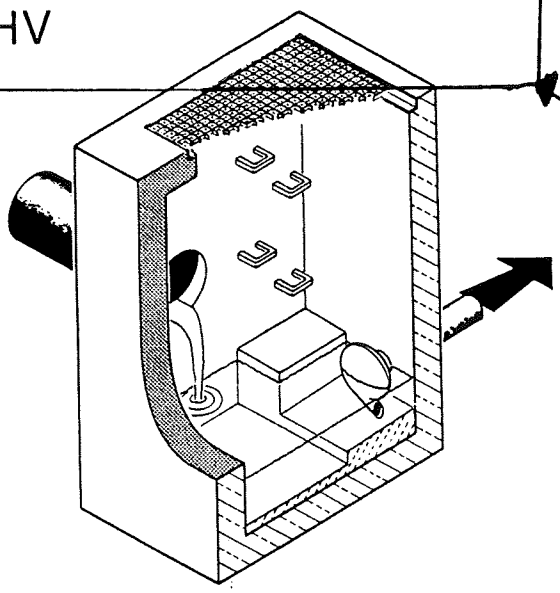
- Remote control

Accuracy: ± 5 % at design flow.



Comparison of a typical 200 IHV 60/3 "HYDROVEX" Valve with an equivalent orifice

Vertical HYDROVEX Valve
VHV



VALVE TYPE RECOMMENDED

Description :

Limits discharges from stormwater catchbasins, by using the storage capacity of the catchbasin to mitigate the storm peak.

Principle :

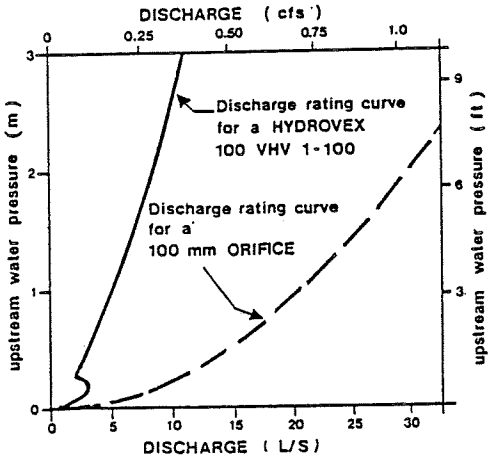
The Vertical "HYDROVEX" Valve operates as a simple orifice up to a given water level in the catchbasin. When incoming flow exceeds the capacity of the VHV the water level rises in the catchbasin as a result of the throttling effect of the vortex valve, and the discharge from the catchbasin to the sewer system is limited.

Advantages :

- Long-lasting stainless steel construction.
- No moving parts.
- No external energy requirements.
- No reduction in flow area to obtain required discharge curve.
- Easy to install.
- Solid one-piece construction.
- Low maintenance.

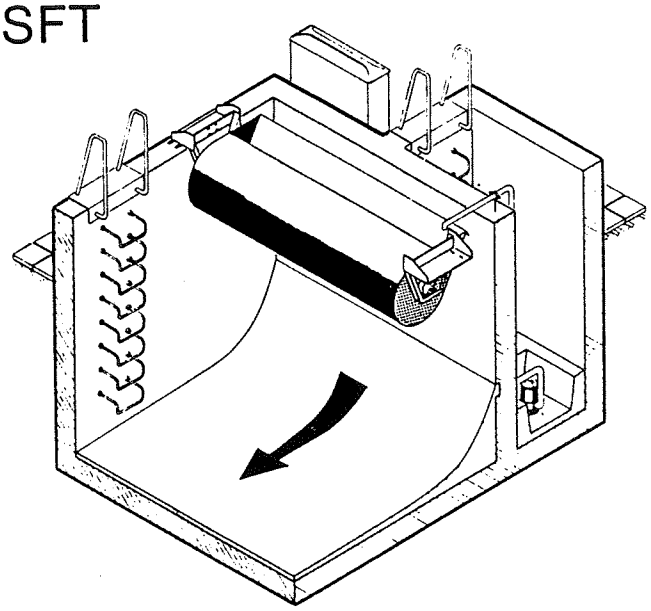
Options : None.

Flow range : From 0.5 to 140 l/s.
0.02 to 4.9 cfs.



Comparison of typical 100 VHV 1-100 "HYDROVEX" Valve with an equivalent orifice

Sediment Flushing Tank
SFT



Description :

The "HYDROVEX" Sediment Flushing Tank is designed so as to fill with water and tip automatically at specified time intervals or at the end of a storm (depending on the type of sensors attached to the unit) to flush any sediments that may be deposited on the chamber or storage reservoir floor back into the main channel.

Principle :

Because of the shape of the tank, it becomes unstable when it is filled beyond a certain level and tips. Its contents are instantly released into the chamber. This results in a high velocity wave sweeping across the chamber floor which carries any sedimented debris back into the main channel.

Advantages :

- Long-lasting stainless steel construction.
- Easy to install.

Options :

- Various types of mounting brackets.
- Digital control.
- Water level indicators (for control).

Accuracy : Not applicable.

JOHN MEUNIER INC.

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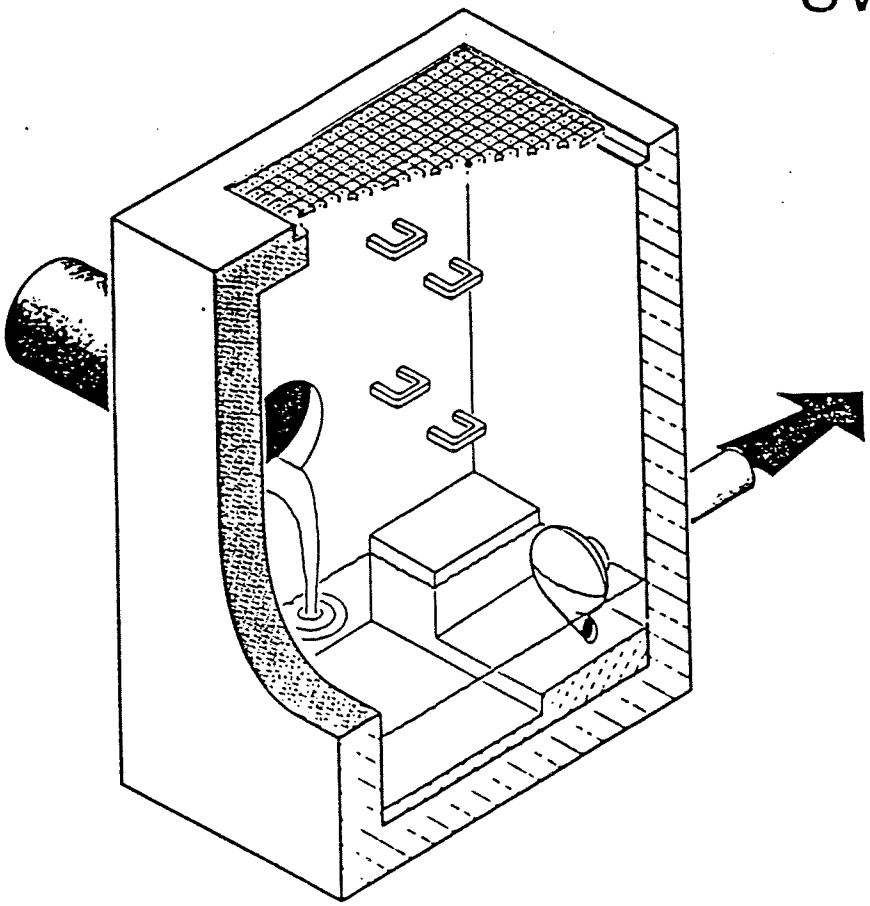
Telephone : (514) 334-7230
Telex : 05-825693
Fax: (514) 334-5070

HYDROVEX

SELECTION, OPERATION & MAINTENANCE MANUAL:

VERTICAL HYDROVEX VALVE VHV SVHV



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INTRODUCTION

One of the major difficulties encountered in urban water-flow management is the runoff generated after a heavy rainfall. In the case of a storm water system excessive flows, if uncontrolled, may overload the system and result in flooding. Due to increased pipe velocities sewer pipe wear is increased dramatically and, as a result, network deterioration is accelerated. In a combined sewer overflow system the wastewater treatment plant may also experience significant increases in flows during storms, thereby resulting in a severe loss of treatment efficiency.

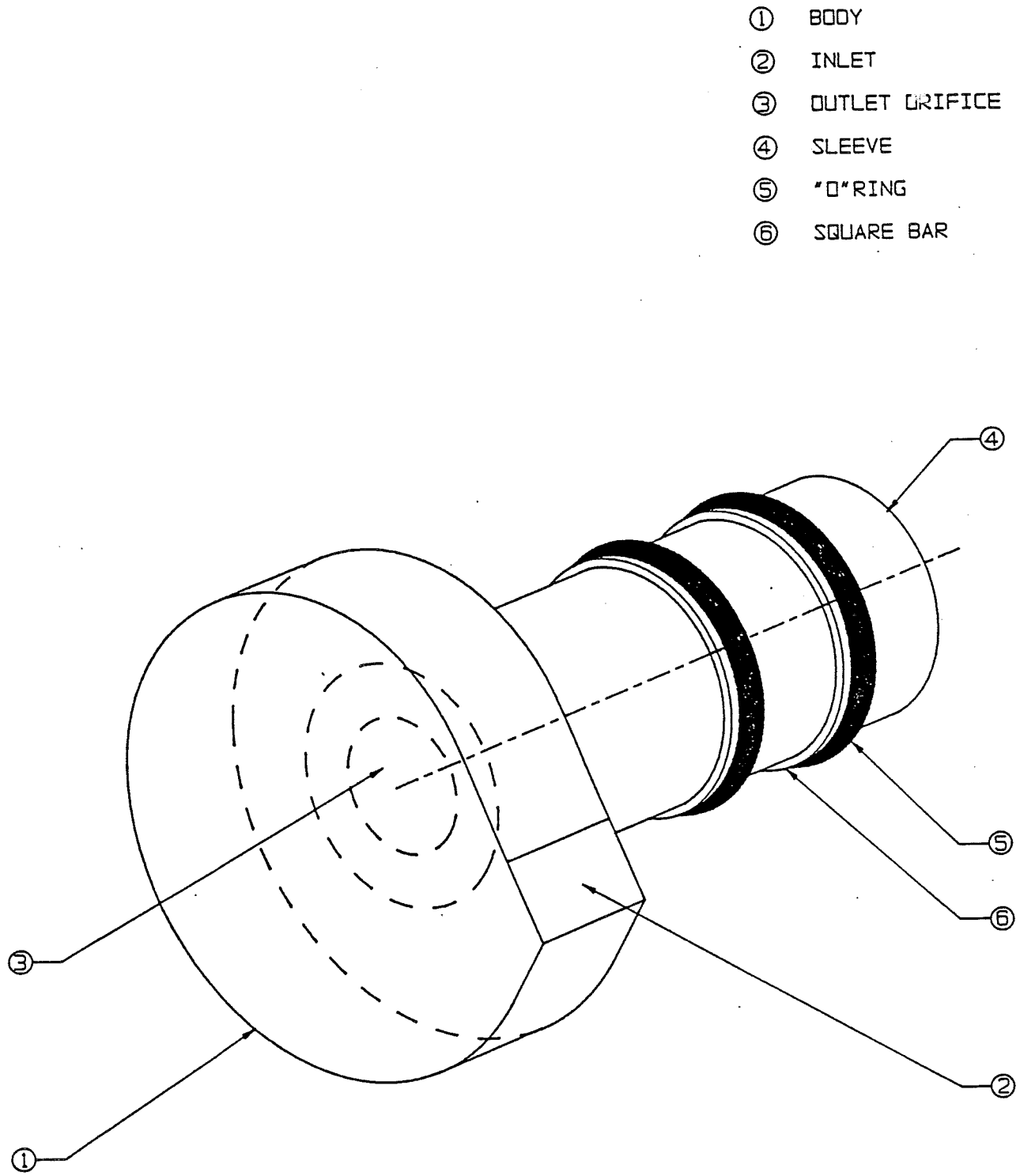
A simple means of controlling excessive water runoff is by controlling excessive flows at their origin (manholes). JOHN MEUNIER INC. manufactures the VHV line of vortex flow regulators to control stormwater overflows in sewer networks as well as manholes.

The vortex flow regulator is designed based on the fluid mechanics principle of the forced vortex. This permits flow regulation to be performed in the absence of any moving parts. As well, human intervention is unnecessary. The operation of the regulator is such that, depending on the upstream head and the discharge, it switches between orifice flow (gravity flow) and vortex flow, in order to control the downstream discharge. Although the concept is quite simple, over 12 years of research has been carried out, and still continues in order to further improve the performance.

The HYDROVEX Vortex Flow Regulator (refer to figure 1) is manufactured entirely of quality stainless steel, and comprises the regulator body (1) (in which flow control takes place) and an outlet sleeve (4) (which is welded to the body). Two rubber "O" rings (5) seal and retain the unit in place. Two stainless steel retaining rings (6) are welded onto the outlet sleeve, in order to ensure that there is no shifting of the "O" rings during installation and use.

"HYDROVEX"™ REGULATOR VHV.

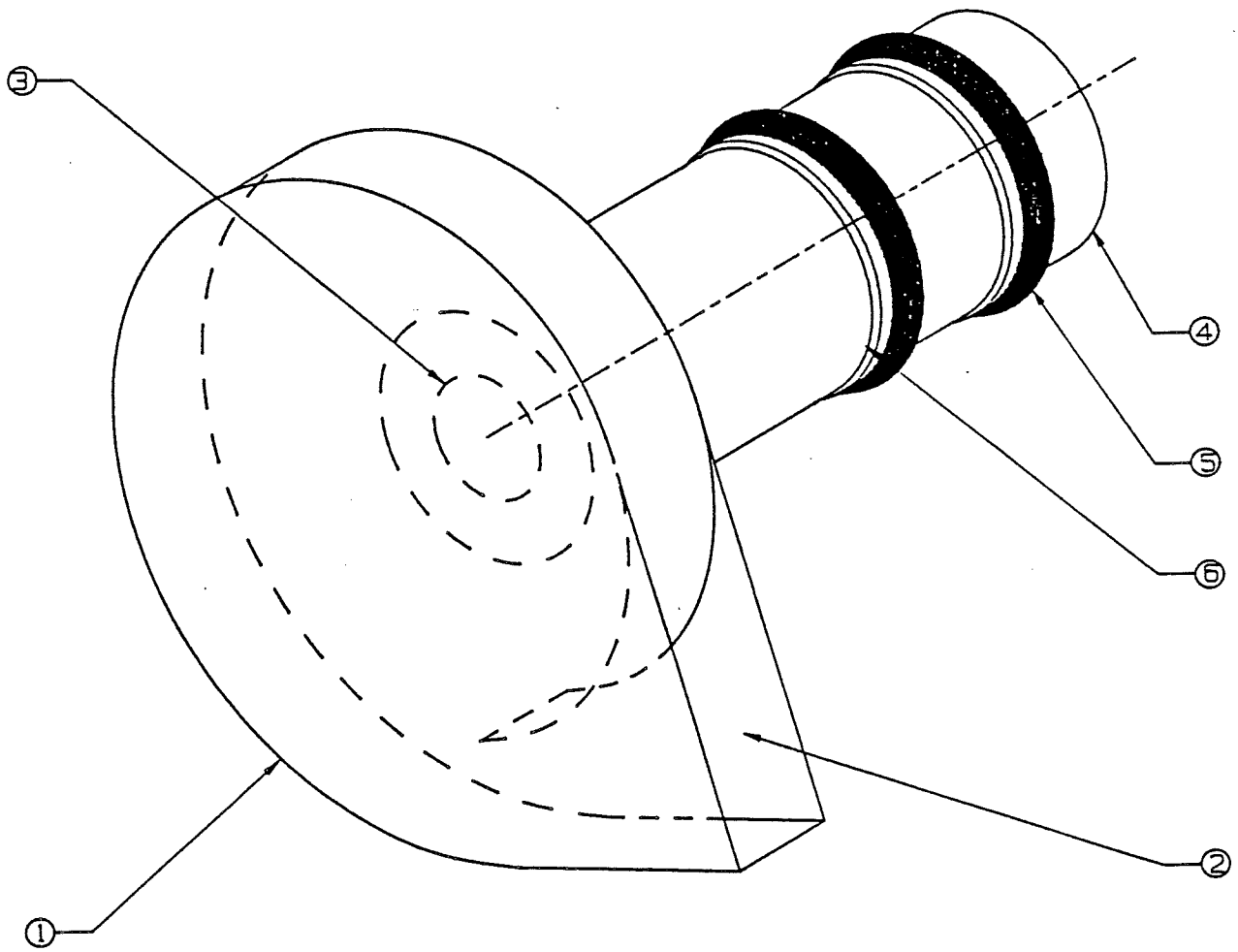
FIGURE 1-VHV



JOHN MEUNIER INC.

"HYDROVEX"™ REGULATOR SVHV.

FIGURE 1-SVHV



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ADVANTAGES OF THE HYDROVEX FLOW REGULATOR

- *The HYDROVEX line of flow regulators, being manufactured entirely of quality stainless steel, are very durable and corrosion resistant.*

- *Having no moving parts they require minimal maintenance.*

- *The geometry of the HYDROVEX flow regulator permits control equal to that of an orifice plate having a cross sectional flow area 4-6 times smaller. This decreases the chance of blockage of the regulator due to sediments and large debris found in stormwater flow.*

- *Installation of the HYDROVEX regulator is quick and straight forward, and is performed after all civil works are completed.*

- *Installation requires no special tools or equipment, and may be carried out by any contractor.*

- *Installation may be carried out in existing structures.*

HYDROVEX SELECTION CRITERIA VHV OR SVHV

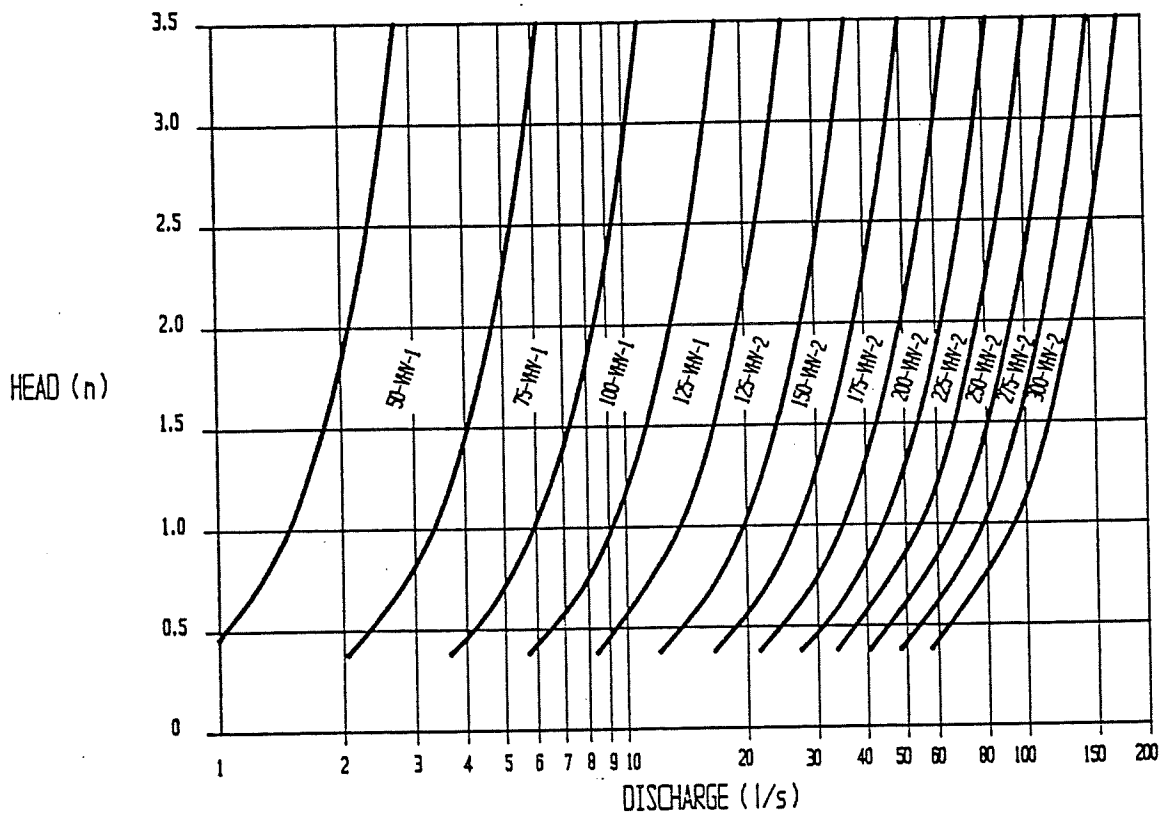
Figure 2 is a graphical representation of the head/discharge range for various VHV or SVHV regulators. This curve is a function of the maximum upstream static water pressure (head) and the discharge. As may be seen from the two curves, the SVHV models have larger cross-sectional flow areas than the VHV's for a given discharge and head.

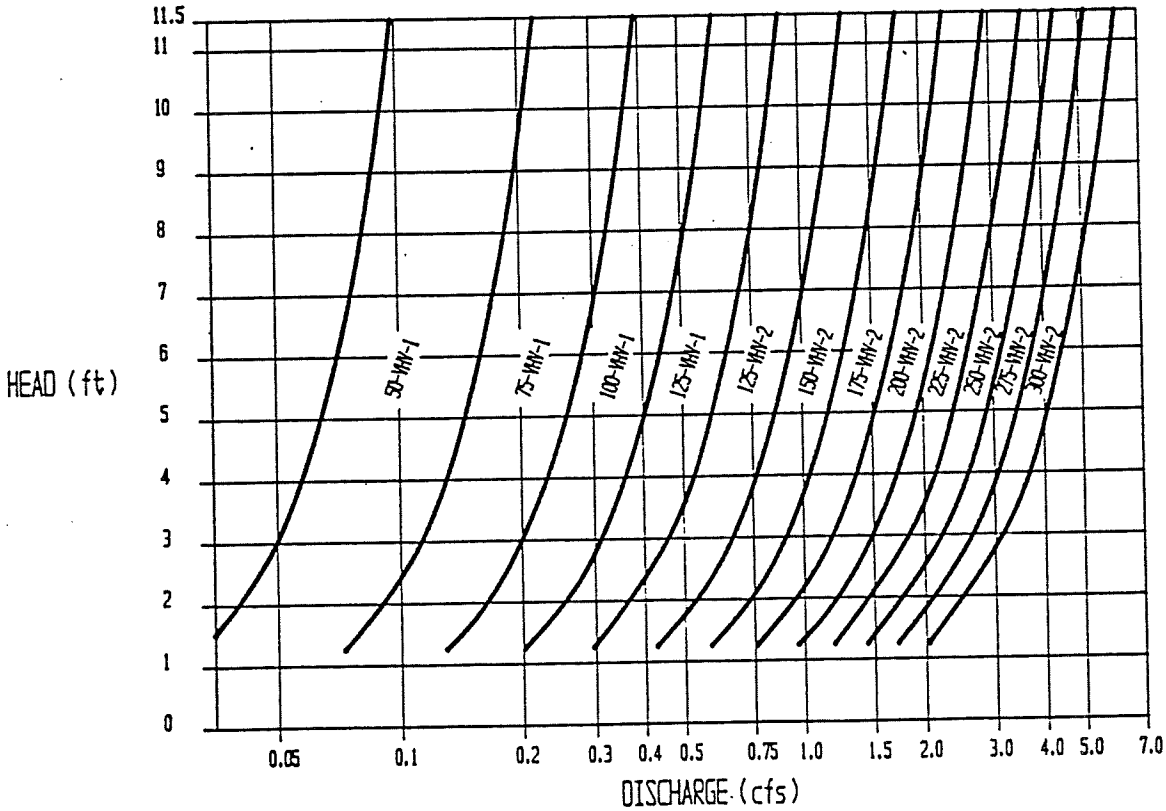
Model selection is performed by determining the maximum design flow at the manhole outlet and the maximum design head at the centerline of the manhole outlet pipe. Each regulator is applicable for operation within a range of head and discharge values, as illustrated by the graph. Using this information one can refer to Figure 2 and determine which model is applicable for a given application. All selection should be verified by JOHN MEUNIER INC. personnel.

Figure 3 shows the difference between flow regulator HYDROVEX model 100 SVHV-1-100 and an equivalent orifice plate. We can see that for the same head the flow regulator controls a discharge approximately four (4) times smaller than the orifice plate.

HEAD Vs DISCHARGE CURVES

FIGURE 2-VHV





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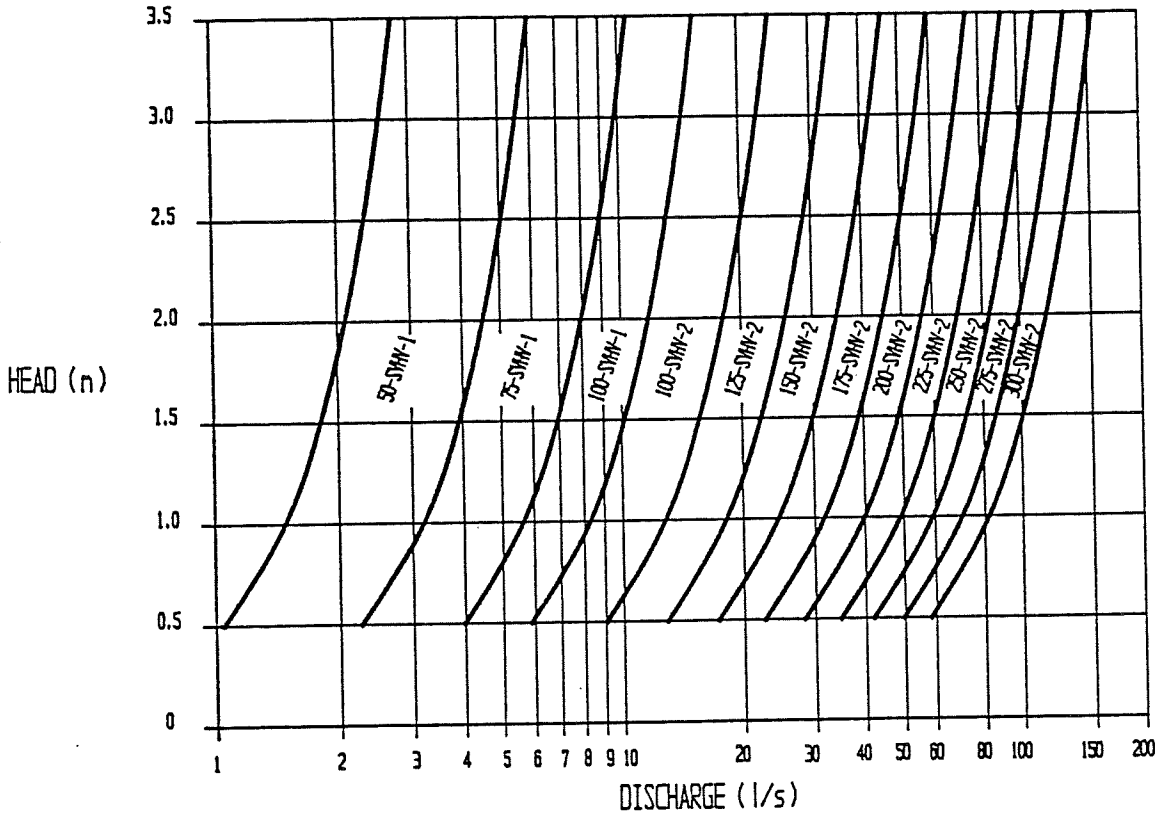
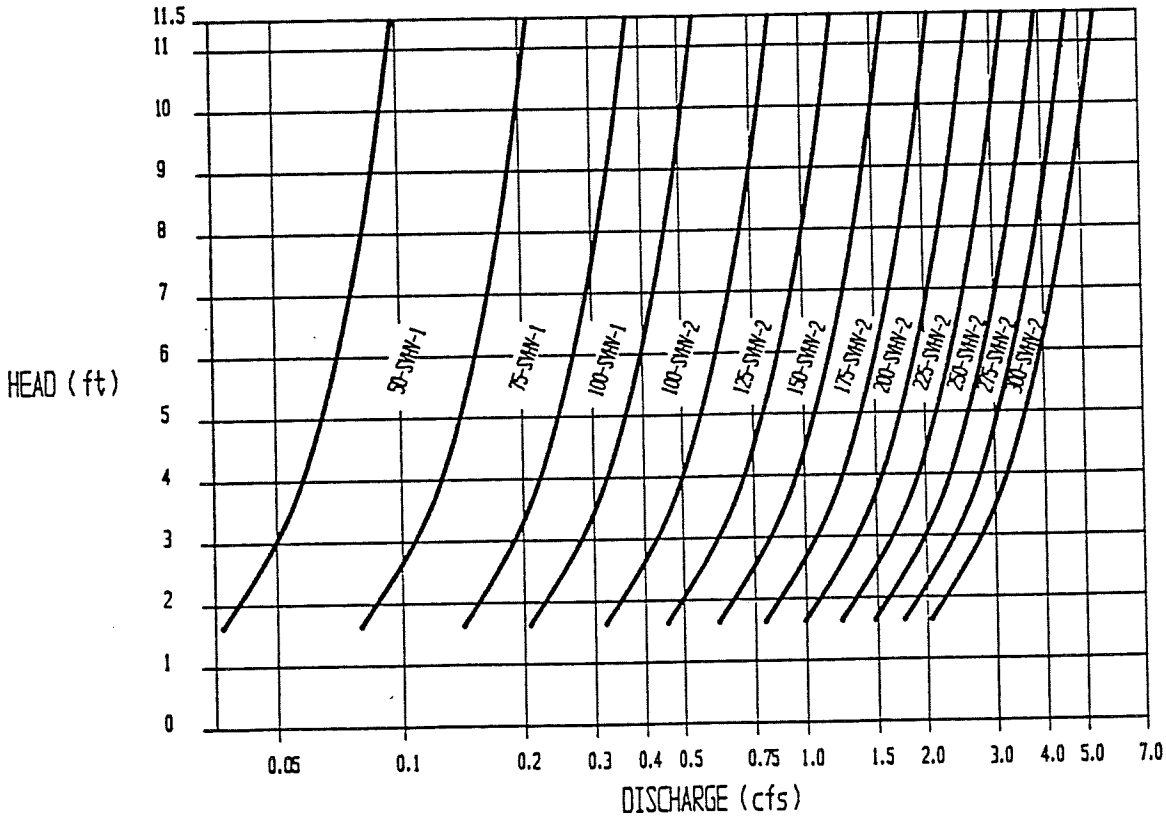
HEAD Vs DISCHARGE CURVES

FIGURE 2-SVHV





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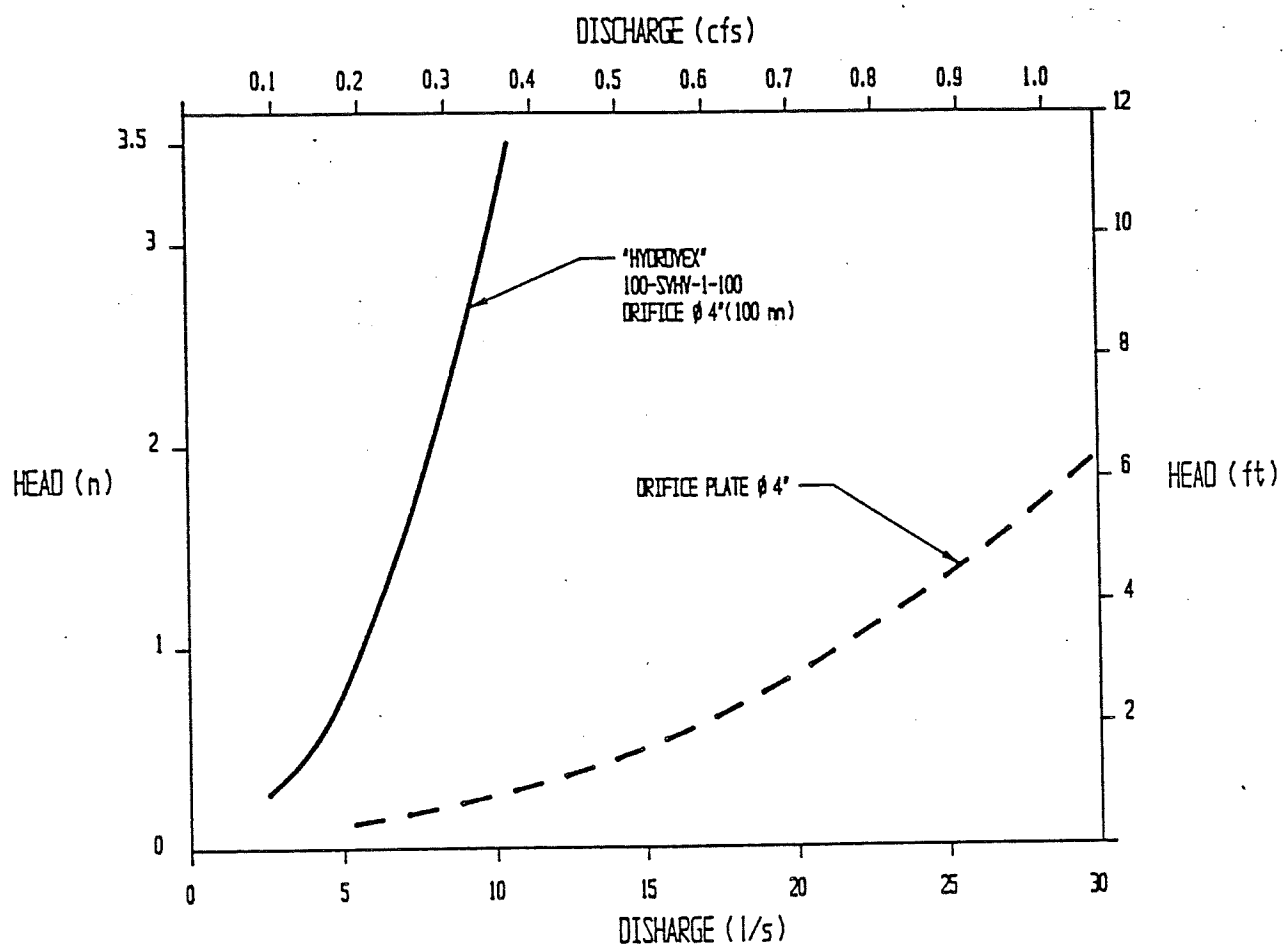


FIGURE 3

Discharge curves for a HYDROVEX 100 SVHV-1-100

and a 4" ϕ orifice plate.

DIMENSIONING OF MANHOLES

Most HYDROVEX models may be installed in a standard 36 inch diameter manhole. All models may also be installed in a rectangular manhole with a minimum dimension of 36 inches (24 inch diameter or rectangular manhole may be used for smaller models).

Note that in the case of a square manhole, be sure that the outlet flow pipe is in the center of the wall to ensure enough clearance for "A" dimension.
A minimum clearance "H" should be established between the floor of the manhole and the invert elevation of the outlet pipe, in order to permit installation of the regulator. Figure 4 gives the various dimensions required for a given regulator.

SPECIFYING THE HYDROVEX VORTEX FLOW REGULATOR

In order to specify a HYDROVEX regulator the following parameters must be defined:

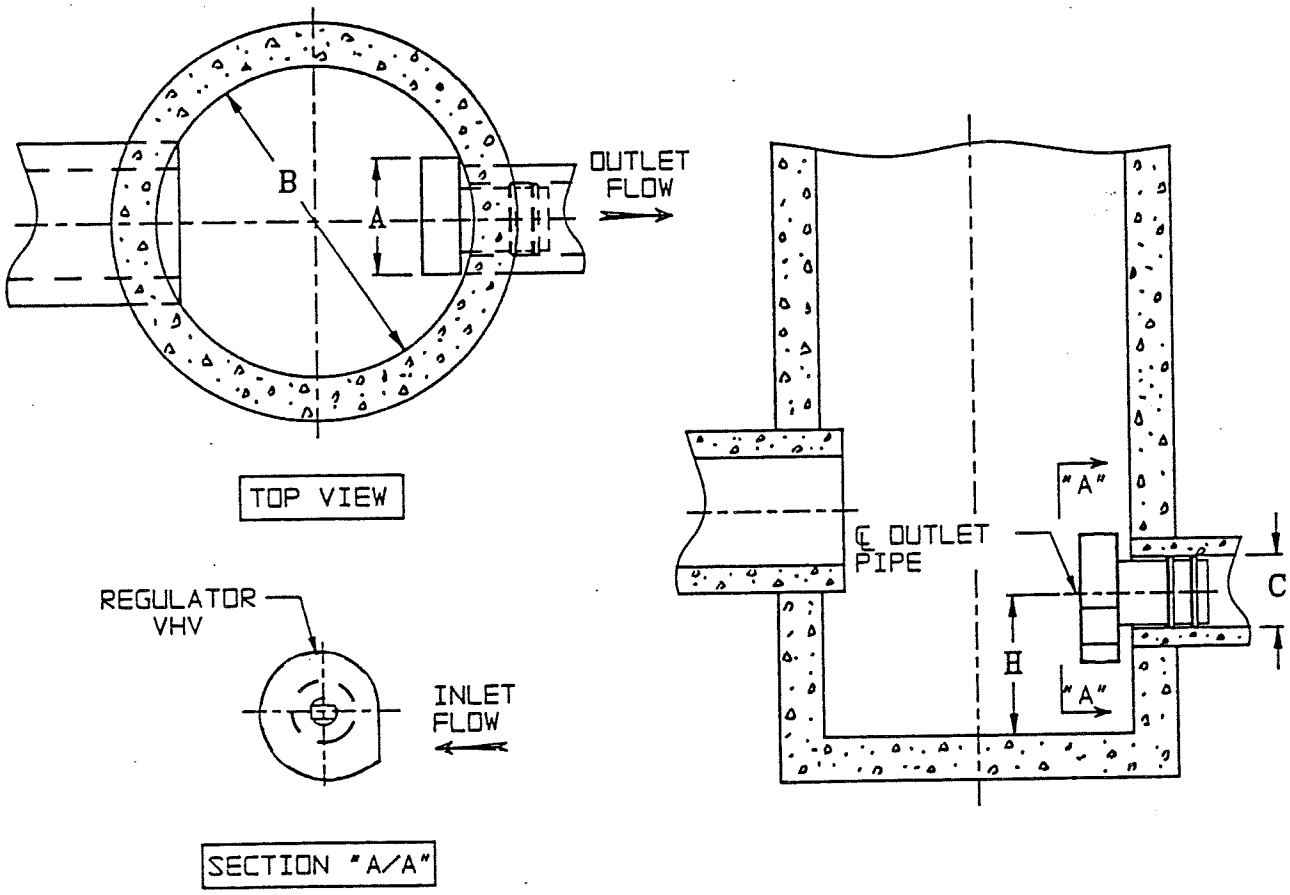
- *The model number (ex.: 100-VHV-1)*
- *The diameter and type of the outlet pipe (ex.: 6" dia. SDR 35 or 8" dia. RCP).*
- *The desired discharge (ex.: 50 l/s or 1.76 CFS).*
- *The upstream head (ex.: 2 m or 6.56 ft.).*
- *The manhole diameter (ex.: 36" dia.).*
- *The minimum clearance "H" (ex.: 10 inches).*
- *The material type (ex.: 304 s/s, 11 Ga. standard).*

REGULATOR TYPICAL INSTALLATION.

FIGURE 4-VHV

HYDROVEX Model No.	Regulator Width A (in.)	Manhole Diameter B (in.)	Outlet pipe * Diameter C (in.)	Minimum Clearance H (in.)
50-VHV-1	10	36	4	4
75-VHV-1	15	36	5	7
100-VHV-1	20	36	6	10
125-VHV-1	12	36	8	5
125-VHV-2	15	36	8	7
150-VHV-2	18	36	8	9
175-VHV-2	21	36	10	11
200-VHV-2	23	48	10	13
225-VHV-2	26	48	12	15
250-VHV-2	29	48	12	17
275-VHV-2	32	48	15	18
300-VHV-2	35	64	15	20

* Minimum recommended



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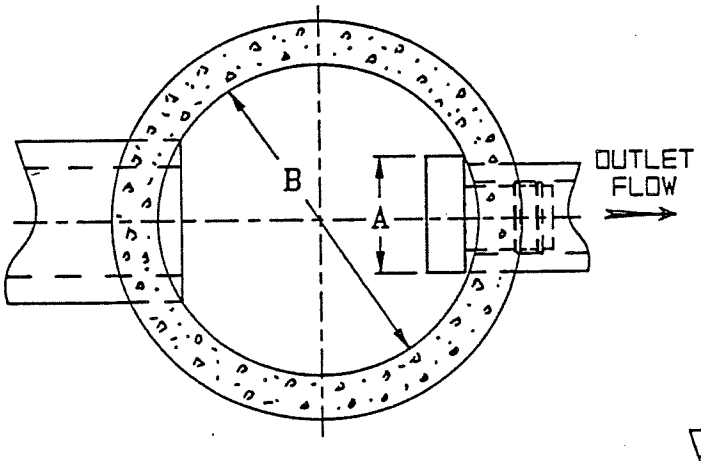
REGULATOR TYPICAL INSTALLATION.

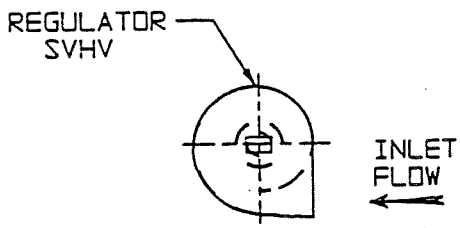
FIGURE 4-SVHV

HYDROVEX Model No.	Regulator Width A (in.)	Manhole Diameter B (in.)	Outlet pipe * Diameter C (in.)	Minimum Clearance H (in.)
50-SVHV-1	12	36	6	7
75-SVHV-1	17	36	6	7
100-SVHV-1	23	36	8	8
100-SVHV-2	15	36	8	8
125-SVHV-2	18	36	8	8
150-SVHV-2	22	36	10	9
175-SVHV-2	24	36	10	9
200-SVHV-2	27	36	12	10
225-SVHV-2	31	42	12	10
250-SVHV-2	34	42	15	12
275-SVHV-2	37	48	15	12
300-SVHV-2	41	48	15	12

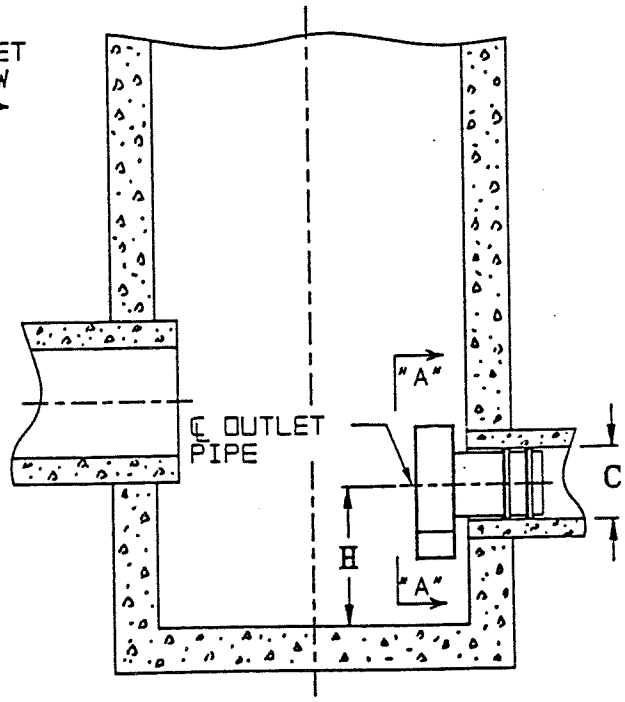
* Minimum recommended



TOP VIEW



SECTION "A/A"



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INSTALLATION

The installation of a HYDROVEX regulator may be undertaken once the manhole and piping is in place. Installation consists of simply fitting the regulator into the outlet pipe of the manhole. JOHN MEUNIER INC. recommends the use of a lubricant on the outlet pipe, in order to facilitate the insertion and orientation of the flow controller.

MAINTENANCE

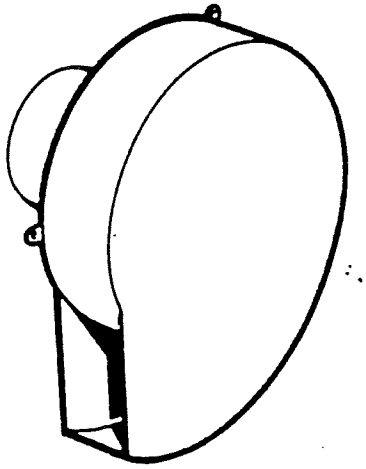
HYDROVEX regulators are manufactured in such a way as to be maintenance free; however, periodic inspection (every 3-6 months) is suggested in order to ensure that neither the inlet nor the outlet have become blocked with debris. The manhole should undergo periodic inspection and cleaning as established by the municipality.

GUARANTEE

The HYDROVEX line of regulators are guaranteed against both design and manufacturing defects for a period of 5 years. Should a unit be defective JOHN MEUNIER INC. is solely responsible for either modification or replacement of the unit, at their discretion.

EGUFLO VORTEX VALVES

TYPE SH

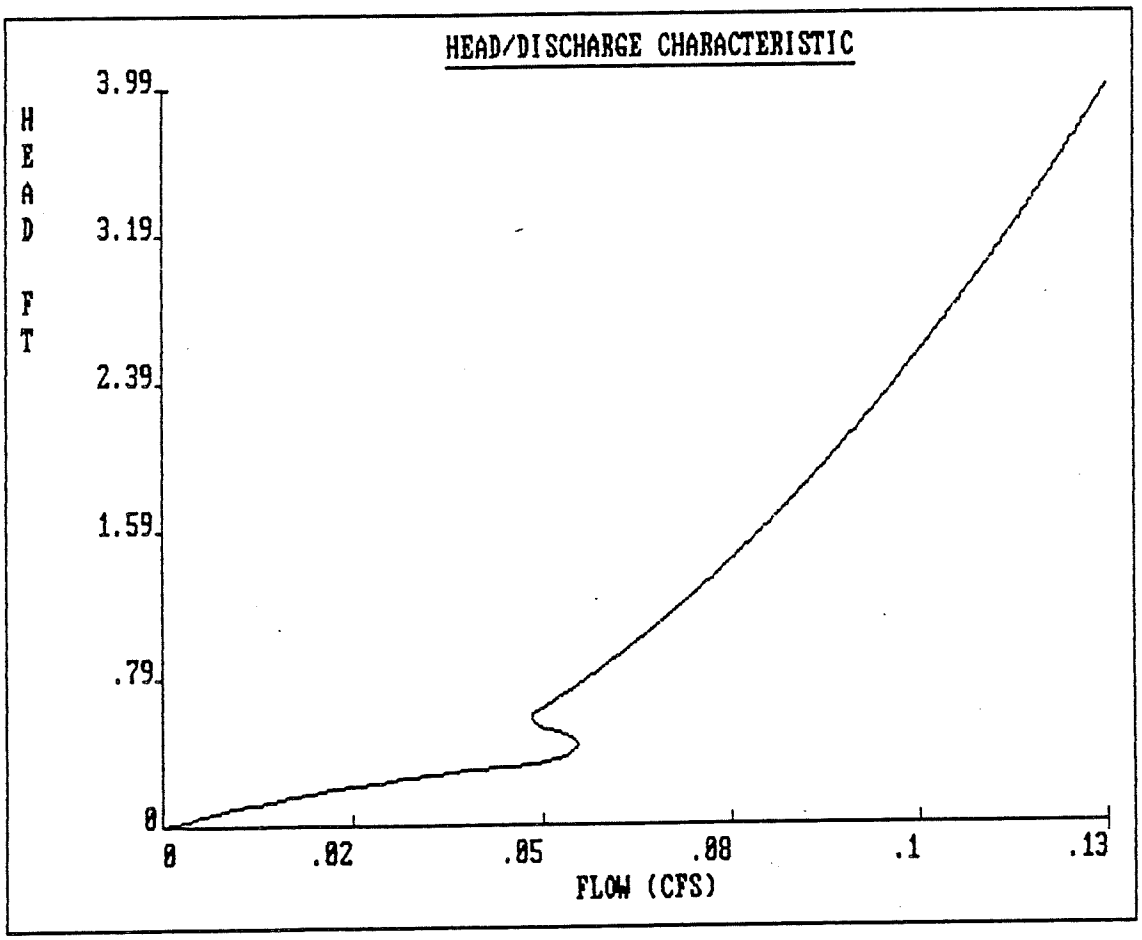


REG-U-FLO VORTEX VALVE DESIGN

JOB:- TYPICAL FLOW CURVES
CASE

REG-U-FLO VORTEX VALVE TYPE 3.00 SH
STANDARD UNIT MOUNTED HORIZONTAL
DIA OF OUTLET = 3.00 INCHES

HEAD (FT)	FLOW (CFS)	HEAD (FT)	FLOW (CFS)
0.200	0.024	2.200	0.101
0.400	0.058	2.400	0.105
0.600	0.053	2.600	0.109
0.800	0.060	2.800	0.114
0.999	0.068	3.000	0.118
1.200	0.074	3.200	0.121
1.400	0.080	3.399	0.125
1.600	0.086	3.600	0.129
1.800	0.091	3.800	0.132
1.999	0.096	3.999	0.136



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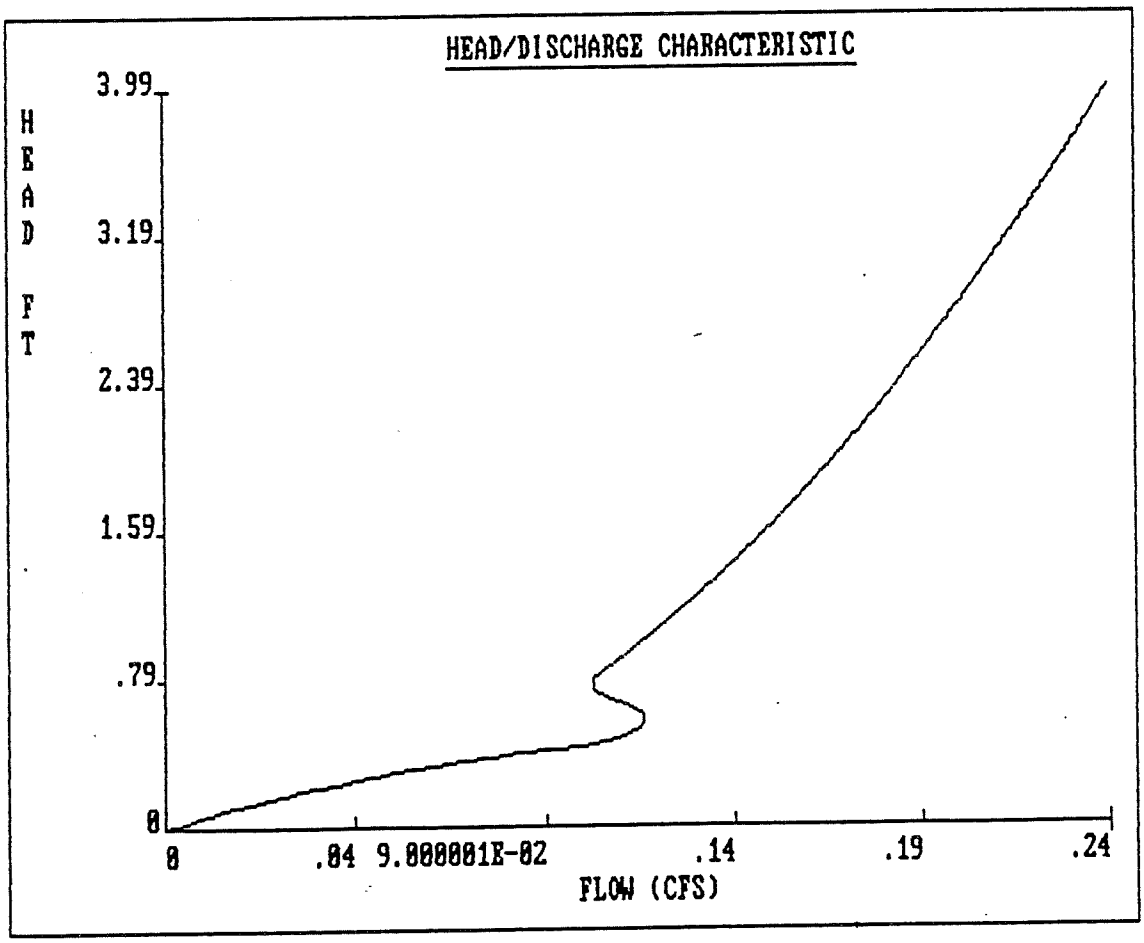
REG-U-FLO VORTEX VALVE DESIGN

JOB:- TYPICAL FLOW CURVES
CASE

REG-U-FLO VORTEX VALVE TYPE 4.00 SH
STANDARD UNIT MOUNTED HORIZONTAL
DIA OF OUTLET = 4.00 INCHES

HEAD (FT)	FLOW (CFS)	HEAD (FT)	FLOW (CFS)
0.200	0.034	2.200	0.179
0.400	0.090	2.400	0.187
0.600	0.122	2.600	0.195
0.800	0.108	2.800	0.202
0.999	0.121	3.000	0.209
1.200	0.132	3.200	0.216
1.400	0.143	3.399	0.223
1.600	0.153	3.600	0.229
1.800	0.162	3.800	0.236
1.999	0.171	3.999	0.242



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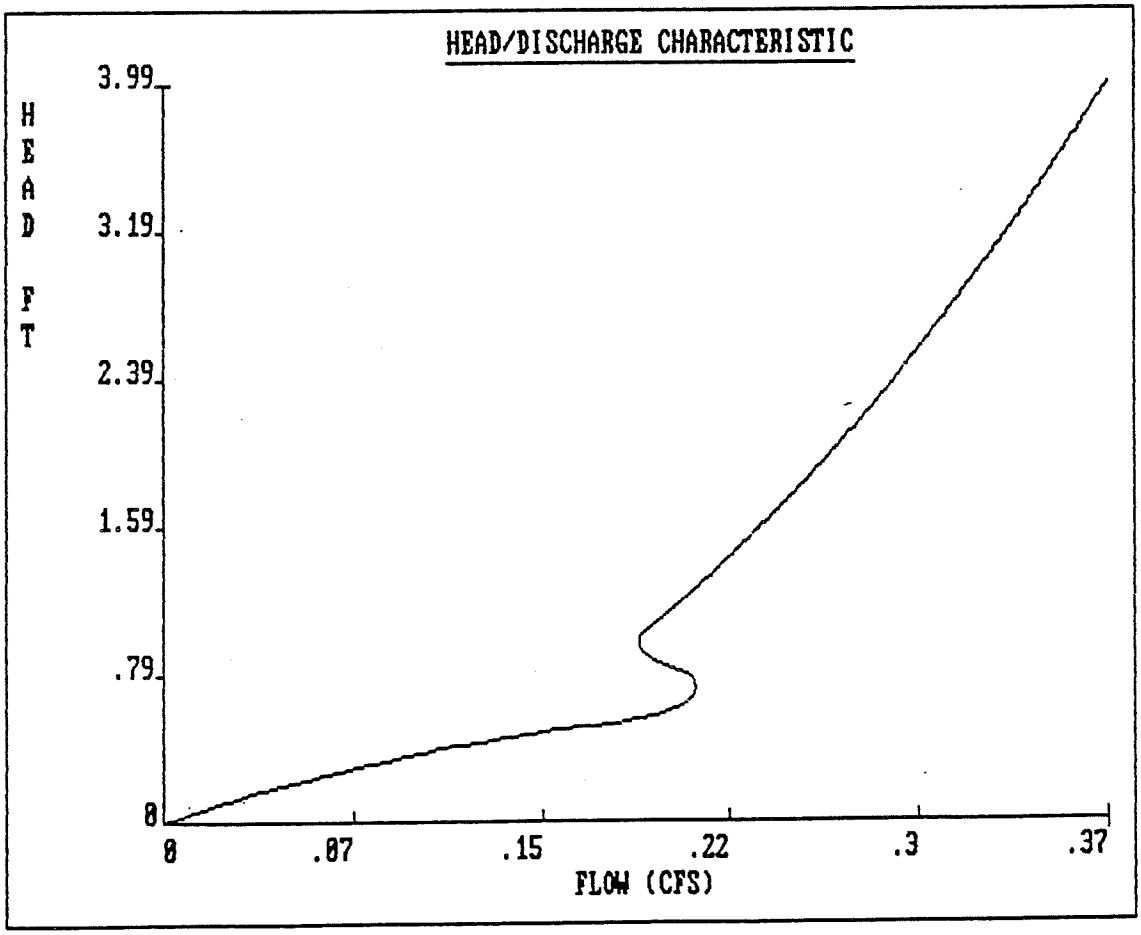
REG-U-FLO VORTEX VALVE DESIGN

JOB:- TYPICAL FLOW CURVES
CASE

REG-U-FLO VORTEX VALVE TYPE 5.00 SH
STANDARD UNIT MOUNTED HORIZONTAL
DIA OF OUTLET = 5.00 INCHES

HEAD (FT)	FLOW (CFS)	HEAD (FT)	FLOW (CFS)
0.200	0.046	2.200	0.280
0.400	0.110	2.400	0.293
0.600	0.201	2.600	0.305
0.800	0.209	2.800	0.316
0.999	0.190	3.000	0.327
1.200	0.207	3.200	0.338
1.400	0.223	3.399	0.349
1.600	0.239	3.600	0.359
1.800	0.253	3.800	0.368
1.999	0.267	3.999	0.378



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APPENDIX C
Detailed Design Analysis of Pipe Replacements and Consolidation Conduits

APPENDIX C
DESIGN ANALYSIS OF CONVEYANCE IMPROVEMENTS AND CONSOLIDATION CONDUITS

A. INTRODUCTION

Sections 5, 6 and 7 of this Improvement Plans technical memorandum (TM) describe the alternative improvement plans analyzed during the Phase 3 study. Each of these plans include conveyance improvements consisting of replacement pipes and/or diversion weirs with consolidation conduits, to meet one of four levels of design storm criteria. This Appendix provides a description of the approach and process whereby these conveyance improvements were developed.

All tunnel, storage and treatment alternatives in the Phase 3 analysis were developed using the existing EXTRAN input file piping and facilities as a template. First, corrections were made to the existing file, in order to improve stability in the numerical solutions and to update the information from the Phase 2 study. Then, this existing system file was edited, i.e. pipes were upsized or a relief pipes were added, to reduce flooding by means of replacing undersized conduits with larger conduits, or relieving the existing conduits with diversion weirs and a system of conduits to convey the diverted flow to tunnels, or storage or treatment facilities. The modeling approach and limitations for these conveyance improvements are discussed in this Appendix.

B. MODIFICATIONS TO EXTRAN SETUP FOR EXISTING SYSTEM

B.1. EXTRAN File Changes

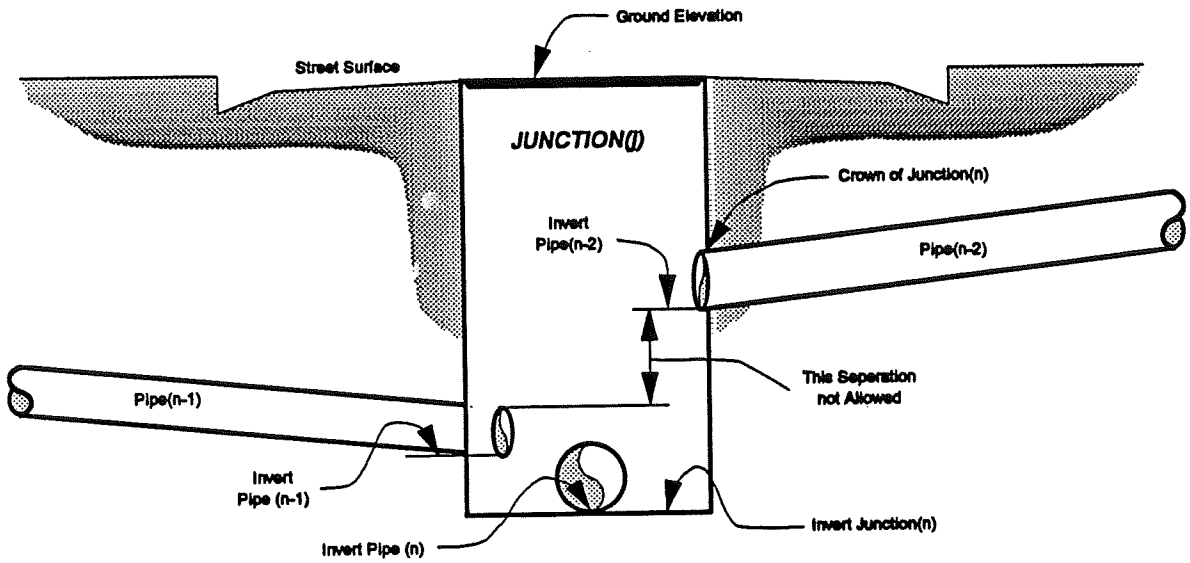
The EXTRAN model setup for the existing Combined Wastewater Collection and Treatment System (CWCS) was used as a base for all proposed alternatives. The CWCS model was updated before the modeling of the alternative plans. Many changes were made to the EXTRAN input file as a result of a quality control checks by the City and HDR to correct input data errors found in the SWMM (RUNOFF and EXTRAN blocks) developed during the Phase 2 study.

Additional changes were made to the EXTRAN files in order to increase the numerical stability of the model. One set of these changes was intended to avoid the difficulty the model has when travel times within a conduit are less than the iteration time step of the model. This problem occurs mainly in short pipe segments or very steep segments. During Phase 2, it was attempted to alleviate these problems by creating an equivalent pipe segment as suggested in the EPA's SWMM Manual. This was done by increasing the length of the pipe and decreasing the Mannings "n" value for the pipe. This did not help the situation significantly, since the conveyance capacity of the conduit remained the same, and consequently the travel time in the pipe did not change.

It was therefore decided that the short segments of pipe, typically less than 200 feet, would be lengthened to create a more stable model. This results in a tradeoff; by lengthening the pipe, the travel time and storage capacity of the conduits would not be accurate, however, these small losses in accuracy were necessary to have a dependable (i.e. hydraulically stable) model setup.

Another change made to the model was with respect to the "dummy" conduits used in Phase 2. Dummy conduits are used in order to avoid the following error message: "****ERROR****SURFACE AREA AT JUNCTION ___ IS ZERO". The surcharge level or junction crown elevation, as computed by EXTRAN, is defined as the crown elevation of the highest pipe connecting to a junction. The junction must not surcharge except when the water surface elevation exceeds the crown of the highest pipe connected to the junction. Figure C-1 illustrates an example junction and definition of terms. The junction would surcharge when the water surface is between the crown of pipe n-1 and the invert of pipe n+1, resulting in the above error.

It was recommended in the EPA's SWMM Manual that a dummy conduit be used to correct this situation. The configuration of this dummy conduit in the Phase 2 input files is shown on the top half of Figure C-2. During Phase 2, a false conduit was added which spanned from the top of pipe n-1 to the invert of pipe n+1. This false conduit then led to a false junction.



DEFINITION OF ELEVATION TERMS FOR A THREE-PIPE JUNCTION

HDR

Figure C-1

During the current Phase 3 study, the configuration of the dummy conduit was changed to that shown on the bottom half of Figure C-2. A new junction was added to which pipe n+1 was connected. A dummy conduit was then used to connect the new junction to the old junction to which pipes n and n-1 are connected. The lengths of these new dummy conduits was chosen to be 500 feet, in order to avoid the instability with short segments as discussed above. The diameters of the dummy conduits were that of pipe n+1.

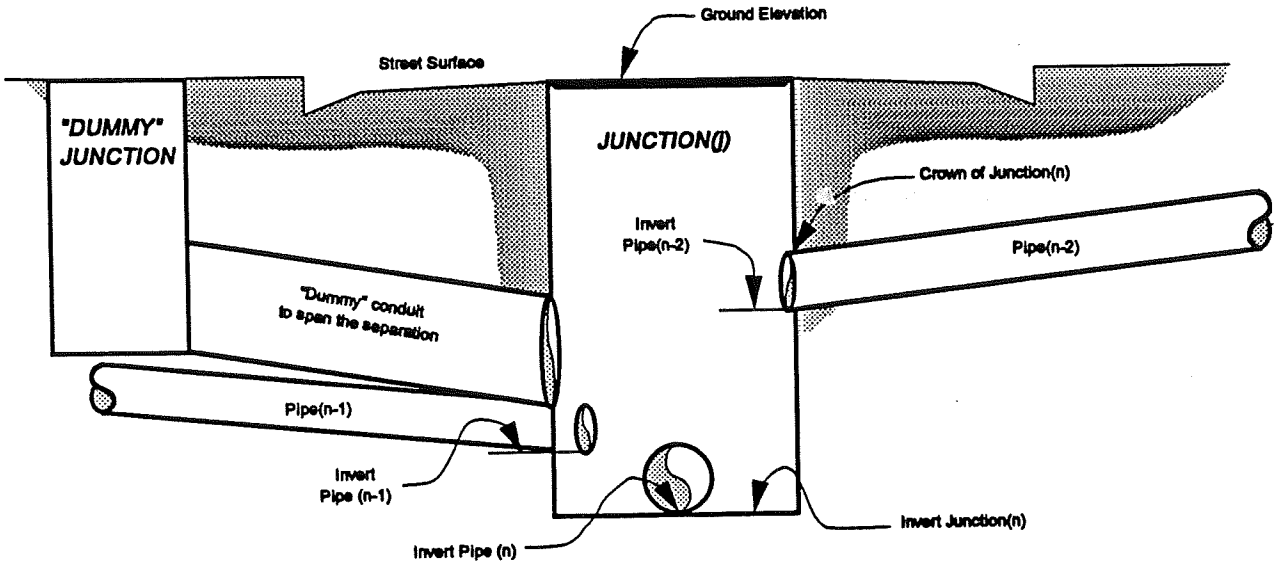
These dummy conduits are needed for model stability, however, they do add extra storage to the system. In summary, the total amount of extra storage is less with the Phase 3 configurations than the Phase 2 configurations, since the diameters of the dummy conduits are often smaller than those required in Phase 2 to span the separation between conduits.

B.2. RUNOFF File Changes

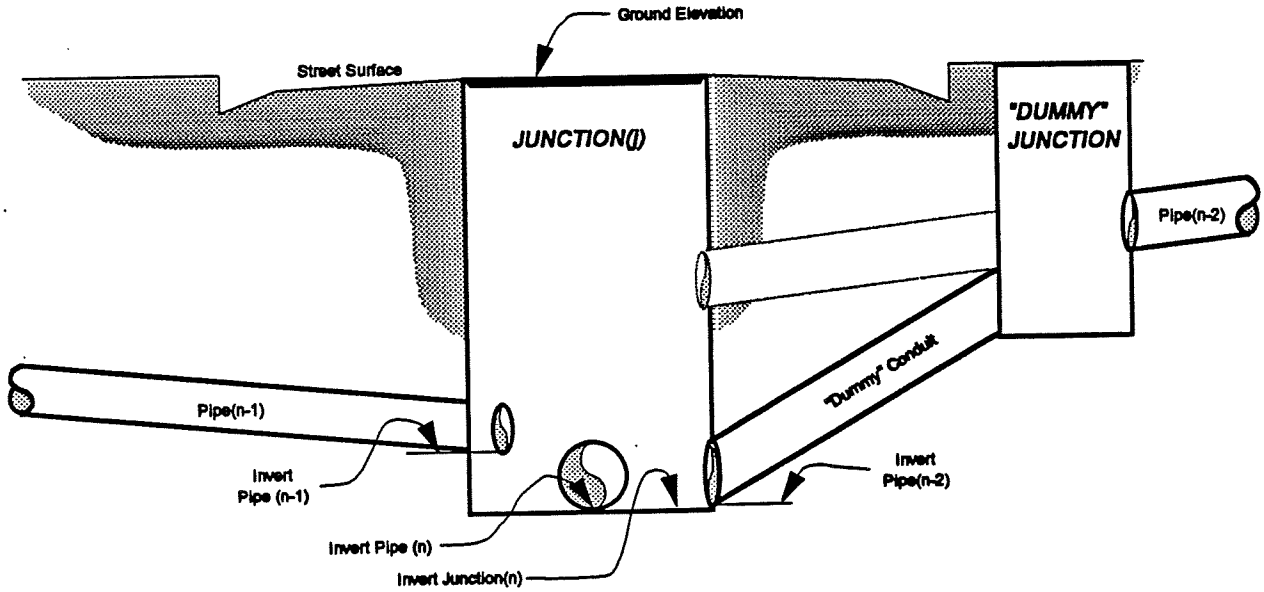
Changes were also made during the Phase 3 study to the RUNOFF files which create the hydrographs contributing to each junction. The design events, the infiltration rate of the soils in the basins, the impervious percentage of each basin, and the constant inflow into each basin were revised as discussed in Section 3 and detailed in the draft technical memorandum entitled Rainfall Analysis, Design Storms, and Flood Volume Maps and the City's TM, Revised SWMM Model Parameters.

C. FLOOD VOLUME ASSESSMENT

The first step in the model setup process for the simulating alternative improvement plans was to determine the cause of flooding and to estimate the best measure for controlling flooding. This process was facilitated by dividing the CWCS model for the entire service area, for existing conditions, into five separate sub-areas that contribute to major interceptors.



"DUMMY " CONDUITS AS MODELED IN THE PHASE 2 STUDY



"DUMMY" CONDUITS AS MODELED IN THE PHASE 3 STUDY

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Figure C-2

Each major interceptor was allowed to free outfall, so that there would be no downstream restriction to cause backwater. Dividing the system into major sub-areas was done to more quickly detect which change in the input caused the program to be instable or terminate. Since the model run times were much shorter for the individual sub-areas than the entire area, more flood control configuration options could be checked and compared for performance.

Next, each of the design storms was applied to each sub-area model, and flooding locations for each design storm were plotted in order to determine where major flooding problems exist and if flooding was due primarily to upstream or downstream control. Flooding problems as indicated by the SWMM model can result from several hydraulic situations, including:

1) The uppermost collectors in the system are typically severely undersized. Flooding results when the runoff inflow contributing to the upstream junctions exceeded the conveyance of these collectors.

2) A specific conduit may be undersized, which causes a hydraulic constriction. This results in water backing up the conduits and junctions, and then onto the streets.

3) A segment of the sewer network is shallow and cannot tolerate surcharged conditions, because the hydraulic grade line (HGL) would lie above ground.

4) The conveyance capacity of small collector conduits is reduced due to a direct connection to a major interceptors with high flows and a high HGL.

D. REPLACEMENT PIPES

A thorough discussion of the modeling and design of the replacement conduits can be found in Section 3, Subsection F.2.

E. DIVERSION WEIRS

Diversion weirs were used to relieve flows from undersized interceptors in the existing system, thereby lowering the hydraulic grade line and consequently reducing flooding. It was attempted to relieve flows only at central locations, where flows are concentrated, in order to reduce the number of weir structures, the length of consolidation conduits and number of tunnel drop shafts. In addition to the location of the weirs, other design considerations included the height and length of the weir. The following is a discussion of these parameters with respect to modeling.

The option for side diversion weirs in the SWMM model was used. The model input requires type of weir, the height of the weir measured from the invert of the junction, the top of the weir, and the length of the weir.

E.1. Weir Height

In practice, diversion weirs would be constructed as adjustable weirs which could vary the amount of water relieved from the existing system. The height of the weirs and their operation in reality would vary with respect to the intensity and duration of the storm and the initial conditions in the system, i.e., amount of storage available. The adjustable operation cannot be modeled by existing SWMM code, which only allows for a constant weir height.

In order to estimate the minimum weir height needed to contain the constant sanitary inflows, a runoff file with no rainfall was applied to the system to check the ratio of constant flow to design flow of the conduits. The model reported ratios typically from 0 to 0.10. This exercise indicated that in order to avoid spilling dry weather inflows into the tunnel system that the weir heights would need to be a minimum of 15 percent of the diameter of the conduit which corresponds to approximately 10 percent of full pipe capacity. Next an appropriate weir height as a ratio of conduit diameter was chosen for each design storm. Since adjustable weirs can not be modeled in SWMM the weir heights were chosen to accommodate the peak flow condition for each of the design storms.

The weir heights for the 10-year and 25-year events were set to one-third the diameter of the existing conduit from which flow was to be relieved. This height would assure the constant inflows would be contained and also would lower the height of the hydraulic grade line sufficiently. Weir heights for the 5-year event were set at 80 percent of the conduit diameter. Most weirs for the 2-year storm were removed or set to 80 percent of the conduit diameter.

The subroutine used in the current EXTRAN version of SWMM produced unstable numerical solutions under certain weir settings and hydraulic conditions as listed below.

1) Downstream surcharging of the diversion weirs creates the greatest modeling problem. Should the downstream conduit become surcharged, the water level will jump to the elevation of the weir, causing excessive flooding and large model continuity errors.

2) Should the height of the weir be set high such that the conduit is near full and a peak wave is applied to the conduit, the water surface elevation may jump dramatically due to the lack of storage volume in the pipe to attenuate the flow. This condition could not be handled by EXTRAN.

3) If several diversion weirs are placed along a conduit, the effect of the various weirs drawing down the hydraulic grade line may cause rapid flow reversals within the conduit and consequent continuity problems.

E.2. Weir Length

A sensitivity analysis was done to determine the most effective, "standardized" weir length. In this analysis, the length of the weir was varied; lengths of 6, 12 and 24 feet were used. The resulting rate of flow over the weir did not change significantly. The height of flow over the weir would drops the weir length increases, therefore, the area and volume of flow over the weir does not change significantly. For the particular junction used in the analysis, the HGL dropped 0.70 feet in lengthening the weir from 6 to 12 feet, and only 0.40 feet in lengthening from 12 to 24 feet, for the test case used in the sensitivity test.

The lengths of the weirs were consequently chosen more for constructabilty rather than for hydraulic effect. The weir lengths used in the model ranged from 10 to 30 feet, with the majority of the weir lengths being 20 feet. The weirs were assumed to be constructed as double-sided weirs, meaning a diversion would exist at each side of the conduit (see Figure 4-7). Therefore most weirs would be 10 foot long on each side of the conduit.

Additional information on the design criteria for diversion weirs can be found in Section 3, Subsection F.2.

F. CONSOLIDATION CONDUITS

The overflow weirs spill excess flows from the existing system into a network of consolidation conduits. These conduits convey combined wastewater to tunnels, storage facilities, or treatment facilities. The design of the consolidation conduits was in accordance with the criteria outlined in Section 3, Subsection F.4.

The conveyance criteria outlined in Section 3 required the new consolidation conduits to flow at approximately 80 percent of the total conduit capacity during the peak of the design event. This criteria proved to be critical as surcharging of the consolidation conduits results in unstable numerical solutions at the diversion weirs, as previously discussed.

In sewer design the conveyance capacity of a pipe depends largely on the diameter of the pipe and the slope at which it is installed. The trade-offs between increasing the slope of the pipe in order to decrease the needed diameter or conversely, using a larger diameter to decrease trenching depth was examined for cost optimization purposes. It was determined that to attempt to place the conduits shallow in order to avoid reduce trenching costs resulted in pipe diameters which required the same or more trenching depth. The slopes and diameters used for the conduit design are considered reasonable for the design flows.

Consolidation conduits for the storage and treatment alternatives collect and convey large flows, some of which have diameters exceeding 12 feet. It was assumed that consolidation conduits up to 12 feet would be constructed with open-trench construction techniques. Above that, the conduits were assumed to be constructed by tunneling.

As described further in Section 5, tunnels are subject to the construction limitation of using only a single diameter for an entire tunnel segment. To be cost effective, tunnel segments must be of constant diameter for a minimum of a mile in length. Consequently, where tunneling was required for consolidation conduit diameters greater than 12 feet, a long run of constant diameter tunnel was used.

G. ORIFICES

For the tunnel alternatives, consolidation conduits connect to the tunnel network through vertical drop shafts. The drop shafts are cost-intensive items, and were placed at central locations in order to reduce the number needed. The drop shafts were modeled as side orifices, which transfers flow from the end consolidation conduit junction to a tunnel junction. The best hydraulic representation of this would be a bottom drop orifice, however this option in SWMM was not functional.

The cross-sectional area of the orifice used for drop shafts was selected as twice the collective cross-sectional area of the consolidation conduits contributing flow to the drop shaft. This cross-sectional area was chosen to ensure that the headloss through the orifice would be minimal, and would consequently more closely represent the transfer of water to the tunnel through a drop shaft. A schematic of the weir, consolidation conduits, and orifices is shown in Figure 5-2.

H. QUALITY CONTROL

Throughout the modeling of alternatives the following quality control checks were exercised. These checks are given below.

1) The connectivity summary of the conduits was checked to ensure that the model setup was correctly.

2) The error in continuity was checked with each run to be certain that a numerically stable solution was obtained.

3) All conduits were checked for excessive velocities, which is an indicator of unstable numerical solutions.

4) The water surface elevations for junctions along major interceptors were checked to ensure that severe oscillations in the water surface elevation did not exist. This phenomena is another indicator of numerical solution problems in the model.

5) The alternatives were also checked for adherence to the design criteria presented in Section 3 of the TM.

APPENDIX D
Detailed Description of Tunnel Hydraulic Analysis

APPENDIX D
DETAILED DESCRIPTION OF TUNNEL HYDRAULICS AND DYNAMICS

A. INTRODUCTION

The main Improvement Plans technical memorandum focuses on obtaining the least-cost improvement plan, therefore, the modeling results discussed in the report concentrate on the cost-related aspects. The purpose of this Appendix is to give some insight into the hydraulic behavior of the tunnels, and the related aspects which influence the least-cost optimization process. As an introduction, the tunnel model development is reviewed. This is followed by a detailed evaluation of the tunnel dynamics and hydraulic behavior.

B. TUNNEL MODELING

B.1. Inflow to Tunnels

Inflow to the tunnels is controlled by the diversion weir/consolidation conduit system, as discussed in the main TM and Appendix C. Flow enters the tunnel through drop shafts, which were modeled as side orifices between the consolidation conduit system and the tunnel.

