City of SACRAMENTO
Department of Utilities

Combined Sewer System Improvement Plan Update

Final Report

December 2015

FINAL

AECOM

Executive Summary

The City of Sacramento (City) owns and operates a combined sewer system (CSS) that conveys residential and commercial wastewater and storm water runoff from approximately 11.7 square miles in downtown Sacramento, East Sacramento, Oak Park, and the Land Park area. There are 5.8 square miles of separated areas of the City north, east, and south of the CSS that contribute sanitary flows to the CSS. The City also includes approximately 76 square miles of separated areas that are not served by the CSS. The CSS serves approximately 205,000 people. The CSS includes four key facilities to manage the collected flow: Sumps 1/1A, Sumps 2/2A, Pioneer Reservoir, and the Combined Wastewater Treatment Plant (CWTP). Sumps 1/1A and 2/2A pump up to 60 million gallons per day (mgd) of flows to the Sacramento Regional County Sanitation District's Regional Wastewater Treatment Plant (SRWTP). Pioneer Reservoir and CWTP provide additional storage and, when needed, primary treatment, and disinfection of combined sewage prior to discharge to the Sacramento River. The CSS is regulated under the August 2015 National Pollutant Discharge Elimination System (NPDES) waste discharge permit, No. CA0079111. The permit allows for CSS discharge to the Sacramento River at six locations: two for primary treated (plus disinfection) effluent, and four that can discharge untreated combined sewage. The secondary treated effluent from SRWTP is discharged to the Sacramento River at a permitted location under a separate NPDES permit.

The City's Department of Utilities (DOU) developed a hydrologic and hydraulic (H&H) numerical computer simulation model of its CSS in the early 1980s using the U.S. Environmental Protection Agency's (EPA's) Storm Water Management Model (SWMM). The customized software application was referred to as SAC-SWMM. The model was continuously updated, maintained, and used throughout the 1990s and early 2000s. In 1995, the City adopted a Long-Term Combined Sewer System Improvement Plan (CSSIP). The CSSIP prescribed improvements to reduce combined sewer overflows (CSOs) to the Sacramento River and the discharge of combined sewage to streets, surface areas, and properties. SAC-SWMM was used to simulate and evaluate many of the alternatives considered for the CSSIP, including increasing pumping and treatment capacity, in-line storage pipelines, and off-line storage facilities.

In September 2008, the DOU entered into a contract with AECOM to provide a CSSIP update. The scope of services included reviewing all relevant information developed since the initial CSSIP; reviewing the existing model relative to other software available in the industry and, based on that analysis, recommending software for use in updating the CSSIP; conducting intensive flow and rainfall monitoring during the winter of 2008–2009; calibrating the model based on that data set and other existing data and information; and using the updated and re-calibrated model to re-analyze seven projects that were to be installed over an approximate 8-year period.

In September 2010, the DOU entered into a contract with AECOM to build on the CSSIP and develop a Phase 2 Update (hereafter "CSSIP update"). The scope of services included updating the sanitary sewer flow estimates, maximizing the performance of the seven projects evaluated in the previous effort, refining the model resolution by adding more pipes,

refining sub-catchment delineation in the six wet areas as defined in the NPDES permit, and evaluating the improvement projects to manage and control outflows and flooding as well as reduce CSO volume and frequency. The scope also included developing a CSS improvement plan that described each of the improvement projects, performed cost/benefit analysis, and developed a prioritization matrix for all the improvement projects. Additional services added to this scope included developing a CSS discharge analysis by simulating an average annual rainfall data set to estimate combined sewer overflows to the river for baseline and project scenarios. The purpose of this final report is to summarize the different analyses and tasks under the CSSIP update and include the technical memoranda and documents developed through this contract.

The first task in the CSSIP update was to refine the sanitary flow representation and the influence of the ground water table during dry weather for the existing conditions model and develop a design flow estimate for future conditions. The equivalent single dwelling (ESD) was used as a unit of sanitary flow rate generation. Four data sets (County Parcel Data, Water Use Consumption, Traffic Zone Analysis, and Pumping Data to SRWTP) were used to generate a per ESD sanitary flow estimate for existing conditions. DOU's interim water conservation plan was used to generate a projected sanitary flow rate per ESD for future conditions. Historical ground water data for nine wells for 40+ years was summarized and compared with the sewer pipe depth. The analysis concluded that the ground water table level did not cause an increase in groundwater infiltration.

The second task in the CSSIP update was to evaluate existing projects to identify opportunities to maximize their storage or conveyance capacity for the 10-year design storm and study its impact on reduced flooding on the surface. Opportunities for optimization were identified with the DOU staff, and the H&H model results were tabulated for total flooding on the surface (outflow from the system and ponding on the street), combined sewer overflow to the river, and total treated flows to the SRWTP. Based on this analysis, optimization of four existing projects was found to provide flood reduction. The optimized configuration of these four existing projects was included in the baseline scenario for identifying additional projects under the CSSIP improvement plan.

The third task in the CSSIP update included refining the H&H model for a higher resolution within the six wet areas, as described in the NPDES permit. Additional smaller-diameter pipes were added in the six wet areas, and the sub-catchment delineation was further refined to improve the resolution of the H&H model. The purpose of improving the model resolution was to improve the accuracy of the flooding results in the wet areas. This task also included a model sensitivity analysis to identify the sensitivity of the H&H model to different simulation inputs, including spatial and temporal variability of design storms, infiltration parameters, and initial storage estimates. The model was not found to be significantly sensitive to any input or model parameter.

The final task in the CSSIP update included using the revised H&H model to simulate the 5-year and 10-year design storms and identify capacity deficiencies using the Innovyze Integrated Catchment Model (ICM) two-dimensional (2D) module. The 2D module allowed simulation of the transport of overland flows generated by outflows from the system or ponding on the surface resulting from a storm event. The overland flooding results as

simulated by the ICM 2D module were validated using historical and anecdotal knowledge of DOU staff. The next steps involved identifying additional projects to address flooding in the six wet areas (first step) and city-wide (next step). These additional projects were simulated in the H&H model to quantify the benefits provided by each project and optimize the project elements, if needed.

Three programs, including water conservation, reduction in rainfall derived inflow and infiltration (RDII) in separated sanitary areas, and green infrastructure were also included at a city-wide level implementation to further reduce sanitary and storm water flows in the combined sewer system during the design storms. The total capital costs for each project and program were estimated based on the actual construction costs for similar facilities built by DOU and adjusted to 2013 dollars. The contingency costs were added to the construction costs to develop the total capital costs. The projects were prioritized using various criteria, including CSS discharges to the river for NPDES compliance; cost-effectiveness (flood reduction per project capital cost); existing sewer age and condition; cost sharing with the water, streets, and parks and recreation departments; and city/community interests. Programs are assumed to be developed throughout the implementation period of the CSSIP. Figure 1 presents the projects and programs that were recommended in the CSSIP. Figure 2 presents the recommended implementation plan for the projects and programs with the cumulative costs.

Lastly, additional services were performed to evaluate CSS discharges to the river for NPDES compliance. Statistical analysis was done using the long-term historical rainfall dataset to find a statically average annual rainfall year (i.e., a typical year). The H&H model was validated for four observed discharge events to the river, and the model accuracy was found to be acceptable to use for this analysis. Existing and near-term project scenarios were simulated for the typical year, and the annual discharge volume and frequency of CSOs to the river were tabulated. The analysis found that CSS discharges to the river using typical year rainfall for existing conditions were slightly higher than the average number of discharges from 2001–2008 reported in the NPDES Permit[1]. However, the typical year was simulated using the operational logic as defined in the NPDES permit, whereas the facilities may have been operated differently during the actual storms in 2001–2008. Also, average of annual rainfall for the 2001-2008 period is less than the typical year rainfall; therefore, a higher CSO frequency in the typical year simulation was expected.

Technical Documents

The following technical memorandums (TMs) and documents are included in this report:

Sewer Flow Projection (July 2011) – TM summarizes relevant information to develop dry weather flow estimates, including sanitary flow and ground water flows in areas proximal to

[1] ORDER NO. R5-2010-0004, NPDES NO. CA0079111

the American and Sacramento Rivers.

Model Sensitivity Analysis (November 2011) – TM summarizes the H&H model's sensitivity to different simulation inputs, including spatial and temporal variability of design storms, infiltration parameters, and initial storage estimates.

Optimization of Existing Projects (November 2011) – TM summarizes the findings of flood reduction in the system for design storms to compare the benefits of optimizing existing and proposed projects.

CSS Discharge Analysis (May 2013) – TM summarizes the evaluation of whether CSS discharges are compliant with the NPDES permit description of discharges. The results of this TM supported the NPDES water quality analysis by characterizing the frequency and volume of treated and untreated CSS discharges for the baseline conditions.

Final CSS Improvement Plan Update (August 2014) – TM summarizes the CSSIP improvement projects and programs identified to reduce flooding in 5-year and 10-year design storms, constructability and cost/benefit analysis, and project prioritization for implementation.

Alhambra Alternatives (April 2015) – TM summarizes the cost and benefit performance summary for the Alhambra Blvd. Tunnel and compares cost/benefit performance for the project alternatives.

Figure 1: CSSIP Projects and Programs

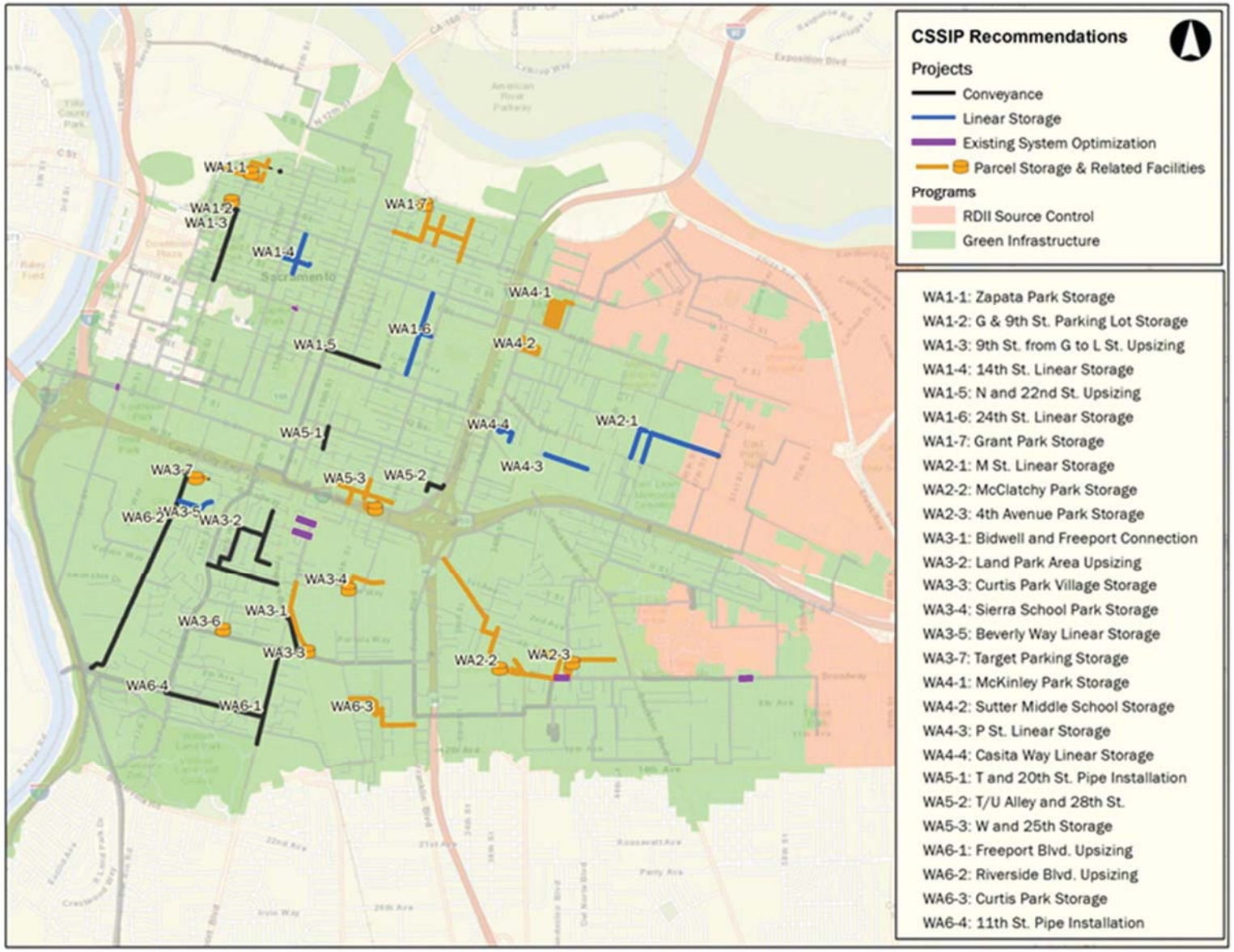
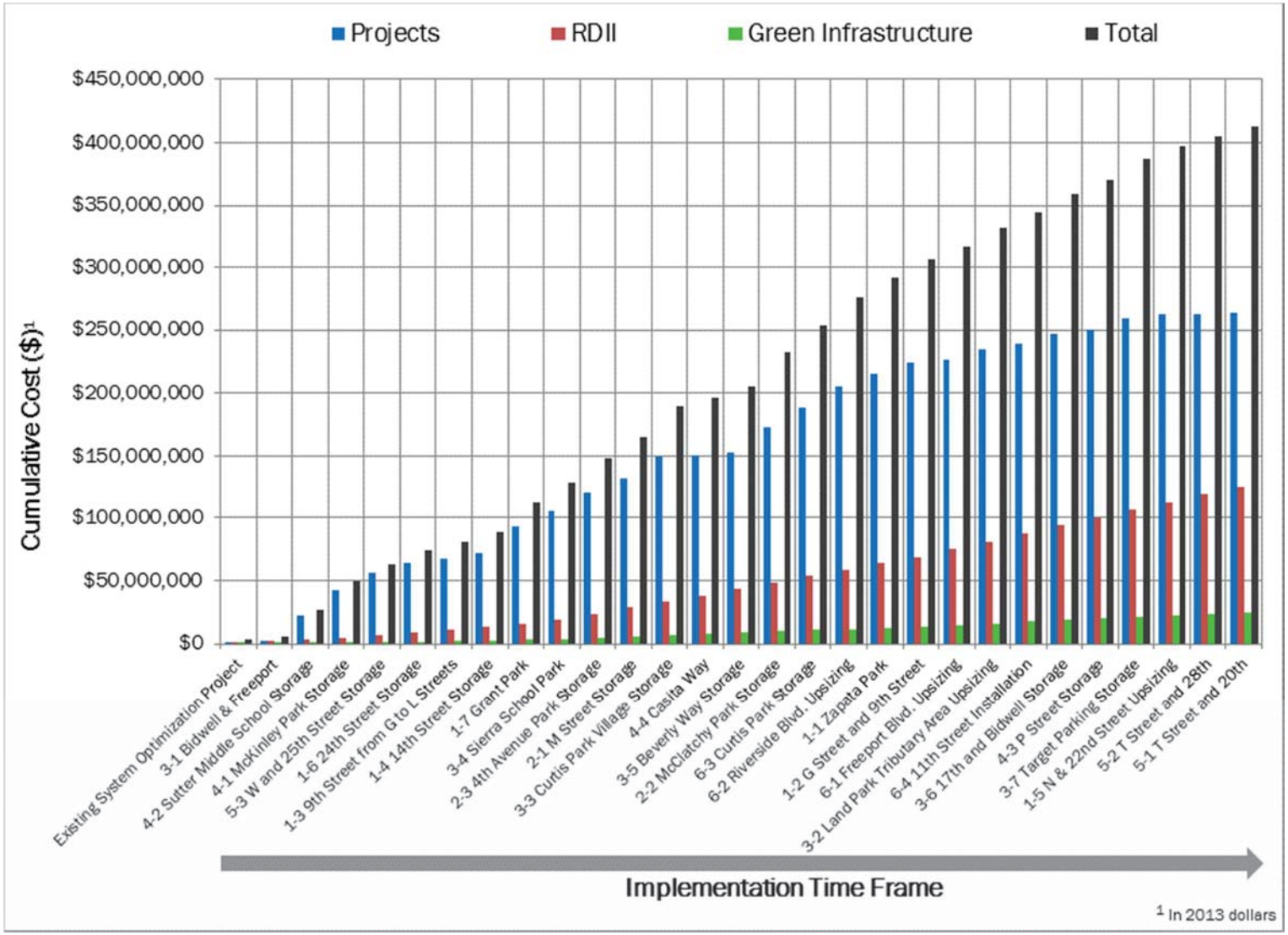


Figure 2: CSSIP Project and Program Implementation Prioritization



City of SACRAMENTO
Department of Utilities

Combined Sewer System (CSS) Improvement Plan Update – Phase 2

Task 2 – Sewer Flow Projection Technical Memorandum

July 11, 2011

Version 10 (Final)

Table of Contents

1.0 Introduction 1

2.0 Background 1

3.0 Methodology 1

3.1 Design Model Landuses 3

3.2 Design Base Sanitary Flow Estimates 4

3.2.1. Sanitary Flow Per Day Per ESD 4

3.2.2. Equivalent Single-Family Dwellings (ESD) 5

3.3 Landuse Based Diurnal Patterns 8

3.4 Sanitary Sub-catchments 9

3.5 Groundwater Infiltration Estimates 11

3.6 Phase 2 Calibrated Model 12

3.7 Phase 2 Design Model 13

3.8 Modification to Wet Weather 14

3.9 Projecting Future Flows 14

3.10 Groundwater Analysis 15

Appendix A – ESD Estimates using Water Billing Data

Appendix B – ESD Estimates using Traffic Analysis Zones data

Appendix C – Sanitary Flow Diurnals for different Landuses

Appendix D – Phase 2 Calibrated Model Result Comparison with Observed Flow

Appendix E – Ground Water Depth for Wells in Study Area

List of Figures

Figure 1: Long Term Dry Weather Flows from Study Area 3
Figure 2: Projected Water Connections in the Study Area 14
Figure 3: Ground Water Wells in Study Area 16

List of Tables

Table 1. Assignment of Sacramento Co. GIS Parcel Landuse to Model Landuse 4
Table 2. Estimating General Equivalent ESD for Parcels 5
Table 3. Estimating Specific Equivalent ESD for Parcels 6
Table 4. Comparing Equivalent ESDs between Method 1 and 2 6
Table 5. Comparing Equivalent ESDs between Method 1 and 3 7
Table 6. Comparing Average Daily Sanitary Flows between Method 1 and 4 8
Table 7. Temporary Flow Meters for Diurnal Patterns 8
Table 8. Peaking Factors 9
Table 9. Sub-catchment Types 10
Table 10. Sanitary Sub-catchments by Landuse Types 10
Table 11. Rainfall Independent Groundwater Infiltration Rate 11
Table 12. Dry Weather Calibration Results 13
Table 13. Design Model Input 13
Table 14. 2030 ESD Estimates 15
Table 15. Average Water Depth in Study Area 17

1.0 Introduction

In September 2008, AECOM under contract with the City of Sacramento (City), Department of Utilities (DOU) updated the Sacramento Combined Sewer System (CSS) model to InfoWorks CS. The efforts to revise, extend, and update the CSS model included extending the physical network representation, updating dry and wet weather flow inputs, refining model assumptions and techniques and validating the model against data obtained during a flow monitoring program conducted as part of the study. This model, referred to as the "Phase 1" model was primarily developed to study the CSS system during rainfall events. The Phase 1 methodologies used to characterize dry weather flow (DWF) were sufficient for the purpose of simulating the combined sewer response to rainfall events. However, the DOU would also like to use the model to analyze DWF changes due to factors such as new development, land use changes, and population increases.