B.2. Tunnel Storage and Conveyance

Tunnels serve as both a storage and a conveyance system. Unlike storage reservoirs, the available storage capacity within a tunnel system is spread throughout the drainage area. The use of tunnels minimizes the extent of the consolidation conduit system, by reducing both the need for and extent of relief or consolidation conduits. The tunnel must, however, provide conveyance capacity to route the tunnel inflow to the available storage within the system. For flood control purposes, tunnels can also be configured with a discharge control station, from which excess flow that cannot be stored in the tunnel is discharged to the river.

The layout and sizing of a tunnel system must satisfy both storage and conveyance requirements. The model development process consisted of setting up the model, testing it for specific conditions to build confidence in the modeling results, and formulating the design requirements applicable to the tunnels. The following aspects were considered:

1) The utilization of the available storage within the system prior to discharge to river.

2) Conveyance limitations and the requirements to provide hydraulic stability in the system. In particular, wave action within the tunnel, extreme flow velocities, steep hydraulic grade lines (thus conveyance limitations) and tunnel water elevations.

3) Tunnel surcharging and the effect on the upstream consolidation system.

4) The tunnel system configuration, and its influence on hydraulic stability and storage utilization.

The results obtained in analyzing the above lead to the formulation of specific tunnel design and modeling guidelines, which are outlined Section 5, Subsections C.1 to C.6. By using these design guidelines, a hydraulically stable tunnel system could be configured that would allow for optimum storage utilization and peak flow conveyance capacity for the applicable design storm event.

B.3. Tunnel Discharge Control

As outlined in the main TM, the approach used in determining the least-cost tunnel alternative included: 1) maximizing storage capacity and evaluating the influence of tunnel diameter on the total project cost; and 2) evaluating the cost implication of reducing the tunnel discharge control elevation for a particular tunnel configuration. These two approaches were adopted as approximations of the City's requirements with regard to CSO and flood control as defined in the TM. The following discussion reviews some aspects considered in the selection of model configurations to simulate the tunnel discharge control

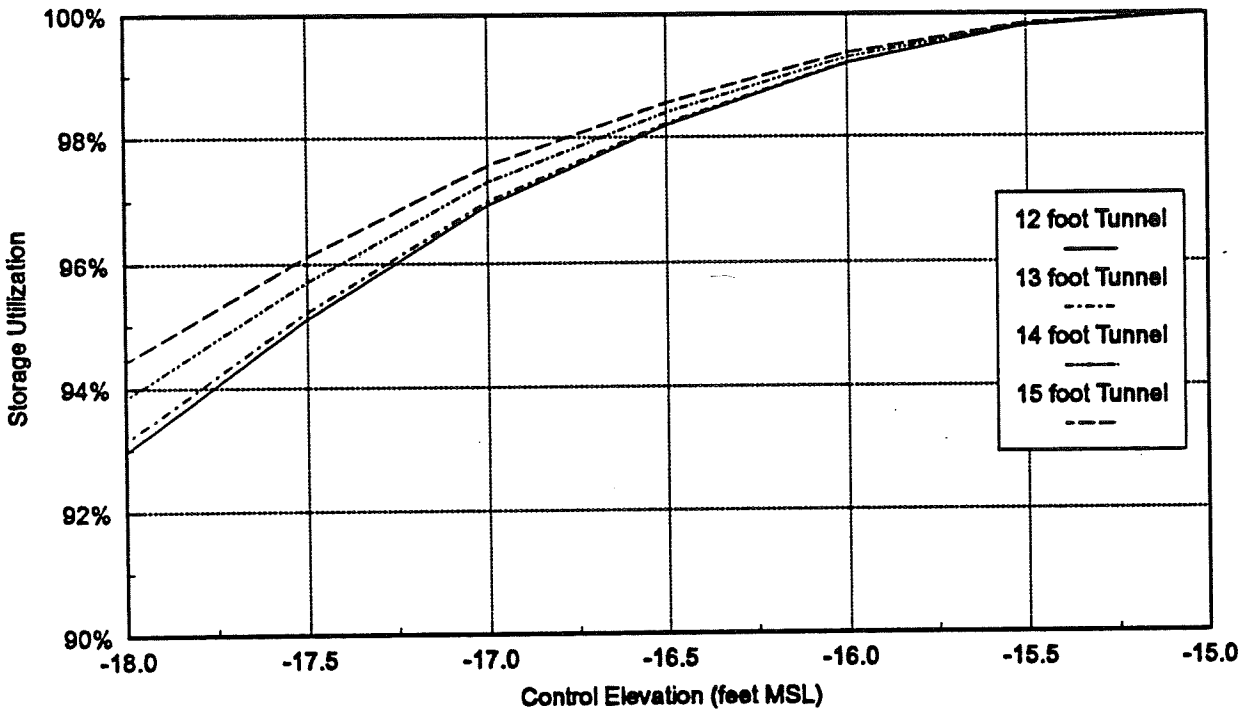
system as defined in the report. In this context, tunnel discharge refers to the discharge of untreated combined wastewater to the river during a storm event.

B.3.a. Maximized Storage Evaluation--

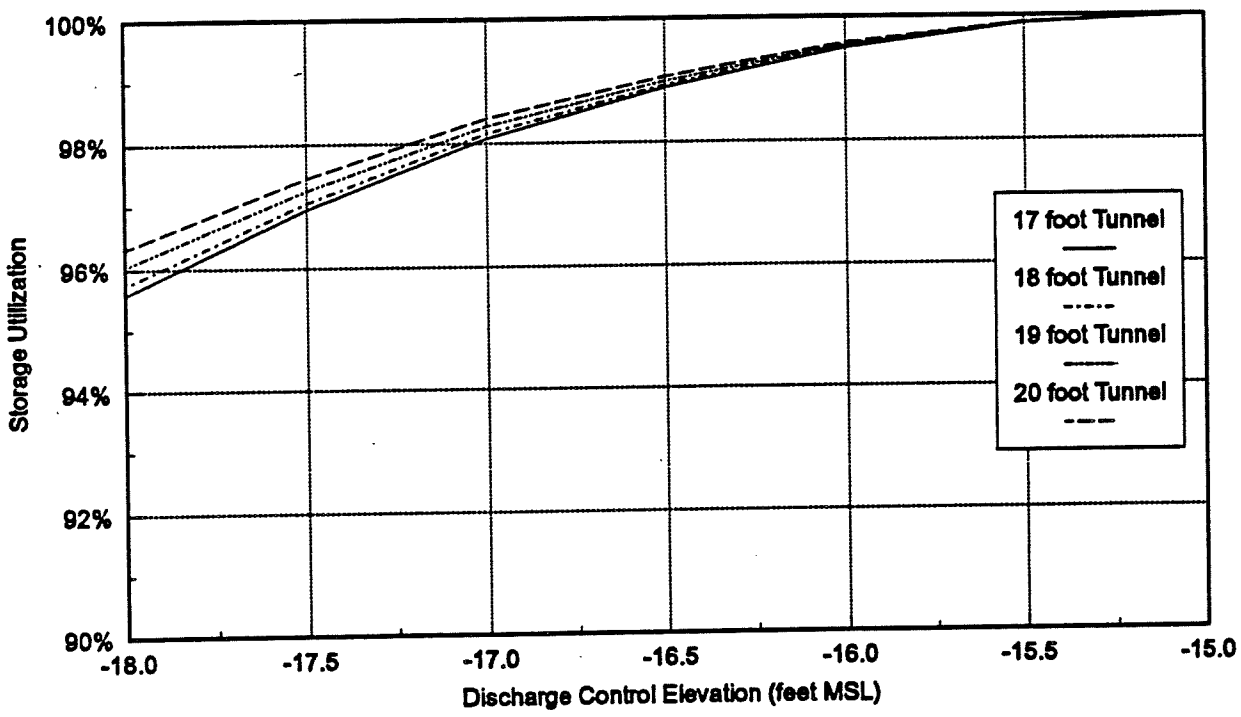
The objective for the maximized storage evaluation was to model discharge form the tunnel when the water elevation in tunnel reached the highest possible discharge control elevation, or when the tunnel is filled to its upper most crown elevation. This approach ensures that the storage volume is completely utilized prior to discharging to the river. By comparing combination of tunnel diameter and discharge rates, the least-cost tunnel diameter configuration could be established. The cost comparisons were based on the tunnel construction cost and the tunnel discharge pumping station cost.

The maximum storage capacity in the tunnel can be obtained by setting the discharge control elevation equal to the uppermost crown elevation in the tunnel system (-15.00 feet MSL). This proved to be an unworkable option, because the surcharging of the tunnel results in unstable hydraulic behavior. Discussion with municipalities operating similar systems indicate that this is a practical limitation which should be addressed in the tunnel design. In order to overcome this problem, a sensitivity analysis was conducted to determine the maximum control elevation at which hydraulic stability could be obtained, even under the most extreme conditions. Several approaches to model the tunnel discharge control were also tested.

An evaluation of storage utilization with the discharge control elevation below the uppermost crown elevation indicated that, even with the control elevation down to 3.0 feet below the crown elevation (-18.00 feet MSL), the available storage utilization efficiency is above 92 percent for tunnel diameters down to 12 feet. This is illustrated in Figure D-1. For control elevations below -18.00 feet MSL, the storage utilization efficiency decreases rapidly. It was concluded that the minimum control elevation for maximized storage configuration would be taken as 3.0 feet below the uppermost crown of the tunnel (i.e. at -18.00 feet MSL).



Storage Utilization for 5-Year Tunnel with Varying Tunnel Diameter Configurations



Storage Utilization for 10-Year Tunnel with Varying Tunnel Diameter Configurations

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Figure D-1

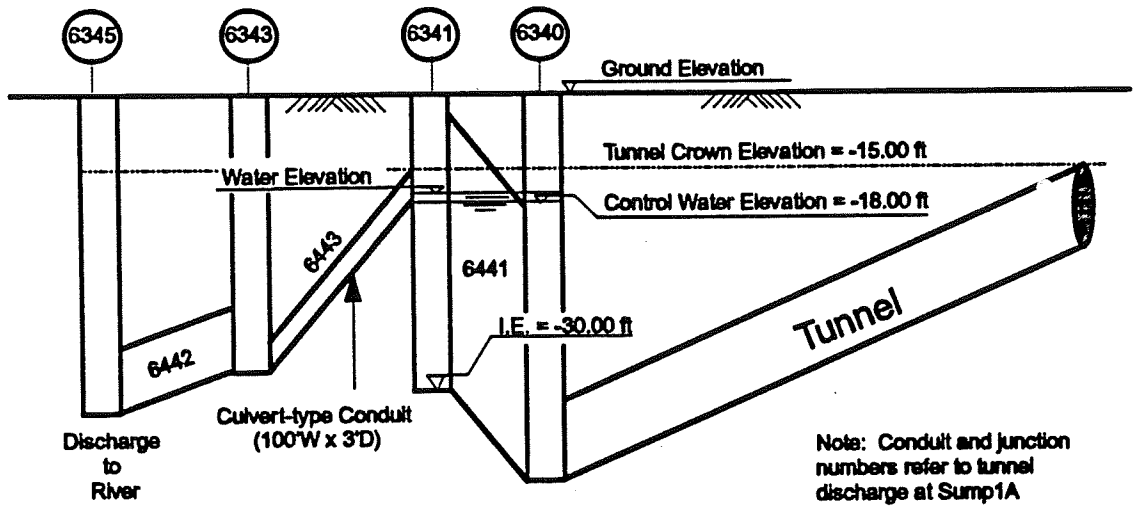
For modeling the tunnel discharge control, which is defined as the rate of discharge and the level at which discharge occurs, three type of facilities were investigated: 1) control weirs; 2) pump systems; and 3) a culvert type discharge conduit. Control weirs, similar to those used in the modeling of the diversion weirs, were tested for modeling the tunnel discharge control system. Control weirs proved to be too sensitive to upstream water level fluctuations, and numerically stable model solutions for maximized storage alternatives could not be obtained. A pump system for discharge control was possible, however, since the tunnel discharge rates were unknown, the selection of a site specific pump system was difficult. This option was ruled out for the maximized storage approach. Pump discharge control systems were, however, used for evaluating the effect of decreasing discharge control elevations, as discussed below.

The culvert type tunnel discharge control, which is illustrated in Figure D-2, proved to be the more general approach that provided both stable hydraulics and numerical modeling solutions. It also provided flexibility, since the model does not require modifications to the discharge control settings for various modeling applications. Tests with this model configuration proved that, by setting the control elevations at 3.0 feet below the uppermost crown elevations of the tunnel (-18.00 ft MSL), stable solutions could be obtained even when tested under the most extreme conditions. This modeling approach was used for all subsequent modeling of tunnels alternatives having maximized tunnel storage configurations.

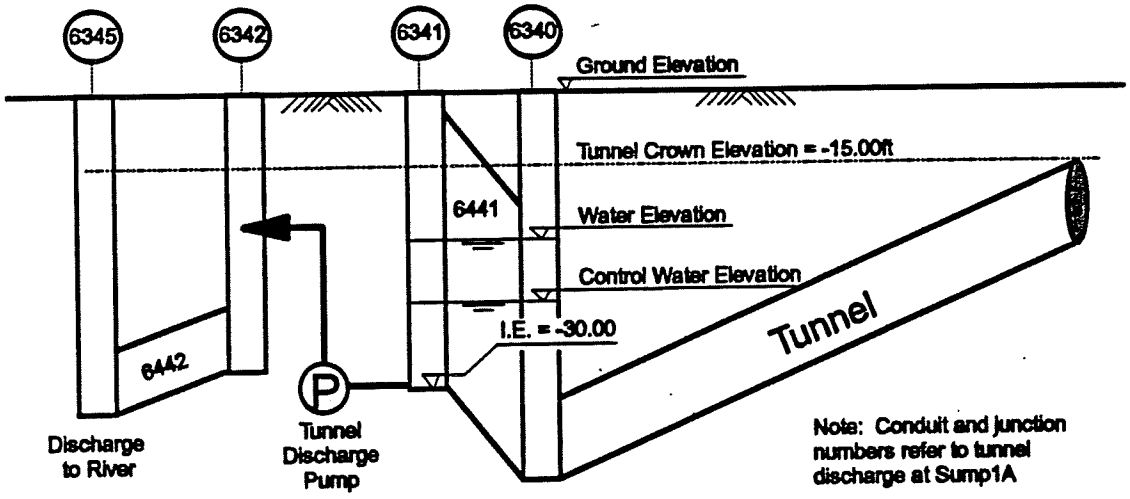
B.3.b. Tunnel Discharge Control Elevations--

The objective of evaluation the tunnel discharge control elevations was to determine the effect of tunnel discharge control elevation variations on the discharge pumping requirements, and thus on the project cost.

Maximum discharge rates from the tunnel to the river for various tunnel configurations were determined from the maximized storage modeling. Tunnel configurations with discharge control for elevations below the maximized storage elevation of -18.00 ft MSL were modeled as pump systems, as shown in Figure D-2.



SCHEMATIC ILLUSTRATION OF THE TUNNEL DISCHARGE CONTROL AS MODELED FOR THE MAXIMIZED STORAGE ALTERNATIVES



SCHEMATIC ILLUSTRATION OF THE TUNNEL DISCHARGE CONTROL AS MODELED FOR THE CONTROL ELEVATION EVALUATION ALTERNATIVES

HDR

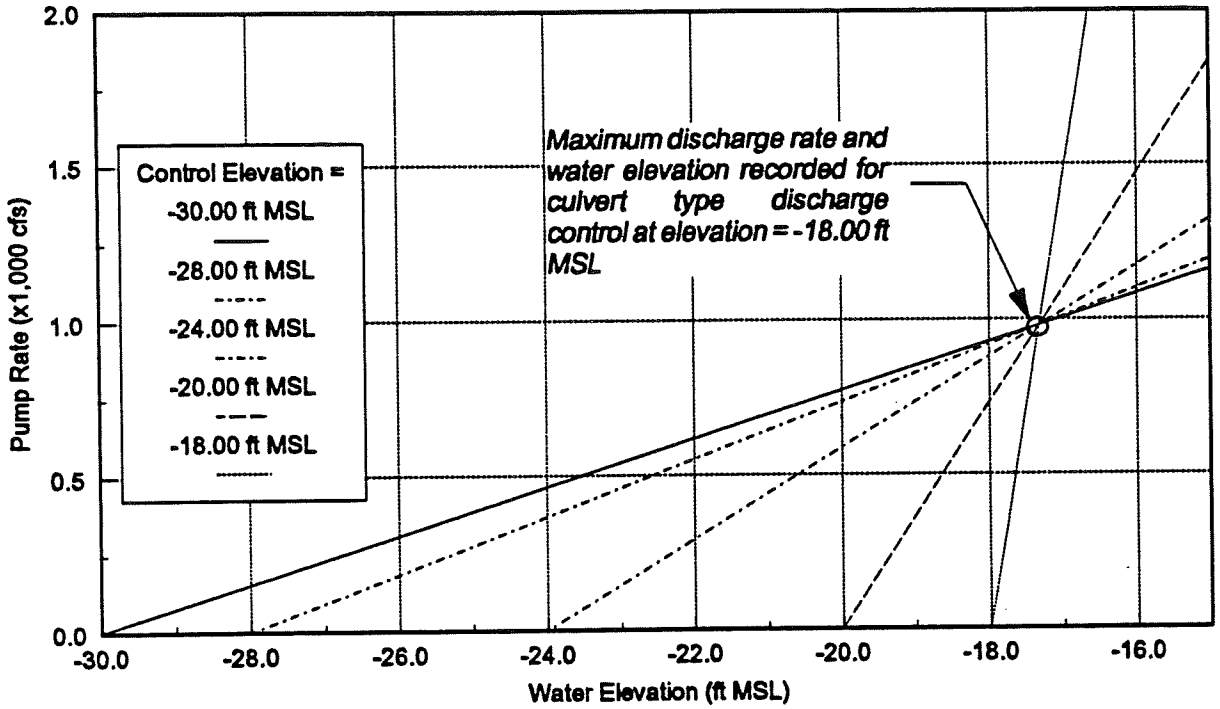
Figure D-2

The pump curves discussed in the main TM provide a linear increase in pumping rate with an increase in the tunnel water elevation of water elevations above the discharge control elevation. For tunnel water elevations below the control elevation, the pumping rate was set equal to zero. The slope of the pump curves were set by two operating points. One was a discharge rate at the outlet control elevation. The other was the maximum discharge rate for that a particular tunnel configuration using the maximized storage model. This is shown in Figure D-3.

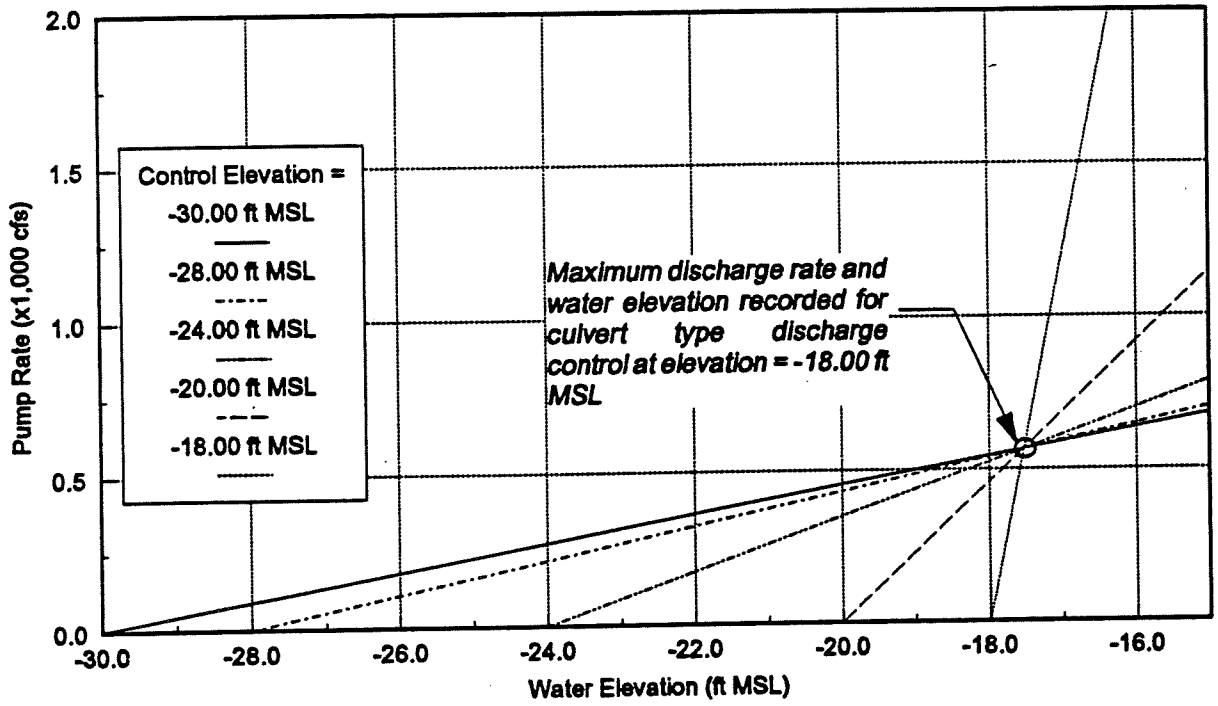
The advantage of using this tunnel discharge modeling approach is that it allows for the attenuation of the discharge peak in the volume available above the discharge control elevation). The tunnel pump requirements were taken to be the maximum rate of discharge to the river for the storm event considered.

B.4. CSO Control Modeling

In the existing CWCS system, Pioneer Reservoir overflow is discharged to the river. For CSO control purposes, this excess flow was diverted to the tunnel instead. Therefore, a CSO occurred from the tunnel system alternatives only when the storm event was large enough to cause a discharge from the tunnels to the river.



Pump Curves Selected for the Sump1a Discharge Pumps of the 17 foot 10-Year Tunnel Configurations.



Pump Curves Selected for the Sump2 Discharge Pumps of the 17 foot 10-Year Tunnel Configurations.

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Figure D-3

C. MODEL HYDRAULICS

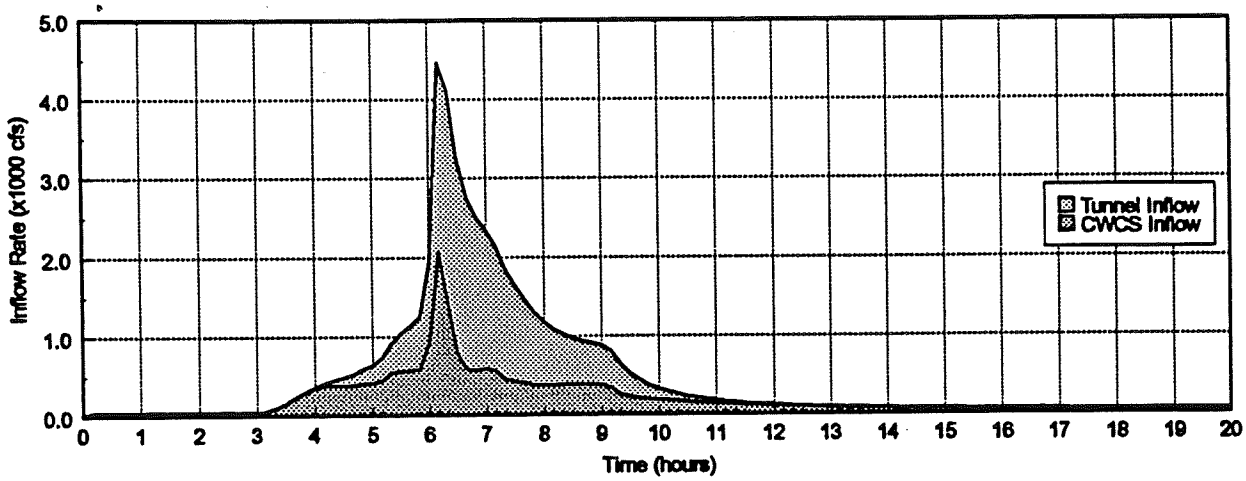
The discussion in this section reviews some aspects of tunnel dynamics and hydraulics that are beneficial to the interpretation of the results as presented in the main TM. For the purpose of this discussion, the points of interest are illustrated using output generated with the 10-year tunnel models. Similar results were obtained with the 5- and 25-year tunnel system models.

C.1. Tunnel Inflow

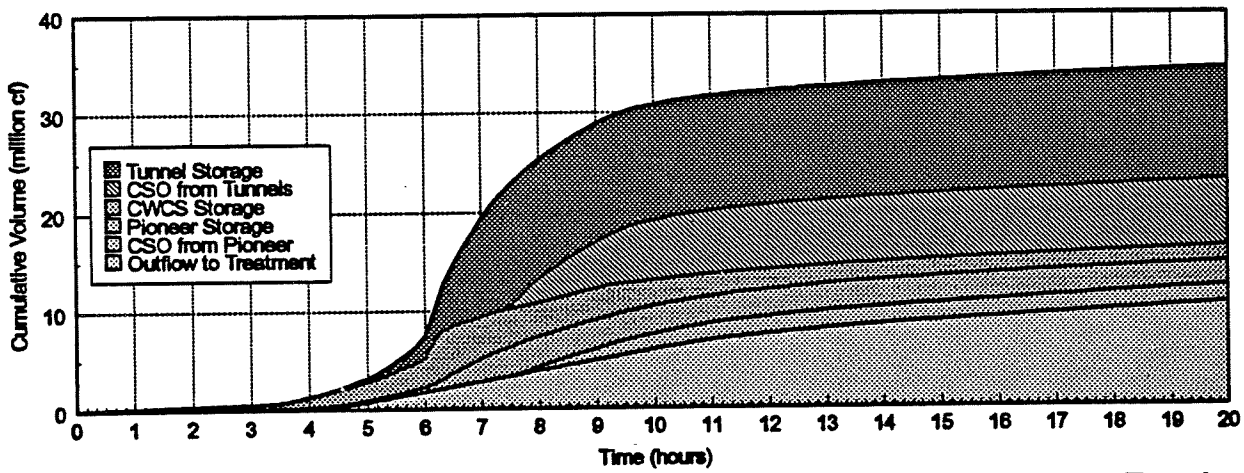
In order to interpret the tunnel hydraulic behavior it is essential to study tunnel inflow pattern. As previously discussed, excess flow in the existing conveyance system is diverted to the tunnel system by means of diversion weirs and consolidation conduits, to alleviate conveyance limitations within the CWCS.

Figure D-4 illustrates how total combined wastewater flow generated in the system is allocated to various components of the existing system and proposed improvements. The sum of the two areas in the top figure represents the total system inflow rate, with the lower area the CWCS inflow rate and the upper area the tunnel inflow rate. The bottom two figures in Figure D-4 show the variation of representative fractions of the total inflow within different sectors of the system with time. The bottom four fractions, CWCS storage, Pioneer Reservoir storage, CSO from Pioneer Reservoir and flow to treatment (SRWTP and CWTP), represent the inflow handled by the existing CWCS system, with the two upper sectors representing the tunnel inflow.

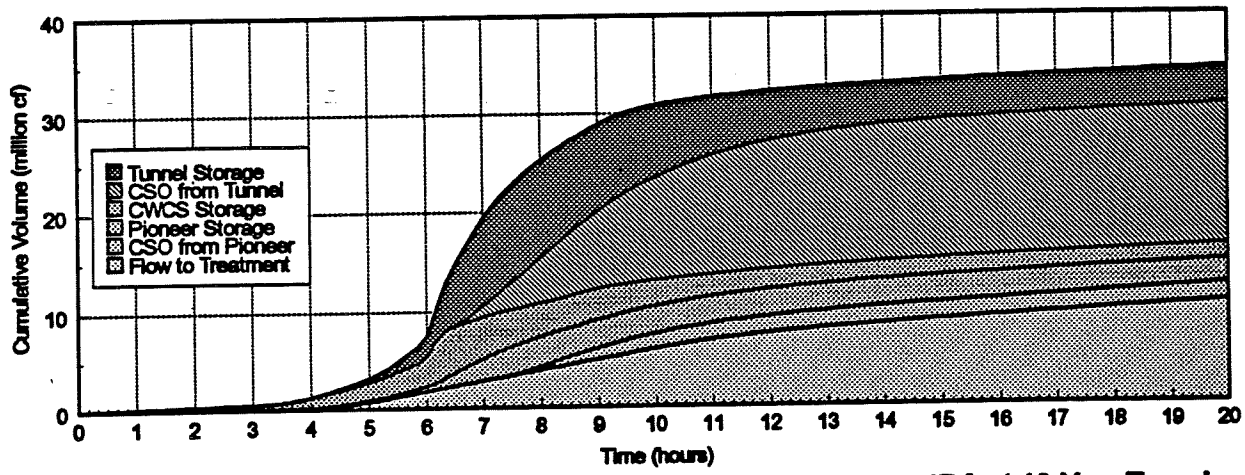
The peak in the CWCS inflow rate, at approximately the 6th hour, does not represent a conveyance rate within the existing CWCS system, but rather a combination of conveyance and storage within the CWCS network. In the bottom two graphs, the increase in CWCS storage at approximately the 6th hour is clearly illustrated.



Tunnel and CWCS Inflow Rates as a Fraction of the Total System Inflow Rate



Cumulative Volumes for Different Sectors of the Total System Using the 17 foot 10-Year Tunnel Configuration with the Discharge Control Elevation = -18.00 ft MSL.



Cumulative Volumes for Different Sectors of the Total System Using the 17 foot 10-Year Tunnel Configuration with the Discharge Control Elevation = -28.0 ft MSL.

HDR

Figure D-4

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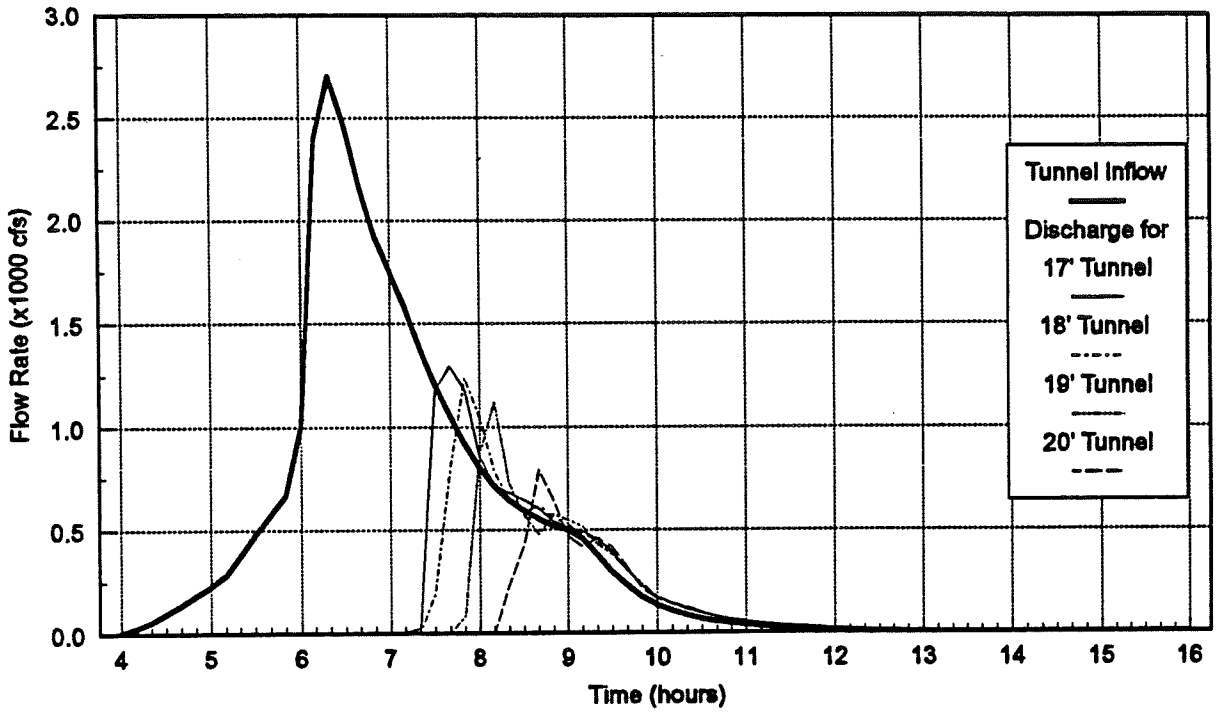
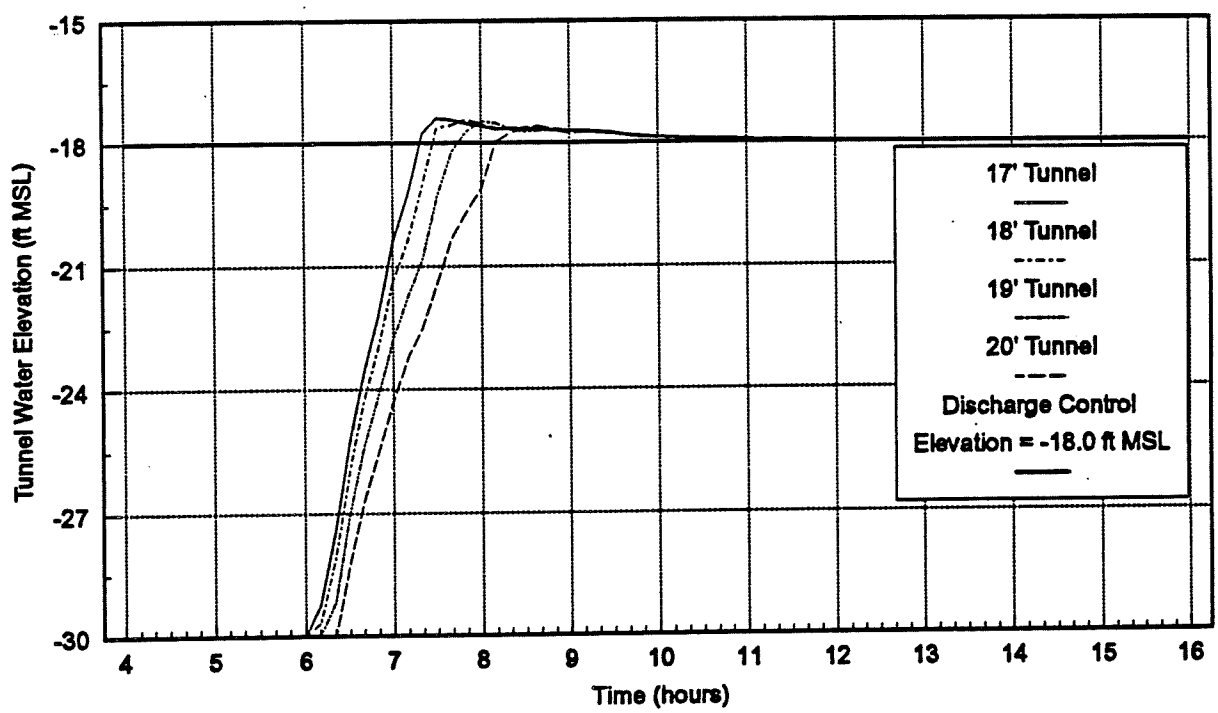
C.2. Storage Optimization

The tunnel hydraulic behavior depends on: 1) the fraction of inflow that can be stored in the system; and 2) the degree of attenuation of the discharge peak provided within the remaining available tunnel storage, i.e. the storage above the discharge control elevation. This can be illustrated by studying the inflow and discharge hydrographs for the 10-year tunnel as shown in Figure D-5. As the tunnel storage increase with increasing tunnel diameter, a larger fraction of the total tunnel inflow is stored in the tunnel. The tunnel inflow in excess of the storage capacity with be discharge to the river. An increase in the tunnel diameter results in a smaller tunnel discharge volume and lower tunnel discharge rate.

A study of the 5-year and 25-year tunnel systems show similar hydraulic behavior patterns. In the 5-year systems, the storage capacity of the 14- and 15-foot tunnels was more than the total tunnel inflow and therefore did not discharge to the river. A summary of the various tunnel statistics is presented in Table D-1.



Tunnel Inflow and Discharge Hydrographs for the 10-Year Tunnel Configuration with Varying Tunnel Diameters and the Discharge Control Elevation = -18.00 ft MSL



Tunnel Water Elevations for the 10-Year Tunnel Configuration with Varying Tunnel Diameters and the Discharge Control Elevation = -18.00 ft MSL

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Figure D-5

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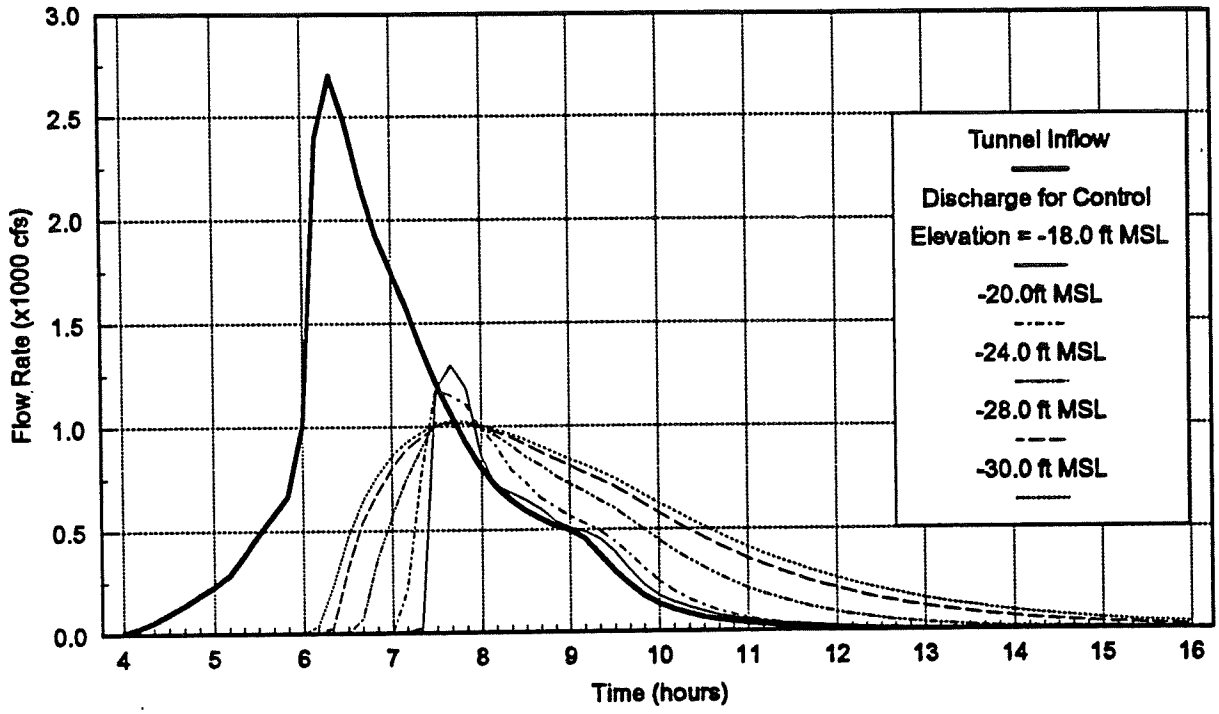
TABLE D-1. SUMMARY OF TUNNEL ANALYSIS STATISTICS FOR MAXIMIZED STORAGE ALTERNATIVES								
Alternative	System Water Balance (10^6 cu.ft.)		Tunnel Water Balance (10^6 cu.ft.)			Tunnel Storage Utilization (%)	Tunnel Discharge as Percentage of	
	Inflow	Discharge	Inflow	Discharge	Storage		Tunnel Inflow	System Outflow
5-Year Tunnel:								
13′ Tunnel	27.0	16.9	6.9	0.9	6.0	100%	12.6%	5.5%
14′ Tunnel	27.0	16.0	6.9	0.0	6.9	97%	0.0%	0.0%
15′ Tunnel	27.0	16.0	6.9	0.0	6.9	84%	0.0%	0.0%
10-Year Tunnel:								
17′ Tunnel	34.5	19.0	18.2	6.7	11.5	100%	36.9%	35.5%
18′ Tunnel	34.5	18.2	18.2	6.0	12.2	100%	32.8%	31.5%
19′ Tunnel	34.5	17.0	18.2	4.8	13.4	100%	26.3%	25.3%
20′ Tunnel	34.5	15.5	18.2	3.3	14.9	100%	18.2%	17.5%
25-Year Tunnel:								
20′ Tunnel	41.5	22.4	24.7	9.8	14.9	100%	39.8%	44.0%
2*20′ Tunnel	41.5	19.7	24.7	7.1	17.6	100%	28.7%	36.1%

C.3. Discharge Control Optimization

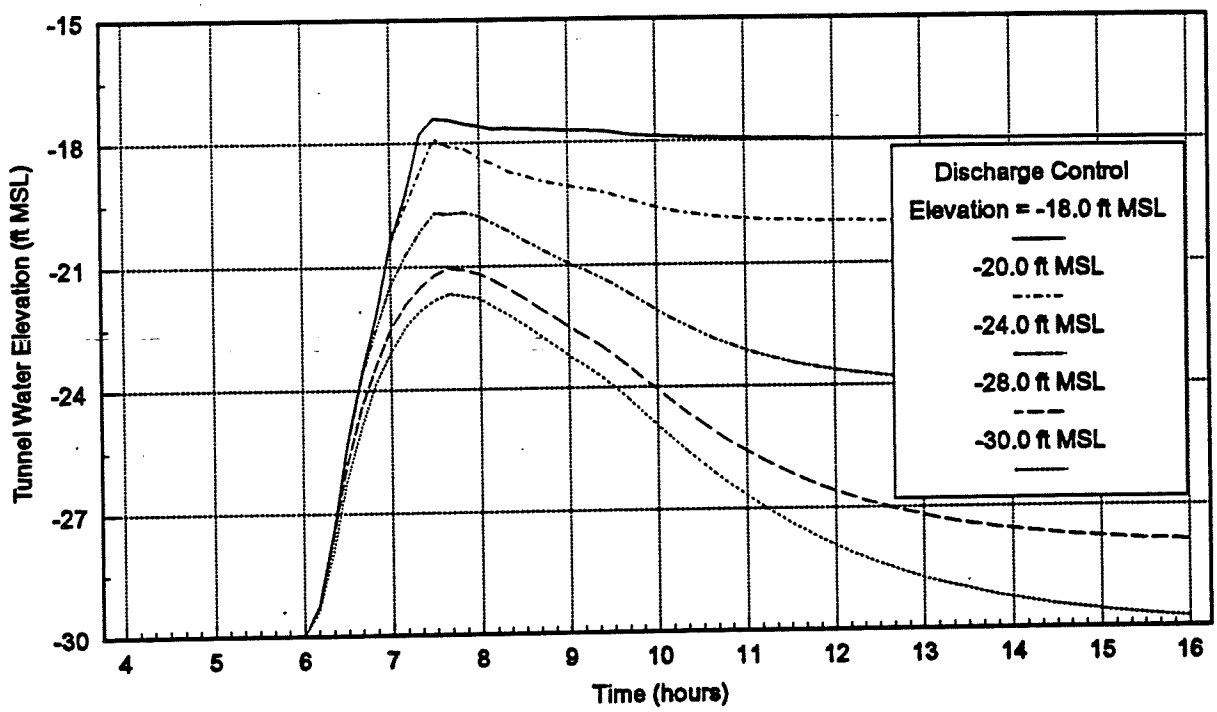
For a given tunnel diameter, lowering the discharge control elevation decreases the available storage prior to discharge, but increases the storage volume above the control level available for tunnel inflow equalization, thus attenuating the peak tunnel discharge rate. This affects the tunnel discharge rate in two ways (refer to Figure D-6). First, the reduced storage capacity causes a larger portion of the tunnel inflow to be discharged to the river at a higher peak discharge rate. Secondly, the available storage above the control elevation provides temporary storage for inflow equalization, resulting in an attenuated peak discharge rate. As illustrated in Figure D-6, the combined effect is a reduction in the peak discharge rate.

For discharge control elevations below a level where discharge from the tunnel occurred before the inflow rate peaked, the attenuated maximum tunnel discharge rate became constant. (See maximum discharge rates for control elevations less than -24 feet MSL in Figure D-6). The same inflow peak (the maximum tunnel inflow rate) had to be attenuated, but with lower control elevations, the available volume for equalizing the inflow is large enough to avoid accelerating the flow rate in the tunnels.

In the case of the 14-foot, 5-year tunnel system, the tunnel storage with the control elevation at the maximized storage setting (-18.00 ft MSL) was sufficient to store the total design storm tunnel inflow. By reducing the discharge control elevation, the inflow exceeds the available storage and discharge occurs. A further reduction in the control elevation did not reduce the discharge rate, but rather, it increased to reach the above mentioned equilibrium.



Tunnel Inflow and Discharge Hydrographs for the 17 foot 10-Year Tunnel Configuration with Varying Discharge Control Elevations



Tunnel Water Elevations for the 17 foot 10-Year Tunnel Configuration with Varying Discharge Control Elevations

HDR

Comparing the inflow and discharge hydrographs, as well as the tunnel water elevation graphs presented in Figures D-5 and D-6, indicates that the volume discharged from the tunnel after the tunnel inflow stops (at approximately 13th hour) is insignificant compared to the total tunnel inflow. It also shows that the higher the discharge control elevation, the closer the tail end of the tunnel discharge hydrograph follows the inflow hydrograph, thus the smaller the volume discharged after inflow stops.

The tunnel model output statistics from the discharge control elevations analysis are summarized in Table D-2.

C.4. CSO Control

For the CSO evaluation, two CSO design storms were considered: the 5/year CSO storm and the 1/year CSO storm. As discussed in the main TM, the 5/year CSO storm did not cause any spill to the tunnels or CSO from the existing system. Therefore, the only CSO control requirements refer to a 1/year CSO storm. Furthermore, only the 5- and 10-year tunnel configurations were evaluated for CSO control, because the flood control configuration for the 25-year tunnel system are sufficient to control the 1/year CSO design storm.

The analysis of the 5- and 10-year tunnel configurations for CSO control concentrate on two aspects: 1) the assessment of the CSO control storage volumes; and 2) the response of the CSO tunnel configuration (i.e. with Pioneer Reservoir overflow diverted to the tunnel) when applying the design storms. The results of these analysis are summarized in the following subsections.

TABLE D-2. SUMMARY OF TUNNEL ANALYSIS STATISTICS FOR VARIATIONS IN THE DISCHARGE CONTROL ELEVATION

Alternative	System Water Balance (10^6 cu. ft.)		Tunnel Water Balance (10^6 cu. ft.)			Tunnel Storage Utilization (%)	Tunnel Discharge as Percentage of	
	Inflow	Discharge	Inflow	Discharge	Storage		Tunnel Inflow	System Outflow
13'- 5-Year Tunnel:								
Discharge Control:								
at -18.00 ft MSL	27.0	16.9	7.0	0.9	6.1	100%	12.6%	5.2%
at -20.00 ft MSL	27.0	17.9	7.0	1.9	5.1	100%	26.9%	10.5%
at -24.00 ft MSL	27.0	20.0	7.0	3.9	3.1	100%	56.1%	19.7%
at -28.00 ft MSL	27.0	21.4	7.0	5.4	1.6	100%	77.1%	25.1%
at -30.00 ft MSL	27.0	21.9	6.9	5.9	1.0	100%	85.2%	26.9%
14'- 5-Year Tunnel:								
Discharge Control:								
at -18.00 ft MSL	27.0	16.0	6.9	0.0	6.9	100%	0.0%	0.0%
at -20.00 ft MSL	27.0	16.9	7.0	0.9	6.1	100%	12.5%	5.2%
at -24.00 ft MSL	27.0	19.2	7.0	3.1	3.9	100%	44.5%	16.3%
at -28.00 ft MSL	27.0	20.8	7.0	4.7	2.3	100%	67.9%	22.8%
at -30.00 ft MSL	27.0	21.3	7.0	5.3	1.7	100%	76.3%	24.9%
10-Year Tunnel:								
Discharge Control:								
at -18.00 ft MSL	34.5	18.9	18.2	6.7	11.5	100%	36.9%	35.5%
at -20.00 ft MSL	34.5	20.1	18.2	7.9	10.3	100%	43.2%	28.0%
at -24.00 ft MSL	34.5	23.3	18.2	11.0	7.2	100%	60.4%	39.2%
at -28.00 ft MSL	34.5	26.6	18.2	14.4	3.8	100%	78.9%	51.2%
at -30.00 ft MSL	34.5	28.2	18.2	15.9	2.3	100%	87.3%	56.6%

C.4.a. CSO Control Storage Volumes--

The tunnel storage requirements for CSO control were assessed as the total tunnel inflow from a 1/year CSO storm event with the following modifications to the design storm tunnel models:

1) Pioneer Reservoir overflow was diverted to the tunnels (thus no direct discharge to the river).

2) Diversion weir settings for the 2-year flood control alternatives were used for both the 5- and 10-year tunnels. This modification forced the maximum flow through the existing CWCS system, therefore maximizing the CWCS utilization and minimizing the tunnel storage requirements.

Since both models (the 5- and 10-year tunnel models) had the same weir settings, the CSO storage requirements for the two tunnel configurations were the same, and was estimated to be approximately 8 million cubic feet. Tunnel inflow from the diversion weirs amounts to 1.3 million cubic feet. The remaining 6.7 million cubic feet is overflow from Pioneer Reservoir to the tunnels.

The 1/year CSO storage volume is more than the 5-year design storm tunnel inflow of 6.9 million cubic feet. The available storage in the 14-foot, 5-year tunnel configuration, which was selected as the least-cost 5-year tunnel alternative, was less than the required CSO storage requirements. Therefore, the larger 15-foot tunnel configuration, which provides sufficient storage for CSO control, was selected as the 1/year 5-year tunnel configuration. The 17-foot, 10-year tunnel with the discharge control elevation at -23.00 feet MSL provided sufficient storage to meet the CSO control requirements for the 1/year CSO design storm event.

C.4.b. CSO/Flood Control Relationships--

By routing both the tunnel inflow and the Pioneer Reservoir overflow to the tunnels, the tunnel dynamics are affected in two ways: 1) the total volume of wastewater diverted to the tunnels increases; and 2) the tunnel discharge rate increases. The extent of the effect on the storage volume and discharge rates due to the above depends on the timing of the Pioneer Reservoir overflow relative to

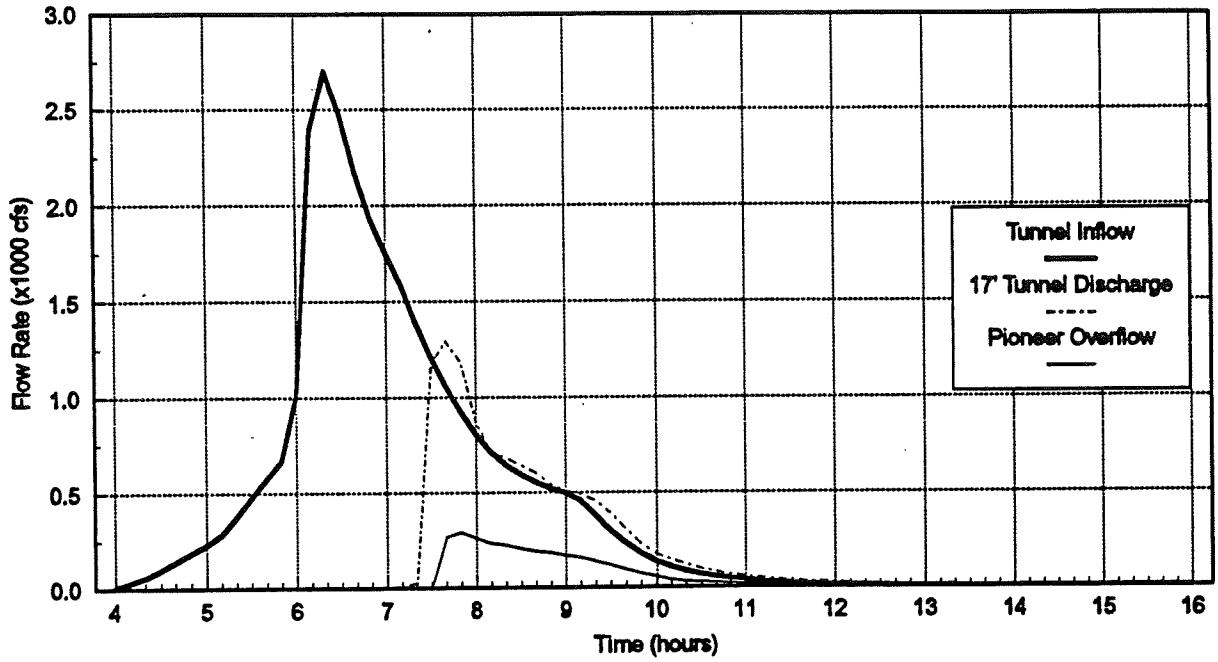
the tunnel inflow from the diversion weirs, and the available storage capacity in the tunnel system. It is also affected by the initial storage volumes in the Pioneer Reservoir and the tunnel system when the event occurs.

For the 10-year tunnels, with both Pioneer Reservoir and the tunnel system empty before the storm event, there is a delay of approximately an hour between the peak in the tunnel inflow and the start of the Pioneer Reservoir overflow as illustrated in Figure D-7. Furthermore, the Pioneer Reservoir overflow for the design event, relative to the total tunnel inflow volume is small. The result is that neither the discharge rate nor the storage in the 10-year tunnels is diversely affected by routing the Pioneer Reservoir overflow to the tunnels.

For the 5-year tunnels, the total tunnel inflow for the 5-Year design storm event (i.e. both the flow diverted from the CWCS and the Pioneer Reservoir overflow), is estimated to be approximately 12.3 million cubic feet. Inflow from the diversion weirs contributes 6.9 million cubic feet and the Pioneer Reservoir overflow contributes the remaining 5.4 million cubic feet.

The storage capacity of the 15-foot tunnel system (8.2 million cubic feet), is sufficient to store the tunnel inflow from the diversion weirs, but is less than the total storage requirement for the 1/year CSO event. Since this tunnel alternative does not include pumping facilities to discharge excess flow to the river, water will back up through the consolidation system to the CWCS causing street flooding for larger storms.

As illustrated in Figure D-7, there is a time delay between the diversion weir inflow peak and the start of the Pioneer Reservoir overflow for the 10-year tunnel system. It will therefore be possible to develop strategies to allow for extensive flexibility in the operation of both the existing system and improvement facilities. This would include the ability for Pioneer Reservoir to overflow directly to the river instead of to the tunnels, once the tunnel volume exceed some predefined control level.



Tunnel Inflow, Tunnel Discharge and Pioneer Reservoir Overflow Hydrographs for the 17 foot 10-Year Tunnel Configuration with the Discharge Control Elevation = -18.0 ft MSL

HDR

Figure D-7

Due to limitations in the EXTRAN program, this operational mode could not be investigated. However, it was assumed that, through the use of adjustable diversion weirs, it will be possible to control inflow the tunnel to ensure the tunnel capacity is not exceeded.

Changing the initial storage volume conditions (i.e. Pioneer Reservoir and the tunnel storage partially full at the start of an event) affects the tunnel dynamics in two ways. Firstly, initial storage volume in the tunnel reduces the available storage to attenuate the peak inflow. The impact would be similar to that of a tunnel system with a smaller diameter; that is, higher tunnel discharge rates and higher discharge pumping requirements would result.

Secondly, initial storage volume in Pioneer Reservoir results in overflows earlier in the storm period. It is anticipated that if Pioneer Reservoir is 100 percent full at the beginning of the event, overflow will commence at approximately the same time as the tunnel inflow peaks. This would also result in higher tunnel discharge rates to the river.

These affects could not be evaluated due to limitations in the existing EXTRAN program. A long-duration storm analysis would give better insight into the effect initial conditions on the tunnel dynamics.

D. SUMMARY AND CONCLUSIONS

The tunnel hydraulic analysis results can be summarized as follows:

1) The tunnel alternatives were configured and sized for both storage and conveyance. The storage requirements were either dictated by the minimum requirements for CSO control or by the most cost effective discharge control alternative.

2) The tunnel discharge control system was modeled as either a culvert type discharge conduit or a discharge control pump station. Both applications provide reliable system simulation. The culvert type discharge control was used for cases where the required discharge rates were unknown.

3) The CSO storage requirements were assessed by diverting both the diversion weir flow and the Pioneer Reservoir overflow to the tunnel. The CSO storage volume was estimated to be the total tunnel inflow.

4) Initial storage volumes in the Pioneer Reservoir and the tunnels at the start of an rain storm event greatly influence the response of the system with regard to discharge rate to the river and the provision of flood control.

5) Operating guidelines should be established to divert Pioneer Reservoir's overflow directly to the river when the tunnel is filled above a predefined control water elevation. This approach will minimize the tunnel discharge pumping requirements (if included into the alternative) and will maximize the flood control protection provided by the tunnel system. These guidelines would include control scenarios for the adjustable diversion weirs that divert flow from the existing system to the tunnel system.

APPENDIX E
Alternative Plan Detailed Cost Spreadsheets

ESTIMATED PROJECT COSTS
5-YEAR TUNNNEL ALTERNATIVE CONFIGURED FOR 1/YR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
TUNNEL PUMP STATION AT SUMP 1A FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNEL PUMP STATION AT SUMP 2 FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNELS				
NEW 15 FOOT DIAMETER	49,200	LF	$2,000.00	$98,400,000
PIONEER DIVERSION TO TUNNEL	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,610	LF	$341.67	$6,700,149
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	17,886	LF	$416.47	$7,448,982
NEW 45 INCH SEWERS	1,600	LF	$456.33	$730,128
NEW 48 INCH SEWERS	5,785	LF	$500.82	$2,897,244
NEW 54 INCH SEWERS	11,290	LF	$570.94	$6,445,913
NEW 60 INCH SEWERS	8,050	LF	$649.83	$5,231,132
NEW 63 INCH SEWERS	800	LF	$688.11	$550,488
NEW 66 INCH SEWERS	4,205	LF	$733.78	$3,085,545
NEW 72 INCH SEWERS	5,680	LF	$809.46	$4,597,733
NEW 78 INCH SEWERS	3,655	LF	$870.84	$3,182,920
NEW 84 INCH SEWERS	3,865	LF	$949.92	$3,671,441
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
TUNNEL DROP SHAFTS				
NEW 3.50 FOOT DROP SHAFT	1	EA	$462,971	$462,971
NEW 4.00 FOOT DROP SHAFT	2	EA	$465,630	$931,260
NEW 4.50 FOOT DROP SHAFT	1	EA	$468,147	$468,147
NEW 5.00 FOOT DROP SHAFT	2	EA	$470,893	$941,786
NEW 5.50 FOOT DROP SHAFT	2	EA	$473,779	$947,557
NEW 6.50 FOOT DROP SHAFT	1	EA	$479,391	$479,391
NEW 8.00 FOOT DROP SHAFT	1	EA	$490,409	$490,409
NEW 8.50 FOOT DROP SHAFT	1	EA	$494,429	$494,429
NEW 9.00 FOOT DROP SHAFT	1	EA	$497,557	$497,557
NEW 12.00 FOOT DROP SHAFT	1	EA	$516,640	$516,640
SUBTOTAL				$173,125,268
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,650,000	$181,650,000
SUBTOTAL				$376,512,868
CONSTRUCTION CONTINGENCY @ 25%				$94,128,217
DESIGN @ 6%				$22,590,772
CONSTRUCTION INSPECTION @ 10%				$37,651,287
ADMINISTRATION, LEGAL @ 3%				$11,295,386
BONDS, INSURANCE, AND MISC. @ 5%				$18,825,643
ESTIMATED TOTAL ALTERNATIVE COST				$561,004,174

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COS
AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
5-YEAR TUNNEL ALTERNATIVE CONFIGURED FOR 5/YEAR CSO CONTROL &FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
TUNNEL PUMP STATION AT SUMP 1A FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNEL PUMP STATION AT SUMP 2 FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNELS				
NEW 14 FOOT DIAMETER	49,200	LF	$1,940.00	$95,448,000
NEW OUTFALL LINE AT SUMP 1A	ALL REQ'D	LS	$750,000	$750,000
NEW OUTFALL LINE AT SUMP 2	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,610	LF	$341.67	$6,700,149
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	17,886	LF	$416.47	$7,448,982
NEW 45 INCH SEWERS	1,600	LF	$456.33	$730,128
NEW 48 INCH SEWERS	5,785	LF	$500.82	$2,897,244
NEW 54 INCH SEWERS	11,290	LF	$570.94	$6,445,913
NEW 60 INCH SEWERS	8,050	LF	$649.83	$5,231,132
NEW 63 INCH SEWERS	800	LF	$688.11	$550,488
NEW 66 INCH SEWERS	4,205	LF	$733.78	$3,085,545
NEW 72 INCH SEWERS	5,680	LF	$809.46	$4,597,733
NEW 78 INCH SEWERS	3,655	LF	$870.84	$3,182,920
NEW 84 INCH SEWERS	3,865	LF	$949.92	$3,671,441
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
TUNNEL DROP SHAFTS				
NEW 3.50 FOOT DROP SHAFT	1	EA	$462,971	$462,971
NEW 4.00 FOOT DROP SHAFT	2	EA	$465,630	$931,260
NEW 4.50 FOOT DROP SHAFT	1	EA	$468,147	$468,147
NEW 5.00 FOOT DROP SHAFT	2	EA	$470,893	$941,786
NEW 5.50 FOOT DROP SHAFT	2	EA	$473,779	$947,557
NEW 6.50 FOOT DROP SHAFT	1	EA	$479,391	$479,391
NEW 8.00 FOOT DROP SHAFT	1	EA	$490,409	$490,409
NEW 8.50 FOOT DROP SHAFT	1	EA	$494,429	$494,429
NEW 9.00 FOOT DROP SHAFT	1	EA	$497,557	$497,557
NEW 12.00 FOOT DROP SHAFT	1	EA	$516,640	$516,640
SUBTOTAL				$171,173,268
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000.00	$181,080,000
SUBTOTAL				$373,990,868
CONSTRUCTION CONTINGENCY @ 25%				$93,497,717
DESIGN @ 6%				$22,439,452
CONSTRUCTION INSPECTION @ 10%				$37,399,087
ADMINISTRATION, LEGAL @ 3%				$11,219,726
BONDS, INSURANCE, AND MISC. @ 5%				$18,699,543
ESTIMATED TOTAL ALTERNATIVE COST				$557,246,394

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COS AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
10-YEAR TUNNEL ALTERNATIVE CONFIGURED FOR 1/YR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
TUNNEL PUMP STATION AT SUMP 1A	430	MGD	$40,771,491	$40,771,491
TUNNEL PUMP STATION AT SUMP 2	265	MGD	$27,680,793	$27,680,793
TUNNEL PUMP STATION AT SUMP 1A FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNEL PUMP STATION AT SUMP 2 FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNELS				
NEW 17 FOOT DIAMETER	26,100	LF	$2,569.00	$67,050,900
NEW 18 FOOT DIAMETER	13,800	LF	$2,881.00	$39,757,800
NEW 19 FOOT DIAMETER	9,300	LF	$3,209.00	$29,843,700
PIONEER DIVERSION TO TUNNEL	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL LINE AT SUMP 1A	ALL REQ'D	LS	$750,000	$750,000
NEW OUTFALL LINE AT SUMP 2	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	14,651	LF	$500.82	$7,337,514
NEW 54 INCH SEWERS	6,295	LF	$570.94	$3,594,067
NEW 60 INCH SEWERS	11,990	LF	$649.83	$7,791,462
NEW 66 INCH SEWERS	4,000	LF	$733.78	$2,935,120
NEW 72 INCH SEWERS	15,435	LF	$809.46	$12,494,015
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	4,700	LF	$949.92	$4,464,624
NEW 90 INCH SEWERS	1,800	LF	$1,078.02	$1,940,436
NEW 96 INCH SEWERS	4,865	LF	$1,134.86	$5,521,094
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
TUNNEL DROP SHAFTS				
NEW 4.00 FOOT DROP SHAFT	1	EA	$465,630	$465,630
NEW 5.00 FOOT DROP SHAFT	2	EA	$470,893	$941,786
NEW 5.50 FOOT DROP SHAFT	1	EA	$473,779	$473,779
NEW 6.00 FOOT DROP SHAFT	1	EA	$476,600	$476,600
NEW 6.50 FOOT DROP SHAFT	2	EA	$479,391	$958,783
NEW 7.00 FOOT DROP SHAFT	1	EA	$482,850	$482,850
NEW 7.50 FOOT DROP SHAFT	1	EA	$487,703	$487,703
NEW 9.00 FOOT DROP SHAFT	1	EA	$497,557	$497,557
NEW 11.00 FOOT DROP SHAFT	1	EA	$510,191	$510,191
NEW 12.00 FOOT DROP SHAFT	2	EA	$516,640	$1,033,279
NEW 14.00 FOOT DROP SHAFT	1	EA	$520,348	$520,348
SUBTOTAL				$296,862,130
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000.00	$198,580,000
SUBTOTAL				$517,179,730
CONSTRUCTION CONTINGENCY @ 25%				$129,294,932
DESIGN @ 6%				$31,030,784
CONSTRUCTION INSPECTION @ 10%				$51,717,973
ADMINISTRATION, LEGAL @ 3%				$15,515,392
BONDS, INSURANCE, AND MISC. @ 5%				$25,858,986
ESTIMATED TOTAL ALTERNATIVE COST				$770,597,797

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COS
AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
10-YEAR TUNNEL ALTERNATIVE CONFIGURED FOR 5/YEAR CSO CONTROL & FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
TUNNEL PUMP STATION AT SUMP 1A	420	MGD	$40,011,171	$40,011,171
TUNNEL PUMP STATION AT SUMP 2	250	MGD	$26,420,062	$26,420,062
TUNNEL PUMP STATION AT SUMP 2 FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNEL PUMP STATION AT SUMP2 FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNELS				
NEW 17 FOOT DIAMETER	26,100	LF	$2,569.00	$67,050,900
NEW 18 FOOT DIAMETER	13,800	LF	$2,881.00	$39,757,800
NEW 19 FOOT DIAMETER	9,300	LF	$3,209.00	$29,843,700
PIONEER DIVERSION TO TUNNEL	ALL REQ'D		$500,000.00	$500,000.00
NEW OUTFALL LINE AT SUMP 1A	ALL REQ'D	LS	$750,000	$750,000
NEW OUTFALL LINE AT SUMP 2	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPINGCONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	14,651	LF	$500.82	$7,337,514
NEW 54 INCH SEWERS	6,295	LF	$570.94	$3,594,067
NEW 60 INCH SEWERS	11,990	LF	$649.83	$7,791,462
NEW 66 INCH SEWERS	4,000	LF	$733.78	$2,935,120
NEW 72 INCH SEWERS	15,435	LF	$809.46	$12,494,015
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	4,700	LF	$949.92	$4,464,624
NEW 90 INCH SEWERS	1,800	LF	$1,078.02	$1,940,436
NEW 96 INCH SEWERS	4,865	LF	$1,134.86	$5,521,094
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
TUNNEL DROP SHAFTS				
NEW 4.00 FOOT DROP SHAFT	1	EA	$465,630	$465,630
NEW 5.00 FOOT DROP SHAFT	2	EA	$470,893	$941,786
NEW 5.50 FOOT DROP SHAFT	1	EA	$473,779	$473,779
NEW 6.00 FOOT DROP SHAFT	1	EA	$476,600	$476,600
NEW 6.50 FOOT DROP SHAFT	2	EA	$479,391	$958,783
NEW 7.00 FOOT DROP SHAFT	1	EA	$482,850	$482,850
NEW 7.50 FOOT DROP SHAFT	1	EA	$487,703	$487,703
NEW 9.00 FOOT DROP SHAFT	1	EA	$497,557	$497,557
NEW 11.00 FOOT DROP SHAFT	1	EA	$510,191	$510,191
NEW 12.00 FOOT DROP SHAFT	2	EA	$516,640	$1,033,279
NEW 14.00 FOOT DROP SHAFT	1	EA	$520,348	$520,348
SUBTOTAL				$294,841,079
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000.00	$198,580,000
SUBTOTAL				$515,158,679
CONSTRUCTION CONTINGENCY @ 25%				$128,789,670
DESIGN @ 6%				$30,909,521
CONSTRUCTION INSPECTION @ 10%				$51,515,868
ADMINISTRATION, LEGAL @ 3%				$15,454,760
BONDS, INSURANCE, AND MISC. @ 5%				$25,757,934
ESTIMATED TOTAL ALTERNATIVE COST				$767,586,431

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COS AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
25-YEAR TUNNEL ALTERNATIVE CONFIGURED FOR FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
TUNNEL PUMP STATION AT SUMP 1A	720	MGD	$61,581,154	$61,581,154
TUNNEL PUMP STATION AT SUMP 2	350	MGD	$34,580,898	$34,580,898
TUNNEL PUMP STATION AT SUMP 1A FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNEL PUMP STATION AT SUMP 2 FOR EMPTYING TUNNEL	14	MGD	$2,633,202	$2,633,202
TUNNELS				
NEW 20 FOOT DIAMETER	58,100	LF	$3,556.00	$206,603,600
NEW OUTFALL LINE AT SUMP 1A	ALL REQ'D	LS	$750,000	$750,000
NEW OUTFALL LINE AT SUMP 2	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 21 INCH SEWERS	600	LF	$147.05	$88,230
NEW 24 INCH SEWERS	3,880	LF	$209.41	$812,511
NEW 27 INCH SEWERS	4,090	LF	$237.49	$971,334
NEW 30 INCH SEWERS	3,620	LF	$269.03	$973,889
NEW 33 INCH SEWERS	3,020	LF	$302.12	$912,402
NEW 36 INCH SEWERS	17,665	LF	$341.67	$6,035,601
NEW 39 INCH SEWERS	6,260	LF	$379.83	$2,377,736
NEW 42 INCH SEWERS	11,900	LF	$416.47	$4,955,993
NEW 45 INCH SEWERS	2,450	LF	$456.33	$1,118,009
NEW 48 INCH SEWERS	16,831	LF	$500.82	$8,429,301
NEW 51 INCH SEWERS	3,430	LF	$535.07	$1,835,290
NEW 54 INCH SEWERS	10,070	LF	$570.94	$5,749,366
NEW 57 INCH SEWERS	7,180	LF	$609.17	$4,373,841
NEW 60 INCH SEWERS	12,865	LF	$649.83	$8,360,063
NEW 66 INCH SEWERS	2,175	LF	$733.78	$1,595,972
NEW 72 INCH SEWERS	16,890	LF	$809.46	$13,671,779
NEW 78 INCH SEWERS	9,030	LF	$870.84	$7,863,685
NEW 84 INCH SEWERS	7,600	LF	$949.92	$7,219,392
NEW 90 INCH SEWERS	11,930	LF	$1,078.02	$12,860,779
NEW 96 INCH SEWERS	3,450	LF	$1,134.86	$3,915,267
NEW 102 INCH SEWERS	980	LF	$1,233.58	$1,208,908
NEW 108 INCH SEWERS	4,650	LF	$1,303.88	$6,063,042
NEW 114 INCH SEWERS	3,400	LF	$1,373.90	$4,671,260
NEW 120 INCH SEWERS	3,965	LF	$1,446.65	$5,735,967
DIVERSION WEIRS				
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	4	EA	$185,691	$742,763
NEW 5.00 FOOT WEIR	2	EA	$194,262	$388,525
NEW 5.50 FOOT WEIR	2	EA	$202,966	$405,932
NEW 6.00 FOOT WEIR	3	EA	$211,846	$635,538
NEW 6.50 FOOT WEIR	6	EA	$220,883	$1,325,300
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	4	EA	$234,681	$938,725
NEW 8.00 FOOT WEIR	3	EA	$244,067	$732,201
NEW 9.00 FOOT WEIR	1	EA	$260,872	$260,872
NEW 9.50 FOOT WEIR	5	EA	$270,735	$1,353,674
NEW 10.00 FOOT WEIR	1	EA	$278,072	$278,072
TUNNEL DROP SHAFTS				
NEW 4.00 FOOT DROP SHAFT	1	EA	$465,630	$465,630
NEW 4.50 FOOT DROP SHAFT	1	EA	$468,147	$468,147
NEW 6.50 FOOT DROP SHAFT	2	EA	$479,391	$958,783
NEW 8.00 FOOT DROP SHAFT	3	EA	$490,409	$1,471,226
NEW 9.50 FOOT DROP SHAFT	2	EA	$500,695	$1,001,391
NEW 11.00 FOOT DROP SHAFT	2	EA	$510,191	$1,020,383
NEW 13.00 FOOT DROP SHAFT	1	EA	$518,501	$518,501
NEW 14.00 FOOT DROP SHAFT	3	EA	$520,348	$1,561,043
NEW 15.00 FOOT DROP SHAFT	1	EA	$522,179	$522,179
NEW 17.00 FOOT DROP SHAFT	1	EA	$525,797	$525,797
SUBTOTAL				$439,298,265
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000.00	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$216,830,000.00	$216,830,000
SUBTOTAL				$677,865,865
CONSTRUCTION CONTINGENCY @ 25%				$169,466,466
DESIGN @ 6%				$40,671,952
CONSTRUCTION INSPECTION @ 10%				$67,786,587
ADMINISTRATION, LEGAL @ 3%				$20,335,976
BONDS, INSURANCE, AND MISC. @ 5%				$33,893,293
ESTIMATED TOTAL ALTERNATIVE COST				$1,010,020,139

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COS AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
2-YEAR UPSIZED CWCS WITH TREATMENT AT PIONEER
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW TREATMENT FACILITY AT PIONEER	31.3	MG	$30,605,000	$30,605,000
SLUDGE PUMPING	3,478	GPM	$1,051,716	$1,051,716
NEW FLUSH WATER INTAKE AT PIONEER	ALL REQ'D	LS	$125,000	$125,000
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
PIONEER CONNECTION TO STORAGE	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 15 INCH SEWERS	1,500	LF	$111.35	$167,025
NEW 18 INCH SEWERS	5,285	LF	$129.35	$683,615
NEW 21 INCH SEWERS	9,160	LF	$147.05	$1,346,978
NEW 24 INCH SEWERS	5,295	LF	$209.41	$1,108,826
NEW 27 INCH SEWERS	8,770	LF	$237.49	$2,082,787
NEW 30 INCH SEWERS	12,765	LF	$269.03	$3,434,168
NEW 33 INCH SEWERS	5,945	LF	$302.12	$1,796,103
NEW 36 INCH SEWERS	5,841	LF	$341.67	$1,995,694
NEW 39 INCH SEWERS	2,610	LF	$379.83	$991,356
NEW 42 INCH SEWERS	3,490	LF	$416.47	
NEW 72 INCH SEWERS	2,350	LF	$809.46	$1,902,231
SUBTOTAL				$49,540,500
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$150,650,000	$150,650,000
SUBTOTAL				$221,928,100
CONSTRUCTION CONTINGENCY @ 25%				$55,482,025
DESIGN @ 6%				$13,315,686
CONSTRUCTION INSPECTION @ 10%				$22,192,810
ADMINISTRATION, LEGAL @ 3%				$6,657,843
BONDS, INSURANCE, AND MISC. @ 5%				$11,096,405
ESTIMATED TOTAL ALTERNATIVE COST				$330,672,869

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTON AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
2-YEAR UPSIZED CWCS WITH STORAGE/TREATMENT AT WPRR SITE
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	41.6	MG	$39,360,000	$39,360,000
NEW PUMP STATION AT WPRR	500	MGD	$46,000,000	$46,000,000
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$456	$3,830,400
NEW 45 INCH DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW TREATMENT FACILITY AT WPRR	3.1	MG	$6,635,000	$6,635,000
SLUDGE PUMPING	344	GPM	$334,755	$334,755
NEW TREATED EFFLUENT PUMP STATION	50	MGD	$7,290,509	$7,290,509
DONNER DIVERSION	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 15 INCH SEWERS	1,500	LF	$111.35	$167,025
NEW 18 INCH SEWERS	5,285	LF	$129.35	$683,615
NEW 21 INCH SEWERS	9,160	LF	$147.05	$1,346,978
NEW 24 INCH SEWERS	5,295	LF	$209.41	$1,108,826
NEW 27 INCH SEWERS	8,770	LF	$237.49	$2,082,787
NEW 30 INCH SEWERS	12,765	LF	$269.03	$3,434,168
NEW 33 INCH SEWERS	5,945	LF	$302.12	$1,796,103
NEW 36 INCH SEWERS	5,841	LF	$341.67	$1,995,694
NEW 39 INCH SEWERS	2,610	LF	$379.83	$991,356
NEW 42 INCH SEWERS	3,490	LF	$416.47	$1,453,480
NEW 72 INCH SEWERS	2,350	LF	$809.46	$1,902,231
SUBTOTAL				$123,162,929
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$150,650,000	$150,650,000
SUBTOTAL				$295,550,529
CONSTRUCTION CONTINGENCY @25%				78057632
DESIGN @ 6%				$17,733,032
CONSTRUCTION INSPECTION @ 10%				$29,555,053
ADMINISTRATION, LEGAL @ 3%				$8,866,516
BONDS, INSURANCE, AND MISC. @ 5%				$14,777,526
ESTIMATED TOTAL ALTERNATIVE COST				$444,540,287

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTON AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
2-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC
CONFIGURED FOR 5/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	40	MGD	$6,098,595	$6,098,595
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$380	$3,192,000
NEW 39 INCH DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
INSTALLED GRAVITY SEWER PIPE				
NEW 15 INCH SEWERS	2,520	LF	$111.35	$280,602
NEW 18 INCH SEWERS	5,365	LF	$129.35	$693,963
NEW 21 INCH SEWERS	9,585	LF	$147.05	$1,409,474
NEW 24 INCH SEWERS	6,345	LF	$209.41	$1,328,706
NEW 27 INCH SEWERS	8,270	LF	$237.49	$1,964,042
NEW 30 INCH SEWERS	10,620	LF	$269.03	$2,857,099
NEW 33 INCH SEWERS	5,945	LF	$302.12	$1,796,103
NEW 36 INCH SEWERS	4,041	LF	$341.67	$1,380,688
NEW 39 INCH SEWERS	1,650	LF	$379.83	$626,720
NEW 42 INCH SEWERS	3,490	LF	$416.47	$1,453,480
NEW 60 INCH SEWERS	2,700	LF	$649.83	$1,754,541
NEW 66 INCH SEWERS	2,600	LF	$733.78	$1,907,828
NEW 69 INCH SEWERS	900	LF	$775.84	$698,256
NEW 72 INCH SEWERS	2,350	LF	$809.46	$1,902,231
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	4	EA	$146,693	$586,774
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
SUBTOTAL				$30,847,972
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$150,650,000	$150,650,000
SUBTOTAL				$203,235,572
CONSTRUCTION CONTINGENCY @ 25%				$50,808,893
DESIGN @ 6%				$12,194,134
CONSTRUCTION INSPECTION @ 10%				$20,323,557
ADMINISTRATION, LEGAL @ 3%				$6,097,067
BONDS, INSURANCE, AND MISC. @ 5%				$10,161,779
ESTIMATED TOTAL ALTERNATIVE COST				$302,821,002

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTON AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
2-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER
CONFIGURED FOR 5/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	40	MGD	$6,098,595	$6,098,595
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$380	$3,192,000
NEW 39 INCH DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
INSTALLED GRAVITY SEWER PIPE				
NEW 15 INCH SEWERS	2,520	LF	$111.35	$280,602
NEW 18 INCH SEWERS	5,365	LF	$129.35	$693,963
NEW 21 INCH SEWERS	9,585	LF	$147.05	$1,409,474
NEW 24 INCH SEWERS	6,345	LF	$209.41	$1,328,706
NEW 27 INCH SEWERS	8,270	LF	$237.49	$1,964,042
NEW 30 INCH SEWERS	10,620	LF	$269.03	$2,857,099
NEW 33 INCH SEWERS	5,945	LF	$302.12	$1,796,103
NEW 36 INCH SEWERS	4,041	LF	$341.67	$1,380,688
NEW 39 INCH SEWERS	1,650	LF	$379.83	$626,720
NEW 42 INCH SEWERS	3,490	LF	$416.47	$1,453,480
NEW 60 INCH SEWERS	2,700	LF	$649.83	$1,754,541
NEW 66 INCH SEWERS	2,600	LF	$733.78	$1,907,828
NEW 69 INCH SEWERS	900	LF	$775.84	$698,256
NEW 72 INCH SEWERS	2,350	LF	$809.46	$1,902,231
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	4	EA	$146,693	$586,774
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
SUBTOTAL				$30,847,972
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$150,650,000	$150,650,000
SUBTOTAL				$203,235,572
CONSTRUCTION CONTINGENCY @ 25%				$50,808,893
DESIGN @ 6%				$12,194,134
CONSTRUCTION INSPECTION @ 10%				$20,323,557
ADMINISTRATION, LEGAL @ 3%				$6,097,067
BONDS, INSURANCE, AND MISC. @ 5%				$10,161,779
ESTIMATED TOTAL ALTERNATIVE COST				$302,821,002

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTON AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
2-YEAR UPSIZED CWCS - CONVEYANCE ALTERNATIVE
CONFIGURED FOR 5/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
INSTALLED GRAVITY SEWER PIPE				
NEW 15 INCH SEWERS	1,500	LF	$111.35	$167,025
NEW 18 INCH SEWERS	5,285	LF	$129.35	$683,615
NEW 21 INCH SEWERS	9,160	LF	$147.05	$1,346,978
NEW 24 INCH SEWERS	5,295	LF	$209.41	$1,108,826
NEW 27 INCH SEWERS	8,770	LF	$237.49	$2,082,787
NEW 30 INCH SEWERS	12,765	LF	$269.03	$3,434,168
NEW 33 INCH SEWERS	5,945	LF	$302.12	$1,796,103
NEW 36 INCH SEWERS	5,841	LF	$341.67	$1,995,694
NEW 39 INCH SEWERS	2,610	LF	$379.83	$991,356
NEW 42 INCH SEWERS	3,490	LF	$416.47	$1,453,480
NEW 72 INCH SEWERS	2,350	LF	$809.46	$1,902,231
SUBTOTAL				$16,962,264
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$150,650,000	$150,650,000
SUBTOTAL				$189,349,864
CONSTRUCTION CONTINGENCY @ 25%				$47,337,466
DESIGN @ 6%				$11,360,992
CONSTRUCTION INSPECTION @ 10%				$18,934,986
ADMINISTRATION, LEGAL @ 3%				$5,680,496
BONDS, INSURANCE, AND MISC. @ 5%				$9,467,493
ESTIMATED TOTAL ALTERNATIVE COST				$282,131,298

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTON AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
2-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	38.4	MG	$36,640,000	$36,640,000
NEW PUMP STATION AT WPRR	535	MGD	$48,558,454	$48,558,454
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$571	$4,796,400
NEW 54 INCH DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW TREATMENT FACILITY AT WPRR	4.7	MG	$4,700,000	$4,700,000
SLUDGE PUMPING	522	GPM	$411,349	$411,349
NEW TREATED EFFLUENT PUMP STATION	75	MGD	$10,083,954	$10,083,954
DONNER DIVERSION	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 15 INCH SEWERS	2,520	LF	$111.35	$280,602
NEW 18 INCH SEWERS	5,365	LF	$129.35	$693,963
NEW 21 INCH SEWERS	9,585	LF	$147.05	$1,409,474
NEW 24 INCH SEWERS	6,345	LF	$209.41	$1,328,706
NEW 27 INCH SEWERS	8,270	LF	$237.49	$1,964,042
NEW 30 INCH SEWERS	10,620	LF	$269.03	$2,857,099
NEW 33 INCH SEWERS	5,945	LF	$302.12	$1,796,103
NEW 36 INCH SEWERS	4,041	LF	$341.67	$1,380,688
NEW 39 INCH SEWERS	1,650	LF	$379.83	$626,720
NEW 42 INCH SEWERS	3,490	LF	$416.47	$1,453,480
NEW 60 INCH SEWERS	2,700	LF	$649.83	$1,754,541
NEW 66 INCH SEWERS	2,600	LF	$733.78	$1,907,828
NEW 69 INCH SEWERS	900	LF	$775.84	$698,256
NEW 72 INCH SEWERS	2,350	LF	$809.46	$1,902,231
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	4	EA	$146,693	$586,774
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
SUBTOTAL				$128,747,533
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$150,650,000	$150,650,000
SUBTOTAL				$301,135,133
CONSTRUCTION CONTINGENCY @ 25%				$75,283,783
DESIGN @ 6%				$18,068,108
CONSTRUCTION INSPECTION @ 10%				$30,113,513
ADMINISTRATION, LEGAL @ 3%				$9,034,054
BONDS, INSURANCE, AND MISC. @ 5%				$15,056,757
ESTIMATED TOTAL ALTERNATIVE COST				$448,691,348

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTON AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
2-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	1.4	MG	$1,400,000	$1,400,000
NEW PUMP STATION AT WPRR	50	MGD	$7,290,509	$7,290,509
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$302	$2,536,800
NEW 33 INCH DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW TREATMENT FACILITY AT WPRR	1.6	MG	$1,600,000	$1,600,000
SLUDGE PUMPING AT WPRR	178	GPM	$241,276	$241,276
NEW TREATED EFFLUENT PUMP STATION	25	MGD	$4,187,298	$4,187,298
NEW TREATMENT FACILITY AT PIONEER	31.3	MG	$30,562,500	$30,562,500
SLUDGE PUMPING AT PIONEER	3,472	GPM	$1,050,884	$1,050,884
NEW FLUSH WATER INTAKE AT PIONEER	ALL REQ'D	LS	$125,000	$125,000
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
PIONEER CONNECTION TO STORAGE	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,000,000	$1,000,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,000,000	$1,000,000
INSTALLED GRAVITY SEWER PIPE				
NEW 15 INCH SEWERS	2,520	LF	$111.35	$280,602
NEW 18 INCH SEWERS	5,365	LF	$129.35	$693,963
NEW 21 INCH SEWERS	9,585	LF	$147.05	$1,409,474
NEW 24 INCH SEWERS	6,345	LF	$209.41	$1,328,706
NEW 27 INCH SEWERS	8,270	LF	$237.49	$1,964,042
NEW 30 INCH SEWERS	10,620	LF	$269.03	$2,857,099
NEW 33 INCH SEWERS	5,945	LF	$302.12	$1,796,103
NEW 36 INCH SEWERS	4,041	LF	$341.67	$1,380,688
NEW 39 INCH SEWERS	1,650	LF	$379.83	$626,720
NEW 42 INCH SEWERS	3,490	LF	$416.47	$1,453,480
NEW 60 INCH SEWERS	2,700	LF	$649.83	$1,754,541
NEW 66 INCH SEWERS	2,600	LF	$733.78	$1,907,828
NEW 69 INCH SEWERS	900	LF	$775.84	$698,256
NEW 72 INCH SEWERS	2,350	LF	$809.46	$1,902,231
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	4	EA	$146,693	$586,774
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
SUBTOTAL				$74,301,643
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$150,650,000	$150,650,000
SUBTOTAL				$246,689,243
CONSTRUCTION CONTINGENCY @ 25%				$61,672,311
DESIGN @ 6%				$14,801,355
CONSTRUCTION INSPECTION @ 10%				$24,668,924
ADMINISTRATION, LEGAL @ 3%				$7,400,677
BONDS, INSURANCE, AND MISC. @ 5%				$12,334,462
ESTIMATED TOTAL ALTERNATIVE COST				$367,566,972

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTON AND MONITORING EQUIPMENT.

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	32.2	MG	$31,370,000	$31,370,000
NEW PUMP STATION AT WPRR	760	MGD	$64,303,217	$64,303,217
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR NEW 11 FOOT DIAMETER	8,400	LF	$1,568	$13,171,200
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW TREATMENT FACILITY AT WPRR	4.7	MG	$4,700,000	$4,700,000
SLUDGE PUMPING	522	GPM	$411,349	$411,349
NEW TREATED EFFLUENT PUMP STATION	75	MGD	$10,083,954	$10,083,954
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,625	LF	$456.33	$1,654,196
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	12,590	LF	$570.94	$7,188,135
NEW 57 INCH SEWERS	3,850	LF	$609.17	$2,345,305
NEW 60 INCH SEWERS	6,150	LF	$649.83	$3,996,455
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	4,455	LF	$870.84	$3,879,592
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	2,600	LF	$1,078.02	$2,802,852
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	1,300	LF	$1,303.88	$1,695,044
NEW 114 INCH SEWERS	500	LF	$1,373.90	$686,950
NEW 132 INCH SEWERS	13,075	LF	$1,567.63	$20,496,762
NEW 192 INCH CONDUIT	12,050	LF	$2,276.00	$27,425,800
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$253,463,275
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$456,280,875
CONSTRUCTION CONTINGENCY @ 25%				$114,070,219
DESIGN @ 6%				$27,376,852
CONSTRUCTION INSPECTION @ 10%				$45,628,087
ADMINISTRATION, LEGAL @ 3%				$13,688,426
BONDS, INSURANCE, AND MISC. @ 5%				$22,814,044
ESTIMATED TOTAL ALTERNATIVE COST				$679,858,503

NOTES:

(1) ESTIMAT4 OF TOTAL PORJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	8.5	MG	$8,500,000	$8,500,000
NEW PUMP STATION AT WPRR	320	MGD	$32,188,579	$32,188,579
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$241	$2,024,400
NEW 2.5 FOOT DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE/TREATMENT AT WPRR	1.6	MG	$5,328,125	$5,328,125
SLUDGE PUMPING AT WPRR	174	GPM	$238,460	$238,460
NEW FINAL EFFLUENT PUMP STATION	25	MGD	$4,187,298	$4,187,298
NEW STORAGE/TREATMENT AT PIONEER	30.[illegible]	MG	$29,755,000	$29,755,000
SLUDGE PUMPING	3,367	GPM	$1,034,944	$1,034,944
NEW PUMP STATION AT PIONEER	440	MGD	$41,528,282	$41,528,282
NEW FLUSH WATER INTAKE AT PIONEER	ALL REQ'D	LS	$125,000	$125,000
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
PIONEER CONNECTION TO STORAGE	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,000,000	$1,000,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,000,000	$1,000,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,625	LF	$456.33	$1,654,196
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	11,590	LF	$570.94	$6,617,195
NEW 60 INCH SEWERS	9,600	LF	$649.83	$6,238,368
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 69 INCH SEWERS	1,100	LF	$775.84	$853,424
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	4,855	LF	$870.84	$4,227,928
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	2,000	LF	$1,078.02	$2,156,040
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	1,300	LF	$1,303.88	$1,695,044
NEW 114 INCH SEWERS	500	LF	$1,373.90	$686,950
NEW 120 INCH SEWERS	900	LF	$1,446.65	$1,301,985
NEW 132 INCH SEWERS	13,075	LF	$1,567.63	$20,496,762
INSTALLED FORCE MAIN SEWER PIPE				
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$232,090,105
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$434,907,705
CONSTRUCTION CONTINGENCY @ 25%				$108,726,926
DESIGN @ 6%				$26,094,462
CONSTRUCTION INSPECTION @ 10%				$43,490,771
ADMINISTRATION, LEGAL @ 3%				$13,047,231
BONDS, INSURANCE, AND MISC. @ 5%				$21,745,385
ESTIMATED TOTAL ALTERNATIVE COST				$648,012,481

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPEMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND STORAGE AT UCD
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	32.2	MG	$31,370,000	$31,370,000
NEW PUMP STATION AT WPRR	590	MGD	$52,512,590	$52,512,590
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$1,135	$9,534,000
NEW 8 FOOT DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW TREATMENT FACILITY AT WPRR	4.7	MG	$4,700,000	$4,700,000
SLUDGE PUMPING	522	GPM	$411,349	$411,349
NEW TREATED EFFLUENT PUMP STATION	75	MGD	$10,083,954	$10,083,954
NEW STORAGE RESERVOIR AT UCD	11	MG	$13,350,000	$13,350,000
NEW PUMP STATION AT UCD	190	MGD	$21,212,151	$21,212,151
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,000,000	$1,000,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,000,000	$1,000,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,625	LF	$456.33	$1,654,196
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	12,590	LF	$570.94	$7,188,135
NEW 57 INCH SEWERS	3,850	LF	$609.17	$2,345,305
NEW 60 INCH SEWERS	5,300	LF	$649.83	$3,444,099
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	2,655	LF	$870.84	$2,312,080
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	3,800	LF	$1,078.02	$4,096,476
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	1,300	LF	$1,303.88	$1,695,044
NEW 114 INCH SEWERS	2,400	LF	$1,373.90	$3,297,360
NEW 132 INCH SEWERS	9,050	LF	$1,567.63	$14,187,052
NEW 192 INCH CONDUIT	12,050	LF	$2,276.00	$27,425,800
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$269,819,434
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$472,637,034
CONSTRUCTION CONTINGENCY @ 25%				$118,159,259
DESIGN @ 6%				$28,358,222
CONSTRUCTION INSPECTION @ 10%				$47,263,703
ADMINISTRATION, LEGAL @ 3%				$14,179,111
BONDS, INSURANCE, AND MISC. @ 5%				$23,631,852
ESTIMATED TOTAL ALTERNATIVE COST				$704,229,181

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT PIONEER, STORAGE AT UCD
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE RESERVOIR AT UCD	24	MG	$24,400,000	$24,400,000
NEW PUMP STATION AT UCD	310	MGD	$31,381,318	$31,381,318
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
NEW STORAGE/TREATMENT AT PIONEER	30.3	MG	$29,755,000	$29,755,000
SLUDGE PUMPING	3,367	GPM	$1,034,944	$1,034,944
NEW PUMP STATION AT PIONEER	440	MGD	$41,528,282	41528282.42
NEW FLUSH WATER INTAKE AT PIONEER	ALL REQ'D	LS	$125,000	$125,000
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
PIONEER CONNECTION TO STORAGE	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,000,000	$1,000,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,000,000	$1,000,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,325	LF	$456.33	$1,517,297
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	12,790	LF	$570.94	$7,302,323
NEW 57 INCH SEWERS	400	LF	$609.17	$243,668
NEW 60 INCH SEWERS	8,750	LF	$649.83	$5,686,013
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	5,455	LF	$870.84	$4,750,432
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	2,650	LF	$1,078.02	$2,856,753
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	3,300	LF	$1,303.88	$4,302,804
NEW 132 INCH SEWERS	9,700	LF	$1,567.63	$15,206,011
NEW 168 INCH CONDUIT	13,550	LF	$1,940.00	$26,287,000
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$257,126,350
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$459,943,950
CONSTRUCTION CONTINGENCY @ 25%				$114,985,987
DESIGN @ 6%				$27,596,637
CONSTRUCTION INSPECTION @ 10%				$45,994,395
ADMINISTRATION, LEGAL @ 3%				$13,798,318
BONDS, INSURANCE, AND MISC. @ 5%				$22,997,197
ESTIMATED TOTAL ALTERNATIVE COST				$685,316,485

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER, STORAGE AT UCD
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	10	MG	$10,000,000	$10,000,000
NEW PUMP STATION AT WPRR	170	MGD	$19,406,221	$19,406,221
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR NEW 2.5 INCH DIAMETER	8,400	LF	$241	$2,024,400
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE/TREATMENT AT WPRR	1.6	MG	$5,328,125	$5,328,125
SLUDGE PUMPING AT WPRR	174	GPM	$238,460	$238,460
NEW TREATED EFFLUENT PUMP STATION	25	MGD	$4,187,298	$4,187,298
NEW STORAGE RESERVOIR AT UCD	11	MG	$13,350,000	$13,350,000
NEW PUMP STATION AT UCD	190	MGD	$21,212,151	$21,212,151
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
NEW STORAGE/TREATMENT AT PIONEER	30.3	MG	$29,755,000	$29,755,000
SLUDGE PUMPING	3,367	GPM	$1,034,944	$1,034,944
NEW PUMP STATION AT PIONEER	440	MGD	$41,528,282	$41,528,282
NEW FLUSH WATER INTAKE AT PIONEER	ALL REQ'D	LS	$125,000	$125,000
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
PIONEER CONNECTION TO STORAGE	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,500,000	$1,500,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,500,000	$1,500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,625	LF	$456.33	$1,654,196
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	11,590	LF	$570.94	$6,617,195
NEW 57 INCH SEWERS	400	LF	$609.17	$243,668
NEW 60 INCH SEWERS	8,750	LF	$649.83	$5,686,013
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 69 INCH SEWERS	1,100	LF	$775.84	$853,424
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	2,655	LF	$870.84	$2,312,080
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	3,800	LF	$1,078.02	$4,096,476
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	1,300	LF	$1,303.88	$1,695,044
NEW 114 INCH SEWERS	2,400	LF	$1,373.90	$3,297,360
NEW 120 INCH SEWERS	900	LF	$1,446.65	$1,301,985
NEW 132 INCH SEWERS	9,050	LF	$1,567.63	$14,187,052
NEW 168 INCH CONDUIT	7,350	LF	$1,940.00	$14,259,000
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$267,392,877
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$470,210,477
CONSTRUCTION CONTINGENCY @ 25%				$117,552,619
DESIGN @ 6%				$28,212,629
CONSTRUCTION INSPECTION @ 10%				$47,021,048
ADMINISTRATION, LEGAL @ 3%				$14,106,314
BONDS, INSURANCE, AND MISC. @ 5%				$23,510,524
ESTIMATED TOTAL ALTERNATIVE COST				$700,613,611

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	760	MGD	$64,303,217	$64,303,217
NEW OUTFALL TO RIVER FROM WPRR NEW 14 FOOT DIAMETER	8,400	LF	$1,940	$16,296,000
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,625	LF	$456.33	$1,654,196
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	12,590	LF	$570.94	$7,188,135
NEW 57 INCH SEWERS	3,850	LF	$609.17	$2,345,305
NEW 60 INCH SEWERS	6,150	LF	$649.83	$3,996,455
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	4,855	LF	$870.84	$4,227,928
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	2,600	LF	$1,078.02	$2,802,852
NEW 96 INCH SEWERS	5,950	LF	$1,134.86	$6,752,417
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	1,300	LF	$1,303.88	$1,695,044
NEW 114 INCH SEWERS	500	LF	$1,373.90	$686,950
NEW 132 INCH SEWERS	13,075	LF	$1,567.63	$20,496,762
NEW 192 INCH CONDUIT	7,500	LF	$2,276.00	$17,070,000
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$203,224,977
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$406,042,577
CONSTRUCTION CONTINGENCY @ 25%				$101,510,644
DESIGN @ 6%				$24,362,555
CONSTRUCTION INSPECTION @ 10%				$40,604,258
ADMINISTRATION, LEGAL @ 3%				$12,181,277
BONDS, INSURANCE, AND MISC. @ 5%				$20,302,129
ESTIMATED TOTAL ALTERNATIVE COST				$605,003,440

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	320	MGD	$32,188,579	$32,188,579
NEW OUTFALL TO RIVER FROM WPRR NEW 9 FOOT DIAMETER	8,400	LF	$1,304	$10,953,600
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW PUMP STATION AT PIONEER	440	MGD	$41,528,282	$41,528,282
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,625	LF	$456.33	$1,654,196
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	11,590	LF	$570.94	$6,617,195
NEW 60 INCH SEWERS	9,600	LF	$649.83	$6,238,368
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 69 INCH SEWERS	1,100	LF	$775.84	$853,424
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	4,855	LF	$870.84	$4,227,928
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	2,000	LF	$1,078.02	$2,156,040
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	1,300	LF	$1,303.88	$1,695,044
NEW 114 INCH SEWERS	500	LF	$1,373.90	$686,950
NEW 120 INCH SEWERS	900	LF	$1,446.65	$1,301,985
NEW 132 INCH SEWERS	13,075	LF	$1,567.63	$20,496,762
INSTALLED FORCE MAIN SEWER PIPE				
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$188,850,479
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$391,668,079
CONSTRUCTION CONTINGENCY @ 25%				$97,917,020
DESIGN @ 6%				$23,500,085
CONSTRUCTION INSPECTION @ 10%				$39,166,808
ADMINISTRATION, LEGAL @ 3%				$11,750,042
BONDS, INSURANCE, AND MISC. @ 5%				$19,583,404
ESTIMATED TOTAL ALTERNATIVE COST				$583,585,438

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND STORAGE AT UCD
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	590	MGD	$52,512,590	$52,512,590
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$1,880	$15,792,000
NEW 13 FOOT DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE RESERVOIR AT UCD	11	MG	$11,000,000	$11,000,000
NEW PUMP STATION AT UCD	190	MGD	$21,212,151	$21,212,151
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,625	LF	$456.33	$1,654,196
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	12,590	LF	$570.94	$7,188,135
NEW 57 INCH SEWERS	3,850	LF	$609.17	$2,345,305
NEW 60 INCH SEWERS	5,300	LF	$649.83	$3,444,099
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	3,055	LF	$870.84	$2,660,416
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	3,800	LF	$1,078.02	$4,096,476
NEW 96 INCH SEWERS	5,950	LF	$1,134.86	$6,752,417
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	1,300	LF	$1,303.88	$1,695,044
NEW 114 INCH SEWERS	2,400	LF	$1,373.90	$3,297,360
NEW 132 INCH SEWERS	9,050	LF	$1,567.63	$14,187,052
NEW 192 INCH CONDUIT	7,500	LF	$2,276.00	$17,070,000
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$220,364,337
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$423,181,937
CONSTRUCTION CONTINGENCY @ 25%				$105,795,484
DESIGN @ 6%				$25,390,916
CONSTRUCTION INSPECTION @ 10%				$42,318,194
ADMINISTRATION, LEGAL @ 3%				$12,695,458
BONDS, INSURANCE, AND MISC. @ 5%				$21,159,097
ESTIMATED TOTAL ALTERNATIVE COST				$630,541,086

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, AQUISTION OF INSPECTION AND MONITORING EQUIPEMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT PIONEER, STORAGE AT UCD
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE RESERVOIR AT UCD	24	MG	$24,400,000	$24,400,000
NEW PUMP STATION AT UCD	310	MGD	$31,381,318	$31,381,318
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
NEW PUMP STATION AT PIONEER	440	MGD	$41,528,282	41528282.42
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,325	LF	$456.33	$1,517,297
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	12,790	LF	$570.94	$7,302,323
NEW 57 INCH SEWERS	400	LF	$609.17	$243,668
NEW 60 INCH SEWERS	8,750	LF	$649.83	$5,686,013
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	5,455	LF	$870.84	$4,750,432
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	2,650	LF	$1,078.02	$2,856,753
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	3,300	LF	$1,303.88	$4,302,804
NEW 132 INCH SEWERS	9,700	LF	$1,567.63	$15,206,011
NEW 168 INCH CONDUIT	13,550	LF	$1,940.00	$26,287,000
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$224,711,406
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$427,529,006
CONSTRUCTION CONTINGENCY @ 25%				$106,882,252
DESIGN @ 6%				$25,651,740
CONSTRUCTION INSPECTION @ 10%				$42,752,901
ADMINISTRATION, LEGAL @ 3%				$12,825,870
BONDS, INSURANCE, AND MISC. @ 5%				$21,376,450
ESTIMATED TOTAL ALTERNATIVE COST				$637,018,219

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
5-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER, STORAGE AT UCD
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	170	MGD	$19,406,221	$19,406,221
NEW OUTFALL TO RIVER FROM WPRR NEW 7 FOOT DIAMETER	8,400	LF	$950	$7,980,000
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE RESERVOIR AT UCD	11	MG	$11,000,000	$11,000,000
NEW PUMP STATION AT UCD	190	MGD	$21,212,151	$21,212,151
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
NEW PUMP STATION AT PIONEER	440	MGD	$41,528,282	$41,528,282
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	4,795	LF	$129.35	$620,233
NEW 21 INCH SEWERS	4,795	LF	$147.05	$705,105
NEW 24 INCH SEWERS	2,540	LF	$209.41	$531,901
NEW 27 INCH SEWERS	14,140	LF	$237.49	$3,358,109
NEW 30 INCH SEWERS	12,140	LF	$269.03	$3,266,024
NEW 33 INCH SEWERS	10,905	LF	$302.12	$3,294,619
NEW 36 INCH SEWERS	19,310	LF	$341.67	$6,597,648
NEW 39 INCH SEWERS	1,370	LF	$379.83	$520,367
NEW 42 INCH SEWERS	14,961	LF	$416.47	$6,230,808
NEW 45 INCH SEWERS	3,625	LF	$456.33	$1,654,196
NEW 48 INCH SEWERS	2,435	LF	$500.82	$1,219,497
NEW 51 INCH SEWERS	1,200	LF	$535.07	$642,084
NEW 54 INCH SEWERS	11,590	LF	$570.94	$6,617,195
NEW 57 INCH SEWERS	400	LF	$609.17	$243,668
NEW 60 INCH SEWERS	8,750	LF	$649.83	$5,686,013
NEW 63 INCH SEWERS	1,600	LF	$688.11	$1,100,976
NEW 66 INCH SEWERS	4,555	LF	$733.78	$3,342,368
NEW 69 INCH SEWERS	1,100	LF	$775.84	$853,424
NEW 72 INCH SEWERS	8,280	LF	$809.46	$6,702,329
NEW 78 INCH SEWERS	2,655	LF	$870.84	$2,312,080
NEW 84 INCH SEWERS	6,565	LF	$949.92	$6,236,225
NEW 90 INCH SEWERS	3,800	LF	$1,078.02	$4,096,476
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	3,000	LF	$1,233.58	$3,700,740
NEW 108 INCH SEWERS	1,300	LF	$1,303.88	$1,695,044
NEW 114 INCH SEWERS	2,400	LF	$1,373.90	$3,297,360
NEW 120 INCH SEWERS	900	LF	$1,446.65	$1,301,985
NEW 132 INCH SEWERS	9,050	LF	$1,567.63	$14,187,052
NEW 168 INCH CONDUIT	7,350	LF	$1,940.00	$14,259,000
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 2.50 FOOT WEIR	1	EA	$146,693	$146,693
NEW 3.00 FOOT WEIR	3	EA	$156,656	$469,968
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.00 FOOT WEIR	4	EA	$177,301	$709,205
NEW 4.50 FOOT WEIR	7	EA	$185,691	$1,299,835
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	4	EA	$202,966	$811,864
NEW 6.00 FOOT WEIR	1	EA	$211,846	$211,846
NEW 6.50 FOOT WEIR	1	EA	$220,883	$220,883
NEW 9.50 FOOT WEIR	1	EA	$270,735	$270,735
SUBTOTAL				$217,329,651
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$181,080,000	$181,080,000
SUBTOTAL				$420,147,251
CONSTRUCTION CONTINGENCY @ 25%				$105,036,813
DESIGN @ 6%				$25,208,835
CONSTRUCTION INSPECTION @ 10%				$42,014,725
ADMINISTRATION, LEGAL @ 3%				$12,604,418
BONDS, INSURANCE, AND MISC. @ 5%				$21,007,363
ESTIMATED TOTAL ALTERNATIVE COST				$626,019,404

NOTES:

(1) ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR COSTS FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

File: str5$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	60	MG	$55,000,000	$55,000,000
NEW PUMP STATION AT WPRR	1,550	MGD	$113,722,706	$113,722,706
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR NEW 18 FOOT DIAMETER	8,400	LF	$2,881	$24,200,400
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	13,951	LF	$500.82	$6,986,940
NEW 54 INCH SEWERS	6,295	LF	$570.94	$3,594,067
NEW 60 INCH SEWERS	11,490	LF	$649.83	$7,466,547
NEW 66 INCH SEWERS	4,800	LF	$733.78	$3,522,144
NEW 69 INCH SEWERS	4,750	LF	$775.84	$3,685,240
NEW 72 INCH SEWERS	14,335	LF	$809.46	$11,603,609
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	4,050	LF	$949.92	$3,847,176
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	7,465	LF	$1,134.86	$8,471,730
NEW 108 INCH SEWERS	2,650	LF	$1,303.88	$3,455,282
NEW 132 INCH SEWERS	5,300	LF	$1,567.63	$8,308,439
NEW 144 INCH SEWERS	1,300	LF	$1,690.91	$2,198,183
NEW 180 INCH CONDUIT	6,525	LF	$2,000.00	$13,050,000
NEW 240 INCH CONDUIT	18,600	LF	$3,556.00	$66,141,600
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$371,625,734
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000.00	$198,580,000
SUBTOTAL				$591,943,334
CONSTRUCTION CONTINGENCY @ 25%				$147,985,833
DESIGN @ 6%				$35,516,600
CONSTRUCTION INSPECTION @ 10%				$59,194,333
ADMINISTRATION, LEGAL @ 3%				$17,758,300
BONDS, INSURANCE, AND MISC. @ 5%				$29,597,167
ESTIMATED TOTAL ALTERNATIVE COST				$881,995,567

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.
FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	2.8	MG	$2,800,000	$2,800,000
NEW PUMP STATION AT WPRR	640	MGD	$56,043,572	$56,043,572
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR NEW 13 FOOT DIAMETER	8,400	LF	$1,880	$15,792,000
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE/TREATMENT AT WPRR	3.1	MG	$3,100,000	$3,100,000
SLUDGE PUMPING AT WPRR	344	GPM	$334,755	$334,755
NEW EFFLUENT PUMP STATION AT WPRR	50	MGD	$7,290,509	$7,290,509
NEW STORAGE/TREATMENT AT PIONEER	29.7	MG	$29,245,000	$29,245,000
SLUDGE PUMPING	3,300	GPM	$1,024,746	$1,024,746
NEW PUMP STATION AT PIONEER	890	MGD	$72,961,513	$72,961,513
NEW FLUSH WATER INTAKE AT PIONEER	ALL REQ'D	LS	$125,000	$125,000
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
PIONEER CONNECTION TO STORAGE	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,000,000	$1,000,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,000,000	$1,000,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	13,951	LF	$500.82	$6,986,940
NEW 54 INCH SEWERS	6,295	LF	$570.94	$3,594,067
NEW 60 INCH SEWERS	11,490	LF	$649.83	$7,466,547
NEW 66 INCH SEWERS	4,800	LF	$733.78	$3,522,144
NEW 69 INCH SEWERS	1,300	LF	$775.84	$1,008,592
NEW 72 INCH SEWERS	16,385	LF	$809.46	$13,263,002
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	5,150	LF	$949.92	$4,892,088
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	7,465	LF	$1,134.86	$8,471,730
NEW 108 INCH SEWERS	2,650	LF	$1,303.88	$3,455,282
NEW 132 INCH SEWERS	5,300	LF	$1,567.63	$8,308,439
NEW 144 INCH SEWERS	2,200	LF	$1,690.91	$3,720,002
NEW 180 INCH CONDUIT	6,525	LF	$2,000.00	$13,050,000
NEW 240 INCH CONDUIT	13,900	LF	$3,556.00	$49,428,400
INSTALLED FORCE MAIN SEWER PIPE				
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$356,255,659
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$576,573,259
CONSTRUCTION CONTINGENCY @ 25%				$144,143,315
DESIGN @ 6%				$34,594,396
CONSTRUCTION INSPECTION @ 10%				$57,657,326
ADMINISTRATION, LEGAL @ 3%				$17,297,198
BONDS, INSURANCE, AND MISC. @ 5%				$28,828,663
ESTIMATED TOTAL ALTERNATIVE COST				$859,094,156

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND UCD
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	60	MG	$55,000,000	$55,000,000
NEW PUMP STATION AT WPRR	1,210	MGD	$93,284,637	$93,284,637
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$1,940	$16,296,000
NEW 14 FOOT DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE RESERVOIR AT UCD	32	MG	$31,200,000	$31,200,000
NEW PUMP STATION AT UCD	370	MGD	$36,152,906	$36,152,906
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,000,000	$1,000,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,000,000	$1,000,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	12,401	LF	$500.82	$6,210,669
NEW 54 INCH SEWERS	6,120	LF	$570.94	$3,494,153
NEW 60 INCH SEWERS	11,490	LF	$649.83	$7,466,547
NEW 66 INCH SEWERS	6,525	LF	$733.78	$4,787,915
NEW 69 INCH SEWERS	4,750	LF	$775.84	$3,685,240
NEW 72 INCH SEWERS	12,985	LF	$809.46	$10,510,838
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	4,050	LF	$949.92	$3,847,176
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	6,165	LF	$1,134.86	$6,996,412
NEW 108 INCH SEWERS	4,450	LF	$1,303.88	$5,802,266
NEW 114 INCH SEWERS	3,375	LF	$1,373.90	$4,636,913
NEW 132 INCH SEWERS	4,800	LF	$1,567.63	$7,524,624
NEW 144 INCH SEWERS	1,300	LF	$1,690.91	$2,198,183
NEW 156 INCH CONDUIT	500	LF	$1,880.00	$940,000
NEW 168 INCH CONDUIT	4,400	LF	$1,940.00	$8,536,000
NEW 240 INCH CONDUIT	18,600	LF	$3,556.00	$66,141,600
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$412,831,127
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$633,148,727
CONSTRUCTION CONTINGENCY @ 25%				$158,287,182
DESIGN @ 6%				$37,988,924
CONSTRUCTION INSPECTION @ 10%				$63,314,873
ADMINISTRATION, LEGAL @ 3%				$18,994,462
BONDS, INSURANCE, AND MISC. @ 5%				$31,657,436
ESTIMATED TOTAL ALTERNATIVE COST				$943,391,603

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT PIONEER, STORAGE AT UCD
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE RESERVOIR AT UCD	63	MG	$57,550,000	$57,550,000
NEW PUMP STATION AT UCD	590	MGD	$52,512,590	$52,512,590
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
NEW STORAGE/TREATMENT AT PIONEER	29.7	MG	$29,245,000	$29,245,000
SLUDGE PUMPING	3,300	GPM	$1,024,746	$1,024,746
NEW PUMP STATION AT PIONEER	890	MGD	$72,961,513	$72,961,513
NEW FLUSH WATER INTAKE AT PIONEER	ALL REQ'D	LS	$125,000	$125,000
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
PIONEER CONNECTION TO STORAGE	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,000,000	$1,000,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,000,000	$1,000,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	12,401	LF	$500.82	$6,210,669
NEW 54 INCH SEWERS	6,120	LF	$570.94	$3,494,153
NEW 60 INCH SEWERS	11,190	LF	$649.83	$7,271,598
NEW 63 INCH SEWERS	1,200	LF	$688.11	$825,732
NEW 66 INCH SEWERS	6,525	LF	$733.78	$4,787,915
NEW 69 INCH SEWERS	1,300	LF	$775.84	$1,008,592
NEW 72 INCH SEWERS	15,235	LF	$809.46	$12,332,123
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	4,050	LF	$949.92	$3,847,176
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	6,165	LF	$1,134.86	$6,996,412
NEW 108 INCH SEWERS	6,100	LF	$1,303.88	$7,953,668
NEW 132 INCH SEWERS	4,800	LF	$1,567.63	$7,524,624
NEW 144 INCH SEWERS	1,300	LF	$1,690.91	$2,198,183
NEW 204 INCH CONDUIT	11,350	LF	$2,569.00	$29,158,150
NEW 240 INCH CONDUIT	13,900	LF	$3,556.00	$49,428,400
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$396,324,953
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$616,642,553
CONSTRUCTION CONTINGENCY @ 25%				$154,160,638
DESIGN @ 6%				$36,998,553
CONSTRUCTION INSPECTION @ 10%				$61,664,255
ADMINISTRATION, LEGAL @ 3%				$18,499,277
BONDS, INSURANCE, AND MISC. @ 5%				$30,832,128
ESTIMATED TOTAL ALTERNATIVE COST				$918,797,405

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER, STORAGE AT UCD
CONFIGURED FOR 1/YEAR CSO CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE AT WPRR	2.8	MG	$2,800,000	$2,800,000
NEW PUMP STATION AT WPRR	310	MGD	$31,381,318	$31,381,318
NEW FLUSH WATER WELL FIELD AT WPRR	ALL REQ'D	LS	$500,000	$500,000
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$1,304	$10,953,600
NEW 9 FOOT DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE/TREATMENT AT PIONEER	3.1	MG	$6,656,250	$6,656,250
SLUDGE PUMPING	347	GPM	$336,089	$336,089
NEW TREATED EFFLUENT PUMP STATION	50	MGD	$7,290,509	72905[illegible].685
NEW STORAGE RESERVOIR AT UCD	32	MG	$31,200,000	$31,200,000
NEW PUMP STATION AT UCD	370	MGD	$36,152,906	$36,152,906
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
NEW STORAGE/TREATMENT AT PIONEER	29.7	MG	$29,245,000	$29,245,000
SLUDGE PUMPING	3,300	GPM	$1,024,746	$1,024,746
NEW PUMP STATION AT PIONEER	890	MGD	$72,961,513	72961513.36
NEW FLUSH WATER INTAKE AT PIONEER	ALL REQ'D	LS	$125,000	$125,000
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
PIONEER CONNECTION TO STORAGE	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,500,000	$1,500,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,500,000	$1,500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	13,951	LF	$500.82	$6,986,940
NEW 54 INCH SEWERS	6,295	LF	$570.94	$3,594,067
NEW 60 INCH SEWERS	11,490	LF	$649.83	$7,466,547
NEW 66 INCH SEWERS	4,800	LF	$733.78	$3,522,144
NEW 69 INCH SEWERS	1,300	LF	$775.84	$1,008,592
NEW 72 INCH SEWERS	14,535	LF	$809.46	$11,765,501
NEW 78 INCH SEWERS	3,505	LF	$870.84	$3,052,294
NEW 84 INCH SEWERS	5,150	LF	$949.92	$4,892,088
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	6,165	LF	$1,134.86	$6,996,412
NEW 108 INCH SEWERS	4,450	LF	$1,303.88	$5,802,266
NEW 132 INCH SEWERS	4,800	LF	$1,567.63	$7,524,624
NEW 144 INCH SEWERS	1,800	LF	$1,690.91	$3,043,638
NEW 156 INCH CONDUIT	5,300	LF	$1,880.00	$9,964,000
NEW 240 INCH CONDUIT	13,900	LF	$3,556.00	$49,428,400
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$394,502,113
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$614,819,713
CONSTRUCTION CONTINGENCY @ 25%				$153,704,928
DESIGN @ 6%				$36,889,183
CONSTRUCTION INSPECTION @ 10%				$61,481,971
ADMINISTRATION, LEGAL @ 3%				$18,444,591
BONDS, INSURANCE, AND MISC. @ 5%				$30,740,986
ESTIMATED TOTAL ALTERNATIVE COST				$916,081,372

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE AT WESTERN PACIFIC
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	1,550	MGD	$113,722,706	$113,722,706
NEW OUTFALL TO RIVER FROM WPRR NEW 20 FOOT DIAMETER	8,400	LF	$3,556	$29,870,400
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,09[illegible]
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	13,951	LF	$500.82	$6,986,940
NEW 54 INCH SEWERS	6,295	LF	$570.94	$3,594,067
NEW 60 INCH SEWERS	11,490	LF	$649.83	$7,466,547
NEW 66 INCH SEWERS	4,800	LF	$733.78	$3,522,144
NEW 69 INCH SEWERS	4,750	LF	$775.84	$3,685,240
NEW 72 INCH SEWERS	14,335	LF	$809.46	$11,603,609
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	4,050	LF	$949.92	$3,847,176
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	7,465	LF	$1,134.86	$8,471,730
NEW 108 INCH SEWERS	2,650	LF	$1,303.88	$3,455,282
NEW 114 INCH SEWERS	400	LF	$1,373.90	$549,560
NEW 132 INCH SEWERS	9,450	LF	$1,567.63	$14,814,104
NEW 144 INCH SEWERS	1,300	LF	$1,690.91	$2,198,183
NEW 180 INCH CONDUIT	6,525	LF	$2,000.00	$13,050,000
NEW 240 INCH CONDUIT	14,050	LF	$3,556.00	$49,961,800
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$311,671,158
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$531,988,758
CONSTRUCTION CONTINGENCY @ 25%				$132,997,190
DESIGN @ 6%				$31,919,325
CONSTRUCTION INSPECTION @ 10%				$53,198,876
ADMINISTRATION, LEGAL @ 3%				$15,959,663
BONDS, INSURANCE, AND MISC. @ 5%				$26,599,438
ESTIMATED TOTAL ALTERNATIVE COST				$792,663,250

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	640	MGD	$56,043,572	$56,043,572
NEW OUTFALL TO RIVER FROM WPRR	8,400	LF	$1,880	$15,792,000
NEW 13 FOOT DIAMETER				
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW PUMP STATION AT PIONEER	890	MGD	$72,961,513	$72,961,513
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	13,951	LF	$500.82	$6,986,940
NEW 54 INCH SEWERS	6,295	LF	$570.94	$3,594,067
NEW 60 INCH SEWERS	11,490	LF	$649.83	$7,466,547
NEW 66 INCH SEWERS	4,800	LF	$733.78	$3,522,144
NEW 69 INCH SEWERS	1,300	LF	$775.84	$1,008,592
NEW 72 INCH SEWERS	16,385	LF	$809.46	$13,263,002
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	5,150	LF	$949.92	$4,892,088
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	7,465	LF	$1,134.86	$8,471,730
NEW 108 INCH SEWERS	2,650	LF	$1,303.88	$3,455,282
NEW 132 INCH SEWERS	5,300	LF	$1,567.63	$8,308,439
NEW 144 INCH SEWERS	2,200	LF	$1,690.91	$3,720,002
NEW 180 INCH CONDUIT	6,525	LF	$2,000.00	$13,050,000
NEW 240 INCH CONDUIT	13,900	LF	$3,556.00	$49,428,400
INSTALLED FORCE MAIN SEWER PIPE				
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$309,335,649
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$529,653,249
CONSTRUCTION CONTINGENCY @ 25%				$132,413,312
DESIGN @ 6%				$31,779,195
CONSTRUCTION INSPECTION @ 10%				$52,965,325
ADMINISTRATION, LEGAL @ 3%				$15,889,597
BONDS, INSURANCE, AND MISC. @ 5%				$26,482,662
ESTIMATED TOTAL ALTERNATIVE COST				$789,183,340

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND UCD
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	1,210	MGD	$93,284,637	$93,284,637
NEW OUTFALL TO RIVER FROM WPRR NEW 18 FOOT DIAMETER	8,400	LF	$2,881	$24,200,400
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE RESERVOIR AT UCD	32	MG	$31,200,000	$31,200,000
NEW PUMP STATION AT UCD	370	MGD	$36,152,906	$36,152,906
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	12,401	LF	$500.82	$6,210,669
NEW 54 INCH SEWERS	6,120	LF	$570.94	$3,494,153
NEW 60 INCH SEWERS	11,490	LF	$649.83	$7,466,547
NEW 66 INCH SEWERS	6,525	LF	$733.78	$4,787,915
NEW 69 INCH SEWERS	4,750	LF	$775.84	$3,685,240
NEW 72 INCH SEWERS	12,985	LF	$809.46	$10,510,838
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	4,050	LF	$949.92	$3,847,176
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	6,165	LF	$1,134.86	$6,996,412
NEW 108 INCH SEWERS	4,450	LF	$1,303.88	$5,802,266
NEW 114 INCH SEWERS	3,775	LF	$1,373.90	$5,186,473
NEW 132 INCH SEWERS	8,950	LF	$1,567.63	$14,030,289
NEW 144 INCH SEWERS	1,300	LF	$1,690.91	$2,198,183
NEW 156 INCH CONDUIT	500	LF	$1,880.00	$940,000
NEW 168 INCH CONDUIT	4,400	LF	$1,940.00	$8,536,000
NEW 240 INCH CONDUIT	14,050	LF	$3,556.00	$49,961,800
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$355,110,952
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$575,428,552
CONSTRUCTION CONTINGENCY @ 25%				$143,857,138
DESIGN @ 6%				$34,525,713
CONSTRUCTION INSPECTION @ 10%				$57,542,855
ADMINISTRATION, LEGAL @ 3%				$17,262,857
BONDS, INSURANCE, AND MISC. @ 5%				$28,771,428
ESTIMATED TOTAL ALTERNATIVE COST				$857,388,542

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT PIONEER, STORAGE AT UCD
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW STORAGE RESERVOIR AT UCD	63	MG	$57,550,000	$57,550,000
NEW PUMP STATION AT UCD	590	MGD	$52,512,590	$52,512,590
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
NEW PUMP STATION AT PIONEER	890	MGD	$72,961,513	$72,961,513
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	12,401	LF	$500.82	$6,210,669
NEW 54 INCH SEWERS	6,120	LF	$570.94	$3,494,153
NEW 60 INCH SEWERS	11,190	LF	$649.83	$7,271,598
NEW 63 INCH SEWERS	1,200	LF	$688.11	$825,732
NEW 66 INCH SEWERS	6,525	LF	$733.78	$4,787,915
NEW 69 INCH SEWERS	1,300	LF	$775.84	$1,008,592
NEW 72 INCH SEWERS	15,235	LF	$809.46	$12,332,123
NEW 78 INCH SEWERS	3,205	LF	$870.84	$2,791,042
NEW 84 INCH SEWERS	4,050	LF	$949.92	$3,847,176
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	6,165	LF	$1,134.86	$6,996,412
NEW 108 INCH SEWERS	6,100	LF	$1,303.88	$7,953,668
NEW 132 INCH SEWERS	4,800	LF	$1,567.63	$7,524,624
NEW 144 INCH SEWERS	1,300	LF	$1,690.91	$2,198,183
NEW 204 INCH CONDUIT	11,350	LF	$2,569.00	$29,158,150
NEW 240 INCH CONDUIT	13,900	LF	$3,556.00	$49,428,400
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$364,430,207
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$584,747,807
CONSTRUCTION CONTINGENCY @ 25%				$146,186,952
DESIGN @ 6%				$35,084,868
CONSTRUCTION INSPECTION @ 10%				$58,474,781
ADMINISTRATION, LEGAL @ 3%				$17,542,434
BONDS, INSURANCE, AND MISC. @ 5%				$29,237,390
ESTIMATED TOTAL ALTERNATIVE COST				$871,274,233

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED TOTAL PROJECT COSTS
10-YEAR STORAGE/TREATMENT AT WESTERN PACIFIC AND PIONEER, STORAGE AT UCD
CONFIGURED FOR 5/YEAR CSO CONTROL AND FLOOD CONTROL

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT WPRR	310	MGD	$31,381,318	$31,381,318
NEW OUTFALL TO RIVER FROM WPRR NEW 9 FOOT DIAMETER	8,400	LF	$1,304	$10,953,600
NEW OUTFALL STRUCTURE AT RIVER	ALL REQ'D	LS	$750,000	$750,000
NEW STORAGE RESERVOIR AT UCD	32	MG	$31,200,000	$31,200,000
NEW PUMP STATION AT UCD	370	MGD	$36,152,906	$36,152,906
NEW FLUSH WATER WELL FIELD AT UCD	ALL REQ'D	LS	$500,000	$500,000
NEW PUMP STATION AT PIONEER	890	MGD	$72,961,513	$72,961,513
NEW OUTFALL LINE AT PIONEER	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	1,020	LF	$129.35	$131,937
NEW 21 INCH SEWERS	2,680	LF	$147.05	$394,094
NEW 24 INCH SEWERS	5,565	LF	$209.41	$1,165,367
NEW 27 INCH SEWERS	6,795	LF	$237.49	$1,613,745
NEW 30 INCH SEWERS	11,530	LF	$269.03	$3,101,916
NEW 33 INCH SEWERS	8,145	LF	$302.12	$2,460,767
NEW 36 INCH SEWERS	16,780	LF	$341.67	$5,733,223
NEW 42 INCH SEWERS	15,960	LF	$416.47	$6,646,861
NEW 48 INCH SEWERS	13,951	LF	$500.82	$6,986,940
NEW 54 INCH SEWERS	6,295	LF	$570.94	$3,594,067
NEW 60 INCH SEWERS	11,490	LF	$649.83	$7,466,547
NEW 66 INCH SEWERS	4,800	LF	$733.78	$3,522,144
NEW 69 INCH SEWERS	1,300	LF	$775.84	$1,008,592
NEW 72 INCH SEWERS	14,535	LF	$809.46	$11,765,501
NEW 78 INCH SEWERS	3,505	LF	$870.84	$3,052,294
NEW 84 INCH SEWERS	5,150	LF	$949.92	$4,892,088
NEW 90 INCH SEWERS	3,300	LF	$1,078.02	$3,557,466
NEW 96 INCH SEWERS	6,165	LF	$1,134.86	$6,996,412
NEW 108 INCH SEWERS	4,450	LF	$1,303.88	$5,802,266
NEW 132 INCH SEWERS	4,800	LF	$1,567.63	$7,524,624
NEW 144 INCH SEWERS	1,800	LF	$1,690.91	$3,043,638
NEW 156 INCH CONDUIT	5,300	LF	$1,880.00	$9,964,000
NEW 240 INCH CONDUIT	13,900	LF	$3,556.00	$49,428,400
INSTALLED FORCE MAIN SEWER PIPE				
NEW 24 INCH SEWERS	2,000	LF	$123.69	$247,380
NEW 120 INCH SEWERS	2,000	LF	$874.83	$1,749,660
DIVERSION WEIRS				
NEW 3.00 FOOT WEIR	2	EA	$156,656	$313,312
NEW 3.50 FOOT WEIR	1	EA	$166,869	$166,869
NEW 4.50 FOOT WEIR	1	EA	$185,691	$185,691
NEW 5.00 FOOT WEIR	3	EA	$194,262	$582,787
NEW 5.50 FOOT WEIR	8	EA	$202,966	$1,623,728
NEW 6.00 FOOT WEIR	4	EA	$211,846	$847,384
NEW 6.50 FOOT WEIR	5	EA	$220,883	$1,104,417
NEW 7.00 FOOT WEIR	3	EA	$227,569	$682,707
NEW 7.50 FOOT WEIR	1	EA	$234,681	$234,681
NEW 8.00 FOOT WEIR	2	EA	$244,067	$488,134
NEW 11.00 FOOT WEIR	1	EA	$295,544	$295,544
SUBTOTAL				$344,024,518
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$564,342,118
CONSTRUCTION CONTINGENCY @ 25%				$141,085,530
DESIGN @ 6%				$33,860,527
CONSTRUCTION INSPECTION @ 10%				$56,434,212
ADMINISTRATION, LEGAL @ 3%				$16,930,264
BONDS, INSURANCE, AND MISC. @ 5%				$28,217,106
ESTIMATED TOTAL ALTERNATIVE COST				$840,869,757

NOTE:
(1) THIS ESTIMATE OF TOTAL PROJECT COST DOES NOT INCLUDE PIPELINE REHABILITATION COSTS, NOR FOR AQUISTION OF INSPECTION AND MONITORING EQUIPMENT.

FILE: Str 10$.wk3

ESTIMATED PROJECT COSTS
10-YEAR SEPARATION ALTERNATIVE - STORM DRAIN

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT SUMP 1	1,080	MGD	$85,176,704	$85,176,704
NEW PUMP STATION AT SUMP 2	1,270	MGD	$96,967,189	$96,967,189
NEW OUTFALL LINE AT SUMP 1	ALL REQ'D	LS	$750,000	$750,000
NEW OUTFALL LINE AT SUMP 2	ALL REQ'D	LS	$750,000	$750,000
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$1,000,000	$1,000,000
MISC. CROSSINGS	ALL REQ'D	LS	$1,000,000	$1,000,000
INSTALLED GRAVITY SEWER PIPE				
NEW 18 INCH SEWERS	200	LF	$129.35	$25,870
NEW 21 INCH SEWERS	1,570	LF	$147.05	$230,869
NEW 24 INCH SEWERS	2,505	LF	$209.41	$524,572
NEW 27 INCH SEWERS	5,150	LF	$237.49	$1,223,074
NEW 30 INCH SEWERS	6,105	LF	$269.03	$1,642,428
NEW 33 INCH SEWERS	11,520	LF	$302.12	$3,480,422
NEW 36 INCH SEWERS	7,600	LF	$341.67	$2,596,692
NEW 39 INCH SEWERS	4,940	LF	$379.83	$1,876,360
NEW 42 INCH SEWERS	18,495	LF	$416.47	$7,702,613
NEW 45 INCH SEWERS	13,785	LF	$456.33	$6,290,509
NEW 48 INCH SEWERS	15,641	LF	$500.82	$7,833,326
NEW 51 INCH SEWERS	14,145	LF	$535.07	$7,568,565
NEW 54 INCH SEWERS	6,400	LF	$570.94	$3,654,016
NEW 57 INCH SEWERS	1,300	LF	$609.17	$791,921
NEW 60 INCH SEWERS	12,035	LF	$649.83	$7,820,704
NEW 63 INCH SEWERS	10,480	LF	$688.11	$7,211,393
NEW 66 INCH SEWERS	5,210	LF	$733.78	$3,822,994
NEW 69 INCH SEWERS	13,850	LF	$775.84	$10,745,384
NEW 72 INCH SEWERS	1,900	LF	$809.46	$1,537,974
NEW 78 INCH SEWERS	6,775	LF	$870.84	$5,899,941
NEW 84 INCH SEWERS	12,375	LF	$949.92	$11,755,260
NEW 90 INCH SEWERS	7,650	LF	$1,078.02	$8,246,853
NEW 96 INCH SEWERS	3,700	LF	$1,134.86	$4,198,982
NEW 102 INCH SEWERS	7,645	LF	$1,233.58	$9,430,719
NEW 108 INCH SEWERS	7,010	LF	$1,303.88	$9,140,199
NEW 114 INCH SEWERS	1,750	LF	$1,373.90	$2,404,325
NEW 120 INCH SEWERS	4,445	LF	$1,446.65	$6,430,359
NEW 132 INCH SEWERS	12,700	LF	$1,567.63	$19,908,901
NEW 228 INCH CONDUIT	14,950	LF	$3,209.00	$47,974,550
NEW 240 INCH CONDUIT	10,800	LF	$3,556.00	$38,404,800
SUBTOTAL				$426,018,467
SUMP 1, 1A, AND 2 REHABILITATION, SHORT TERM	ALL REQ'D	LS	$2,746,500	$2,746,500
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$198,580,000	$198,580,000
SUBTOTAL				$628,107,567
CONSTRUCTION CONTINGENCY @ 25%				$157,026,892
DESIGN @ 6%				$37,686,454
CONSTRUCTION INSPECTION @ 10%				$62,810,757
ADMINISTRATION, LEGAL @ 3%				$18,843,227
BONDS, INSURANCE, AND MISC. @ 5%				$31,405,378
ESTIMATED TOTAL ALTERNATIVE COST				$935,880,275

ESTIMATED TOTAL PROJECT COSTS
SEPARATION ALTERNATIVE - SANITARY SEWER

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT SUMP 2	110	MGD	$13,699,224	$13,699,224
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 8 INCH SEWERS	8,425	LF	$73.48	$619,038
NEW 10 INCH SEWERS	29,385	LF	$82.90	$2,435,969
NEW 12 INCH SEWERS	25,670	LF	$94.77	$2,432,746
NEW 15 INCH SEWERS	36,731	LF	$111.35	$4,089,997
NEW 18 INCH SEWERS	26,865	LF	$129.35	$3,474,988
NEW 21 INCH SEWERS	10,085	LF	$147.05	$1,482,999
NEW 24 INCH SEWERS	20,220	LF	$209.41	$4,234,270
NEW 27 INCH SEWERS	13,195	LF	$237.49	$3,133,681
NEW 30 INCH SEWERS	13,105	LF	$269.03	$3,525,638
NEW 33 INCH SEWERS	9,375	LF	$302.12	$2,832,375
NEW 36 INCH SEWERS	8,050	LF	$341.67	$2,750,444
NEW 39 INCH SEWERS	8,010	LF	$379.83	$3,042,438
NEW 42 INCH SEWERS	2,430	LF	$416.47	$1,012,022
NEW 45 INCH SEWERS	5,465	LF	$456.33	$2,493,843
NEW 54 INCH SEWERS	1,700	LF	$570.94	$970,598
NEW 57 INCH SEWERS	4,330	LF	$609.17	$2,637,706
NEW 60 INCH SEWERS	5,770	LF	$649.83	$3,749,519
NEW 63 INCH SEWERS	800	LF	$688.11	$550,488
NEW 66 INCH SEWERS	3,300	LF	$733.78	$2,421,474
NEW 69 INCH SEWERS	550	LF	$775.84	$426,712
NEW 72 INCH SEWERS	1,600	LF	$809.46	$1,295,136
NEW 78 INCH SEWERS	6,020	LF	$870.84	$5,242,457
NEW 84 INCH SEWERS	2,250	LF	$949.92	$2,137,320
NEW 96 INCH SEWERS	1,800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	900	LF	$1,233.58	$1,110,222
SUBTOTAL				$74,844,053
SUMP 1, 1A, AND 2 REHABILITATION, LONG TERM	ALL REQ'D	LS	$20,975,000	$20,975,000
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
*COLLECTION SYSTEM UPGRADE	ALL REQ'D	LS	$214,910,000	$214,910,000
SUBTOTAL				$311,491,653
CONSTRUCTION CONTINGENCY @ 25%				$77,872,913
DESIGN @ 6%				$18,689,499
CONSTRUCTION INSPECTION @ 10%				$31,149,165
ADMINISTRATION, LEGAL @ 3%				$9,344,750
BONDS, INSURANCE, AND MISC. @ 5%				$15,574,583
ESTIMATED TOTAL ALTERNATIVE COST				$464,122,562

NOTES:
(1) NEW COLLECTION SYSTEM SEWERS ARE SEWERS NOT MODELED IN SWIMM.

File: SSO1.wk1

APPENDIX F
INLET CONTROL PROJECT
BY MR. JOHN TOMKO

John J. Tomko, P.E.
Hazardous Materials and
Water Quality Consultant

JJT MEMORANDUM

TO: Gary Reents
Ken Fischer
Craig Chalmers

FROM: John Tomko, Consulting Engineer

DATE: May 14, 1992

SUBJECT: INLET CONTROL PROJECT

Attached is a report entitled "Inlet Control Project - SWMM Basin #7623". This report indicates that a 10-year inlet control project can be constructed in this SWMM basin for approximately $650,000. The analytical procedures used in this report are described in the report, its appendices, and in a master spreadsheet included on a computer disk.

Also, submitted under this memorandum are the original survey field notes collected for three other SWMM basins, #3491 (K and 16th Street), #5681 (E and 25th Street), and #1561 (25th Street and 4th Avenue). Since these are the only copies of these data sheets, I suggest that additional copies of these survey sheets be made. In addition, a 1"- 200' map of SWMM Basin #5681 with some of the field data mapped is likewise submitted.

If you have any questions, please contact me at 455-5712.

Respectfully,

John J. Tomko
Consultant

John J. Tomko, P.E.
Hazardous Materials and
Water Quality Consultant

JJT MEMORANDUM

TO: Gary Reents
Ken Fischer
Craig Chalmers

FROM: John Tomko, Consulting Engineer

DATE: May 12, 1992

SUBJECT: INLET CONTROL PROJECT - SWNN BASIN # 7623

1) STEADY-STATE OUTLET HYDRAULIC CAPACITY

Several SWMM runs were conducted to determine the steady-state hydraulic capacity of the existing combined sewer collection system. Constant precipitation rates were routed through the SWMM model. The objective of this approach was to identify the greatest precipitation rate that does not cause street flooding. Since freeways will not be used for street storage, all SWMM runs were done with a 10-year storm hyteograph for the freeway basins. This analysis indicated that system-wide flooding was reduced to under 10% of the total watershed inflow at a constant precipitation rate of 0.2 inches/hour. (See Appendix 1 for a complete description and a summary of the results of this analysis.)

Based upon the above analysis it was apparent that to achieve the system-wide zero flooding objective, a very low precipitation rate (less than 0.15 inches/hour) would be necessary. However, this low rate would result in an under-utilization of a majority of the sewer mains and laterals within the system. For the 0.2 inches/hour precipitation rate, almost half of the sewer lines were at or under 80% of capacity, and almost a third were at 50% or less. (See Appendix 2.)

Therefore, for planning purposes, it was determined that the 0.2 inches/hour precipitation rate would be utilized to determine the steady-state flow capacity of the combined sewer system basins that do not experience flooding at this rate. For those basins that do experience flooding at this rate, the precipitation rate is reduced until flooding in that basin in eliminated and that reduced rate is then used to estimate that basin's steady-state outlet capacity.

For SWMM Basin 7623 the constant precipitation rate was 0.19 inches/hour. (See Tables 1, 2, and 3.) This precipitation rate resulted in a steady-state basin outlet capacity of approximately 4.5 cfs. This value represents the restricted (maximum allowable) flow rate) from SWMM Basin 7623.

2) RUNOFF STORAGE REQUIREMENTS FOR DESIGN STORMS

The total runoff storage volumes for the 5-year and 10-year design storms were calculated using a spreadsheet hydrograph curve integration technique using the design storm's runoff hydrographs from the RUNOFF block of the SWMM model and the basin's steady-state outlet flow rate of 4.5 cfs.

Figure 1 shows the flow hydrographs for the 10-year design storm and the steady-state outlet flow rate of 4.5 cfs. The volume of runoff requiring storage is the area under the 10-year storm hydrograph and above the 4.5 cfs (steady-state) flow hydrograph. This volume is represented in Figure 2 as the normalized flow (NOR. FLOW).

Table 4 contains the spreadsheet used to calculated the 10-year storage volume requirement for Basin 7623. The runoff volume requiring storage for Basin 7623 during the 10-year design storm is 46,226 cubic feet.

Figure 3 shows the flow hydrographs for the 5-year design storm and the steady-state outlet flow rate of 4.5 cfs. The volume of runoff requiring storage is the area under the 10-year storm hydrograph and above the 4.5 cfs (steady-state) flow hydrograph. This volume is represented in Figure 4 as the normalized flow (NOR. FLOW).

Table 5 contains the spreadsheet used to calculated the 5-year storage volume requirement for Basin 7623. The runoff volume requiring storage for Basin 7623 during the 5-year design storm is 23,822 cubic feet.

3) TOPOGRAPHIC SURVEY

A topographic survey of the basin was conducted by city staff. Survey data (Appendix 3) included the following data at each DI (low point):

a) ROW width.

b) Street pavement width.

c) Type of curb - roll curb or vertical curb.

d) Height of curb.

e) Road crown grades.

f) ROW grades, from top of curb to edge of pavement.

g) Distance from each DI to next upstream DI (or to gutter line high point) and the longitudinal gutter grades between these points.

4) CHARACTERIZATION OF THE DI SUBCATCHMENTS

The study area encompasses the Phase 3 boundaries of SWMM Basin 7623. The study area is delineated on a 1 inch to 100 feet scaled map (see Appendix 4). The study area is south of Folsom Blvd and west of 42nd Street.

Each DI's tributary subcatchment is delineated on the Appendix 4 map. The delineation was based upon the survey data results and the assumption that lots drain toward the streets. Each subcatchment's area was then calculated. The master spreadsheet used in this analysis is contained in Appendix 5 (a disc copy of this spreadsheet, in Lotus format, has been provided to the city).

5) DISTRIBUTION OF BASIN'S STEADY-STATE OUTFLOW RATE

Since this basin is residential, the steady-state outflow rate of 4.5 cfs (Section 2) was distributed to each DI on an area-weighted basis. The restriction flow rate for each DI (designated DI_a) was based on the following equation:

$$Q_a = Q_{Basin} * A_a \ / \ SUM \ (i=1 \ TO \ i=n) \ A_i$$

where n = the number of subcatchments within the basin.

The restricted flow for each DI in the 42nd Street basin is shown in Table 6. The restricted DI flow rates ranged from 0.03 to 0.33 cfs.

6) DISTRIBUTION OF BASINS 10-YEAR FLOODING VOLUME

The basin's 10-year flooding volume of 46,226 cubic feet (see Table 4 and Section 2) was distributed to each catchment based upon a weighted average of the subcatchment's area. The flooding volume at each subcatchment (designated A_a) is:

$$V_a = V_{Basin} * A_a \ / \ SUM \ (i=1 \ TO \ i=n) \ A_i$$

where n = the number of subcatchments within the basin.

The 10-year flooding volume for each subcatchment is shown in Table 7. The flooding volumes ranged from 550 to 7045 cubic feet.

7) CALCULATE THE MAXIMUM ALLOWABLE STREET FLOODING VOLUME AT EACH SUBCATCHMENT

The maximum allowable street storage volume in each subcatchment was estimated based upon the survey data regarding street cross-sections and longitudinal gutter grades (see master spreadsheet in Appendix 5). The maximum allowable extent of flooding is up to the street's ROW boundaries, approximately 10 feet from the top of the curb. Appendix 6 contains the equations used to estimate the maximum storage volumes for each subcatchment.

The maximum storage volumes for each subcatchment is shown in Table 7. The storage volumes ranged from 56 to 5766 cubic feet.

8) DETERMINE EXCESS RUNOFF VOLUMES

The excess 10-year runoff volumes for each subcatchment was calculated by subtracting the maximum street storage volumes from the 10-year flooding volumes. Positive excess runoff volumes signify that insufficient street storage exists at that subcatchment to store that subcatchment's entire 10-year runoff volume. Therefore, the excess volume was either routed to the next downstream subcatchment (pass-over DIs), or subsurface storage in pipes was provided at location (sump DIs).

The excess 10-year runoff volumes for each subcatchment is shown is Table 7. There were 10 locations with excess runoff volumes. Volumes ranged from 176 to 3941 cubic feet.

9) RUNOFF CONTROL IMPROVEMENTS

Based upon the above analysis, an estimate was made of the runoff control improvements necessary to contain the 10-year storm runoff within the street ROWs. These improvements are shown on Figure 5 and tabulated, by subcatchment, in Table 7. The study area for this report is the Phase 3 SWMM Basin #7623, and includes the area south of Folsom Blvd and west of 42nd Street. Within this 31.8 acre basin, 12 asphalt road berms, 34 restricted-flow drop inlets, and 6 below-ground pipe storage facilities will be required. The pipe storage facilities will be near-surface gravity-emptying 36 or 48 inch diameter pipes (see Table 8).

10) COST ESTIMATE

The cost of the inlet control improvements for this basin are based upon HDR's Phase 3 cost estimates for constructing new drop inlets and sewer pipes, and Donohue and Associates's 1984 cost of asphalt berms for the community of Skokie, Illinois. The HDR unit pipe costs were reduced to account for the shallow burial of the pipes.

The Donohue 1984 unit cost estimate for the asphalt berms of $8300 was inflated to $11,400 using a 4% annual inflation rate. The cost breakdown for the proposed improvements are shown in Table 8. The total cost, including a 20% contingency, is $642,250.