In September 2010, AECOM signed a new contract with DOU that includes a task to develop an approach and methodologies to refine and update the DWF characteristics in the model. The InfoWorks CS model developed under this contract will be referred to as the "Phase 2" model. The "Phase 2" model will include a calibrated model that will provide the same calibration results as "Phase 1", and a design model that represents DWF for design conditions. This technical memorandum documents the methodologies used to refine the DWF characterization in the Phase 2 model, and summarizes the results of the updated DWF characterization.

2.0 Background

DWF in the CSS is flow that occurs during dry weather, without a rainfall dependent component. It is the base sanitary flow as well as rainfall independent infiltration. In InfoWorks, the components of DWF (i.e. base sanitary flow and rainfall independent infiltration) are loaded into the model using the sub-catchments element, which represent parcels, buildings, sub-basins, etc. The base sanitary flow is primarily the waste water generated by the population living and working in the study area as well as wastewater generated by commercial and industrial processes. This base sanitary flow in InfoWorks is expressed using a population value and flow per capita value for each sub-catchment. Additionally, the base sanitary flow varies every hour and typically follows a repeatable weekday and weekend day 24-hour pattern; it is different for each landuse (i.e. residential, commercial, industrial, etc). A diurnal pattern is a unitless variation of base sanitary flow over a 24-hour period.

In the Phase 1 model, each sub-catchment was comprised of several different landuse types. Population and flow per capita estimates were developed based on landuse types, but then lumped together at a sub-catchment level. Weekday and weekend diurnal patterns were developed based on data from the 13 flow meters installed during the Phase 1 flow monitoring program. Sub-catchments draining to a given flow meter were assigned the same diurnal pattern independent of landuse. This Phase 1 approach of lumped population, flow per capita and diurnal patterns per sub-catchment for base sanitary flow characterization provided good validation results when compared with flow monitoring data. However, it did not enable tracking of future changes in landuse nor enable DOU staff to see the impact of land use development and changes over time. The Phase 2 approach improves upon the previous dry-weather representation and enables DOU to better use the model for planning and design.

3.0 Methodology

The purpose of this task was to refine the model characterization of CSS DWF. InfoWorks uses sub-catchments to load the DWF into the model. The refinement included developing landuse-based sub-

catchments and developing landuse-based DWF input parameters for the sub-catchments. In general, the steps included developing landuse specific model sanitary sub-catchments in the model, loading dry weather flows for each landuse specific sanitary sub-catchment, calibrating the model to 2009 data, and adjusting the calibrated model to develop a design model that will allow DOU to use the model for planning and design scenarios. The methodology is divided into three steps below:

Step 1: Develop Phase 2 Model Sub-catchments

The purpose of this step was to develop landuse-based sanitary sub-catchments, and load dry weather flows for each sub-catchment. The parcel shapefile from Sacramento County GIS was used as the source for developing landuse-based sub-catchments. Each CSS parcel was assigned a modeling landuse type (Section 3.1) and dry weather flow loads were estimated using City's Design and Procedure Manual, DOU guideline documents, customer water billing data, TAZ analysis and other sources. Based on the discussion with DOU staff, the team decided to use equivalent single family dwelling (ESD) as a representation of population rather than actual population (Section 3.2). Flow meter data collected for the Phase 2 study were analyzed and landuse-based diurnals were developed and assigned to the parcels (Section 3.3). The parcels were grouped to form sub-catchments of a single landuse type (Section 3.4). The sanitary sub-catchments from the Phase 1 model were replaced by the Phase 2 sanitary sub-catchments, thus developing the Phase 2 design model sub-catchments.

Step 2: Develop Phase 2 Calibrated Model

The Phase 2 design model sub-catchments were developed as described in Step 1 using DOU manuals, guidelines and data. The subcatchments were then calibrated to the 2009 flow monitoring data and from a systemwide total. The 2009 systemwide DWF was determined based on the long-term DWF pumped from the study area to the Sacramento County Regional Sanitation District (SCRSD) wastewater treatment plant (Figure 1).

The long-term DWF excludes rainfall related storm flows and storage dewatering. It does include rainfall independent infiltration. Figure 1 indicates an overall declining DWF contribution from the CSS. This is believed to be related to water conservation measures that have been promoted and implemented, sewer system rehabilitation, better construction that protects against infiltration and inflow, and the economic downturn. In 2009, over the period of the flow monitoring program, the average systemwide DWF was 16.2 million gallons per day (mgd). The Phase 2 model was validated against the 2009 monitoring data by adjusting the base sanitary flow per ESD. The total ESD values for the study area from Step 1 were not changed in the calibrated model.

Step 3: Develop Phase 2 Design Model

DOU indicated their preference that average DWF be developed consistent with the City of Sacramento's Design and Procedure Manual. Following these design procedures, the overall DWF was calculated as 24.2 mgd, based on 70,650 ESDs at 310 gallons per day (gpd) per ESD, and 2.3 mgd rainfall independent infiltration. As seen in Figure 1, the highest recorded study area average DWF is during years 2002 to 2004 with a value of 20 mgd. While the calculated design value of 24.2 mgd is higher, this value is based on City's current design standards and consistent with other studies done by DOU staff. After the Phase 2 calibrated model conditions were established, the Phase 2 design model was finalized by changing the unit flow per ESD to 310 gpd/ESD and the rainfall independent infiltration flow rates (Section 3.5) to meet DOU's design flow recommendation of 24.2 mgd. The study area spatial distribution of ESDs and landuse diurnals are the same for both the Phase 2 calibrated and design models.

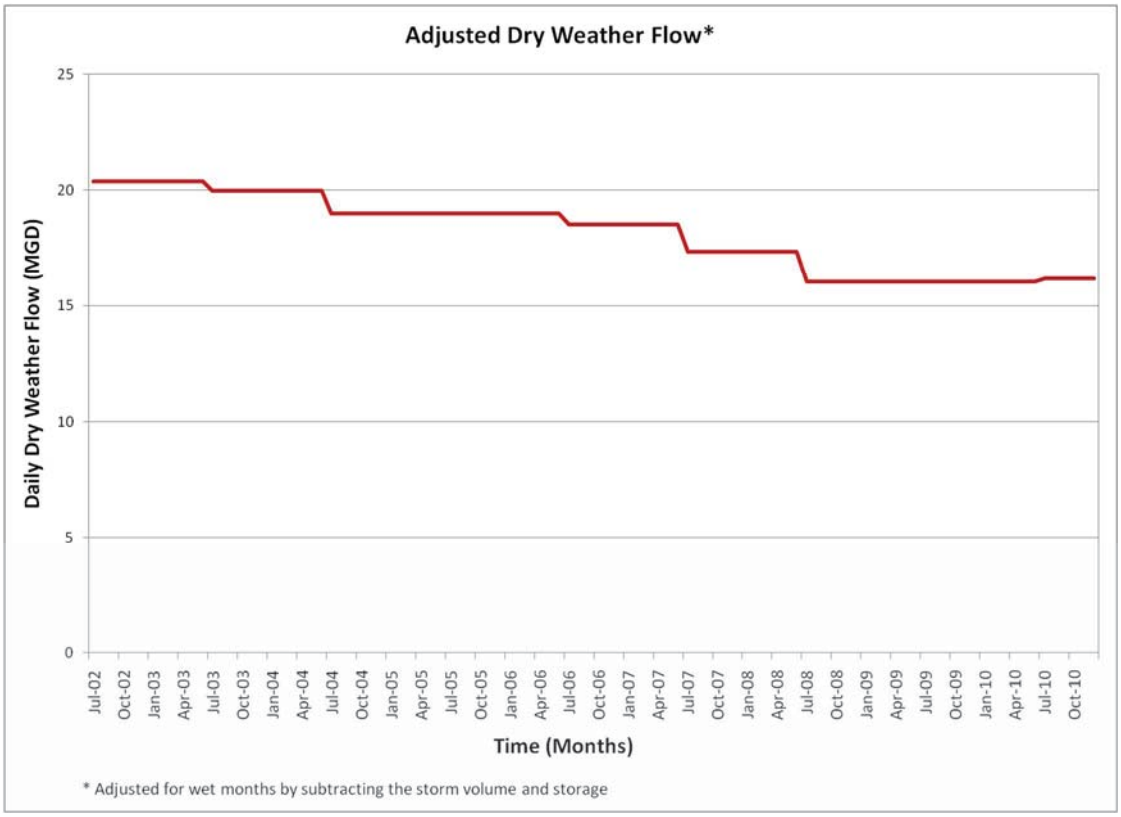


Figure 1. Long Term Dry Weather Flows from Study Area (provided by DOU)

3.1 Design Model Landuses

Parcel shapefiles, and detailed landuse descriptions for each parcel were available from Sacramento County GIS. The different detailed landuse types in the parcel dataset were grouped into the following 7 landuse types for the purposes of modeling:

1. Residential - Single Family
2. Residential - Multi Family
3. Commercial - High Rise
4. Commercial - Low Rise
5. Industrial
6. Not Contributing
7. Other

Table 1 presents a cross reference of the General and Specific attributes contained in the Sacramento County GIS to the 7 modeling landuse types.

Table 1. Assignment of Sacramento Co. GIS Parcel Landuse to Model Landuse

"General" Attribute	"Specific/Detail/Unit" Attribute	Landuse for Modeling
Residential	Unit = 1	Residential - Single Family
	Unit >= 2	Residential - Multi Family
Office	Detail = "Multiple Stories"	Commercial - High Rise
	Detail <> "Multiple Stories"	Commercial - Low Rise
Retail/Commercial	-	Commercial - Low Rise
Industrial	-	Industrial
Care/Health	Specific = "Residential Care Facility"	Residential - Multi Family
	Specific = "Retirement Home"	Residential - Multi Family
	Specific = "'Day Nursery"	Commercial - Low Rise
	Specific = "Skilled Nursing Facility"	Commercial - Low Rise
	Specific = "Acute Care Hospital"	Commercial - High Rise
	Specific = "Cemetary"	Not Contributing
Public/Utilities	Specific = "Public Housing"	Residential - Multi Family
	Specific = "SBE Housing"	Residential - Multi Family
	Specific = "City"	Commercial – Low Rise
	Specific = "County"	Commercial – Low Rise
	Specific = "Federal"	Commercial – Low Rise
	Specific = "Special District"	Commercial – Low Rise
	Specific = "State"	Commercial – Low Rise
	Specific = "Public School"	Commercial – Low Rise
Church/Welfare	-	Other
Recreational	-	Other
Miscellaneous	-	Other
Vacant	-	Not Contributing
No Data	-	Not Contributing
Exceptions: 1.UC Medical Center located along Stockton Blvd is classified under Commercial – High Rise. 2.Parks or open lands will be classified under Non-contributing.		

3.2 Design Base Sanitary Flow Estimates

The base sanitary flow is primarily the average daily waste water generated by the population living and working in the study area as well as wastewater generated by commercial and industrial processes. In InfoWorks CS, the base sanitary flow for each sub-catchment is generated as flow per day per ESD times the number of ESDs.

3.2.1. Base Sanitary Flow Per Day Per ESD

Each sanitary sub-catchment was assigned a design unit base sanitary flow of 310 gpd/ESD based on an analysis conducted independently by DOU[1] (as part of Sewer Impact Fee studies). The 310 gpd/ESD value is the design model unit flow per ESD. The design model is different than the 2009 calibration model. The unit flow value in the calibration model was adjusted downward to meet the 2009 calibrated model study area target flow of 16.2 mgd.

[1] Preliminary Hydraulic Analysis Separated Sewer Basins, November 2009, DOU Engineering Services Division, Pg 4.

3.2.2. Equivalent Single-Family Dwellings (ESD)

Based on discussion with DOU staff, the team decided to use ESD as a representation of population rather than actual population. This approach is consistent with other similar analyses conducted independently by DOU (as part of Sewer Development Fee studies). The flow contributions from other types of landuses (Commercial, Industrial, etc) was also expressed in terms of ESDs, reflecting appropriate sewage generation.

Four distinct sources of information and methods were used to develop and compare the ESD estimates for the sub-catchments. Method 1 provides a general approach to estimate ESDs for sub-catchments in the entire CSS, whereas Methods 2, 3 and 4 refine and validate the accuracy of ESD estimates developed using the Method 1 approach. The four methods are described below.

i. Method 1 - County of Sacramento's parcel data
ii. Method 2 - Water use consumption data
iii. Method 3 - Traffic analysis zone (TAZ) data
iv. Method 4 - Average dry-weather wastewater pumping data to SRCSD

i. <u>Method 1 - County of Sacramento's Parcel data</u>

Initial estimates of ESDs were developed using the County of Sacramento's Parcel data and DOU Technical Memorandum[2]. There are 33,897 parcels in the CSS with each parcel associated with a distinct identifier (Assessor's Parcel Number (APN)), the net parcel lot size, and a detailed landuse that includes number of units, specific landuse type, etc. Using the landuse categories listed in Table 1, and guidance on ESD Estimates from the DOU Technical Memorandum, the equations listed in Table 2 were utilized for ESD estimates for each of the 33,897 parcels.

Table 2. Estimating General Equivalent ESD for Parcels

Parcel Landuse for Modeling	Factor	Equivalent ESD
Residential - Single Family	-	1
Residential - Multi Family	Units	Units*0.75
Commercial – Low Rise	NPLS (Acres)	NPLS * 6
Commercial – High Rise	NPLS Acres)	NPLS * 12
Industrial	NPLS (Acres)	NPLS * 9
Other	NPLS (Acres)	NPLS * 6
Not-Contributing	-	0
NPLS = Net Parcel Lot Size in acres		

After the initial ESD estimates were developed, the City's design manual[3] was used to refine the ESD estimates for a smaller set of landuse types. The City's design manual provides ESD estimates for specific landuses such as offices, retail, warehouses, restaurants, churches, medical/dental facilities and banks. The ESD estimate was refined using the City's design manual where parcel data were available to this detail. Table 3 provides the ESD estimates from the City's design manual for these specific landuses. The total number of ESDs calculated using Method 1 for the CSS area is 70,650.

[2] Preliminary Hydraulic Analysis Separated Sewer Basins, November 2009, DOU Engineering Services Division, Pg 4.
[3] City of Sacramento's Design and Procedure Manual, September 1990.

Table 3. Estimating Specific Equivalent ESD for Parcels

Detailed Parcel Landuse Description	ESDs per 1000 sq. ft. of gross floor area
Office	0.2
Retail	0.2
Warehouses	0.1
Restaurants	1.9
Church	0.2
Medical/Dental	0.4
Banks	0.3

ii. Method 2 - Water use data

Water use data were available from 2007 to 2009 for a limited number of customer accounts. They were used to estimate ESDs. There are 3,178 distinct water use customer account numbers with monthly consumption data from Jan-Mar 2009 (same period as Phase 1 flow monitoring). Each customer account number has a parcel APN associated with it. For these 3,178 customer accounts, average monthly water use was calculated. ESDs were estimated assuming a 90% rate of return to the sewer system and 310 gpd/ESD.

Method 2 ESD estimates were then used to refine and validate Method 1 ESD estimates for each of the 3,178 accounts. The customer account number was joined to the Parcel data using the APN field, of which 3,164 accounts matched. ESDs using the water use data (Method 2) were compared with ESDs estimated using parcel data (Method 1). Appendix A shows a snapshot of the first 50 out of the 3,164 matched customer accounts, associated APN number, and estimated ESDs using Method 1 and Method 2. Table 4 shows the summary of ESDs estimated using Method 1 and Method 2, by the 7 model landuses. A few parcels under the “Not Contributing” classification have associated water consumption data but are believed to be parks or open area with irrigation demands.

Table 4. Comparing Equivalent ESDs between Method 1 and 2

Landuse for Modeling	# of Customer Accounts	Total ESDs using Method 2	Total ESDs using Method 1	% Difference in ESDs between Method 1 & 2
Residential - Single Family	771	521	771	-32%
Residential - Multi Family	697	2302	5058	-54%
Commercial - Low Rise	1310	6129	5529	11%
Commercial - High Rise	90	1527	797	92%
Industrial	150	741	1244	-40%
Other	105	416	410	1%
Not Contributing	41	657	0	-
Total	**3164**	**12293**	**13810**	**-11%**

Since Method 2 is based on actual water consumption data, ESD estimates using Method 2 are considered more accurate, than Method 1 which estimates ESDs based on land-use type.

Therefore, with the exception of the *Not Contributing* land use, the ESD estimate will be generated using Method 2 for the 3,164 parcels for which water use data was available. The remaining 30,733 (i.e. 33,897 – 3,164) parcels will utilize the Method 1 approach to estimate the equivalent ESDs. During dry weather calibration, if ESDs are found to need adjustment, the ESD for landuses with larger % differences in Table 3 will be adjusted before the ESDs for landuses with smaller % differences.

iii. Method 3 - Traffic analysis zone (TAZ) data

Sacramento Council of Governments (SACOG) has analyzed various traffic analysis zones (TAZs), including the County of Sacramento for years 2005, 2013, among others. For the TAZs that are within the CSS area, the estimates for residential dwelling units (single and multi-family) and employment count were summarized. Appendix B shows a map of the 114 TAZs that are within the CSS area and provides a table of the number of residential (single and multi-family) dwelling units and employment count for years 2005 and 2013.

Table 5 summarizes the number of residential dwelling units and employment counts for the 2005 and 2013 dataset. Using a straight-line method, these values were interpolated for 2009. The single and multi-family residential dwelling units were converted into ESDs and compared to Method 1 ESD estimates. As there is no straightforward approach to converting employment count to ESDs, the ESDs for other landuse types were not compared with Method 1. The small % difference between Method 1 and 3 residential values suggests that Method 1 is acceptable for estimating residential ESDs for the Phase 2 model.

Table 5. Comparing Equivalent ESDs between Method 1 and 3

Landuse for Modeling	Source	2005 TAZ	2013 TAZ	2009 TAZ[1]	Method 3 ESDs (Based on 2009 TAZ)	Method 1 ESDs	% Difference in ESDs between Methods 1 & 3
Residential – Single Family	Single Family Dwelling Units	19,439	20,228	19,834	19,834	21,881	9%
Residential - Multi Family	Multi Family Dwelling Units[2]	26,631	30,238	28,435	21,551	16,593	-30%
Commercial (Low and High Rise), Industrial and Other	Total Employment Count	202,019	209,944	205,981	-	32,176	-

[1] Straight line interpolation between 2005 and 2013 values
[2] ESDs = 0.75 * UNITS

iv. Method 4 - DOU's long-term wastewater pumping rate data

DOU provided long-term monthly flow data from the CSS pumped to the SCRSD wastewater treatment plant from 2002 to 2010 (Figure 1). Table 6 lists the average DWF pumped to the regional plant from the CSS per year over the period. For each year of the period, ESDs were

estimated for the entire CSS by dividing the DWF by 310 gpd/ESD. This value is then compared with ESDs from Method 1. The comparison suggests that the Method 1 calculated ESDs result in a conservative DWF design flow as compared to CSS DWF data from the 2002-2010 pumping record and is therefore appropriate as a basis for the Phase 2 design model.

Table 6. Comparing Average Daily Sanitary Flows between Method 1 and 4

Year	Average SCRSD Dry Weather Flow (MGD)	Method 4 ESDs = Flow/310 gpd	Method 1 ESDs	% Difference in ESDs between Method 1 and 4
2002	20.4	65,806	70,650	-7%
2003	20.2	65,161	70,650	-8%
2004	19.5	62,903	70,650	-12%
2005	19.0	61,290	70,650	-15%
2006	18.8	60,645	70,650	-16%
2007	17.9	57,742	70,650	-22%
2008	16.7	53,871	70,650	-31%
2009	16.0	51,613	70,650	-37%
2010	16.1	51,935	70,650	-36%

3.3 Landuse Based Diurnal Patterns

Diurnal pattern is the representation of the 24-hour time variation of base sanitary flow. It is often different for different landuse types. Industry standards and literature review provide “typical” diurnal patterns for different landuse types listed in Table 1 and used in the model. AECOM recommended installing flow meters for a 1-2 week period to capture this diurnal variation of flow for some of the most common landuse types. Table 7 lists the four flow meters installed, the manholes, primary landuse they intended to capture, and the installation period. For the rest of the landuses listed in Table 1, including Industrial and Other, a generic diurnal pattern was derived from industry standards and calibrated using the Phase 1 monitoring data.