FIGURES

42ND ST. RCS

FIGURE 1

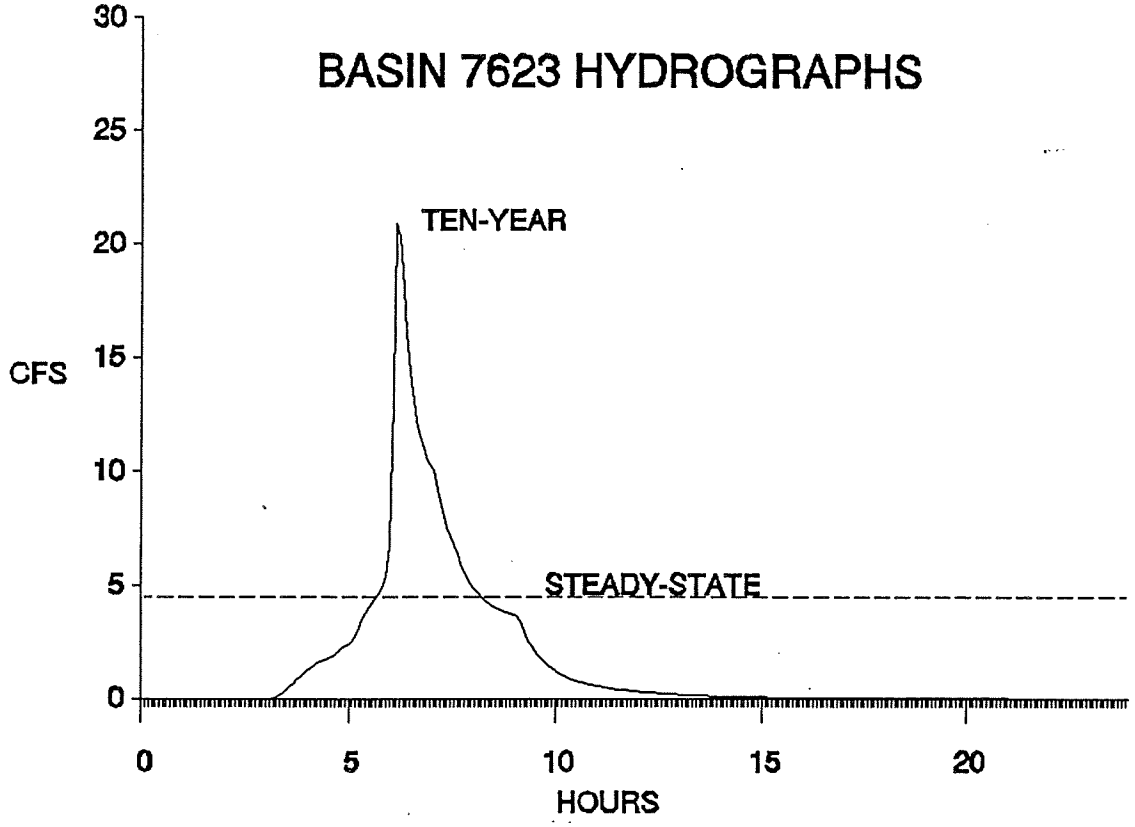

BASIN 7623 HYDROGRAPHS
TEN-YEAR
STEADY-STATE
CFS
30
25
20
15
10
5
0
0
5
10
15
20
HOURS

FIGURE 2

10-YEAR NORMALIZED HYDROGRAPH

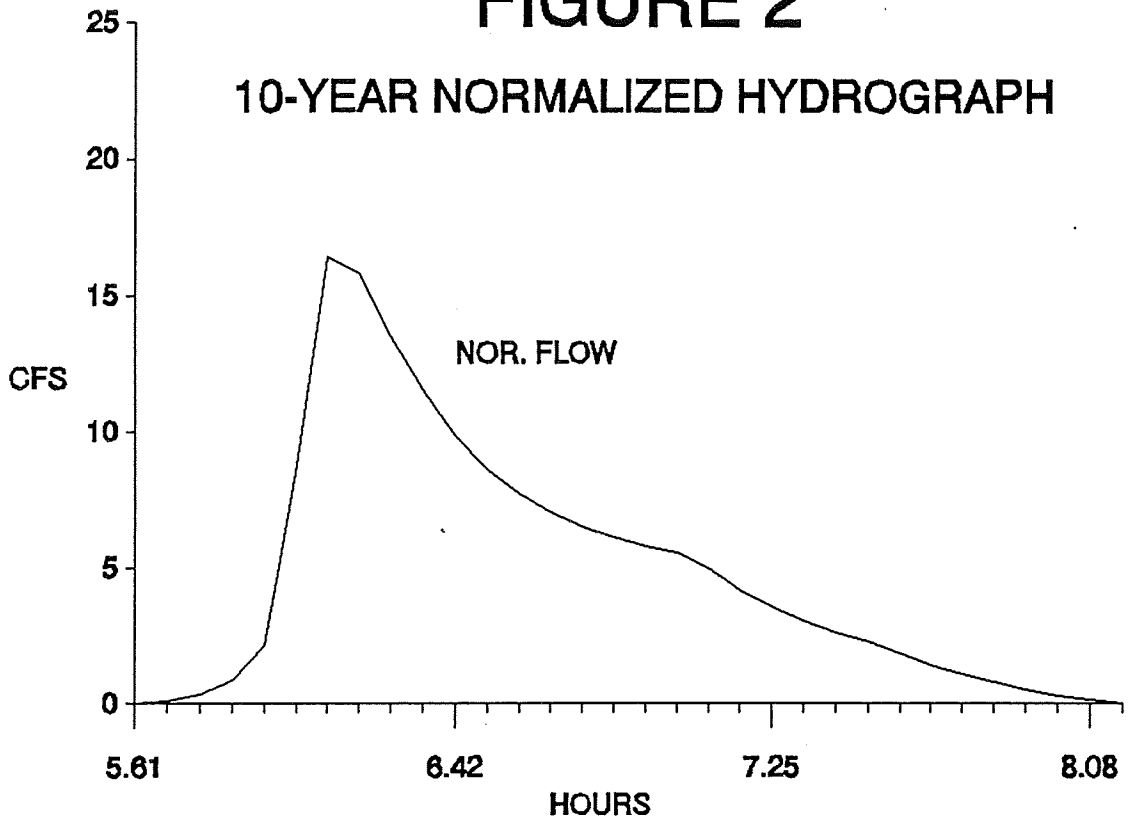


FIGURE 3

BASIN 7623 HYDROGRAPHS

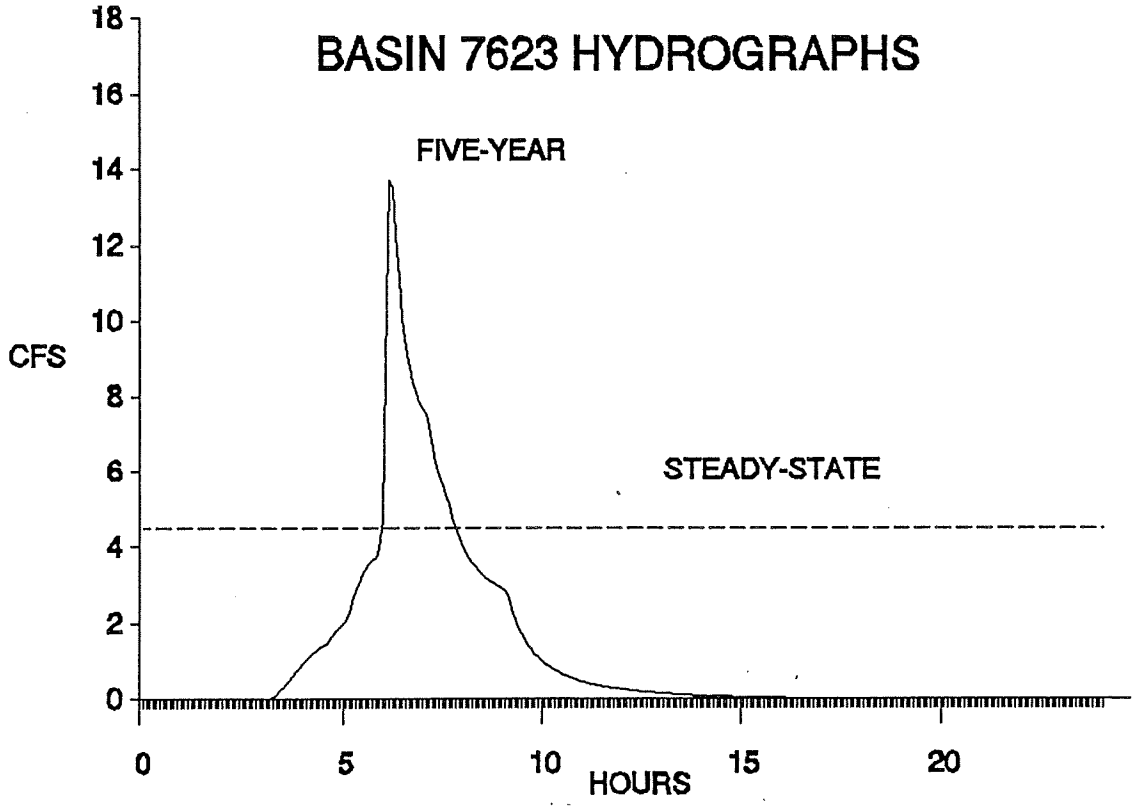
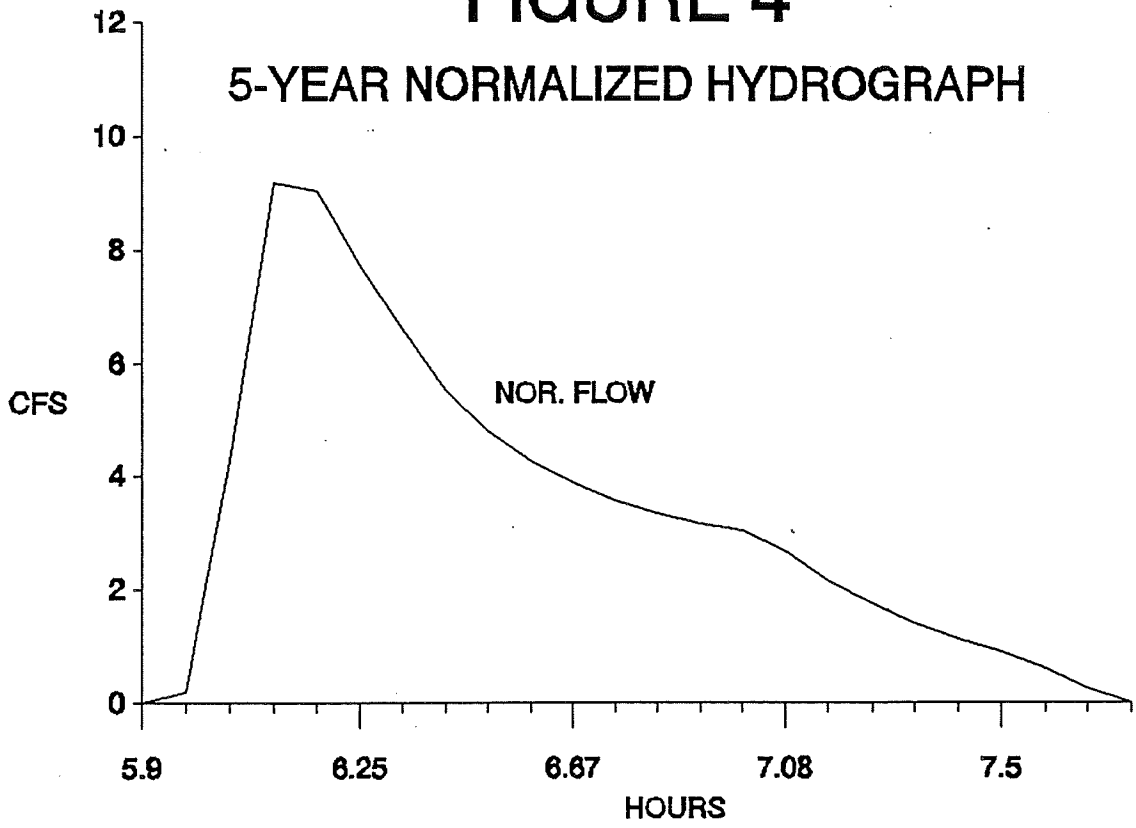
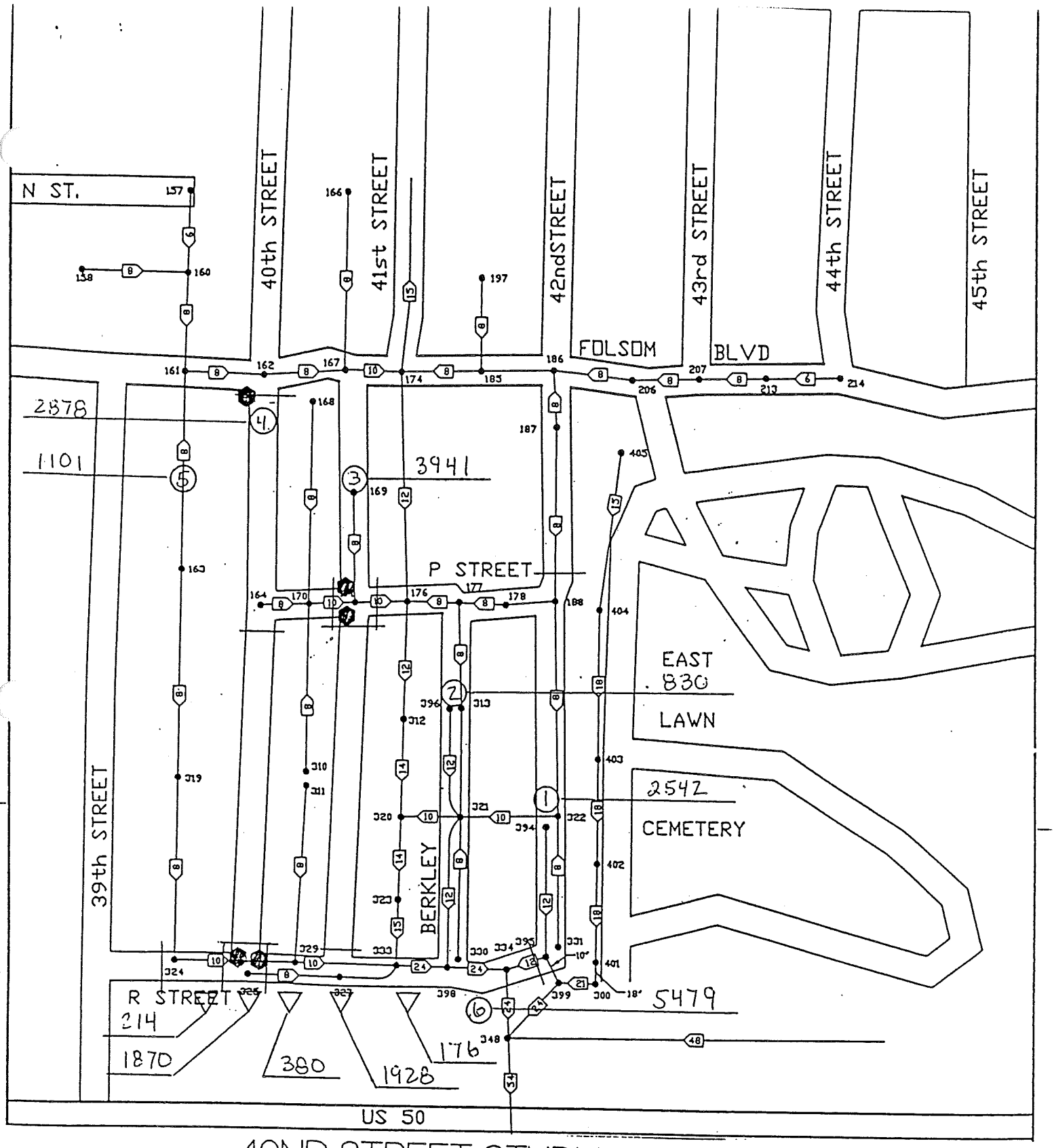


FIGURE 4

5-YEAR NORMALIZED HYDROGRAPH





42ND STREET STUDY AREA

BERM

NEW DI

830

PIPE STORAGE

TO DITCH

FIGURE 5

TABLES

42ND ST. RCS

TABLE 1

FLOODING NODES FOR 10-YEAR STORM AT FREEWAYS
AND .2 IN\HR CONSTANT RAINFALL ELSEWHERE
BAS10FR2.DAT. PIPE102.OUT [FLD10-2.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	FLOOD REENTANCE	STREET FLOODING
151	324200	5	0	134374
162	0	0	0	57668
165	359402	9	0	5329
173	254827	9	0	33480
178	137910	5	0	290140
185	0	0	42	65080
186	540392	6	0	45533
187	93982	2	0	73345
191	298570	10	0	268618
192	539696	21	24400	58775
193	642057	31	0	696730
195	283514	10	0	25871
336	407477	13	0	1154
353	202444	4	0	3117
366	266126	6	23	24
559	466943	10	0	77705
568	311386	10	0	13585
596	114792	6	91	1305
598	232333	10	1	368
740	206180	14	0	62543
744	778451	18	0	1557195
762	**427113**	**34**	**0**	**508071**
780	184697	8	0	257960
786	239449	5	0	18666
787	347736	13	0	98457
940	196699	5	4233	272218
941	467491	12	0	186362
947	197763	6	0	25612
949	250276	5	0	91537
971	253396	3	3	21423
973	295817	6	37	9406
974	444312	5	0	84675
COUNT 32				TOTAL 5,046,326

TOTAL WATERSHED INFLOW = 70,100,000
% FLOODING = 7%

TABLE 2

FLOODING NODES FOR 10-YEAR STORM AT FREEWAYS AND 0.195 IN/HR RAINFALL ELSEWHERE BAS10195.DAT, PP195.OUT [FLD1095.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	314215	5	0	119514
162	0	0	0	1756
173	245976	9	0	2052
178	132585	4	0	268245
185	0	0	37	51326
186	524818	6	0	33559
187	90595	0	0	39200
191	288071	10	0	233765
192	508711	19	28606	34789
193	613583	28	0	662892
559	453265	10	0	47717
568	304342	9	0	2893
740	198990	13	0	53468
744	753578	17	0	1100628
762	**413958**	**31**	**0**	**205812**
780	178375	6	0	195275
787	336427	15	0	70293
940	190643	5	5876	181174
941	451596	10	0	171429
947	191046	6	0	1246
949	241672	4	0	80293
971	245525	3	10	8562
973	286856	6	799	172
974	430934	5	0	39601
COUNT 24				TOTAL 3,605,661

TOTAL INFLOW = 68,000,000
% FLOODING = 5%

TABLE 3

FLOODING NODES FOR 10-YEAR STORM AT FREEWAYS AND 0.19 IN/HR RAINFALL ELSEWHERE BAS10F19.DAT, PIP10F19.OUT [FLD1019.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	304241	4	0	104359
178	127340	4	0	245322
185	0	0	25	37296
186	508958	5	0	21791
187	86995	0	0	8100
191	277622	9	0	191824
192	477957	17	34746	13544
193	585081	26	0	628482
366	250716	5	15	15
559	439484	9	0	20582
740	192005	13	0	43895
744	728634	18	0	142801
762				**0**
780	171935	6	0	130535
787	325060	14	0	42530
940	184603	5	8768	113591
941	435572	9	0	156416
949	233038	4	0	61506
971	237680	3	22	311
974	417583	4	0	4976
COUNT				TOTAL
20				1,967,876

TOTAL INFLOW = 65,800,000
% FLOODING = 3%

TABLE 4
BASIN 7623 - 10 YEAR STORAGE REQUIREMENT

TIME (HOURS)	10-YR FLOW (CFS)	S-ST. FLOW (CFS)	NOR. FLOW (CFS)	VOLUME (CUBIC FT)
5.61	4.5	4.5	0	
5.67	4.64	4.5	0.14	15.1
5.75	4.87	4.5	0.37	73.4
5.83	5.38	4.5	0.88	180.0
5.92	6.68	4.5	2.18	495.7
6	13.26	4.5	8.76	1575.4
6.08	20.93	4.5	16.43	3627.4
6.17	20.33	4.5	15.83	5226.1
6.25	18.03	4.5	13.53	4227.8
6.33	16.09	4.5	11.59	3617.3
6.42	14.35	4.5	9.85	3473.3
6.5	13.12	4.5	8.62	2659.7
6.58	12.22	4.5	7.72	2353.0
6.67	11.55	4.5	7.05	2392.7
6.75	11.03	4.5	6.53	1955.5
6.83	10.63	4.5	6.13	1823.0
6.92	10.32	4.5	5.82	1935.9
7	10.07	4.5	5.57	1640.2
7.08	9.51	4.5	5.01	1523.5
7.17	8.71	4.5	4.21	1493.6
7.25	8.07	4.5	3.57	1120.3
7.33	7.56	4.5	3.06	954.7
7.42	7.14	4.5	2.64	923.4
7.5	6.79	4.5	2.29	709.9
7.58	6.39	4.5	1.89	601.9
7.67	5.96	4.5	1.46	542.7
7.75	5.61	4.5	1.11	370.1
7.83	5.31	4.5	0.81	276.5
7.92	5.05	4.5	0.55	220.3
8	4.83	4.5	0.33	126.7
8.08	4.65	4.5	0.15	69.1
8.16	4.5	4.5	0	21.6
	130,153		TOTAL	46,226.0

TOTAL 10-YEAR RUNOFF VOLUME = 130,153 CU.FT.

VOLUME REQUIRING BASIN-WIDE STORAGE BASED ON A STEADY-STATE OUTLET FLOW OF 4.5 CFS = 46,226 CU. FT.

TABLE 5
BASIN 7623 - 5 YEAR STORAGE REQUIREMENT

TIME (HOURS)	5-YR FLOW (CFS)	S-ST. FLOW (CFS)	NOR. FLOW (CFS)	VOLUME (CUBIC FT)
5.9	4.5	4.5	0	
5.92	4.7	4.5	0.2	7.2
6	8.77	4.5	4.27	643.7
6.08	13.68	4.5	9.18	1936.8
6.17	13.54	4.5	9.04	2951.6
6.25	12.25	4.5	7.75	2417.8
6.33	11.12	4.5	6.62	2069.3
6.42	10.06	4.5	5.56	1973.2
6.5	9.32	4.5	4.82	1494.7
6.58	8.78	4.5	4.28	1310.4
6.67	8.38	4.5	3.88	1321.9
6.75	8.08	4.5	3.58	1074.2
6.83	7.85	4.5	3.35	997.9
6.92	7.67	4.5	3.17	1056.2
7	7.54	4.5	3.04	894.2
7.08	7.18	4.5	2.68	823.7
7.17	6.66	4.5	2.16	784.1
7.25	6.24	4.5	1.74	561.6
7.33	5.9	4.5	1.4	452.2
7.42	5.62	4.5	1.12	408.2
7.5	5.39	4.5	0.89	289.4
7.58	5.1	4.5	0.6	214.6
7.67	4.76	4.5	0.26	139.3
7.75	4.5	4.5	0	37.4
	98,219		TOTAL	23,822

TOTAL 5-YEAR RUNOFF VOLUME = 98,219 CU.FT.

VOLUME REQUIRING BASIN-WIDE STORAGE BASED ON A STEADY-STATE OUTLET FLOW OF 4.5 CFS = 23,822 CU.FT

APPENDIX 1

CONSTANT PRECIPITATION SWMM RUNS

42ND ST. RCS

John J. Tomko, P.E.
Hazardous Materials and
Water Quality Consultant

JJT MEMORANDUM

TO: Gary Reents
Ken Fischer
Craig Chalmers

FROM: John Tomko, Consulting Engineer

DATE: April 6, 1992

SUBJECT: SWMM RUNS TO DETERMINE THE CAPACITY OF THE EXISTING COMBINED SEWER SYSTEM, SECTION 2, TASK 1

STEP 1 - CONSTANT PRECIPITATION RATES

Several SWMM runs were conducted to determine the steady state hydraulic capacity of the existing combined sewer collection system. Since freeways will not be used for street storage, all SWMM runs were done with a 10-year storm hyteograph for the freeway basins. Initially, six SWMM runs were done with constant precipitation hyteographs varying from 0.25 to 0.15 in/hr. A summary table of the street flooding results for these six runs are included as Attachments 1 through 6.

As expected, street flooding decreased with a decrease in the constant precipitation rate. The following table summarizes the results of this analysis.

PRECIPITATION RATE (IN/HR)	# OF FLOODING SITES	FLOOD VOLUMES (FT^3*10^6)	TOTAL WATERSHED INFLOW (FT^3*10^6)	% FLOODING
0.25	87	75.1	92.3	81
0.21	58	14.2	74.4	19
0.20	32	5.0	70.1	7
0.19	19	2.0	65.8	3
0.18	13	1.1	61.5	2
0.15	4	0.3	48.9	0.7

As previously discussed with City staff, the precipitation rate that resulted in a percent flooding value of 10% was then used to conduct a more in-depth basin-by-basin analysis of street flooding. Therefore, the 0.20 in/hr precipitation rate was selected (See Attachment 8).

STEP 2 - IN-DEPTH ANALYSIS OF STREET FLOODING

The 32 flooding locations (SWMM nodes) assoiciated with the 0.20 in/hr precipitation rate were identified (See Attachment 3). In addition, the upstream SWMM basins that contribute flow to these nodes, during non-surcharged flow conditions, were also identified (See Attachments 7 and 8).

The in-depth analysis of the street flooding consisted of a series of SWMM runs with decreasing precipitation rates at those contributory (upstream) basins to the flood-prone nodes. Attachments 9 through 14 show the hyteograph distribution for each of the SWMM runs and the resulting decrease in flooding characteristics. Following is a detailed discussion on these SWMM runs.

SWMM RUN # 1 - ATTACHMENT 3

All basins at precipitation rate = 0.20 in/hr.

Flooding volume = 5.0 million cubic feet.

Number of flooding junctions = 32.

Total watershed inflow = 70.1 million cubic feet.

Percent flooding = 7.0 %.

SWMM RUN # 2 - ATTACHMENT 9

Basins in Attachment 8 (except node 947) at precipitation rate = 0.195 in/hr.

All other basins at precipitation rate = 0.20 in/hr.

Flooding volume = 4.4 million cubic feet.

Number of flooding junctions = 26.

Total watershed inflow = 69.6 million cubic feet.

Percent flooding = 6.0 %.

SWMM RUN # 3 - ATTACHMENT 10

Basins 1651, 3361, 3531, 3661, 3991, 5961, and 5981 at precipitation rate = 0.195 in/hr.

Remaining basins in Attachment 8 at precipitation rate = 0.185

in/hr.

All other basins at precipitation rate = 0.20 in/hr.

Flooding volume = 3.4 million cubic feet.

Number of flooding junctions = 21.

Total watershed inflow = 68.8 million cubic feet.

Percent flooding = 5.0 %.

SWMM RUN # 4 - ATTACHMENT 11

Basins 1651, 3361, 3531, 3661, 3991, 5961, and 5981 at precipitation rate = 0.195 in/hr.

Basins 1631, 1501, 1502, 1591, 1521, 1551, 1531, 1731, 1741, 1951, 5681, 7861, and 9731 at precipitation rate = 0.185 in/hr.

Remaining basins in Attachment 8 at precipitation rate = 0.170 in/hr.

All other basins at precipitation rate = 0.20 in/hr.

Flooding volume = 1.9 million cubic feet.

Number of flooding junctions = 17.

Total watershed inflow = 67.9 million cubic feet.

Percent flooding = 3.0 %.

SWMM RUN # 5 - ATTACHMENT 12

Basins 1651, 3361, 3531, 3661, 3991, 5961, and 5981 at precipitation rate = 0.195 in/hr.

Basins 1631, 1501, 1502, 1591, 1521, 1551, 1531, 1731, 1741, 1951, 5681, 7861, and 9731 at precipitation rate = 0.185 in/hr.

Basins 9711 and 9741 at precipitation rate = 0.170 in/hr.

Remaining basins in Attachment 8 at precipitation rate = 0.160 in/hr.

All other basins at precipitation rate = 0.20 in/hr.

Flooding volume = 1.1 million cubic feet.

Number of flooding junctions = 12.

Total watershed inflow = 67.4 million cubic feet.

Percent flooding = 1.4 %.

SWMM RUN # 6 - ATTACHMENT 13

Basins 1651, 3361, 3531, 3661, 3991, 5961, and 5981 at precipitation rate = 0.195 in/hr.

Basins 1631, 1501, 1502, 1591, 1521, 1551, 1531, 1731, 1741, 1951, 5681, 7861, and 9731 at precipitation rate = 0.185 in/hr.

Basins 9711 and 9741 at precipitation rate = 0.170 in/hr.

Remaining basins in Attachment 8 at precipitation rate = 0.160 in/hr.

All other basins at precipitation rate = 0.196 in/hr.

Flooding volume = 0.9 million cubic feet.

Number of flooding junctions = 11.

Total watershed inflow = 66.0 million cubic feet.

Percent flooding = 1.3 %.

SWMM RUN # 7 - ATTACHMENT 14

Basins 1651, 3361, 3531, 3661, 3991, 5961, and 5981 at precipitation rate = 0.195 in/hr.

Basins 1631, 1501, 1502, 1591, 1521, 1551, 1531, 1731, 1741, 1951, 5681, 7861, and 9731 at precipitation rate = 0.185 in/hr.

Basins 9711 and 9741 at precipitation rate = 0.170 in/hr.

Remaining basins in Attachment 8 at precipitation rate = 0.160 in/hr.

All other basins at precpitation rate = 0.196 in/hr, except basins 1931 and 1932 precipitation rate = 0 in/hr.

Flooding volume = 0.5 million cubic feet.

Number of flooding junctions = 11.

Total watershed inflow = 65.6 million cubic feet.

Percent flooding = 0.9 %.

ATTACHMENT 1

FLOODING NODES FOR 10-YEAR STORM AT FREEWAYS
AND 0.25 IN\HR CONSTANT RAINFALL ELSEWHERE
BAS10F25.DAT, PIP10F25.OUT [FLD10F25.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
130	449079	34	585	100397
151	424300	11	0	278729
152	405476	6	21	686376
159	374514	10	2	85375
160	579950	15	19	1011210
162	0	0	0	395868
165	492128	15	0	579191
166	417433	10	0	241825
170	0	0	1014	674579
172	455884	18	1	91224
173	342476	13	0	162169
174	226299	6	3	3
177	289931	8	10786	62906
178	191116	10	0	485259
182	236336	10	24	17534
185	0	0	1644	358090
186	698855	9	0	184776
187	128866	3	0	611162
191	404512	14	0	455066
192	854683	27	13347	506055
193	930837	41	0	1022076
194	111529	4	350	146772
195	399247	15	0	126972
301	107074	6	0	3444694
309	185406	2	0	116267
323	616485	15	10	393972
332	306409	15	21806	48283
333	422969	5	0	345668
336	513447	20	0	256620
337	197886	6	0	32252
342	223842	4	91371	324510
343	601016	10	217	531292
349	575515	10	10	1572
353	257718	8	0	150455
354	36719	4	0	374282
357	59000	5	148	19200
359	0	0	0	166559
366	343703	8	1053	47918
399	644845	18	0	158488
511	328115	3	23	13168940
548	762553	14	0	39093
550	353224	10	0	604930
557	223461	10	6860	66137
558	171860	6	191	637903

559	605747	17	0	422471
568	431061	14	0	252566
569	341038	9	92336	26195
586	173888	5	0	61537
587	112892	5	4739	5931
589	0	0	27	294118
593	563966	10	2	1316141
596	158907	9	0	102983
598	298430	15	0	180896
710	159850	4	198	20870
719	320581	12	7868	8770437
722	353605	6	0	15820110
738	595781	13	193936	310081
739	0	0	0	3953725
740	278049	18	0	134345
744	1029621	27	4	3309822
750	0	0	1	1
762	591498	43	2	2007338
770	18719	0	2	2
779	293303	8	0	109393
780	250159	9	0	805712
781	496994	11	0	89381
785	356926	6	8112	39643
786	314755	10	0	180928
787	462041	20	0	290606
798	0	0	0	514192
799	382623	5	52622	16
913	135134	0	1029	1029
931	20160	0	161	16763
937	596161	15	0	308016
938	591849	12	332	632
940	257986	5	7527	551539
941	628280	18	0	346857
947	266913	9	0	342251
948	174306	8	11260	43347
949	337660	5	0	187225
950	486521	17	0	692141
955	235879	12	0	1219647
956	0	0	9	1952874
970	594094	19	83928	697
971	332322	5	40	153932
973	387031	8	0	425937
974	579646	11	0	658262
COUNT				TOTAL
87				75,133,238

TOTAL WATERSHED INFLOW = 92,300,000
% FLOODING = 81%

ATTACHMENT 2

FLOODING NODES FOR 10-YEAR STORM AT FREEWAYS AND 0.21 IN\HR CONSTANT RAINFALL ELSEWHERE BAS10F21.DAT, PIP10F21.OUT [FLD10F21.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	344192	6	0	173977
159	295097	9	0	776
160	452913	9	1	302049
162	0	0	0	241309
165	385767	12	0	372523
166	330098	5	88	85624
172	373312	13	1614	14435
173	272223	10	1	60902
178	148464	6	0	331135
185	0	0	230	112325
186	572023	8	0	70861
187	100951	2	0	316364
191	319561	8	0	320698
192	601910	20	19090	139678
193	699263	32	0	763247
194	79809	5	3552	8533
195	306436	12	2	47137
301	80300	4	0	121033
309	156841	0	0	71921
333	327616	5	0	184697
336	428811	14	0	78451
337	165379	4	150	150
353	213405	4	0	35611
354	15361	2	0	110009
359	0	0	8	6346
361	572024	32	16	15
365	173830	5	1	1
366	281621	8	106	108
550	280630	8	221	169664
558	142609	4	497	349376
559	494584	13	0	207144
568	327617	11	0	78855
586	134217	3	0	3793
593	454970	6	71	74
596	121272	8	0	12928
598	245414	8	0	90305
722	287642	8	836	2882852
739	0	0	0	378901
740	220389	13	0	78415
744	828375	20	0	2160837
762	453015	35	0	883235
780	197779	5	0	528843
781	393775	6	0	10751
786	254619	6	0	98032

787	370540	14	0	142980
798	0	0	0	94651
799	313103	6	38038	11
937	475905	9	2	454
940	208858	6	3534	428288
941	499479	13	0	216633
947	211593	8	0	219639
949	267696	5	0	109889
950	381665	12	0	366683
955	173802	10	62	1141
956	0	0	940	287857
971	269167	4	21	57778
973	313918	8	0	98600
974	471317	6	0	274675
COUNT				TOTAL
58				14,203,199

TOTAL WATERSHED INFLOW = 74,400,000
% FLOODING = 19%

ATTACHMENT 3

FLOODING NODES FOR 10-YEAR STORM AT FREEWAYS AND .2 IN\HR CONSTANT RAINFALL ELSEWHERE BAS10FR2.DAT. PIPE102.OUT [FLD102.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	FLOOD REENTANCE	STREET FLOODING
151	324200	5	0	134374
162	0	0	0	57668
165	359402	9	0	5329
173	254827	9	0	33480
178	137910	5	0	290140
185	0	0	42	65080
186	540392	6	0	45533
187	93982	2	0	73345
191	298570	10	0	268618
192	539696	21	24400	58775
193	642057	31	0	696730
195	283514	10	0	25871
336	407477	13	0	1154
353	202444	4	0	3117
366	266126	6	23	24
559	466943	10	0	77705
568	311386	10	0	13585
596	114792	6	91	1305
598	232333	10	1	368
740	206180	14	0	62543
744	778451	18	0	1557195
762	427113	34	0	508071
780	184697	8	0	257960
786	239449	5	0	18666
787	347736	13	0	98457
940	196699	5	4233	272218
941	467491	12	0	186362
947	197763	6	0	25612
949	250276	5	0	91537
971	253396	3	3	21423
973	295817	6	37	9406
974	444312	5	0	84675
COUNT				TOTAL
32				5,046,326

TOTAL WATERSHED INFLOW = 70,100,000
% FLOODING = 7%

ATTACHMENT 3-1

ATTACHMENT 3 CONDITIONS WITH THREE EAST SACRAMENTO INTERTIE SEWERS TO DOWNTOWN DISCONNECTED WITH FREE OUTFALL CONDITIONS
BAS10FR2.DAT, PIP10.DAT, PIP1.OUT, FLD3-1.PLN

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	FLOOD REENTANCE	STREET FLOODING
151	324200	5	0	123938
173	254827	9	0	31451
178	137910	5	0	288954
185	0	0	74	48545
186	540392	6	0	45506
187	93982	2	0	17659
191	298570	10	0	233670
192	539696	21	25948	51289
193	642057	31	0	696747
195	283514	10	0	16835
353	202444	4	0	3117
366	266126	6	8	8
568	311386	10	0	11656
740	206180	14	0	59810
744	778451	18	0	1203543
762	427113	34	0	281686
780	184697	8	0	184850
786	239449	5	0	7748
787	347736	13	0	90130
940	196699	5	6795	152597
941	467491	12	0	186365
947	197763	6	0	643
949	250276	5	0	83503
971	253396	3	19	17722
973	295817	6	287	13
974	444312	5	0	29320
26				3,867,305

TOTAL WATERSHED INFLOW = 70,100,000
% FLOODING = 5.5%

ATTACHMENT 4

FLOODING NODES FOR 10-YEAR STORM AT FREEWAYS AND 0.19 IN/HR RAINFALL ELSEWHERE BAS10F19.DAT, PIP10F19.OUT [FLD10F19.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	304241	4	0	104359
178	127340	4	0	245322
185	0	0	25	37296
186	508958	5	0	21791
187	86995	0	0	8100
191	277622	9	0	191824
192	477957	17	34746	13544
193	585081	26	0	628482
366	250716	5	15	15
559	439484	9	0	20582
740	192005	13	0	43895
744	728634	18	0	142801
780	171935	6	0	130535
787	325060	14	0	42530
940	184603	5	8768	113591
941	435572	9	0	156416
949	233038	4	0	61506
971	237680	3	22	311
974	417583	4	0	4976
COUNT				TOTAL
19				1,967,876

TOTAL INFLOW = 65,800,000
% FLOODING = 3%

ATTACHMENT 5

FLOODING NODES FOR 10 YEAR STORM AT FREEWAYS AND 0.18 IN/HR ELSEWHERE.
BAS10F18.DAT, PIP10F18.OUT [FLD10F18.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	284542	8	0	76795
178	116606	6	0	191464
185	0	0	0	13543
186	477464	4	0	1143
191	256741	8	0	102781
193	528584	24	0	557873
366	235354	5	21	21
740	177668	10	0	18400
780	158973	5	0	9880
940	172484	4	15118	20417
941	403903	11	0	126537
949	215821	4	0	23586
971	222054	2	168	182
COUNT				TOTAL
13				1,142,622

TOTAL INFLOW = 61,500,000
% FLOOD = 2%

ATTACHMENT 6

FLOODING NODES FOR 10-YEAR STORM AT FREEWAYS AND 0.15 IN/HR ELSEWHERE.
BAS10FR1.DAT, PIPE10.OUT [FLD10.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	225378	4	0	10673
191	194633	8	0	12027
193	361593	19	0	312278
941	309630	6	0	3009
COUNT				TOTAL
4				337,987

TOTAL INFLOW = 48,900,000
% FLOOD = 0.7%

ATTACHMENT 7

UPSTREAM BASINS FOR JUNCTIONS IDENTIFIED
IN ATTACHMENT 3

NODE	BASINS
151	1511
162	1631
	1501
	1502
	1591
	1521
	1511
	1551
	1531
165	1651
173	1731
	1741
178	1781
	1791
185	1861
186	1861
187	1871
	1881
	1891
191	1911
192	1921
	1931
	1932
193	1931
	1932
195	1951
336	3361
353	3531
366	3661
	3991
559	5591
568	5681
596	5961
598	5961
	5981
740	7401
744	7622
	7623
	7601
	7492
	7491
	7521
	7481
	7482
	7461
	7451
	7441
	7442

NODE	BASINS
762	7622
	7623
780	7801
	7811
786	7861
787	7871
940	9401
	9411
941	9411
949	9491
971	9711
973	9731
974	9741

NO UPSTREAM BASINS
FOR #947-BASINS
LISTED ARE POTENTIAL
BACKFLOW BASINS

947	9452
	9501
	9361
	9282
	9291
	9371
	9451
	9251

ATTACHMENT 8

ATTACHMENT 7's 53 UPSTREAM BASINS IN NUMERICAL ORDER AND NODE 947's POTENTIAL BACKFLOW BASINS

BASIN	BASIN	BASIN	BASIN
1501	1891	7442	9411
1502	1911	7451	9491
1511	1921	7461	9711
1521	1931	7481	9731
1531	1932	7482	9741
1551	1951	7491	
1591	3361	7492	NODE 947
1631	3531	7521	9251
1651	3661	7601	9282
1731	3991	7622	9291
1741	5591	7623	9361
1781	5681	7801	9371
1791	5961	7811	9451
1861	5981	7861	9452
1871	7401	7871	9501
1881	7441	9401	

ATTACHMENT 9

10-YEAR STORM AT FREEWAYS
.195 IN/HR AT 53 BASINS (ATTACHMENT 8) AND
.2 IN/HR AT REMAINING BASINS.
RED195.DAT, PIPE195.OUT [FLD195.PLN]

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JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	314215	5	0	122582
162	0	0	0	35298
173	245976	9	0	16151
178	132585	4	0	271924
185	0	0	1	63294
186	524818	6	0	33646
187	90595	0	0	59956
191	288071	10	0	244255
192	527664	19	24001	49122
193	613583	28	0	665797
195	272006	9	0	8056
559	453265	10	0	63643
568	304342	9	0	5893
740	198990	13	0	56171
744	753578	17	0	1424953
762	413958	31	0	364862
780	178375	6	0	233676
786	232068	5	0	8774
787	341670	12	0	89628
940	190643	5	3977	237232
941	451596	10	0	171777
947	197763	6	0	23087
949	241672	4	0	83148
971	245525	3	0	14268
973	286856	6	159	397
974	430934	5	0	58161
COUNT				TOTAL
26				4,405,751

TOTAL WATERSHED INFLOW = 69,600,000
% FLOODING = 6%

ATTACHMENT 10

10-YEAR STORM AT FREEWAYS
.195 IN/HR AT 7 BASINS (HYTE #4)
.185 IN/HR AT OTHER ATTACHMENT 8 BASINS (HYTE #3)
.2 IN/HR AT REMAINING BASINS
RED185.DAT, PIPE185.OUT [FLD185.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	294310	8	0	100642
178	121978	8	0	234973
185	0	0	0	59637
186	493306	5	0	10991
187	83642	0	0	36836
191	267161	8	0	200936
192	503880	19	24099	30842
193	556863	27	0	604307
366	258534	6	1	1
559	425702	8	0	38958
740	184843	12	0	44313
744	703666	17	0	1160532
762	388082	29	0	217289
780	165453	6	0	191275
787	329528	15	0	79366
940	178540	4	4179	185564
941	419745	9	0	143411
947	197763	6	0	19540
949	224358	4	0	65249
971	229885	3	7	1913
974	404232	8	0	11757
COUNT				TOTAL
21				3,438,314

TOTAL WATERSHED INFLOW = 68,800,000
% FLOODING = 5%

ATTACHMENT 11

10-YEAR STORM AT FREEWAYS
.195 AT 7 BASINS (#4)
.185 AT 13 BASINS (#3)
.170 AT OTHER ATTACHMENT 8 BASINS (#5)
.2 AT OTHER REMAINING BASINS
RED170.DAT, PIPE170.OUT [FLD170.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	264762	6	0	74639
178	106156	5	0	178036
185	0	0	0	34344
187	73446	4	0	7054
191	235938	6	0	126169
192	468859	18	27991	8285
193	472377	24	0	513392
559	384777	9	0	8589
740	163589	9	0	27160
744	629491	13	0	477556
762	349291	25	0	6302
780	146083	5	0	124155
787	311317	14	0	58080
940	160399	4	3595	111518
941	372373	10	0	101880
947	197763	6	0	14405
949	198666	3	0	36953
COUNT				TOTAL
17				1,908,517

TOTAL INFLOW = 67,900,000
% FLOODING = 3%

ATTACHMENT 12

10-YEAR STORM AT FREEWAYS (#2)
.195 AT 7 BASINS (#4)
.185 AT 13 BASINS (#3)
.170 AT 2 BASINS (#6)
.160 AT OTHER ATTACHMENT 8 BASINS (#5)
.2 AT OTHER REMAINING BASINS (#1)
RED160.DAT, PIPE160.OUT [FLD160.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	245028	5	0	57886
178	95740	5	0	135208
185	0	0	0	13071
191	215182	5	0	99770
193	416730	21	0	454359
740	149367	10	0	15988
780	133211	4	0	86124
787	299424	13	0	42519
940	148156	3	2941	80734
941	340855	8	0	75513
947	197763	6	0	11336
949	181673	3	0	17200
COUNT				TOTAL
12				1,089,708

TOTAL INFLOW = 67,400,000
% FLOODING = 1.4%

ATTACHMENT 13

10-YEAR STORM AT FREEWAYS (#2)
.195 AT 7 BASINS (#4)
.185 AT 13 BASINS (#3)
.170 AT 2 BASINS (#6)
.160 AT OTHER ATTACHMENT 8 BASINS (#5)
.196 AT OTHER REMAINING BASINS (#1)
RED196.DAT, PIPE196.OUT [FLD196.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	245028	5	0	54956
178	95740	5	0	127042
185	0	0	0	4459
191	215182	5	0	65702
193	416730	21	0	449627
740	149367	10	0	11372
780	133211	4	0	47886
787	295079	13	0	22069
940	148156	3	6171	51138
941	340855	8	0	74371
949	181673	3	0	10459
COUNT				TOTAL
11				919,081

TOTAL INFLOW = 66,000,000
% FLOODING = 1.3%

ATTACHMENT 14

10-YEAR STORM AT FREEWAYS (#2)
.195 AT 7 BASINS (#4)
.185 AT 13 BASINS (#3)
.170 AT 2 BASINS (#6)
.160 AT OTHER ATTACHMENT 8 BASINS (#5)
.196 AT OTHER REMAINING BASINS (#1)
0 IN/HR AT BASINS 1931 AND 1932 (#7)
REDO.DAT, PIPEO.OUT [FLDO.PLN]

JUNCTION	WATERSHED INFLOW	PIPE OUTFLOW	STREET REENTANCE	STREET FLOODING
151	245028	5	0	54649
178	95740	5	0	119306
185	0	0	0	3386
191	215182	5	0	75285
193	23761	0	0	99401
740	149367	10	0	11125
780	133211	4	0	44555
787	295079	13	0	22519
940	148156	3	6222	49840
941	340855	8	0	74416
949	181673	3	0	9941
COUNT				TOTAL
11				564,423

TOTAL INFLOW = 65,600,000
% FLOODING = .9%

APPENDIX 2

Q/QMAX CONDUIT VALUES AT 0.2 IN/HR

42ND ST. RCS

APPENDIX 2 - Q/QMAX VALUES AT 0.2 "/ HR

50% OR LESS		80% OR LESS	
CONDUIT	Q/QMAX	CONDUIT	Q/QMAX
855	0.1	856	0.1
1225	0.1	855	0.1
1224	0.2	1225	0.1
639	0.2	1224	0.2
270	0.2	639	0.2
869	0.2	270	0.2
1222	0.2	869	0.2
868	0.2	1222	0.2
852	0.2	868	0.2
853	0.2	852	0.2
246	0.2	853	0.2
637	0.2	246	0.2
831	0.2	637	0.2
1034	0.2	831	0.2
849	0.2	1034	0.2
828	0.2	849	0.2
874	0.2	828	0.2
232	0.2	874	0.2
825	0.2	232	0.2
691	0.2	825	0.2
236	0.2	691	0.2
1032	0.2	236	0.2
88526	0.2	1032	0.2
829	0.2	88526	0.2
826	0.2	829	0.2
88188	0.3	826	0.2
875	0.3	88188	0.3
460	0.3	875	0.3
235	0.3	460	0.3
274	0.3	235	0.3
88143	0.3	274	0.3
862	0.3	88143	0.3
88522	0.3	862	0.3
480	0.3	88522	0.3
632	0.3	480	0.3
1033	0.3	632	0.3
1035	0.3	1033	0.3
854	0.3	1035	0.3
1044	0.3	854	0.3
1213	0.3	1044	0.3
835	0.3	1213	0.3
676	0.3	835	0.3
88428	0.4	676	0.3
638	0.4	88428	0.4
678	0.4	638	0.4
677	0.4	678	0.4
88351	0.4	677	0.4
423	0.4	88351	0.4
240	0.4	423	0.4
1069	0.4	240	0.4
681	0.4	1069	0.4

APPENDIX 2 - Q/QMAX VALUES AT 0.2 "/ HR

50% OR LESS	
CONDUIT	Q/QMAX
629	0.4
1042	0.4
898	0.4
848	0.4
1022	0.4
636	0.4
220	0.4
88908	0.4
607	0.4
692	0.4
1228	0.4
281	0.4
1025	0.5
899	0.5
845	0.5
444	0.5
684	0.5
429	0.5
1012	0.5
846	0.5
1019	0.5
204	0.5
882	0.5
1018	0.5
289	0.5
834	0.5
1013	0.5
248	0.5
245	0.5
457	0.5
244	0.5
COUNT	**133**
%	**32.5**

80% OR LESS	
CONDUIT	Q/QMAX
681	0.4
629	0.4
1042	0.4
898	0.4
848	0.4
1022	0.4
636	0.4
220	0.4
88908	0.4
607	0.4
692	0.4
1228	0.4
281	0.4
1025	0.5
899	0.5
845	0.5
444	0.5
684	0.5
429	0.5
1012	0.5
846	0.5
1019	0.5
204	0.5
882	0.5
1018	0.5
289	0.5
834	0.5
1013	0.5
248	0.5
245	0.5
457	0.5
244	0.5
243	0.6
280	0.6
450	0.6
221	0.6
1010	0.6
445	0.6
1226	0.6
699	0.6
421	0.6
693	0.6
689	0.6
430	0.6
254	0.6
675	0.6
667	0.6
206	0.6
233	0.6
1016	0.6
897	0.6

APPENDIX 2 - Q/QMAX VALUES AT 0.2 "/ HR

50% OR LESS		80% OR LESS	
CONDUIT	Q/QMAX	CONDUIT	Q/QMAX
		1009	0.6
		669	0.6
		896	0.6
		407	0.6
		1037	0.6
		858	0.6
		255	0.6
		690	0.6
		844	0.6
		282	0.6
		695	0.7
		434	0.7
		433	0.7
		688	0.7
		685	0.7
		673	0.7
		88739	0.7
		649	0.7
		627	0.7
		402	0.7
		657	0.7
		465	0.7
		462	0.7
		833	0.7
		443	0.8
		442	0.8
		697	0.8
		696	0.8
		435	0.8
		694	0.8
		205	0.8
		672	0.8
		671	0.8
		886	0.8
		616	0.8
		1026	0.8
		628	0.8
		1015	0.8
		202	0.8
		209	0.8
		839	0.8
		838	0.8
		COUNT	**195**
		%	**47.7**

APPENDIX 5

MASTER SPREADSHEET

42ND ST. RCS

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	AREA	DI #	ST. LTH.	ST. S	X-SEC.	A
1 & 2	1	0.91	88E	296	0.31	8R	23.18
	2	0.68	88W	292	0.29	8L	23.18
28 & 29	28	1.28	94N	502	0.43	7L	21.3
	29	1.39	22S	512	0.39	7R	21.3
30 & 31	30	0.91	22S	367	0.29	7R	21.3
	31	1.06	94N	376	0.28	7L	21.3
45 & 46	45	0.28	95N	223	0.88	1R	22.55
	46	0.19	95E	38	0.47	7R	21.3
3 & 27	3	0.21	78N	105	0.96	9L	22.5
	27	0.15	78S	105	0.112	9R	22.5
4 & 26	4	0.27	78N	101	0.52	9L	22.5
	26	0.11	78S	88	0.78	9R	22.5
22 & 25	22	0.38	96N	248	0.46	10L	22.02
	25	0.42	96E	245	0.18	10R	22.02
23 & 24	23	1.88	96N	650	0.4	10L	22.02
	24	1.88	96E	650	0.4	10R	22.02
43 & 44	43	0.18	33W	106	0.28	2L	24.18
	44	0.07	28W	117	0.21	2R	24.18
40′ & 42	40′	0.06	28S	111	0.36	2R	
	42	0.08	33W	96	0.22	2L	24.18
5 & 21	5	0.38	76N	132	0.5	11L	22.45
	21	0.12	76S	91	0.7	11R	22.45
6 & 20	6	0.13	76N	99	0.73	11L	22.45
	20	0.12	76S	91	0.91	11R	22.45
8 & 9	8	0.59	69E	226	0.31	13L	22.33
	9	0.6	69W	243	0.27	13R	22.33
7 & 10	7	0.42	69E	175	0.46	13L	22.33
	10	0.73	69W	165	0.44	13R	22.33
17′ & 17	17′	0.24	71NW	190	0.97	11L	22.45
	17	0.24	71SW	177	1.09	11R	22.45
18 & 19	18	1.16	71W	462	0.78	12R	23.28
	19	1.13	71E	470	0.79	12L	23.28

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	AREA	DI #	ST. LTH.	ST. S	X-SEC.	A
32 & 33	32	1.07	27N	425	0.56	3R	24.35
	33	1.14	27NW	425	0.59	3L	24.35
40 & 41	40	0.13	25S	231	0.34	4R	23.78
	41	0.25	25N	187	0.124	4L	23.78
11 & 12	11	1.18	62S	508	0.4	25L	6.79
	12	1.21	62W	467	0.31	25R	6.79
15 & 16	15	1.63	64W	331	0.29	14L	25.09
	16	0.75	64E	336	0.3	14R	25.09
34 & 35	34	1.44	25N	557	0.34	5R	23.89
	35	1.93	25NW	578	0.36	5L	23.89
37 & 38	37	0.31	25NW	105	1.8	4L	23.78
	38	0.35	25S	317	2.18	4R	23.78
36 & 48	36	1	24W	162	0.3	6R	25.58
	48	0.17	NEW	160	2.18	6	
13 & 14	13	0.26	63N	111	0.95	ALLEY	
	14	0.54	63N	143	0.57	ALLEY	
39 & 47	39	0.52	25S	700	0.6	DITCH	
	47	0.11		110	0.6	DITCH	

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	B	B′	C	D	E	F
1 & 2	1	22.92	22.74	23.16	23.84	21.17	10
	2	23.01	22.79	23.25	23.86	21.17	10
28 & 29	28	20.99	20.69	21.12	21.65	21.42	10
	29	20.97	20.71	21.18	21.6	21.42	10
30 & 31	30	20.97	20.71	21.18	21.6	21.42	10
	31	20.99	20.69	21.12	21.65	21.42	10
45 & 46	45	22.39	22.07	22.5	23.46	15.25	10
	46	20.97	20.71	21.18	21.6	21.42	10
3 & 27	3	22.33	22.14	22.57	23.07	21.17	10
	27	22.23	21.93	22.33	23.13	21.17	10
4 & 26	4	22.33	22.14	22.57	23.07	21.17	10
	26	22.23	21.93	22.33	23.13	21.17	10
22 & 25	22	21.71	21.39	21.89	22.56	21.08	10
	25	21.79	21.43	21.96	23.01	21.08	10
23 & 24	23	21.71	21.39	21.89	22.56	21.08	10
	24	21.79	21.43	21.96	23.01	21.08	10
43 & 44	43	24.08	23.89	24.27	24.74	15.5	10
	44	23.9	23.64	24.05	SHRUB	15.5	SHRUB
40′ & 42	40′						
	42	24.08	23.89	24.27	24.74	15.5	10
5 & 21	5	22.21	21.9	22.43	22.72	21.17	10
	21	22.22	21.74	22.24	22.6	21.17	10
6 & 20	6	22.21	21.9	22.43	22.72	21.17	10
	20	22.22	21.74	22.24	22.6	21.17	10
8 & 9	8	22	21.76	22.19	22.7	21.17	10
	9	22.08	21.82	22.28	22.77	21.17	10
7 & 10	7	22	21.76	22.19	22.7	21.17	10
	10	22.08	21.82	22.28	22.77	21.17	10
17′ & 17	17′	22.21	21.9	22.43	22.72	21.17	10
	17	22.22	21.74	22.24	22.6	21.17	10
18 & 19	18	22.91	22.68	23.09	24.65	21.08	10
	19	22.87	22.7	23.07	23.54	21.08	10

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	B	B′	C	D	E	F
32 & 33	32	24.3	24	24.39	24.86	21	10
	33	24.1	23.82	24.28	25.11	21	10
40 & 41	40	23.55	23.29	23.6	SHRUB	15.17	SHRUB
	41	23.58	23.37	23.78	24.76	15.17	10
11 & 12	11	6.56	6.32	6.72	7.02	21	10
	12	6.54	6.33	6.74	7.84	21	10
15 & 16	15	24.76	24.55	24.95	26.04	21	10
	16	24.86	24.64	25.07	26.02	21	10
34 & 35	34	23.6	23.39	23.8	24.62	21.25	10
	35	23.7	23.47	23.89	24.5	21.25	10
37 & 38	37	23.58	23.37	23.78	24.76	15.17	10
	38	23.55	23.29	23.6	SHRUB	15.17	SHRUB
36 & 48	36	25.45	25.15	25.63	26.62	15.08	10
	48						
13 & 14	13		0		0.2		
	14		0		0.2		
39 & 47	39						
	47						

TOTAL

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	AREA	ST. LEN.	LOWEST D SECTION	SROAD	SWALK
1 & 2	1 2	1.5	282	YES	0.0031	0.068
28 & 29	28 29	2.67	492	YES	0.0039	0.042
30 & 31	30 31	1.97	357	YES	0.0029	0.042
45 & 46	45 46	0.47	213	YES	0.0088	0.096
3 & 27	3 27	0.36	95	YES	0.0096	0.05
4 & 26	4 26	0.38	91	YES	0.0052	0.05
22 & 25	22 25	0.8	235	YES	0.0046	0.067
23 & 24	23 24	3.76	640	YES	0.004	0.067
43 & 44	43 44	0.25	96	YES	0.0028	0.047
40′ & 42	40′ 42	0.14	86	YES	0.0022	0.047
5 & 21	5 21	0.5	100	YES	0.007	0.036
6 & 20	6 20	0.25	90	YES	0.0091	0.036
8 & 9	8 9	1.19	216	YES	0.0031	0.051
7 & 10	7 10	0.91	155	YES	0.0046	0.051
17′ & 17	17′ 17	0.48	167	YES	0.019	0.036
18 & 19	18 19	2.29	452	YES	0.0079	0.047

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	AREA	ST. LEN.	LOWEST D SECTION	SROAD	SWALK
32 & 33	32	2.21	415	YES	0.0056	0.047
	33					
40 & 41	40					
	41	0.38	180	YES	0.00124	0.098
11 & 12	11	2.39	457	YES	0.004	0.03
	12					
15 & 16	15					
	16	2.38	330	YES	0.003	0.095
34 & 35	34					
	35	3.37	547	YES	0.0036	0.061
37 & 38	37	0.49	110	41	0.018	0.098
	38					
36 & 48	36	1.17	152	YES	0.003	0.099
	48					
13 & 14	13	0.26		ALLEY	0.0095	0.02
	14	0.54		ALLEY	0.0057	0.02
39 & 47	39	0.52	600	DITCH	0.006	
	47	0.11	110	DITCH	0.006	
		31.7				

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	D-C	D-B′	(D-C)/ SROAD	(D-B′)/ SROAD
1 & 2	1 2	0.68	1.1	219	355
28 & 29	28 29	0.42	0.89	108	228
30 & 31	30 31	0.42	0.89	145	307
45 & 46	45 46	0.96	1.39	109	158
3 & 27	3 27	0.5	0.93	52	97
4 & 26	4 26	0.5	0.93	96	179
22 & 25	22 25	0.67	1.17	146	254
23 & 24	23 24	0.67	1.17	168	293
43 & 44	43 44	0.47	0.85	168	304
40′ & 42	40′ 42	0.47	0.85	214	386
5 & 21	5 21	0.36	0.86	51	123
6 & 20	6 20	0.36	0.86	40	95
8 & 9	8 9	0.51	0.94	165	303
7 & 10	7 10	0.51	0.94	111	204
17′ & 17	17′ 17	0.36	0.86	19	45
18 & 19	18 19	0.47	0.84	59	106

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	D-C	D-B′	(D-C)/ SROAD	(D-B′)/ SROAD
32 & 33	32 33	0.47	0.86	84	154
40 & 41	40 41	0.98	1.39	790	1121
11 & 12	11 12	0.3	0.7	75	175
15 & 16	15 16	0.95	1.38	317	460
34 & 35	34 35	0.61	1.03	169	286
37 & 38	37 38	0.98	1.39	54	77
36 & 48	36 48	0.99	1.47	330	490
13 & 14	13 14		0.2 0.2		
39 & 47	39 47				

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	(C-B')/ SROAD	SCROWN	ST. LEN.	EQN #
1 & 2	1 2	135		282	2
28 & 29	28 29	121		492	1
30 & 31	30 31	162		357	1
45 & 46	45 46	49		213	1
3 & 27	3 27	45		95	2
4 & 26	4 26	83		91	2
22 & 25	22 25	109		235	2
23 & 24	23 24	125		640	1
43 & 44	43 44	136	0.0129	96	3
40' & 42	40' 42	173	0.0129	86	3
5 & 21	5 21	71		100	2
6 & 20	6 20	55		90	2
8 & 9	8 9	139		216	2
7 & 10	7 10	93		155	2
17' & 17	17' 17	26		167	1
18 & 19	18 19	47		452	1

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	(C-B')/ SROAD	SCROWN	ST. LEN.	EQN #
32 & 33	32	70		415	1
	33				
40 & 41	40				
	41	331	0.0264	180	3
11 & 12	11	100		457	1
	12				
15 & 16	15				
	16	143		330	2
34 & 35	34				
	35	117		547	1
37 & 38	37	23		110	1
	38				
36 & 48	36	160	0.0172	152	3
	48				
13 & 14	13			21	ALLEY
	14			35	ALLEY
39 & 47	39				
	47				

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	D-C	C-B′	D-B′	STLEN* SROAD	VOLUME 1
1 & 2	1 2	0.68	0.42	1.1	0.87	1054
28 & 29	28 29	0.42	0.47	0.89		1778
30 & 31	30 31	0.42	0.47	0.89		2391
45 & 46	45 46	0.96	0.43	1.39		2550
3 & 27	3 27	0.5	0.43	0.93	0.91	419
4 & 26	4 26	0.5	0.43	0.93	0.47	5
22 & 25	22 25	0.67	0.5	1.17	1.08	1235
23 & 24	23 24	0.67	0.5	1.17		3401
43 & 44	43 44		0.38		0.27	269
40′ & 42	40′ 42		0.38		0.19	119
5 & 21	5 21	0.36	0.5	0.86	0.70	87
6 & 20	6 20	0.36	0.5	0.86	0.82	189
8 & 9	8 9	0.51	0.43	0.94	0.67	275
7 & 10	7 10	0.51	0.43	0.94	0.71	266
17′ & 17	17′ 17	0.36	0.5	0.86		320
18 & 19	18 19	0.47	0.37	0.84		873

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	D-C	C-B′	D-B′	STLEN* SROAD	VOLUME 1
32 & 33	32	0.47	0.39	0.86		1269
	33					
40 & 41	40					
	41				0.22	170
11 & 12	11	0.3	0.4	0.7		1019
	12					
15 & 16	15					
	16	0.95	0.43	1.38	0.99	1615
34 & 35	34					
	35	0.61	0.42	1.03		3005
37 & 38	37	0.98	0.41	1.39		1271
	38					
36 & 48	36				0.46	917
	48					
13 & 14	13					
	14					
39 & 47	39					
	47					

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	VOLUME 2	VOLUME 3	TOTAL VOLUME	10-YEAR VOLUME
1 & 2	1	769	356	2179	2170
	2				
28 & 29	28				
	29			1778	3862
30 & 31	30			2391	2850
	31				
45 & 46	45			2550	680
	46				
3 & 27	3	270	120	809	521
	27				
4 & 26	4	45	222	272	550
	26				
22 & 25	22	789	340	2365	1157
	25				
23 & 24	23			3401	5439
	24				
43 & 44	43			269	362
	44				
40′ & 42	40′				
	42			119	203
5 & 21	5				
	21	179	223	489	723
6 & 20	6				
	20	219	172	580	362
8 & 9	8	415	373	1063	1721
	9				
7 & 10	7	331	251	848	1316
	10				
17′ & 17	17′				
	17			320	694
18 & 19	18				
	19			873	3313

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	5-YEAR VOLUME
1 & 2	1	1132
	2	
28 & 29	28	
	29	2015
30 & 31	30	1487
	31	
45 & 46	45	355
	46	
3 & 27	3	272
	27	
4 & 26	4	287
	26	
22 & 25	22	604
	25	
23 & 24	23	2838
	24	
43 & 44	43	189
	44	
40′ & 42	40′	
	42	106
5 & 21	5	
	21	377
6 & 20	6	
	20	189
8 & 9	8	898
	9	
7 & 10	7	687
	10	
17′ & 17	17′	
	17	362
18 & 19	18	
	19	1728

MASTER SPREADSHEET-42ND. ST. RCS

CATCHMENTS	CATCHMENT	5-YEAR VOLUME
32 & 33	32	1668
	33	
40 & 41	40	
	41	287
11 & 12	11	1804
	12	
15 & 16	15	
	16	1796
34 & 35	34	
	35	2543
37 & 38	37	370
	38	
36 & 48	36	883
	48	
13 & 14	13	196
	14	408
39 & 47	39	392
	47	83
		23,955

APPENDIX 6

MAXIMUM STORAGE VOLUME EQUATIONS

42ND ST. RCS

APPENDIX N

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 7

BENEFIT/COST ANALYSIS FOR THE PHASE 3 STUDY

Paula M. Arsenault
HDR Engineering, Inc.

August 13, 1992

TECHNICAL MEMORANDUM
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN
PHASE 3, TASK 7

PREPARED BY: Paula Arsenault, HDR Engineering, Inc.
DATE: July 1, 1992
SUBJECT: Benefit/Cost Analysis for the Phase 3 Study

A. INTRODUCTION

This technical memorandum (TM) presents the benefit/cost analysis for the least-cost alternative improvement plans developed during the Phase 3 improvement planning study. This TM:

- Defines the objective of and approach to the benefit/cost evaluation;
- Presents the improvement plan benefits on a qualitative and quantitative basis;
- Compares alternatives in which the sewers remain combined, discusses the sewer separation alternatives, and then compares the two types of alternatives; and
- Identifies a recommended plan.

Task 6 of the Phase 3 study involved developing improvement plans to control street outflows and flooding for 2-, 5-, 10- and 25-year return period design storms, and to control CSOs for 1/year and 5/year design events. Other alternatives, such as the sewer separation alternatives, were also developed to determine the cost of complete outflow and CSO elimination. The Separate Sanitary Sewer Alternative will eliminate sewage outflows, but provides minimal flood control improvement. The Separate Storm Drain Alternative will both eliminate outflows and improve flood control to a 10-year design storm level.

A Baseline Rehabilitation Alternative was also defined in Task 6. This alternative represents the minimum level of effort required at this time to keep the existing system serviceable. Although it costs $88 million, this alternative

does not include flood, CSO, or outflow control improvements. It therefore has no benefits regarding these major project objectives and consequently can be considered "incomplete". For this reason the Baseline Rehabilitation Alternative is excluded from the following benefit/cost analysis. It should be noted, however, that each of the least-cost projects includes rehabilitation improvements, but the cost of those improvements varies from alternative to alternative due to different levels of conveyance system piping replacement.

The analytical procedures and results for the Task 6 work are presented in the TM entitled *Improvement Plans*. Least-cost improvement plans are summarized in Section 12 of that TM, and are also presented in Table 1 here. A summary of the major facilities included in the alternatives in which the sewers remain combined is presented in Table 2.

Because of the excessive cost of the 25-year flood control alternative, the City decided to eliminate it from further consideration. The capital cost of this alternative makes it obviously unaffordable.

B. OBJECTIVE OF THE ANALYSIS

The purpose of this analysis is to select a recommended improvement plan for the combined sewer system based on technical and regulatory factors. Affordability factors are addressed in a technical memorandum entitled *Rate Assessment for Least-Cost Improvement Plans*. The selection is achieved through a benefit/cost comparison for controlling outflows, CSOs and flooding within the combined sewer service area.

The benefit/cost analysis and selection of the recommended plan is based on:

- Cease and Desist Order No. 90-198
- Flooding in the streets during wet weather

Flood control is not an issue addressed in the Cease and Desist Order. The evaluation of alternatives for flood control was completed upon the City's request.

Controlling outflows (i.e. sewage backing out of the conveyance system onto streets and into structures during storms) is the focus of the Cease and Desist Order. An evaluation of CSO frequency and impacts, and improvements to control CSOs is required both by the Cease and Desist Order and the NPDES permit for the CWCS. The CWCS NPDES permit requires compliance with the State and National CSO Control Strategies, as well as a water quality sampling and reporting program. Evaluation of flood control costs and benefits were requested by the City.

TABLE 1. SUMMARY OF COST ESTIMATES FOR LEAST-COST ALTERNATIVE IMPROVEMENT PLANS ($ millions)[4]

Flood Control Design Storm	Least-Cost Alternative[5]	CSO Control Design Storm	
		1/Year CSO	5/Year CSO
Alternatives Wherein Sewers Remained Combined			
2-year	Upsize Existing CWCS Conveyance w/ Treatment at Pioneer Reservoir Site	$ 363	
5-year	15-foot Tunnel System	$ 587	
10-year	17-foot Tunnel System	$ 797	
2-year	Upsize Existing CWCS Conveyance		$ 312
5-year	14-foot Tunnel System		$ 584
10-year	17-foot Tunnel System		$ 793
25-year	20-foot Tunnel System	$ 1,035[1]	$ 1,035[1]
Sewer Separation Alternatives			
N.A.[2]	New Sanitary System[3]	$ 381	

1 This alternative controls both the 1/year and 5/year CSO control design storms.
2 The flood control capability of the existing system would remain unchanged from the existing system, which approximately controls flooding for the 1.5- to 2-year storm.
3 The existing piping would serve as the storm drain system, with minimal flood control improvements.
4 Includes all rehabilitation/repair and reliability improvement costs for the CWCS conveyance system and major facilities.
5 See the Phase 3 TM entitled Improvement Plans for a complete description of these alternative improvement plans. Section 12 of this TM summarizes the least-cost plans. A least-cost plan was identified for each group of alternatives satisfying a combination of a flood and CSO control design storms.

TABLE 2. SUMMARY OF MAJOR FACILITIES INCLUDED IN LEAST-COST ALTERNATIVE IMPROVEMENT PLANS WHEREIN SEWERS REMAIN COMBINED

Description	CSO Design Storm	2-Year Flood Control Design Storm	5-Year Flood Control Design Storm	10-Year Flood Control Design Storm	25-Year Flood Control
Tunnels w/ Consolidation Conduit and Weir System	1/Year		consolidation conduit/weir system; 15-foot tunnel; CSO elevation at -18.00 feet MSL; 14-mgd emptying stations at Sump 2 and and 1A to SRWTP	consolidation conduit/weir system; 17-, 18- and 19-foot tunnel segments); CSO elevation at -23.00 feet MSL; 430-mgd and 265-mgd CSO stations at Sump 1A and 2, respectively; 14-mgd emptying stations at Sump 2 and 1A to SRWTP	consolidation conduit/weir system; 20-foot tunnel; CSO elevation at -18.00 feet MSL; 720-mgd and 350-mgd CSO stations at Sump 1A and 2, respectively; 14-mgd emptying stations at Sump 2 and 1A to SRWTP
	5/Year		consolidation conduit/weir system; 14-foot tunnel; CSO elevation at -18.00 feet MSL; 14-mgd emptying stations at Sump 2 and 1A to SRWTP	consolidation conduit/weir system; 17-, 18- and 19-foot tunnel segments); CSO elevation at <-24.00 feet MSL; 420-mgd and 250-mgd CSO stations at Sump 1A and 2, respectively; 14-mgd emptying stations at Sump 2 and 1A to SRWTP	
Upsized CWCS Conveyance w/ Primary Treatment Plant at Pioneer Reservoir Site	1/Year	upsize existing laterals and mains; 500-mgd primary treatment plant; gravity effluent discharge			
Upsized CWCS Conveyance	5/Year	upsize existing laterals and mains			

For combined sewer alternatives, it was necessary to directly relate outflow control frequency to flood control frequency, due to the infeasibility of quantifying and locating system outflows. A discussion of issues relating to the analysis of outflows is provided in the TM entitled *Approach to Outflow Analysis*.

Based on the requirements of the Cease and Desist Order and the CWCS NPDES permit, the benefit/cost analysis focuses on the following criteria, in order of importance:

1. Outflow control
2. CSO control
3. Flood control

The next section presents the intended versus implemented approaches to the benefit/cost analysis.

C. APPROACH TO ANALYSIS

C.1. Phase 3 Scope of Work

The Phase 3 Scope of Work (SOW) identified a step-wise procedure for conducting a benefit/cost analysis. The basis for comparison of projects and selection of the recommended improvement plan would be defined by the "knee" of the benefit/cost curve. This is the point on the curve at which the cost of flood and CSO control improvements begin to rapidly outweigh the benefits.

In order to determine flood control benefits of the various least-cost improvement plans, the City proposed to use the 25-year design storm as a relative measure, which is the basis for the Phase 3 SOW. Flood control benefits for the plans would then be defined in terms of the volume of system-wide flooding that is reduced for a 25-year, 6-hour design storm relative to the volume that would occur in the existing system for the 25-year storm event. CSO control benefits are defined in terms of the average annual CSOs remaining following the construction of improvements. The Phase 3 analysis used design storms as a measure of CSO control: the 5-events or 1-event per year design

storms. Therefore, this approach depended solely on the use of design storms in determining project benefits.

This procedure basically consists of two steps:

1) Prepare a comparison of project costs to flood control benefits for the two sets of the four least-cost improvement plans. Each set of plans represents a group of plans that meet a CSO control level (i.e. the 5/year and 1/year design storms). Graphically identify the knee-of-the-curve point on each curve.

2) Review the results of the analysis and select a final recommended improvement plan.

A continuous simulation version of SSWMM91 was also proposed to develop the statistical information for measuring the relative benefits of alternative improvement plans. The use of annual statistics developed through continuous simulation is generally used in benefit/cost analyses for combined systems. However, difficulties with achieving reliable results from the continuous SSWMM91, did not allow for the completion of the Phase 3 analysis as described in the Phase 3 SOW (i.e. use of a 25-year design event as a relative measure) or as proposed by a continuous simulation approach.

C.2. Revisions and Enhancements to the Phase 3 SOW Approach

Due to SSWMM91 modeling difficulties, the approach developed in the Phase 3 SOW had to be modified to compare benefits on a qualitative basis, and to use only design storm information. In addition to the cost curve development identified in the Phase 3 SOW, several other factors were considered in the analysis. These are described below, along with an explanation of how the corresponding information was generated and its use in the analysis.

1) **Duration of flooding.** Duration of flooding can only be measured for the existing system during the flood control design storm, because the alternative plans are designed to control flooding for that design storm.

For instance, the flood duration benefits for the 2-year least-cost plans (one for each level of CSO control) are measured relative to the duration of flooding during a 2-year storm imposed on the existing system. Duration is taken from the SSWMM91 outputs for total system-wide flooding versus time.

2) **Volume of flooding.** The approach for this parameter is identical to the flooding duration. Values for this parameter are taken from the SSWMM91 output of system-wide flood volume versus time, and selecting the peak volume.

3) **CSO volume.** CSO control benefits for alternative improvement plans can only be measured using a design storm approach for the existing system. Since the 5/year storm does not cause an overflow from the existing system (it can be handled by the existing facilities), a cost versus benefits analysis cannot be determined. For the 1/year CSO storms and all of the flood control design storms, CSO volume can be computed from the SSWMM91 output hydrographs.

4) **CSO duration.** The approach for this parameter is identical to CSO volume.

5) **Wastewater volume treated at the SRWTP.** This is the total volume that receives secondary treatment at SRWTP either during the storm or as the conveyance system and storage facilities are emptied following the rainfall event. It was assumed that, in general, increasing the volume of combined wastewater volume given secondary treatment would be preferred over discharging to the Sacramento River or primary treatment.

6) **Wastewater volume treated at the existing CWTP.** This is the volume routed to the CWTP during an event, until the inflow rate to Sump 2 recedes to 60 mgd, following a storm. In addition, wastewater volume routed to new treatment facilities is identified and included in the total volume given primary treatment for an alternative.

D. IMPROVEMENT PLAN BENEFITS AND COSTS

D.1. Benefits Summary Data

Table 3 presents the results of the benefits analysis for the existing system and for the least-cost alternatives, for alternatives that keep the sewers combined. Outflows from the conveyance system onto streets could not be quantified, and can only be measured relative to flood control. Therefore, they were not included in the table.

The two separation alternatives developed must be compared on an essentially qualitative basis with combined sewer alternatives (i.e improvement plans such as the tunnel systems wherein the sewers remain combined). The separation alternatives are the only plans that ensure all combined system outflows and public health issues related to the Cease and Desist Order will be eliminated.

The following presents the benefit/cost comparison first for the least-cost plans for the combined sewer alternatives. Incremental benefits of the 1/year versus 5/year CSO design storm levels are first discussed. Then, cost-effectiveness with regard to street outflow and flood control is discussed. Finally, a comparison of these results to the sewer separation alternatives is addressed.

TABLE 3. SUMMARY OF BENEFITS FOR LEAST-COST ALTERNATIVE IMPROVEMENT PLANS

COMPARISON PARAMETER	EXISTING CWCS						LEAST-COST ALTERNATIVES WHEREIN SEWERS REMAIN COMBINED							
	CSO STORM		FLOOD CONTROL DESIGN STORMS				2-Yr Upsized CWCS		5-Yr Tunnels		10-Yr Tunnels		25-Yr Tunnels	
	5/year	1/year	2-year	5-year	10-year	25-year	CSO Storm		CSO Storm		CSO Storm		CSO Storm	
							5/year	1/year	5/year	1/year	5/year	1/year	5/year	1/year
1. Duration of Flooding (hrs)	N.A.	N.A.	6.0	9.3	12.3	15.0	0.0	4.3	0.0	4.3	0.0	4.2	-	-
2. Volume of Flooding (MG)	N.A.	N.A.	0.5	33.5	73.8	118.9	0.0	2.8	0.0	1.2	0.0	1.0	-	-
3. CSO Volumes (MG)	0.0	60.4	25.7	83.5	125.1	163.3	0.0	0.0	0.0	0.0	0.0	0.0	-	-
4. CSO Durations (hrs)	0.0	14.0	6.8	8.7	10.7	12.2	-	-	-	-	-	-	-	-
5. Volume Wastewater Treated at SRWTP (MG):														
During the Storm	39.83	70.26	40.49	40.61	40.78	40.87	39.46	69.96	39.46	69.91	39.46	69.93	39.46	-
Stored in Pioneer Reservoir	27.16	25.70	24.03	25.14	26.44	27.72	25.82	25.42	25.82	25.70	25.82	25.59	25.82	-
Stored in the CWCS	5.81	5.86	5.47	6.57	7.83	9.45	5.89	5.59	5.89	5.55	5.89	5.92	5.89	-
Stored in New Facilities	-	-	-	-	-	-	-	-	-	59.48	-	59.68	-	-
TOTAL SRWTP TREATMENT	72.80	101.82	69.98	72.32	75.05	78.04	71.16	100.97	71.16	160.64	71.16	161.13	71.16	-
6. Volume Wastewater To Primary Treatment (MG):														
Diverted to CWTP	40.71	97.53	35.57	47.32	59.22	70.58	40.91	97.99	40.91	96.24	40.91	96.62	40.91	-
Diverted to New Facilities	-	-	-	-	-	-	-	58.07	-	-	-	-	-	-
TOTAL PRIMARY TREATMENT	40.71	97.53	35.57	47.32	59.22	70.58	40.91	156.06	40.91	96.24	**40.91**	**96.92**	**40.91**	-

Note: Volumes for the 1/year CSO design storm are larger than the 2-year flood control design storm because of the long duration of the CSO control storms.

D.2. Discussion of Sewer Separation Alternatives

The primary advantage of the sewer separation alternatives is that sewage outflows are completely eliminated. This achieves the primary requirement of the Cease and Desist Order. In addition, CSOs would be completely eliminated by these alternatives. The primary disadvantage of the separation alternatives is that additional stormwater and associated pollutants would be discharged to the Sacramento River, above the 130 mgd that it is assumed will receive primary treatment at the CWTP.

The stormwater outfalls associated with these alternatives would be subject to the EPA's requirements for obtaining new stormwater NPDES permits. However, with either of these alternatives, the existing CWTP and Pioneer Reservoir facilities could be used for stormwater storage and treatment. This would provide the City with an existing, extensive stormwater management program that may counter the overall increase of urban stormwater discharges to the Sacramento River. The extent of the mitigation of pollutant discharges is unknown.

The cost of installing a new storm drain system is substantially higher than installing a new sanitary system. In fact, a new 10-year drainage system is more costly than constructing a combined sewer tunnel system sized for a 10-year storm. On the basis of measuring only flood control benefits and costs, the new storm drain system should be eliminated from further consideration.

The new sanitary system would provide minimal flood control improvements. A small amount of additional capacity would be made available in the existing collection and conveyance system for storm drainage with the removal of sanitary flows.

Considering the primary objectives of the benefit/cost analysis, i.e. outflow control, the new sanitary system would be selected as the preferred alternative over the new storm drain system.

D.3. Comparison of Least-Cost Combined Sewer Alternatives

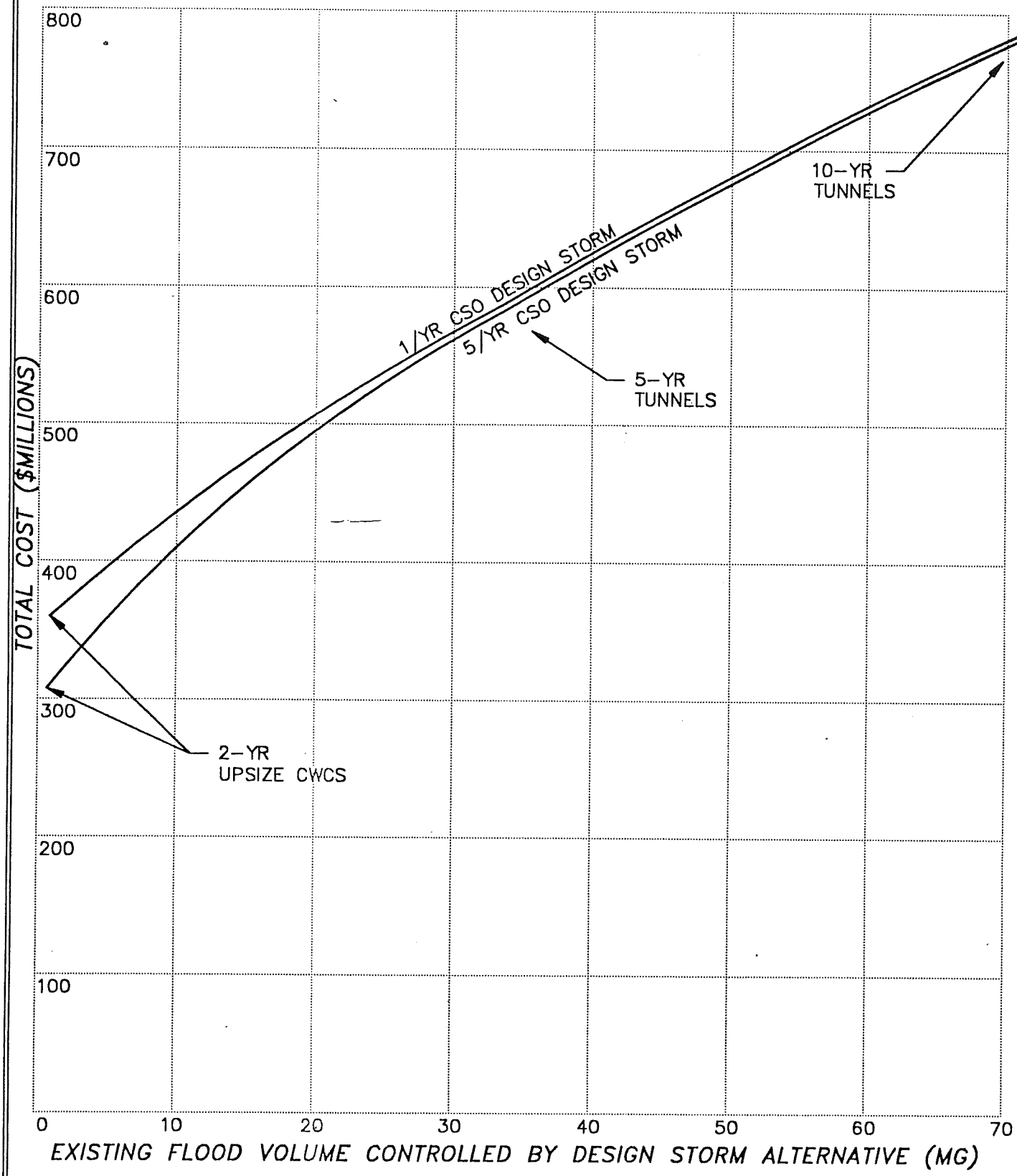
D.3.a. CSO Control--

Figures 1 and 2 illustrate total cost curves, (i.e. total cost versus design storm volume controlled and design storm frequency, respectively) for combined sewer alternatives. These curves illustrate the fact that the incremental cost of reducing CSOs from the 5/year to 1/year design storm levels is insignificant for the 5- and 10-year tunnel system alternatives ($3 and $4 million, respectively), and well within the accuracy of planning level cost estimates. For the 2-year alternative, the difference is $51 million or approximately 16 percent of the "base" 5/year CSO alternative cost. However, this is still within the accuracy of these planning level estimates. Therefore, the 1/year CSO event is selected as the most appropriate control level for Sacramento.

D.3.b. Outflow and Flood Control--

Although Figures 1 and 2 provide an illustration of the incremental costs associated with flood control, a "knee-of-the-curve" is not produced for selecting the most cost-effective level of flood control for the combined sewer alternatives. Therefore, unit cost computations were prepared for comparing the various levels of CSO control, which are presented in Table 4. These were calculated using the total cost of a particular improvement plan, as presented in Table 1, divided by the flood volumes for a given flood control design storm for the existing system from Table 3 (i.e the volume that would be controlled by a given alternative improvement plan).

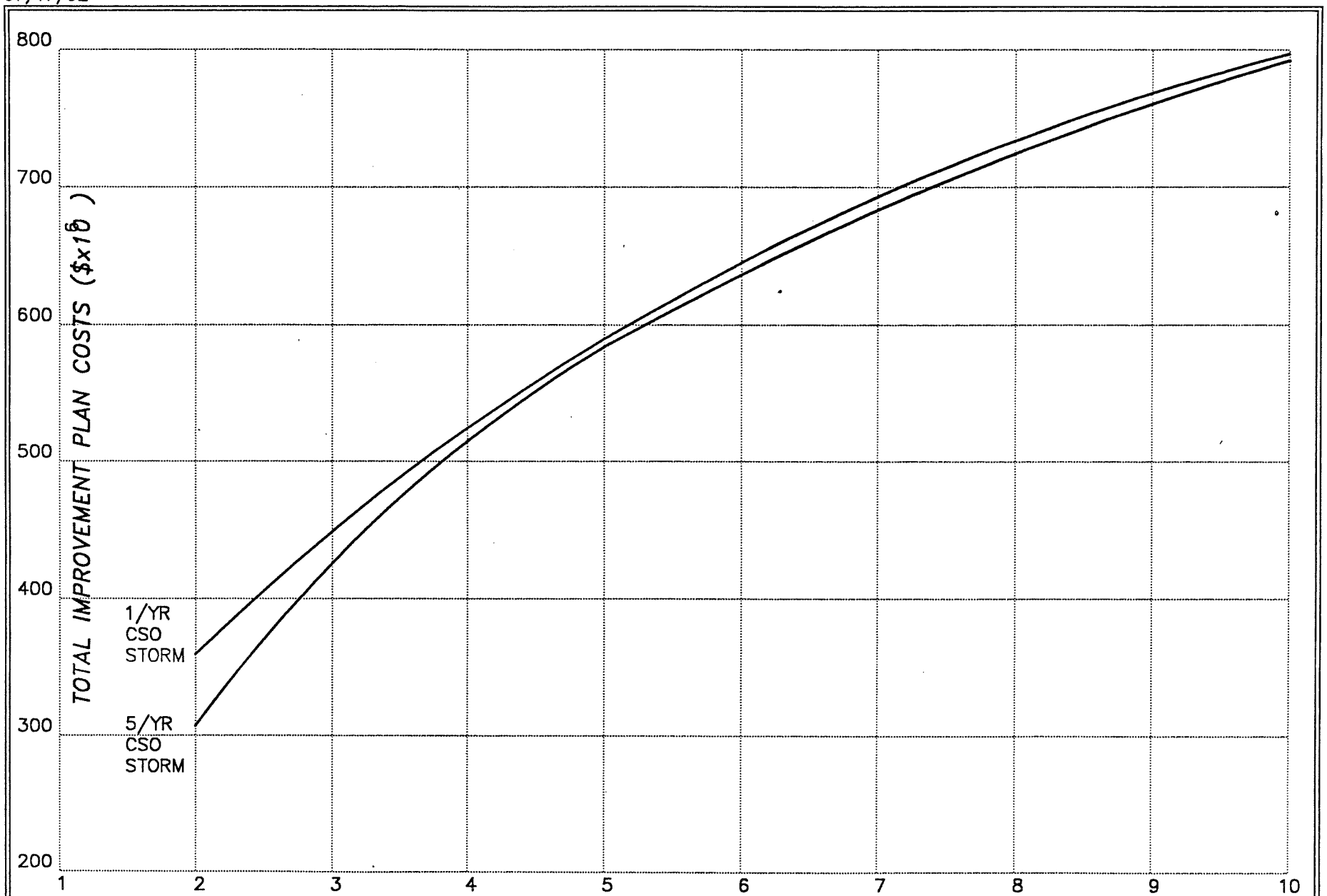
Unit cost versus design storm volume controlled (relative to the existing system) was plotted for the least-cost alternatives, and is shown in Figure 3. The results indicate that the 5-year tunnel system is the most cost-effective alternative for controlling flooding.



TOTAL COST CURVES FOR LEAST-COST ALTERNATIVES
WHEREIN SEWERS REMAIN COMBINED

HDR

Figure 1



TOTAL PLAN COST VS.
FLOOD CONTROL FREQUENCY

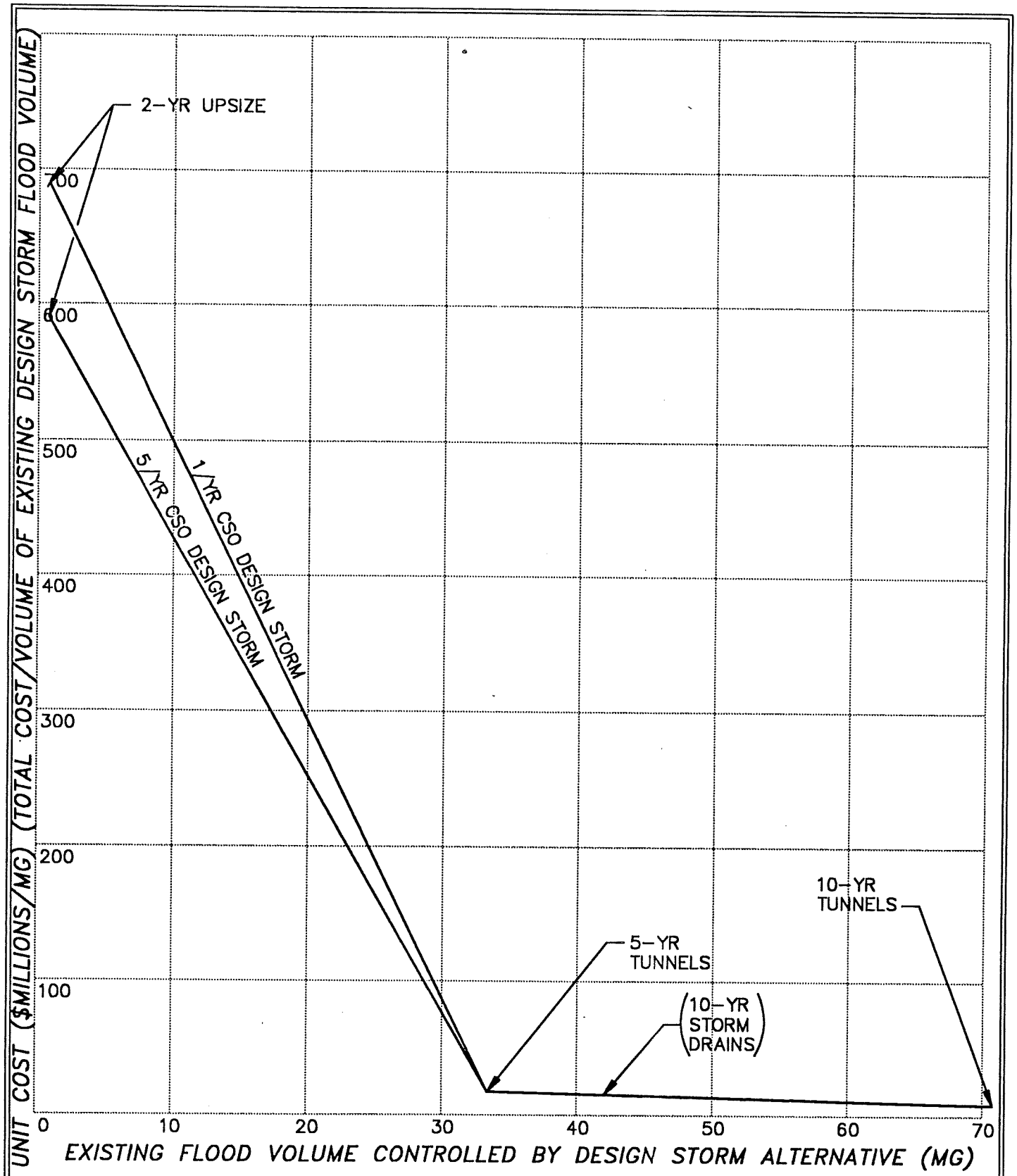
HDR

Figure 2

TABLE 4. SUMMARY OF FLOOD CONTROL UNIT COSTS FOR LEAST-COST ALTERNATIVE IMPROVEMENT PLANS			
Flood Control Design Storm	Least-Cost Alternative	Unit Cost ($ millions/MG)[1]	
		1/Year CSO	5/Year CSO
Alternatives Wherein Sewers Remained Combined			
2-year	Upsize Existing CWCS Conveyance w/ Treatment at Pioneer Reservoir Site	$ 698	
5-year	15-foot Tunnel System	$ 17.4	
10-year	17-foot Tunnel System	$ 10.8	
2-year	Upsize Existing CWCS Conveyance		$ 600
5-year	14-foot Tunnel System		$ 17.5
10-year	17-foot Tunnel System		$ 10.7
Alternatives Wherein Sewers Are Separated			
N.A.	New Sanitary System	N.A.[2]	

1 Total improvement plan costs divided by the existing design storm flood volume that would be controlled by a given alternative improvement plan.

2 No flood control improvements to the existing sewers are included in this alternative.



UNIT COST CURVES FOR LEAST-COST ALTERNATIVES WHEREIN SEWERS REMAIN COMBINED

HDR

Figure 3

Because street outflows are directly related to the level of flood control provided by an improvement plan, it is reasonable to assume that the 5-year tunnel system is the most cost-effective plan for controlling, but not eliminating, outflows. For this level of flood control, it would be reasonable to select the 1/year CSO control level.

D.3.c. Treatment Volumes--

The volume of combined wastewater that receives treatment by the alternative improvement plans increases for all of the 1/year improvement plans, because of the increased level of CSO control compared to the existing system. The amount of wastewater that would otherwise be discharged to the river during the 1/year event is treated either at the SRWTP (5- and 10-year tunnel systems), or by a new treatment facility near Pioneer Reservoir (2-year alternative). Assuming that a secondary level of treatment is preferred over primary treatment for wet weather flows, then the 5- and 10-year plans would be the most beneficial alternatives. Each of these treats approximately the same volume of water to secondary treatment levels.

D.4. Comparison of Combined Sewer Alternatives to Separation Alternatives

Based on the criteria set forth for the improvement plan and this benefit/cost analysis, the new Separate Sanitary Sewer Alternative would be selected over any of the alternatives wherein the sewers remain combined. However, as previously mentioned, this alternative provides only minimal improvement in the level of flood control. Relative to only flood control, the 5-year tunnel system would be selected, since the 2-year alternative provides only a marginal amount of additional flood control beyond the existing conveyance system. The new sanitary alternative would also provide the City with a new, long-term wastewater collection system with at least a 50-year life.

Any of the alternatives for which the sewers remain combined would require a long-term rehabilitation/replacement program to replace the existing, aging laterals/mains and interceptors that would remain in service as part of these alternative plans. (The cost of upsizing and replacing the collection system to provide improved flood control is included in these alternatives.) Maintenance

costs would also be higher over the remaining life of the existing conveyance piping for the combined sewer alternatives. Although the existing piping would remain in service as the storm drain system for this alternative, the potential consequences of sewage piping failure is greater than the consequences of storm drainage piping failure. Therefore, the new sanitary collection system provides greater safeguards for the City.

E. RECOMMENDED PLAN

Based on a benefit/cost analysis that establishes meeting the requirements of the Cease and Desist Order as the primary goal, the recommended plan is the Separate Sanitary Sewer Alternative.

F. SUMMARY

The following summarizes the key elements and conclusions of the benefit/cost analysis:

- The analysis and selection of the recommended plan is based on two major issues: 1) Cease and Desist Order No. 90-198 which addresses outflows and conformance with the National and State CSO Strategies; and 2) controlling flooding in the streets. The latter criteria was requested for inclusion and evaluation by the City.

- Since this improvement planning study was completed in response to the Cease and Desist Order, the primary criteria for the benefit/cost analysis are ranked in the following order of importance:

    1. Outflow control.
    2. CSO control.
    3. Flood control.

- The separation alternatives are the only plans that ensure all system outflows will be controlled during wet weather and that all CSOs will be eliminated.

- The Separate Storm Drain Alternative is not cost effective from a flood control standpoint, and is 2.5 times more expensive than the Separate Sanitary Sewer Alternative.

- The Separate Sanitary Sewer Alternative is selected as the recommended plan, based on the primary requirement of complying with the Cease and Desist Order.

- The most cost-effective alternative for controlling flooding is the 5-Year Tunnel System Alternative that controls the 1/year CSO design storm.

APPENDIX O

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 8

RATE ASSESSMENT FOR LEAST-COST IMPROVEMENT PLANS

Jerry Troyan
HDR Engineering, Inc.

August 13, 1992

TECHNICAL MEMORANDUM

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN

PHASE 3, TASK 8

PREPARED BY: Jerry Troyan, HDR Engineering, Inc.

DATE: July 1, 1992

SUBJECT: Rate Assessment for Least-Cost Improvement Plans

A. INTRODUCTION

This technical memorandum (TM) presents a brief comparison of the rate increases necessary for both the Baseline Rehabilitation Alternative and the least-cost alternative plans identified in the July 1, 1992 TM for Task 6 entitled, *Improvement Plans*. Both the dollar values of required rate increases and a measure of affordability of the new rates, based on the U.S. EPA Index of Hardship Threshold (EPA Index), are provided. The analysis incorporates several key assumptions, such as the rate of expenditure for construction, which are important to consider in reviewing the results. The objective of this TM is to present an initial assessment of the impact to the City's existing rate structure for all the identified least-cost plans, including rehabilitation costs, as one means of selecting a recommended plan.

The information in this TM is the result of data collected, and work performed, by both HDR Engineering and Bartle Wells Associates.

Development of a revenue program for the recommended plan is provided in a separate Phase 3 TM. That technical memorandum also contains detailed tabulations of the numbers and types of sewer user accounts, individual monthly user fees, and the City's existing budget for the combined sewer project, which are not included in this memorandum.

All tables referenced in this document are located at the end of the text.

B. APPROACH AND ANALYSIS ASSUMPTIONS

The approach and assumptions listed below in this memorandum are the same as those used in the Phase 2 study, with two exceptions. First, the Phase 2 study assumed an even capital expenditure stream over 10 years. The evaluation herein considers an approximated phasing schedule for the Baseline Rehabilitation Alternative and each of the least-cost plans.

B.1. Revenue Analysis

Key assumptions in estimating increases in revenues and corresponding fees include the following:

- Total project costs are phased for construction periods using moderate rates of construction as shown in Table 1. The first year of each project, however, is designated for design, permitting and related activities. Details of the procedure used to define these construction periods are given in the Task 7 TM entitled *Construction Phasing for the Recommended Improvement Plan*.

- The Baseline Rehabilitation Alternative represents the minimum work required to keep the existing facilities and conveyance system reliable. An estimated phasing schedule for the components of this work is shown in Table 2.

- The costs and phasing of rehabilitation work to Sumps 1, 1A and 2, and Pioneer Reservoir are common to all alternatives, as is the cost of new inspection and maintenance equipment. The construction cost of rehabilitation to the existing conveyance system varies from alternative to alternative, from $51 million for the Baseline Rehabilitation Alternative to $21 million for the 25-Year Tunnel System Alternative. The latter improvement plan has lower rehabilitation costs because more of the existing conveyance sewers are upsized (replaced) for flood control, making rehabilitation unnecessary.

- Project costs are escalated at an inflation rate of 4 percent.

- Operation and maintenance cost increases (above existing expenditures) for all alternatives are assumed to be the same as those used in the Phase 2 study, i.e. $1 million per year. Lower costs are used in the first two years to represent required increased maintenance in the existing sewer system. These costs also are escalated at 4 percent per year.

- Based on an evaluation of City billing reports for September 1991 and December 1991, including both sewer service charges and storm drainage charges, the distribution of revenue by account category is as follows:

Category	Percent of Revenue
Residential	74.4
Commercial	20.1
Metered	5.5
Total	100.0

- Financing for all alternatives is assumed to be by a 20-year issue revenue supported bond, at a net interest cost of 7.00 percent. The estimated annual debt service per million dollars of construction cost for these bonds is $101,000.

B.2. Plan Affordability

Assessment of the relative affordability of the alternatives is based on the increases determined in the revenue analyses for the least-cost plans, on residential monthly fees, the Household Effective Buying Income for the Sacramento area, and the EPA Index. Key assumptions regarding the latter three factors are as follows:

- This affordability assessment is made for a typical single-family residence. Residential accounts provide 74 percent of the City's service

charge revenue. The analysis further assumes that all capital costs of the alternative plans are supported entirely by service charge increases, i.e. no revenue is assumed from either connection fees or state loans.

- No other major capital expenditure programs by the City are included in this analysis.

- A typical six- to seven-room City residence now pays combined monthly fees of $24.28. Future increases are assumed to come only from City sewer and storm drainage fees, not from the Sacramento County Regional Sanitation District fee. The District is presently updating its wastewater treatment master plan. Projects being considered in that planning process could have a severe impact on the District's fees. As described in more detail in the Task 8 TM entitled *Revenue Program for the Recommended Plan*, potential rate increases could range from inflationary increases to roughly four times that amount by 2004 to 2010.

- The affordability of sewer service charges is assumed to be related to household income in the area. In 1990, the median household effective buying income (HEBI) for Sacramento was $27,200, as reported by *Sales and Marketing Management* in its August, 1991 issue.

- Residential fees and the HEBI are assumed to escalate at 4 percent annually.

- The EPA Index of household affordability used in this memorandum is 2 percent of the median HEBI. This is the informal guideline used by the USEPA in reviewing mandated sewer-related expenditures. The EPA Index for Sacramento, based on the 1990 HEBI of $27,200 would be $544/year, or $45.33/month.

C. RATE ASSESSMENT

C.1. Revenue Analysis

Tables 3 through 11 reflect preliminary financing plans for the least-cost alternative plans based on the previously described assumptions. Each table presents information in three parts. Part I provides inflated project costs using increases in the City of San Francisco Construction Cost Index, current values of which are found in *Engineering News Record* magazine. The ENR index is assumed to increase at 4 percent annually, like other inflationary increases in this analysis.

Table 4, for example, shows that the inflated annual construction costs for Alternative No. 1 will reach a maximum of $73.5 million in 1998/1999. Part II of these tables converts the inflated project costs to monthly additional revenue requirements; it also allocates these requirements to customer classifications assuming continuation of the existing allocation methods. For example, Table 4 also shows that in 1998/1999 in Alternative No. 1, residential users would be assessed an additional $4.6 million per month. Part III of these tables shows the cumulative additional revenue required and compares it to the current monthly revenue of $2.333 million on a percentage basis. Table 4, for example, shows that for Alternative No. 1 in 1998/1999, $1.311 million dollars per month more revenue would be required, a 56.2-percent increase above the current $2.333 million/month value. Examples of these calculations are attached as an appendix to this TM.

Cumulative additional revenue requirements in the last year of construction of the alternatives range from a low of a 186.4-percent increase, or an additional $4.35 million per month for Alternative No. 2, to a 558.5-percent increase, or an additional $13.03 million per month for Alternative No. 8. The Baseline Rehabilitation Project by itself would require a 42.0-percent increase in revenue requirements, or an additional $980,000 per month.

C.2. Plan Affordability

Tables 12 through 20 show the projected total monthly residential sewer and storm drainage rates for the estimated construction period of each of the least-cost alternatives. This rate includes existing average City sewer and storm drainage fees (including the Regional Sanitation District sewer fee) plus the additional fees required by each alternative plan. In addition, the EPA Index for each year is also shown in these tables. It should be emphasized that this affordability analysis assumes: 1) the absence of any other major capital expenditure programs; and 2) no significant increase in the Regional Sanitation District fee.

A summary comparison of estimated monthly fees required for each least-cost alternative to corresponding EPA Index fees is provided in Table 21. As indicated in the table, required total residential fees for each alternative vary as widely as the total project costs. Using the 2-percent EPA Index as guide, the Baseline Rehabilitation Project, and Alternatives 1, 2, 3, 4, and 5 (the 2- and 5-year flood control alternatives, and the Separate Sanitary Sewer Alternative) have residential fees that are always less than the EPA Index fee. However, the Baseline Rehabilitation Project is not a complete alternative. Alternatives 6, 7, and 8 (the tunnel alternatives for 10-year and 25-year flood control) all significantly exceed the affordable monthly fee. The year in which the maximum percent of the EBI occurs also varies, depending on the construction requirements indicated by the total annual budgets in Table 1. However, this variation is not significant in the affordability analysis.

Affordability analyses were also performed for shorter construction periods than those shown in Table 1. An estimate was made of the shortest practical construction period for the least-cost plans as the basis for this second analysis. As would be expected, all of the alternatives become less affordable as the construction period is shortened. For example, as shown in Tables 22 and 23, the maximum percent of the EBI for the Separate Sanitary Sewer Alternative increases from 1.56 percent for a 27-year period to 1.83 percent for its 11-year minimum construction period. The most important result from the affordability analysis for the shorter period, however, is that the <u>relative</u> affordability of

the least-cost projects remains the same, while all the projects become less affordable or more affordable as the construction period is shortened or lengthened, respectively.

D. SUMMARY

Based on a preliminary screening of the least-cost plans using a comparison to the EPA Index of Hardship Threshold, the Separate Sanitary Sewer Alternative, the 2-year flood control alternatives, and the 5-year flood control alternatives, are judged affordable. The three other complete alternative plans (two for 10-year flood control and one for 25-year flood control) are judged as significantly exceeding the EPA Index. The Baseline Rehabilitation Project is also judged to be affordable, but does not qualify as a complete alternative.

In addition, it should be noted that the EPA also recognizes that the affordability of a project is dependent on a community's individual mix of public expenditures, and the relative importance of the sewer-related project. Therefore, considering the other large capital costs, the City of Sacramento is pursuing (other local flood control projects, SAFCA flood control, stormwater quality control, solid waste, etc.), all the alternatives become less affordable.

E. CONCLUSIONS

The primary conclusions of the preceding affordability analysis are as follows:

- The Separate Sanitary Sewer Alternative is judged to be affordable.
- The least-cost alternative plans for 2-year flood control and 5-year flood control are also judged to be affordable.

- The relative ranking of affordable projects, from most affordable to least affordable, is:

    1. The 2-year flood control projects.
    2. Separate Sanitary Sewer Alternative.
    3. The 5-year flood control projects.

TABLE 1
SUMMARY OF PRELIMINARY TOTAL ANNUAL EXPENDITURES (X $1000)
FOR LEAST–COST ALTERNATIVES

YEAR	BASELINE REHAB	ALT.# 1 SEP.SANITARY SEWERS	ALT.# 2 2–YEAR 5/YR CSO	ALT.# 3 2–YEAR 1/YR CSO	ALT.# 4 5–YEAR 5/YR CSO	ALT.# 5 5–YEAR 1/YR CSO	ALT.# 6 10–YEAR 5/YR CSO	ALT.# 7 10–YEAR 1/YR CSO	ALT.# 8 25–YEAR
1	$3,100	$3,100	$3,100	$3,700	$4,400	$4,400	$4,800	$4,800	$5,300
2	$18,700	$18,700	$18,300	$35,200	$48,500	$48,800	$60,400	$60,400	$71,400
3	$13,700	$13,700	$40,400	$57,300	$72,900	$73,200	$108,800	$109,700	$133,200
4	$13,100	$13,100	$38,800	$55,000	$86,200	$86,800	$130,800	$132,500	$194,500
5	$10,800	$12,40[illegible]	$14,800	$14,800	$60,200	$60,900	$94,600	$95,000	$123,700
6	$18,900	$58,100	$13,400	$13,500	$65,200	$66,200	$101,400	$101,700	$141,200
7	$9,300	$27,500	$10,200	$10,200	$27,600	$27,600	$51,900	$52,300	$70,300
8		$18,200	$10,200	$10,200	$10,000	$10,000	$10,900	$10,900	$39,800
9		$18,200	$10,200	$10,200	$10,000	$10,000	$11,000	$11,000	$11,100
10		$18,200	$10,200	$10,200	$10,000	$10,000	$10,900	$10,900	$11,100
11		$18,200	$10,200	$10,200	$10,000	$10,000	$11,000	$11,000	$11,100
12		$18,200	$10,200	$10,200	$10,000	$10,000	$10,900	$10,900	$11,100
13		$18,300	$10,200	$10,200	$10,000	$10,000	$11,000	$11,000	$11,100
14		$17,800	$10,200	$10,200	$10,000	$10,000	$10,900	$10,900	$11,100
15		$8,300	$10,200	$10,200	$10,000	$10,000	$11,000	$11,000	$11,100
16		$8,300	$10,200	$10,200	$10,000	$10,000	$10,900	$10,900	$11,100
17		$8,300	$10,200	$10,200	$10,000	$10,000	$11,000	$11,000	$11,100
18		$8,300	$10,200	$10,200	$10,000	$10,000	$10,900	$10,900	$11,100
19		$8,300	$10,200	$10,200	$10,000	$10,000	$11,000	$11,000	$11,100
20		$8,300	$10,200	$10,200	$10,000	$10,000	$10,900	$10,900	$11,100
21		$8,300	$10,200	$10,200	$10,000	$10,000	$11,000	$11,000	$11,100
22		$8,300	$10,200	$10,200	$10,000	$10,000	$10,900	$10,900	$11,100
23		$8,300	$10,200	$10,200	$10,000	$10,000	$11,000	$11,000	$11,100
24		$8,300	$9,800	$9,800	$10,000	$10,000	$10,900	$10,900	$11,100
25		$8,400			$10,000	$10,000	$11,000	$11,000	$11,100
26		$8,300			$10,000	$10,000	$10,900	$10,900	$11,100
27		$8,000			$10,000	$10,000	$11,000	$11,000	$11,100
28					$10,000	$10,000	$10,900	$10,900	$11,100
29					$9,600	$9,600	$10,500	$10,500	$11,100
30									$11,100
31									$10,700
TOTAL	$87,600	$381,400	$312,000	$362,700	$584,600	$587,500	$793,100	$796,800	$1,034,300

NOTES:
(1) All values in 1992 dollars, based on planning–level information.
(2) Alternatives 2 to 8 are:
2 – Upsized CWCS Conveyance – 2–Year Flood Control, 5/Yr CSO Storms
3 – Upsized CWCS Conveyance with Treatment at Pioneer Site – 2–Year Flood Control, 1/Year CSO Storms
4 – Tunnels – 5–Year Flood Control, 5/Yr CSO Storms
5 – Tunnels – 5–Year Flood Control, 1/Yr CSO Storms
6 – Tunnels – 10–Year Flood Control, 5/Yr CSO Storms
7 – Tunnels – 10–Year Flood Control, 1/Yr CSO Storms
8 – Tunnels – 25–Year Flood Control Storm

TABLE 2. PROJECT PHASING ANNUAL BUDGET ESTIMATES
($1000)
BASELINE REHABILITATION PROJECT

Year	Design	Sumps 1, 1A & 2 Rehab	Pioneer Reservoir Improv.	Existing Conveyance System & MH Rehab	New Inspect. & Maintenance Equipment	Total Annual Budget[1]
1	$881				$2,145	$3,026
2	$548	$11,358	$1,091	$5,740		$18,736
3	$525			$13,196		$13,721
4	$416			$12,658		$13,073
5	$770			$10,038		$10,808
6	$385	$9,318		$9,163		$18,867
7		$9,318				$9,318
TOTAL	$3,526	$29,994	$1,091	$50,795	$2,145	$87,551

NOTES:

1 1992 Dollars.

TABLE 12

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

BASELINE REHABILITATION ALTERNATIVE

ASSUMPTIONS:

* Project size (x $1000)	$87,551
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900																								
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17																								
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (x $1,000) (3)	$67	$237	$417	$556	$666	$877	$980																								
Increase over current revenue (x $1,000) (3)	2.9%	10.2%	17.9%	23.8%	28.5%	37.6%	42.0%																								
Required residential fee–total (4)	$25.72	$27.90	$30.19	$32.25	$34.14	$36.79	$38.72																								
Required fee as a percent of HEBI	1.03%	1.07%	1.12%	1.15%	1.17%	1.21%	1.23%																								

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 13

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.1, SEPARATE SANITARY SEWERS

ASSUMPTIONS:

* Project size (x $1000)	$381,397
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300	$46,100	$47,900	$49,800	$51,800	$53,900	$56,100	$58,300	$60,600	$63,000	$65,500	$68,100	$70,800	$73,600	$76,500	$79,600	$82,800				
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83	$76.83	$79.83	$83.00	$86.33	$89.83	$93.50	$97.17	$101.00	$105.00	$109.17	$113.50	$118.00	$122.67	$127.50	$132.67	$138.00				
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (x $1,000) (3)	$67	$237	$417	$556	$683	$1,311	$1,616	$1,826	$2,054	$2,280	$2,526	$2,771	$3,036	$3,295	$3,430	$3,571	$3,707	$3,858	$4,016	$4,169	$4,338	$4,514	$4,696	$4,876	$5,072	$5,275	$5,480				
Increase over current revenue (x $1,000) (3)	2.9%	10.2%	17.9%	23.8%	29.3%	56.2%	69.3%	78.2%	88.0%	97.7%	106.3%	118.8%	130.1%	141.2%	147.0%	153.0%	158.9%	165.4%	172.1%	178.7%	185.9%	193.5%	201.3%	209.0%	217.4%	226.1%	234.9%				
Required residential fee–total (4)	$25.72	$27.90	$30.19	$32.25	$34.26	$39.79	$43.12	$45.85	$48.76	$51.70	$54.84	$58.03	$61.42	$64.82	$67.44	$70.16	$72.92	$75.86	$78.92	$82.02	$85.32	$88.75	$92.31	$95.95	$99.79	$103.79	$107.89				
Required fee as a percent of HEBI	1.03%	1.07%	1.12%	1.15%	1.17%	1.31%	1.37%	1.40%	1.43%	1.46%	1.49%	1.51%	1.54%	1.56%	1.56%	1.56%	1.56%	1.56%	1.56%	1.56%	1.56%	1.56%	1.56%	1.56%	1.57%	1.56%	1.56%				

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 14

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.2, 2-YEAR FLOOD CONTROL, 5/YEAR CSO

ASSUMPTIONS:

* Project size (x $1000)	$311,897
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300	$46,100	$47,900	$49,800	$51,800	$53,900	$56,100	$58,300	$60,600	$63,000	$65,500	$68,100	$70,800	$73,600							
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83	$76.83	$79.83	$83.00	$86.33	$89.83	$93.50	$97.17	$101.00	$105.00	$109.17	$113.50	$118.00	$122.67							
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (x $1,000) (3)	$67	$232	$864	$1,057	$1,208	$1,361	$1,474	$1,592	$1,724	$1,851	$1,993	$2,130	$2,283	$2,432	$2,597	$2,768	$2,935	$3,120	$3,311	$3,498	$3,704	$3,917	$4,139	$4,350							
Increase over current revenue (x $1,000) (3)	2.9%	10.0%	28.5%	45.3%	51.8%	58.3%	63.2%	68.2%	73.9%	79.3%	85.4%	91.3%	97.9%	104.3%	111.3%	118.6%	125.8%	133.7%	141.9%	149.9%	158.7%	167.9%	177.4%	186.4%							
Required residential fee–total (4)	$25.71	$27.87	$31.91	$35.71	$37.89	$40.13	$42.14	$44.23	$46.48	$48.74	$51.18	$53.60	$56.21	$58.86	$61.68	$64.61	$67.59	$70.76	$74.04	$77.39	$80.93	$84.62	$88.46	$92.31							
Required fee as a percent of HEBI	1.03%	1.07%	1.18%	1.27%	1.30%	1.32%	1.33%	1.35%	1.36%	1.37%	1.39%	1.40%	1.41%	1.42%	1.43%	1.44%	1.45%	1.46%	1.47%	1.47%	1.48%	1.49%	1.50%	1.51%							

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 15

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.3, 2-YEAR FLOOD CONTROL, 1/YEAR CSO

ASSUMPTIONS:

= Project size (x $1000)	$362,605
= Average residential sewer fee – City	$7.07 /month
= Average residential storm fee – City	$9.06 /month
= Average residential system fees – City	$16.13 /month
= Average residential sewer fee – Regional	$8.15 /month
= Average residential fees – total (1)	$24.28 /month
= Household effective buying income (1990)	$27,200
= Inflation rate	4.00%
= EPA Hardship Threshold index/year	2.00%

	Financing Year																																	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300	$46,100	$47,900	$49,800	$51,800	$53,900	$56,100	$58,300	$60,600	$63,000	$65,500	$68,100	$70,800	$73,600							
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83	$76.83	$79.83	$83.00	$86.33	$89.83	$93.50	$97.17	$101.00	$105.00	$109.17	$113.50	$118.00	$122.67							
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (x $1,000) (3)	$73	$392	$985	$1,537	$1,688	$1,841	$1,954	$2,072	$2,204	$2,331	$2,473	$2,610	$2,763	$2,912	$3,077	$3,248	$3,415	$3,600	$3,791	$3,978	$4,184	$4,397	$4,618	$4,830							
Increase over current revenue (x $1,000) (3)	3.1%	16.8%	42.2%	65.9%	72.4%	78.9%	83.8%	88.8%	94.5%	99.9%	106.0%	111.9%	118.4%	124.8%	131.9%	139.2%	146.4%	154.3%	162.5%	170.5%	179.3%	188.5%	198.0%	207.0%							
Required residential fee–total (4)	$25.75	$28.97	$34.12	$39.03	$41.21	$43.45	$45.48	$47.55	$49.79	$52.06	$54.48	$56.92	$59.53	$62.18	$65.00	$67.93	$70.91	$74.07	$77.36	$80.71	$84.25	$87.94	$91.77	$95.63							
Required fee as a percent of HEBI	1.03%	1.11%	1.26%	1.39%	1.41%	1.43%	1.44%	1.45%	1.46%	1.47%	1.48%	1.48%	1.49%	1.50%	1.51%	1.51%	1.52%	1.52%	1.53%	1.54%	1.54%	1.55%	1.56%	1.56%							

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 16

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.4, 5-YEAR FLOOD CONTROL, 5/YEAR CSO

ASSUMPTIONS:

* Project size (x $1000)	$584,293
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300	$46,100	$47,900	$49,800	$51,800	$53,900	$56,100	$58,300	$60,600	$63,000	$65,500	$68,100	$70,800	$73,600	$76,500	$79,600	$82,800	$86,100	$89,500		
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83	$76.83	$79.83	$83.00	$86.33	$89.83	$93.50	$97.17	$101.00	$105.00	$109.17	$113.50	$118.00	$122.67	$127.50	$132.67	$138.00	$143.50	$149.17		
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (x $1,000) (3)	$78	$520	$1,260	$2,119	$2,736	$3,439	$3,745	$3,861	$3,990	$4,115	$4,254	$4,389	$4,539	$4,684	$4,846	$5,013	$5,177	$5,357	$5,543	$5,728	$5,930	$6,139	$6,356	$6,572	$6,805	$7,049	$7,301	$7,563	$7,824		
Increase over current revenue (x $1,000) (3)	3.3%	22.3%	54.0%	90.8%	117.3%	147.4%	160.5%	165.5%	171.0%	176.4%	182.4%	188.1%	194.5%	200.8%	207.7%	214.9%	221.9%	229.6%	237.6%	245.5%	254.2%	263.1%	272.4%	281.7%	291.7%	302.1%	312.9%	324.2%	335.4%		
Required residential fee–total (4)	$25.79	$29.86	$36.02	$43.06	$48.45	$54.50	$57.85	$59.92	$62.15	$64.39	$66.79	$69.22	$71.81	$74.43	$77.23	$80.14	$83.09	$86.22	$89.48	$92.80	$96.32	$99.99	$103.79	$107.67	$111.78	$116.05	$120.49	$125.10	$129.82		
Required fee as a percent of HEBI	1.03%	1.15%	1.33%	1.53%	1.66%	1.80%	1.83%	1.83%	1.82%	1.81%	1.81%	1.80%	1.80%	1.79%	1.79%	1.78%	1.78%	1.77%	1.77%	1.77%	1.76%	1.76%	1.76%	1.76%	1.75%	1.75%	1.75%	1.74%	1.74%		

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 17

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.5, 5-YEAR FLOOD CONTROL, 1/YEAR CSO

ASSUMPTIONS:

* Project size (x $1000)	$587,201
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300	$46,100	$47,900	$49,800	$51,800	$53,900	$56,100	$58,300	$60,600	$63,000	$65,500	$68,100	$70,800	$73,600	$76,500	$79,600	$82,800	$86,100	$89,500		
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83	$76.83	$79.83	$83.00	$86.33	$89.83	$93.50	$97.17	$101.00	$105.00	$109.17	$113.50	$118.00	$122.67	$127.50	$132.67	$138.00	$143.50	$149.17		
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (x $1,000) (3)	$78	$523	$1,266	$2,130	$2,754	$3,469	$3,776	$3,891	$4,021	$4,145	$4,285	$4,419	$4,569	$4,715	$4,876	$5,044	$5,207	$5,387	$5,574	$5,758	$5,960	$6,169	$6,367	$6,602	$6,836	$7,079	$7,331	$7,593	$7,855		
Increase over current revenue (x $1,000) (3)	3.4%	22.4%	54.3%	91.3%	118.1%	148.7%	161.8%	166.8%	172.3%	177.7%	183.7%	189.4%	195.9%	202.1%	209.0%	216.2%	223.2%	230.9%	238.9%	246.8%	255.5%	264.4%	273.7%	283.0%	293.0%	303.4%	314.2%	325.5%	336.7%		
Required residential fee–total (4)	$25.79	$29.87	$36.07	$43.13	$48.58	$54.71	$58.06	$60.13	$62.36	$64.60	$67.00	$69.43	$72.02	$74.64	$77.44	$80.35	$83.30	$86.43	$89.69	$93.01	$96.53	$100.20	$104.00	$107.88	$111.99	$116.28	$120.70	$125.31	$130.03		
Required fee as a percent of HEBI	1.03%	1.15%	1.34%	1.54%	1.67%	1.80%	1.84%	1.83%	1.83%	1.82%	1.81%	1.81%	1.80%	1.80%	1.79%	1.79%	1.78%	1.78%	1.78%	1.77%	1.77%	1.77%	1.76%	1.76%	1.76%	1.75%	1.75%	1.75%	1.74%		

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 18

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.6, 10-YEAR FLOOD CONTROL, 5/YEAR CSO

ASSUMPTIONS:

* Project size (x $1000)	$793,195
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300	$46,100	$47,900	$49,800	$51,600	$53,900	$56,100	$58,300	$60,600	$63,000	$65,500	$68,100	$70,800	$73,600	$76,500	$79,600	$82,600	$86,100	$89,500		
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83	$76.83	$79.83	$83.00	$86.33	$89.83	$93.50	$97.17	$101.00	$105.00	$109.17	$113.50	$118.00	$122.67	$127.50	$132.67	$138.00	$143.50	$149.17		
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (x $1,000) (3)	$82	$632	$1,711	$3,010	$3,979	$5,068	$5,643	$5,770	$5,911	$6,047	$6,200	$6,347	$6,510	$6,670	$6,848	$7,028	$7,208	$7,404	$7,609	$7,811	$8,031	$8,259	$8,497	$8,733	$8,989	$9,255	$9,531	$9,817	$10,103		
Increase over current revenue (x $1,000) (3)	3.5%	27.1%	73.4%	129.0%	170.5%	217.2%	241.9%	247.3%	253.4%	259.2%	265.7%	272.0%	279.0%	285.9%	293.4%	301.3%	308.9%	317.4%	326.1%	334.8%	344.2%	354.0%	364.2%	374.3%	385.3%	396.7%	408.5%	420.8%	433.0%		
Required residential fee–total (4)	$25.82	$30.63	$39.14	$49.21	$57.05	$65.76	$70.97	$73.12	$75.43	$77.75	$80.24	$82.75	$85.44	$88.16	$91.06	$94.07	$97.13	$100.38	$103.76	$107.20	$110.86	$114.64	$118.59	$122.62	$126.87	$131.30	$135.90	$140.68	$145.57		
Required fee as a percent of HEBI	1.03%	1.18%	1.45%	1.75%	1.96%	2.17%	2.25%	2.23%	2.21%	2.19%	2.17%	2.15%	2.14%	2.12%	2.11%	2.09%	2.08%	2.07%	2.05%	2.04%	2.03%	2.02%	2.01%	2.00%	1.99%	1.98%	1.97%	1.96%	1.95%		

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 19

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.7, 10-YEAR FLOOD CONRTROL, 1/YEAR CSO

ASSUMPTIONS:

* Project size (x $1000)	$796,951
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300	$46,100	$47,900	$49,800	$51,800	$53,900	$56,100	$58,300	$60,600	$63,000	$65,500	$68,100	$70,800	$73,600	$76,500	$79,600	$82,800	$86,100	$89,500		
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83	$76.83	$79.83	$83.00	$86.33	$89.83	$93.50	$97.17	$101.00	$105.00	$109.17	$113.50	$118.00	$122.67	$127.50	$132.67	$138.00	$143.50	$149.17		
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (x $1,000) (3)	$82	$833	$1,721	$3,036	$4,009	$5,102	$5,681	$5,808	$5,949	$6,085	$6,237	$6,385	$6,548	$6,708	$6,884	$7,066	$7,245	$7,442	$7,647	$7,849	$8,069	$8,297	$8,534	$8,771	$9,027	$9,293	$9,569	$9,855	$10,141		
Increase over current revenue (x $1,000) (3)	3.5%	27.1%	73.8%	130.1%	171.8%	218.7%	243.5%	248.9%	255.0%	260.8%	267.4%	273.7%	280.7%	287.5%	295.1%	302.9%	310.6%	319.0%	327.8%	336.4%	345.9%	355.6%	365.8%	375.9%	386.9%	398.3%	410.1%	422.4%	434.7%		
Required residential fee–total (4)	$25.82	$30.64	$39.21	$49.39	$57.26	$66.00	$71.23	$73.38	$75.69	$78.01	$80.50	$83.02	$85.70	$88.42	$91.32	$94.33	$97.39	$100.64	$104.02	$107.47	$111.12	$114.91	$118.85	$122.88	$127.14	$131.56	$136.16	$140.94	$145.83		
Required fee as a percent of HEBI	1.03%	1.18%	1.45%	1.76%	1.96%	2.18%	2.26%	2.23%	2.22%	2.20%	2.18%	2.16%	2.15%	2.13%	2.12%	2.10%	2.08%	2.07%	2.06%	2.05%	2.04%	2.02%	2.01%	2.00%	1.99%	1.98%	1.97%	1.96%	1.96%		

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 20

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.8, 25-YEAR FLOOD CONTROL

ASSUMPTIONS:

* Project size (x $1000)	$1,034,927
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300	$46,100	$47,900	$49,800	$51,800	$53,900	$56,100	$58,300	$60,600	$63,000	$65,500	$68,100	$70,800	$73,600	$76,500	$79,600	$82,800	$86,100	$89,500	$93,100	$96,800
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83	$76.83	$79.83	$83.00	$86.33	$89.83	$93.50	$97.17	$101.00	$105.00	$109.17	$113.50	$118.00	$122.67	$127.50	$132.67	$138.00	$143.50	$149.17	$155.17	$161.33
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required–cum. (X $1,000) (3)	$87	$737	$2,047	$3,972	$5,239	$6,753	$7,532	$7,991	$8,134	$8,273	$8,427	$8,577	$8,742	$8,905	$9,083	$9,268	$9,451	$9,650	$9,857	$10,062	$10,286	$10,518	$10,759	$10,999	$11,259	$11,529	$11,809	$12,100	$12,402	$12,716	$13,030
Increase over current revenue (x $1,000) (3)	3.7%	31.6%	87.8%	170.3%	224.6%	289.5%	322.9%	342.5%	348.6%	354.6%	361.2%	367.6%	374.7%	381.7%	389.3%	397.3%	405.1%	413.6%	422.5%	431.3%	440.9%	450.9%	461.2%	471.5%	482.6%	494.2%	506.2%	518.7%	531.6%	545.1%	558.5%
Required residential fee–total (4)	$25.85	$31.35	$41.47	$55.87	$65.76	$77.41	$84.03	$88.48	$90.80	$93.14	$95.64	$98.17	$100.87	$103.61	$106.53	$109.55	$112.64	$115.91	$119.30	$122.77	$126.45	$130.26	$134.23	$138.28	$142.57	$147.03	$151.66	$156.47	$161.47	$166.67	$171.99
Required fee as a percent of HEBI	1.03%	1.21%	1.54%	1.99%	2.25%	2.55%	2.66%	2.69%	2.66%	2.62%	2.59%	2.56%	2.53%	2.50%	2.47%	2.44%	2.41%	2.39%	2.36%	2.34%	2.32%	2.30%	2.28%	2.25%	2.24%	2.22%	2.20%	2.18%	2.16%	2.15%	2.13%

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

TABLE 3

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

BASELINE REHABILITATION ALTERNATIVE

ASSUMPTIONS:

* Project Size (X $1000)	$57,551
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I–CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$3,100	$18,700	$13,700	$13,100	$10,800	$18,900	$9,300																								
ENR–S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193																								
Inflated Construction Cost	3224	20,200	15,400	15,300	13,100	23,900	12,200																								
II–TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333																								
Additional capital expense	269	1,700	1,300	1,300	1,100	2,000	1,000																								
Additional O&M expense	40	40	90	100	100	110	110																								
Additional revenue required – total	309	1,740	1,390	1,400	1,200	2,110	1,110																								
Residential (74.4%)	230	1,290	1,030	1,040	890	1,570	830																								
Commercial (20.1%)	62	350	280	280	240	420	220																								
Metered (5.5%)	17	100	80	80	70	120	60																								
III–REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service – current year issue	$27	$170	$130	$129	$110	$201	$103																								
O&M expense	40	40	90	100	100	110	110																								
Additional revenue required – cumulative	67	237	417	556	666	877	980																								
Increase over current revenue	2.9%	10.2%	17.9%	23.8%	28.5%	37.6%	42.0%																								

(1) City revenue only – excludes revenue from Regional Sanitation District fee

TABLE 4

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.1, SEPARATE SANITARY SEWERS

ASSUMPTIONS:

* Project Size (X $1000)	$381,397
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6225
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I—CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$3,100	$18,700	$13,700	$13,100	$12,400	$58,100	$27,500	$18,200	$18,200	$18,200	$18,200	$18,200	$18,200	$17,800	$8,300	$8,300	$8,300	$8,300	$8,300	$8,300	$8,300	$8,300	$8,300	$8,300	$8,300	$8,300	$8,000				
ENR—S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585	9,968	10,367	10,781	11,213	11,661	12,128	12,613	13,117	13,642	14,188	14,755	15,345	15,959	16,597	17,261	17,952				
Inflated Construction Cost	3224	20,200	15,400	15,300	15,100	73,500	36,200	24,900	25,900	26,900	28,000	29,100	30,300	30,800	14,900	15,500	16,200	16,800	17,500	18,200	18,900	19,700	20,500	21,300	22,100	23,000	23,100				
II—TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333				
Additional capital expense	289	1,700	1,300	1,300	1,300	6,100	3,000	2,100	2,200	2,200	2,300	2,400	2,500	2,600	1,200	1,300	1,400	1,400	1,500	1,500	1,600	1,600	1,700	1,800	1,800	1,900	1,900				
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240				
Additional revenue required – total	309	1,740	1,390	1,400	1,400	6,210	3,110	2,210	2,320	2,320	2,430	2,530	2,640	2,740	1,350	1,460	1,560	1,570	1,680	1,680	1,790	1,800	1,910	2,010	2,020	2,130	2,140				
Residential (74.4%)	230	1,290	1,030	1,040	1,040	4,620	2,310	1,640	1,730	1,730	1,810	1,880	1,960	2,040	1,000	1,090	1,160	1,170	1,250	1,250	1,330	1,340	1,420	1,500	1,500	1,580	1,590				
Commercial (20.1%)	62	350	280	280	280	1,250	630	440	470	470	490	510	530	550	270	290	310	320	340	340	360	360	380	400	410	430	430				
Metered (5.5%)	17	100	80	80	80	340	170	120	130	130	130	140	150	150	70	80	90	90	90	90	100	100	110	110	110	120	120				
III—REVENUE SUPPORTED BOND ISSUE MONTHLY COSTS (X $1,000)																															
Debt service – current year issue	$27	$170	$130	$129	$127	$619	$305	$210	$218	$228	$236	$245	$255	$259	$125	$130	$136	$141	$147	$153	$159	$166	$173	$179	$186	$194	$194				
O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240				
Additional revenue required – cumulative	67	237	417	556	683	1,311	1,616	1,826	2,054	2,280	2,526	2,771	3,036	3,295	3,430	3,571	3,707	3,858	4,016	4,169	4,338	4,514	4,696	4,876	5,072	5,275	5,480				
Increase over current revenue	2.9%	10.2%	17.9%	23.8%	29.3%	56.2%	69.3%	78.2%	88.0%	97.7%	108.3%	118.8%	130.1%	141.2%	147.0%	153.0%	158.9%	165.4%	172.1%	178.7%	185.9%	193.5%	201.3%	209.0%	217.4%	226.1%	234.9%				

(1) City revenue only – excludes revenue from Regional Sanitation District fee

TABLE 5

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.2, 2-YEAR FLOOD CONTROL, 5/YEAR CSO

ASSUMPTIONS:

* Project Size (X $1000)	$311,897
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I-CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$3,033	$18,254	$40,369	$38,837	$14,766	$13,449	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$10,199	$9,792							
ENR-S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585	9,968	10,367	10,781	11,213	11,661	12,128	12,613	13,117	13,642	14,188	14,755	15,345	15,959							
Inflated Construction Cost	3154	19,700	45,400	45,400	18,000	17,000	13,400	14,000	14,500	15,100	15,700	16,300	17,000	17,700	18,400	19,100	19,900	20,700	21,500	22,300	23,200	24,200	25,100	25,100							
II-TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333							
Additional capital expense	263	1,600	3,800	3,800	1,500	1,400	1,100	1,200	1,200	1,300	1,300	1,400	1,400	1,500	1,500	1,600	1,700	1,700	1,800	1,900	1,900	2,000	2,100	2,100							
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210							
Additional revenue required - total	303	1,640	3,890	3,900	1,600	1,510	1,210	1,310	1,320	1,420	1,430	1,530	1,540	1,640	1,650	1,760	1,860	1,870	1,980	2,080	2,090	2,200	2,310	2,310							
Residential (74.4%)	225	1,220	2,890	2,900	1,190	1,120	900	970	980	1,060	1,060	1,140	1,150	1,220	1,230	1,310	1,380	1,390	1,470	1,550	1,550	1,640	1,720	1,720							
Commercial (20.1%)	61	330	780	780	320	300	240	260	270	290	290	310	310	330	330	350	370	380	400	420	420	440	460	460							
Metered (5.5%)	17	90	210	210	90	80	70	70	70	80	80	80	80	90	90	100	100	100	110	110	110	120	130	130							
III-REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service - current year issue	$27	$166	$382	$382	$152	$143	$113	$118	$122	$127	$132	$137	$143	$149	$155	$161	$167	$174	$181	$188	$195	$204	$211	$211							
O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210							
Additional revenue required - cumulative	67	232	664	1,057	1,208	1,361	1,474	1,592	1,724	1,851	1,993	2,150	2,283	2,432	2,597	2,768	2,935	3,120	3,311	3,498	3,704	3,917	4,139	4,350							
Increase over current revenue	2.9%	10.0%	28.5%	45.3%	51.8%	58.3%	63.2%	68.2%	73.9%	79.3%	85.4%	91.3%	97.9%	104.3%	111.3%	118.6%	125.8%	133.7%	141.9%	149.9%	158.7%	167.9%	177.4%	186.4%							

(1) City revenue only - excludes revenue from Regional Sanitation District fee

TABLE 6

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.3, 2-YEAR FLOOD CONTROL, 1/YEAR CSO

ASSUMPTIONS:

* Project Size (X $1000)	$362,605
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I-CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$3,730	$35,155	$57,291	$55,056	$14,768	$13,448	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$10,198	$9,792							
ENR-S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585	9,968	10,367	10,781	11,213	11,661	12,128	12,613	13,117	13,642	14,188	14,755	15,345	15,959							
Inflated Construction Cost	3879	36,000	64,400	64,400	18,000	17,000	13,400	14,000	14,500	15,100	15,700	16,300	17,000	17,700	18,400	19,100	19,900	20,700	21,500	22,300	23,200	24,200	25,100	25,100							
II-TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333							
Additional capital expense	323	3,200	5,400	5,400	1,500	1,400	1,100	1,200	1,200	1,300	1,300	1,400	1,400	1,500	1,500	1,600	1,700	1,700	1,800	1,900	1,900	2,000	2,100	2,100							
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210							
Additional revenue required - total	363	3,240	5,490	5,500	1,600	1,510	1,210	1,310	1,320	1,420	1,430	1,530	1,540	1,640	1,650	1,760	1,860	1,870	1,980	2,080	2,090	2,200	2,310	2,310							
Residential (74.4%)	270	2,410	4,080	4,090	1,190	1,120	900	970	980	1,050	1,060	1,140	1,150	1,220	1,230	1,310	1,380	1,390	1,470	1,550	1,550	1,640	1,720	1,720							
Commercial (20.1%)	73	650	1,100	1,110	320	300	240	260	270	290	290	310	310	330	330	350	370	380	400	420	420	440	460	460							
Metered (5.5%)	20	180	300	300	90	80	70	70	70	80	80	80	80	90	90	100	100	100	110	110	110	120	130	130							
III-REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service - current year issue	$33	$320	$542	$542	$152	$143	$113	$118	$122	$127	$132	$137	$143	$149	$155	$161	$167	$174	$181	$188	$195	$204	$211	$211							
O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210							
Additional revenue required - cumulative	73	392	985	1,537	1,688	1,841	1,954	2,072	2,204	2,331	2,473	2,610	2,763	2,912	3,077	3,248	3,415	3,600	3,791	3,978	4,164	4,397	4,618	4,830							
Increase over current revenue	3.1%	16.8%	42.2%	65.9%	72.4%	78.9%	83.8%	88.8%	94.5%	99.9%	106.0%	111.9%	118.4%	124.8%	131.9%	139.2%	146.4%	154.3%	162.5%	170.5%	179.3%	188.5%	198.0%	207.0%							

(1) City revenue only - excludes revenue from Regional Sanitation District fee

TABLE 7

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.4, 5-YEAR FLOOD CONTROL, 5/YEAR CSO

ASSUMPTIONS:

* Project Size (X $1000)	$564,293
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I-CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$4,356	$48,543	$72,931	$86,215	$60,169	$65,158	$27,640	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,591		
ENR-S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585	9,968	10,367	10,781	11,213	11,661	12,128	12,613	13,117	13,642	14,188	14,755	15,345	15,959	16,597	17,261	17,952	18,670	19,417		
Inflated Construction Cost	4530	52,500	82,000	100,900	73,200	82,400	36,400	13,700	14,200	14,800	15,400	16,000	16,600	17,300	18,000	18,700	19,400	20,200	21,000	21,900	22,800	23,700	24,600	25,600	26,600	27,700	28,800	29,900	29,900		
II-TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333		
Additional capital expense	378	4,400	6,800	8,400	6,100	6,900	3,000	1,100	1,200	1,200	1,300	1,300	1,400	1,400	1,500	1,600	1,600	1,700	1,800	1,800	1,900	2,000	2,100	2,100	2,200	2,300	2,400	2,500	2,500		
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260		
Additional revenue required - total	418	4,440	6,890	8,500	6,200	7,010	3,110	1,210	1,320	1,320	1,430	1,430	1,540	1,540	1,650	1,760	1,760	1,870	1,980	1,980	2,090	2,200	2,310	2,310	2,420	2,530	2,640	2,750	2,760		
Residential (74.4%)	311	3,300	5,130	6,320	4,610	5,220	2,310	900	980	980	1,060	1,060	1,150	1,150	1,230	1,310	1,310	1,390	1,470	1,470	1,550	1,640	1,720	1,720	1,800	1,880	1,960	2,050	2,050		
Commercial (20.1%)	84	890	1,380	1,710	1,250	1,410	630	240	270	270	290	290	310	310	330	350	350	380	400	400	420	440	460	460	490	510	530	550	550		
Metered (5.5%)	23	240	380	470	340	390	170	70	70	70	80	80	80	80	90	100	100	100	110	110	110	120	130	130	130	140	150	150	150		
III-REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service - current year issue	$38	$442	$690	$849	$616	$694	$306	$115	$120	$125	$130	$135	$140	$146	$152	$157	$163	$170	$177	$184	$192	$199	$207	$215	$224	$233	$242	$252	$252		
O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260		
Additional revenue required - cumulative	78	520	1,260	2,119	2,736	3,439	3,745	3,861	3,990	4,115	4,254	4,389	4,539	4,684	4,846	5,013	5,177	5,357	5,543	5,728	5,930	6,139	6,356	6,572	6,805	7,049	7,301	7,563	7,824		
Increase over current revenue	3.3%	22.3%	54.0%	90.8%	117.3%	147.4%	160.5%	165.5%	171.0%	176.4%	182.4%	188.1%	194.5%	200.8%	207.7%	214.9%	221.9%	229.6%	237.6%	245.5%	254.2%	263.1%	272.4%	281.7%	291.7%	302.1%	312.9%	324.2%	335.4%		

(1) City revenue only - excludes revenue from Regional Sanitation District fee

TABLE 8

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.5, 5-YEAR FLOOD CONTROL, 1/YEAR CSO

ASSUMPTIONS:

* Project Size (X $1000)	$567,201
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I-CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$4,368	$46,822	$73,222	$96,794	$60,883	$56,191	$27,640	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,985	$9,591		
ENR-S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585	9,968	10,367	10,781	11,213	11,661	12,128	12,613	13,117	13,642	14,188	14,755	15,345	15,959	16,597	17,261	17,952	18,670	19,417		
Inflated Construction Cost	4543	52,800	82,400	101,500	74,100	83,800	36,400	13,700	14,200	14,800	15,400	16,000	16,600	17,300	18,000	18,700	19,400	20,200	21,000	21,900	22,800	23,700	24,600	25,600	26,600	27,700	28,800	29,900	29,900		
II-TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333		
Additional capital expense	379	4,400	6,900	8,500	6,200	7,000	3,000	1,100	1,200	1,200	1,300	1,300	1,400	1,400	1,500	1,600	1,600	1,700	1,800	1,800	1,900	2,000	2,100	2,100	2,200	2,300	2,400	2,500	2,500		
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260		
Additional revenue required - total	419	4,440	6,990	8,600	6,300	7,110	3,110	1,210	1,320	1,320	1,430	1,430	1,540	1,540	1,650	1,760	1,760	1,870	1,980	1,980	2,090	2,200	2,310	2,310	2,420	2,530	2,640	2,750	2,760		
Residential (74.4%)	312	3,300	5,200	6,400	4,690	5,290	2,310	900	980	980	1,060	1,060	1,150	1,150	1,230	1,310	1,310	1,390	1,470	1,470	1,550	1,640	1,720	1,720	1,800	1,880	1,960	2,050	2,050		
Commercial (20.1%)	84	890	1,400	1,730	1,270	1,430	630	240	270	270	290	290	310	310	330	350	350	380	400	400	420	440	460	460	490	510	530	550	550		
Metered (5.5%)	23	240	380	470	350	390	170	70	70	70	80	80	80	80	90	100	100	100	110	110	110	120	130	130	130	140	150	150	150		
III-REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service - current year issue	$36	$444	$694	$854	$624	$705	$306	$115	$120	$125	$130	$135	$140	$146	$152	$157	$163	$170	$177	$184	$192	$199	$207	$215	$224	$233	$242	$252	$252		
O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260		
Additional revenue required - cumulative	78	523	1,286	2,130	2,754	3,469	3,776	3,891	4,021	4,145	4,285	4,419	4,569	4,715	4,876	5,044	5,207	5,367	5,574	5,758	5,960	6,169	6,387	6,602	6,836	7,079	7,331	7,593	7,855		
Increase over current revenue	3.4%	22.4%	54.3%	91.3%	118.1%	148.7%	161.8%	166.8%	172.3%	177.7%	183.7%	189.4%	195.9%	202.1%	209.0%	216.2%	223.2%	230.9%	238.9%	246.8%	255.5%	264.4%	273.7%	283.0%	293.0%	303.4%	314.2%	325.5%	336.7%		

(1) City revenue only - excludes revenue from Regional Sanitation District fee

TABLE 9

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.6, 10-YEAR FLOOD CONTROL, 5/YEAR CSO

ASSUMPTIONS:

* Project Size (X $1000)	$793,195
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I-CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$4,843	$60,396	$108,765	$130,636	$94,614	$101,351	$51,941	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,517		
ENR-S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585	9,968	10,367	10,781	11,213	11,661	12,128	12,613	13,117	13,642	14,188	14,755	15,345	15,959	16,597	17,261	17,952	18,670	19,417		
Inflated Construction Cost	5037	65,300	122,300	153,100	115,100	128,200	66,400	15,000	15,600	16,200	16,900	17,500	18,200	19,000	19,700	20,500	21,300	22,200	23,100	24,000	25,000	25,900	27,000	28,100	29,200	30,400	31,600	32,800	32,800		
II-TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333		
Additional capital expense	420	5,400	10,200	12,800	9,600	10,700	5,700	1,300	1,300	1,400	1,400	1,500	1,500	1,600	1,600	1,700	1,800	1,900	1,900	2,000	2,100	2,200	2,300	2,300	2,400	2,500	2,600	2,700	2,700		
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260		
Additional revenue required - total	460	5,440	10,290	12,900	9,700	10,810	5,810	1,410	1,420	1,520	1,530	1,630	1,640	1,740	1,750	1,860	1,960	2,070	2,080	2,180	2,290	2,400	2,510	2,510	2,620	2,730	2,840	2,950	2,960		
Residential (74.4%)	342	4,050	7,660	9,600	7,220	8,040	4,320	1,050	1,060	1,130	1,140	1,210	1,220	1,290	1,300	1,380	1,460	1,540	1,550	1,620	1,700	1,790	1,870	1,870	1,950	2,030	2,110	2,190	2,200		
Commercial (20.1%)	92	1,090	2,070	2,590	1,950	2,170	1,170	280	290	310	310	330	330	350	350	370	390	420	420	440	460	480	500	500	530	550	570	590	590		
Metered (5.5%)	25	300	570	710	530	590	320	80	80	80	80	90	90	100	100	100	110	110	110	120	130	130	140	140	140	150	160	160	160		
III-REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service - current year issue	$42	$550	$1,029	$1,289	$969	$1,079	$576	$126	$131	$136	$142	$147	$153	$160	$166	$173	$179	$187	$194	$202	$210	$218	$227	$237	$246	$256	$266	$276	$276		
O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260		
Additional revenue required - cumulative	82	632	1,711	3,010	3,979	5,068	5,643	5,770	5,911	6,047	6,200	6,347	6,510	6,670	6,846	7,026	7,208	7,404	7,609	7,811	8,031	8,259	8,497	8,733	8,989	9,255	9,531	9,817	10,103		
Increase over current revenue	3.5%	27.1%	73.4%	129.0%	170.5%	217.2%	241.9%	247.3%	253.4%	259.2%	265.7%	272.0%	279.0%	285.9%	293.4%	301.3%	308.9%	317.4%	326.1%	334.8%	344.2%	354.0%	364.2%	374.3%	385.3%	396.7%	408.5%	420.8%	433.0%		

(1) City revenue only -- excludes revenue from Regional Sanitation District fee

TABLE 10

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.7, 10-YEAR FLOOD CONTROL, 1/YEAR CSO

ASSUMPTIONS:

* Project Size (X $1000)	$796,951
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I-CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$4,844	$50,433	$109,734	$132,467	$94,991	$101,728	$52,303	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,949	$10,517		
ENR-S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585	9,968	10,367	10,781	11,213	11,661	12,128	12,613	13,117	13,642	14,188	14,755	15,345	15,959	16,597	17,261	17,952	18,670	19,417		
Inflated Construction Cost	5038	65,400	123,400	155,000	115,600	128,700	68,800	15,000	15,600	16,200	16,900	17,500	18,200	19,000	19,700	20,500	21,300	22,200	23,100	24,000	25,000	25,900	27,000	28,100	29,200	30,400	31,600	32,800	32,800		
II-TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333		
Additional capital expense	420	5,500	10,300	12,900	9,600	10,700	5,700	1,300	1,300	1,400	1,400	1,500	1,500	1,600	1,600	1,700	1,800	1,900	1,900	2,000	2,100	2,200	2,300	2,300	2,400	2,500	2,600	2,700	2,700		
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260		
Additional revenue required - total	460	5,540	10,390	13,000	9,700	10,810	5,810	1,410	1,420	1,520	1,530	1,630	1,640	1,740	1,750	1,860	1,960	2,070	2,080	2,180	2,290	2,400	2,510	2,510	2,620	2,730	2,840	2,950	2,960		
Residential (74.4%)	342	4,120	7,730	9,670	7,220	8,040	4,320	1,050	1,060	1,130	1,140	1,210	1,220	1,290	1,300	1,380	1,460	1,540	1,550	1,620	1,700	1,790	1,870	1,870	1,950	2,030	2,110	2,190	2,200		
Commercial (20.1%)	92	1,110	2,090	2,610	1,950	2,170	1,170	280	290	310	310	330	330	350	350	370	390	420	420	440	460	480	500	500	530	550	570	590	590		
Metered (5.5%)	25	300	570	720	530	590	320	80	80	80	80	90	90	100	100	100	110	110	110	120	130	130	140	140	140	150	160	160	160		
III-REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service - current year issue	$42	$550	$1,039	$1,305	$973	$1,083	$579	$125	$131	$136	$142	$147	$153	$160	$166	$173	$179	$187	$194	$202	$210	$218	$227	$237	$246	$256	$266	$276	$276		
O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260		
Additional revenue required - cumulative	82	633	1,721	3,036	4,009	5,102	5,681	5,806	5,949	6,085	6,237	6,365	6,548	6,708	6,864	7,056	7,245	7,442	7,647	7,849	8,069	8,297	8,534	8,771	9,027	9,293	9,569	9,855	10,141		
Increase over current revenue	3.5%	27.1%	73.8%	130.1%	171.8%	218.7%	243.5%	248.9%	255.0%	260.8%	267.4%	273.7%	280.7%	287.5%	295.1%	302.9%	310.6%	319.0%	327.8%	336.4%	345.9%	355.6%	365.8%	375.9%	386.9%	398.3%	410.1%	422.4%	434.7%		

(1) City revenue only - excludes revenue from Regional Sanitation District fee

TABLE 11

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.8, 25-YEAR FLOOD CONTROL

ASSUMPTIONS:

* Project Size (X $1000)	$1,034,927
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I-CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$5,343	$71,370	$133,167	$194,465	$123,702	$141,214	$70,346	$39,769	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$11,129	$10,692
ENR-S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585	9,968	10,367	10,781	11,213	11,661	12,128	12,613	13,117	13,642	14,188	14,755	15,345	15,959	16,597	17,261	17,952	18,670	19,417	20,193	21,001
Inflated Construction Cost	5557	77,200	149,800	227,500	150,500	178,700	92,800	54,500	15,800	16,500	17,100	17,800	18,500	19,300	20,000	20,800	21,700	22,500	23,400	24,400	25,400	26,400	27,400	28,500	29,700	30,900	32,100	33,400	34,700	36,100	36,100
II-TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333
Additional capital expense	463	6,400	12,500	19,000	12,500	14,900	7,700	4,500	1,300	1,400	1,400	1,500	1,500	1,600	1,700	1,700	1,800	1,900	2,000	2,000	2,100	2,200	2,300	2,400	2,500	2,600	2,700	2,800	2,900	3,000	3,000
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260	270	280
Additional revenue required - total	503	6,440	12,590	19,100	12,600	15,010	7,810	4,610	1,420	1,520	1,530	1,630	1,640	1,740	1,850	1,860	1,960	2,070	2,180	2,180	2,290	2,400	2,510	2,610	2,720	2,830	2,940	3,050	3,160	3,270	3,280
Residential (74.4%)	374	4,790	9,370	14,210	9,370	11,170	5,810	3,430	1,060	1,130	1,140	1,210	1,220	1,290	1,380	1,380	1,460	1,540	1,620	1,620	1,700	1,790	1,870	1,940	2,020	2,110	2,190	2,270	2,350	2,430	2,440
Commercial (20.1%)	101	1,290	2,530	3,840	2,530	3,020	1,570	930	290	310	310	320	330	350	370	370	390	420	440	440	460	480	500	520	550	570	590	610	640	660	660
Metered (5.5%)	28	350	690	1,050	690	830	430	250	80	80	80	90	90	100	100	100	110	110	120	120	130	130	140	140	150	160	160	170	170	180	180
III-REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service - current year issue	$47	$650	$1,261	$1,915	$1,267	$1,504	$779	$459	$133	$139	$144	$150	$156	$162	$168	$175	$183	$189	$197	$205	$214	$222	$231	$240	$250	$260	$270	$281	$292	$304	$304
O&M expense	40	40	90	100	100	110	110	110	120	120	130	130	140	140	150	160	160	170	180	180	190	200	210	210	220	230	240	250	260	270	280
Additional revenue required - cumulative	87	757	2,047	3,972	5,239	6,753	7,532	7,991	8,134	8,273	8,427	8,577	8,742	8,905	9,083	9,268	9,451	9,650	9,857	10,062	10,286	10,518	10,759	10,999	11,259	11,529	11,809	12,100	12,402	12,716	13,050
Increase over current revenue	3.7%	31.6%	87.8%	170.3%	224.6%	289.5%	322.9%	342.5%	348.6%	354.6%	361.2%	367.6%	374.7%	381.7%	389.3%	397.3%	405.1%	413.6%	422.5%	431.3%	440.9%	450.9%	461.2%	471.5%	482.6%	494.2%	506.2%	518.7%	531.6%	545.1%	558.5%

(1) City revenue only -- excludes revenue from Regional Sanitation District fee

TABLE 21. SUMMARY OF ESTIMATED AND AFFORDABLE RATES FOR LEAST-COST ALTERNATIVE PLANS				
Alternative	Maximum Percent of EBI	Year Maximum Percent Occurs	Monthly Rates in Year Maximum Percent Occurs	
			Required for Project	EPA Threshold Rate
Baseline Rehabilitation Project	1.23	1999/2000	$38.72	$63.17
1. Separate Sanitary Sewers	1.56	2019/2020	$107.89	$138.00
2. Upsized CWCS Conveyance for 2-Year Flood Control and 5/Year CSO Control.	1.51	2016/2017	$92.31	$122.67
3. Upsized CWCS Conveyance and Storage/Treatment at Pioneer Site for 2-Year Flood Control and 1/Year CSO Control.	1.56	2016/2017	$95.63	$122.67
4. Tunnels for 5-year Flood Control and 5/Year CSO Control.	1.83	1999/2000	$57.85	$63.17
5. Tunnels for 5-Year Flood Control and 1/Year CSO Control.	1.84	1999/2000	$58.06	$63.17
6. Tunnels for 10-Year Flood Control and 5/Year CSO Control.	2.25	1999/2000	$70.97	$63.17
7. Tunnels for 10-year Flood Control and 1/Year CSO Control.	2.26	1999/2000	$71.23	$63.17
8. Tunnels for 25-Year Flood Control.	2.69	2000/2001	$88.48	$65.67

APPENDIX A

EXAMPLE CALCULATIONS FOR REVENUE AND AFFORDABILITY ANALYSES

EXAMPLE: ALTERNATIVE 1, SEPARATE SANITARY SEWER SYSTEM
27-YEAR CONSTRUCTION SCHEDULE
YEAR 1999/2000 CALCULATIONS

REVENUE ANALYSIS

PART 1 - CONSTRUCTION COSTS (ANNUAL) (x $1,000)

(1) 1992 Phased Dollars = $27,500 (input)
(2) ENR-SF Construction Cost Index = 8,193 (inflated @ 4%)
(3) Inflated Construction Cost = $\frac{8,193}{6,226}$ x $27,500 = $36,200

PART II - TOTAL MONTHLY REVENUE REQUIREMENTS (x $1,000)

(4) Current Revenue = $2,333 (input)
(5) Additional Capital Expense = $\frac{\$36,200}{12}$ = $3,000

(6) Additional O&M Expense = $110 (input)
(7) Additional Revenue Required = $3,110 (sum (5) + (6))
(8) Residential (74.4%) = .744 x $3,110 = $2,310
(9) Commercial (20.1%) = .201 x $3,110 = $630
(10) Metered (5.5%) = .055 x $3,110 = $170

PART III - REVENUE SUPPORTED BOND ISSUE
MONTHLY COSTS (x $1,000)

(11) Debt Service - current year issue = $\frac{\$101,000}{1,000}$ x $\frac{\$36,200 \div 12}{1,000}$ = $305

(12) O&M expense = $110 (from (6) above)
(13) Additional revenue required - cumulative =
$1,311* + $305 + $110 - $110* = $1,616
(*Values from preceding year, 1998/1999)

(14) Increase over current revenue = $\frac{1,616}{2,333}$ = 69.3%

PLAN AFFORDABILITY

(1) Household Effective Buying Income = $27,200 x $(1.04)^9$ = $37,900
(2) Affordable Residential Fee/Month = $37,900 x 0.02 ÷ 12 = $63.17

Affordability (Monthly) Assuming Revenue Supported Bond Issue

(3) Additional revenue required - cum. (x 1,000) = $1,616 (from Revenue Analysis, (13))

(4) Increase over current revenue = 69.3% (from Revenue Analysis, (14))

(5) Required residential fee - total =
($16.13* x 0.693 + ($24.28* x $(1.04)^7$) = $43.12
(*Current residential fees)

(6) Required fee as a percent of HEBI =

$$\frac{\$43.12 \times 12}{\$37,900} = 1.37\%$$

TABLE 22

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN REVENUE ANALYSIS

ALTERNATIVE NO.1 , SEPARATE SANITARY SEWERS (11 YEARS)

ASSUMPTIONS:

* Project Size (X $1000)	$381,397
* Additional O&M cost/yr. (X $1000)	1,000
* ENR SF Construction Index (3/92)	6226
* Inflation Rate	4.00%
* Debt service/year/million	$101,000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
I—CONSTRUCTION COSTS (ANNUAL) (x $1000)																															
1992 Phased Dollars	$3,100	$18,700	$13,700	$13,100	$13,800	$92,600	$62,100	$52,800	$51,900	$30,400	$29,200																				
ENR—S.F. Construction Index	6,475	6,734	7,003	7,284	7,575	7,878	8,193	8,521	8,862	9,216	9,585																				
Inflated Construction Cost	3224	20,200	15,400	15,300	16,800	117,200	81,700	72,300	73,900	45,000	45,000																				
II—TOTAL MONTHLY REVENUE REQUIREMENTS (x $1000)																															
Current revenue (1)	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333	2,333																				
Additional capital expense	269	1,700	1,300	1,300	1,400	9,800	6,800	6,000	6,200	3,800	3,800																				
Additional O&M expense	40	40	90	100	100	110	110	110	120	120	130																				
Additional revenue required – total	309	1,740	1,390	1,400	1,500	9,910	6,910	6,110	6,320	3,920	3,930																				
Residential (74.4%)	230	1,290	1,030	1,040	1,120	7,370	5,140	4,550	4,700	2,920	2,920																				
Commercial (20.1%)	62	350	280	280	300	1,990	1,390	1,230	1,270	790	790																				
Metered (5.5%)	17	100	80	80	80	550	380	340	350	220	220																				
III—REVENUE SUPPORTED BOND ISSUE																															
MONTHLY COSTS (X $1,000)																															
Debt service – current year issue	$27	$170	$130	$129	$141	$986	$688	$809	$622	$379	$379																				
O&M expense	40	40	90	100	100	110	110	110	120	120	130																				
Additional revenue required – cumulative	67	237	417	556	697	1,693	2,381	2,990	3,622	4,000	4,389																				
Increase over current revenue	2.9%	10.2%	17.9%	23.8%	29.9%	72.6%	102.1%	128.1%	155.2%	171.5%	188.1%																				

(1) City revenue only – excludes revenue from Regional Sanitation District fee

TABLE 23

CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT PLAN AFFORDABILITY

ALTERNATIVE NO.1, SEPARATE SANITARY SEWERS (11 YEARS)

ASSUMPTIONS:

* Project size (x $1000)	$381,397
* Average residential sewer fee – City	$7.07 /month
* Average residential storm fee – City	$9.06 /month
* Average residential system fees – City	$16.13 /month
* Average residential sewer fee – Regional	$8.15 /month
* Average residential fees – total (1)	$24.28 /month
* Household effective buying income (1990)	$27,200
* Inflation rate	4.00%
* EPA Hardship Threshold index/year	2.00%

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
	Financing Year																														
	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	2020/21	2021/22	2022/23	2023/24
HOUSEHOLD EFFECTIVE BUYING INCOME	$30,000	$31,200	$32,400	$33,700	$35,000	$36,400	$37,900	$39,400	$41,000	$42,600	$44,300																				
AFFORDABLE RESIDENTIAL FEE/MONTH (2)	$50.00	$52.00	$54.00	$56.17	$58.33	$60.67	$63.17	$65.67	$68.33	$71.00	$73.83																				
AFFORDABILITY (MONTHLY) ASSUMING REVENUE SUPPORTED BOND ISSUE																															
Additional revenue required – cum. (x $1,000) (3)	$67	$237	$417	$556	$697	$1,693	$2,381	$2,990	$3,622	$4,000	$4,389																				
Increase over current revenue (x $1,000) (3)	2.9%	10.2%	17.9%	23.8%	29.9%	72.6%	102.1%	128.1%	155.2%	171.5%	188.1%																				
Required residential fee – total (4)	$25.72	$27.90	$30.19	$32.25	$34.36	$42.43	$48.41	$53.90	$59.60	$63.60	$67.72																				
Required fee as a percent of HEBI	1.03%	1.07%	1.12%	1.15%	1.18%	1.40%	1.53%	1.64%	1.74%	1.79%	1.83%																				

(1) Includes rate increases for 1991/92

(2) EPA index times household effective buying income

(3) From Table 2

(4) Includes inflationary increase, and is the sum of fees for city sewer and storm drains, plus Regional Sanitation District.

APPENDIX P

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 7

CONSTRUCTION PHASING SCHEDULE FOR RECOMMENDED IMPROVEMENT PLAN

Greg Jones
HDR Engineering, Inc.

August 13, 1992

TECHNICAL MEMORANDUM
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN
PHASE 3, TASK 7

PREPARED BY: Greg Jones, HDR Engineering, Inc.
DATE: July 1, 1992
SUBJECT: Construction Phasing Schedule for Recommended Improvement Plan

A. INTRODUCTION

The recommended improvement plan identified in the Phase 3 technical memorandum entitled *Benefit/Cost Analysis*, is the Separate Sanitary Sewer Alternative. This plan involves constructing a new, Separate sanitary sewer system; the existing, combined sewer conveyance system would remain in place to serve as a stormwater-only drainage system. The total estimated cost for this alternative improvement plan is $381 million (1992 dollars), including contingencies and other related, non-capital improvement costs, such as engineering and administration. Also included is a program of rehabilitation oriented improvements for the existing pipes and pumping stations. Table 1 is a detailed cost estimate for the Separate Sanitary Sewer Alternative improvement plan.

Constructing a new sanitary sewer system was selected as the recommended improvement plan from the Phase 3 analysis, because it meets the key requirements of the Cease and Desist Order and the provisions of the NPDES permit for the Combined Wastewater Collection and Treatment System (CWCS). Sewer separation is the only alternative plan that will ensure that sewage outflows (combined sewage outflows to streets and structures during wet weather) are eliminated. The potential human health threat posed by outflows was the basis for the Regional Board imposing a Cease and Desist Order on the City. In addition, the Order and the renewed CWCS NPDES Permit require compliance with National and State CSO Strategies. Sewer separation would also eliminate CSOs, which is required by these strategies.