Table 7. Temporary Flow Meters for Diurnal Patterns

Installation ID	Manhole	Pipe Diameter	Modeling Landuse Represented	Installation Period
U09-545D-01	609CC14	15”	Commercial – High Rise	01/11/2011-02/03/2011
U09-545D-02	813CC14	12”	Commercial – Low Rise	01/11/2011-02/03/2011
U09-545D-03E[1]	609DD16	18”	Commercial – Low Rise	01/11/2011-02/03/2011
U09-545D-04	910EE18	12”	Residential – Single & Multi Family	01/11/2011-02/03/2011
[1] Diurnal developed from this meter is not used in the model				

DOU staff compared the peaking factor for the system's average base sanitary flow with other utilities, and recommended a design peaking factor. DOU's recommended peaking factor is expressed as[4]:

Peaking Factor (PF) = 1.9 x Q^ -0.1
Where,
Q = Average Daily Flow (mgd)
Minimum peaking factor = 1.5
Maximum peaking factor = 3.0

Table 8 compares the peaking factor in the diurnal patterns developed from the Phase 2 flow metering data for the period of 01/11/2011-02/03/2011, with the peaking factors recommended by DOU staff.

Table 8. Peaking Factors

Installation ID	Manhole	Modeling Landuse Represented	Average Dry Sanitary Flow, Q (mgd)	Peaking Factor from the diurnal	Peaking Factor recommended by DOU staff = 1.9 x Q^ -0.1
U09-545D-01	609CC14	Commercial – High Rise	0.13	1.76	2.42
U09-545D-02	813CC14	Commercial – Low Rise	0.024	1.96	2.76
U09-545D-03E[1]	609DD16	Commercial – Low Rise	0.08	2.05	2.55
U09-545D-04	910EE18	Residential – Single & Multi Family	0.067	2.00	2.49

[1] Diurnal developed from this meter is not used in the model

The peaking factors in the landuse based flow meter diurnal patterns are lower than the factors calculated using the DOU recommended equation. The landuse based diurnal patterns were adjusted so that the peaking factors matched the design peaking factor recommended by DOU. Appendix C shows the landuse based weekday and weekend diurnal patterns calculated from the flow monitoring data, and the diurnal patterns adjusted to match the design peaking factor. The adjusted diurnal patterns are used in the calibrated and design models.

3.4 Sanitary Sub-catchments

The Phase 1 model included 430 sewershed subcatchments with average areas of approximately 26 acres. They were based on the original Sacramento SWMM drainage subcatchments and represented areas that generate base sanitary flow, wet weather inflow, or rainfall dependent inflow and infiltration (RDII). DOU staff expanded and refined the Phase 1 model network, and subdivided the 430 sanitary subcatchments further into 929 subcatchments to distribute the sanitary flows to every node in the expanded network. The boundaries of the subcatchments were adjusted to reflect that CSS sewer flows are typically collected in the alley and back yard mains. The added level of detail more accurately distributed the sewer flows in the model.

[4] Developed by Gary Gulseth, Department of Utilities, City of Sacramento.

This expanded model network with 929 sanitary sub-catchments was provided to AECOM by DOU at the start of the Phase 2 project, and is referred as the starting Phase 2 model. Table 9 lists the three types of sub-catchments in the Phase 2 model, total count and description for each type.

Table 9. Sub-catchment Types

Sub-catchment type "system_type"	Starting Phase 2 Count*	Description
Sanitary	929	Sub-catchments that contribute DWF due to population living or working and industrial/commercial processes in the sub-catchment
Storm	1,457	Sub-catchments that contribute to surface runoff and RDII
Other	169	Sub-catchments that do not contribute to surface runoff (as they belong to a separate stormwater system) but contribute RDII
Total	**2,555**	
*As-of the February 2011 InfoWorks CS Model provided by DOU		

For the final Phase 2 model, additional subcatchment detail was implemented based on parcel landuse data.
Each parcel was routed to the same model node as the starting phase 2 model sub-catchment of which it is a part. Next, the parcels with same landuse type and draining to the same model node were grouped together to develop a single landuse "sanitary" sub-catchment. Due to limitations in the number of nodes allowed by the DOU's InfoWorks license, the team decided not to extend model pipes to capture all new subcatchments. Therefore, there are instances where more than one subcatchment drains to the same node.

Table 10 lists the final Phase 2 model count of "sanitary" sub-catchments by model land use classification. Note that the "storm" and "other" sub-catchments are not sub-divided and remain the same as they were in the starting Phase 2 model. The number of sanitary subcatchments will continue to grow as DOU staff continues to add details to the pipe network in the wet areas.

Table 10. Sanitary Sub-catchments by Landuse Types

Sub-catchment type "system_type"	Landuse for Modeling	Final Phase 2 Count
Sanitary	Residential - Single Family	2,917
	Residential - Multi Family	2,405
	Commercial – High Rise	119
	Commercial – Low Rise	1,394
	Industrial	212
	Other	242
	Not Contributing	1,785
Total Sanitary Sub-catchments		**9,074**

3.5 Groundwater Infiltration Estimates

Groundwater infiltration is rainfall independent groundwater that enters the sewer system through cracks or defective joints in the pipes and manhole walls. The magnitude of groundwater infiltration depends on the condition of the sewers and the depth of groundwater relative to the collection system. Section 3.10 documents the average depth of groundwater in the study area, and inconclusive correlation with the river levels.

Groundwater Infiltration Estimates in Calibrated Model: For the Phase 1 model calibration, groundwater infiltration flow was estimated at 0.88 times the minimum daily flow, based on literature recommendations derived empirically. The groundwater infiltration was distributed evenly amongst parcels upstream of the flow monitors. The Phase 2 calibrated model used the same groundwater infiltration rates as the Phase 1 model. Table 11 lists the 13 flow meters, and the groundwater infiltration rates estimated for the sub-catchments tributary to that flow meter and applied in the Phase 2 model. Note that, in Table 11, three meters have zero groundwater infiltration rate. For these meters, the groundwater infiltration tributary from its upstream meters was higher than that estimated at the meter itself. The groundwater infiltration was therefore set to zero, if not it would have lead to higher modeled groundwater and average dry weather flows at the meter.

Table 11. Rainfall Independent Groundwater Infiltration Rate

Flow Meter	Tributary Area (acres)	Average Dry Weather Flow (mgd)	Average Dry Sanitary Flow (mgd)	Average Groundwater Infiltration Flow (mgd)	Groundwater Infiltration Rate (gpd/acre)
C61-02-01B	678.5	2.32	1.59	0.73	1100
C61-02-02A	46.6	0.12	0.07	0.05	1100
C61-02-03B	552.4	0.52	0.52	0.00	0
C61-02-04A	575.1	0.60	0.60	0.00	0
C61-02-05A	850.9	0.46	0.46	0.00	0
C61-02-06A	250.1	1.62	1.16	0.46	1800
C61-02-07A	711.8	1.53	0.61	0.92	1300
C61-02-08A	391.6	1.08	0.66	0.42	1100
C61-02-09A	718.6	1.25	0.67	0.58	800
C61-02-10A	2579.1	2.26	1.30	0.96	400
C61-02-12B	457.3	0.78	0.34	0.44	1000
C61-02-16ADFM	538.3	0.68	0.27	0.42	800
C61-02-18A	637.8	1.10	0.63	0.47	700
Unmetered area	930.0	1.9	1.3	0.58	600
Total Study Area	**9,920**	**16.2**	**10.2**	**6.0**	**600**
* Tributary area and flows for downstream meters do not include upstream meter area or flows.					

Groundwater Infiltration Estimates in Design Model: The Phase 1 monitoring period was during the wet weather season and may have exhibited higher groundwater infiltration than might have been observed during the dry season. The method used to estimate groundwater infiltration for the Phase 1 model was considered by the team to over-predict the groundwater infiltration. Therefore, groundwater infiltration for the Phase 2 design model was implemented using a previously developed DOU method. DOU's method for estimating groundwater infiltration considers the old age of the collection system, recent monitoring data and data collected from the Sacramento Regional Wastewater Treatment Plant. DOU recommends 300 gpd/acre of groundwater infiltration for the

collection system in the CSS area, provided the groundwater is 15 feet or less in depth. Based on the ground water analysis (Section 3.10), the ground water in the study area was observed to be deeper than the collection system. However, as a conservative estimate, all the “sanitary” sub-catchments in the design model were assigned groundwater infiltration rates of 300 gpd/acre.

3.6 Phase 2 Calibrated Model

Once the initial estimates of DWF parameters were established, a check of the model calibration was performed, and unit flow per ESD (in GPD) values was adjusted to 140 gpd/ESD to match recorded data. The landuse diurnal patterns for Commercial Low Rise and High Rise were also adjusted to allow better matching of peaks for the flow meters that had contributions from these two landuse types.

The total study area DWF for the calibration model was 16.2 mgd made up of rainfall independent groundwater infiltration of 6.0 mgd and 10.2 mgd base sanitary flow. Table 12 lists the 13 flow meters, observed and Phase 2 modeled DWF at each flow meter and the % difference between the two. Appendix D hydrographically presents the observed vs. modeled flow for each meter during the dry weather period.

The remaining DWF calibration discrepancies between the 2009 monitoring data and model results were not further reconciled to reduce difference because that would involve some significant re-allocation of ESDs which would compromise the intended use of a design model. Furthermore, because of the sequential locations of the meters, improvement between observed and modeled at one location could worsen the match at a downstream location.

Table 12. Dry Weather Calibration Results

Flow Meter	Data Period	Observed Flow (MG)	Modeled Dry Weather Flow (MG)	% Difference with Observed Flow
C61-02-01B	1/8/2009-1/18/2009	25.1	14.7	41%
C61-02-02A	1/8/2009-1/18/2009	2.1	1.2	43%
C61-02-03B	1/8/2009-1/18/2009	33.5	23.1	31%
C61-02-04A	1/8/2009-1/18/2009	72.0	70.9	2%
C61-02-05A	1/8/2009-1/18/2009	32.9	44.8	-36%
C61-02-06A	1/8/2009-1/18/2009	12.6	9.0	29%
C61-02-07A	1/8/2009-1/18/2009	16.2	16.3	-1%
C61-02-08A	1/8/2009-1/18/2009	58.7	64.8	-10%
C61-02-09A	1/8/2009-1/18/2009	13.2	13.6	-3%
C61-02-10A	1/8/2009-1/18/2009	36.1	39.7	-10%
C61-02-12B	1/8/2009-1/18/2009	8.9	9.6	-8%
C61-02-16ADFM	1/8/2009-1/18/2009	50.7	62.4	-23%
C61-02-18A	1/8/2009-1/18/2009	13.0	11.7	10%
Sump 2	1/8/2009-1/18/2009	160.7	170.1	-6%

3.7 Phase 2 Design Model

The ESDs and landuse diurnal patterns in the design model were kept the same as in the calibrated model; however the unit flow per ESD in the design model was set to 310 gpd/ESD and the groundwater infiltration was set to 300 gpd/acre. The total DWF in the design model meets DOU's design goal of 24.2 mgd. Table 13 lists the baseflow and ESD estimates for the CSS study area for design model.

Table 13. Design Model Input

CSS Area	Tributary Area (Acres)	Groundwater Infiltration (MGD)	ESDs	Average Sanitary Flow = ESDs * Unit Flow (MGD)	Average Dry Weather Flow (MGD)
Total	7,720.0	2.3	70,650	21.9	24.2

3.8 Modification to Wet Weather

This TM addresses only the DWF component of model flow generation. Stormwater runoff in the CSS and RDII in the separate sanitary portions will be represented using wet weather flow generation parameters assigned to subcatchments developed as part of the above DWF procedure. If the model is extended to include additional pipes draining the additional Phase 2 subcatchments, the wet weather parameters will not need to be subdivided. However, assigning the wet weather parameters to additional subcatchments will require verification of the wet-weather model response. At such time, the results of the wet-weather verification must be reviewed with DOU to discuss any additional effort required for refining wet-weather calibration.

3.9 Projecting Future Flows

DOU's Interim Water Conservation Plan[5] provides projections for growth of water connections in the City. Figure 2 presents projected number of water connections for Single Family, Multi Family, Commercial, Industrial and Other landuse types through the year 2030.

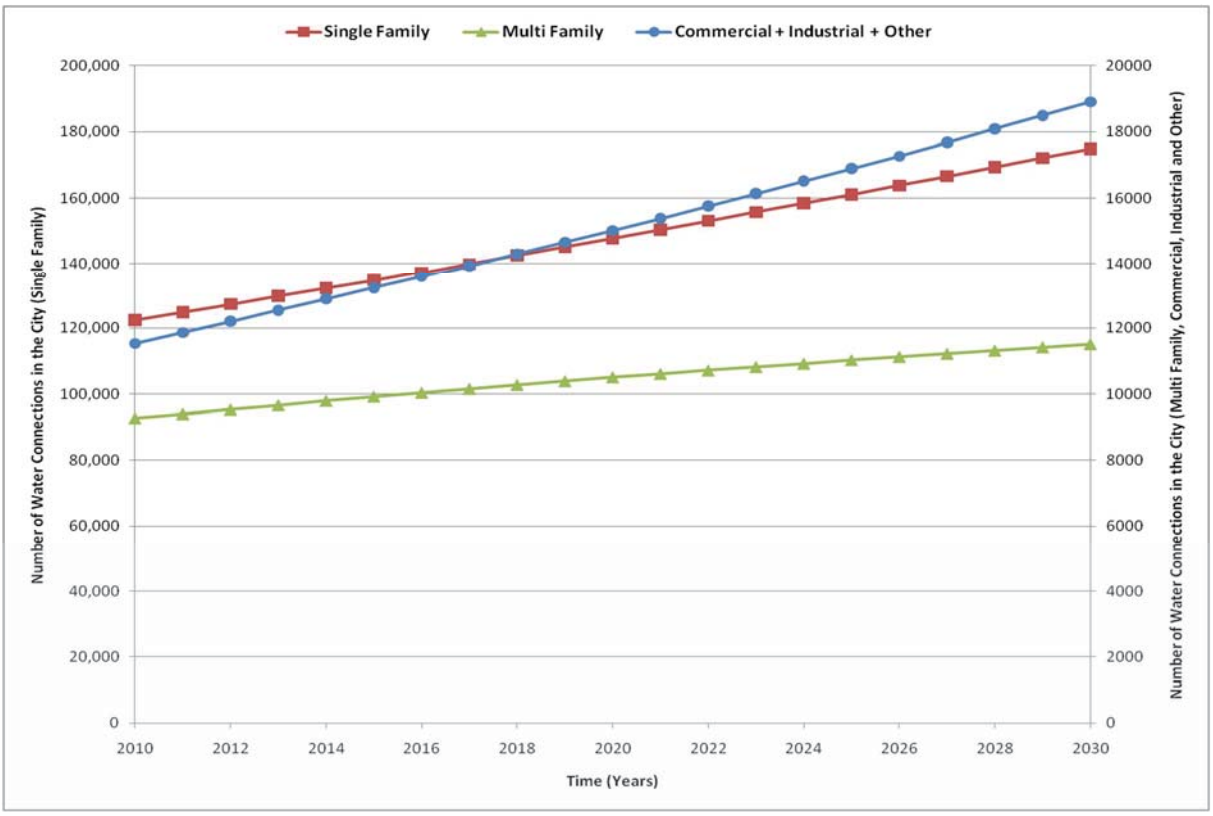


Figure 2. Projected Water Connections in the Study Area

[5] City of Sacramento Interim Water Conservation Plan, October 2009 (Revised January 2010).

Table 14 calculates the projected growth of number of water connections in 2030, as compared to 2010 (in percentage) for the different landuse types. If the same projected growth rate is applied to the number of ESDs in the study area for the current design model (2010), an ESD estimate for 2030 planning model can be obtained. Table 14 applies these growth rates, and estimates the total ESDs for the 2030 planning model.

Table 14. 2030 ESD Estimates

Landuse Type	Number of Water Connections in 2010	Number of Water Connections in 2030	Projected Growth in Number of Water Connections from 2010 to 2030 (%)	ESDs in 2010	ESDs in 2030
Residential – Single Family	122,368	174,885	43%	21,881	31,290
Residential – Multi Family	9,273	11,510	24%	16,593	20,576
Commercial + Industrial + Other	11,858	19,309	63%	32,176	52,446
Total	-	-	-	**70,650**	**104,312**

The ESD estimates in the 2030 planning model will be increased according to Table 14, while sanitary flow per ESD will be unchanged at 310 gpd/ESD. Different 2030 planning scenarios can be modeled by changing the sanitary flow per ESD assumption.

3.10 Groundwater Analysis

Historical ground water data was available from the Integrated Water Resources Information System (IWRIS) maintained by State of California's Department of Water Resources. Data for nine ground water wells within the CSS study area was collected and analyzed. Figure 3 presents the location of these wells in the study area.

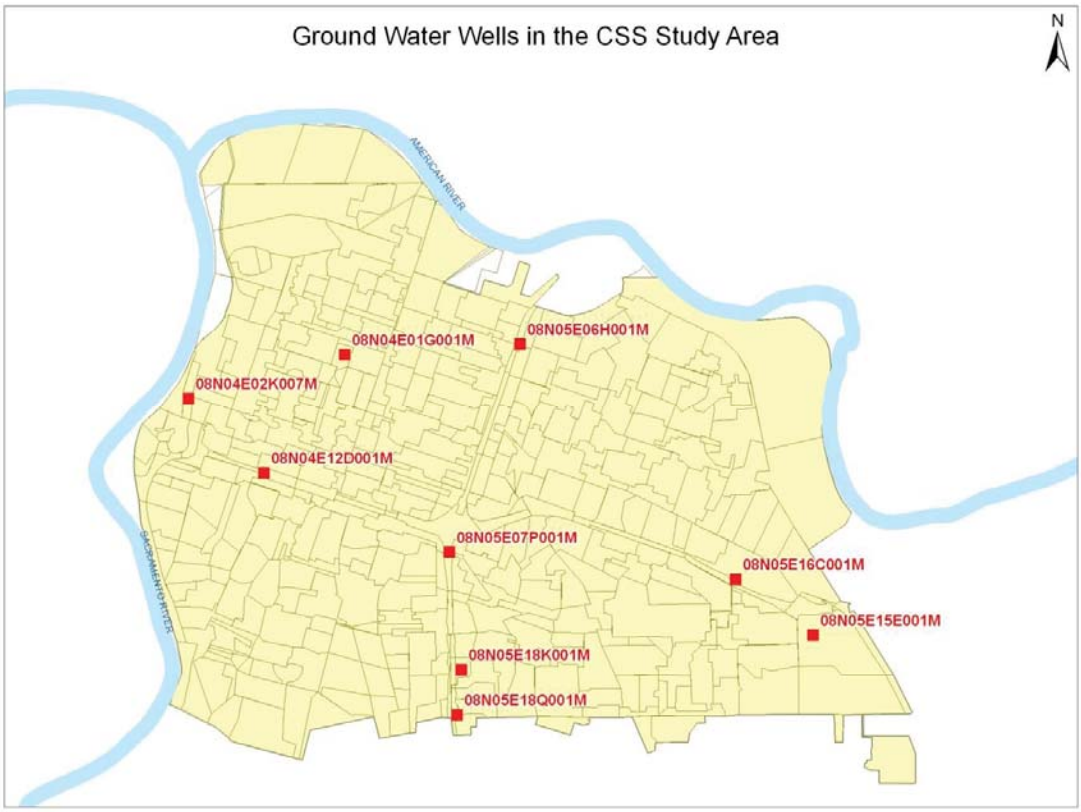


Figure 3:

Ground Water Wells in Study Area

These wells record and report water surface elevation and water depth from ground surface for a historical period of up to 40+ years. Appendix E presents the depth of water from ground surface for each of these nine wells. The ground surface elevation used by these wells is within +/- 1 foot difference from the ground surface elevation of sewer pipes in the Phase 2 model. Table 15 presents an average depth of ground water for these nine wells, and the depth of the pipe invert from ground surface in the vicinity of the well. From the table, it can be summarized that most of the pipes in the study area are above the ground water table. It can be concluded that ground water infiltration into the collection system is not significant, especially during non-rainfall periods.