TABLE 1
ESTIMATED TOTAL PROJECT COSTS
SEPARATION ALTERNATIVE – SANITARY SEWER

ITEM	QUANTITY	UNITS	UNIT PRICE ($)	COST ($)
NEW PUMP STATION AT SUMP 2	110	MGD	$13,699,224	$13,699,224
MISC. PIPING CONNECTIONS	ALL REQ'D	LS	$500,000	$500,000
MISC. CROSSINGS	ALL REQ'D	LS	$500,000	$500,000
INSTALLED GRAVITY SEWER PIPE				
NEW 8 INCH SEWERS	8425	LF	$66.11	$556,977
NEW 10 INCH SEWERS	29385	LF	$79.85	$2,346,392
NEW 12 INCH SEWERS	25670	LF	$94.[illegible]7	$2,432,746
NEW 15 INCH SEWERS	36731	LF	$111.35	$4,089,997
NEW 18 INCH SEWERS	26865	LF	$129.35	$3,474,988
NEW 21 INCH SEWERS	10085	LF	$147.05	$1,482,999
NEW 24 INCH SEWERS	20220	LF	$209.41	$4,234,270
NEW 27 INCH SEWERS	13195	LF	$237.49	$3,133,681
NEW 30 INCH SEWERS	13105	LF	$269.03	$3,525,638
NEW 33 INCH SEWERS	9375	LF	$302.12	$2,832,375
NEW 36 INCH SEWERS	8050	LF	$341.67	$2,750,444
NEW 39 INCH SEWERS	8010	LF	$379.83	$3,042,438
NEW 42 INCH SEWERS	2430	LF	$416.47	$1,012,022
NEW 45 INCH SEWERS	5465	LF	$456.33	$2,493,843
NEW 54 INCH SEWERS	1700	LF	$570.94	$970,598
NEW 57 INCH SEWERS	4330	LF	$609.17	$2,637,706
NEW 60 INCH SEWERS	5770	LF	$649.83	$3,749,519
NEW 63 INCH SEWERS	800	LF	$688.11	$550,488
NEW 66 INCH SEWERS	3300	LF	$733.78	$2,421,474
NEW 69 INCH SEWERS	550	LF	$775.84	$426,712
NEW 72 INCH SEWERS	1600	LF	$809.46	$1,295,136
NEW 78 INCH SEWERS	6020	LF	$870.84	$5,242,457
NEW 84 INCH SEWERS	2250	LF	$949.92	$2,137,320
NEW 96 INCH SEWERS	1800	LF	$1,134.86	$2,042,748
NEW 102 INCH SEWERS	900	LF	$1,233.58	$1,110,222
NEW COLLECTION SYSTEM SEWERS (1)	ALL REQ'D	LS	$122,520,000	$122,520,000
SUBTOTAL				$197,212,414
SUMP 1, 1A AND 2 REHABILITATION,	ALL REQ'D	LS	$20,975,000	$20,975,000
LONG TERM				
PIONEER RESERVOIR IMPROVEMENTS	ALL REQ'D	LS	$762,600	$762,600
COLLECTION SYSTEM ROOT, GROUT	ALL REQ'D	LS	$20,607,000	$20,607,000
REHAB AND REPAIR				
MANHOLE REHABILITATION AND REPAIR	ALL REQ'D	LS	$7,716,000	$7,715,500
LATERAL/MAIN ROOT, GROUT	ALL REQ'D	LS	$7,199,000	$7,198,749
REHAB AND REPAIR				
NEW INSPECTION/MAINT. EQUIP	ALL REQ'D	LS	$1,500,000	$1,500,000
SUBTOTAL				$255,971,263
CONSTRUCTION CONTINGENCY @ 25%				$63,992,816
DESIGN @ 6%				$15,358,276
CONSTRUCTION INSPECTION @ 10%				$25,597,126
ADMINISTRATION, LEGAL @ 3%				$7,679,138
BONDS, INSURANCE, AND MISC. @ 5%				$12,798,563
ESTIMATED TOTAL ALTERNATIVE COST				$381,397,182

(1) NEW COLLECTION SYSTEM SEWERS ARE SEWERS NOT MODELED IN SWMM.

FILE: SS01$.WK1
DATE: JUNE 30, 1992

However, as stated in the technical memorandum entitled *Analytical Approach to Outflow Analysis* (dated February 13, 1992), accurately quantifying wet weather outflows (e.g. volume, frequency) from the combined sewer system is impractical. A relative comparison of alternative improvement plans with regard to outflow quantities therefore could not be completed for inclusion in the benefit/cost analysis for Phase 3 alternatives. By comparison, quantifying flooding within and CSOs from the combined system has been accomplished through extensive computer modeling efforts. Therefore, the selection of the recommended plan was based primarily on a qualitative evaluation and requirement for complying with the Regional Board's Order.

The cost for implementing the comprehensive, recommended improvement plan is substantial. Constructing a new sanitary sewer system, including mains, interceptors and collection system piping and service connections, is estimated to cost $197 million for capital improvements alone, not including contingencies. All of the benefits of this expenditure, i.e. existing outflows that would be eliminated, have not been quantified. Given this fact, the City is proposing to delay implementation of the capital improvements associated with sewer separation. This would allow the City to collect additional information on the occurrence of outflows through a monitoring program. In addition, the City would implement rehabilitation and reliability improvements to the conveyance system and major CWCS facilities during this period. Thus, the overall improvement plan would need to be phased to allow for a period of data collection and rehabilitation work prior to implementing sewer separation. At the conclusion of the data collection period, sewer separation, as well as other alternatives, could be re-evaluated from a benefit/cost standpoint.

To implement this improvement plan, it is anticipated that the work will be broken up into multiple contracts and spread over several years. This is essential to minimize disruptive impacts to City residents, including the interruption of vehicular traffic flow. Advanced planning to schedule overall project construction phasing is needed. This is useful for several reasons. It provides an estimate of annual capital improvement expenditures required for the project. In addition, project phasing scheduling can be useful in coordinating the proposed work with other planned projects within the study area.

This technical memorandum presents a recommended construction phasing schedule for the sanitary sewer separation alternative. The annual expenditure information developed from the phasing analysis was used in the overall project financial analysis, which is Task 8 of the Phase 3 study.

B. COMPONENTS OF THE RECOMMENDED PLAN

The recommended improvement plan involves the construction of a new sanitary sewer system for the entire combined sewer service area. As described in the Phase 3 technical memorandum entitled, *Improvement Plans*, this improvement plan was configured as a new gravity collection system draining the entire combined sewer service area to a new sanitary sewer pumping station located near Sump 2. Individual components of this improvement plan are discussed below.

B.1. New Separate Sanitary Sewer System

B.1.a. Sanitary Conveyance System--

New sanitary sewers were routed to parallel the existing combined sewer collection system where possible. This would facilitate the connection of existing sanitary sewer laterals to the new sewer system. In order to match the inverts of the service laterals, the new sewer was assumed to be laid at the same elevation or lower than the existing combined sewer. Adjustments to the existing sewer alignment were made to avoid conflict with existing major, large-diameter sewers.

B.1.b. New Sanitary Sewer Pumping Station--

All sewers drain by gravity flow to a new pumping station near Sump 2. The capacity of this station would be 110 mgd, based on estimates of peak sanitary wet weather flows. Average design flow through the pumping station was estimated to be 50 mgd with a daily peak of 108 mgd. These estimates of peak and average flows are conservative; current sanitary flows are known to be lower than the flows predicted in the model. The modeling analysis utilized diurnal curves that included a peaking factor from the City's *Design and Procedures Manual*, resulting in conservative (large) pipe and pumping station sizing.

B.2. Rehabilitation/Reliability Improvements

B.2.a. Rehabilitation of Sumps 1, 1A and 2--

The Phase 2 study recommended rehabilitation upgrades to the existing Sumps 1, 1A and 2. These upgrades are summarized in Table 2.

TABLE 2. RECOMMENDED LONG-TERM[1] IMPROVEMENTS TO SUMPS 1, 1A AND 2

Sump	Description[2]
1	Seismic modifications
1A	Seismic modifications Wet well expansion Addition of mechanical bar screens
2	Seismic modifications Corrosion control Flow Control Structure sluice gate modifications Conversion to hypochlorite chlorination Improve fire protection system Provide electrical power supply redundancy Add mechanical bar screens Rehabilitate wet well Upgrade/modify pumps and piping

1 Long-term is defined as improvements for approximately 25 to 50 years remaining life.
2 See Phase 2 technical memorandum entitled Evaluation of Reliability and Remaining Life at Existing CWCS Pumping Station Facilities, dated July 1, 1991.

B.2.b. Pioneer Reservoir Improvements--

In addition to the Phase 2 recommendations for Sumps 1, 1A and 2, improvements were also identified for the Pioneer Reservoir. The recommended improvements consist of monitoring the existing structural cracks in the 120-inch RCP outfall line to the river, replacing the 42-inch flushwater intake line, and adding to the capacity of the existing odor control equipment.

B.2.c. Rehabilitation of Existing Combined Sewers--

The Separate Sanitary Sewer Alternative would leave the existing combined sewers in place to serve as the stormwater-only drainage system. Based on a sewer televising program conducted during early 1991 and records available from the City, an evaluation was made of the structural condition of the combined system piping. It was concluded that a major root control and grouting program is needed, and rehabilitation of a significant portion of the system is required. These efforts are not presently included in the City's routine sewer maintenance program.

B.3. Sampling and Monitoring

During the next four to five years, the City will be continuing to sample street floodwaters and monitoring flows in order to determine the presence of sewage outflows onto the streets and to calibrate the hydraulic models for the system. The cost of this program is considered to be an operations and maintenance cost rather than a capital cost, and is therefore included in the financial analysis (Task 8 of the Phase 3 study).

C. CONSTRUCTION PHASING CONSIDERATIONS

Setting a schedule for the project involves determining the sequence of individual project components. This sequence is dictated by several issues, not the least of which is the need to provide additional justification for the new Separate sewer system. Other considerations include permitting, physical construction requirements, public safety and infrastructure protection considerations and the scheduling of other projects planned within the proximity of the project area. These issues are reviewed below.

C.1. Collection of Additional Data

As previously mentioned, the Regional Board's Cease and Desist Order states that the need to provide control of outflows is in the interest of protecting the public health and safety. However, the extent of the hazard created by the existing combined sewer system has not been quantified by extensive data

collection. This is the basis for deferring the design and construction of a new separate sanitary system until additional data can be collected to justify continuing with the project.

The phasing schedule proposed in this memorandum is therefore separated into two major phases. The first is the upgrade and rehabilitation of existing infrastructure. Concurrent with the rehabilitation work would be the collection of additional water quality and flow data over several years to confirm the need for the sewer separation work. The second phase is the construction of a new separate sanitary sewer system.

C.2. Preliminary Design and Permitting

The first items to be scheduled are the project preliminary design and permitting for the new facilities. Preliminary design would address the general layout of the entire project in sufficient detail to prepare an environmental impact report. Issues to be addressed would include noise, traffic control, air quality, construction dewatering, disposal of construction wastes, maintaining emergency services during the project, and other issues considered necessary according to the California Environmental Quality Act (CEQA).

C.3. Public Safety and Infrastructure Protection

The upgrade of Sumps 1, 1A and 2 and the Pioneer Reservoir upgrade are portions of the project that should be considered priority items from a public safety and infrastructure protection standpoint. Specifically, the seismic deficiencies and the gas chlorination system pose hazards to the City staff and the residents of the City. In addition, improvement of hydraulic capabilities at Sumps 1A and 2 also improves the ability of the CWCS to provide the service area with flood protection. For these reasons, these items of work should be scheduled early in the overall plan implementation.

C.4. Physical Construction Requirements

For the separated sewer project, the construction of the sewers should proceed initially from the downstream end of the collection system. This is preferred from a constructibility standpoint, and allows new connections to be placed into service immediately. To receive the sanitary flows contributed by the new sewers, the sanitary sewer pumping station near Sump 2 must be scheduled as one of the first components of the new sanitary sewer system completed.

C.5. Forthcoming Projects

Projects other than the sewer separation project that would be in progress concurrently with the sewer project could compound noise, traffic congestion, and other construction related problems. Scheduling the sewer project work around these other projects should be accomplished where possible to minimize the overall impact to City residents.

To identify upcoming projects in the study area, the City Planning Department was contacted and asked about their knowledge of future developments. Major projects they identified are discussed below.

C.5.a. Street Improvements/Light Rail Extension--

Major street and light rail projects identified to date are still in the planning stage. The City's *General Plan* identifies a need for another transportation corridor across the American River. The City Planning Department indicates this proposed corridor may serve vehicular traffic, the light rail or both. As of this time, the location of this corridor has not been identified. An overall downtown planning study is scheduled to begin later this year to identify improvements in traffic circulation within the City.

The *General Plan* calls for the light rail system to be extended to the south towards Elk Grove. The scheduling of this project has not yet been established. A preliminary routing for this extension is presented in the *General Plan*, which runs west out of Old Sacramento to the Sacramento River, then south along the river along the existing Southern Pacific Railroad right-of-way, out of the combined sewer service area.

C.5.b. Southern Pacific/Richards Boulevard Redevelopment Plan--
This project encompasses a portion of the existing combined sewer service area. The first ground breaking effort has not been scheduled. The City Planning Department estimates that construction will begin within the next 5 years, with subsequent efforts phased over an extended period. Development will include residential and commercial areas and areas set aside for parks.

C.5.c. Shriners Hospital Project--
A new Shriners Hospital is to be constructed at the UC Davis Medical Center. Construction is scheduled for completion no later than mid-1995.

C.5.d. Regional Interceptor Master Plan--
The County Regional Sanitation District is currently preparing a master plan for expanding its interceptor system. The routing of these new sewers could impact the layout and scheduling for the Separate Sanitary Sewer Alternative.

C.5.e. Summary of Forthcoming Projects--
The projects identified by the City Planning Department as being considered for the foreseeable future are: 1) the construction of a new additional transportation corridor across the American River; 2) extension of the light rail system south to Elk Grove; and 3) Southern Pacific/Richards Boulevard redevelopment.

Based on the major proposed projects within close proximity to the study area, no single project was identified as interfering with the construction of the new sanitary sewer work.

D. PROPOSED CONSTRUCTION PHASING

Tables 3, 4 and 5 are optional construction phasing schedules for the Separate Sanitary Sewer Alternative. The following discussion presents the assumptions used in preparing the data presented in these tables. The final choice of a recommended implementation schedule cannot be made however, until the City has an opportunity to evaluate financial factors, and other considerations.

TABLE 3. PROJECT PHASING ANNUAL BUDGET ESTIMATES
(THOUSANDS OF DOLLARS)
SEPARATION ALTERNATIVE, NEW SANITARY
27 YEAR PROJECT PERIOD

YEAR	DESIGN	NEW SANITARY PUMP STATION (4)	NEW SANITARY MAINS/ INTERCPT (1)	NEW SANITARY COLLECTION SYSTEM (2)	SUMPS 1, 1A &2 REHAB	PIONEER RESERVOIR IMPROV.	EXISTING COLLECTION SYSTEM & MH REHAB	NEW INSPECT. & MAINTENANCE EQUIPMENT	TOTAL ANNUAL BUDGET (3)
1	$976							$2,145	$3,121
2	$544				$11,358	$1,091	$5,740		$18,733
3	$522						$13,196		$13,718
4	$412						$12,658		$13,069
5	$2,387						$10,038		$12,426
6	$1,121	$21,020	$9,532	$7,964	$9,318		$9,163		$58,118
7	$730		$9,532	$7,964	$9,318				$27,544
8	$730		$9,532	$7,964					$18,226
9	$730		$9,532	$7,964					$18,226
10	$730		$9,532	$7,964					$18,226
11	$730		$9,532	$7,964					$18,226
12	$730		$9,532	$7,964					$18,226
13	$730		$9,532	$7,964					$18,226
14	$330		$9,532	$7,964					$17,826
15	$330			$7,964					$8,294
16	$330			$7,964					$8,294
17	$330			$7,964					$8,294
18	$330			$7,964					$8,294
19	$330			$7,964					$8,294
20	$330			$7,964					$8,294
21	$330			$7,964					$8,294
22	$330			$7,964					$8,294
23	$330			$7,964					$8,294
24	$330			$7,964					$8,294
25	$330			$7,964					$8,294
26	$330			$7,964					$8,294
27				$7,964					$7,964
TOTAL	$15,358	$21,020	$85,790	$175,204	$29,994	$1,091	$50,795	$2,145	$381,397

NOTES:
(1) ASSUMES CONSTRUCTION OVER 9 YEARS
(2) ASSUMES WORK SPREAD OVER 22 YEARS
(3) 1992 DOLLARS
(4) INCLUDES MISCELLANEOUS CONNECTIONS AND CROSSINGS

TABLE 4. PROJECT PHASING ANNUAL BUDGET ESTIMATES
(THOUSANDS OF DOLLARS)
SEPARATION ALTERNATIVE, NEW SANITARY
15 YEAR PROJECT PERIOD

YEAR	DESIGN	NEW SANITARY PUMP STATION (4)	NEW SANITARY MAINS/ INTERCPT (1)	NEW SANITARY COLLECTION SYSTEM (2)	SUMPS 1, 1A &2 REHAB	PIONEER RESERVOIR IMPROV.	EXISTING COLLECTION SYSTEM & MH REHAB	NEW INSPECT. & MAINTENANCE EQUIPMENT	TOTAL ANNUAL BUDGET (3)
1	$976							$2,145	$3,121
2	$544				$11,358	$1,091	$5,740		$18,733
3	$522						$13,196		$13,718
4	$412						$12,658		$13,069
5	$1,648						$10,038		$11,686
6	$1,581	$21,020			$9,318		$9,163		$41,083
7	$1,190		$28,597		$9,318				$39,105
8	$1,190		$28,597						$29,787
9	$1,216		$28,597						$29,813
10	$1,216			$29,201					$30,416
11	$1,216			$29,201					$30,416
12	$1,216			$29,201					$30,416
13	$1,216			$29,201					$30,416
14	$1,216			$29,201					$30,416
15				$29,201					$29,201
TOTAL	$15,358	$21,020	$85,790	$175,204	$29,994	$1,091	$50,795	$2,145	$381,397

NOTES:
(1) ASSUMES CONSTRUCTION OVER 3 YEARS
(2) ASSUMES WORK SPREAD OVER 6 YEARS
(3) 1992 DOLLARS
(4) INCLUDES MISCELLANEOUS CONNECTIONS AND CROSSINGS

TABLE 5. PROJECT PHASING ANNUAL BUDGET ESTIMATES
(THOUSANDS OF DOLLARS)
SEPARATION ALTERNATIVE, NEW SANITARY
11 YEAR PROJECT PERIOD

YEAR	DESIGN	NEW SANITARY PUMP STATION (4)	NEW SANITARY MAINS/ INTERCPT (1)	NEW SANITARY COLLECTION SYSTEM (2)	SUMPS 1, 1A &2 REHAB	PIONEER RESERVOIR IMPROV.	EXISTING COLLECTION SYSTEM & MH REHAB	NEW INSPECT. & MAINTENANCE EQUIPMENT	TOTAL ANNUAL BUDGET (3)
1	$976							$2,145	$3,121
2	$544				$11,358	$1,091	$5,740		$18,733
3	$522						$13,196		$13,718
4	$412						$12,658		$13,069
5	$3,767						$10,038		$13,805
6	$2,500	$21,020	$21,448	$29,201	$9,318		$9,163		$92,650
7	$2,109		$21,448	$29,201	$9,318				$62,076
8	$2,109		$21,448	$29,201					$52,758
9	$1,209		$21,448	$29,201					$51,858
10	$1,209			$29,201					$30,410
11				$29,201					$29,201
TOTAL	$15,358	$21,020	$85,790	$175,204	$29,994	$1,091	$50,795	$2,145	$381,397

NOTES:
(1) ASSUMES CONSTRUCTION OVER 4 YEARS
(2) ASSUMES WORK SPREAD OVER 6 YEARS
(3) 1992 DOLLARS
(4) INCLUDES MISCELLANEOUS CONNECTIONS AND CROSSINGS

D.1. GENERAL COMMENTS

The three construction phasing options presented in Tables 3, 4 and 5 each have a different total project term consisting of 27, 15 and 11 years, respectively. The 27-year option is based on the premise that one sewer contract would be active per year within the study area. The 15-year option assumes that up to 4 contracts would be active within the study area for much of the overall project term. This may mean that one contractor would be active in each of the downtown quadrants during a given construction season. The 11-year option assumes that between 4 and 6 contractors would be active simultaneously.

The 27-year schedule represents a moderate rate of construction in which there would be at least one construction project scheduled every year throughout the 27-year period. The primary assumptions used in this alternative are that average installation rates are 200 feet per day for small diameter sewers and 120 feet per day for larger sanitary mains and interceptors. These are acceptable rates for collection sewers within the downtown area. This volume of work could be handled by one contractor.

The 15-year schedule is fairly aggressive. There would be multiple construction projects scheduled every year in this option. The basic difference between this schedule and the 27-year schedule is the assumption that four contractors or construction crews would be at work in different areas of the City. In this case, the sewer installation rates are approximately 480 and 800 feet per day for large and small sewers, respectively. The effects of this change are: 1) to reduce the length of the new sanitary sewer construction from 22 years to 6 years; and 2) to reduce the length of construction for mains and interceptors from 9 years to 3 years. However, consideration is also given to equalizing the annual construction costs by delaying the start of collection system construction until after the mains and interceptors are in place. This annual volume of work could be split up into multiple construction contracts. This schedule would require a concerted management effort on behalf of the City.

The 11-year schedule is very aggressive and represents the minimum feasible time schedule in which the project could be completed. The sewer construction rate

is assumed to be the same as for the 15-year schedule. However, the construction of both collection sewers and the larger mains and interceptors would begin together, thereby increasing the number of multiple construction contracts underway simultaneously. This schedule would require an intense management effort on behalf of the City. It would also result in the greatest simultaneous disruption to City streets during sewer installation.

The general advantages and disadvantages of reducing the project term length from 27 to a shorter period such as 15 or 11 years are summarized in Table 6.

TABLE 6. ADVANTAGES AND DISADVANTAGES OF REDUCED PROJECT TERM LENGTHS

Advantages	Disadvantages
Sanitary protection afforded by separated sewer system available to public within shorter time frame.	Higher annual project costs with greater increases to utility rates.
Lower overall cost due to less inflation impacts and lower interest costs.	More widespread obstruction/disruption to vehicle and pedestrian traffic within study area.
	More intensive construction contract administrative effort required by City.

D.2. Non-Construction Project Costs

Table 1 presented the estimated total project cost for the separation alternative. At the bottom of this table are non-construction project costs. These include construction contingencies, design, construction inspection, project administration and legal fees and bonds, insurance and miscellaneous expenses.

Of these non-construction costs, the design fees were estimated to be 6 percent of the total construction costs. These design fees are included under the column heading "Design" in Tables 3, 4 and 5. The other non-construction costs are

distributed over the non-design cost data in the tables. For example, the costs shown in the "New Sanitary Pumping Station" include both construction costs and the non-construction costs listed above, but exclude the design costs.

The design costs include a project permitting expense for compliance with the CEQA. It is assumed that the initial permitting study would find no negative impacts, and that a negative declaration would be filed for the rehabilitation of the existing sewers, the Pioneer Reservoir improvements and the rehabilitation of Sumps 1, 1A and 2 as isolated projects. Permitting for the new separated sewer work would also require an initial study under CEQA, which would be completed in the year prior to the initiation of project design for the separated sewer system.

D.3. Rehabilitation of Sumps 1, 1A and 2

As outlined in Table 2, the recommended upgrades to Sumps 1 and 1A included seismic modifications to both pumping stations, and wet well modifications and the addition of mechanical bar screens to Sump 1A. These modifications can all be initiated early in the overall project phasing assuming there are no CEQA permitting requirements for this work.

For Sump 2 the recommended rehabilitation program is more extensive. This work would be split into two distinct projects. The division of work to be done in each of the projects is outlined in Table 7.

This division of work for the rehabilitation of Sump 2 has been proposed to avoid having the site subjected to major heavy construction activity more than once. The construction of the new sanitary sewage pumping station would be sited at or near Sump 2, and is proposed for construction following the collection of additional field data to verify the need for the overall sewer separation project. It is suggested that the major rehabilitation work for Sump 2 begin coincident with the construction of this sanitary sewage pumping station.

TABLE 7. RECOMMENDED PROJECT PHASING FOR SUMP 2 REHABILITATION	
Project Sequencing	**Description of Project Work**
Initial Phase	Seismic Modifications Corrosion Control Flow Control Structure Sluice Gate Modifications Conversion to Hypochlorite Chlorination Improve Fire Protection System Install Additional Odor Control Facilities Provide Electrical Power Supply Redundancy
Follow-up Phase	Add Mechanical Bar Screens Modify/Upgrade Wetwell Pumps and Piping

D.4. Pioneer Reservoir Improvements

The Pioneer Reservoir improvements (i.e. monitoring the 120-inch outfall line for structural integrity, replacement of the 42-inch flushwater line and provision of additional odor control facilities) are scheduled for construction in the second year of the overall program for all three project term options (i.e. the 27-, 15- and 11-year options). This work would begin after the completion of design efforts in the first year.

D.5. Existing Sewer Rehabilitation

The sewer rehabilitation program costs were generated based on the evaluation of the existing collection system as outlined in the Phase 2 technical memorandum entitled *Condition Evaluation of Combined Sewer Conveyance System*. Estimates of cost and scheduling for the sewer rehabilitation program are summarized in Table 8.

D.6. New Sanitary Sewers

Tables 3, 4 and 5 all include two columns for new sanitary sewers, one for "Mains/Interceptors" and the other for the local "Collection System." These are discussed below.

TABLE 8. SEWER REHABILITATION PHASING AND COSTS		
Year	Description of Work	Cost[1]
1[2]	A. Purchase sewer inspection equipment B. Complete 1/3 of total root control program for 6- to 16-inch sewers (i.e. collection system) C. Complete 1/2 of root control in 18- to 36-inch sewers (lateral/main system) D. Complete 1/6 of grouting and testing for collection system E. Complete 1/4 of grouting and testing for laterals/mains F. Complete 1/10 of lateral/main rehabilitation G. 1/5 of manhole rehabilitation	$4,014,000
2	A. 1/3 of root control for collection system B. 1/2 of root control for lateral/main system C. 1/3 of grouting and testing for collection system D. 1/2 of grouting and testing for laterals/mains E. 1/4 of collection system rehabilitation F. 1/5 of lateral/main rehabilitation G. 1/5 of manhole rehabilitation	$9,612,000
3	A. 1/3 of root control for collection system B. 1/3 of grouting and testing for collection system C. 1/4 of grouting and testing for laterals/mains D. 1/4 of collection system rehabilitation E. 1/5 of lateral/main rehabilitation F. 1/5 of manhole rehabilitation	$8,851,000
4	A. 1/6 of grouting and testing for collection system B. 1/4 of collection system rehabilitation C. 1/5 of lateral/main rehabilitation D. 1/5 of manhole rehabilitation	$ 7,020,000
5	A. 1/4 of collection system rehabilitation B. 3/10 of lateral/main rehabilitation C. 1/5 of manhole rehabilitation	$6,408,000

1 1992 dollars, excluding contingencies, engineering, legal and administrative costs.
2 Corresponds to fiscal year 1992/93.

D.6.a. New Sanitary Mains/Interceptors--

The costs identified as "New Sanitary Mains/Interceptors" in Tables 3, 4 and 5 represent the costs for constructing new separated sewers which were modeled in the Sacramento SWMM (SSWMM91). The terms "laterals/mains" and "interceptors" are

typically used to represent larger sewers. In the case of the separation alternative, the new sanitary sewers modeled in the SSWMM91 ranged down to 8-inch diameter conduits. This nomenclature was used because of the procedure used to model the new sanitary sewer system. The conduits listed in the SSWMM91 input data file representing the existing combined sewer collection system (consisting of mains and interceptors) were down-sized as required to convey only sanitary flows. Consequently the conduits in the new sanitary sewer model are referred to as mains and interceptors to be consistent.

As indicated in Tables 3, 4 and 5, construction of these "Mains and Interceptors" has been scheduled in the initial years of the new sewer construction program. These sewers include the larger downstream sewers in the collection system and must necessarily be completed first prior to the system being placed into service. Scheduling for these sewers was based on a 10-month construction season, with production rates of 120 linear feet per day per contractor or construction crew. This rate does not include the final setting of manhole frames, paving or clean-up. The total lineal footage of new sewers modeled in the SSWMM91 is approximately 246,000 feet. Depending upon the number of contractors or construction crews active in the study area, this work can be accomplished in 3 to 9 years.

D.6.b. New Sanitary Collection System--

The costs in Tables 3, 4 and 5 for the "New Sanitary Collection System" represent the costs for constructing new separate sanitary sewers that are not modeled in the SSWMM91. The assumptions used in estimating these costs are discussed below.

The total cost of the connections was estimated as the product of an estimated 24,359 service connections and a unit cost of $2450 per connection. The total footage of collection system piping to be installed (not modeled in SSWMM91) is 951,000 feet. This is the total footage of existing collection system piping, excluding the sanitary-only backyard easement pipes. These estimates of service area acreage, service connection density, and length of collection system piping are documented in the technical memorandum entitled *Collection System Hydraulic Evaluation, Improvements and Costs.* The new collection system sewers were assumed to be 8-inch sewer at a unit cost of $66.11 per foot. This new 8-inch

pipe plus the service lateral connections amounted to a total of $122.5 million, without non-construction project costs.

The proposed schedule for constructing these new collection system sewers was based on the following. Construction was assumed to be distributed over 6 to 22 years, depending upon how many contractors or construction crews were active within the study area at the same time. As with the "Mains and Interceptors", the "New Sanitary Collection System" work was assumed to occur within a 10-month construction period each year. An average production rate of approximately 200 feet per day was assumed per construction contractor, which is a typical rate for small-diameter, shallow-bury pipeline work within the downtown area.

D.6.c. New Sanitary Sewer Pumping Station--

The new sanitary pumping station construction must also be initiated at the beginning of work on the new sanitary conveyance system. It was assumed that this project would be completed within a year, although site construction limitations may require extension of this time estimate.

E. SUMMARY

The Separate Sanitary Sewer Alternative includes three components: 1) rehabilitation of the existing combined wastewater collection system, 2) monitoring and data collection focused on determining the severity and extent of the outflow problem, and 3) the construction of the new sanitary sewer system.

- Rehabilitation work will include a major root control and grouting program along with other repairs to existing sewers, plus improvements at Pioneer Reservoir and Sumps 1, 1A, and 2.

- The monitoring and data collection program would be conducted over the initial 4 to 5 years of the project, concurrent with rehabilitation work, and would include sampling street floodwaters and monitoring flows to determine the presence of sewage outflows onto the streets.

- The new sanitary sewer system was sized for this study based on estimates of sanitary flow rates converted into a diurnal curve for use in the SSWMM91. The system generally parallels the alignment of the existing combined sewer system, and conveys wastewater to the Sump 2 site from which is pumped to the SRWTP. Preliminary quantity estimates include over 1 million linear feet of new sanitary sewer ranging in size from 8 inches to 102 inches.

The construction phasing analysis included consideration of preliminary design and permitting activities, physical construction requirements, and public safety and infrastructure protection. The schedules for several other specific construction projects within and close to the study area were reviewed, and no single project was seen as slowing the progress of the new sanitary sewer system.

Three optional project phasing schedules with estimated total annual costs were prepared for the Separate Sanitary Sewer Alternative plan. A decision on the final implementation schedule however, must be based on financial considerations and other information not included in this study. One assumption used in preparing all these schedules is that the City will defer the construction of a new sanitary conveyance system for a few years. During this time, water quality and flow data will be collected to confirm the need (i.e., outflow control) for the project. Rehabilitation and repair of the existing pumping stations, Pioneer Reservoir and the combined sewer conveyance system are scheduled concurrently with this data collection and analysis period. If the decision is made to go ahead with the new sanitary sewer system, the scheduled work would begin with the sanitary sewer pumping station and larger gravity sewers. To avoid staging two major construction projects within the vicinity of Sump 2 within a 5-year period, the major earthwork and structural modifications to Sump 2 would be deferred until the construction of the new sanitary sewer pumping station near Sump 2. The total length of the overall construction period would be dependent upon the number of contractors that can work simultaneously within the study area.

APPENDIX Q

CITY OF SACRAMENTO

COMBINED SEWER SYSTEM IMPROVEMENT AND IMPLEMENTATION PLAN

TECHNICAL MEMORANDUM

PHASE 3, TASK 8

REVENUE PROGRAM FOR RECOMMENDED IMPROVEMENT PLAN

Kent South
Bartle Wells Associates

August 13, 1992

TECHNICAL MEMORANDUM
CITY OF SACRAMENTO COMBINED SEWER IMPROVEMENT AND IMPLEMENTATION PLAN
PHASE 3, TASK 8

PREPARED BY: Kent South, Bartle Wells Associates
DATE: July 1, 1992
SUBJECT: Revenue Program for the Recommended Improvement Plan

A. INTRODUCTION

Least-cost alternative improvement projects for the Combined Wastewater Collection and Treatment System (CWCS) were determined in the Task 6 technical memorandum (TM) entitled *Improvement Plans*. In this TM, cost estimates were also prepared for a Baseline Rehabilitation Project. The alternative projects were also evaluated with regard to their affordability in the Task 8 TM entitled *Rate Assessment for Least-Cost Alternative Plans*, which assumed the use of 20-year revenue bond financing, no new revenue sources, and a generalized increase in City-wide residential sewer fees as an indicator of rate impacts. Finally, a TM titled *Benefit/Cost Analysis for the Phase 3 Study* was prepared in Task 7. From these analyses and cost estimates the City selected the Separate Sanitary Sewer Alternative as the recommended plan. The estimated cost of this alternative is $381.4 million (1992 dollars).

Three alternative construction schedules for the recommended plan are described in the Task 7 TM titled *Construction Phasing Schedule for the Recommended Improvement Plan*. These include a 27-year construction period, a 15-year period, and an 11-year period. Revenue requirements for the 27-year and 15-year periods were examined as part of the work in this TM, but not the 11-year period. The 11-year period was eliminated from further financial analyses due to its distinct disadvantages, not only in terms of affordability, but also because it would involve the greatest simultaneous disruption to City streets during sanitary sewer installation, and would severely restrict the City's flexibility to respond to future changes in regulatory or financial conditions.

This TM describes the results of work performed in Subtask 8.2, and includes a review of existing sewer system resources and rate base. It also examines and recommends other non-user revenue sources to offset or mitigate the impact of recommended plan capital costs on existing users, specifically:

- State Revolving Loan funds (availability and application process)
- Connection charges

The user fee impact of the recommended plan is projected through the year of maximum cumulative debt service cost. For the 27-year construction phasing schedule, maximum cumulative debt service cost and rate impact is projected to occur in the year 2016/17. For the 15-year construction schedule, the comparable date is 2004/05. Subsequent to that time, rate increases would be limited to those due to inflation and new capital projects beyond the combined sewer improvement plan. The projection assumes use of revenue bonds to finance the project. Debt service costs included in the projection were estimated by Bartle Wells Associates based on schedules developed by a financing team working for the City Treasurer's Office. Increased user fee projections are included for four rate allocation options, namely:

1) **Option 1:** Increase rates equally for all sanitary sewer and storm drainage customers in the City.

2) **Option 2:** Increase rates for combined service area customers only, both sanitary sewer and storm drainage. Increase rates for other users for 4-percent inflationary factor only.

3) **Option 3:** Increase sanitary sewer rates only (entire City). Increase rates for other users for 4-percent inflationary factor only.

4) **Option 4:** Increase rates for all Sacramento Regional customers (including City customers) rather than increasing City user rates.

Finally, user fees are compared with fees charged by comparable agencies.

B. EXISTING SEWER FEES

Table 1 shows the City's 1991/92 monthly user fee schedule for both sanitary sewer and storm drainage accounts. It includes fee increases adopted to implement the 1991/92 budget, as follows:

- Sewer fees - 15% increase over 1990/91 rates for residential
- Storm drainage fees - 7% increase over 1990/91 for residential

Table 2 details total monthly sewer rates paid by a typical single-family residence (i.e., 6-7 rooms) in the City area. No separate fee was calculated or assessed for combined system users.

The City does not charge a connection charge for new development for storm drainage service to recover its cost of existing infrastructure or the cost of future improvements allocated to growth. It does charge a small system development fee for sanitary sewer connections.

C. 1991/92 SEWER BUDGET

Table 3 shows budgeted revenues, expenses, and fund balances for both sewer and storm drainage enterprise funds for 1991/92. Charges for services (i.e., user fees) are estimated to total $28.0 million and provide 84.0 percent of total enterprise revenue. The budget shows that approximately $1.1 million of fund balance will be required to provide for estimated operating and capital improvement expenses, leaving a fund balance of $1.2 million, or approximately 4.4 percent of annual operation and maintenance (O&M) costs at June 30, 1992. It is recommended that future City budgets and projections of revenues and expenses include a minimum $1.0 million fund balance for contingencies.

The 1991/92 budget is used as a base for projecting future revenues, expenses, and required rate increases.

TABLE 1. SEWER SERVICE AND STORM DRAINAGE MONTHLY FEES, 1991/92		
Monthly Fees[1]	Sewer Service	Storm Drainage
Residential[2]		
1-3 rooms	$4.64	$6.02
4-5 rooms	$5.89	$7.67
6-7 rooms[3]	$7.07	$9.06
8-9 rooms	$8.18	$10.70
10-15 rooms	$9.38	$12.20
Over 15 rooms, each additional room	$0.66	$0.75
Commercial[4]		
Minimum	$5.75	$15.43
Per 10,000 sq.ft. service area	n/a	$6.71
Per 1,000 sq.ft. floor area	--[5]	--
Hotel or motel, per room	$1.09	--
Rest home, per person	$1.75	--
School, per 100 pupils	$9.73	--
Service stations & car lots	--[6]	--
Metered Sewer Use		
5/8" x 3/4" - 1,200 cf maximum	$6.89	--
3/4" - 1,700 cf maximum	$6.89	--
1" - 2,100 cf maximum	$7.46	--
1-1/2" - 3,700 cf maximum	$12.74	--
2" - 6,200 cf maximum	$21.29	--
3" - 12,500 cf maximum	$42.47	--
4" - 21,800 cf maximum	$74.37	--
6" - 50,000 cf maximum	$148.65	--
8" - 106,200 cf maximum	$265.38	--
10" - 168,700 cf maximum	$371.66	--
12" - 262,500 cf maximum	$530.93	--
Per 100 cf over maximum	$0.29	--

1 Does not include Sacramento Regional fees ($8.15 for residential).
2 Single-family and each multiple dwelling unit.
3 Typical residential use.
4 Does not include special sewer rates.
5 Varies by category from $0.97 to $21.30.
6 Varies by meter size.
Source: City rate ordinance.

TABLE 2. MONTHLY SEWER RATES PAID BY SINGLE-FAMILY RESIDENCE	
Sanitary sewer	$7.07
Storm drainage	$9.06
Total City rate	$16.13
Sacramento Regional (not included in Table 1)	$8.15
Total user rate	$24.28

TABLE 3. 1991/92 BUDGET (x $1000)			
	Sewer	Storm Drain	Total
Beginning fund balance	$655	$1,698	$2,353
Revenues			
Charges for services	$8,811	$19,189	$28,000
Use of money	$215	$2,320	$2,535
Intergovernmental	$1,415	$628	$2,043
Miscellaneous	$510	$265	$775
Total revenues	$10,951	$22,402	$33,353
Operating Expenses			
Employee services	$3,615	$6,787	$10,402
Services & supplies	$3,483	$10,933	$14,416
Equipment	$302	$709	$1,011
Debt service	$575	$1,189	$1,764
Total expenses	$7,975	$19,618	$27,593
Net operating revenue (cost)	$2,976	$2,784	$5,760
Capital improvements	($1,834)	($4,898)	($6,732)
Transfers in	$50	$1,653	$1,703
Transfers out	($1,483)	($394)	($1,877)
Net change in fund balance	($291)	($855)	($1,146)
Ending fund balance	$364	$843	$1,207

Source: City budget, 1991/92

D. CUSTOMER RATE BASE/REVENUE ESTIMATES

D.1. Account History and Growth

Table 4 shows the number of sanitary sewer and storm drainage accounts for the most recent four-year period based on City billing records. The number of storm drainage accounts exceeded sewer accounts by approximately 36,000, or 49 percent, in September 1991. This difference is attributable primarily to two factors:

- Use of septic tanks
- City customers connected to Sacramento Regional collector system (particularly in areas annexed to the City or new-growth areas)

Table 4 also shows that the average annual growth rate for City accounts in each enterprise amounts to the following:

Sanitary sewer	0.88%
Storm drainage	2.61%

Table 5 shows the history of equivalent single-family dwellings (ESDs) for Sacramento Regional service areas during the period 1984 through 1990, as well as the average annual growth. The average annual growth of 0.89 percent shown for the City of Sacramento compares closely with the growth rate of 0.88 percent based on City billings records shown in Table 4. Sanitary sewer system growth within City boundaries but not connected to the City system (see second growth factor above) occurs in County Sanitation District No. 1, and is shown separately in Table 5. The average annual growth for the total Regional system of 2.84 percent compares closely with the storm drainage account growth of 2.61 percent, shown in Table 4.

TABLE 4. NUMBER OF ACCOUNTS HISTORY FOR CITY AREA		
Date	Sewer Accounts[1]	Storm Drainage Accounts[2]
June 1987	71,386	98,781
September 1988	71,989	100,878
December 1989	72,791	107,787
September 1990	73,661	108,828
September 1991	74,093	110,206
Average annual increase[3]	0.88%	2.61%

1 City accounts only.
2 Includes City residents billed by Sacramento Regional CSD and City residents on septic tanks.
3 4½-year period.

TABLE 5. HISTORY OF EQUIVALENT SINGLE-FAMILY DWELLINGS AND USER RATES					
	Equivalent Single-Family Dwellings (ESD)				Residential Service Charge (per mo.)
Year	City of Sacramento	CSD No. 1[1]	City of Folsom	Total System	
1984	102,598	225,102	6,450	334,150	$5.30
1985	103,444	231,083	6,625	341,152	$5.30
1986	105,000	237,678	7,700	350,378	$5.30
1987	105,988	251,895	8,525	366,408	$7.30
1988	106,100	261,173	9,700	376,973	$7.35
1989	106,500	273,038	10,000	389,538	$7.40
1990	108,181	275,954	11,100	395,235	$7.45
Average annual increase	0.89%	3.45%	9.47%	2.84%	5.84%

1 Includes many development projects located in the City. However, collection system design results in service provision by CSD No. 1. As a result, much of the growth in the City is occurring in service areas of CSD No. 1.

Source: Sacramento Regional CSD, 1990 Annual Report.

For purposes of future revenue projections, it is recommended that a conservative annual growth estimate be used, as follows:

City Combined Sewer System:	0.0%
City Sanitary Sewer System:	0.7%
City Storm Drainage System:	2.0%
County Sanitation District No. 1:	2.0%

D.2. Allocation of Accounts and Revenue by User Group

Table 6 shows the allocation of customer accounts and revenues by user group based on billing reports for September and December 1991. Over 74 percent of user fee revenue is derived from residential users. This estimated allocation is slightly higher but consistent with the 72-percent estimate presented in the *Phase 2 Detailed Technical Report*. This high residential rate base is consistent with findings of the Association of Metropolitan Sewage Agencies in its *1990 Financial Survey*. Future fee increases will similarly be allocated largely to residential users.

TABLE 6. SERVICE CHARGE REVENUE ANALYSIS - SEPTEMBER & DECEMBER 1991

User Group	Sewer Service		Storm Drainage		Total Revenue	Per-cent
	Accounts	Revenue	Accounts	Revenue		
Residential	69,760	$1,154,579	103,922	$2,404,903	$3,559,482	74.4%
Commercial	2,606	$84,761	5,552	$875,088	$959,849	20.1%
Metered	1,329	$266,552	--	--	$266,552	5.5%
Total	73,695	$1,505,892	110,474	$3,279,991	$4,785,883	100.0%

Source: Based on September and December 1991 billing reports.

D.3. Estimated City Equivalent Dwelling Units (EDUs)

For purposes of estimating future rate increases it is easier to use an equivalent dwelling unit (EDU) rate, which is essentially equivalent to the Regional Sanitation District's ESD, rather than separate rate calculations for each rate category. For example, the City's fee ordinance contains over 50 sanitary and 15 storm drainage customer categories. Appendix A contains a customer data base model with a history of customers in each category over the period December 1989 through September 1991. This model also shows estimated monthly revenue based on September 1991 billing records.

Table 7 shows the calculation to derive EDUs in the City based on the rate for a typical single-family user. In summary, it shows:

Sewer EDUs	103,854
Storm drainage EDUs	176,500

EDUs exceed the number of accounts because the average commercial and metered account pays more than an average single-family residence. The estimated 103,854 sewer EDUs compares closely with Sacramento Regional's 1990 estimate of 108,181 EDUs for the City area, shown in Table 5.

TABLE 7. ESTIMATED EQUIVALENT DWELLING UNITS

	Sewer	Storm Drainage
Estimated revenue, 1991/92	$8,811,000	$19,189,000
Typical single-family user rate[1]	$7.07	$9.06
Number of accounts (September 1991)	74,093	110,203
Equivalent dwelling units[2]	103,854	176,500

1 6-7 rooms.
2 Revenue per month (line 1 ÷ 12) ÷ typical user rate (line 2).
Source: 1991/92 approved budget.

D.4. Sacramento Regional ESDs

Rate Option 4 allocates capital costs to all Sacramento Regional users rather than to City users. Sacramento Regional estimates 450,000 ESDs in 1991/92.

E. ESTIMATED COMBINED SEWER SYSTEM ACCOUNTS

To facilitate a rate impact projection applied only to the combined sewer service area, it is necessary to estimate accounts and EDUs applicable to that area (see Rate Option 2 in Section A). The City completed an estimate of accounts by comparing assessor's parcel numbers with billing account records for the affected area with the following results:

Combined system area only (storm and sanitary): 24,359 accounts
Separated area contributing sanitary to combined system: 9,375 accounts

For purposes of evaluating Rate Option 2 for the recommended plan, accounts in the combined system area only will be separately identified. The rationale for application of a separate rate to such accounts is that the combined flow of both storm and sanitary is a primary cause of outflows and overflows from the system. That rationale would not apply to the separated area, since only sanitary flow is contributed to the collector system.

Table 8 shows that approximately 31 percent of total City sanitary sewer accounts are within the combined system area or approximately 31,900 EDUs.

TABLE 8. ESTIMATED COMBINED SEWER USER ACCOUNTS/EDUs

	Estimated Combined System	Other	Total
Accounts	24,359	54,974	79,333
Percent	30.7%	69.3%	100.0%
EDUs	31,883	71,971	103,854[1]

1 See Table 7.
Source: City Department of Data Management Analysis (April 13, 1992).

F. RECOMMENDED IMPROVEMENT PLAN

The City selected the Separate Sanitary Sewer Alternative as the recommended plan. The total estimated cost for this alternative improvement plan is $381.4 million (1992 dollars). Two construction phasing schedules for this alternative are contained in the TM titled *Construction Phasing Schedule for Recommended Improvement Plan.*

Additional O&M expenses are estimated at $1.0 million per year and are phased in over a three-year period beginning in 1993/94. Also, during the first five years, the City will continue to sample street floodwaters and monitor flows to determine sewage outflows and calibrate hydraulic models for the system. The $400,000 estimated annual cost of this system is considered to be an O&M cost, and is added to the $1.0 million annual O&M costs cited above for purposes of rate impact analysis.

F.1. 27-Year Construction Phasing

Table 9 shows inflated construction costs, bond issue sizes, and estimated debt service costs resulting from a 27-year construction phasing schedule. The bond program assumes 25-year revenue bonds sold at an average interest cost of 7.5 percent, with 5-percent earnings on construction and reserve funds. It shows a total bond program of $647.7 million and maximum debt service of $60.0 million in the 25th year.

Table 10 is a detailed schedule of debt service showing nine bond issues, total debt service of $1,501 million, and debt service ending in 2040/41. Table 12 is a detailed schedule of debt service showing five bond issues, total debt service of $1,329 million, and debt service ending in 2028/29.

F.2. 15-Year Construction Phasing

Table 11 shows inflated construction costs, bond issue sizes, and estimated debt service costs resulting from a 15-year construction phasing schedule. It shows a total bond program of $573.6 million and maximum debt service of $53.2 million in the 13th year.

TABLE 9. ESTIMATED DEBT SERVICE - RECOMMENDED PROJECT (X $1000)
(Project Phasing Option 1 - 27-Year Construction Period)

Year	Construction Cost		Bond Issue Size[2,3]	Cumulative Annual Debt Svc[2]
	1992 Dollars	Inflated Dollars[1]		
1	$3,121	$3,200	$40,500	$0
2	$18,733	$20,300		$3,754
3	$13,718	$15,400	$108,100	$3,754
4	$13,069	$15,300		$13,775
5	$12,426	$15,100		$13,775
6	$58,118	$73,500	$90,500	$13,775
7	$27,544	$36,200		$22,165
8	$18,226	$24,900		$22,165
9	$18,226	$25,900	$87,700	$22,165
10	$18,226	$27,000		$30,294
11	$18,226	$28,100		$30,294
12	$18,226	$29,200	$79,100	$30,294
13	$18,226	$30,300		$37,627
14	$17,826	$30,900		$37,627
15	$8,294	$14,900	$50,400	$37,627
16	$8,294	$15,500		$42,299
17	$8,294	$16,200		$42,299
18	$8,294	$16,800	$56,800	$42,299
19	$8,294	$17,500		$47,564
20	$8,294	$18,200		$47,564
21	$8,294	$18,900	$63,900	$47,564
22	$8,294	$19,700		$53,488
23	$8,294	$20,400		$53,488
24	$8,294	$21,300	$70,700	$53,488
25	$8,294	$22,100		$60,042
26	$8,294	$23,000		$60,042
27	$7,958	$22,900		$56,288
Total	$381,397	$622,700	$647,700	

1 Inflated 4% annually.
2 Estimates provided by Bartle Wells Associates based on comparable data supplied by City Treasurer's Office.
3 Includes cost of issuance and debt service reserve.

TABLE 10. ESTIMATED DEBT SERVICE SCHEDULE - $381 MILLION RECOMMENDED PROJECT (X $1000) (Project Phasing Option 1 - 27-Year Construction Period)[1]

Bond--->	$40.500	$108.100	$90.500	$87.700	$79.100	$50.400	$56.800	$63.900	$70.700	$647.700
Debt Svc Year										Total Annual
1993/94	$3.754									$3.754
1994/95	$3.754									$3.754
1995/96	$3.754	$10.021								$13.775
1996/97	$3.754	$10.021								$13.775
1997/98	$3.754	$10.021								$13.775
1998/99	$3.754	$10.021	$8.390							$22.165
1999/00	$3.754	$10.021	$8.390							$22.165
2000/01	$3.754	$10.021	$8.390							$22.165
2001/02	$3.754	$10.021	$8.390	$8.129						$30.294
2002/03	$3.754	$10.021	$8.390	$8.129						$30.294
2003/04	$3.754	$10.021	$8.390	$8.129						$30.294
2004/05	$3.754	$10.021	$8.390	$8.129	$7.333					$37.627
2005/06	$3.754	$10.021	$8.390	$8.129	$7.333					$37.627
2006/07	$3.754	$10.021	$8.390	$8.129	$7.333					$37.627
2007/08	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602				$42.229
2008/09	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602				$42.229
2009/10	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602				$42.229
2010/11	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602	$5.335			$47.564
2011/12	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602	$5.335			$47.564
2012/13	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602	$5.335			$47.564
2013/14	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602	$5.335	$5.924		$53.488
2014/15	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602	$5.335	$5.924		$53.488
2015/16	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602	$5.335	$5.924		$53.488
2016/17	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$60.042
2017/18	$3.754	$10.021	$8.390	$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$60.042
2018/19		$10.021	$8.390	$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$56.288
2019/20		$10.021	$8.390	$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$56.288
2020/21			$8.390	$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$46.267
2021/22			$8.390	$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$46.267
2022/23			$8.390	$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$46.267
2023/24				$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$37.877
2024/25				$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$37.877
2025/26				$8.129	$7.333	$4.602	$5.335	$5.924	$6.554	$37.877
2026/27					$7.333	$4.602	$5.335	$5.924	$6.554	$29.748
2027/28					$7.333	$4.602	$5.335	$5.924	$6.554	$29.748
2028/29					$7.333	$4.602	$5.335	$5.924	$6.554	$29.748
2029/30						$4.602	$5.335	$5.924	$6.554	$22.415
2030/31						$4.602	$5.335	$5.924	$6.554	$22.415
2031/32						$4.602	$5.335	$5.924	$6.554	$22.415
2032/33							$5.335	$5.924	$6.554	$17.813
2033/34							$5.335	$5.924	$6.554	$17.813
2034/35							$5.335	$5.924	$6.554	$17.813
2035/36								$5.924	$6.554	$12.478
2036/37								$5.924	$6.554	$12.478
2037/38								$5.924	$6.554	$12.478
2038/39									$6.554	$6.554
2039/40									$6.554	$6.554
2040/41									$6.554	$6.554
TOTALS	$93.850	$250.525	$209.750	$203.225	$183.325	$115.050	$133.375	$148.100	$163.850	$1.501.050

1 Assumes average interest cost of 7.5%, 25-year maturities, and 5% earnings on construction and reserve funds during construction.

TABLE 11. ESTIMATED DEBT SERVICE - RECOMMENDED PROJECT (X $1000) (Project Phasing Option 2 - 15-year Construction Period)				
Year	Construction Cost		Bond Issue Size[2,3]	Cumulative Annual Debt Service[2]
	1992 Dollars	Inflated Dollars[1]		
1	$3,121	$3,200	$40,500	$0
2	$18,733	$20,300		$3,754
3	$13,718	$15,400	$84,800	$3,754
4	$13,069	$15,300		$11,615
5	$11,686	$14,200		$11,615
6	$41,083	$52,000	$140,100	$11,615
7	$39,105	$51,500		$24,603
8	$29,787	$40,800		$24,603
9	$29,813	$42,400	$146,100	$24,603
10	$30,416	$45,000		$38,146
11	$30,416	$46,800		$38,146
12	$30,416	$48,700	162,100	$38,146
13	$30,416	$50,600		$53,173
14	$30,416	$52,700		$53,173
15	$29,202	$52,600		$53,173
Total	$381,397	$551,500	$573,600	

1 Inflated 4% annually.
2 Estimates provided by Bartle Wells Associates based on comparable data supplied by City Treasurer's Office.
3 Includes cost of issuance and debt service reserve.

TABLE 12. ESTIMATED DEBT SERVICE SCHEDULE - $381 MILLION RECOMMENDED PROJECT (X $1000)
(Project Phasing Option 2 - 27-Year Construction Period)[1]

Bond--->	$40,500	$84,800	$140,100	$146,100	$162,100	$573,600
Debt Svc Year						Total Annual
1993/94	$3,754					$3,754
1994/95	$3,754					$3,754
1995/96	$3,754	$7,861				$11,615
1996/97	$3,754	$7,861				$11,615
1997/98	$3,754	$7,861				$11,615
1998/99	$3,754	$7,861	$12,988			$24,603
1999/00	$3,754	$7,861	$12,988			$24,603
2000/01	$3,754	$7,861	$12,988			$24,603
2001/02	$3,754	$7,861	$12,988	$13,543		$38,146
2002/03	$3,754	$7,861	$12,988	$13,543		$38,146
2003/04	$3,754	$7,861	$12,988	$13,543		$38,146
2004/05	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2005/06	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2006/07	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2007/08	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2008/09	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2009/10	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2010/11	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2011/12	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2012/13	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2013/14	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2014/15	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2015/16	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2016/17	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2017/18	$3,754	$7,861	$12,988	$13,543	$15,027	$53,173
2018/19		$7,861	$12,988	$13,543	$15,027	$49,419
2019/20		$7,861	$12,988	$13,543	$15,027	$49,419
2020/21			$12,988	$13,543	$15,027	$41,558
2021/22			$12,988	$13,543	$15,027	$41,558
2022/23			$12,988	$13,543	$15,027	$41,558
2023/24				$13,543	$15,027	$28,570
2024/25				$13,543	$15,027	$28,570
2025/26				$13,543	$15,027	$28,570
2026/27					$15,027	$15,027
2027/28					$15,027	$15,027
2028/29					$15,027	$15,027
2029/30						$0
TOTALS	$93,850	$196,525	$324,700	$338,575	$375,675	$1,329,325

1 Assumes average interest cost of 7.5%, 25-year maturities, and 5% earnings on construction and reserve funds during construction.

G. FEDERAL AND STATE LOAN SOURCES AVAILABLE FOR PROJECT COSTS

G.1. Federal Grant/Loan Sources

For the 13-year period from 1973 through 1985, California was allotted $3.8 billion for federal Clean Water grants. In general, these grants paid for 75 percent of eligible project costs. That federal program was discontinued. The federal Clean Water Act now provides for a State Revolving Fund (SRF) to assist municipalities in meeting federal wastewater discharge requirements. The program is capitalized in part by federal funds. The SRF is administered by the state; no grants or loans are currently available directly from the federal program.

G.2. State Revolving Loan Fund

The State Water Resources Control Board (SWRCB) adopted an SRF program in 1988. It provides low-cost loans to public agencies for wastewater projects. The program is funded in part from federal funds which are capitalized through the loan program. Funds are loaned at one-half of the interest rate on the state's most recent general obligation bond sale and provide a low-cost source of funds.

Availability of loan funds is determined by federal funding available on a fiscal year basis. In order to participate in the next bonding cycle (fiscal 1993), an initial loan request was submitted to the Regional Board by the City on March 19, 1992. However, the City was advised that a specific project must be designated to be considered for the state's priority list. The Regional Board suggested that, upon selection of a specific project, the City should submit a mid-year resolution requesting addition to the priority list.

In fiscal 1992, the SWRCB limited funding for any one entity to $20,000,000, or 20 percent of its yearly allotment, whichever is less. The Association of Metropolitan Sewerage Agencies advises that federal funding for fiscal 1993 will not be known until September 1992. A funding level similar to fiscal 1992 would limit the City to a maximum loan of $20,000,000. The City should pursue

each step of the application process to mitigate the cost to users. Receipt of additional loan funds after fiscal 1993 would further mitigate the user fee impact.

In summary, certain loan restrictions apply. Specific loan provisions include:

- Terms up to 25 years
- Repayment begins one year after completion of construction
- No funds advanced during design phase
- No reimbursement for design costs

Appendix B contains materials which describe the administration and operational requirements of the loan program, namely:

- SWRCB policy manual for the SRF (excerpts) dated September 20, 1990
- Sample loan contract
- Loan contract standard conditions
- SWRCB 1993 priority list discussion

H. CONNECTION CHARGE

H.1. Basis and Methods

The City currently levies connection charges (called system development fees) for new water and sanitary sewer connections but not for storm drainage. The sanitary sewer charge for a 4-inch residential connection is $123.54 and increases for larger connections applicable to multiple units and commercial users. CH_2M Hill in its March 1991 *Rate Structure Review* recommended that the City adopt a storm drainage impact (or connection) fee.

New development should pay its fair share of capital costs and then pay user charges on an equal basis with existing users. Existing customers have provided funds for debt retirement, cash financing of improvements, and additions to the system through their past payment of user rates or through

capital contributions. Collecting a front-end capital payment in the form of a connection charge is a typical method of recovering these costs. Connection charges are also called impact fees or system development fees. Such fees initiated or increased after January 1, 1989 are also subject to compliance with Government Code §6600-66008 relating to the equity of the fee (discussed further below).

Several approaches have been used by other agencies to set connection charges that meet the test of equity, including:

- System buy-in method
- Marginal cost of expansion
- Buildout cost (combination of both methods above)

H.2. System Buy-In Method

Capital improvement projects designed to control street flooding, CSOs, and outflows are intended to correct problems with the existing system; no portion of the cost has been identified as growth-related. The system buy-in method is one way in which new customers can be required to pay an amount equal to the net investment in the existing system.

After new customers have paid their front-end capital contribution toward backup plant facilities, they become equivalent to existing customers and share the responsibility for existing assets and liabilities, including the retirement of bonded indebtedness. When additional costs are incurred for system improvements or replacements, all customers share in the costs.

However, the majority of new growth in the City is expected to occur in the North Natomas area where drainage facilities are proposed to be constructed by revenue from a Mello-Roos district. In that case, a system buy-in charge would not be appropriate. In other areas, such a charge could be applied.

Because customer use characteristics vary, it is necessary to develop a system of customer equivalents for the calculation of the buy-in amount. The

calculations which follow use the number of EDUs developed in Table 7 as the customer equivalent.

H.3. Buy-In Charge Calculation

Table 13 shows the calculation to develop the current value of existing facilities. The 1988 system values are based on data developed by CH_2M Hill in its March 1991 *Rate Structure Review*. Values developed in that study were updated to a current value through application of the March 1992, 20-Cities index ratio from *Engineering News-Record* (ENR) magazine.

TABLE 13. CURRENT VALUE OF EXISTING FACILITIES, MARCH 1992 (x $1000)

	Value		ENR Index[1]	Current Value
	Amount	Date		
Sewer System				
Land[2]	$1,098	1960	830	$6,517
Treatment plant	$15,000	1988	4568	$16,176
Mains	$82,387	1988	4568	$88,844
Laterals	$52,250	1988	4568	$56,345
Pump stations	$7,887	1988	4568	$8,502
Construction in progress[2]	$3,500	1991	4889	$3,526
Total	$162,122			$179,910
Storm Drainage System				
Mains	$96,054	1988	4568	$103,582
Secondary mains	$597,067	1988	4568	$643,860
Canals & channels	$155,891	1988	4568	$168,108
Levees	$27,730	1988	4568	$29,903
Pump stations	$58,470	1988	4568	$63,052
Construction in progress[2]	$3,172	1991	4889	$3,420
Total	$938,384			$1,011,926

1 20-Cities Index; March 1992 index = 4926.
2 From audited financial statement of the City, June 30, 1991; all other value amounts from CH_2M Hill Rate Structure Review, March 1991 (replacement value less depreciation).

The City issued debt to finance acquisition of certain facility costs and other facilities were contributed by developers. Table 14 adjusts current facility value by outstanding debt and the portion of facilities provided by developers, to arrive at a net buy-in value to be shared by new customers.

Table 14 also shows a buy-in value per estimated EDU, as follows:

Sanitary sewer	$1,350
Storm drainage	1,620

TABLE 14. SYSTEM FACILITIES BUY-IN VALUE - MARCH 1992

	Sewer System		Storm Drainage	
	Percent	Amount	Percent	Amount
Current value of existing facilities (x $1000)		$179,910		$1,011,926
Less: Contributed capital portion (x $1000)	20.1%[1]	$36,162	70.6%[2]	$714,420
Less: Outstanding debt (x $1000)	100.0%	$3,363	100.0%	$11,525
Net buy-in value of existing facilities (x $1000)		$140,385		$285,981
EDUs		103,800		176,500
Facilities buy-in value per EDU		$1,352		$1,620
Recommended initial connection charge		$1,350		$1,620
Existing system development fee		$123		$ --

1 Developer contributed capital ÷ fixed asset book value (before depreciation) (14,805 ÷ 73,641 = 20.1%).

2 Developer contributed capital ÷ fixed asset book value (before depreciation) (25,982 ÷ 36,782 = 70.6%).

Source: 1990/91 audited financial statements of the City.

H.4. Documentation Necessary to Establish or Increase a Connection Charge

From the standpoint of equity to the City's customers and potential customers, if the City adopts a connection charge, it is important to document that charges do not exceed the cost of service, thereby becoming a tax. Further, Government Code §66000-66003 (AB 1600) added specific procedural requirements for adoption of or increases to developer fees charged to finance public facilities. This legislation became effective January 1, 1989. The City should make certain, documented determinations regarding the purpose and use of charges.

Additionally, the City must determine that there is a reasonable relationship between the amount of the charge and the cost of facilities attributable to the capital infrastructure system on which the charge is imposed.

The analysis in Subsection H.3. determined the buy-in value of existing facilities. The recommended charge is based on a generally accepted unit of measurement (the value per equivalent dwelling unit), to establish the relationship between its public facility cost and the charge. The City can use this analysis, data, and recommended charge schedule as documentation for the required charge determinations.

H.5. Impact on User Rates

The Task 8 technical memorandum entitled *Rate Assessment of Least-Cost Alternatives*, concluded that user rates required for the recommended plan fell within affordability guidelines. Such rates do not consider connection charges or potential increases in other utility rates, such as regional sewer, water, flood control, and solid waste rate increases that have an overall effect on utility rate affordability. Additional revenue generated from connection charges would have the effect of mitigating user fee increases. However, connection charge revenue is not subject to accurate projection since it is growth-dependent. Therefore, such revenue is not included in rate-impact revenue projections that follow.

I. USER FEE IMPACT - RECOMMENDED PLAN

I.1. Rate Options

Based on the City's selection of the Separate Sanitary Sewer Alternative as the recommended plan, this section estimates the rate impact of implementing that plan for each of four rate allocation options, namely:

1) **Option 1:** Increase rates equally for all sanitary sewer and storm drainage customers throughout the City.

2) **Option 2:** Increase rates for combined service area customers only, both sanitary sewer and storm drainage. Increase rates for users outside the combined area and Sacramento Regional CSD by 4 percent inflationary factor only.

3) **Option 3:** Increase sanitary sewer rates only (entire City). Increase rates for storm drainage accounts and Sacramento Regional by 4 percent inflationary factor only.

4) **Option 4:** Increase rates for all Sacramento Regional customers (including City customers) rather than increasing City user rates. Further, the rate impact for both a 27-year and 15-year construction phasing schedule is estimated for each of the four rate options above.

I.2. Key Assumptions

Key assumptions in estimating the rate impact for each option include the following:

- 1991/92 budget data are a base for revenue and expense projections.

- Ending fund balance in each fiscal year should exceed $1.0 million to provide a minimum contingency allowance.

- A conservative annual growth rate in EDUs and ESDs is shown in Table 15.

- Number of combined system customers based on estimates discussed in Subsection E and Table 8 of this TM.

- Operation and maintenance costs escalated at an annual inflation rate of 4 percent.

- Options 1 through 3 include an inflationary increase only in the current $8.15 Sacramento Regional County Sanitation District monthly user charge. Using this assumption, the District charge would increase to $16.51 in 2009/10. The District is, however, revising its treatment plant master plan, which includes consideration of significant new capital project costs. A District master plan Technical Memorandum No. 3 on Task 900, entitled *Funding Alternatives Analysis and Recommendation* and dated December 1991, indicates a range of possible monthly user charges for each of four capital project scenarios through the year 2010, as shown in Table 16. If the District proceeds with any alternative other than Scenario 1, the total user rate paid by City system users would be increased by $20 to $56 per month.

- Additional debt service costs for the recommended plan based on estimates from the City treasurer's office and its financing team. Estimates are based on a bond program structured to meet the construction schedule and inflated construction cost estimates for the recommended plan.

- Revenue bond debt service coverage requirements equal to at least 1.2 times net revenue.

The tables that follow show the rate impact for the City's recommended improvement plan through the year of maximum cumulative debt service cost, at

which time all bonds are assumed to have been sold. To the extent possible, rates have been stabilized by regular annual increases rather than less frequent but larger increases at the time of each bond sale.

TABLE 15. SEWER SYSTEM GROWTH RATE		
	Recent Past Growth[1]	Rate Impact Assumption
City sanitary sewer	0.88%	0.7%
City storm drainage	2.61%	2.0%
Sacramento Regional system	2.84%	2.0%

1 See Tables 4 and 5.

TABLE 16. POTENTIAL USER RATES - SACRAMENTO REGIONAL			
		Required Charge	
Scenario*	Current Charge	2004	2010
1	$8	$16	$18
2	$8	$31	$36
3	$8	$58	$68
4	$8	$60	$72

* Scenario descriptions:

1 Secondary treatment.
2 Intermediate metals and ammonia reduction in addition to Scenario 1.
3 Advanced metals reduction in addition to Scenario 1.
4 Advanced metals and ammonia reduction in addition to Scenario 1.

I.3. Rate Option 1: 27-Year Construction Phasing

In this option, required user rates would not exceed the affordability guidelines as represented by the EPA Index of Hardship Threshold (EPA Index). However, an initial rate increase of 24 percent is required for 1992/93. Significant additional rate increases averaging 8.6 percent per year would also be required through the year 1997/98. Beyond that year, required rate increases would not be greater than an inflationary increase.

Table 17 summarizes the rate impact to the first year of maximum annual debt service (2016/17). Required total City user charges in that year would be $56.51, an increase of $13.41 or 31 percent, compared to 1991/92 rates inflated to 2016/17.

TABLE 17. SUMMARY OF MONTHLY RATE IMPACT; 27-YEAR CONSTRUCTION PHASING RATE OPTION 1

Rate	1991/92 Actual	2004/05		Increase[2]	
		Inflation Only (4%)	Total[1]	Amount	Percent
Sanitary sewer	$7.07	$18.85	$24.77	$5.92	31%
Storm drainage	$9.06	$24.15	$31.74	$7.59	31%
Total City user charge	$16.13	$43.00	$56.51	$13.41	31%
Sacramento Regional charge	$8.15	$21.73	$21.73	$0.00	0%
Total user charge	$24.28	$64.73	$78.24	$13.41	21%
Affordability - Index rate	$46.23	$123.24	$123.24	n/a	n/a
Affordability - percent of EBI[3]	1.05%	1.05%	1.27%	n/a	n/a

1 For inflation plus project cost.
2 Due to project cost only.
3 EPA Index is 2%.

I.4. Rate Option 1: 15-Year Construction Phasing

In this option, required user rates would not exceed the EPA Index for affordability. However, an initial rate increase of 24 percent is required for 1992/93. Significant additional rate increases averaging 7.7 percent per year would also be required through the year 2004/05. Beyond that year, required rate increases would not be greater than an inflationary increase.

Table 18 summarizes the rate impact to the first year of maximum annual debt service (2004/05). Required total City user charges in that year would be $48.51, an increase of $21.65, or 81 percent compared to 1991/92 rates inflated to 2004/05.

TABLE 18. SUMMARY OF MONTHLY RATE IMPACT; 15-YEAR CONSTRUCTION PHASING RATE OPTION 1

Rate	1991/92 Actual	2004/05		Increase[2]	
		Inflation Only (4%)	Total[1]	Amount	Percent
Sanitary sewer	$7.07	$11.77	$21.26	$9.49	81%
Storm drainage	$9.06	$15.09	$27.25	$12.16	81%
Total City user charge	$16.13	$26.86	$48.51	$21.65	81%
Sacramento Regional charge	$8.15	$13.57	$13.57	$0.00	0%
Total user charge	$24.28	$40.43	$62.08	$21.65	54%
Affordability - Index rate	$46.23	$76.97	$76.97	n/a	n/a
Affordability - percent of EBI[3]	1.05%	1.05%	1.61%	n/a	n/a

1 For inflation plus project cost.
2 Due to project cost only.
3 EPA Index is 2%.

I.5. Rate Option 2: 27-Year Construction Phasing

In this option, required user rates would exceed the EPA Index for affordability. An initial rate increase of 70 percent is required for combined system users in 1992/93. Significant additional rate increases averaging 14.0 percent per year would also be required for combined system users through the year 2002/03. Beyond that year, required rate increases would not be greater than an inflationary increase.

Table 19 summarizes the rate impact to the first year of maximum annual debt service (2016/17). Required highest total City user charges in that year would be $130.94, an increase of $87.94, or 205 percent compared to 1991/92 rates inflated to 2016/17.

TABLE 19. SUMMARY OF MONTHLY RATE IMPACT; 27-YEAR CONSTRUCTION PHASING RATE OPTION 2

Rate	1991/92 Actual	2016/17		Increase[2]	
		Inflation Only (4%)	Total[1]	Amount	Percent
Sanitary sewer - combined system	$7.07	$18.85	$57.39	$38.54	205%
Sanitary sewer - other	$7.07	$18.85	$18.85	$0.00	0%
Storm drainage - combined system	$9.06	$24.15	$73.55	$49.40	205%
Storm drainage - other	$9.06	$24.15	$24.15	$0.00	0%
Highest total City user charge	$16.13	$43.00	$130.94	$87.94	205%
Sacramento Regional charge	$8.15	$21.73	$21.73	$0.00	0%
Total user charge	$24.28	$64.73	$152.67	$87.94	136%
Affordability - Index rate	$46.23	$123.24	$123.24	n/a	n/a
Affordability - percent of EBI[3]	1.05%	1.05%	2.48%	n/a	n/a

1 For inflation plus project cost.
2 Due to project cost only.
3 EPA Index is 2%.

I.6. Rate Option 2: 15-Year Construction Phasing

In this option, required user rates would exceed the EPA Index for affordability. An initial rate increase of 70 percent is required for combined system users in 1992/93. Significant additional rate increases averaging 15.0 percent per year would also be required for combined system users through the year 2004/05. Beyond that year, required rate increases would not be greater than an inflationary increase.

Table 20 summarizes the rate impact to the first year of maximum annual debt service (2004/05). Required highest total City user charges in that year would be $146.61, an increase of $119.75, or 446 percent compared to 1991/92 rates inflated to 2004/05.

TABLE 20. SUMMARY OF MONTHLY RATE IMPACT; 15-YEAR CONSTRUCTION PHASING

RATE OPTION 2

Rate	1991/92 Actual	2004/05		Increase[2]	
		Inflation Only (4%)	Total[1]	Amount	Percent
Sanitary sewer - combined system	$7.07	$11.77	$64.26	$52.49	446%
Sanitary sewer - other	$7.07	$11.77	$11.77	$0.00	0%
Storm drainage - combined system	$9.06	$15.09	$82.35	$67.26	446%
Storm drainage - other	$9.06	$15.09	$15.09	$0.00	0%
Highest total City user charge	$16.13	$26.86	$146.61	$119.75	446%
Sacramento Regional charge	$8.15	$13.57	$13.57	$0.00	0%
Total user charge	$24.28	$40.43	$160.18	$119.75	296%
Affordability - Index rate	$46.23	$76.97	$76.97	n/a	n/a
Affordability - percent of EBI[3]	1.05%	1.05%	4.16%	n/a	n/a

1 For inflation plus project cost.
2 Due to project cost only.
3 EPA Index is 2%.

I.7. Rate Option 3: 27-Year Construction Phasing

In this option, required user rates would not exceed the EPA Index for affordability. However, an initial rate increase of 50 percent is required for sanitary sewer system users in 1992/93. Significant additional rate increases averaging 11.9 percent per year would also be required for sanitary sewer system users through the year 2002/03. Beyond that year, required rate increases would not be greater than an inflationary increase.

Table 21 summarizes the rate impact to the first year of maximum annual debt service (2016/17). Required total City user charges in that year would be $65.33, an increase of $22.33, or 52 percent compared to 1991/92 rates inflated to 2016/17.

TABLE 21. SUMMARY OF MONTHLY RATE IMPACT; 27-YEAR CONSTRUCTION PHASING

RATE OPTION 3

Rate	1991/92 Actual	2016/17		Increase[2]	
		Inflation Only (4%)	Total[1]	Amount	Percent
Sanitary sewer	$7.07	$18.85	$41.18	$22.33	119%
Storm drainage	$9.06	$24.15	$24.15	$0.00	0%
Total City user charge	$16.13	$43.00	$65.33	$22.33	52%
Sacramento Regional charge	$8.15	$21.73	$21.73	$0.00	0%
Total user charge	$24.28	$64.73	$87.06	$22.33	35%
Affordability - Index rate	$46.23	$123.24	$123.24	n/a	n/a
Affordability - percent of EBI[3]	1.05%	1.05%	1.41%	n/a	n/a

1 For inflation plus project cost.
2 Due to project cost only.
3 EPA Index is 2%.

I.8. Rate Option 3: 15-Year Construction Phasing

In this option, required user rates would closely approximate the EPA Index rate. An initial rate increase of 50 percent is required for sanitary sewer system users for 1992/93. Significant additional rate increases averaging 12.8 percent per year would also be required for sanitary sewer system users through the year 2004/05. Beyond that year, required rate increases would not be greater than an inflationary increase.

Table 22 summarizes the rate impact to the first year of maximum annual debt service (2004/05). Required total City user charges in that year would be $59.98, an increase of $33.12, or 123 percent compared to 1991/92 rates inflated to 2004/05.

TABLE 22. SUMMARY OF MONTHLY RATE IMPACT; 15-YEAR CONSTRUCTION PHASING

RATE OPTION 3

Rate	1991/92 Actual	2004/05		Increase[2]	
		Inflation Only (4%)	Total[1]	Amount	Percent
Sanitary sewer-	$7.07	$11.77	$44.89	$33.12	281%
Storm drainage	$9.06	$15.09	$15.09	$0.00	0%
Total City user charge	$16.13	$26.86	$59.98	$33.12	123%
Sacramento Regional charge	$8.15	$13.57	$13.57	$0.00	0%
Total user charge	$24.28	$40.43	$73.55	$33.12	82%
Affordability - Index rate	$46.23	$76.97	$76.97	n/a	n/a
Affordability - percent of EBI[3]	1.05%	1.05%	1.91%	n/a	n/a

1 For inflation plus project cost.
2 Due to project cost only.
3 EPA Index is 2%.

I.9. Rate Option 4: 27-Year Construction Phasing

Table 23 summarizes the rate impact to the first year of maximum annual debt service (2016/17). Required total City user charges in that year would be only $43.00, since City rates would increase by inflation only. Sacramento Regional CSD rates show an increase of $6.43 for capital purposes, which would equate to a 30-percent increase in its rate compared to 1991/92 rates inflated to 2016/17.

TABLE 23. SUMMARY OF MONTHLY RATE IMPACT; 27-YEAR CONSTRUCTION PHASING

RATE OPTION 4

Rate	1991/92 Actual	2016/17		Increase[2]	
		Inflation Only (4%)	Total[1]	Amount	Percent
Sanitary sewer	$7.07	$18.85	$18.85	$0.00	0%
Storm drainage	$9.06	$24.15	$24.15	$0.00	0%
Total City user charge	$16.13	$43.00	$43.00	$0.00	0%
Sacramento Regional charge - O&M	$8.15	$21.73	$21.73	$0.00	0%
Sacramento Regional charge - capital	$0.00	$0.00	$6.43	$6.43	n/a
Total Sacramento Regional charge	$8.15	$21.73	$28.16	$6.43	30%
Total user charge	$24.28	$64.73	$71.16	$6.43	10%
Affordability - Index rate	$46.23	$123.24	$123.24	n/a	n/a
Affordability - percent of EBI[3]	1.05%	1.05%	1.16%	n/a	n/a

1 For inflation plus project cost.
2 Due to project cost only.
3 EPA Index is 2%.

I.10. Rate Option 4: 15-Year Construction Phasing

Table 24 summarizes the rate impact to the first year of maximum annual debt service (2004/05). Required total City user charges in that year would be only $26.86 since City rates increased by inflation only. Sacramento Regional CSD rates show an increase of $6.94 for capital purposes, which would equate to a 51 percent increase in its rate compared to 1991/92 rates inflated to 2004/05.

TABLE 24. SUMMARY OF MONTHLY RATE IMPACT; 15-YEAR CONSTRUCTION PHASING

RATE OPTION 4

Rate	1991/92 Actual	2004/05		Increase[2]	
		Inflation Only (4%)	Total[1]	Amount	Percent
Sanitary sewer	$7.07	$11.77	$11.77	$0.00	0%
Storm drainage	$9.06	$15.09	$15.09	$0.00	0%
Total City user charge	$16.13	$26.86	$26.86	$0.00	0%
Sacramento Regional charge - O&M	$8.15	$13.57	$13.57	$0.00	0%
Sacramento Regional charge - capital	$0.00	$0.00	$6.94	$6.94	n/a
Total Sacramento Regional charge	$8.15	$13.57	$20.51	$6.94	51%
Total user charge	$24.28	$40.43	$47.37	$6.94	17%
Affordability - Index rate	$46.23	$76.97	$76.97	n/a	n/a
Affordability - percent of EBI[3]	1.05%	1.05%	1.23%	n/a	n/a

1 For inflation plus project cost.
2 Due to project cost only.
3 EPA Index is 2%.

I.11. Summary of User Rate and Debt Service Impact

Table 25 shows a summary of required user rate and debt service impact for the recommended plan for each of the four rate options and two construction phasing schedules.

Funding of the recommended plan falls within the affordability index for Rate Options 1, 3, and 4 but not under Rate Option 2. In each rate option, the 27-year construction phasing schedule provides a more affordable annual financing plan for system users than the 15-year schedule. However, the 15-year schedule results in a debt service savings of $171,725,000, or approximately 11 percent. In addition, debt service would end 12 years earlier (i.e., in the year 2028/29).

J. RATE COMPARISON WITH COMPARABLE AGENCIES

J.1. Rate Surveys

The State Water Resources Control Board (SWRCB), the Association of Metropolitan Sewerage Agencies (AMSA), and Black & Veatch have conducted surveys of service charges for sanitary sewer. SWRCB and Black & Veatch compiled data on monthly service fees and connection charges from public agencies in California. SWRCB's survey data is as of July 1, 1990. Black & Veatch's survey results are more recent, as of January 1991. AMSA's survey was nationwide and covered most of the nation's largest wastewater treatment agencies. AMSA's survey was the most comprehensive of the three surveys and included information on finances, rates, and operations.

Black & Veatch's survey presents the most recent information in a form that is easiest to compare to Sacramento. Table 26 is a summary of the survey results for January 1991. The survey involved more than 300 California cities and counties.

TABLE 25. USER RATE IMPACT SUMMARY

Rate Option:	Option 1[1]		Option 2[2]		Option 3[3]		Option 4[4]	
Construction Phasing	27-Year	15-Year	27-Year	15-Year	27-Year	15-Year	27-Year	15-Year
Year	2016/17	2004/05	2016/17	2004/05	2016/17	2004/05	2016/17	2004/05
Inflated 1991/92 Total City User Rate	$43.00	$26.86	$43.00	$26.86	$43.00	$26.86	$43.00	$26.86
Required User Rates								
Highest Total City Rate	$56.51	$48.51	$130.94	$146.61	$65.33	$59.98	$43.00	$26.86
Highest Total User Rate[5]	$78.24	$62.08	$152.67	$160.18	$87.06	$73.55	$71.16	$47.37
Affordability Index Rate	$123.24	$76.97	$123.24	$76.97	$123.24	$76.97	$123.24	$76.97
Percent of EBI[6]	1.27%	1.61%	2.48%	4.16%	1.41%	1.91%	1.16%	1.23%
Debt Service								
Maximum Annual (x $1000)	$60,042	$53,173	$60,042	$53,173	$60,042	$53,173	$60,042	$53,173
Total (x $1000)	$1,501,050	$1,329,325	$1,501,050	$1,329,325	$1,501,050	$1,329,326	$1,501,050	$1,329,325
Payable through Year	2040/41	2028/29	2040/41	2028/29	2040/41	2028/29	2040/41	2028/29

1 Increases applied equally to sanitary and storm drainage users in the City.
2 Increases applied only to combined system users.
3 Increases applied only to sanitary system users.
4 Sacramento Regional CSD capital rate; no increase in City rates.
5 Includes Sacramento Regional rate.
6 Effective buying income. EPA Index is 2%.

Table 26 shows the average, median, low, and high fees for monthly sewer use and one-time connection charges, all of which are broken down by population. The user fees and connection charges apply to a typical single-family residential customer.

TABLE 26. CALIFORNIA CITIES AND DISTRICTS: SEWER SERVICE FEE COMPARISON[1,2]

Population	Cities & Districts Surveyed	Average	Median	Low	High
Up to 1,000					
Monthly fees	19	$24.05	$18.00	$8.33	$88.75
Connection charge		$1,893.00	$1,000.00	$150.00	$4,472.00
1,001 to 10,000					
Monthly fees	86	$12.62	$10.30	$0.00[3]	$55.50
Connection charge		$1,669.00	$1,400.00	$0.00	$6,500.00
10,001 to 50,000					
Monthly fees	108	$10.30	$9.60	$3.54	$27.40
Connection charge		$1,887.00	$1,667.00	$0.00	$8,025.00
50,001 to 100,000					
Monthly fees	48	$9.61	$8.90	$3.92	$23.52
Connection charge		$2,176.00	$2,184.00	$328.00	$4,851.00
Over 100,000					
Monthly fees	41	$10.04	$8.75	$4.08	$30.09
Connection charge		$1,841.00	$1,565.00	$0.00	$4,810.00

1 Single-family residential monthly fees.
2 Excludes ad valorem taxes.
3 Los Angeles County Sanitation District No.27 currently collects sufficient funds through ad valorem taxes to pay for O&M without collecting a sewer charge.
Source: Black & Veatch, Sewer Service Survey (1991).

The data for cities and districts with population over 100,000 is the most comparable to the City of Sacramento, whose population is about 347,000. The Black & Veatch survey shows that for sewer agencies with over 100,000 population, the average monthly service fee is $10.04. This compares to the total service fee of $15.22 for Sacramento. City of Sacramento residents pay two monthly service fees of:

Average City residential fee	$ 7.07
Sacramento Regional residential fee	8.15
Total	$15.22

The average connection charge for agencies with populations over 100,000, shown in Black & Veatch survey, is $1,841, compared to Sacramento's current connection charge of $124.

J.2. Comparison of Sewer Fees and Charges in Nearby Cities

A comparison of sewer fees and charges was made between Sacramento and some nearby cities. Table 27 compares the current monthly service fees and connection charges of six selected cities with Sacramento's. The cities were chosen on the basis of proximity to Sacramento and similarity in population to Sacramento. The selected cities include: Bakersfield, Concord, Fresno, San Francisco, San Jose, and Stockton. The information shown in this table is based on Bartle Wells Associates' telephone survey in April 1992. The fees and charges are for a single-family residence.

Table 27 reveals that Sacramento's current total monthly user fee is already higher than all selected cities except San Jose and San Francisco. Conversely, Sacramento's current connection charge is the lowest of the charges shown in the table.

J.3. Monthly User Fees for Four Utility Services

A comparison was made of combined monthly user fees for four utility services, including sanitary sewer, storm drainage, water, and solid waste. Once again,

Sacramento's monthly user fees are compared to those for Bakersfield, Concord, Fresno, San Francisco, San Jose, and Stockton.

TABLE 27. SELECTED CALIFORNIA CITIES SEWER USER FEE AND CHARGE COMPARISON[1]		
City	Monthly User Fee	Connection Charge
Bakersfield	$8.50	$900
Concord	$13.00	$1,750
Fresno	$5.46	$200
San Francisco	$31.34	None
San Jose	$15.30	$1,227
Stockton	$12.99	$2,674
Sacramento	$15.15	$124

1 Typical single-family residence.
Source: Bartle Wells Associates telephone survey (April 1992).

Table 28 shows the comparisons and is based on a telephone survey conducted by Bartle Wells Associates in April 1992. The user fees are for a typical single-family residence. Certain utility services are provided by public agencies, while others are supplied by private companies. The cities of Sacramento, Stockton, and San Jose are the only cities surveyed by Bartle Wells Associates that have instituted specific user fees for storm drainage.

Table 28 reveals that Sacramento's combined monthly user fee of $42.88 lies in the middle of the total fees charged by the other seven cities and is below the average of $51.39 for all eight surveyed cities.

J.4. Rate Comparison Summary

The rate surveys and comparisons indicate that Sacramento's current total sewer user fee for a single-family residence is already high when compared to

the user fees of other California cities and districts. Conversely, Sacramento's connection charge is low compared to these same cities.

TABLE 28. SELECTED CALIFORNIA CITIES: UTILITY USER FEE COMPARISON[1]

City	Sanitary Sewer	Storm Drainage	Water[2]	Solid Waste[3]	Total
Bakersfield	$8.50	$ --	$13.79	$8.92	$31.21
Concord	$13.00	$ --	$39.25	$24.05[4]	$76.30
Fresno	$5.46	$ --	$8.84[5]	$14.45[4]	$28.75
Oakland	$9.36[2]	$ --	$21.30	$24.76[4]	$55.42
San Francisco	$31.34[2]	$ --	$16.20	$9.38	$56.92
San Jose	$15.30	$27.96	$32.58	$9.80	$85.64
Stockton	$12.99	$2.10	$11.96	$6.92	$33.97
Sacramento	$15.15	$9.06	$10.37	$8.30[6]	$42.88
Average	$13.89	$13.04	$19.29	$13.32	$51.39

1 Typical single-family residence
2 Based on 15 hcf usage per month where applicable.
3 One standard (32-gallon) can.
4 Two-can option or equivalent.
5 Flat rate, up to 6,000-sq.ft. lot.
6 One 90-gallon can.
Source: Bartle Wells Associates telephone survey (April 1992).

However, when looking at four utility services (sanitary sewer, storm drainage, water, and solid waste), Sacramento's combined monthly user fees are lower than the average of those for the surveyed cities.

K. CONCLUSIONS AND RECOMMENDATIONS

- The recommended plan (Separate Sanitary Sewer Alternative) would cost approximately $381.4 million (1992 dollars). Construction and financing would be phased over either a 27-year period ending in 2019/20 or a 15-year period ending in 2007/08.

- An analysis was made of the rate impacts of implementing the Separate Sanitary Sewer Alternative as the recommended plan. Two construction phasing options were evaluated, applying rate increases on the basis of four different rate options. User rates in the analysis do not consider potential increases in other utility rates (such as regional sewer, water, flood control, and solid waste), that also have an effect on utility rate affordability. Consequently, even though comparisons made in this report indicate that a phasing option appears affordable, the City must consider the user rate impacts of all its projects before making a final decision.

- Funding of the recommended plan using 25-year revenue bonds would be affordable under Rate Options 1, 3, and 4 but not under Rate Option 2 (paid by combined system users only), in comparison with the EPA Threshold Affordability Index. However, the index is only a guide. The EPA recognizes that project affordability depends on a community's individual mix of public expenditures and the relative importance of sewer-related projects.

- The 27-year construction phasing schedule provides a more affordable annual financing plan for system users. It also results in spreading the costs of such improvements to more future users, namely those paying their share of debt service costs for the 12-year period after the year 2028/29 when 15-year construction phasing debt service would be retired.

- The 27-year construction phasing schedule would allow the City the flexibility to accelerate its construction schedule at any time for reasons such as an improvement in affordability due to such factors as:

  Significant connection charge revenue
  Significant state loans from the State Revolving Fund

- The 15-year construction phasing schedule results in a less affordable financing plan for system users in the early years, but also results in

a reduction of total debt service costs due to lower inflated construction costs, bond issue size, and total interest.

- The City should consider adopting a connection charge for both sanitary and storm drainage systems designed to ensure that new development pays its share of project costs.

- Upon approval of the recommended plan by the City Council, a State Revolving Fund loan application should be submitted to the Regional Water Quality Control Board as the first step for inclusion on the State's priority list. Approval of a loan (limited to $20 million per entity) would further mitigate the financial impact on user fees.

- The comparable-agency rate study shows that the City's total sewer user fees are already high compared to other selected California cities.

- A survey of four utility service fee totals shows the City's fees to be lower than the average for other California cities.

APPENDIX A

CITY OF SACRAMENTO COMBINED SEWER SYSTEM
CUSTOMER DATABASE
DECEMBER 1989 THROUGH SEPTEMBER 1991

Bartle Wells Associates

CITY OF SACRAMENTO COMBINED SEWER PROJECT
CUSTOMER DATA BASE

SEWER CUSTOMERS	Accounts December 1989	Accounts September 1990	Accounts September 1991	Units/multi.	Rate	Estimated Monthly Revenue Sept. 1991
Single-family						
*1-3 rooms	977	976	965	965	4.64	$4,478
*4-5 rooms	23,828	23,801	23,481	23,481	5.89	138,303
*6-7 rooms	34,279	34,485	34,501	34,501	7.07	243,922
*8-9 rooms	742	951	1,309	1,309	8.18	10,708
*10-15 rooms	47	99	200	200	9.38	1,876
over 15 rooms	21	3	11	11	0.66	7
Sub-total	59,894	60,315	60,467	60,467		399,294
Multi-family						
*1-3 rooms	3,062	3,018	2,956	12,895	4.64	59,833
*4-5 rooms	5,392	5,387	5,326	16,727	5.89	98,522
*6-7 rooms	839	842	852	1,513	7.07	10,697
*8-9 rooms	10	12	15	25	8.18	205
*10-15 rooms	9	6	5	245	9.38	2,298
Sub-total	9,312	9,265	9,154	31,405		171,554
Total residential	69,206	69,580	69,621	91,872		570,848
Commercial						
Bakery	1	1	1	2	10.23	20
Barber	105	94	85	64	5.49	351
Dining	155	143	125	201	11.00	2,211
Drug store	13	12	11	23	3.60	83
Furniture	1	1	0	0	1.18	0
Garage	21	18	19	84	0.97	81
Halls	56	54	52	252	3.18	801
Hotel	95	86	84	1,355	1.09	1,477
Market	58	57	50	107	4.60	492
Office	726	640	610	2,652	3.71	9,839
Church	146	142	138	912	1.63	1,487
Rest home	5	6	6	117	1.75	205
School	76	76	75	360	9.73	3,503
Service station/car lot	33	28	25		varies	145
Store	787	687	630	1,899	3.83	7,273
Theater	1	1	2	13	4.52	59
Warehouse	469	423	419	3,772	0.92	3,470
Total commercial	2,748	2,469	2,332	11,813		31,498

Metered sewer use						
5/8"	23	134	83	(1)	6.89	1,200
3/4"	13	63	26	(1)	6.89	925
1"	197	345	625	(1)	7.46	13,000
1 1/2"	101	220	350	(1)	12.74	13,300
2"	178	319	441	(1)	21.29	54,600
3"	46	63	128	(1)	42.47	46,000
4"	20	38	84	(1)	74.37	33,500
6"	24	31	25	(1)	148.65	25,000
8"	3	5	6	(1)	265.38	11,400
10"	0	0	2	(1)	371.66	1,800
12"	0	0	1	(1)	530.93	2,200
Total metered	605	1,218	1,771			202,925
Air conditioning systems						
no conser. device	62	56	54	(1)	9.17	3,200
conser. device	0	100	98	(1)	0.89	3,100
Evaporation coolers						
no conser. device	0	7	7	(1)	1.23	50
conser. device	0	63	52	(1)	0.35	200
Refrigeration system						
no conser. device	36	34	30	(1)	9.17	600
conser. device	24	26	23	(1)	0.89	200
Swimming pool	2	2	1	(1)	(1)	15
Dining facility	90	88	86	152	7.11	1,081
Gymnasium	17	17	17	673	3.29	2,214
Hydraulic elevator	1	1	1	1	28.24	28
Total sewer utility	72,791	73,661	74,093			$815,959
Estimated Annual Revenue						$9,792,000

(1) Varies by usage

STORM DRAINAGE CUSTOMERS	Accounts December 1989	Accounts September 1990	Accounts September 1991	Units/multi.	Rate	Estimated Monthly Revenue
Single-family						
*1-3 rooms	4,323	1,166	1,146	1,146	6.02	$6,899
*4-5 rooms	38,010	31,592	31,284	31,284	7.67	239,948
*6-7 rooms	50,012	49,547	50,263	50,263	9.06	455,383
*8-9 rooms	9,108	9,538	10,282	10,282	10.70	110,017
*10-15 rooms	655	701	859	859	12.20	10,480
over 15 rooms	234	172	172	172	0.95	163
Sub-total	102,342	92,716	94,006	94,006		822,891
Multi-family						
*1-3 rooms	14	3,214	3,182	15,231	6.02	91,691
*4-5 rooms	21	6,400	6,378	32,601	7.67	250,050
*6-7 rooms	6	1,009	1,022	2,413	9.06	21,862
*8-9 rooms	0	29	34	35	10.70	375
*10-15 rooms	0	12	14	19	12.20	232
over 15 rooms	12	10	10	435	0.95	413
Sub-total	53	10,674	10,640	50,734		364,622
Total residential	102,395	103,390	104,646	144,740		1,187,512
Commercial						
minimum rate	2	7	7	7	6.71	47
gross surface area	5,390	5,431	5,553	28,837	15.43	444,955
Total commercial	5,392	5,438	5,560	28,844		445,002
Total storm drain utility	107,787	108,828	110,206	173,584		$1,632,514
Estimated Annual Revenue						$19,590,000

APPENDIX B

STATE REVOLVING FUND (SRF)
ADMINISTRATION AND OPERATIONAL REQUIREMENTS

APPENDIX B

CONTENTS

- Policy Manual for Implementing SRF
- Sample SRF Loan Contract Between State Water Resources Control Board (SWRCB) and Loan Recipient
- Loan Contract Standard Conditions
- Memorandum from SWRCB to Regional Officer Dated January 8, 1992 Discussing 1993 SRF Priority List and Application Deadlines
- Memorandum from City to Regional Board Requesting Addition of CSO Improvement Project to Status Priority List

BARTLE WELLS ASSOCIATES
APR 15 1991
File

State Water Resources Control Board

POLICY FOR IMPLEMENTING THE STATE REVOLVING FUND FOR CONSTRUCTION OF WASTEWATER TREATMENT FACILITIES

"Loans for clean water"

S R F

AUGUST 18, 1988
AMENDED SEPTEMBER 20, 1990

DIVISION OF CLEAN WATER PROGRAMS

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I. INTRODUCTION

The Federal Clean Water Act provides for the creation of a State Revolving Fund (SRF) Loan Program capitalized in part by federal funds. The Federal Clean Water Act authorizes loan funding for construction of publicly owned treatment works (POTWs), for implementation of a nonpoint source pollution control management program, and for the development and implementation of an estuary conservation and management program. This policy document only addresses the issuance of loans for wastewater treatment works. Separate programs will be developed to implement loan funding for the other two possible uses. The SRF is intended to provide loans in perpetuity for construction of POTWs, and implementation of nonpoint source and estuary conservation projects using state and federal funds.

On November 15, 1987, the State Board approved a Concept Paper for conversion of California's grant program to the SRF Loan Program. The Concept Paper addressed the major issues facing the State Board and the schedule for implementing the new loan program. The schedule approved by the State Board calls for the conversion to a full loan program on October 1, 1988. Federal Fiscal Year (FFY) 1989 is the first year that the states can convert 100 percent of their funds to the loan program.

II. PURPOSE AND OBJECTIVE

The primary purpose of the SRF Loan Program is to implement the Federal Clean Water Act and various state laws including the Clean Water Bond Law of 1984, by assisting in the financing of POTWs necessary to prevent water pollution and thereby protect and promote the health, safety and welfare of the inhabitants of the state.

III. DEFINITIONS

The following words where used in this policy shall have the meaning hereafter ascribed to them:

(a) "Allowance" means an amount based on a percentage of the accepted bid for an eligible project to help defray the planning, design, and construction engineering and administration costs of the project.

(b) "Applicant" or "loan applicant" means a city, town, district, or other public body (including an intermunicipal agency of two or more of the foregoing entities) created under state law, or an Indian tribe or an authorized Indian tribal organization having jurisdiction over disposal of sewage, industrial

wastes, or other waste, or a designated and approved management agency under Section 208 of the Act.

(c) "Areawide Waste Treatment Management Plan" means a plan prepared in conformance with Section 208 of the federal Clean Water Act.

(d) "Binding loan commitment" means the date the loan contract is signed by the Department of General Services.

(e) "Completion of Construction" means the date, as determined by the Division after consultation with the loan recipient, that the work of building and erection of the project is substantially complete.

(f) "Division" means the Division of Loans and Grants, State Water Resources Control Board, or any other segment of the State Board authorized to administer the State Revolving Fund Program.

(g) "Eligible project cost" means that portion of the total cost of construction of a project which is eligible for loan assistance from the State Revolving Fund pursuant to federal and state laws, rules, regulations, policies and guidelines.

(h) "Environmental Document" means either an initial study or an Environmental Impact Report (EIR) prepared in accordance with the California Environmental Quality Act (CEQA).

(i) "EPA" means the United States Environmental Protection Agency.

(j) "Estuary enhancement project" means any programs, devices, methods, or systems used to attain or maintain water quality in an estuary nominated by the Administrator of EPA which assures protection of public water supplies and protection and propagation of a balanced, indigenous population of shellfish, fish, and wildlife, and allows recreational activities in and on the water.

(k) "Excessive infiltration/inflow" means a flow rate in excess of acceptable threshold values as defined by federal regulations (currently see 40 CFR 35.2120).

(l) "Facilities plan approval" means approval by the Division of the project concept being proposed by the loan applicant.

(m) "Federal Clean Water Act" means the Federal Water Pollution Control Act (33 U.S.C.A. 1251 et seq.) including any amendments thereto.

(n) "Initiation of construction" means the date that the notice to proceed with the work is issued for the project, or if the notice is not required, the date of commencement of building and erection of the project.

(o) "Loan contract" means the written agreement, and any amendments thereto, between the State Board and a loan applicant in which the terms, provisions, and conditions governing the SRF loan are stated.

(p) "Loan recipient" means an applicant who has executed a loan contract.

(q) "Nonpoint source project" means any programs, devices, methods, or systems used for preventing abating, reducing, transporting, separating, storing, treating, recycling, reclaiming, or disposing of pollutants from nondistinct, unconfined sources, including return flows from irrigated agriculture.

(r) "Preliminary loan commitment" means a statement in a Facilities Plan Approval Letter which indicates the estimated amount of loan assistance for the project.

(s) "Project completion" means the date, as determined by the Division after consultation with the loan recipient, that operation of the treatment works is initiated or is capable of being initiated, whichever comes first.

(t) "Project" means any distinguishable segment or segments of a POTW which can be bid separately, and for which loan assistance is being requested or provided.

(u) "Regional Board" means the appropriate California Regional Water Quality Control Board.

(v) "State Board" means the State Water Resources Control Board.

(w) "Storm drainage project" means any programs, devices, methods, or systems used for preventing, abating, reducing, transporting, separating, storing, treating, recycling, or disposing of pollutants arising or flowing in storm drainage that is transported in pipes, culverts, tunnels, ditches, wells, channels, conduits, from urban or rural areas to surface or ground waters of the state.

(x) "Treatment works" has the same meaning as in the Federal Clean Water Act.

(y) "Water Quality Assessment" means a report prepared by the State Board to identify the water quality conditions in the waters of the state.

(z) "Water Quality Control Plan" means a State Board approved plan adopted pursuant to Division 7 of the Water Code designating or establishing beneficial use and water quality objectives for water within a specified area and a program of implementation needed to achieve these objectives.

IV. PRIORITY SYSTEM

The primary purpose of this section is to implement a Priority System for providing SRF loan assistance for the planning, design, and construction of wastewater treatment, nonpoint source, storm drainage, and estuary enhancement projects and programs eligible under Title VI of the federal Clean Water Act.

A. Implementation and Objectives

The primary method of implementing the SRF Loan Program is through the administration of Title VI of the federal Clean Water Act. The primary objective of the Priority System is to rank treatment works, storm drainage, nonpoint source, and estuary enhancement projects and programs such that activities receiving assistance contribute to the protection of the health, safety, and welfare of the inhabitants of the state.

B. Development of Regional Board Project Priority List

1. Annually, each Regional Board Executive Officer shall develop a Project Priority List for the Region. With the concurrence of the Regional Board, the Executive Officer may directly transmit the list to the Division.

2. Regional Board Project Priority Lists shall be transmitted to the Division no later the April 1 of each year.

3. Such lists shall be consistent with all applicable water quality control plans.

C. Development of Statewide Project Priority List

Annually, after review of the Regional Project Priority Lists, the State Board shall adopt a statewide Project Priority List (Statewide List). The Statewide List shall identify those projects for which assistance from the SRF Loan Program is expected during the succeeding five-year planning period.

1. The Statewide List shall be adopted by the State Board not later than September 15 of each year.

2. The fundable portion of the Statewide List shall include those wastewater treatment, nonpoint source, storm drainage and estuary enhancement projects scheduled for planning, design, and/or construction loan assistance during the first year of the five-year planning period. The extended portion of the Statewide List shall include all projects scheduled for planning, design, and/or construction loans during the following four years.

3. Each project on the Statewide List shall be in conformance with applicable Water Quality Control Plans and with any applicable Areawide Waste Treatment Management Plan.

4. Placement of a project on the Statewide List shall not constitute a commitment to provide loan assistance.

D. Amendment of Statewide Project Priority List

The State Board may modify the Statewide List at any time.

E. Priority Classes

Each Project shall be assigned to one of the following priority classes:

1. Class A -- Public Health Problems. a) Wastewater treatment works projects required to alleviate significant and documented public health hazards involving demonstrated contamination as defined in Section 13050(k) of the Water Code, and where a Regional Board has, adopted a prohibition for elimination of discharges from individual treatment systems and such prohibition has been approved by the State Board, or approved a local moratorium prohibiting the construction of new

individual systems (see Appendix A), or adopted a cease and desist order; and b) nonpoint source, storm drainage pollution and estuary enhancement projects required to comply with prohibitions, postings, limitations, or warnings that have been imposed by responsible health authorities, and where the Regional Board has concurred with the findings of the health authority and has established a time schedule for correction or elimination of the discharge.

2. Class B -- Pollution of Impaired Water Bodies. Projects required to correct conditions where the water quality objectives for an impaired water body are not being attained. The project must improve the quality of a water body listed in the State Board's Water Quality Assessment as an impaired water body.

3. Class C -- Compliance With Requirements. Projects necessary to comply with waste discharge requirements or other formally imposed regulatory requirements imposed by the State Board or Regional Board. Projects necessary for correction of threatened violations of existing waste discharge requirements.

4. Class D -- Project Serving as Preventative Measures Against Additional Water Quality Degradation for Impaired of Unimpaired Water Bodies. Projects which would control discharges to impaired or unimpaired waters, where correction of such discharges may, or may not be required through formally adopted waste discharge requirements. Includes projects to provide additional wastewater treatment capacity.

5. Class E -- Water Reclamation Projects. Projects which reclaim water in a cost-effective manner for beneficial uses.

6. Class F -- Other Projects. Projects not included in any of the other priority classes.

F. Project Ranking

Projects within each priority class shall be ranked on the basis of readiness to proceed. The project with the earliest estimated date for award of a loan contract will be ranked above a project with a later estimated date. In the case of a tie, the project

discharging to the water body with the greater resource value will be ranked higher.

G. Restrictions and Adjustments

1. If a project falls in more than one priority class, the full project shall be placed in the highest priority class applicable to the project, except as specifically ordered by the State Board.

2. If the priority classification of a project is in any way dependent upon state, county, or local action, or upon State or Regional Board action, only action taken prior to the adoption or amendment to the Priority List will be considered.

3. The Priority List may be adjusted by the State Board at its option for good cause subject to approval by EPA.

4. The State Board reserves the right to transfer treatment works from one priority class to another priority class, to change project ranking, to reduce the eligible cost of any project, and to allocate available funds among one or more priority classes when the State Board determines such action to be necessary or appropriate for effective and equitable use of available monies. Such action will only be taken after public hearing.

5. When appropriate, the State Board may create a set-aside for the purposes of assuring that SRF assistance will be available for nonpoint source, estuary enhancement, and storm drainage projects and programs contained on the fundable portion of the Priority List.

H. Management of the Priority List

Before a concept approval is given, a project implementation schedule shall be proposed by the applicant and approved in writing by the appropriate Division. The appropriate Division shall monitor and compare progress on the project to the established schedule to assure that the loan recipient is proceeding expeditiously with the project.

If at any time the appropriate Division determines that progress has slipped sufficiently to push the award of a loan beyond the end of the scheduled federal fiscal year, the State Board may add a project, or projects of

approximately equal value from the planning portion of the Statewide List may be added to the fundable portion of the Statewide list. After such additions, all projects on the Statewide List will continue to compete on an as ready basis for available funds.

I. Funding of Projects

Except as may from time to time be directed by the State Board, projects on the fundable portion of the Statewide List will receive funding on an as ready basis.

J. Project Removal and Changes

Projects shall not be removed from the Priority List unless:

- The State Board so instructs; or
- The project has received a binding loan commitment.

V. OVERALL LOAN PROGRAM PROCEDURES

In order to obtain a loan, the following documents must be submitted:

- A complete Project Report;
- A complete Environmental Document for the project;
- An acceptable draft Revenue Program;
- Final plans and specifications for the project, and an engineer's estimate of project costs;
- Documentation that the MBE/WBE positive effort steps have been addressed in the award of subagreements for the project.

The loan applicant will be required to submit the Project Report, Environmental Document, and the draft Revenue Program to obtain early Board approval of the loan. Once these documents are approved, the Division will prepare an initial loan approval package for the State Board's consideration. For projects with estimated costs greater than $25 million, the initial State Board approval will set the due date for the first repayment and the length of the repayment period.

The actual loan contract will not be offered until the prospective loan applicant has completed facilities planning and received facilities plan approval, applied for and received initial State Board approval of the loan, completed project design, received plans and specifications approval and anticipates bid solicitation in the immediate future. The loan application and a portion of the supporting documentation (final plans and specifications and acceptable MBE/WBE documentation) can be submitted 60 days in advance of the date when the loan applicant expects to advertise for bids for the project provided the State Board has previously approved the project for a loan.

Because this will be a loan program rather than a grant program, the review given to a project will not be to the same level of detail as in the grant program. The State Board does, however, have the obligation to assure that proposed projects are cost effective, likely to meet project performance standards and effluent limitations and are in compliance with all other requirements mandated by the Federal Clean Water Act. Minimal reviews will take place as necessary to provide such assurances.

The Division will expeditiously review, comment on, and approve, conditionally approve, or disapprove all facility planning documents. Approval or conditional approval will be set forth in a Facilities Plan Approval Letter issued by the Division. After Facilities Plan Approval, the Division will take the project to the State Board for the initial loan approval. The actual loan contract will be prepared and transmitted to the loan applicant for signature at the time of the approval to award the construction contract. Appendix B is a flow diagram of the overall loan application procedure.

The major decisions regarding project eligibility determinations will be at the facilities planning stage. After the State Board has made the initial loan approval, subsequent design reviews will be for the purpose of making sure the proposed design is conceptually the same as the one approved by the State Board.

The loan program will be structured so that assistance and review by the Division is available at the option of the prospective applicant. Prospective applicants are strongly urged to take advantage of Division services which are available to minimize the risk of possible project delay due to failure to meet loan program requirements. The possibility of problems can be minimized through early consultation with the Division.

VI. REFINANCING EXISTING DEBT OBLIGATIONS

Under Section 603(d)(2), the SRF may be used to buy or refinance local debt obligations at or below market rates where the debt was incurred after March 7, 1985. In order to be eligible for refinancing, project construction must have been initiated after October 1, 1988. Agencies requesting refinancing must meet all the requirements contained in this Policy, including being reachable on the fundable portion of the Priority List before assistance will be offered. Agencies planning to ask for refinancing of a future project should submit required documents for our review as the project proceeds. Refinancing will not be offered to projects that fail to meet the requirements contained in this Policy. Only local debt obligations for an eligible wastewater facility can be refinanced.

VII. FACILITIES PLANNING

Loan applicants will be required to complete a facilities planning process. A complete facilities plan will include a Project Report, a complete Environmental Document and a draft Revenue Program.

To facilitate expeditious loan approval, the Division will offer consultation regarding specific program requirements during facilities planning. When a municipality is placed on the fundable portion of the Priority List, the Division will issue a letter offering assistance at the option of the prospective loan applicant. The assistance available will include guidance for (1) identifying project alternatives, (2) selecting the cost-effective alternative, (3) preparing the Project Report, the Revenue Program and the Environmental Document. The Division will also offer assistance to aid municipalities in assuring that they have the ability to administer and manage the construction of the proposed treatment works.

A. Project Report

The facilities planning process is the first major step toward award of loan assistance. The Project Report is the loan applicant's record of why the selected project best meets local needs. An effective, well thought-out Project Report is a very necessary element of a successful project.

A Project Report shall be prepared and submitted as part of the loan application process. While early submittal is not a program requirement, the Division strongly recommends that loan applicants submit the Project Report for review well in advance of commencing design.

The Project Report must contain, as appropriate, the following:

1. A statement of project needs and benefits, including a discussion of the water quality benefits of the project or the public health or water quality problems to be corrected.

2. A cost effectiveness evaluation of alternatives. The evaluations presented must include an evaluation of the alternative of upgrading operation and maintenance of the existing facility to improve effluent quality [currently see 40 CFR 35.2030 (b)(3)(iii)].

3. An evaluation of alternative methods for reuse or ultimate disposal of treated wastewater and sludge material resulting from the treatment process [currently see 40 CFR 35.2030 (b)(3)(iv)].

4. An evaluation of opportunities to reduce the use of, or recover, energy [currently see 40 CFR 35.2030(b)(3)(vi)].

5. An evaluation of the potential for construction of revenue producing facilities [currently see 40 CFR 35.2030(b)(3)(v)].

6. An evaluation of the potential open space and recreation opportunities associated with the project [currently see 40 CFR 35.2030(b)(5)].

7. A cost-effectiveness analysis of the feasible conventional, innovative and alternative wastewater treatment works, processes and techniques capable of meeting effluent, water quality, and public health needs and requirements over the design life of the facility. The analysis must recognize environmental and other nonmonetary considerations in arriving at the cost-effective solution [currently see 40 CFR 35.2030(b)(3)].

8. An evaluation of the non-existence or possible existence of excessive infiltration/inflow in the existing sewer system [currently see 40 CFR 35.2030(b)(4)].

9. Cost information on total capital costs, and annual operation and maintenance costs, as well as the estimated annual or monthly costs to residential and industrial users for all of the alternatives [currently see 40 CFR 35.2030 (b)(8)(ii)].

10. A discussion of the population projections, flows and loadings used to estimate needed future capacity.

11. A discussion of the anticipated eligible capacity for the project and how this capacity was derived (see Section VII.B.1. of this policy).

12. A description of the Best Practicable Wastewater Treatment Technology [currently see 40 CFR 35.2005(b)(7)].

13. A summary of public participation required as part of the environmental review process.

14. The following must be submitted for the selected alternative:

    a. A detailed description of the selected alternative and the complete waste treatment system of which it is a part.

    b. A summary of relevant design criteria (i.e., design flow, peak flows, daily BOD loadings, daily suspended solids loadings, overflow rates, detention times, sludge production, etc.).

    c. The estimated capital construction and annual operation and maintenance costs and a description of the anticipated manner in which all the costs will be financed.

    d. A summary of the cost impacts on wastewater system users.

    e. A summary of the energy requirements of the selected project.

    f. A summary of the significant environmental impacts of the selected project and any proposed mitigation measures.

    a. A copy of any proposed intermunicipal service agreements necessary for the project.

    h. Loan applicants must comply with the Civil Rights Act of 1964. Where minority populations are included in the facilities planning area, the Facility Plan must show that such areas will be served or excluded from service only for reasons of cost-effectiveness (currently see 40 CFR Part 7).

i. A description of operation and maintenance requirements.

j. A demonstration that the selected alternative is consistent with any applicable approved water quality management plan (currently see 40 CFR 35.2102).

k. A summary of public participation.

The Division will review the Project Report, the Revenue Program, the environmental documents, and where appropriate, issue a Facilities Plan Approval Letter and take the project to the State Board for approval of the loan. The loan amount approved will be based on the engineer's estimate of the eligible costs. Only in the event of a major cost increase (in excess of 50 percent of the engineer's estimate) in the project would it be necessary to go back to the State Board to reaffirm the approval.

The actual loan contract will be prepared and transmitted to the loan applicant for signature when the project reaches the approval to award stage.

B. Eligible Project

The categories of wastewater treatment projects eligible for loan assistance are as follows:

Secondary Treatment .....Category I
Advanced Treatment.....Category II
Infiltration/Inflow Correction.....Category IIIA
Major Sewer System Rehabilitation....Category IIIB
New Collection Systems.....Category IVA
New Interceptors.....Category IVB
Combined Sewer Overflow Correction...Category V

In accordance with Section 201(g)(1) of the Federal Clean Water Act, annual loan funding available for projects in Categories IIIB, IVA, and V is limited to a maximum of 20 percent of the state's yearly allotment. The State Board will determine annually the amount of funding which will be provided and the categories that will be placed on the fundable portion of the Priority List.

Individual eligibility determinations will be kept to a minimum. Consistent with the State Board's approved November 1987 Concept Paper, the following are categories of eligible and ineligible costs:

Eligible	Ineligible
Treatment works, including new collection systems in existing communities and on-site solids handling facilities	Land and rights of way
Twelve (12) years of reserve capacity for treatment plants and forty (40) years for interceptors and outfalls	Construction contingency
Administration buildings	Construction change orders and claims
Process control systems	Motor Vehicles
Reasonable landscaping measures	House laterals for collection systems, and any inhouse facilities
Mitigation measures (except for land) mandated by state and/or federal agencies	Decorative items (Art work, sculptures, reflective ponds, fountains, etc.)
Allowance for planning, design and construction engineering services and administration	

Cost categories, other than reserve capacity are intended to be completely eligible or ineligible. Eligible and ineligible items must be segregated and bid separately. Partially eligible items must be bid separately so that the Division can determine the eligible share for each item. Totally eligible as well as ineligible items, can be bid as single items.

1. Capacity Funding Limitations

The eligible capacity shall be determined using average dry weather flow and appropriate peak flows in accordance with population and per capita flow estimates provided by the loan applicant. Project capacity must be consistent with environmental constraints.

Eligible capacity for treatment plants will be up to a period of 12 years from the estimated date of initiation of construction. Allocation between eligible and ineligible costs shall be determined on an incremental basis. Eligible capacity for multiple phased projects shall be determined on a case by case basis.

Eligible capacity for interceptors and outfalls will be up to 40 years from the estimated date of initiation of construction. Allocation between eligible and ineligible costs shall be determined on an incremental basis.

Eligible capacity shall be calculated by multiplying the appropriate local population projection by an appropriate local per capita flow figure. The flow thus calculated will be deemed to include all the eligible project flows (residential, commercial, federal facilities, industrial, and infiltration/ inflow). Eligible capacity will be determined at the facilities planning stage.

The loan applicant will be responsible for documenting, in the facilities plan, the peaking factors used for the project.

Eligible capacity will be adjusted at the time of approval to award to reflect the latest estimated date for initiation of construction.

2. Collection Systems

To be eligible for loan assistance, a sewage collection system must comply with the requirements of Section 211(a) of the Federal Clean Water Act. Construction of new collection systems necessary to serve existing communities will be eligible for loan assistance. Collection systems which will primarily serve undeveloped areas will not be eligible for loan assistance.

3. Population and Flow Projections

Locally approved [i.e. Regional Council of Governments, appropriate local planning agencies, or Areawide Water Quality (208) Management Plans] population and flow projections will be used to determine the eligible project capacity. Depending on the apparent reasonableness of the local projection, the Division may require additional documentation to support the reasonableness of the projection. A discussion of the local projections should be included in the facilities plan.

4. Federal Facilities

Eligible capacity will include flows from federal facilities discharging to municipal systems.

C. Revenue Program

Section 204(b)(1) of the Federal Clean Water Act requires communities to develop user charge systems and to have the legal, institutional, managerial, and financial capability to construct, operate, and maintain the treatment works. [currently see 40 CFR 35.2208, 35.2130, 35.2140, 35.2214. 35.2122, 35.2110 and 35.2206(a)]. The State Board will adopt revised Revenue Program Guidelines for the SRF program. The loan applicant will be required to prepare a Revenue Program in accordance with the Revenue Program Guidelines (see Appendix C), adopt a sewer use ordinance (currently see 40 CFR 35.2208), and establish an acceptable dedicated source or sources of revenue to repay the loan.

The loan applicant will be required to identify and make projections of the amount of revenue available from specific sources necessary to repay the loan.

A draft Revenue Program must be prepared and submitted with the Project Report. The draft Revenue Program shall be updated as appropriate prior to submission of the formal loan application. Facilities Plan Approval will not be issued until the Division has approved the draft Revenue Program.

As indicated, the loan recipient will be required to demonstrate, at the time of the actual loan application (at the approval to award stage), that a "dedicated" source of revenue is available to repay the loan. Revenue will be considered dedicated when the local community passes an ordinance or a resolution committing a source or sources of funds for repayment. The resolution or ordinance dedicating a source of funding for repayment of the loan must be adopted before issuance of the loan contract.

A final Revenue Program will be due at the time of 90 percent pay out of the loan funds. The final approved Revenue Program should be reviewed periodically during the useful life of the project and modified as necessary.

D. Environmental Impact Analyses

The Federal Clean Water Act requires that municipalities applying for loans complete an environmental review process similar to that required by the National Environmental Policy Act (NEPA).

All applicants for loans must meet the requirements of the California Environmental Quality Act (CEQA). CEQA requires that all public agencies applying for financial assistance

must prepare either an Initial Study and Negative Declaration or an Environmental Impact Report (EIR).

The loan applicant must submit the draft environmental documents to the State Clearinghouse for comments. The documents must also be distributed for review and comment to other responsible (as defined in CEQA) local, state, and federal agencies within the project's service area. The Division, as a responsible agency, will review and comment on the draft documents received from the State Clearinghouse.

As a minimum, the appropriate environmental document shall discuss impacts on:

1. Threatened or endangered species;
2. Wetlands;
3. Farmland;
4. Fish and wildlife;
5. Cultural resources;
6. Open spaces;
7. Environmentally sensitive areas;
8. Air quality;
9. Water quality;
10. Consistency with land use plans;
11. Energy use;
12. Long-term versus short-term trade offs;
13. Growth inducement;
14. Any other significant environmental effects of the proposed project.
15. Floodplain
16. Consistency with areawide planning such as basin plans and Section 208 plans.

The Division shall review the final environmental documentation to determine if it meets all the environmental review requirements. Once these requirements are met, the Division will prepare a summary of the environmental impacts for inclusion in the State Board agenda item for approval of the loan. At the same time, the Division will prepare a Notice of Decision to issue the loan for adoption by the State Board. The adopted notice will then be filed with the Governor's Office of Planning and Research (Clearinghouse) and the appropriate County Clerk.

Public participation is required on all projects before issuance of the loan. Loan applicants must hold at least one public hearing on the draft Project Report and Environmental Document. A loan will not be issued until this public participation requirement has been fulfilled.

Loan applicants are also required to carry through with feasible mitigation measures agreed upon during the environmental review process. The Division will condition the loan contract to require compliance with agreed upon mitigation measures.

In the event that a loan applicant proposes to use a previously prepared EIR, they must follow the review procedures in Section 15153(b) of the State EIR Guidelines. In brief, this section requires that the loan applicant review the proposed project with an Initial Study to determine if the earlier EIR adequately covers the environmental setting, significant impacts, and the alternatives and mitigation measures related to each significant impact. If the earlier EIR is adequate in these areas, the loan applicant shall use that EIR as the draft EIR for the proposed project and submit it for the usual review procedures for any draft EIR. It is also possible for the loan applicant to use an earlier EIR as part of an Initial Study to document a finding that a later project will not have a significant effect. In this case, a Negative Declaration shall be prepared.

VIII. PLANS AND SPECIFICATIONS

The primary focus of the review of the final plans and specifications will be to conduct a cursory check to make sure that the project, as designed, is consistent with the project approved in facilities planning. To accomplish this, a cursory plans and specifications check will be done to determine if the design is consistent with the facilities described in the Facilities Plan, that the proposed project, in concept, will meet discharge requirements, and that it will comply with applicable federal and state loan program requirements.

A municipality, at its option, may submit ten percent plans for review if it would like an initial review prior to finalizing the project plans and specifications. Municipalities may also request, at their option, a detailed design review by the Division with any comments resulting to be forwarded to the municipality and its consultant. Federal regulations require that loan applicants with estimated project costs greater than $10 million conduct a value engineering (VE) study (currently see 40 CFR 35.2114). The VE recommendations shall be submitted with the final plans and specifications. The VE submittal shall describe the VE recommendations that have been implemented. If any VE recommendations are not being implemented, the submittal shall discuss the reasons why.

IX. COMPLIANCE WITH OTHER FEDERAL STATUTES AND AUTHORITIES

In its January 28, 1988 Final Guidance for State Revolving Funds, EPA has stated that a number of other federal laws and authorities will be applied to activities supported with SRF funds directly made available by capitalization grants. Appendix D contains a current list of these other laws and authorities. Appendix D will be updated as necessary to coincide with federal requirements.

Loan applicants will be required, prior to issuance of the loan contract, to certify that they have complied, or will comply, with all the other federal authorities determined by EPA as applying to the loan program.

X. ALLOWANCES FOR PLANNING, DESIGN, AND CONSTRUCTION ENGINEERING AND ADMINISTRATION SERVICES

A fixed amount of loan funds will be provided to the loan recipient to partially offset costs for planning, design, construction management, administration, and prime engineering. The sum of the allowance plus the eligible low construction bid amount will equal the total amount of the loan.

The planning and design allowances, established by the cost curves in Appendix E, are the EPA allowance tables from Appendix B of 40 CFR Part 35. The allowance for construction engineering and administration is based on typical costs experienced on past Clean Water Grant projects. The allowance should not be construed to represent a limit for each project. Needs must be considered on the basis of specific factors associated with that project. Such consideration could result in engineering costs which are appropriate and which are higher or lower than the amount of the allowance. There is a separate table for pipeline projects and one for treatment facilities. The tables in Appendix E also take into account projects that may have received previous grants for planning and/or for design.

The actual allowance will be calculated by a computer program based on the eligible low bid amount. The tables included in this policy should be used to only estimate the amount of the allowance.

XI. LOAN APPLICATION

Appendix F contains a sample loan application. The loan application should be submitted once the Division has issued a Facilities Plan Approval letter and approved the final plans and specifications. The Division has prepared a loan contract checklist that will assure all program requirements

have been met before award of loan assistance. The checklist is contained in Appendix G.

XII. APPROVAL TO AWARD (ATA)

The loan applicant will be required to submit an ATA package to the Division for review and approval prior to the applicant awarding the construction contract.

Appendix H contains the ATA form that must be submitted with the request to award the construction contract.

The Division will use the lowest acceptable bid to determine the final eligible loan amount. The Division will also review the ATA request package to assure that both the loan applicant, its consultants, and its contractors have complied with the MBE/WBE positive effort requirements. Neither the EPA nor the State Board will be involved in resolving bid disputes. Bid dispute resolution will be the sole responsibility of the loan applicant. ATA will not be issued until all bid disputes have been resolved by the loan applicant.

XIII. AWARD OF LOAN ASSISTANCE AND PROJECT CONTINGENCY REQUIREMENT

Once the ATA package has been submitted and is complete, the Division will prepare the loan contract and mail it to the loan applicant for signature. Appendix I contains a sample loan contract.

The amount of the loan will be limited to the sum of the lowest acceptable bid cost of the eligible project, plus an allowance for planning, design, and construction engineering and construction administration costs. At the time of the submittal of the ATA package, the applicant must demonstrate that an amount equal to at least five percent of the estimated project costs will be available for project contingencies. The contingency will not be eligible for loan assistance.

The final eligible project capacity and loan amount will be determined at ATA based on the estimated date for initiation of construction for the project. Once the final loan contract is executed, all project changes that result in cost increases will be the responsibility of the loan recipient. The loan amount will not be adjusted due to project changes after ATA has been issued. The final loan amount, however, may be subsequently adjusted to account for any later resolution of funding disputes decided in favor of the loan recipient.

The loan recipient will have a maximum of 90 days from the signing of the loan contract to award the prime construction contract. An extension may be granted by the Division in the event of unusual circumstances.

XIV. LOAN DISBURSEMENTS

Disbursements to loan recipients will be subject to any EPA requirements or limitations.

EPA plans to make cash available to the state under a Letter of Credit (LOC) approach. Under this approach, the State Board may not receive the federal cash until a cost reimbursement request is received from the loan recipient. Costs may be submitted for payment on a monthly basis. Costs submitted to the State Board for payment must be due and payable to the project contractors. It will not, however, be necessary for the loan recipient to have actually paid the costs before requesting payment under the loan contract.

A. Loan Disbursement Requests

The loan recipient will receive several copies of the SRF Disbursement Request, Form No. 259 (see Appendix J) upon execution of the loan contract. The Division will complete all necessary information on the SRF Disbursement Request Form except for Column G, "Completed To Date" (Units/%) and the authorized representative's signature and date. A copy of each processed request will be sent to the recipient to show the amount paid by the Division.

B. Planning and Design Allowance Requests

A request for disbursement of the planning and design allowance can be submitted as soon as the loan contract is executed. The disbursement amount is based on a calculated amount using the federal planning and design tables. The amount for the planning and design allowance will be shown on the SRF Disbursement Request Form sent to the recipient. The SRF Disbursement Request Form must be signed and dated by the authorized representative.

C. Construction Progress Payment Requests

A request for a construction progress payment can be submitted to the Division on a monthly basis. The recipient will enter the work completed to date in column G of the SRF Disbursement Request Form. The SRF Disbursement Request Form must be signed and dated by the authorized representative. With the submittal, the recipient must

include a copy of the construction contractor's progress pay estimate (signed by both the recipient and the contractor). The loan disbursement will be based on the amount of money currently due and payable to the contractor for eligible bid items minus any amounts previously paid by the Division.

The allowance for construction engineering and administration will be paid as a percentage of the eligible cost owed the contractor to date versus the total eligible low bid (i.e., total for column H/total for column F on the SRF Disbursement Request Form). It is not necessary to include invoices or other documentation for the disbursement for engineering or administration expenses.

The loan recipient should assure that adequate local funding is available to pay the contractor in case the loan disbursement is not processed before payment to the contractor is due.

D. Division Assistance

If you need assistance, or have any questions regarding submittal of a request for a loan disbursement, please contact the Chief of the Payments Unit at (916) 739-4365.

XV. CONSTRUCTION

A. Labor Wage Provisions

The loan recipient must comply with Davis-Bacon wage determinations (currently see 29 CFR Part 5). The final plans and specifications will be reviewed to make sure the proper wage determination provisions have been inserted in the specifications. The loan recipient will be responsible for assuring compliance with applicable labor laws.

B. Oversite Construction Inspections

The Division may conduct oversite inspections during construction. The primary purpose of any inspection will be to ascertain that the loan recipient is constructing the approved project according to applicable contract requirements. The first inspection will normally be held at the 10 percent completion point with subsequent inspections conducted if deemed necessary by the Division.

C. Final Construction Inspection

The Division will conduct a final construction inspection. The primary purpose of the final inspection will

be to ascertain that the proposed project was constructed according to the plans and specifications. At the time of the Division's final inspection, the project completion date will be established for purposes of beginning the one-year project performance certification period.

D. Minority Business Enterprise/Womens Business Enterprise (MBE/WBE)

1. MBE/WBE (Professional Services)

The applicant must comply with federal regulations governing the participation of MBE/WBE firms for all professional services (includes planning, design, and construction). Applicants will be required to complete the Minority and Women's Business Utilization Report contained in Appendix K for professional services. The Report must be filed quarterly according to the instructions contained with the Report. Copies of the completed forms must also be submitted to the Grants Administration Branch (currently see 40 CFR Part 33). MBE/WBE Information (Form No. 4700-5) and Self-Certification forms (see Appendix L) must be submitted with the loan application package.

Loan recipients are expected to meet the fair share MBE/WBE percentages set each year by the Board and EPA. Agencies not meeting these fair share goals must submit documentation which demonstrates compliance with the affirmative steps identified in federal regulation (currently see 40 CFR Part 33.240). Failure to initially comply with MBE/WBE procurement requirements will result in the service or contract in question being identified as ineligible for loan participation.

2. MBE/WBE (Construction)

The applicant must insert the "Contract Provisions Relative to MBE/WBE" (see Appendix L) and Form 4700-5 (see Appendix K) in its contract bidding documents. Furthermore, a prebid conference with the prospective bidders is strongly encouraged to explain the MBE/WBE "positive effort" requirement.

The applicant must submit three MBE/WBE forms with the ATA request package. These are Form 4700-5 (that was submitted with the bid), the MBE/WBE self-certification forms, and the applicant MBE/WBE certification form (see Appendix L). In the event the bidder does not meet the MBE and WBE fair share

goals, or the historical average, whichever is higher, for each construction contract, additional documentation must be submitted to the Division by the loan applicant to demonstrate that the low bidder complied with the positive effort steps listed in the Contract Provisions for MBE/WBE Utilization in Appendix L.

If acceptable positive effort steps have not been taken, the Division will neither authorize the award of the construction contract nor issue the loan contract. Selecting the next low, responsive, responsible bidder, or rebidding the contract, are possible alternatives for the applicant if loan assistance is still desired.

E. Project Performance Certification

The date when project operations are capable of being initiated will be set by the Division after consultation with the loan recipient in order to set the one-year performance certification procedure.

Prior to the loan recipient's performance certification, the Division will conduct a performance certification inspection to make sure all the project facilities are operating and capable of meeting discharge standards.

After one year of operation, loan recipients will be required to submit a certification that the project meets, or fails to meet, performance standards.

Project performance standards include meeting any applicable design specifications and effluent requirements. For nondischarging projects such as interceptor sewers, pump stations, etc., the project performance standards include meeting only the design specifications, and where appropriate, the elimination of overflows and reduction of infiltration/inflow. At the option of the loan recipient, the Division will assist in developing the applicable performance standards.

If the project cannot be affirmatively certified, the loan recipient must prepare a Corrective Action Report which includes an analysis of the cause of the project's failure to meet the performance standards and an estimate of the nature, scope and cost to proceed with the necessary corrective action. The Division will conduct follow-up inspections as necessary to determine if the project meets performance standards. The cost of

the corrective action will not be eligible for loan assistance.

XVI. REPAYMENTS

Interest will accrue on all loan payments as of the date payment is made. Each loan recipient will be given a repayment schedule after completion of contruction. The interest rate is currently set at one-half the rate the state paid on the most recent sale of general obligation bonds.

The Division will prepare a repayment spread sheet that includes:

- The interest rate;
- Amount of dollars loaned;
- Accrued interest;
- The final principal amount of the loan due including accrued interest;
- A complete amortization table; and
- Information regarding penalties for late repayment.

Information on the amount of money necessary to repay the loan in full, at any given point in time, will be available on request.

The first annual loan repayment will be due one year after completion of construction for projects costing less than $25 million. For projects costing greater than $25 million, the date for the first annual repayment and the loan term will be determined by the State Board after facilities planning is completed and after negotiation with the loan applicant. The resulting repayment will begin no sooner than one year after the first loan payment or no later than one year after completion of construction. The loans will be fully amortized not later than 20 years after completion of construction of the project. The amount to be repaid will include the amount loaned plus accrued interest (a sample repayment table is included in Appendix M). The Division will normally send a repayment notice 30 days before the date each repayment is due, but prompt repayment remains the responsibility of the loan recipient.

There will be a penalty for late repayments. The penalty will be one-tenth of one percent (0.1%) per day on the amount due. A 30 day grace period will be allowed. However, if the payment is not received by the end of the grace period, the penalty

will be assessed from the repayment due date. Any penalties collected will be deposited in the SRF account to be made available for loans.

Penalties assessed will not change the principal balance of the loan contract. Such penalties will be treated as a separate account in addition to the annual repayment due.

All repayments are to be sent to:

Chief, Grants Reimbursement Section
Division of Clean Water Programs
P.O. Box 944212
Sacramento, CA 94244-2120

XVII. RECORD KEEPING REQUIREMENTS

Loan recipients are required to maintain separate project accounts in accordance with generally accepted government accounting standards. More specifically, (currently see 40 CFR 30.500), the following records must be maintained:

- Accounts accurately depicting amounts received and expended for the project, including all funds received from the SRF;
- Program income data;
- Total cost of the project;

Appropriate records must be maintained for a period of at least three years after completion of the project.

XVIII. BOARD RESERVATION OF AUTHORITY

Prior to award of any loan contract, the State Board reserves the right to modify this Policy as necessary to provide for effective and equitable use of SRF funds, including:

- Reducing the eligible loan amount for any project; or
- Adjusting the terms for repayment of the loan consistent with applicable statutes.

Any such action will be taken only in a manner consistent with applicable federal and state requirements and after any prospective loan applicant, adversely affected by the action, has had an opportunity to comment on the proposed action.

XIX. DISPUTES

Disputes under this policy are expected to be minimal. Any dispute which is not otherwise resolved shall be referred to the Chief of the Division, or his delegatee, for decision. The decision by the Chief of the Division, or his delegatee, shall be reviewable by the State Board, at the option of the State Board.

In the event that a Division decision is not acceptable to a loan applicant, the applicant must appeal to the State Board within 30 days. The Office of the Chief Counsel of the State Board will prepare a summary of the dispute and make recommendations relative to its final resolution, which will be provided to the State Board's Executive Director and all the State Board Members. Upon the motion of any State Board Member, the State Board will undertake to review and resolve the dispute in the manner determined by the State Board. Should the State Board determine not to review the Division decision, this decision will represent a final agency action on the dispute.

These provisions do not pertain to disputes under an executed loan contract. Such disputes shall be resolved in accordance with the disputes resolution provisions of the contract.

RECEIVED

OCT 1 0 1991

STATE REVOLVING LOAN FUND PROGRAM
LOAN CONTRACT
BETWEEN THE
STATE WATER RESOURCES CONTROL BOARD

This loan contract is made this between the State of California, acting by and through the State Water Resources Control Board, hereafter referred to as the "State Board", and

WHEREAS:

1. The Federal Clean Water Act (33 U.S.C.A. § 1251 et seq.) and State law (Division 7, Chapter 6.5, California Water Code) authorize the State Board to enter into contracts with municipalities and other public agencies for financial assistance for construction of publicly owned treatment works; and

2. The Agency has made application for a loan related to construction of the Project hereafter described, and said Project has been determined by the State Board to be eligible for a loan pursuant to applicable Federal and State laws, rules, regulations and guidance; and

3. The State Board has authorized loan funding for the Project hereafter described. (See State Board Resolution No. 90-89, adopted on August 16, 1990.)

NOW, THEREFORE, It is Agreed as Follows:

SECTION 1. PROJECT DESCRIPTION.

SECTION 2. INCORPORATION OF DOCUMENTS; ORDER OF PRECEDENCE; GENERAL AGENCY COMMITMENTS.

This contract incorporates herein, or by reference, the documents listed below. In the event of any inconsistency in the contract documents, except as otherwise specifically provided, the inconsistency shall be resolved by giving precedence in the following order: (1) the provisions of this loan contract document (2) the Loan Contract Special Conditions (Exhibit A);, (3) the Approved Plans and Specifications and Applicable Effluent Limitations (Exhibit B); (4) the Plans and Specifications Approval Letters (Exhibit C); (5) the Facilities Plan Approval Letter (Exhibit D); (6) the Preliminary Loan Repayment Schedule and any amendments thereto (Exhibit E); (7) the Loan Contract Standard Conditions (Exhibit F); (8) the Nondiscrimination Clause (Exhibit G).

The Agency accepts and agrees to comply with all terms, provisions, conditions, and commitments of this contract, including all incorporated documents, and to fulfill all assurances, declarations, representations, and commitments made by the Agency in its application, accompanying documents, and communications filed in support of its request for loan.

SECTION 3. ESTIMATED COST OF PROJECT.

The estimated total cost of the Project, including associated planning and design costs, is seven million dollars ($7,000,000) to the Agency .

SECTION 4. MAXIMUM LOAN AMOUNT.

Subject to all of the terms, provisions, and conditions of this contract, and subject to the availability of State and Federal funds, the State Board will loan the sum of five million three hundred fifty three thousand six hundred twenty-one dollars ($5,353,621) to the Agency. This amount is based on the revised Approval to Award budget letter dated December 18, 1990.

SECTION 5. INTEREST RATE.

The loan interest rate shall be 3.3 percent per annum. Interest on any loan funds disbursed to the Agency shall begin to accrue as of the date of each disbursement.

SECTION 6. COMPLETION OF PROJECT.

The Agency agrees to expeditiously proceed with and complete construction of the Project in substantial accordance with Project plans and specifications approved by the State Board.

SECTION 7. PROJECT CERTIFICATION.

One year after Project completion, the Agency shall certify to the State Board whether or not the Project, as of that date, meets applicable design specifications and effluent limitations. If the Agency cannot certify that the Project meets such specifications and limitations at that time, the Agency will, at its own expense and in a timely manner, expeditiously make all needed corrections and perform all additional work necessary to allow affirmative certification for the Project.

SECTION 8. DISBURSEMENT.

Loan funds will be disbursed in accordance with the disbursement provisions of Exhibit F attached hereto.

SECTION 9. REPAYMENT OF LOAN.

Loan funds shall be repaid in accordance with the provisions of Exhibit F, attached hereto, and as indicated in appropriate Loan Repayment Schedules in Exhibit E, also attached hereto.

SECTION 10. DEDICATED SOURCE OF REVENUE.

The Agency, in cooperation with the Cities of San Bernardino and Colton, shall at all times maintain a dedicated source of revenue sufficient to provide reasonable assurance of repayment of this loan. The dedicated source of revenue shall comply with the requirements of the federal Clean Water Act and any applicable Federal and State regulations and shall have received State Board approval.

SECTION 11. FINANCIAL MANAGEMENT SYSTEM AND STANDARDS.

The Agency agrees to comply with Federal standards for financial management systems. The Agency agrees that, at a minimum, its fiscal control and accounting procedures will be sufficient to permit preparation of reports required by the Federal government and tracing of loan funds to a level of expenditure adequate to establish that such funds have not been used in violation of Federal or State law or the terms of this contract. To the extent applicable, the Agency agrees to be bound by and to comply with the provisions and requirements of the Federal Single Audit Act of 1984 (Pub. L. 98-502) and Office of Management and Budget (OMB) Circular No. A-128.

SECTION 12. ACCOUNTING STANDARDS.

The Agency will maintain separate Project accounts in accordance with generally accepted government accounting standards including those contained in the "Standards for Audit of Governmental Organizations, Programs, Activities and Functions" promulgated by the U. S. General Accounting Office.

SECTION 13. PROHIBITED CONTRACTS.

The Agency agrees that it will preclude its contractors and subcontractors from contracting with any party which is debarred, suspended, or otherwise excluded from or ineligible for participation in federal assistance programs under Executive Order 12549, "Debarment and Suspension".

SECTION 14. COPYRIGHTS.

The Agency agrees that the U. S. Environmental Protection Agency shall have a royalty-free, nonexclusive and irrevocable license to reproduce, publish, otherwise use, and to authorize others to use, for federal government purposes: (a) The copyright in any work developed under this loan contract; and (b) any rights of copyright which the Agency may purchase where costs of such purchase are reimbursed with loan funds.

SECTION 15. USER CHARGE SYSTEM.

The Agency shall ensure that the Cities of San Bernardino and Colton adopt and maintain in effect a user charge system which at all times complies with the requirements of Section 204(b)(1) of the Federal Clean Water Act and applicable Federal and State rules, regulations and guidelines.

SECTION 16. OPERATION AND MAINTENANCE.

The Agency agrees to properly staff, operate and maintain all portions of the Project during its useful life in accordance with all applicable state and federal laws, rules and regulations.

SECTION 17. USEFUL LIFE OF PROJECT.

For purposes of this contract, the parties agree that the useful life of the Project is 20 years from and after Project completion. The demonstration project will only be operated for one year. However, the RIX demonstration site will either be incorporated into a full-scale RIX project or used as the site for an alternate treatment process to meet the applicable effluent limitations.

SECTION 18. ASSIGNABILITY.

To the extent permitted by Federal and State laws, rules, and regulations, the State Board may sell, pledge, assign, grant a security interest in, or otherwise encumber this contract and any right, or rights hereunder, including any payment or payments to be received hereunder.

SECTION 19. STATE REVIEWS AND INDEMNIFICATION.

The parties agree that review or approval of project plans and specifications by the State Board is for administrative purposes only and does not relieve the Agency of its responsibility to properly plan, design, construct, operate, and maintain the Project. As between the State Board and the Agency, the Agency agrees that it has sole responsibility for proper planning, design, construction, operation, and maintenance of the Project, and the Agency agrees to indemnify the State Board, the State of California and their officers, agents, and employees against and to hold the same free and harmless from any and all claims, demands, damages, losses, costs, expenses, or liability due or incident to planning, design, construction, operation, or maintenance of the Project.

EXHIBIT F

LOAN CONTRACT STANDARD CONDITIONS

ARTICLE 1. DEFINITIONS.

(A) "Allowance" means an amount based on a percentage of the accepted bid for an eligible project to help defray the planning, design, and construction engineering and administration costs of the Project.

(B) "Authorized Representative"

(C) "Completion of construction" means the date, as determined by the Division after consultation with the Agency, that the work of building and erection of the Project is substantially complete.

(D) "Dedicated source of revenue" means and includes special assessments, general taxes, general obligation bonds, revenue bonds, user charges, or other sources of income which are consistent with federal requirements.

(E) "Force Account" means the use of the Agency's own employees or equipment for construction of the Project.

(F) "Division" means the Division of Clean Water Programs of the State Water Resources Control Board, or any other segment of the State Board authorized to administer the State Revolving Fund.

(G) "Initiation of construction" means the date that the notice to proceed with work is issued for the Project, or, if notice to proceed is not required, the date of commencement of building and erection of the Project.

(H) "Official Representatives"

(1) The State Board Loan Coordinator shall be the Division Chief of the Division of Clean Water Programs.

(2) The State Board Loan Coordinator shall be the State Board's representative for administration of the contract and shall have authority to make determinations and findings with respect to any controversy arising under or in connection with interpretation of the contract. Disputes shall be resolved in accordance with SECTION 2 of the signed contract.

(3) The Agency Loan Representative shall be the General Manager (or his designee), who shall administer the contract and who shall have full authority to act on behalf of the Agency, including authority to execute disbursement requests. All communications given to the loan representative shall be as binding as if given to the Agency.

(4) Either party may change its Official Representative upon written notice to the other party.

(I) "Project completion" means the date, as determined by the Division after consultation with the Agency, that operation of the Project is initiated or is capable of being initiated, whichever comes first.

(J) "Revenue Program" means a system of charges, fees, or other means of income production adopted by the Agency which provides for recovery of appropriate capital costs of the Project, generates adequate income to reasonably assure repayment of loan funds under this contract, generates adequate income to provide for reasonable operation and maintenance of the Project, and provides adequate income for reasonable future expansion and improvement of the Project.

ARTICLE 2. COMPLIANCE WITH LAWS, REGULATIONS, ETC.

(A) The Agency agrees that it will, at all times, comply with and require its contractors and subcontractors to comply with all applicable federal and state laws, rules, guidelines, regulations, and requirements. Without limitation of the foregoing, the Agency agrees that, to the extent applicable, the Agency will:

(1) Comply with the requirements of 40 CFR Part 31;

(2) Comply with, implement and fulfill all environmental mitigation measures committed to by the Agency as a part of its request for loan or which may otherwise be required by this contract.

(3) Comply with the State Water Resources Control Board's "Policy for Implementing the State Revolving Fund for Construction of Wastewater Treatment Facilities", August 18, 1988; and

(4) Comply with, and require its contractors and subcontractors on the Project to comply with federal and state minority and women-owned business enterprise (MBE/WBE) requirements.

ARTICLE 3. AWARD OF CONSTRUCTION CONTRACTS; NOTIFICATION OF AWARD AND INITIATION OF CONSTRUCTION.

(A) The Agency agrees to award the prime construction contract for the Project within 90 days after execution of this contract by the Agency.

(B) The Agency agrees to promptly notify the Division in writing both of the award of the prime construction contract for the Project and of initiation of construction of the Project. The Agency shall include with the notice of initiation of construction a statement as to the anticipated date of completion of construction of the Project.

ARTICLE 4. CONSTRUCTION ACTIVITIES; NOTIFICATIONS; PROTECTION OF ARCHEOLOGICAL AND HISTORICAL RESOURCES.

(A) The Agency agrees to promptly notify the Division in writing of:

(1) Any substantial change in scope of the Project. The Agency agrees that no substantial change in the scope of the Project will be undertaken until written notice of the proposed change has been provided to the Division and the Division has given written approval for such change;

(2) Cessation of all major construction work on the Project where such cessation of work is expected to or does extend for a period of 30 days or more;

(3) Any circumstance, combination of circumstances, or condition, which is expected to or does delay completion of construction for a period of 90 days or more beyond the estimated date of completion of construction previously provided to the Division;

(4) Discovery of any potential archeological or historical resource. Should a potential archeological or historical resource be discovered during construction of the Project, the Agency agrees that all work in the area of the find will cease until a qualified archeologist has evaluated the situation and made recommendations regarding preservation of the resource, and the Division has determined what actions should be taken to protect and preserve the resource. The Agency agrees to implement appropriate actions as directed by the Division; and

(5) Completion of construction of the Project.

ARTICLE 5. PROJECT ACCESS.

(A) The Agency agrees to insure that the State Board, or any authorized representative thereof, will have suitable access to the Project site at all reasonable times during Project construction and thereafter for the useful life of the Project.

ARTICLE 6. PROJECT COMPLETION; INITIATION OF OPERATIONS.

(A) Upon completion of construction of the Project, the Agency agrees to expeditiously initiate Project operations. At the time of completion of construction, the Division, after consultation with the Agency, will establish a reasonable estimated Project completion date, and the Agency agrees to make all reasonable efforts to meet the date so established. Such date shall be binding upon the Agency unless modified in writing by the Division upon a showing of good cause by the Agency. Extension of the Project completion date by the Division shall not be unreasonably withheld.

ARTICLE 7. REVENUE PROGRAM.

(A) The Agency agrees to prepare and provide an acceptable final Revenue Program for the full-scale project (revenue program is waived for the demonstration project) to the Division at the time of 90 percent payout of loan funds. Further loan disbursements may be withheld until an acceptable final Revenue Program is submitted. The Agency further agrees to periodically review and modify the Revenue Program as necessary to assure reasonable adequacy of the Revenue Program. The final Revenue Program and all modifications thereof shall be consistent with applicable guidelines and shall be to the reasonable satisfaction of the Division. The Division may review the Agency's records to assure compliance with the approved Revenue Program at any time during the useful life of the Project.

ARTICLE 8. CONTINUOUS USE OF PROJECT; LEASE OR DISPOSAL OF PROJECT.

(A) The Agency agrees that it will not abandon, substantially discontinue use of, lease, or dispose of the Project or any significant part or portion thereof during the useful life of the Project without prior written approval of the Division. Such approval may be conditioned as determined to be appropriate by the Division, including a condition requiring repayment of all or any portion of all remaining loan funds covered by this contract together with accrued interest and any penalty assessments which may be due.

ARTICLE 9. REPORTS.

(A) The Agency agrees to expeditiously provide, during construction of the Project and thereafter during the useful life of the Project,

such reports, data, and information as may be reasonably required by the Division, including but not limited to material necessary or appropriate for evaluation of the State Revolving Fund Loan Program or to fulfill any reporting requirements of the federal government.

ARTICLE 10. LOAN DISBURSEMENTS; AVAILABILITY OF FUNDS.

(A) Except as may be otherwise provided in this contract, loan amounts will be disbursed as follows:

(1) Upon execution and return of this loan contract, the Agency may request immediate disbursement of any planning and design allowance included in the loan amount through submission of Disbursement Request Form 260, or any amendment thereto, duly completed and executed. The planning and design allowance amount due, up to the allowance, will be disbursed promptly upon request after approval of this contract by the State Department of General Services;

(2) Additional loan funds will be promptly disbursed to the Agency on account of disbursement costs incurred by the Agency upon receipt of Disbursement Request Form 260, or any amendment thereto, duly completed and executed, provided that disbursement shall not be made more frequently than once a month. The Agency agrees that it will not request disbursement for any Project cost until such cost has been incurred and is due and payable to Project contractors, although it is agreed that actual disbursement of such cost by the Agency is not required as a condition of disbursement request. If required by the Division, the Agency agrees to provide a certification with each disbursement request that costs shown in the disbursement request have been incurred and are due and payable at the time of the request and that the funds received will all be paid to the contractors and vendors within thirty (30) days from receipt of the funds; and

(3) Notwithstanding any other provision of this contract, no disbursement shall be required at any time or in any manner which is in violation of or in conflict with federal or state laws, rules, or regulations.

(B) The State Board's obligation to disburse any sum to the Agency under any provision of this contract is contingent upon the availability of sufficient funds to permit the disbursements provided for herein. In the event that sufficient funds do not become available for reasons beyond the reasonable control of the State Board, such as failure of the federal or state government to appropriate funds necessary for disbursement of loan amounts, the State Board shall not be obligated to make any disbursements to the Agency under this contract. This provision shall be construed as a condition precedent to the obligation of the State Board to make any disbursements under this contract. Nothing in this contract shall be construed to provide the Agency with a right of priority for disbursement over any other

agency. If any disbursements which are otherwise due to the Agency under this contract are deferred because of unavailability of sufficient funds, such disbursements will promptly be made to the Agency when sufficient funds do become available.

ARTICLE 11. WITHHOLDING OF LOAN DISBURSEMENTS.

(A) The Division may withhold all or any portion of the loan funds provided for by this contract in the event that:

(1) The Agency has materially violated, or threatens to materially violate, any term, provision, condition, or commitment of this contract;

(2) The Agency fails to maintain reasonable progress toward completion of the Project; or

(3) An acceptable Revenue Program is not submitted at the time of 90 percent payout of loan funds for the full-scale project. Submittal of a revenue program is waived for the demonstration project.

ARTICLE 12. RECORDS.

(A) Without limitation of the requirement to maintain Project accounts in accordance with generally accepted government accounting standards, the Agency agrees to:

(1) Establish an official file for the Project which shall adequately document all significant actions relative to the Project;

(2) Establish separate accounts which will adequately and accurately depict all amounts received and expended on the Project, including all loan funds received under this contract;

(3) Establish separate accounts which will adequately depict all income received which is attributable to the Project, specifically including any income attributable to loan funds disbursed under this contract;

(4) Establish an accounting system which will accurately depict final total costs of the Project, including both direct and indirect costs;

(5) Establish such accounts and maintain such records as may be necessary for the State to fulfill federal reporting requirements, including any and all reporting requirements under federal tax statutes or regulations; and

(6) If Force Account is used by the Agency for any phase of the Project, other than for planning, design and construction engineering, and administration provided for by allowance, accounts will be

established which reasonably document all employee hours charged to the Project and the associated tasks performed by each employee. Indirect force account costs may be paid with prior written approval by the Division of the Agency's indirect cost proposal.

(B) The Agency agrees to require Project contractors and sub-contractors to maintain books, records, and other material relative to the Project in accordance with generally accepted accounting standards, and to require such contractors and subcontractors to retain such books, records, and other material for a minimum of three years after Project completion. The Agency further agrees to require that such books, records, and other material shall be subject at all reasonable times to inspection, copying, and audit by the State Board and by the U. S. Environmental Protection Agency, or any authorized representatives thereof.

(C) The Agency agrees to retain its Project records for a minimum of three years after Project completion, and for such longer period as may be required for the State to fulfill federal reporting requirements under federal tax statutes and regulations. All Agency records relative to the Project shall be subject at all reasonable times to inspection, copying and audit by the State Board and the U. S. Environmental Protection Agency, or any authorized representatives thereof.

ARTICLE 13. FINAL PROJECT REPORTS; AUDIT.

(A) Within 90 days after Project completion, the Agency agrees to submit to the Division a final Project summary report which briefly describes the Project as completed, its features, estimated capacity, and apparent treatment capability. The report shall note any unusual operational problems that have been encountered and shall indicate whether the Agency expects to be able to certify that the Project meets applicable design specifications and effluent limitations one year after Project completion.

(B) Within 120 days after Project completion, the Agency agrees to provide to the Division a final cost summary report on the Project. The summary shall include at a minimum, a statement of:

(1) Total Project costs;

(2) Total Project costs which are eligible for loan funding under the Board's loan program and this contract;

(3) The total amount of loan funds received; and

(4) The amount of interest earned, if any, on loan funds before disbursement on account of incurred Project costs. If no interest has been earned, this fact shall be expressly stated.