Table 15. Average Water Depth in Study Area

GW Well	Average depth of water from ground surface[1] (ft)	Depth of pipe from ground surface[2] (ft)
08N04E02K007M	16.2	9.5
08N04E01G001M	19.5	11.2
08N05E06H001M	21.2	13.7
08N04E12D001M	21.3	8.8
08N05E16C001M	42.9	16.7
08N05E18K001M	27.2	11.7
08N05E07P001M	25.9	18.1
08N05E18Q001M	30.4	9.1
08N05E15E001M	45.8	11.7

[1] Ground surface elevation is surveyed or interpolated from USGS topographic data.
[2] Depth of pipes is estimated from Phase 2 model, and the ground surface elevation of the Phase 2 model pipes is within +/- 1 feet difference of the ground water wells.

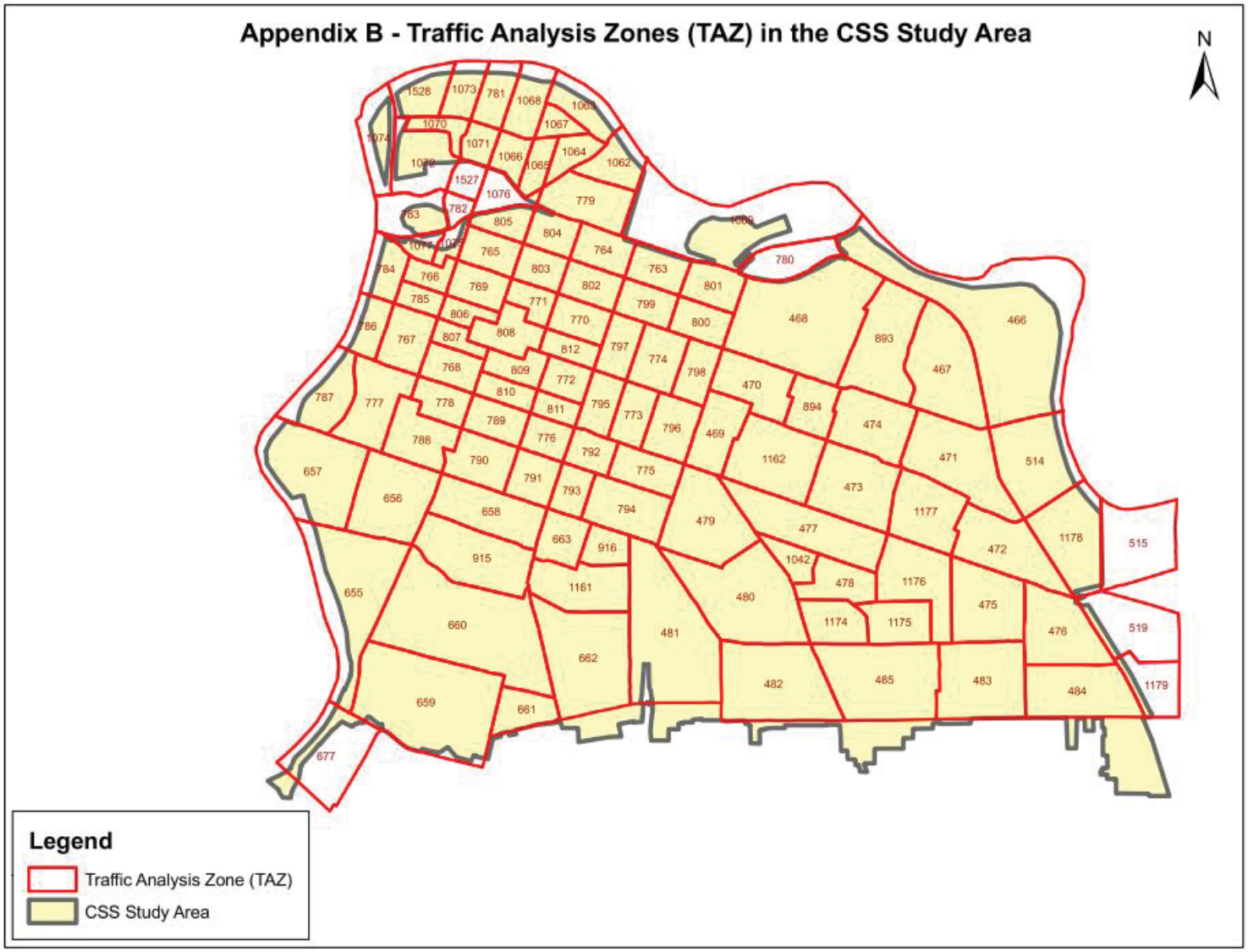
The hourly river stage level data for Sacramento and American river was also available on the IWRIS website. The river stage levels were compared in conjunction with the ground water levels for an extended time period; the comparison did not point to any correlation between the river levels and ground water depth. The analysis could not conclude if the change in river levels had any impact on the ground water depth.

Appendix A – ESD Estimates using Water Billing Data

Appendix A - Population Estimates using Water Billing Data								
#	ACCOUNT NO	Assessor Parcel Number (APN)	Landuse for Modeling	Equivalent ESDs using Method 1	Average Monthly Water Use	Average Daily Water Use	Average Sanitary Flow	Equivalent ESDs using Method 2
					(Cubic Feet)	(GPD)	(GPD)	
1	0006526261	602360170000	Residential - Single Family	1.0	2688.0	670.2	603.2	1.9
2	0008734000	00901330040000	Commercial - Low Rise	1.3	2396.7	597.6	537.8	1.7
3	0009560417	00201650080000	Residential - Multi Family	2.3	1485.3	370.3	333.3	1.1
4	0009734000	00201320020000	Residential - Multi Family	2.3	282.3	70.4	63.4	0.2
5	0010834000	00301250230000	Residential - Single Family	1.0	363.7	90.7	81.6	0.3
6	0013734000	00201640110000	Commercial - Low Rise	0.9	393.3	98.1	88.3	0.3
7	0017644000	01101830150000	Commercial - High Rise	4.6	483.3	120.5	108.5	0.3
8	0023144000	00902940080000	Commercial - Low Rise	0.8	41973.3	10465.4	9418.8	30.4
9	0024734000	00701020270000	Commercial - Low Rise	0.9	483.3	120.5	108.5	0.3
10	0025144000	00601750180000	Residential - Multi Family	15.0	6543.3	1631.5	1468.3	4.7
11	0029734000	00901510100000	Commercial - Low Rise	0.4	557.7	139.0	125.1	0.4
12	0031882551	01502410210000	Residential - Single Family	1.0	1690.0	421.4	379.2	1.2
13	0032834000	00300950280000	Industrial	8.7	3118.3	777.5	699.8	2.3
14	0037734000	01001660380000	Commercial - Low Rise	0.9	876.7	218.6	196.7	0.6
15	0040834000	01200530070000	Residential - Single Family	1.0	333.3	83.1	74.8	0.2
16	0044734000	00701060010000	Commercial - Low Rise	18.2	15866.7	3956.1	3560.5	11.5
17	0044863938	01301620010000	Commercial - Low Rise	0.7	2182.0	544.0	489.6	1.6
18	0046452712	00201160200000	Residential - Multi Family	3.0	970.0	241.9	217.7	0.7
19	0052844000	01000620200000	Commercial - High Rise	3.5	2200.0	548.5	493.7	1.6
20	0053284442	01000740070000	Residential - Single Family	1.0	800.7	199.6	179.7	0.6
21	0053734000	00700150110000	Residential - Multi Family	3.0	1586.7	395.6	356.0	1.1
22	0054844000	00201750110000	Residential - Multi Family	1.5	409.7	102.1	91.9	0.3
23	0061734000	00200860280000	Other	1.7	1658.7	413.6	372.2	1.2
24	0062734000	00201260180000	Residential - Multi Family	4.5	74647.0	18612.0	16750.8	54.0
25	0064734000	00602310080000	Commercial - Low Rise	0.9	4283.3	1068.0	961.2	3.1

26	0068208215	00900820020000	Residential - Single Family	1.0	791.3	197.3	177.6	0.6
27	0069501435	01003730020000	Commercial - Low Rise	0.8	827.0	206.2	185.6	0.6
28	0078734000	00602240260000	Commercial - Low Rise	0.8	716.7	178.7	160.8	0.5
29	0083734000	00700910010000	Other	3.5	13546.7	3377.6	3039.9	9.8
30	0087734000	00201050020000	Residential - Multi Family	1.5	1573.3	392.3	353.1	1.1
31	0091908559	01402110030000	Residential - Single Family	1.0	3626.0	904.1	813.7	2.6
32	0092734000	00301720090000	Residential - Multi Family	2.3	184.0	45.9	41.3	0.1
33	0093144000	01002150100000	Commercial - Low Rise	3.4	10833.3	2701.1	2431.0	7.8
34	0095734000	00900640080000	Other	2.2	157.7	39.3	35.4	0.1
35	0105844000	00700940260000	Residential - Single Family	1.0	605.3	150.9	135.8	0.4
36	0106734000	00900960210000	Commercial - Low Rise	0.4	330.0	82.3	74.1	0.2
37	0107844000	00703800300000	Residential - Single Family	1.0	475.3	118.5	106.7	0.3
38	0109076743	00701440100000	Residential - Single Family	1.0	3378.7	842.4	758.2	2.4
39	0109647185	00602840350000	Residential - Single Family	1.0	423.3	105.6	95.0	0.3
40	0109734000	00201650120000	Residential - Multi Family	1.5	567.7	141.5	127.4	0.4
41	0110944000	01301530270000	Residential - Single Family	1.0	770.0	192.0	172.8	0.6
42	0112834000	00300950290000	Industrial	2.2	97.7	24.4	21.9	0.1
43	0113734000	00201740160000	Residential - Multi Family	24.0	14773.3	3683.5	3315.1	10.7
44	0114734000	00601010170000	Commercial - Low Rise	0.4	423.3	105.6	95.0	0.3
45	0117644000	01401230320000	Commercial - Low Rise	4.2	15066.7	3756.6	3381.0	10.9
46	0117844000	00703800210000	Residential - Single Family	1.0	525.7	131.1	118.0	0.4
47	0120834000	00301250100000	Residential - Single Family	1.0	461.0	114.9	103.4	0.3
48	0123934000	00902860090000	Industrial	3.7	8033.3	2003.0	1802.7	5.8
49	0125144000	00601760050000	Commercial - Low Rise	0.4	595.0	148.4	133.5	0.4
50	0127431586	00301250200000	Residential - Multi Family	6.0	2196.7	547.7	492.9	1.6

Appendix B – ESD Estimates using Traffic Analysis Zones data

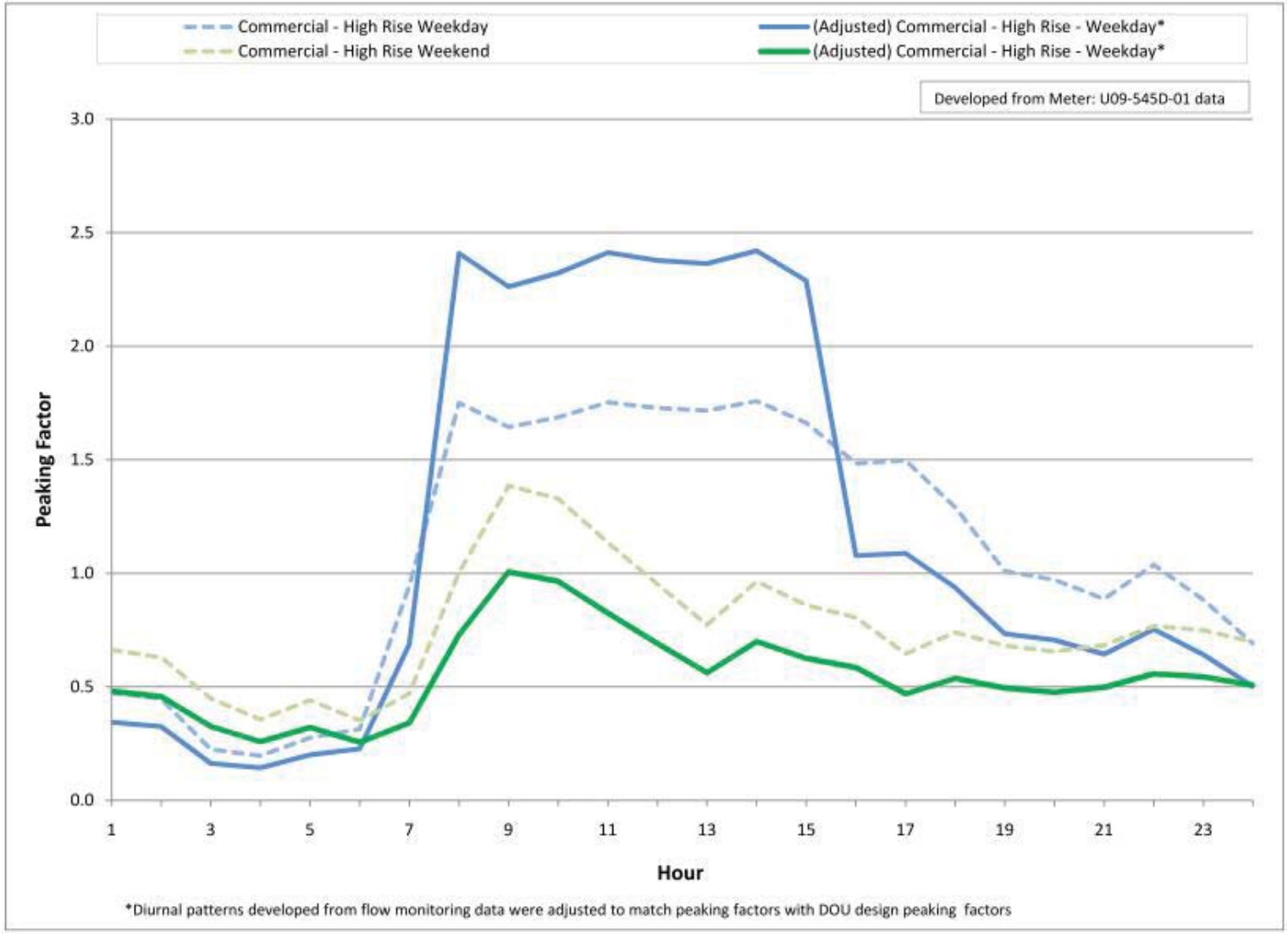


Appendix B – Population Estimates using Traffic Analysis Zones data						
	2005			2013		
TAZ	Total Single Family Dwelling Units	Total Multi Family Dwelling Units	Employment (Retail, Office, Industrial, Public, Other)	Total Single Family Dwelling Units	Total Multi Family Dwelling Units	Employment (Retail, Office, Industrial, Public, Other)
1073	0	0	1217	0	0	1217
781	0	0	1219	0	0	1219
1068	0	0	4326	0	0	4326
1528	0	0	1599	0	0	1599
1063	0	0	568	0	0	568
1074	0	0	190	0	0	190
1067	0	0	886	0	0	886
1070	0	0	495	0	0	495
1072	25	53	449	25	53	493
1071	0	0	659	0	0	659
1062	16	11	415	16	11	415
1066	0	0	1462	0	0	1462
1064	74	223	276	74	223	276
1065	0	0	480	0	0	480
1069	1	0	202	1	0	201
1527	0	0	17	0	0	17
1076	0	0	18	0	0	18
779	17	40	1363	17	40	1363
783	0	0	0	0	0	0
782	0	0	0	0	0	0
805	20	199	1164	20	221	1188
804	27	278	698	28	289	704
466	1455	223	252	1455	223	252
1075	0	0	1440	0	0	1440
764	33	89	651	33	89	651
765	7	351	3312	7	354	3326
1077	0	0	14	0	0	14
784	0	33	2191	0	44	2237
780	0	0	1	0	0	1
763	112	355	175	112	355	175
803	43	557	949	43	557	949
766	2	250	4173	2	250	4173
468	1134	441	1691	1134	441	1691
802	38	725	272	38	725	272
769	0	30	16304	0	78	16627
801	57	287	539	57	287	539
799	101	770	67	101	770	67
893	796	185	102	796	185	102
771	0	27	6107	0	35	6288
785	0	9	4226	0	146	7395
786	0	173	293	10	234	345

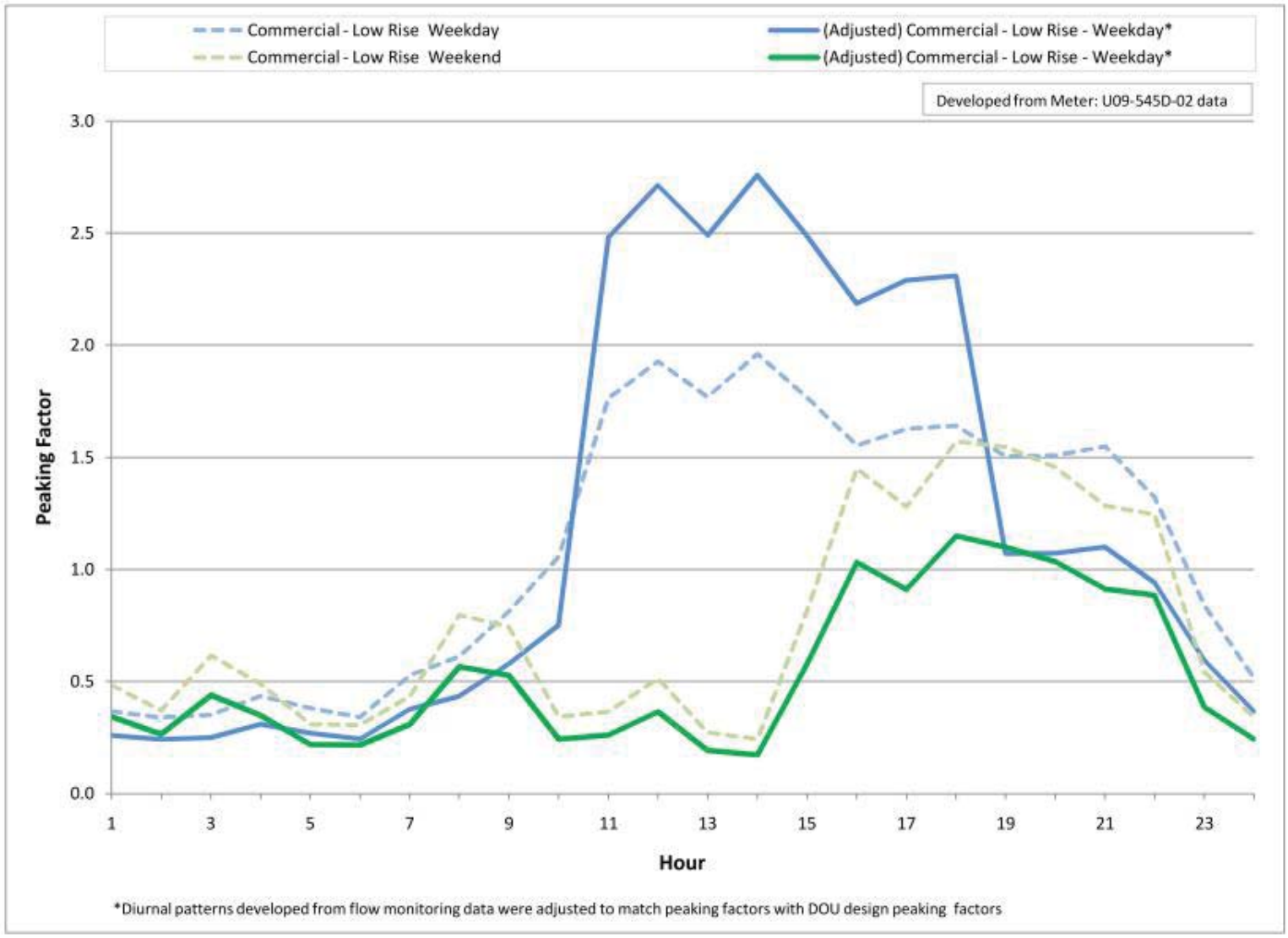
770	5	356	2081	5	379	2089
806	0	136	9626	0	162	10214
800	61	430	377	61	430	377
467	673	157	3119	673	157	3119
808	0	2	5251	0	47	5821
767	0	1070	11442	9	1124	11545
797	20	643	2122	20	643	2122
807	1	3	7546	1	9	8020
774	35	845	1786	35	860	1789
812	5	324	3414	5	324	3414
798	14	81	3592	14	81	3592
768	3	136	14672	34	391	14814
787	0	0	287	0	0	287
809	0	955	3593	0	955	3593
470	242	355	796	242	355	796
777	56	442	2978	107	796	3476
772	15	776	970	15	776	970
795	45	728	892	46	764	864
810	27	366	465	62	570	592
894	184	142	1935	184	142	1935
778	37	285	947	70	575	1020
773	72	814	223	73	842	281
474	478	180	228	478	180	228
469	143	180	3565	143	180	3565
811	11	131	290	49	415	585
796	22	365	3296	22	373	3366
789	52	310	2824	99	603	2965
788	50	287	552	52	301	555
471	539	184	949	539	184	953
514	5	30	2240	5	30	2240
1162	594	249	211	594	249	211
776	29	270	924	76	561	1187
657	112	334	1627	112	334	1643
792	56	329	473	69	453	543
790	58	375	1098	58	375	1098
473	425	157	564	425	157	564
775	58	485	1278	60	508	1302
791	43	211	855	43	211	855
656	308	370	925	308	370	939
793	57	235	667	57	235	667
479	261	502	2057	261	502	2057
1177	294	159	1153	294	159	1153
658	360	347	646	360	347	646
794	54	381	609	54	381	609
477	410	217	403	410	217	403
1178	0	32	358	12	223	372
515	188	565	756	195	670	719