The report shall be accompanied by such other financial information as may be reasonably required by the Division to verify Agency entitlement to loan funds, to assure program integrity of the State Revolving Fund Loan Program, and to comply with any federal requirements. The report shall be certified as correct by a duly authorized representative of the Agency.

(C) The Division, at its option, may call for an audit of financial information relative to the Project, where the Division determines that an audit is desirable to assure program integrity or where such an audit becomes necessary because of federal requirements. Where such an audit is called for, the audit shall be performed by a Certified Public Accountant independent of the Agency and at the cost of the Agency. The audit shall be in the form required by the Division.

ARTICLE 14. REPAYMENT; PENALTIES; AGENCY OBLIGATION.

(A) The loan amount, together with all interest accruing thereon, shall be repaid in full not later than 20 years after the completion of construction. Repayment shall be made in 20 annual installments, with the first repayment due one year after the completion of construction, with annual repayment installments due thereafter until the loan amount and all accrued interest has been paid in full.

The annual repayments will be based on the following calculations. First, calculate the sum of (1) the disbursements during the year, (2) plus the accrued interest from date of disbursement to the end of the year for each disbursement during the year, plus (3) the remaining balance from the previous year, plus (4) the accrued interest from the beginning of the year to the end of the year for the remaining balance from the previous year. Second, calculate the present worth of this sum back to the beginning of the year using the formula (1) below. Third, calculate the repayment amount using the formula (2) below for a standard fully amortized loan with equal annual repayments. The remaining balance is the previous balance, plus the disbursements, plus the accrued interest on both, less the repayment. Repayment calculations will be made beginning one year after the completion of construction and continuing each year thereafter for 20 years. Exhibit E is a sample repayment schedule based on the provisions of this article and a hypothetical disbursement schedule. The actual repayments will be based on actual disbursements.

(1) $PW = \frac{D}{(1 + i)}$

PW = Present worth
D = Sum of remaining balance from previous year, plus current year disbursements, plus accrued interest on both
R = Repayment amount
i = Interest rate per annum
n = 20 years less the number of repayments scheduled to have been made

(2) $R = PW \frac{i}{1 - (i + 1)^{-n}}$

(B) Upon completion of construction of the Project and submission and necessary reports, the Division will prepare an appropriate Loan Repayment Schedule and supply the same to the Agency. The Loan Repayment Schedule may be amended as necessary to accurately reflect amounts due under this contract. Any amended Loan Repayment Schedule which is necessary will be prepared by the Division and furnished to the Agency.

(C) The Agency agree to make each loan payment on or before the due date therefor. A ten-day grace period will be allowed. A penalty in the amount of one-tenth of one percent (0.1%) of the amount due will be due for each day of nonpayment. For purposes of penalty assessment, payment will be deemed to have been made if payment is deposited in the U. S. mail within the grace period with postage prepaid and properly addressed. Any penalties assessed will not be added to the loan balance but will be treated as a separate account and obligation of the Agency.

(D) The Agency as a whole is obligated to make all payments required by this contract to the State Board, notwithstanding any individual default by its constituents or others in the payment to the Agency of taxes, assessments, tolls, or other charges levied by the Agency. The Agency shall provide for the punctual payment to the State Board of all amounts which become due under this contract and which are received from constituents or others in the payment to the Agency of taxes, assessments, tolls, or other charges levied by the Agency. In the event of failure, neglect or refusal of any officer of the Agency to levy or cause to be levied any tax or assessment necessary to provide payment by the Agency under this contract, to enforce or to collect such tax or assessment, or to pay over to the State Board any money collected on the tax or assessment necessary to satisfy any amount due under this contract, the State Board may take such action in a court of competent jurisdiction as it deems necessary to compel the performance of all duties relating to the levying and collection

of the taxes or assessments and the payment of the money collected therefrom to the State Board. Action taken pursuant hereto shall not deprive the State Board of, or limit the application of, any other remedy provided by law or by this contract.

ARTICLE 15. TERMINATION; IMMEDIATE REPAYMENT; INTEREST.

(A) This contract may be terminated by written notice during construction of the Project, or thereafter at any time prior to complete repayment by the Agency, at the option of the State Board, upon violation by the Agency of any material provision of this loan contract after such violation has been called to the attention of the Agency and after failure of the Agency to bring itself into compliance with the provisions of this contract within a reasonable time as established by the Division.

(B) In the event of such termination, the Agency agrees, upon demand, to immediately repay to the State Board an amount equal to the current balance due on the loan, including accrued interest, and all penalty assessments due. In the event of termination, interest shall accrue on all amounts due at the highest legal rate of interest from the date that notice of termination is mailed to the Agency to the date of full repayment by the Agency.

ARTICLE 16. DAMAGES FOR BREACH AFFECTING TAX EXEMPT STATUS.

(A) In the event that any breach of any of the provisions of this contract by the Agency shall result in the loss of tax exempt status for any state bonds, or if such breach shall result in an obligation on the part of the State to reimburse the federal government by reason of any arbitrage profits, the Agency shall immediately reimburse the State in an amount equal to any damages paid by or loss incurred by the State due to such breach.

ARTICLE 17. DISPUTES.

(A) Any dispute arising under this contract which is not otherwise disposed of by agreement shall be decided by the Chief of the Division, or his authorized representative. The decision shall be reduced to writing and a copy thereof furnished to the Agency and to the State Board's Executive Director. The decision of the Division shall be final and conclusive unless, within thirty (30) calendar days after mailing of the Division decision to the Agency, the Agency mails or otherwise furnishes a written appeal of the decision to the State Board's Executive Director. The decision of the State Board's Executive Director shall be final and conclusive unless determined by a court of competent jurisdiction to have been fraudulent, or capricious, or arbitrary, or so grossly erroneous as necessarily to imply bad faith, or not supported by substantial evidence. In

connection with any appeal under this clause, the Agency shall be afforded an opportunity to be heard and to offer evidence in support of its appeal. Pending final decision of a dispute hereunder, the Agency shall continue to fulfill and comply with all the terms, provisions, commitments, and requirements of this loan contract.

(B) This clause does not preclude consideration of legal questions, provided that nothing herein shall be construed to make final the decision of the State Board, or any official or representative thereof, or any question on law.

ARTICLE 18. STATE BOARD ACTION; COSTS AND ATTORNEY FEES.

(A) The Agency agrees that any remedy provided in this contract is in addition to and not in derogation of any other legal or equitable remedy available to the State Board as a result of breach of this contract by the Agency, whether such breach occurs before or after completion of the Project, and exercise of any remedy provided by this contract by the State Board shall not preclude the State Board from pursuing any legal remedy or right which would otherwise be available. In the event of litigation between the parties hereto arising from this contract, it is agreed that the prevailing party shall be entitled to such reasonable costs and/or attorney fees as may be ordered by the court entertaining such litigation.

ARTICLE 19. WAIVER.

(A) The parties hereto may, from time to time, waive any rights under this contract unless such waiver is contrary to law, provided that any such waiver must be in writing and must be signed by the party making such waiver.

ARTICLE 20. AMENDMENT.

(A) This contract may be amended at any time by mutual written agreement of the parties.

NONDISCRIMINATION CLAUSE

(OCP - 1)

1. During the performance of this contract, contractor and its subcontractors shall not unlawfully discriminate against any employee or applicant for employment because of race, religion, color, national origin, ancestry, physical handicap, medical condition, marital status, age (over 40) or sex. Contractors and subcontractors shall insure that the evaluation and treatment of their employees and applicants for employment are free of such discrimination. Contractors and subcontractors shall comply with the provisions of the Fair Employment and Housing Act (Government Code, Section 12900 et seq.) and the applicable regulations promulgated thereunder (California Administrative Code, Title 2, Section 7285.0 et seq.). The applicable regulations of the Fair Employment and Housing Commission implementing Government Code, Section 12990, set forth in Chapter 5 of Division 4 of Title 2 of the California Administrative Code are incorporated into this contract by reference and made a part hereof as if set forth in full. Contractor and its subcontractors shall give written notice of their obligations under this clause to labor organizations with which they have a collective bargaining or other agreement.

2. This contractor shall include the nondiscrimination and compliance provisions of this clause in all subcontracts to perform work under the contract.

State of California

Memorandum

To : Regional Board Executive Officers

Date : JAN - 8 1992

Walt Pettit
Executive Director

From : **STATE WATER RESOURCES CONTROL BOARD**

Subject: FEDERAL FISCAL YEAR (FFY) 1993 STATE REVOLVING FUND (SRF) LOAN PROGRAM PRIORITY LIST -- REQUEST FOR REGIONAL BOARD INPUT

Attached is a copy of the preliminary FFY 1993 SRF Loan Program Priority List (Attachment 1). Attachment 2 is the proposed schedule for adopting the FFY 1993 Priority List.

The proposed FFY 1993 Priority List will be developed on the assumption that the State Board will: (1) continue to include projects in Priority Classes A through E on the fundable portion of the list; (2) set the amount of SRF funds (Cap) that any one agency can receive in FFY 1993 at $20 million; (3) use the priority system adopted by the State Board on September 20, 1990 (Attachment 3); and, (4) not fund major sewer rehabilitation projects in FFY 1993.

State regulations require that the Regional Board solicit comments before submittal to the State Board. It is extremely important that you review and recommend any schedule changes you think are appropriate. Since the criteria used to rank projects is based on the readiness to proceed, the estimated date for the start of construction is extremely important. We will attempt to accommodate your recommendations within the limitations of the project's Priority Class and the availability of loan funds. Other changes you may wish to recommend include changes in Priority Class, revision of project descriptions and costs, addition of new projects, and deletion of projects.

Any state, county, State Board, or Regional Board enforcement action necessary for a particular priority classification must be taken prior to the adoption or amendment of the Priority List (Chapter IV.G.2. of the Policy for Implementing the State Revolving Fund as amended on September 20, 1990).

We also request your assistance in making sure that consideration and public input is received regarding addition of nonpoint source and storm drainage projects to the FFY 1993 Priority List. Please contact Bill Campbell of the Division of Water Quality at (916) 657-1043 or ATSS 8-437-1043 for information.

Each request for a priority class elevation or addition of a new project should contain the following:

1. A transmittal letter summarizing the Regional Board request;

2. Adequate documentation of an existing public health or water quality problem if the request is for Priority Class A or B;

3. A copy of the appropriately signed Regional Board Enforcement Order for Class A, B or C requests;

4. Copies of any staff reports prepared;

5. A completed copy of the Project Priority History/Information Sheet (Attachment 4) for all project changes and for addition of new projects.

We anticipate that California will have approximately $100 million in SRF loan monies in FFY 1993. In preparing the FFY 1993 Priority List, the State Board will conduct the consistency reviews necessary to ascertain compliance with areawide waste treatment management (208) plans and water quality control plans.

Your recommended changes with the necessary documentation must be submitted to our office by April 30, 1992.

If you have an questions regarding the development of your Priority List, please contact Gus Atkins at (916) 739-4410 or ATSS 497-4410.

Attachments (4)

cc: Fresno, Victorville, Redding Branch Offices, (w/attachments)
Regional Board Priority List Coordinators (w/attachments)

Barbara Evoy (w/attachments)
Program Control Unit

Stan Martinson, DWQ (w/attachments)
Bill Campbell, DWQ (w/attachments)

DEPARTMENT OF UTILITIES

DIVISION OF
FLOOD CONTROL AND SEWERS

CITY OF SACRAMENTO
CALIFORNIA

1391 - 35TH AVENUE
SACRAMENTO, CA
95822-2911

916-264-7250

March 19, 1992
920194:CC:ar

Mr. Bill Crooks
California Regional Water Quality Control Board
Central Valley Region
3443 Routier Road
Sacramento, CA 95827-3098

SUBJECT: **ADDITION OF THE CITY OF SACRAMENTO'S COMBINED SEWER SYSTEM IMPROVEMENT PROJECT TO THE STATE REVOLVING FUND LOAN PROGRAM LIST.**

Dear Bill:

The City is officially requesting to be added to the California Regional Water Quality Control Board's (RWQCB) priority list of agencies eligible for funding from the State Revolving Fund. This request is based on the forthcoming improvement project for the City's combined sewer system. This request is for the maximum amount of funds that could be made available from the State Fund.

As you know, the combined sewer system improvement project will be in direct response to the RWQCB's Cease and Desist Order No. 90-198 adopted on June 22, 1990, and amended on September 6, 1991. The City is currently in it's third phase (Phase 3) of study addressing the requirements of the Cease and Desist Orders. The current order requires the City to submit a Phase 3 Detailed Technical Report to the RWQCB by July 1, 1992. This report is required to recommend a combined sewer improvement plan, along with proposed funding mechanisms. The State Revolving Fund will be one of the proposed funding mechanisms recommend by the City.

Although a final combined sewer improvement plan has not been recommended to the RWQCB. The Phase 2 report estimated the least cost alternative designed for a 10 year - 6 hour storm event vould cost approximately $575 million. This least cost improvement plan included the construction of a deep tunnel system used for conveyance and storage along with collection system and pumping facility improvements. The magnitude of this cost was such that the City does not believe this plan to be affordable. The RWQCB concurred, and on September 6, 1991, ordered the City to perform a Phase 3 study investigating other alternatives.

It is our understanding that the City can be added to the priority list of agencies requesting State Revolving Fund Loan Program funds even though the final project has not been selected or approved. It is also our understanding that when the project is selected this information can be forwarded to Mr. Paul Marshall, the RWQCB's State Revolving Fund Loan Program Manager, Paul will then make the proper loan information revisions.

We respectfully request that your office confirm that we have been added to the RWQCB's priority list for State Revolving Funds. Also, please confirm that the City's understanding of the information required by the RWQCB and the contact person are correct.

If you have any questions please contact Gary Reents at 264-7250.

Sincerely,

Donald M. Dodge

Donald M. Dodge, Director
Department of Utilities

cc: Albert McCollam, Manager, Flood Control & Sewers Division
Gary Reents, Supervising Engineer
Dave Affleck, Deputy City Treasurer
Jerry Troyan, HDR
Combined Sewer System File

APPENDIX C

DETAILED CALCULATIONS FOR MONTHLY RATE IMPACT OF RECOMMENDED PLAN

TABLE USER FEE IMPACT-RECOMMENDED PLAN

PROJECT PHASING OPTION 2 - 15 YEAR CONSTRUCTION PERIOD

Rate Option 1 - All users.

BARTLE WELLS ASSOCIATES

SCSOREC7 6/25/92

	Annual Increase	Budget 1991/92	Pro forma 1992/93	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08
Number of EDU's (1)																		
1 Combined sewer system			31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900
2 Sanitary sewer-other		103,800	72,600	73,300	74,000	74,700	75,400	76,200	77,000	77,800	78,600	79,400	80,200	81,000	81,800	82,600	83,400	84,200
3 Total sewer	0.7%	103,800	104,500	105,200	105,900	106,600	107,300	108,100	108,900	109,700	110,500	111,300	112,100	112,900	113,700	114,500	115,300	116,100
4 Storm drainage	2.0%	176,500	180,000	183,600	187,300	191,000	194,800	198,700	202,700	206,800	210,900	215,100	219,400	223,800	228,300	232,900	237,600	242,400
5 Average monthly user rate/EDU: Increase(2)---)			24%	10%	8%	8%	8%	7%	7%	7%	7%	8%	8%	7%	7%	4%	0%	0%
6 Combined sewer system		$7.07	$8.77	$9.64	$10.41	$11.25	$12.15	$13.00	$13.91	$14.88	$15.92	$17.20	$18.57	$19.87	$21.26	$22.12	$22.12	$22.12
7 Sanitary sewer-other		$7.07	$8.77	$9.64	$10.41	$11.25	$12.15	$13.00	$13.91	$14.88	$15.92	$17.20	$18.57	$19.87	$21.26	$22.12	$22.12	$22.12
8 Storm drainage-combined system		$9.06	$11.23	$12.36	$13.35	$14.41	$15.57	$16.66	$17.82	$19.07	$20.41	$22.04	$23.80	$25.47	$27.25	$28.34	$28.34	$28.34
9 Storm drainage-other		$9.06	$11.23	$12.36	$13.35	$14.41	$15.57	$16.66	$17.82	$19.07	$20.41	$22.04	$23.80	$25.47	$27.25	$28.34	$28.34	$28.34
10 Sacramento Regional rate		$8.15	$8.48	$8.82	$9.17	$9.53	$9.92	$10.31	$10.72	$11.15	$11.60	$12.06	$12.55	$13.05	$13.57	$14.11	$14.68	$15.26
11 Highest total user rate		$24.28	$28.48	$30.82	$32.93	$35.20	$37.63	$39.97	$42.46	$45.11	$47.93	$51.30	$54.92	$58.39	$62.08	$64.57	$65.13	$65.72
12 Affordability Index rate		$46.23	$48.08	$50.00	$52.00	$54.08	$56.24	$58.49	$60.83	$63.27	$65.80	$68.43	$71.17	$74.01	$76.97	$80.05	$83.25	$86.58
13 Beginning Fund Balance (X$000)		$2,353	$1,200	$760	$2,366	$6,372	$4,617	$6,062	$10,907	$6,064	$4,621	$7,178	$1,292	$1,106	$6,520	$3,107	$3,394	$3,181
Revenue (3)																		
14 Combined sewer user charge		0	3,000	3,700	4,000	4,300	4,700	5,000	5,300	5,700	6,100	6,600	7,100	7,600	8,100	8,500	8,500	8,500
15 Other sanitary user charge		8,811	6,900	8,500	9,200	10,100	11,000	11,900	12,900	13,900	15,000	16,400	17,900	19,300	20,900	21,900	22,100	22,300
16 Storm drainage user charge		19,189	21,900	27,200	30,000	33,000	36,400	39,700	43,400	47,300	51,600	56,900	62,700	68,400	74,700	79,200	80,800	82,400
17 Interest income		2,043	0	100	200	300	300	400	400	300	300	200	100	200	200	200	200	100
18 Other		3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310
19 Total revenues		33,353	35,110	42,810	46,710	51,010	55,710	60,310	65,310	70,510	76,310	83,410	91,110	98,810	107,210	113,110	114,910	116,610
Operating Expenses																		
20 existing (4)		25,829	26,900	28,000	29,100	30,300	31,500	32,800	34,100	35,500	36,900	38,400	39,900	41,500	43,200	44,900	46,700	48,600
21 additional		0	400	900	1,000	1,700	1,700	1,300	1,400	1,500	1,600	1,700	1,800	1,900	2,000	2,100	2,200	2,300
22 Other (net transfer)		174	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23 Total operating expenses		26,003	27,300	28,900	30,100	32,000	33,200	34,100	35,500	37,000	38,500	40,100	41,700	43,400	45,200	47,000	48,900	50,900
24 Net Revenue		7,350	7,810	13,910	16,610	19,010	22,510	26,210	29,810	33,510	37,810	43,310	49,410	55,410	62,010	66,110	66,010	65,710
Debt Service																		
25 existing		1,764	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750
26 additional		0	0	3,754	3,754	11,615	11,615	11,615	24,603	24,603	24,603	38,146	38,146	38,146	53,173	53,173	53,173	53,173
27 Total debt service		1,764	1,750	5,504	5,504	13,365	13,365	13,365	26,353	26,353	26,353	39,896	39,896	39,896	54,923	54,923	54,923	54,923
28 Capital improvements		6,732	6,500	6,800	7,100	7,400	7,700	8,000	8,300	8,600	8,900	9,300	9,700	10,100	10,500	10,900	11,300	11,800
29 Net revenue/(expense)		(1,146)	(440)	1,606	4,006	(1,755)	1,445	4,845	(4,843)	(1,443)	2,557	(5,886)	(186)	5,414	(3,413)	287	(213)	(1,013)
30 Ending fund balance		$1,207	$760	$2,366	$6,372	$4,617	$6,062	$10,907	$6,064	$4,621	$7,178	$1,292	$1,106	$6,520	$3,107	$3,394	$3,181	$2,168
31 Coverage Ratio (line 24 ÷ line 13/line 27)		5.50	5.15	2.67	3.45	1.90	2.03	2.41	1.55	1.50	1.61	1.27	1.27	1.42	1.25	1.26	1.26	1.25
33 Bond Issue - Current			40,500		84,800			140,100			146,100			162,100				
34 Bond Issue - Cumulative			40,500	40,500	125,300	125,300	125,300	265,400	265,400	265,400	411,500	411,500	411,500	573,600	573,600	573,600	573,600	573,600

(1) Equivalent dwelling units

(2) Shows increase in impacted rate(s) only; other rates increased at 4% inflation rate.

(3) Assumes rates increased 1/1/93 in 1992/93 and on 7/1 in all subsequent years.

(4) Inflated at 4% annually beginning in 1992/93.

TABLE _ USER FEE IMPACT-RECOMMENDED PLAN
PROJECT PHASING OPTION 1 - 27 YEAR CONSTRUCTION SCHEDULE
Rate Option 2 - Combined system users only

BARTLE WELLS ASSOCIATES
SCSOREC2 7/6/92

	Annual Increase	Budget 1991/92	Pro forma 1992/93	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06
Number of EDU's (1)																
1 Combined sewer system			31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900
2 Sanitary sewer-other		103,800	72,600	73,300	74,000	74,700	75,400	76,200	77,000	77,800	78,600	79,400	80,200	81,000	81,800	82,600
3 Total sewer	0.7%	103,800	104,500	105,200	105,900	106,600	107,300	108,100	108,900	109,700	110,500	111,300	112,100	112,900	113,700	114,500
4 Storm drainage	2.0%	176,500	180,000	183,600	187,300	191,000	194,800	198,700	202,700	206,800	210,900	215,100	219,400	223,800	228,300	232,900
5 Average monthly user rate/EDU: Increase(2)---			70%	36%	25%	20%	17%	10%	10%	5%	5%	10%	10%	4%	4%	4%
6 Combined sewer system		$7.07	$12.02	$16.35	$19.62	$23.54	$27.54	$30.29	$33.32	$34.99	$36.74	$40.41	$44.45	$46.23	$48.08	$50.00
7 Sanitary sewer-other		$7.07	$7.35	$7.65	$7.95	$8.27	$8.60	$8.95	$9.30	$9.68	$10.06	$10.47	$10.88	$11.32	$11.77	$12.24
8 Storm drainage-combined system		$9.06	$15.40	$20.95	$25.14	$30.16	$35.29	$38.82	$42.70	$44.84	$47.08	$51.79	$56.97	$59.24	$61.61	$64.08
9 Storm drainage-other		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69
10 Sacramento Regional rate		$8.15	$8.48	$8.82	$9.17	$9.53	$9.92	$10.31	$10.72	$11.15	$11.60	$12.06	$12.55	$13.05	$13.57	$14.11
11 Highest total user rate		$24.28	$35.90	$46.11	$53.92	$63.24	$72.75	$79.43	$86.75	$90.99	$95.42	$104.26	$113.97	$118.52	$123.27	$128.20
12 Affordability Index rate		$46.23	$48.08	$50.00	$52.00	$54.08	$56.24	$58.49	$60.83	$63.27	$65.80	$68.43	$71.17	$74.01	$76.97	$80.05
13 Beginning Fund Balance (X$000)		$2,353	$1,200	$60	$1,066	$5,072	$1,857	$2,242	$5,727	$3,722	$3,317	$4,812	$1,678	$2,344	$5,110	$2,643
Revenue (3)																
14 Combined sewer user charge		0	3,700	6,300	7,500	9,000	10,500	11,600	12,800	13,400	14,100	15,500	17,000	17,700	18,400	19,100
15 Other sanitary user charge		8,811	6,300	6,700	7,100	7,400	7,800	8,200	8,600	9,000	9,500	10,000	10,500	11,000	11,600	12,100
16 Storm drainage user charge		19,189	21,100	25,900	28,600	31,800	35,100	37,800	40,800	43,200	45,700	49,300	53,200	56,100	59,100	62,400
17 Interest income		2,043	0	0	200	200	100	200	200	200	200	200	100	200	200	100
18 Other		3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310
19 Total revenues		33,353	34,410	42,210	46,710	51,710	56,810	61,110	65,710	69,110	72,810	78,310	84,110	88,310	92,610	97,010
Operating Expenses																
20 existing (4)		25,829	26,900	28,000	29,100	30,300	31,500	32,800	34,100	35,500	36,900	38,400	39,900	41,500	43,200	44,900
21 additional		0	400	900	1,000	1,700	1,700	1,300	1,400	1,500	1,600	1,700	1,800	1,900	2,000	2,100
22 Other (net transfer)		174	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23 Total operating expenses		26,003	27,300	28,900	30,100	32,000	33,200	34,100	35,500	37,000	38,500	40,100	41,700	43,400	45,200	47,000
24 Net Revenue		7,350	7,110	13,310	16,610	19,710	23,610	27,010	30,210	32,110	34,310	38,210	42,410	44,910	47,410	50,010
Debt Service																
25 existing		1,764	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750
26 additional		0	0	3,754	3,754	13,775	13,775	13,775	22,165	22,165	22,165	30,294	30,294	30,294	37,627	37,627
27 Total debt service		1,764	1,750	5,504	5,504	15,525	15,525	15,525	23,915	23,915	23,915	32,044	32,044	32,044	39,377	39,377
28 Capital improvements		6,732	6,500	6,800	7,100	7,400	7,700	8,000	8,300	8,600	8,900	9,300	9,700	10,100	10,500	10,900
29 Net revenue/(expense)		(1,146)	(1,140)	1,006	4,006	(3,215)	385	3,485	(2,005)	(405)	1,495	(3,134)	666	2,766	(2,467)	(267)
30 Ending fund balance		$1,207	$60	$1,066	$5,072	$1,857	$2,242	$5,727	$3,722	$3,317	$4,812	$1,678	$2,344	$5,110	$2,643	$2,376
31 Coverage Ratio (line 24 + line 13/line 27)		5.50	4.75	2.42	3.21	1.60	1.64	1.88	1.50	1.53	1.57	1.34	1.38	1.47	1.33	1.34
32 Bond Issue - Current			40,500		108,100			90,500			87,700			79,100		
33 Bond Issue - Cumulative			40,500	40,500	148,600	148,600	148,600	239,100	239,100	239,100	326,800	326,800	326,800	405,900	405,900	405,900

	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20
Number of EDU's (1)														
1 Combined sewer system	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900
2 Sanitary sewer-other	83,400	84,200	85,000	85,800	86,600	87,400	88,200	89,000	89,800	90,700	91,600	92,500	93,400	94,300
3 Total sewer	115,300	116,100	116,900	117,700	118,500	119,300	120,100	120,900	121,700	122,600	123,500	124,400	125,300	126,200
4 Storm drainage	237,600	242,400	247,200	252,100	257,100	262,200	267,400	272,700	278,200	283,800	289,500	295,300	301,200	307,200
5 Average monthly user rate/EDU: Increase(2)---	4%	0%	3%	0%	0%	0%	3%	3%	0%	0%	1%	0%	0%	0%
6 Combined sewer system	$52.00	$52.00	$53.56	$53.56	$53.56	$53.56	$55.17	$56.83	$56.83	$56.83	$57.39	$57.39	$57.39	$57.39
7 Sanitary sewer-other	$12.73	$13.24	$13.77	$14.32	$14.90	$15.49	$16.11	$16.76	$17.43	$18.12	$18.85	$19.60	$20.39	$21.20
8 Storm drainage-combined system	$66.64	$66.64	$68.64	$68.64	$68.64	$68.64	$70.70	$72.82	$72.82	$72.82	$73.55	$73.55	$73.55	$73.55
9 Storm drainage-other	$16.32	$16.97	$17.65	$18.35	$19.09	$19.85	$20.65	$21.47	$22.33	$23.22	$24.15	$25.12	$26.12	$27.17
10 Sacramento Regional rate	$14.68	$15.26	$15.88	$16.51	$17.17	$17.86	$18.57	$19.31	$20.09	$20.89	$21.73	$22.60	$23.50	$24.44
11 Highest total user rate	$133.32	$133.91	$138.08	$138.72	$139.38	$140.06	$144.44	$148.96	$149.74	$150.54	$152.67	$153.54	$154.44	$155.35
12 Affordability Index rate	$83.25	$86.58	$90.05	$93.65	$97.40	$101.29	$105.34	$109.56	$113.94	$118.50	$123.24	$128.17	$133.29	$138.62
13 Beginning Fund Balance (X$000)	$2,376	$4,709	$3,170	$3,831	$5,492	$3,088	$1,684	$3,080	$1,452	$1,524	$3,396	$1,214	$1,232	$7,504
Revenue (3)														
14 Combined sewer user charge	19,900	19,900	20,500	20,500	20,500	20,500	21,100	21,800	21,800	21,800	22,000	22,000	22,000	22,000
15 Other sanitary user charge	12,700	13,400	14,000	14,700	15,500	16,200	17,100	17,900	18,800	19,700	20,700	21,800	22,800	24,000
16 Storm drainage user charge	65,800	68,400	71,900	74,800	77,900	81,100	85,400	89,900	93,900	98,100	102,800	107,500	112,600	117,900
17 Interest income	200	200	200	200	200	100	100	100	100	100	100	100	200	600
18 Other	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310
19 Total revenues	101,910	105,210	109,910	113,510	117,410	121,210	127,010	133,010	137,910	143,010	148,910	154,710	160,910	167,810
Operating Expenses														
20 existing (4)	46,700	48,600	50,500	52,500	54,600	56,800	59,100	61,500	64,000	66,600	69,300	72,100	75,000	78,000
21 additional	2,200	2,300	2,400	2,500	2,600	2,700	2,800	2,900	3,000	3,100	3,200	3,300	3,400	3,500
22 Other (net transfer)	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23 Total operating expenses	48,900	50,900	52,900	55,000	57,200	59,500	61,900	64,400	67,000	69,700	72,500	75,400	78,400	81,500
24 Net Revenue	53,010	54,310	57,010	58,510	60,210	61,710	65,110	68,610	70,910	73,310	76,410	79,310	82,510	86,310
Debt Service														
25 existing	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750
26 additional	37,627	42,299	42,299	42,299	47,564	47,564	47,564	53,488	53,488	53,488	60,042	60,042	56,288	56,288
27 Total debt service	39,377	44,049	44,049	44,049	49,314	49,314	49,314	55,238	55,238	55,238	61,792	61,792	58,038	58,038
28 Capital improvements	11,300	11,800	12,300	12,800	13,300	13,800	14,400	15,000	15,600	16,200	16,800	17,500	18,200	18,900
29 Net revenue/(expense)	2,333	(1,539)	661	1,661	(2,404)	(1,404)	1,396	(1,628)	72	1,872	(2,182)	18	6,272	9,372
30 Ending fund balance	$4,709	$3,170	$3,831	$5,492	$3,088	$1,684	$3,080	$1,452	$1,524	$3,396	$1,214	$1,232	$7,504	$16,876
31 Coverage Ratio (line 24 + line 13/line 27)	1.41	1.34	1.37	1.42	1.33	1.31	1.35	1.30	1.31	1.35	1.29	1.30	1.44	1.62
32 Bond Issue - Current	50,400			56,800			63,900			70,700				
33 Bond Issue - Cumulative	456,300	456,300	456,300	513,100	513,100	513,100	577,000	577,000	577,000	647,700	647,700	647,700	647,700	647,700

(1) Equivalent dwelling units
(2) Shows increase in impacted rate(s) only; other rates increased at 4% inflation rate.
(3) Assumes rates increased 1/1/93 in 1992/93 and on 7/1 in all subsequent years.
(4) Inflated at 4% annually beginning in 1992/93.

TABLE __ USER FEE IMPACT-RECOMMENDED PLAN
PROJECT PHASING OPTION 1 - 15 YEAR CONSTRUCTION SCHEDULE
Rate Option 2 - Combined system users only

BARTLE WELLS ASSOCIATES
SCSOREC5 7/6/92

	Annual Increase	Budget 1991/92	Pro forma 1992/93	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08
Number of EDU's (1)																		
1 Combined sewer system			31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900
2 Sanitary sewer-other		103,800	72,600	73,300	74,000	74,700	75,400	76,200	77,000	77,800	78,600	79,400	80,200	81,000	81,800	82,600	83,400	84,200
3 Total sewer	0.7%	103,800	104,500	105,200	105,900	106,600	107,300	108,100	108,900	109,700	110,500	111,300	112,100	112,900	113,700	114,500	115,300	116,100
4 Storm drainage	2.0%	176,500	180,000	183,600	187,300	191,000	194,800	198,700	202,700	206,800	210,900	215,100	219,400	223,800	228,300	232,900	237,600	242,400
5 Average monthly user rate/EDU: Increase(2)---			70%	36%	16%	16%	16%	14%	14%	14%	12%	12%	12%	10%	10%	4%	0%	0%
6 Combined sewer system		$7.07	$12.02	$16.35	$18.96	$21.99	$25.51	$29.09	$33.16	$37.80	$42.34	$47.42	$53.11	$58.42	$64.26	$66.83	$66.83	$66.83
7 Sanitary sewer-other		$7.07	$7.35	$7.65	$7.95	$8.27	$8.60	$8.95	$9.30	$9.68	$10.06	$10.47	$10.88	$11.32	$11.77	$12.24	$12.73	$13.24
8 Storm drainage-combined system		$9.06	$15.40	$20.95	$24.30	$28.19	$32.70	$37.27	$42.49	$48.44	$54.25	$60.76	$68.05	$74.86	$82.35	$85.64	$85.64	$85.64
9 Storm drainage-other		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69	$16.32	$16.97
10 Sacramento Regional rate		$8.15	$8.48	$8.82	$9.17	$9.53	$9.92	$10.31	$10.72	$11.15	$11.60	$12.06	$12.55	$13.05	$13.57	$14.11	$14.68	$15.26
11 Highest total user rate		$24.28	$35.90	$46.11	$52.43	$59.72	$68.13	$76.67	$86.37	$97.39	$108.19	$120.24	$133.71	$146.33	$160.18	$166.58	$167.15	$167.73
12 Affordability Index rate		$46.23	$48.08	$50.00	$52.00	$54.08	$56.24	$58.49	$60.83	$63.27	$65.80	$68.43	$71.17	$74.01	$76.97	$80.05	$83.25	$86.58
13 Beginning Fund Balance (X$000)		$2,353	$1,200	$60	$1,066	$4,472	$2,017	$2,862	$7,507	$2,964	$2,521	$6,478	$1,592	$1,906	$7,520	$3,807	$2,794	$2,481
Revenue (3)																		
14 Combined sewer user charge		0	3,700	6,300	7,300	8,400	9,800	11,100	12,700	14,500	16,200	18,200	20,300	22,400	24,600	25,600	25,600	25,600
15 Other sanitary user charge		8,811	6,300	6,700	7,100	7,400	7,800	8,200	8,600	9,000	9,500	10,000	10,500	11,000	11,600	12,100	12,700	13,400
16 Storm drainage user charge		19,189	21,100	25,900	28,300	31,000	34,100	37,200	40,700	44,600	48,500	52,700	57,400	62,100	67,100	70,600	73,100	75,600
17 Interest income		2,043	0	0	100	200	100	300	300	100	200	200	100	200	300	200	100	100
18 Other		3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310
19 Total revenues		33,353	34,410	42,210	46,110	50,310	55,110	60,110	65,610	71,510	77,710	84,410	91,610	99,010	106,910	111,810	114,810	118,010
Operating Expenses																		
20 existing (4)		25,829	26,900	28,000	29,100	30,300	31,500	32,800	34,100	35,500	36,900	38,400	39,900	41,500	43,200	44,900	46,700	48,600
21 additional		0	400	900	1,000	1,700	1,700	1,300	1,400	1,500	1,600	1,700	1,800	1,900	2,000	2,100	2,200	2,300
22 Other (net transfer)		174	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23 Total operating expenses		26,003	27,300	28,900	30,100	32,000	33,200	34,100	35,500	37,000	38,500	40,100	41,700	43,400	45,200	47,000	48,900	50,900
24 Net Revenue		7,350	7,110	13,310	16,010	18,310	21,910	26,010	30,110	34,510	39,210	44,310	49,910	55,610	61,710	64,810	65,910	67,110
Debt Service																		
25 existing		1,764	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750
26 additional		0	0	3,754	3,754	11,615	11,615	11,615	24,603	24,603	24,603	38,146	38,146	38,146	53,173	53,173	53,173	53,173
27 Total debt service		1,764	1,750	5,504	5,504	13,365	13,365	13,365	26,353	26,353	26,353	39,896	39,896	39,896	54,923	54,923	54,923	54,923
28 Capital improvements		6,732	6,500	6,800	7,100	7,400	7,700	8,000	8,300	8,600	8,900	9,300	9,700	10,100	10,500	10,900	11,300	11,800
29 Net revenue/(expense)		(1,146)	(1,140)	1,006	3,406	(2,455)	845	4,645	(4,543)	(443)	3,957	(4,886)	314	5,614	(3,713)	(1,013)	(313)	387
30 Ending fund balance		$1,207	$60	$1,066	$4,472	$2,017	$2,862	$7,507	$2,964	$2,521	$6,478	$1,592	$1,906	$7,520	$3,807	$2,794	$2,481	$2,868
31 Coverage Ratio (line 24 ÷ line 13/line 27)		5.50	4.75	2.43	3.10	1.70	1.79	2.16	1.43	1.42	1.58	1.27	1.29	1.44	1.26	1.25	1.25	1.27
32 Bond Issue - Current			40,500		84,800			140,100			146,100			162,100				
33 Bond Issue - Cumulative			40,500	40,500	125,300	125,300	125,300	265,400	265,400	265,400	411,500	411,500	411,500	573,600	573,600	573,600	573,600	573,600

(1) Equivalent dwelling units
(2) Shows increase in impacted rate(s) only; other rates increased at 4% inflation rate.
(3) Assumes rates increased 1/1/93 in 1992/93 and on 7/1 in all subsequent years.
(4) Inflated at 4% annually beginning in 1992/93.

TABLE __ USER FEE IMPACT-RECOMMENDED PLAN
PROJECT PHASING SCHEDULE 1 - 27 YEAR CONSTRUCTION SCHEDULE
Rate Option 3 - Sanitary system users only

BARTLE WELLS ASSOCIATES
SCSOREC3 7/6/92

	Annual Increase	Budget 1991/92	Pro forma 1992/93	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06
Number of EDU's (1)																
1 Combined sewer system			31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900
2 Sanitary sewer-other		103,800	72,600	73,300	74,000	74,700	75,400	76,200	77,000	77,800	78,600	79,400	80,200	81,000	81,800	82,600
3 Total sewer	0.7%	103,800	104,500	105,200	105,900	106,600	107,300	108,100	108,900	109,700	110,500	111,300	112,100	112,900	113,700	114,500
4 Storm drainage	2.0%	176,500	180,000	183,600	187,300	191,000	194,800	198,700	202,700	206,800	210,900	215,100	219,400	223,800	228,300	232,900
5 Average monthly user rate/EDU: Increase(2)—>			50%	30%	18%	15%	15%	10%	5%	5%	5%	10%	8%	4%	4%	4%
6 Combined sewer system		$7.07	$10.61	$13.79	$16.27	$18.71	$21.51	$23.67	$24.85	$26.09	$27.40	$30.14	$32.55	$33.85	$35.20	$36.61
7 Sanitary sewer-other		$7.07	$10.61	$13.79	$16.27	$18.71	$21.51	$23.67	$24.85	$26.09	$27.40	$30.14	$32.55	$33.85	$35.20	$36.61
8 Storm drainage-combined system		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69
9 Storm drainage-other		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69
10 Sacramento Regional rate		$8.15	$8.48	$8.82	$9.17	$9.53	$9.92	$10.31	$10.72	$11.15	$11.60	$12.06	$12.55	$13.05	$13.57	$14.11
11 Highest total user rate		$24.28	$28.50	$32.40	$35.63	$38.84	$42.45	$45.44	$47.50	$49.64	$51.89	$55.61	$59.04	$61.40	$63.86	$66.41
12 Affordability Index rate		$46.23	$48.08	$50.00	$52.00	$54.08	$56.24	$58.49	$60.83	$63.27	$65.80	$68.43	$71.17	$74.01	$76.97	$80.05
13 Beginning Fund Balance (X$000)		$2,353	$1,200	$60	$1,166	$5,472	$2,357	$2,842	$6,727	$4,122	$3,317	$4,412	$1,178	$1,644	$4,510	$2,343
Revenue (3)																
14 Combined sewer user charge		0	3,400	5,300	6,200	7,200	8,200	9,100	9,500	10,000	10,500	11,500	12,500	13,000	13,500	14,000
15 Other sanitary user charge		8,811	7,700	12,100	14,400	16,800	19,500	21,600	23,000	24,400	25,800	28,700	31,300	32,900	34,600	36,300
16 Storm drainage user charge		19,189	20,000	21,600	22,900	24,300	25,800	27,300	29,000	30,800	32,600	34,600	36,700	39,000	41,300	43,800
17 Interest income		2,043	0	0	200	200	100	200	300	200	200	100	100	200	200	100
18 Other		3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310
19 Total revenues		33,353	34,410	42,310	47,010	51,810	56,910	61,510	65,110	68,710	72,410	78,210	83,910	88,410	92,910	97,510
Operating Expenses																
20 existing (4)		25,829	26,900	28,000	29,100	30,300	31,500	32,800	34,100	35,500	36,900	38,400	39,900	41,500	43,200	44,900
21 additional		0	400	900	1,000	1,700	1,700	1,300	1,400	1,500	1,600	1,700	1,800	1,900	2,000	2,100
22 Other (net transfer)		174	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23 Total operating expenses		26,003	27,300	28,900	30,100	32,000	33,200	34,100	35,500	37,000	38,500	40,100	41,700	43,400	45,200	47,000
24 Net Revenue		7,350	7,110	13,410	16,910	19,810	23,710	27,410	29,610	31,710	33,910	38,110	42,210	45,010	47,710	50,510
Debt Service																
25 existing		1,764	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750
26 additional		0	0	3,754	3,754	13,775	13,775	13,775	22,165	22,165	22,165	30,294	30,294	30,294	37,627	37,627
27 Total debt service		1,764	1,750	5,504	5,504	15,525	15,525	15,525	23,915	23,915	23,915	32,044	32,044	32,044	39,377	39,377
28 Capital improvements		6,732	6,500	6,800	7,100	7,400	7,700	8,000	8,300	8,600	8,900	9,300	9,700	10,100	10,500	10,900
29 Net revenue/(expense)		(1,146)	(1,140)	1,106	4,306	(3,115)	485	3,885	(2,605)	(805)	1,095	(3,234)	466	2,866	(2,167)	233
30 Ending fund balance		$1,207	$60	$1,166	$5,472	$2,357	$2,842	$6,727	$4,122	$3,317	$4,412	$1,178	$1,644	$4,510	$2,343	$2,576
31 Coverage Ratio (line 24 ÷ line 13/line 27)		5.50	4.75	2.45	3.28	1.63	1.68	1.95	1.52	1.50	1.56	1.33	1.35	1.46	1.33	1.34
32 Bond Issue - Current			40,500		108,100			90,500			87,700			79,100		
33 Bond Issue - Cumulative			40,500	40,500	148,600	148,600	148,600	239,100	239,100	239,100	326,800	326,800	326,800	405,900	405,900	405,900

	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20
Number of EDU's (1)														
1 Combined sewer system	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900
2 Sanitary sewer-other	83,400	84,200	85,000	85,800	86,600	87,400	88,200	89,000	89,800	90,700	91,600	92,500	93,400	94,300
3 Total sewer	115,300	116,100	116,900	117,700	118,500	119,300	120,100	120,900	121,700	122,600	123,500	124,400	125,300	126,200
4 Storm drainage	237,600	242,400	247,200	252,100	257,100	262,200	267,400	272,700	278,200	283,800	289,500	295,300	301,200	307,200
5 Average monthly user rate/EDU: Increase(2)—>	0%	4%	0%	0%	4%	0%	0%	4%	0%	0%	0%	0%	0%	0%
6 Combined sewer system	$36.61	$38.08	$38.08	$38.08	$39.60	$39.60	$39.60	$41.18	$41.18	$41.18	$41.18	$41.18	$41.18	$41.18
7 Sanitary sewer-other	$36.61	$38.08	$38.08	$38.08	$39.60	$39.60	$39.60	$41.18	$41.18	$41.18	$41.18	$41.18	$41.18	$41.18
8 Storm drainage-combined system	$16.32	$16.97	$17.65	$18.35	$19.09	$19.85	$20.65	$21.47	$22.33	$23.22	$24.15	$25.12	$26.12	$27.17
9 Storm drainage-other	$16.32	$16.97	$17.65	$18.35	$19.09	$19.85	$20.65	$21.47	$22.33	$23.22	$24.15	$25.12	$26.12	$27.17
10 Sacramento Regional rate	$14.68	$15.26	$15.88	$16.51	$17.17	$17.86	$18.57	$19.31	$20.09	$20.89	$21.73	$22.60	$23.50	$24.44
11 Highest total user rate	$67.60	$70.31	$71.60	$72.94	$75.86	$77.31	$78.82	$81.97	$83.60	$85.30	$87.06	$88.90	$90.80	$92.79
12 Affordability Index rate	$83.25	$86.58	$90.05	$93.65	$97.40	$101.29	$105.34	$109.56	$113.94	$118.50	$123.24	$128.17	$133.29	$138.62
13 Beginning Fund Balance (X$000)	$2,576	$3,609	$2,870	$2,831	$3,792	$2,788	$2,784	$3,980	$2,952	$3,324	$5,396	$2,814	$2,032	$7,304
Revenue (3)														
14 Combined sewer user charge	14,000	14,600	14,600	14,600	15,200	15,200	15,200	15,800	15,800	15,800	15,800	15,800	15,800	15,800
15 Other sanitary user charge	36,600	38,500	38,800	39,200	41,200	41,500	41,900	44,000	44,400	44,800	45,300	45,700	46,200	46,600
16 Storm drainage user charge	46,500	49,400	52,400	55,500	58,900	62,500	66,200	70,300	74,500	79,100	83,900	89,000	94,400	100,200
17 Interest income	200	200	100	200	200	100	200	200	200	200	200	100	200	500
18 Other	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310
19 Total revenues	100,610	106,010	109,210	112,810	118,810	122,610	126,810	133,610	138,210	143,210	148,510	153,910	159,910	166,410
Operating Expenses														
20 existing (4)	46,700	48,600	50,500	52,500	54,600	56,800	59,100	61,500	64,000	66,600	69,300	72,100	75,000	78,000
21 additional	2,200	2,300	2,400	2,500	2,600	2,700	2,800	2,900	3,000	3,100	3,200	3,300	3,400	3,500
22 Other (net transfer)	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23 Total operating expenses	48,900	50,900	52,900	55,000	57,200	59,500	61,900	64,400	67,000	69,700	72,500	75,400	78,400	81,500
24 Net Revenue	51,710	55,110	56,310	57,810	61,610	63,110	64,910	69,210	71,210	73,510	76,010	78,510	81,510	84,910
Debt Service														
25 existing	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750
26 additional	37,627	42,299	42,299	42,299	47,564	47,564	47,564	53,488	53,488	53,488	60,042	60,042	56,288	58,288
27 Total debt service	39,377	44,049	44,049	44,049	49,314	49,314	49,314	55,238	55,238	55,238	61,792	61,792	58,038	60,038
28 Capital improvements	11,300	11,800	12,300	12,800	13,300	13,800	14,400	15,000	15,600	16,200	16,800	17,500	18,200	18,900
29 Net revenue/(expense)	1,033	(739)	(39)	961	(1,004)	(4)	1,196	(1,028)	372	2,072	(2,582)	(782)	5,272	5,972
30 Ending fund balance	$3,609	$2,870	$2,831	$3,792	$2,788	$2,784	$3,980	$2,952	$3,324	$5,396	$2,814	$2,032	$7,304	$13,276
31 Coverage Ratio (line 24 ÷ line 13/line 27)	1.38	1.33	1.34	1.38	1.33	1.34	1.37	1.32	1.34	1.39	1.32	1.32	1.44	1.54
32 Bond Issue - Current	50,400			56,800			63,900			70,700				
33 Bond Issue - Cumulative	456,300	456,300	456,300	513,100	513,100	513,100	577,000	577,000	577,000	647,700	647,700	647,700	647,700	647,700

(1) Equivalent dwelling units
(2) Shows increase in impacted rate(s) only; other rates increased at 4% inflation rate.
(3) Assumes rates increased 1/1/93 in 1992/93 and on 7/1 in all subsequent years.
(4) Inflated at 4% annually beginning in 1992/93.

TABLE __ USER FEE IMPACT-RECOMMENDED PLAN
PROJECT PHASING SCHEDULE 1 - 15 YEAR CONSTRUCTION SCHEDULE
Rate Option 3 - Sanitary system users only

BARTLE WELLS ASSOCIATES
SCSOREC8 7/6/92

	Annual Increase	Budget 1991/92	Pro forma 1992/93	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08
Number of EDU's (1)																		
1 Combined sewer system			31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900
2 Sanitary sewer-other		103,800	72,600	73,300	74,000	74,700	75,400	76,200	77,000	77,800	78,600	79,400	80,200	81,000	81,800	82,600	83,400	84,200
3 Total sewer	0.7%	103,800	104,500	105,200	105,900	106,600	107,300	108,100	108,900	109,700	110,500	111,300	112,100	112,900	113,700	114,500	115,300	116,100
4 Storm drainage	2.0%	176,500	180,000	183,600	187,300	191,000	194,800	198,700	202,700	206,800	210,900	215,100	219,400	223,800	228,300	232,900	237,600	242,400
5 Average monthly user rate/EDU: Increase(2)---)			50%	30%	15%	10%	10%	15%	15%	15%	9%	9%	9%	9%	9%	6%	0%	0%
6 Combined sewer system		$7.07	$10.61	$13.79	$15.85	$17.44	$19.18	$22.06	$25.37	$29.18	$31.80	$34.66	$37.78	$41.18	$44.89	$47.58	$47.58	$47.58
7 Sanitary sewer-other		$7.07	$10.61	$13.79	$15.85	$17.44	$19.18	$22.06	$25.37	$29.18	$31.80	$34.66	$37.78	$41.18	$44.89	$47.58	$47.58	$47.58
8 Storm drainage-combined system		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69	$16.32	$16.97
9 Storm drainage-other		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69	$16.32	$16.97
10 Sacramento Regional rate		$8.15	$8.48	$8.82	$9.17	$9.53	$9.92	$10.31	$10.72	$11.15	$11.60	$12.06	$12.55	$13.05	$13.57	$14.11	$14.68	$15.26
11 Highest total user rate		$24.28	$28.50	$32.40	$35.21	$37.57	$40.12	$43.84	$48.02	$52.73	$56.30	$60.14	$64.28	$68.74	$73.55	$77.39	$78.58	$79.82
12 Affordability Index rate		$46.23	$48.08	$50.00	$52.00	$54.08	$56.24	$58.49	$60.83	$63.27	$65.80	$68.43	$71.17	$74.01	$76.97	$80.05	$83.25	$86.58
13 Beginning Fund Balance (X$000)		$2,353	$1,200	$60	$1,166	$5,072	$2,417	$2,062	$6,007	$1,464	$2,121	$6,678	$1,792	$1,506	$6,420	$1,923	$1,726	$2,329
Revenue (3)																		
14 Combined sewer user charge		0	3,400	5,300	6,100	6,700	7,300	8,400	9,700	11,200	12,200	13,300	14,500	15,800	17,200	18,200	18,200	18,200
15 Other sanitary user charge		8,811	7,700	12,100	14,100	15,600	17,400	20,200	23,400	27,200	30,000	33,000	36,400	40,000	44,100	47,200	47,600	48,100
16 Storm drainage user charge		19,189	20,000	21,600	22,900	24,300	25,800	27,300	29,000	30,800	32,600	34,600	36,700	39,000	41,300	43,800	46,500	49,400
17 Interest income		2,043	0	0	200	200	100	200	200	100	200	200	100	200	200	100	100	200
Other		3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310
19 Total revenues		33,353	34,410	42,310	46,610	50,110	53,910	59,410	65,610	72,610	78,310	84,410	91,010	98,310	106,110	112,610	115,710	119,210
Operating Expenses																		
20 existing (4)		25,829	26,900	28,000	29,100	30,300	31,500	32,800	34,100	35,500	36,900	38,400	39,900	41,500	43,200	44,900	46,700	48,600
21 additional		0	400	900	1,000	1,700	1,700	1,300	1,400	1,500	1,600	1,700	1,800	1,900	2,000	2,100	2,200	2,300
22 Other (net transfer)		174	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23 Total operating expenses		26,003	27,300	28,900	30,100	32,000	33,200	34,100	35,500	37,000	38,500	40,100	41,700	43,400	45,200	47,000	48,900	50,900
24 Net Revenue		7,350	7,110	13,410	16,510	18,110	20,710	25,310	30,110	35,610	39,810	44,310	49,310	54,910	60,910	65,610	66,810	68,310
Debt Service																		
25 existing		1,764	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750
26 additional		0	0	3,754	3,754	11,615	11,615	11,615	24,603	24,603	24,603	38,146	38,146	38,146	53,157	53,157	53,157	53,157
27 Total debt service		1,764	1,750	5,504	5,504	13,365	13,365	13,365	26,353	26,353	26,353	39,896	39,896	39,896	54,907	54,907	54,907	54,907
28 Capital improvements		6,732	6,500	6,800	7,100	7,400	7,700	8,000	8,300	8,600	8,900	9,300	9,700	10,100	10,500	10,900	11,300	11,800
29 Net revenue/(expense)		(1,146)	(1,140)	1,106	3,906	(2,655)	(355)	3,945	(4,543)	657	4,557	(4,886)	(286)	4,914	(4,497)	(197)	603	1,603
30 Ending fund balance		$1,207	$60	$1,166	$5,072	$2,417	$2,062	$6,007	$1,464	$2,121	$6,678	$1,792	$1,506	$6,420	$1,923	$1,726	$2,329	$3,932
31 Coverage Ratio (line 24 + line 13/line 27)		5.50	4.75	2.45	3.21	1.73	1.73	2.05	1.37	1.41	1.59	1.28	1.28	1.41	1.23	1.23	1.25	1.29
32 Bond Issue - Current			40,500		84,800			140,100			146,100			162,100				
33 Bond Issue - Cumulative			40,500	40,500	125,300	125,300	125,300	265,400	265,400	265,400	411,500	411,500	411,500	573,600	573,600	573,600	573,600	573,600

(1) Equivalent dwelling units
(2) Shows increase in impacted rate(s) only; other rates increased at 4% inflation rate.
(3) Assumes rates increased 1/1/93 in 1992/93 and on 7/1 in all subsequent years.
(4) Inflated at 4% annually beginning in 1992/93.

TABLE __ USER FEE IMPACT-RECOMMENDED PLAN
PROJECT PHASING SCHEDULE 1 - 27 YEAR CONSTRUCTION SCHEDULE
Rate Option 4 - Sacramento Regional users only

BARTLE WELLS ASSOCIATES
SCSOREC4 7/6/92

	Annual Increase	Budget 1991/92	Pro forma 1992/93	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	
Number of City EDU's (1)																															
1 Combined sewer system			31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	1
2 Sanitary sewer-other		103,800	72,600	73,300	74,000	74,700	75,400	76,200	77,000	77,800	78,600	79,400	80,200	81,000	81,800	82,600	83,400	84,200	85,000	85,800	86,600	87,400	88,200	89,000	89,800	90,700	91,600	92,500	93,400	94,300	2
3 Total sewer	0.7%	103,800	104,500	105,200	105,900	106,600	107,300	108,100	108,900	109,700	110,500	111,300	112,100	112,900	113,700	114,500	115,300	116,100	116,900	117,700	118,500	119,300	120,100	120,900	121,700	122,600	123,500	124,400	125,300	126,200	3
4 Storm drainage	2.0%	176,500	180,000	183,600	187,300	191,000	194,800	198,700	202,700	206,800	210,900	215,100	219,400	223,800	228,300	232,900	237,600	242,400	247,200	252,100	257,100	262,200	267,400	272,700	278,200	283,800	289,500	295,300	301,200	307,200	4
5 Number of Sac. Regional ESD's(2)	2.0%	450,000	459,000	468,200	477,600	487,200	496,900	506,800	516,900	527,200	537,700	548,500	559,500	570,700	582,100	593,700	605,600	617,700	630,100	642,700	655,600	668,700	682,100	695,700	709,600	723,800	738,300	753,100	768,200	783,600	5
City rates/month:																															
6 Combined sewer system		$7.07	$7.35	$7.65	$7.95	$8.27	$8.60	$8.95	$9.30	$9.68	$10.06	$10.47	$10.88	$11.32	$11.77	$12.24	$12.73	$13.24	$13.77	$14.32	$14.90	$15.49	$16.11	$16.76	$17.43	$18.12	$18.85	$19.60	$20.39	$21.20	6
7 Sanitary sewer-other		$7.07	$7.35	$7.65	$7.95	$8.27	$8.60	$8.95	$9.30	$9.68	$10.06	$10.47	$10.88	$11.32	$11.77	$12.24	$12.73	$13.24	$13.77	$14.32	$14.90	$15.49	$16.11	$16.76	$17.43	$18.12	$18.85	$19.60	$20.39	$21.20	7
8 Storm drainage-combined system		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69	$16.32	$16.97	$17.65	$18.35	$19.09	$19.85	$20.65	$21.47	$22.33	$23.22	$24.15	$25.12	$26.12	$27.17	8
9 Storm drainage-other		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69	$16.32	$16.97	$17.65	$18.35	$19.09	$19.85	$20.65	$21.47	$22.33	$23.22	$24.15	$25.12	$26.12	$27.17	9
Sacramento Regional rate/month:																															
10 -O&M		$8.15	$8.48	$8.82	$9.17	$9.53	$9.92	$10.31	$10.72	$11.15	$11.60	$12.06	$12.55	$13.05	$13.57	$14.11	$14.68	$15.26	$15.88	$16.51	$17.17	$17.86	$18.57	$19.31	$20.09	$20.89	$21.73	$22.60	$23.50	$24.44	10
11 -Capital rate increase-current year(3)——			5%	5%	5%	5%	5%	5%	5%	3%	0%	3%	3%	3%	3%	0%	2%	0%	0%	0%	2%	0%	0%	3%	0%	0%	0%	0%	0%	0%	11
12 -Capital rate (cumulative)		$0.00	$0.42	$0.86	$1.32	$1.80	$2.30	$2.81	$3.35	$3.68	$3.68	$4.04	$4.42	$4.81	$5.22	$5.22	$5.51	$5.51	$5.51	$5.51	$5.86	$5.86	$5.86	$6.44	$6.44	$6.44	$6.44	$6.44	$6.44	$6.44	12
13 -Sacramento Regional rate -Total		$8.15	$8.90	$9.68	$10.49	$11.33	$12.21	$13.12	$14.07	$14.84	$15.28	$16.11	$16.97	$17.86	$18.79	$19.33	$20.19	$20.78	$21.39	$22.02	$23.03	$23.71	$24.43	$25.75	$26.52	$27.33	$28.16	$29.03	$29.93	$30.87	13
14 Highest total user rate		$24.28	$25.68	$27.13	$28.63	$30.20	$31.84	$33.53	$35.30	$36.91	$38.24	$39.98	$41.80	$43.68	$45.65	$47.26	$49.24	$50.99	$52.81	$54.70	$57.01	$59.06	$61.18	$63.98	$66.28	$68.67	$71.16	$73.75	$76.44	$79.24	14
15 Affordability Index rate		$46.23	$48.08	$50.00	$52.00	$54.08	$56.24	$58.49	$60.83	$63.27	$65.80	$68.43	$71.17	$74.01	$76.97	$80.05	$83.25	$86.58	$90.05	$93.65	$97.40	$101.29	$105.34	$109.56	$113.94	$118.50	$123.24	$128.17	$133.29	$138.62	15
16 Beginning Capital Fund Balance	(X$000)		$0	$1,200	$2,446	$6,492	$3,517	$3,642	$7,267	$6,202	$7,637	$9,672	$6,378	$6,084	$9,190	$8,463	$8,436	$11,409	$10,610	$10,511	$11,212	$10,248	$10,184	$11,020	$11,832	$13,744	$16,956	$14,714	$13,572	$17,384	16
17 Sac. Regional-Additional capital revenue(4)			1,200	4,900	7,600	10,500	13,700	17,100	20,800	23,300	23,800	26,600	29,700	33,000	36,500	37,200	40,100	40,900	41,700	42,500	46,100	47,000	47,900	53,700	54,800	55,900	57,000	58,200	59,300	60,500	17
18 Interest income			0	100	200	300	200	300	300	300	400	400	300	400	400	400	500	600	500	500	500	500	500	600	600	800	800	700	800	1,000	18
19 Total Capital funds available			1,200	6,200	10,246	17,292	17,417	21,042	28,367	29,802	31,837	36,672	36,378	39,484	46,090	46,063	49,036	52,909	52,810	53,511	57,812	57,748	58,584	65,320	67,232	70,444	74,756	73,614	73,672	78,884	19
20 Capital Project debt service			0	3,754	3,754	13,775	13,775	13,775	22,165	22,165	22,165	30,294	30,294	30,294	37,627	37,627	37,627	42,299	42,299	42,299	47,564	47,564	47,564	53,488	53,488	53,488	60,042	60,042	56,288	56,288	20
22 Ending fund balance			$1,200	$2,446	$6,492	$3,517	$3,642	$7,267	$6,202	$7,637	$9,672	$6,378	$6,084	$9,190	$8,463	$8,436	$11,409	$10,610	$10,511	$11,212	$10,248	$10,184	$11,020	$11,832	$13,744	$16,956	$14,714	$13,572	$17,384	$22,596	22
23 Coverage Ratio (line 19/line 20)			N.A.	N.A.	2.73	1.26	1.26	1.53	1.28	1.34	1.44	1.21	1.20	1.30	1.22	1.22	1.30	1.25	1.25	1.27	1.22	1.21	1.23	1.22	1.26	1.32	1.25	1.23	1.31	1.40	23
24 Bond Issue - Current			$40,500		$108,100			$90,500			$87,700			$79,100			$50,400			$56,800			$63,900			$70,700					24
25 Bond Issue - Cumulative			$40,500	$40,500	$148,600	$148,600	$148,600	$239,100	$239,100	$239,100	$326,800	$326,800	$326,800	$405,900	$405,900	$405,900	$456,300	$456,300	$456,300	$513,100	$513,100	$513,100	$577,000	$577,000	$577,000	$647,700	$647,700	$647,700	$647,700	$647,700	25

(1) Equivalent dwelling units
(2) Equivalent single family dwellings
(3) Shows increase in impacted rate(s) only; other rates increased at 4% inflation rate.
(4) Assumes rates increased 1/1/93 in 1992/93 and on 7/1 in all subsequent years.

TABLE __ USER FEE IMPACT-RECOMMENDED PLAN
PROJECT PHASING SCHEDULE 1 - 15 YEAR CONSTRUCTION SCHEDULE
Rate Option 4 - Sacramento Regional users only

BARTLE WELLS ASSOCIATES
SCSOREC9 7/6/92

	Annual Increase	Budget 1991/92	1992/93	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08
			Pro forma															
Number of City EDU's (1)																		
1 Combined sewer system			31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900
2 Sanitary sewer-other		103,800	72,600	73,300	74,000	74,700	75,400	76,200	77,000	77,800	78,600	79,400	80,200	81,000	81,800	82,600	83,400	84,200
3 Total sewer	0.7%	103,800	104,500	105,200	105,900	106,600	107,300	108,100	108,900	109,700	110,500	111,300	112,100	112,900	113,700	114,500	115,300	116,100
4 Storm drainage	2.0%	176,500	180,000	183,600	187,300	191,000	194,800	198,700	202,700	206,800	210,900	215,100	219,400	223,800	228,300	232,900	237,600	242,400
5 Number of Sac. Regional ESD's(2)	2.0%	450,000	459,000	468,200	477,600	487,200	496,900	506,800	516,900	527,200	537,700	548,500	559,500	570,700	582,100	593,700	605,600	617,700
City rates/month:																		
6 Combined sewer system		$7.07	$7.35	$7.65	$7.95	$8.27	$8.60	$8.95	$9.30	$9.68	$10.06	$10.47	$10.88	$11.32	$11.77	$12.24	$12.73	$13.24
7 Sanitary sewer-other		$7.07	$7.35	$7.65	$7.95	$8.27	$8.60	$8.95	$9.30	$9.68	$10.06	$10.47	$10.88	$11.32	$11.77	$12.24	$12.73	$13.24
8 Storm drainage-combined system		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69	$16.32	$16.97
9 Storm drainage-other		$9.06	$9.42	$9.80	$10.19	$10.60	$11.02	$11.46	$11.92	$12.40	$12.90	$13.41	$13.95	$14.51	$15.09	$15.69	$16.32	$16.97
Sacramento Regional rate/month:																		
10 -O&M		$8.15	$8.48	$8.82	$9.17	$9.53	$9.92	$10.31	$10.72	$11.15	$11.60	$12.06	$12.55	$13.05	$13.57	$14.11	$14.68	$15.26
11 -Capital rate increase-current year(3)----)			5%	5%	5%	4%	4%	5%	5%	5%	7%	6%	5%	4%	4%	3%	0%	0%
12 -Capital rate (cumulative)		$0.00	$0.42	$0.86	$1.32	$1.70	$2.10	$2.62	$3.15	$3.71	$4.52	$5.25	$5.87	$6.40	$6.94	$7.36	$7.36	$7.36
13 -Sacramento Regional rate -Total		$8.15	$8.90	$9.68	$10.49	$11.24	$12.02	$12.93	$13.88	$14.86	$16.12	$17.31	$18.42	$19.44	$20.51	$21.47	$22.04	$22.63
14 Highest total user rate		$24.28	$25.68	$27.13	$28.63	$30.11	$31.64	$33.34	$35.10	$36.94	$39.08	$41.19	$43.25	$45.27	$47.37	$49.41	$51.09	$52.84
15 Affordability Index rate		$46.23	$48.08	$50.00	$52.00	$54.08	$56.24	$58.49	$60.83	$63.27	$65.80	$68.43	$71.17	$74.01	$76.97	$80.05	$83.25	$86.58
16 Beginning Capital Fund Balance (X$000)			$0	$1,200	$2,446	$6,492	$5,177	$6,362	$11,047	$6,444	$5,641	$10,638	$7,492	$9,146	$15,400	$11,443	$11,286	$12,229
17 Sac. Regional-Additional capital revenue(4)			1,200	4,900	7,600	10,000	12,500	15,900	19,600	23,500	29,200	34,500	39,400	43,800	48,500	52,400	53,500	54,600
18 Interest income			0	100	200	300	300	400	400	300	400	500	400	600	700	600	600	700
19 Total Capital funds available			1,200	6,200	10,246	16,792	17,977	22,662	31,047	30,244	35,241	45,638	47,292	53,546	64,600	64,443	65,386	67,529
20 Capital Project debt service			0	3,754	3,754	11,615	11,615	11,615	24,603	24,603	24,603	38,146	38,146	38,146	53,157	53,157	53,157	53,157
22 Ending fund balance			$1,200	$2,446	$6,492	$5,177	$6,362	$11,047	$6,444	$5,641	$10,638	$7,492	$9,146	$15,400	$11,443	$11,286	$12,229	$14,372
23 Coverage Ratio (line 19/line 20)			N.A.	N.A.	2.73	1.45	1.55	1.95	1.26	1.23	1.43	1.20	1.24	1.40	1.22	1.21	1.23	1.27
24 Bond Issue - Current			$40,500		$84,800			$140,100			$146,100			$162,100				
25 Bond Issue - Cumulative			$40,500	$40,500	$125,300	$125,300	$125,300	$265,400	$265,400	$265,400	$411,500	$411,500	$411,500	$573,600	$573,600	$573,600	$573,600	$573,600

(1) Equivalent dwelling units
(2) Equivalent single family dwellings
(3) Shows increase in impacted rate(s) only; other rates increased at 4% inflation rate.
(4) Assumes rates increased 1/1/93 in 1992/93 and on 7/1 in all subsequent years.

TABLE __ USER FEE IMPACT-RECOMMENDED PLAN
PROJECT PHASING OPTION 1 - 27 YEAR CONSTRUCTION SCHEDULE
Rate Option 1 - All users

BARTLE WELLS ASSOCIATES
SCSOREC1 6/26/92

	Annual Increase	Budget 1991/92	1992/93	1993/94	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00	2000/01	2001/02	2002/03	2003/04	2004/05	2005/06	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13	2013/14	2014/15	2015/16	2016/17	2017/18	2018/19	2019/20	
			Pro forma																												
Number of EDU's (1)																															
1 Combined sewer system			31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	31,900	1
2 Sanitary sewer-other		103,800	72,600	73,300	74,000	74,700	75,400	76,200	77,000	77,800	78,600	79,400	80,200	81,000	81,800	82,600	83,400	84,200	85,000	85,800	86,600	87,400	88,200	89,000	89,800	90,700	91,600	92,500	93,400	94,300	2
3 Total sewer	0.7%	103,800	104,500	105,200	105,900	106,600	107,300	108,100	108,900	109,700	110,500	111,300	112,100	112,900	113,700	114,500	115,300	116,100	116,900	117,700	118,500	119,300	120,100	120,900	121,700	122,600	123,500	124,400	125,300	126,200	3
4 Storm drainage	2.0%	176,500	180,000	183,600	187,300	191,000	194,800	198,700	202,700	206,800	210,900	215,100	219,400	223,800	228,300	232,900	237,600	242,400	247,200	252,100	257,100	262,200	267,400	272,700	278,200	283,800	289,500	295,300	301,200	307,200	4
5 Average monthly user rate/EDU: Increase(2)--->			24%	10%	10%	8%	8%	7%	5%	5%	5%	4%	5%	5%	5%	4%	0%	4%	0%	4%	4%	3%	0%	3%	0%	4%	4%	0%	0%	0%	5
6 Combined sewer system		$7.07	$8.77	$9.64	$10.61	$11.46	$12.37	$13.24	$13.90	$14.60	$15.33	$15.94	$16.74	$17.57	$18.45	$19.19	$19.19	$19.96	$19.96	$20.76	$21.59	$22.23	$22.23	$22.90	$22.90	$23.82	$24.77	$24.77	$24.77	$24.77	6
7 Sanitary sewer-other		$7.07	$8.77	$9.64	$10.61	$11.46	$12.37	$13.24	$13.90	$14.60	$15.33	$15.94	$16.74	$17.57	$18.45	$19.19	$19.19	$19.96	$19.96	$20.76	$21.59	$22.23	$22.23	$22.90	$22.90	$23.82	$24.77	$24.77	$24.77	$24.77	7
8 Storm drainage-combined system		$9.06	$11.23	$12.36	$13.59	$14.68	$15.86	$16.97	$17.81	$18.70	$19.64	$20.43	$21.45	$22.52	$23.64	$24.59	$24.59	$25.57	$25.57	$26.60	$27.66	$28.49	$28.49	$29.35	$29.35	$30.52	$31.74	$31.74	$31.74	$31.74	8
9 Storm drainage-other		$9.06	$11.23	$12.36	$13.59	$14.68	$15.86	$16.97	$17.81	$18.70	$19.64	$20.43	$21.45	$22.52	$23.64	$24.59	$24.59	$25.57	$25.57	$26.60	$27.66	$28.49	$28.49	$29.35	$29.35	$30.52	$31.74	$31.74	$31.74	$31.74	9
10 Sacramento Regional rate		$8.15	$8.48	$8.82	$9.17	$9.53	$9.92	$10.31	$10.72	$11.15	$11.60	$12.06	$12.55	$13.05	$13.57	$14.11	$14.68	$15.26	$15.88	$16.51	$17.17	$17.86	$18.57	$19.31	$20.09	$20.89	$21.73	$22.60	$23.50	$24.44	10
11 Highest total user rate		$24.29	$28.48	$30.82	$33.37	$35.67	$38.14	$40.52	$42.44	$44.45	$46.57	$48.43	$50.73	$53.14	$55.67	$57.89	$58.46	$60.80	$61.41	$63.86	$66.42	$68.58	$69.30	$71.56	$72.33	$75.23	$78.24	$79.10	$80.01	$80.95	11
12 Affordability Index rate		$46.23	$48.08	$50.00	$52.00	$54.08	$56.24	$58.49	$60.83	$63.27	$65.80	$68.43	$71.17	$74.01	$76.97	$80.05	$83.25	$86.59	$90.05	$93.65	$97.40	$101.29	$105.34	$109.56	$113.94	$118.50	$123.24	$128.17	$133.29	$138.62	12
13 Beginning Fund Balance (X$000)		$2,353	$1,200	$760	$2,366	$7,272	$4,257	$4,342	$8,027	$5,322	$5,017	$7,412	$3,578	$2,644	$4,910	$3,343	$4,576	$5,109	$4,140	$2,371	$3,902	$3,598	$5,894	$7,290	$5,462	$2,534	$3,706	$3,124	$1,342	$2,114	13
Revenue (3)																															
14 Combined sewer user charge		0	3,000	3,700	4,100	4,400	4,700	5,100	5,300	5,600	5,900	6,100	6,400	6,700	7,100	7,300	7,300	7,600	7,600	7,900	8,300	8,500	8,500	8,800	8,800	9,100	9,500	9,500	9,500	9,500	14
15 Other sanitary user charge		8,811	6,900	8,500	9,400	10,300	11,200	12,100	12,800	13,600	14,500	15,200	16,100	17,100	18,100	19,000	19,200	20,200	20,400	21,400	22,400	23,300	23,500	24,500	24,700	25,900	27,200	27,500	27,800	28,000	15
16 Storm drainage user charge		19,159	21,900	27,200	30,600	33,600	37,100	40,500	43,300	46,400	49,700	52,700	56,500	60,500	64,800	68,700	70,100	74,400	75,900	80,500	85,300	89,600	91,400	96,000	98,000	103,900	110,300	112,500	114,700	117,000	16
17 Interest income		2,043	0	100	200	300	200	300	300	300	300	300	200	200	200	200	200	200	200	200	200	200	300	300	200	200	200	100	100	100	17
18 Other		3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	3,310	18
19 Total revenues		33,353	35,110	42,810	47,610	51,910	56,510	61,310	65,010	69,210	73,710	77,610	82,510	87,810	93,510	98,510	100,110	105,710	107,410	113,310	119,510	124,910	127,010	132,910	135,010	142,410	150,510	152,910	155,410	157,910	19
Operating Expenses																															
20 existing (4)		25,829	26,900	28,000	29,100	30,300	31,500	32,800	34,100	35,500	36,900	38,400	39,900	41,500	43,200	44,900	46,700	48,600	50,500	52,500	54,600	56,800	59,100	61,500	64,000	66,600	69,300	72,100	75,000	78,000	20
21 additional		0	400	900	1,000	1,700	1,700	1,300	1,400	1,500	1,600	1,700	1,800	1,900	2,000	2,100	2,200	2,300	2,400	2,500	2,600	2,700	2,800	2,900	3,000	3,100	3,200	3,300	3,400	3,500	21
22 Other (net transfer)		174	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	22
23 Total operating expenses		26,003	27,300	28,900	30,100	32,000	33,200	34,100	35,500	37,000	38,500	40,100	41,700	43,400	45,200	47,000	48,900	50,900	52,900	55,000	57,200	59,500	61,900	64,400	67,000	69,700	72,500	75,400	78,400	81,500	23
24 Net Revenue		7,350	7,810	13,910	17,510	19,910	23,310	27,210	29,510	32,210	35,210	37,510	40,810	44,410	48,310	51,510	51,210	54,810	54,510	58,310	62,310	65,410	65,110	68,510	68,010	72,710	78,010	77,510	77,010	76,410	24
Debt Service																															
25 existing		1,764	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	1,750	25
26 additional		0	0	3,754	3,754	13,775	13,775	13,775	22,165	22,165	22,165	30,294	30,294	30,294	37,627	37,627	37,627	42,229	42,229	42,229	47,564	47,564	47,564	53,588	53,588	53,588	60,042	60,042	56,288	56,288	26
27 Total debt service		1,764	1,750	5,504	5,504	15,525	15,525	15,525	23,915	23,915	23,915	32,044	32,044	32,044	39,377	39,377	39,377	43,979	43,979	43,979	49,314	49,314	49,314	55,338	55,338	55,338	61,792	61,792	58,038	58,038	27
28 Capital improvements		6,720	6,500	6,800	7,100	7,400	7,700	8,000	8,300	8,600	8,900	9,300	9,700	10,100	10,500	10,900	11,300	11,800	12,300	12,800	13,300	13,800	14,400	15,000	15,600	16,200	16,800	17,500	18,200	18,900	28
29 Net revenue/(expense)		(1,166)	(440)	1,606	4,906	(3,015)	85	3,685	(2,705)	(305)	2,395	(3,234)	(924)	2,266	(1,567)	1,233	533	(969)	(1,769)	1,521	(304)	2,296	1,396	(1,828)	(2,928)	1,172	(582)	(1,782)	772	(525)	1,586	29
30 Ending fund balance		$1,187	$760	$2,366	$7,272	$4,257	$4,342	$8,027	$5,322	$5,017	$7,412	$3,578	$2,644	$4,910	$3,343	$4,576	$5,109	$4,140	$2,371	$3,902	$3,598	$5,894	$7,290	$5,462	$2,534	$3,706	$3,124	$1,342	$2,114	$1,586	30
31 Coverage Ratio (line 24 ÷ line 13/line 27)		5.53	5.15	2.67	3.61	1.75	1.78	2.02	1.57	1.57	1.68	1.45	1.39	1.47	1.35	1.39	1.42	1.36	1.33	1.38	1.34	1.40	1.44	1.37	1.33	1.36	1.32	1.30	1.35	1.35	31
32 Bond Issue - Current			40,500		108,100			90,500			87,700			79,100			50,400			56,200			63,900			70,700					32
33 Bond Issue - Cumulative			40,500	40,500	148,600	148,600	148,600	239,100	239,100	239,100	326,800	326,800	326,800	405,900	405,900	405,900	456,300	456,300	456,300	512,500	513,100	513,100	577,000	577,000	577,000	647,700	647,700	647,700	647,700	647,700	33

(1) Equivalent dwelling units
(2) Shows increase in impacted rate(s) only; other rates increased at 4% inflation rate.
(3) Assumes rates increased 1/1/93 in 1992/93 and on 7/1 in all subsequent years.
(4) Inflated at 4% annually beginning in 1992/93.