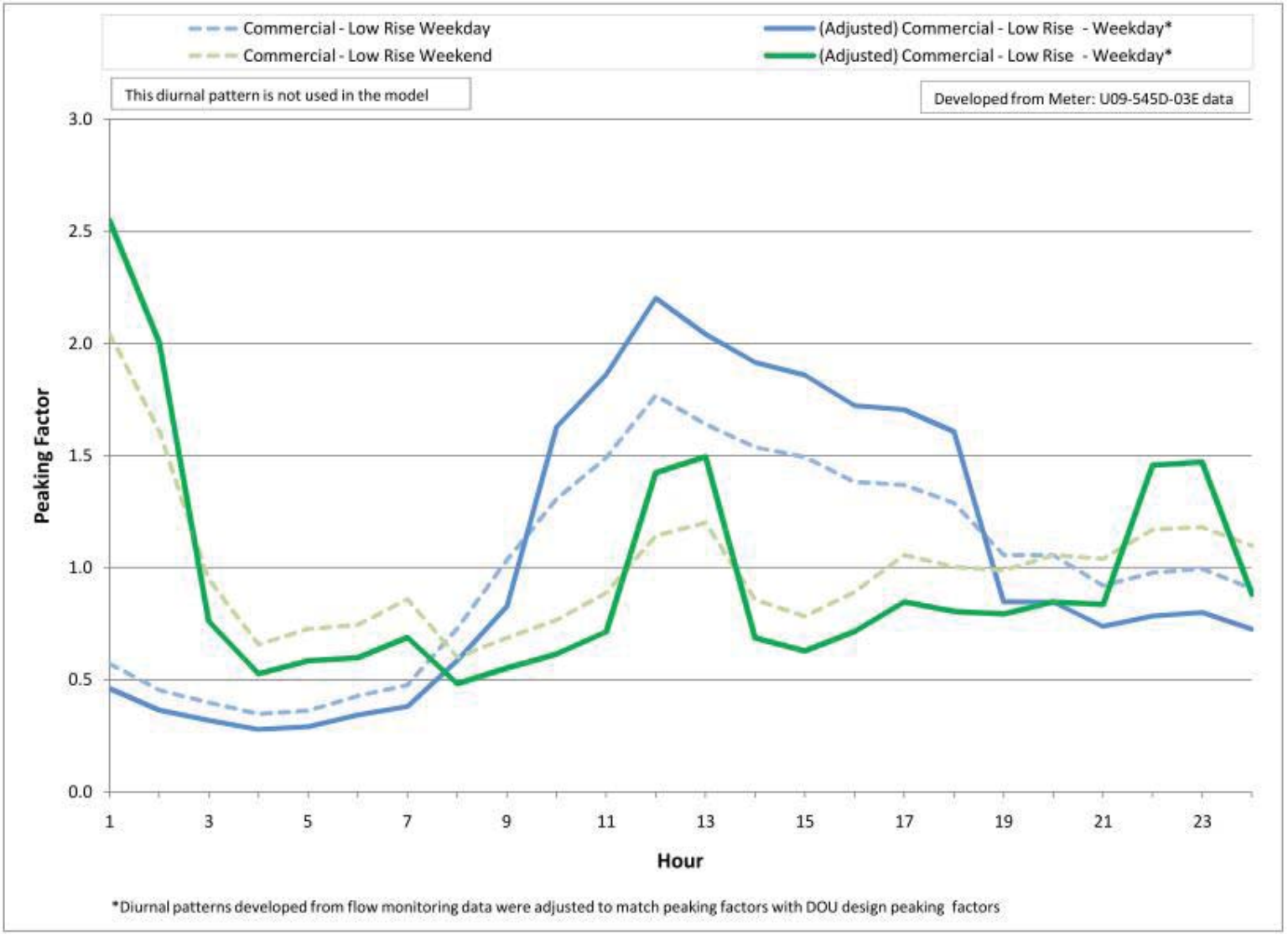
472	142	79	2127	167	495	2053
663	41	89	792	41	89	792
915	687	271	142	687	271	142
655	622	267	235	622	267	235
916	70	209	4463	70	209	4463
1042	0	0	6082	0	0	6082
1176	464	168	63	464	168	63
481	688	776	1691	690	784	1693
480	634	791	1321	634	791	1321
660	1365	428	702	1365	428	702
478	35	13	2195	35	13	2196
475	373	132	3049	373	145	3057
1161	449	308	38	449	308	38
476	58	72	1414	76	116	1297
519	3	58	2066	3	58	2248
662	507	253	79	878	370	267
1174	0	0	2337	0	0	2337
1175	106	257	552	139	290	552
482	714	469	358	714	469	358
659	345	19	208	345	19	208
483	605	184	70	605	184	70
485	858	456	642	858	456	642
1179	4	1	851	4	1	1216
484	191	58	1283	191	58	1707
661	0	0	2132	0	0	1726
677	417	36	0	417	36	0
Total	**19439**	**26631**	**202019**	**20228**	**30238**	**209944**

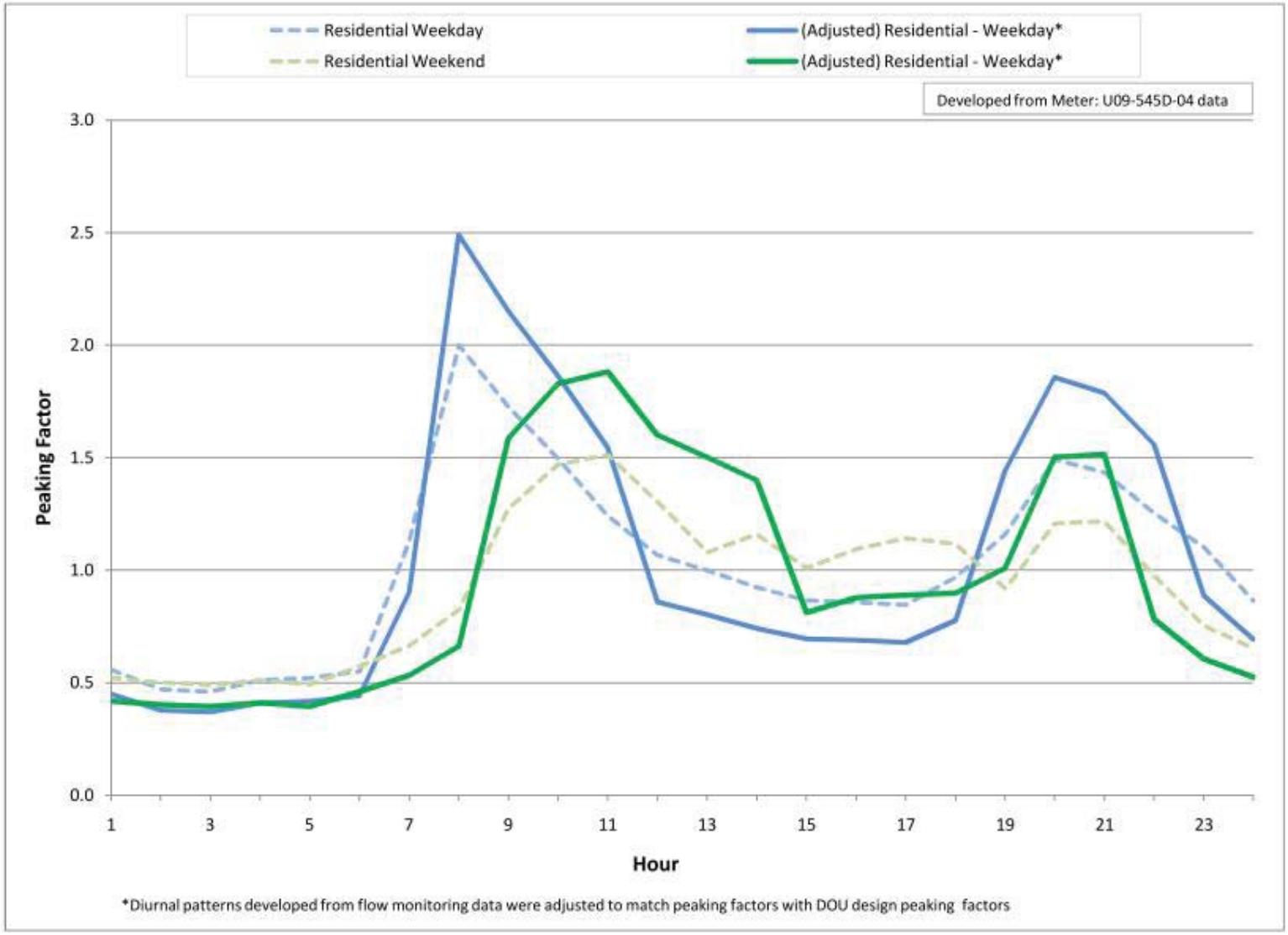
Appendix C – Sanitary Flow Diurnals for different Landuses

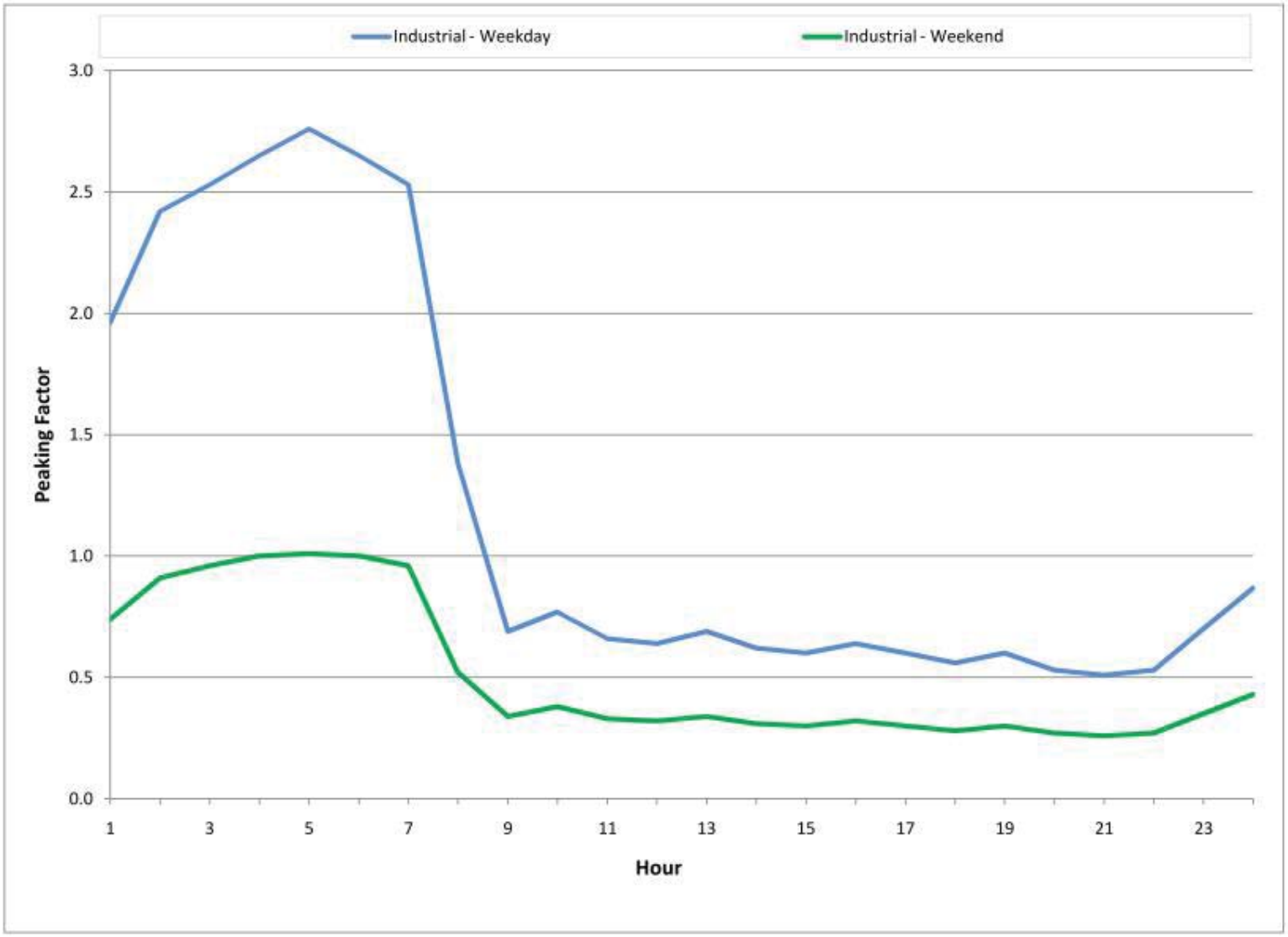


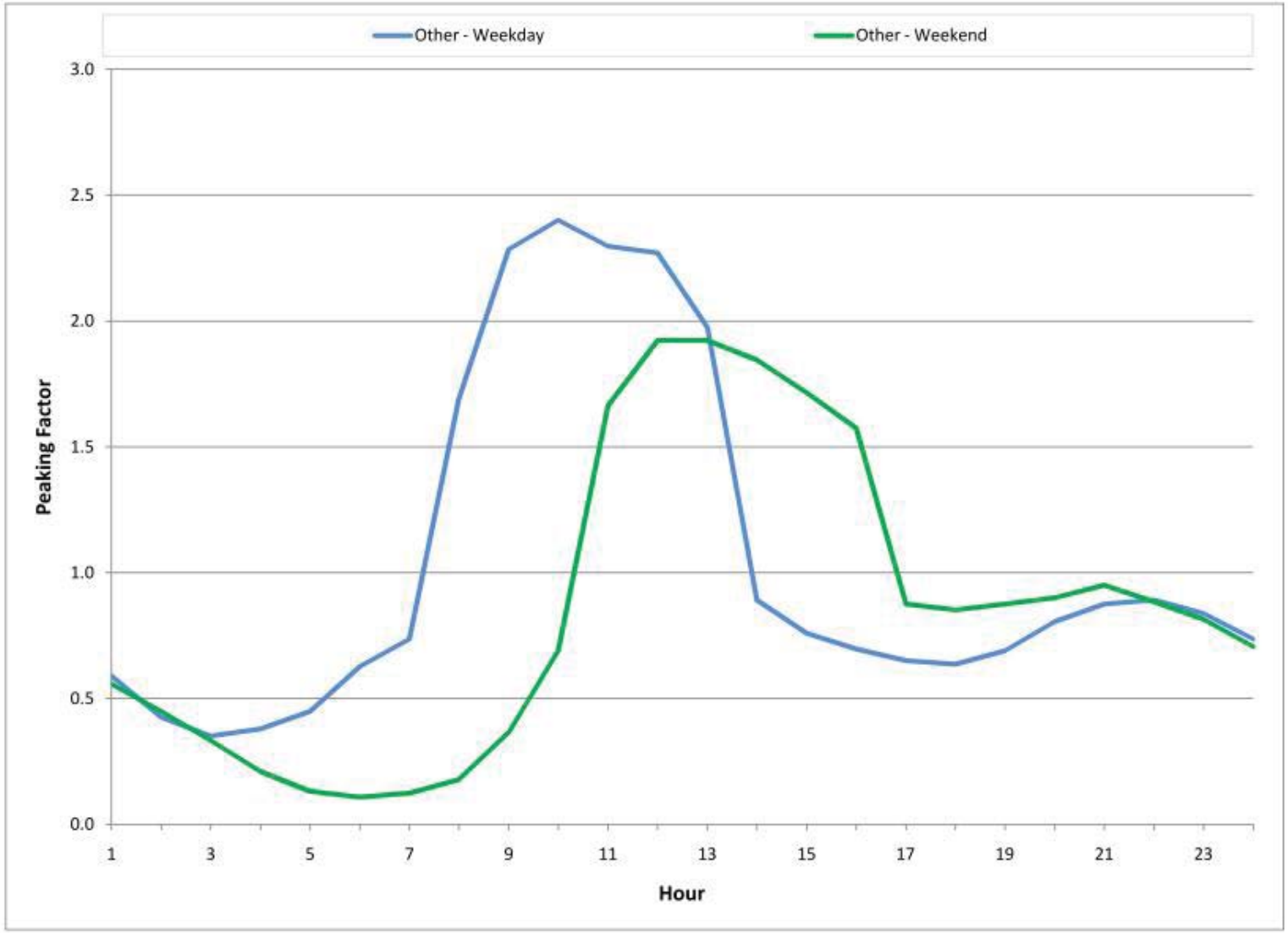
*Diurnal patterns developed from flow monitoring data were adjusted to match peaking factors with DOU design peaking factors


Commercial - Low Rise Weekday
Commercial - Low Rise Weekend
(Adjusted) Commercial - Low Rise - Weekday*
(Adjusted) Commercial - Low Rise - Weekday*
Developed from Meter: U09-545D-02 data
Peaking Factor
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1
3
5
7
9
11
13
15
17
19
21
23
Hour
*Diurnal patterns developed from flow monitoring data were adjusted to match peaking factors with DOU design peaking factors


Commercial - Low Rise Weekday
Commercial - Low Rise Weekend
(Adjusted) Commercial - Low Rise - Weekday*
(Adjusted) Commercial - Low Rise - Weekday*
This diurnal pattern is not used in the model
Developed from Meter: U09-545D-03E data
Peaking Factor
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1
3
5
7
9
11
13
15
17
19
21
23
Hour
*Diurnal patterns developed from flow monitoring data were adjusted to match peaking factors with DOU design peaking factors


Residential Weekday
Residential Weekend
(Adjusted) Residential - Weekday*
(Adjusted) Residential - Weekday*
Developed from Meter: U09-545D-04 data
Peaking Factor
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1
3
5
7
9
11
13
15
17
19
21
23
Hour
*Diurnal patterns developed from flow monitoring data were adjusted to match peaking factors with DOU design peaking factors

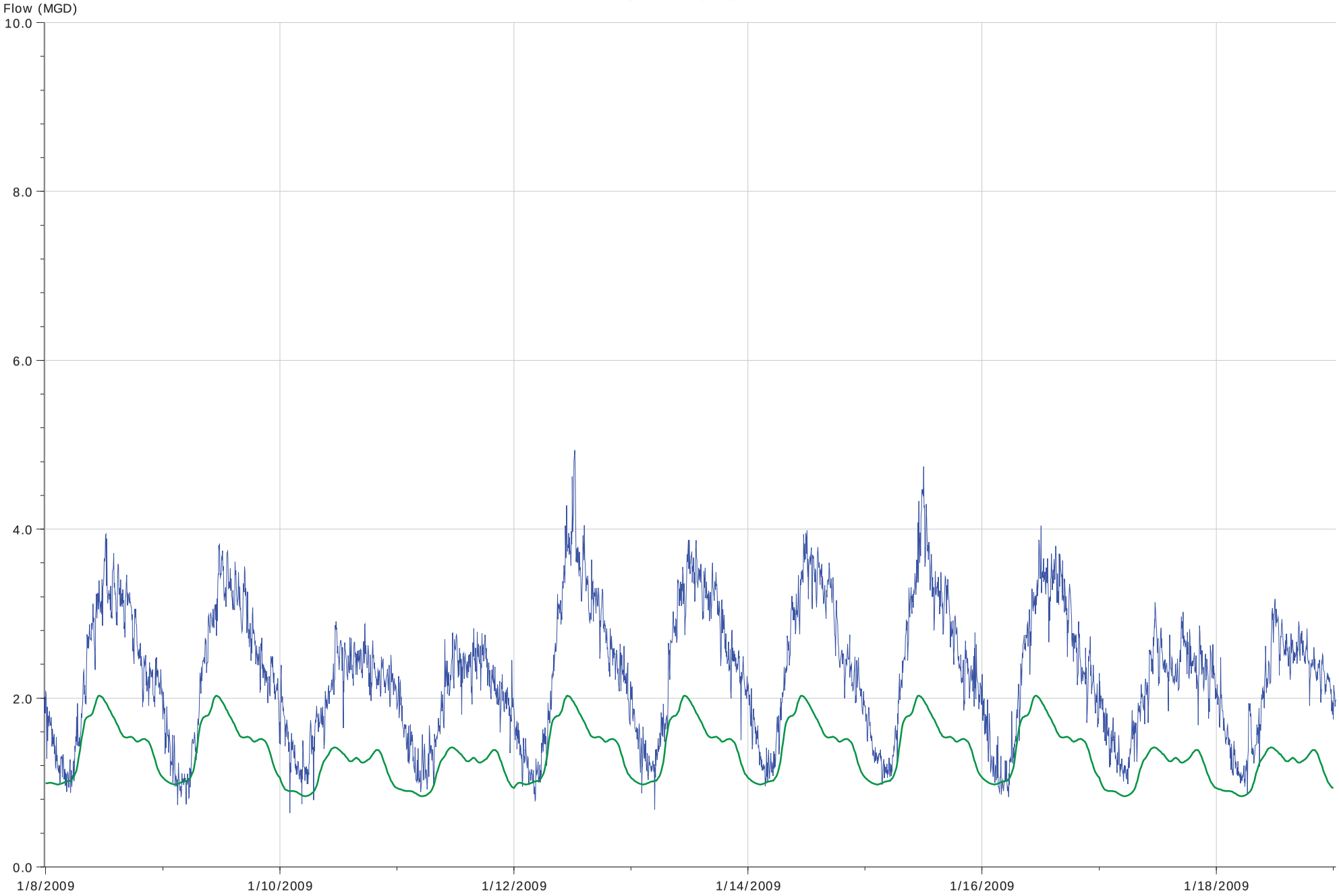

Industrial - Weekday
Industrial - Weekend
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Peaking Factor
1
3
5
7
9
11
13
15
17
19
21
23
Hour


Other - Weekday
Other - Weekend
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Peaking Factor
1
3
5
7
9
11
13
15
17
19
21
23
Hour

Appendix D – Phase 2 Calibrated Model Result Comparison with Observed Flow

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 1 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

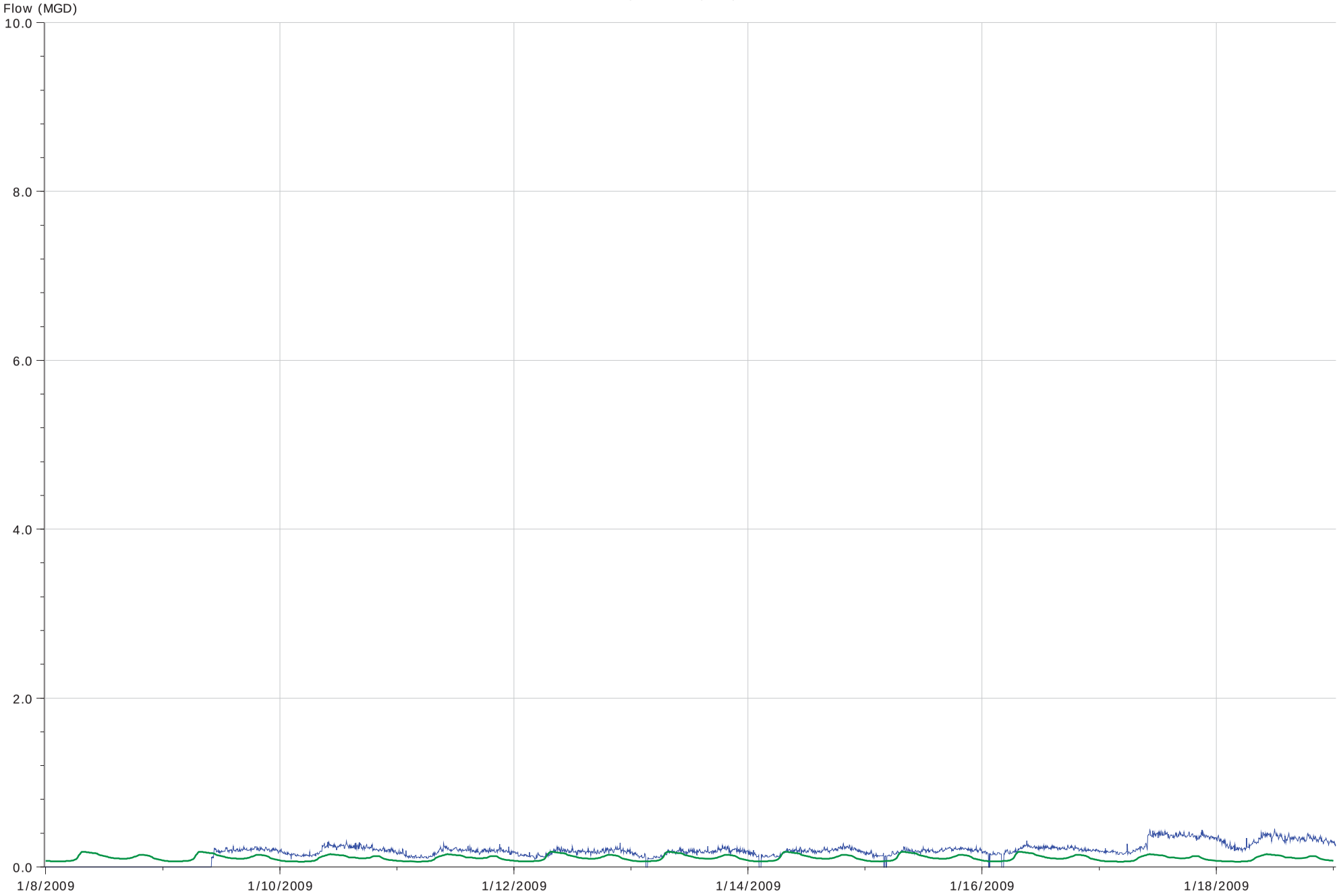
Flow Survey Location (Obs.) 1 212EE13.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	0.639	4.936	25.092
Modeled	0.838	2.028	14.727

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 2 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

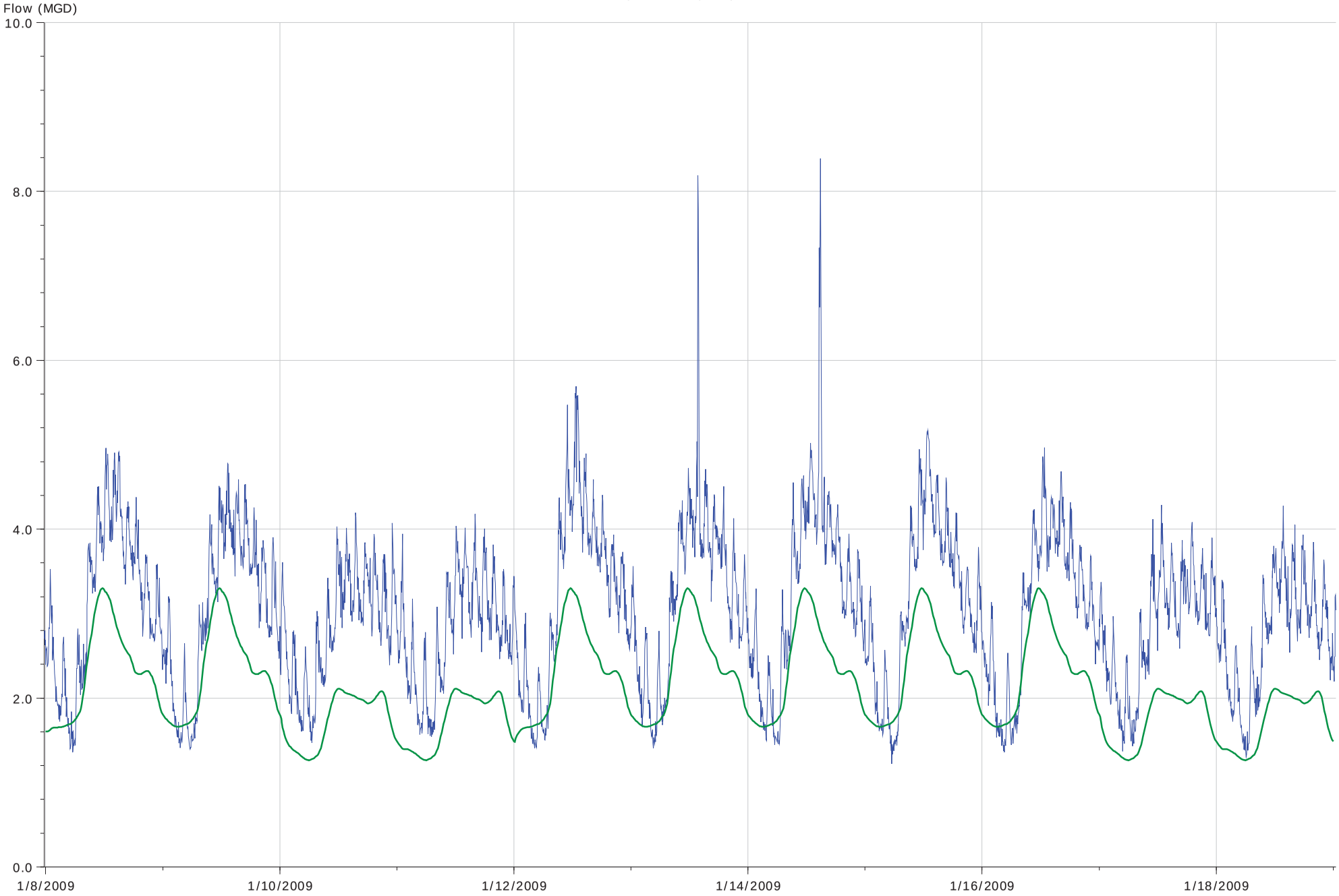
Flow Survey Location (Obs.) 2 916EE14.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	-0.156	0.449	2.010
Modeled	0.062	0.179	1.189

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 3 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

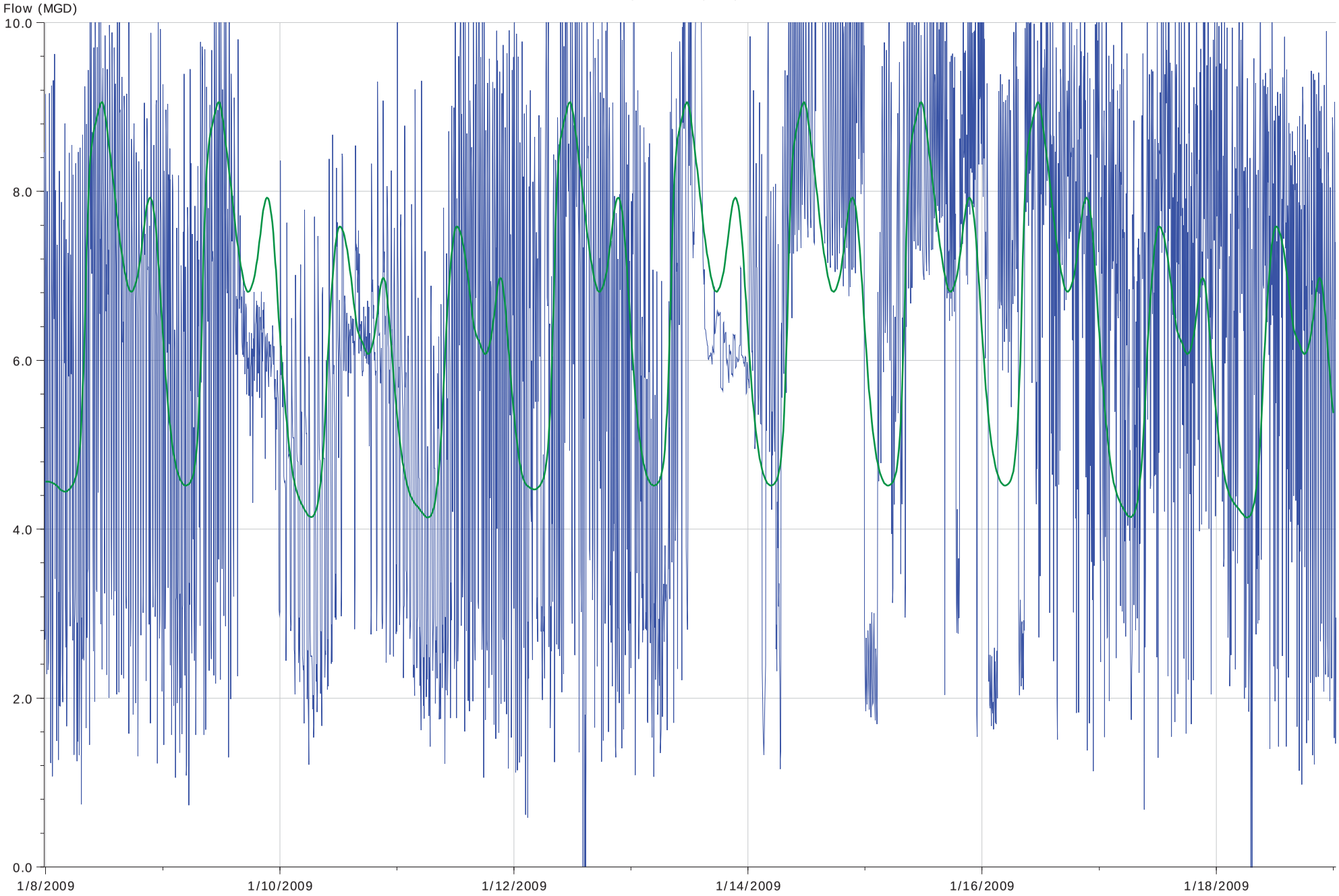
Flow Survey Location (Obs.) 3 104HH13.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	1.220	8.389	33.488
Modeled	1.265	3.299	23.150

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 4 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

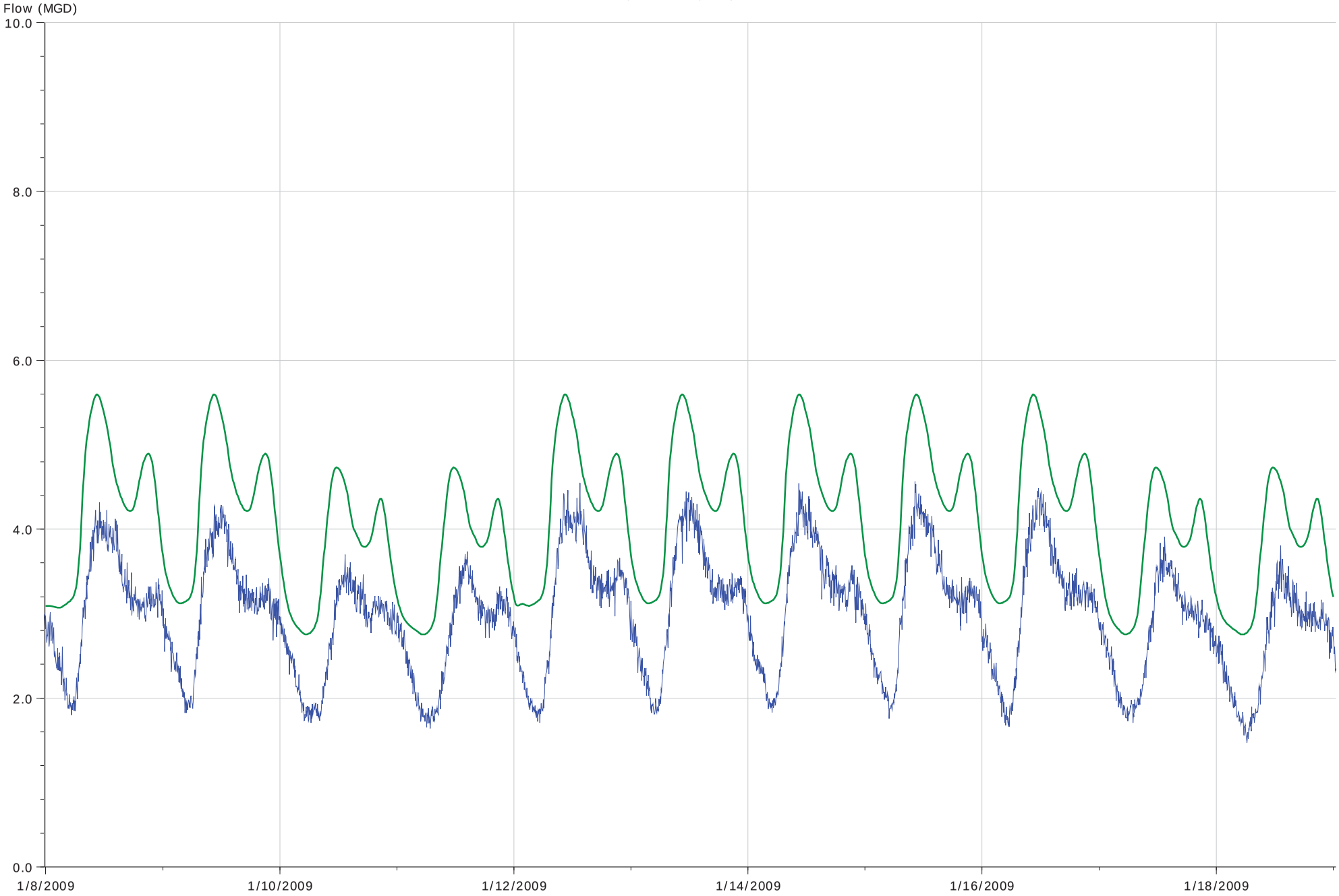
Flow Survey Location (Obs.) 4 503I113.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	-1.270	26.626	71.862
Modeled	4.139	9.054	70.945

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 5 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

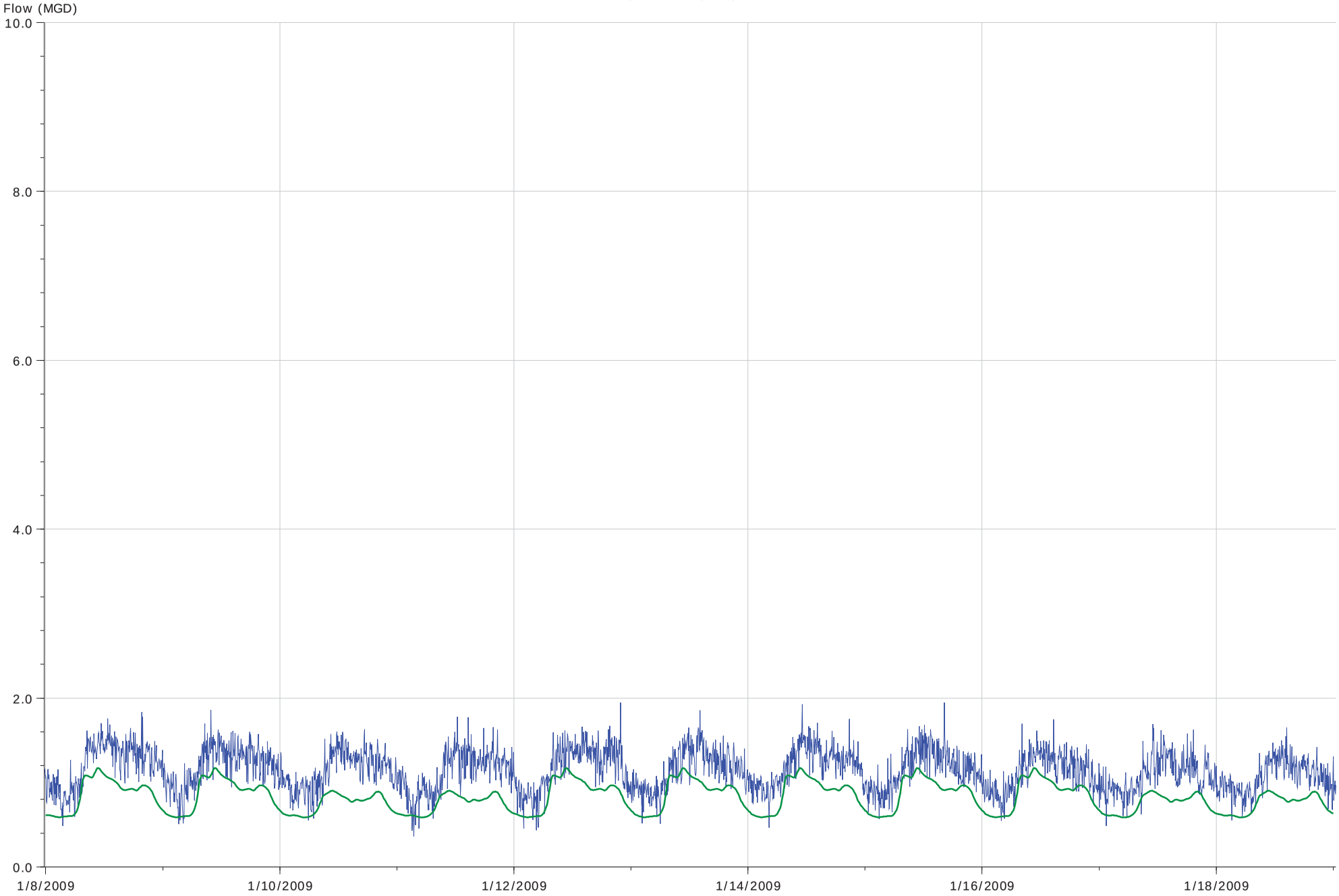
Flow Survey Location (Obs.) 5 819DD15.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	1.474	4.561	32.874
Modeled	2.752	5.598	44.847

Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

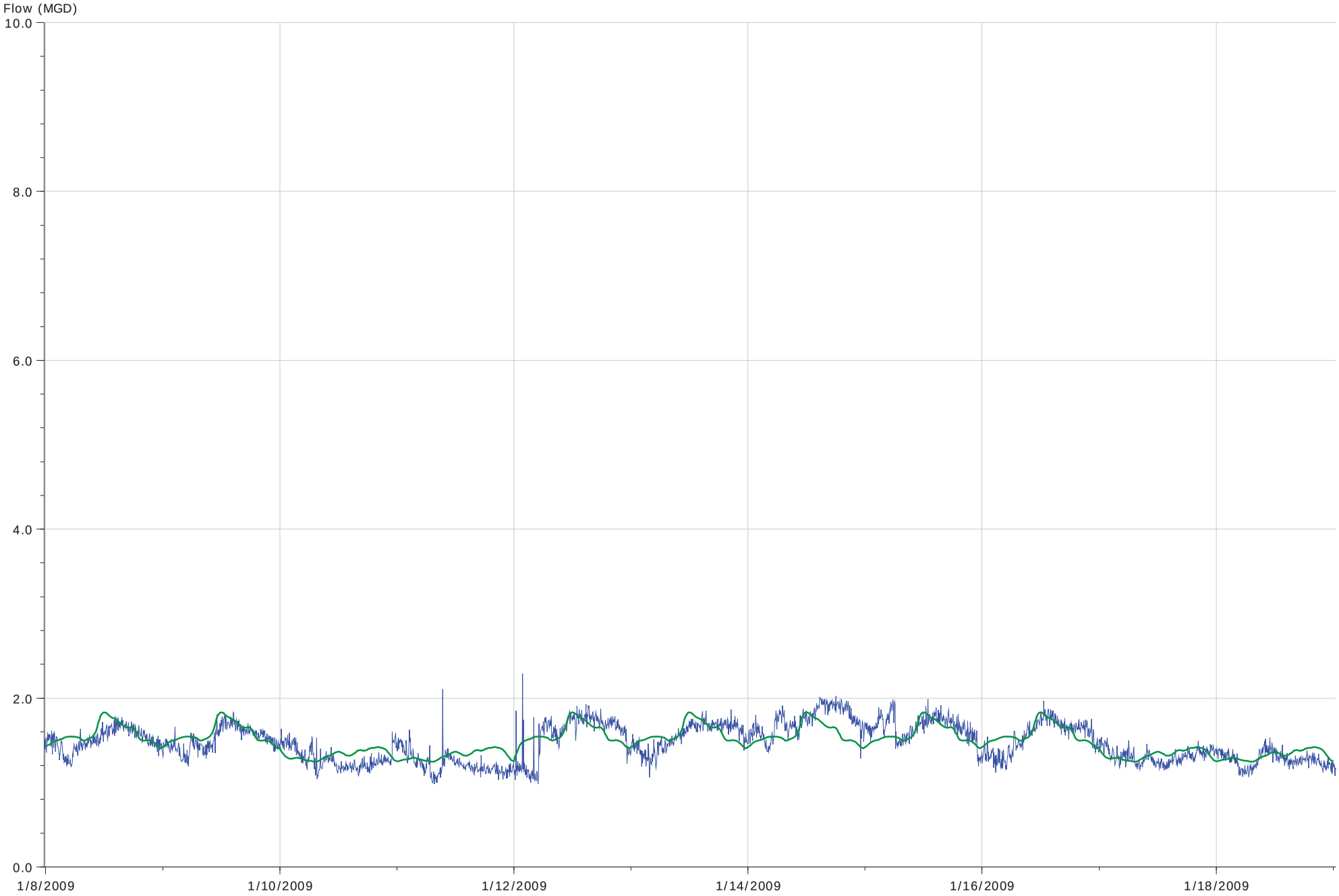
Flow Survey Location (Obs.) 6 308EE15.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	0.365	1.947	12.641
Modeled	0.587	1.172	9.063

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 7 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

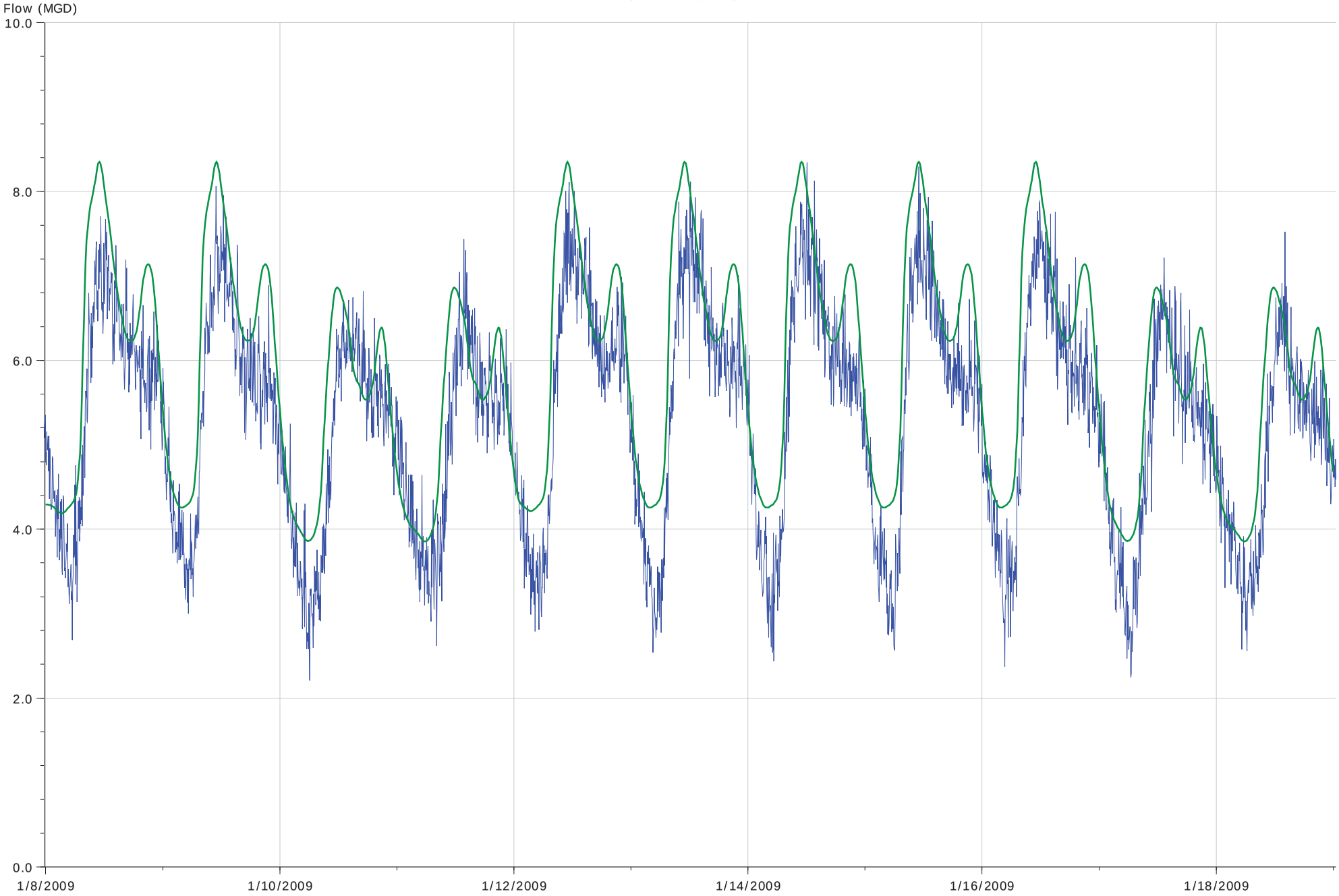
Flow Survey Location (Obs.) 7 405BB16.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	0.981	2.289	16.204
Modeled	1.244	1.831	16.347

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 8 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

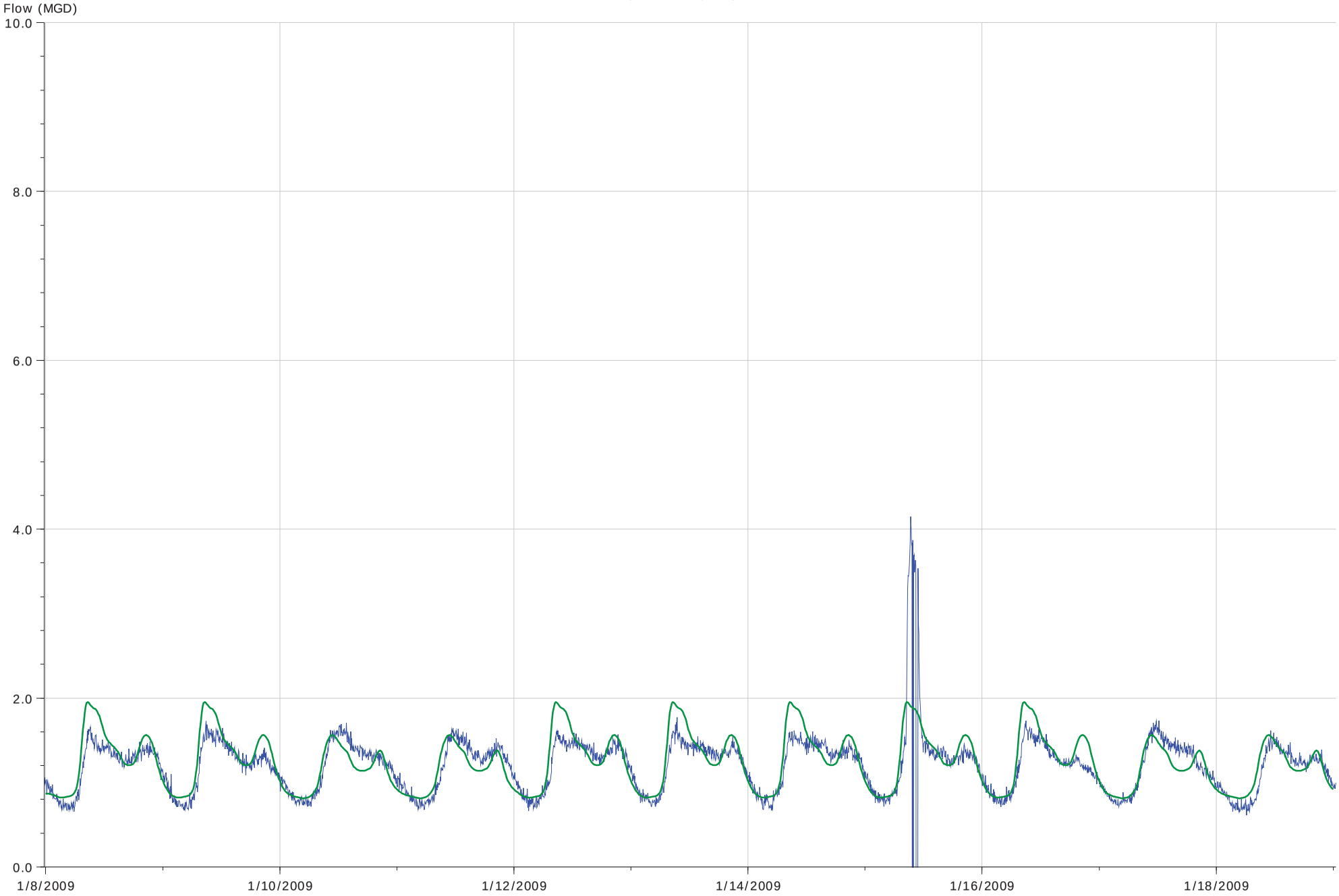
Flow Survey Location (Obs.) 8 913FF14.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	2.207	8.346	58.602
Modeled	3.855	8.352	64.896

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 9 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

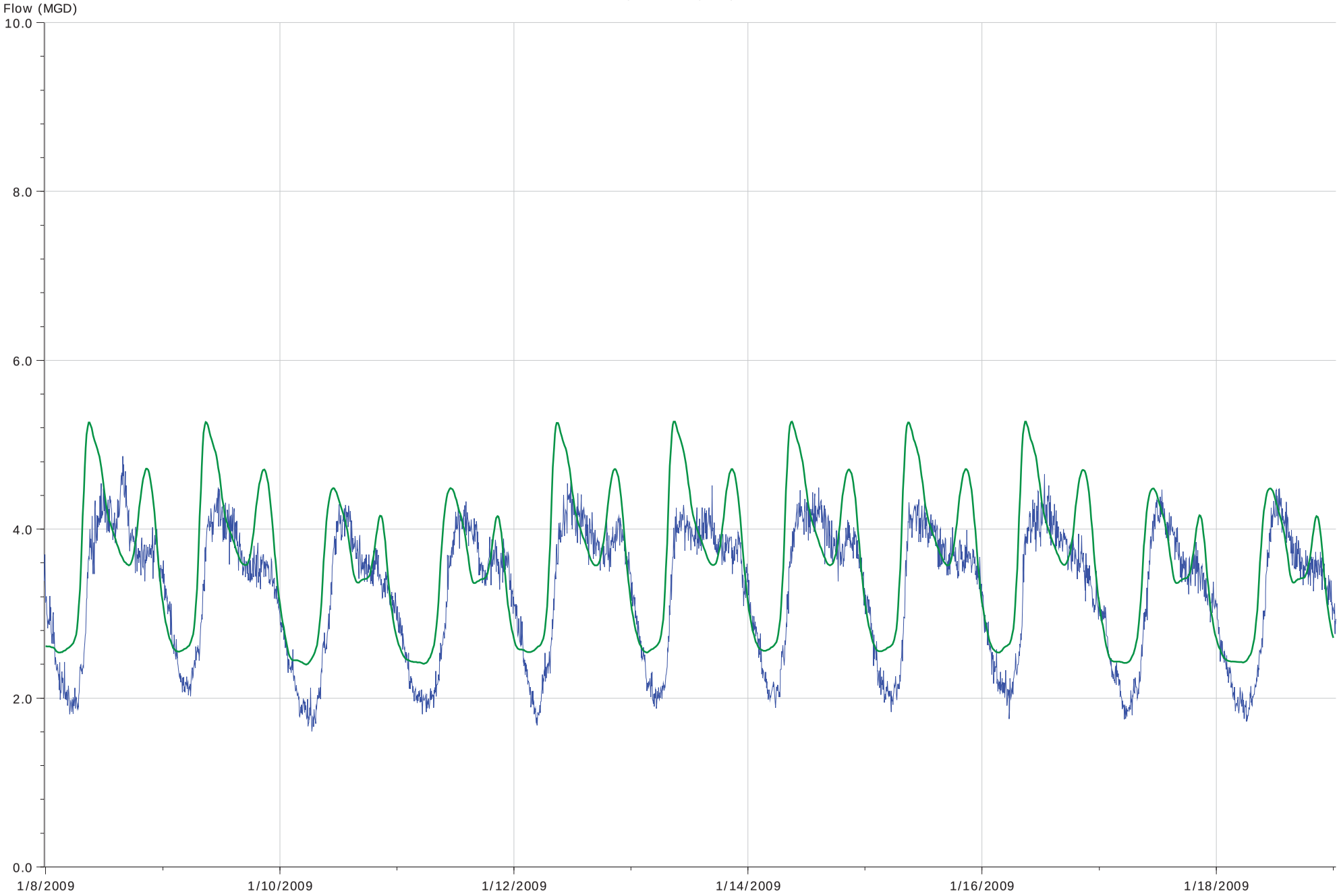
Flow Survey Location (Obs.) 9 714EE17.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	-3.734	4.147	13.215
Modeled	0.814	1.952	13.676

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 10 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

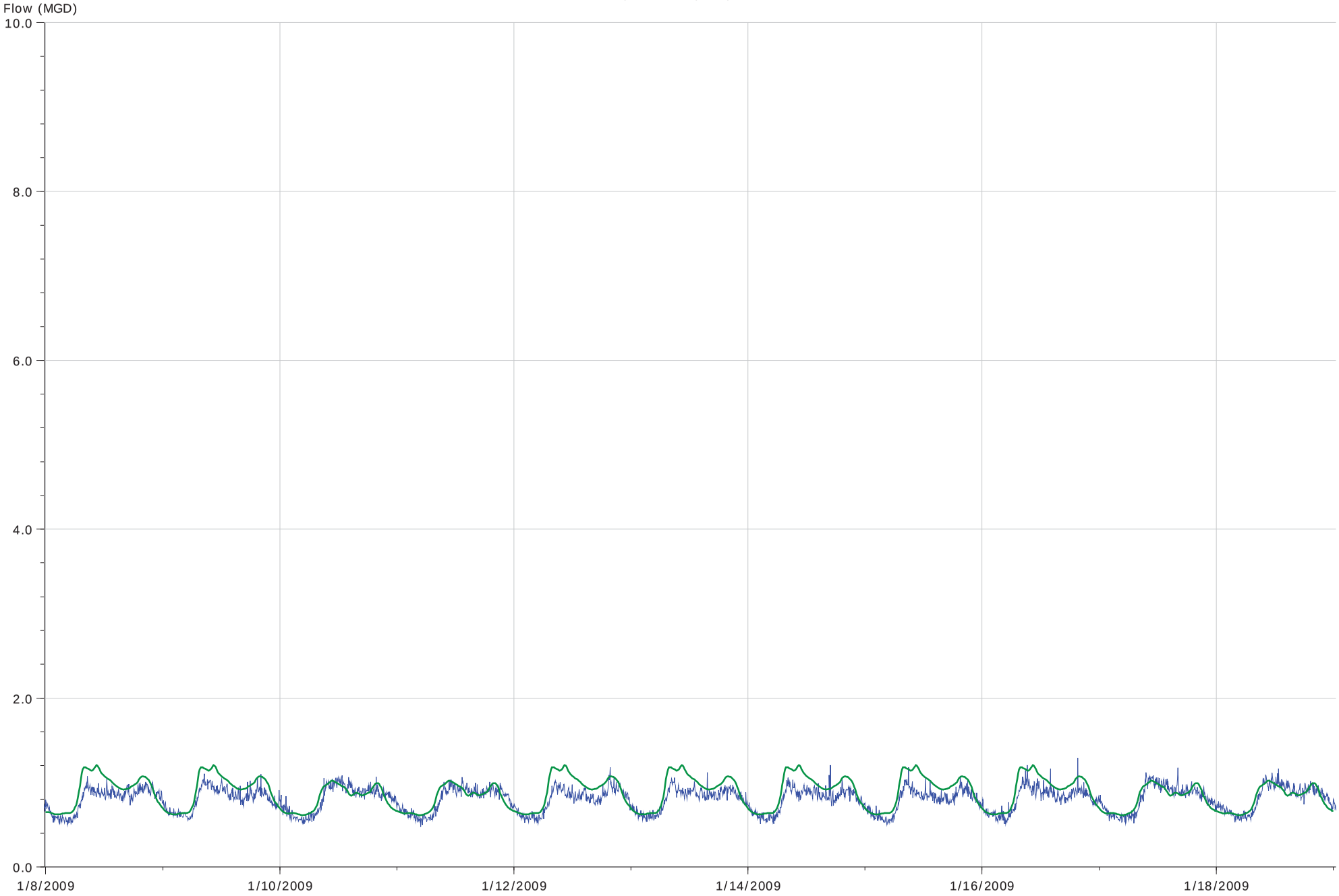
Flow Survey Location (Obs.) 10 504II16.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	1.606	4.867	36.087
Modeled	2.397	5.276	39.775

Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

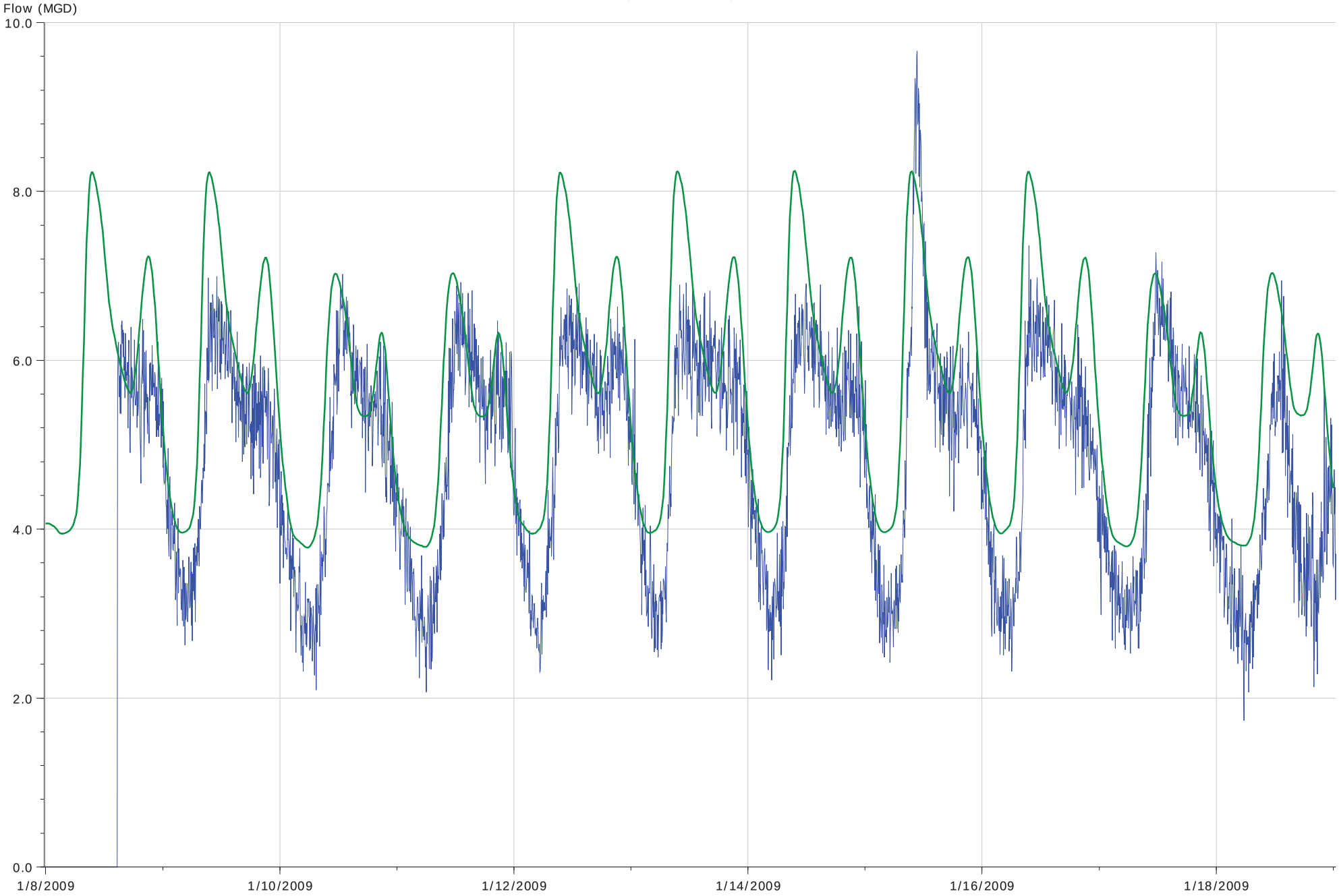
Flow Survey Location (Obs.) 12 302GG18.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	0.478	1.294	8.865
Modeled	0.611	1.204	9.620

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 12 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

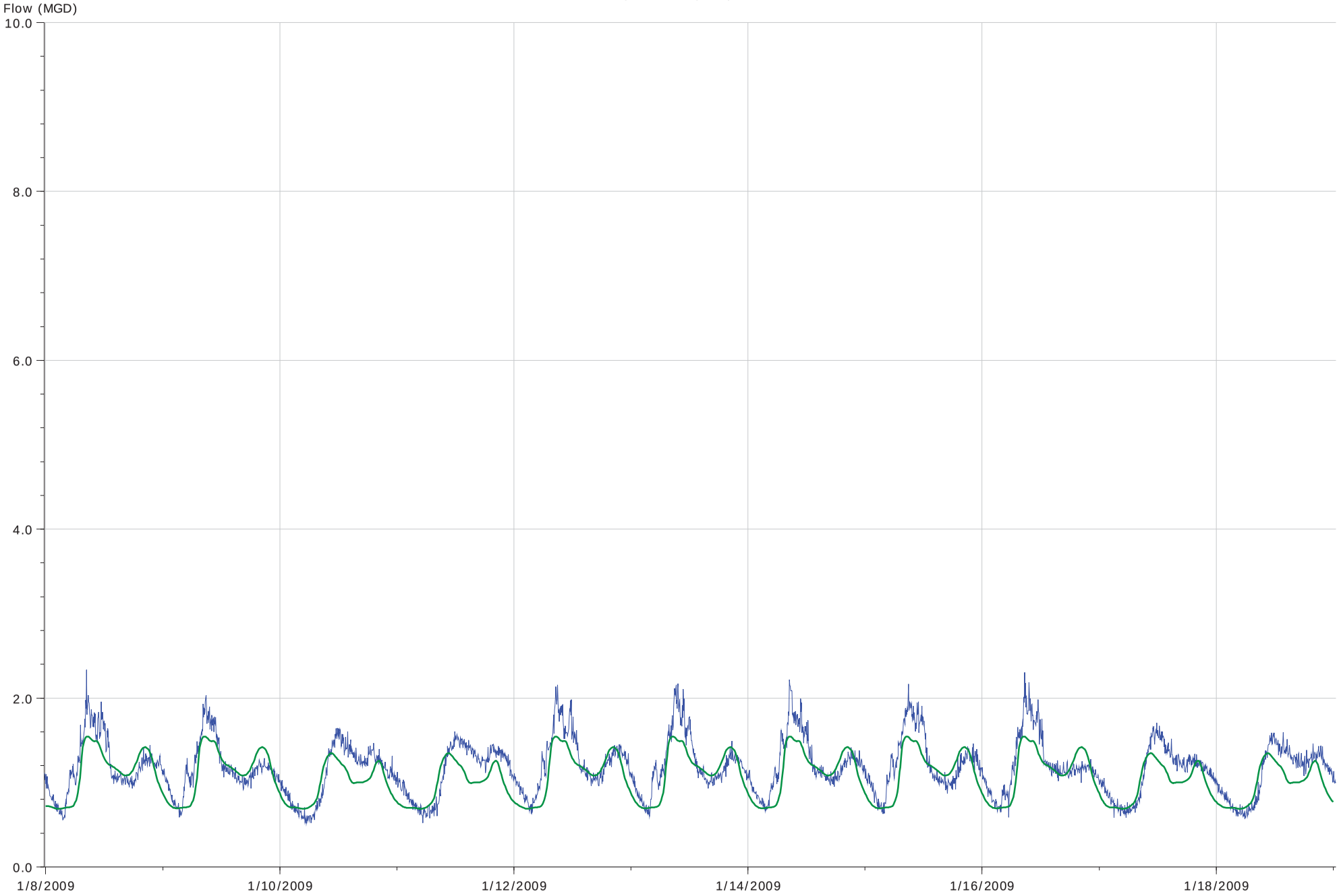
Flow Survey Location (Obs.) 16 314II15.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	0.000	9.664	50.489
Modeled	3.780	8.244	62.415

Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 13 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

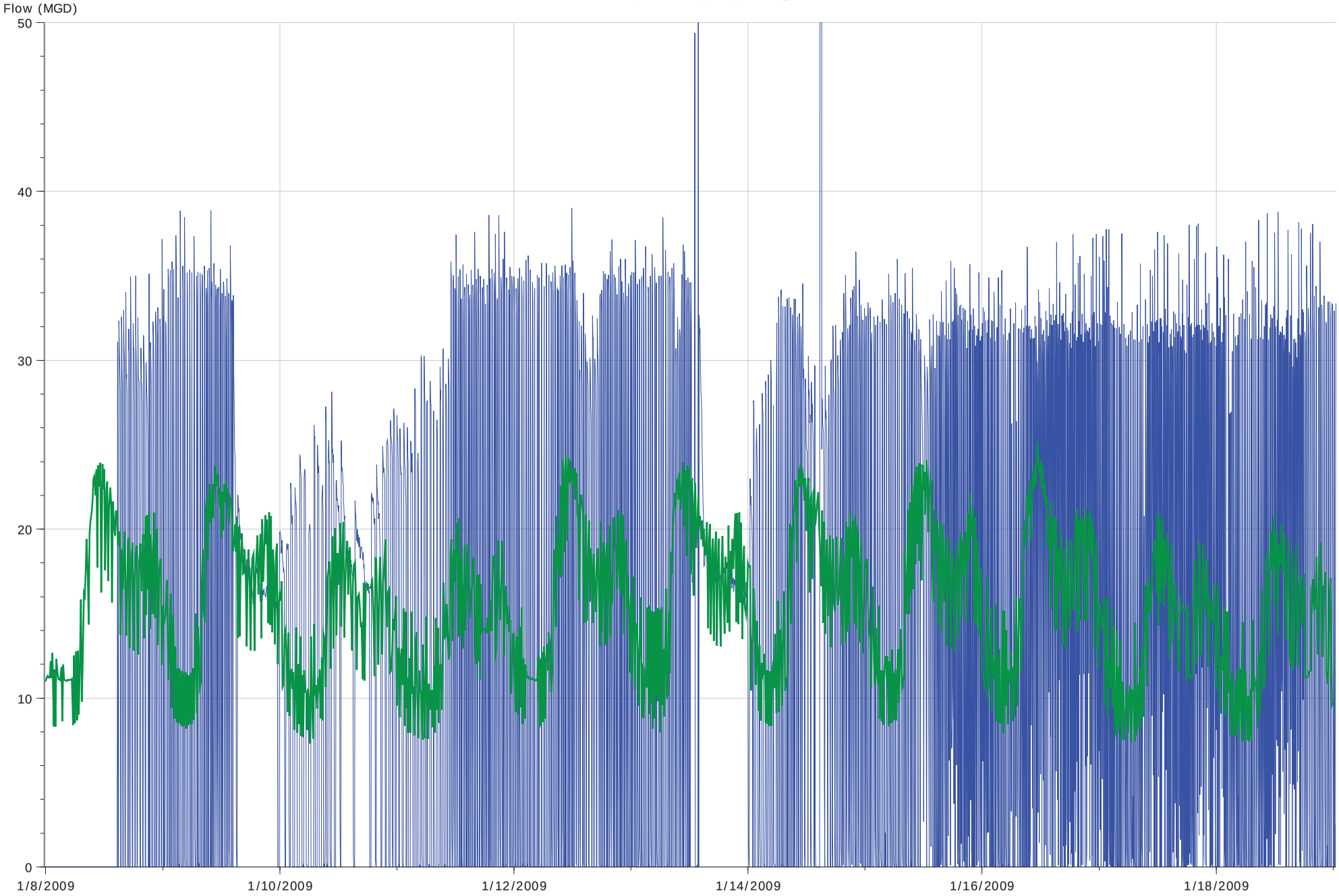
Flow Survey Location (Obs.) 18 307CC18.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	0.518	2.336	12.975
Modeled	0.687	1.545	11.717

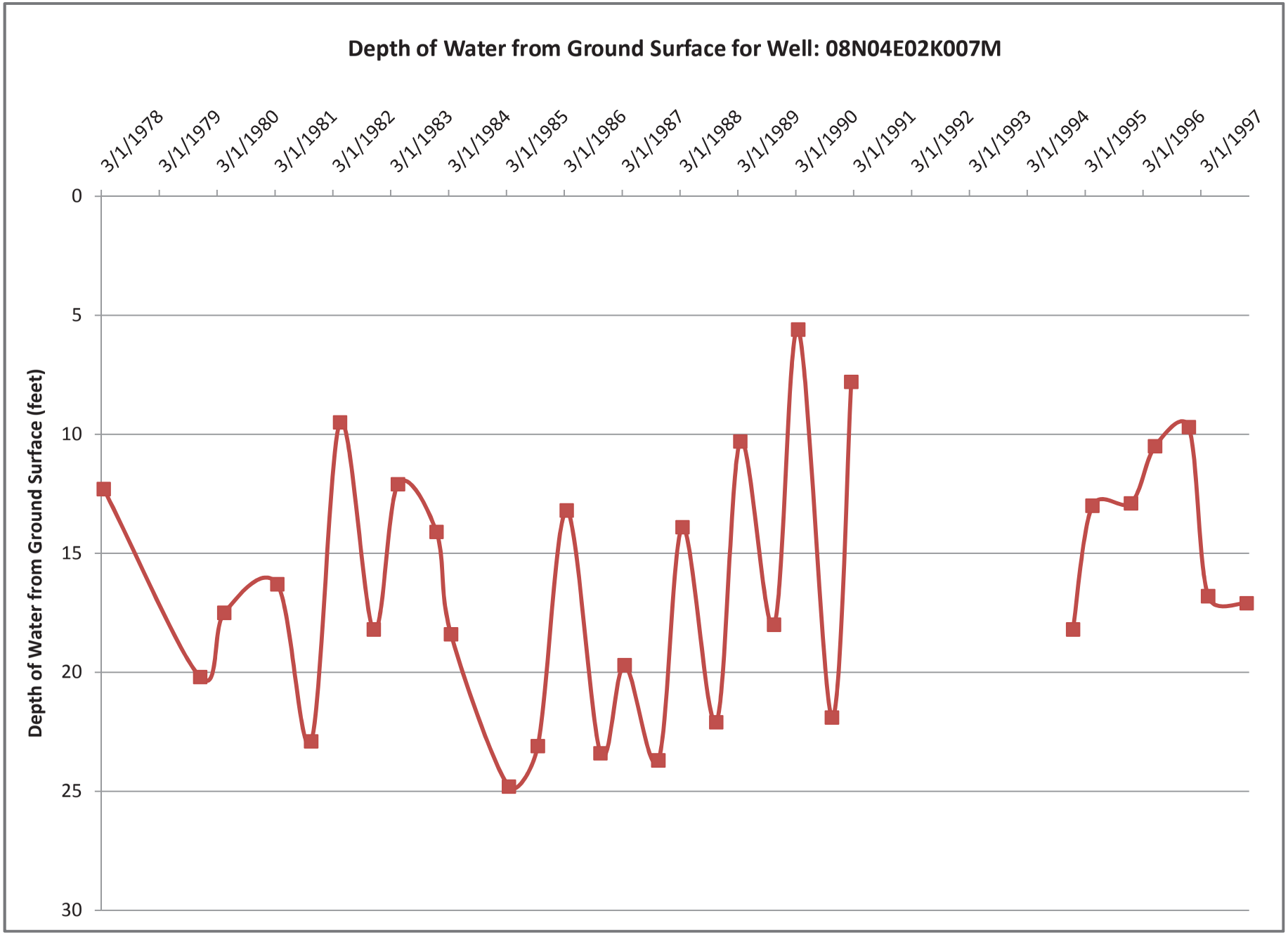
Observed / Predicted Plot Produced by chokshim (5/19/2011 12:14:08 PM) Page 14 of 15
Flow Survey: >Catchment Group>Flow Survey Group>Flow Survey_2009 (2/23/2011 11:49:40 AM)
Sim: >Catchment Group>Run Group>DWCalibration_Ver13>DWF (5/19/2011 12:13:55 PM)
Graph Template: >Catchment Group>Graph Template Group>Graph Template_2009 (5/18/2011 2:20:12 PM)

Flow Survey Location (Obs.) S2 S2_28.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Observed	0.000	79.420	160.080
Modeled	7.310	25.044	170.189

Appendix E – Ground Water Depth for Wells in Study Area


Depth of Water from Ground Surface for Well: 08N04E02K007M
Depth of Water from Ground Surface (feet)
0
5
10
15
20
25
30
3/1/1978
3/1/1979
3/1/1980
3/1/1981
3/1/1982
3/1/1983
3/1/1984
3/1/1985
3/1/1986
3/1/1987
3/1/1988
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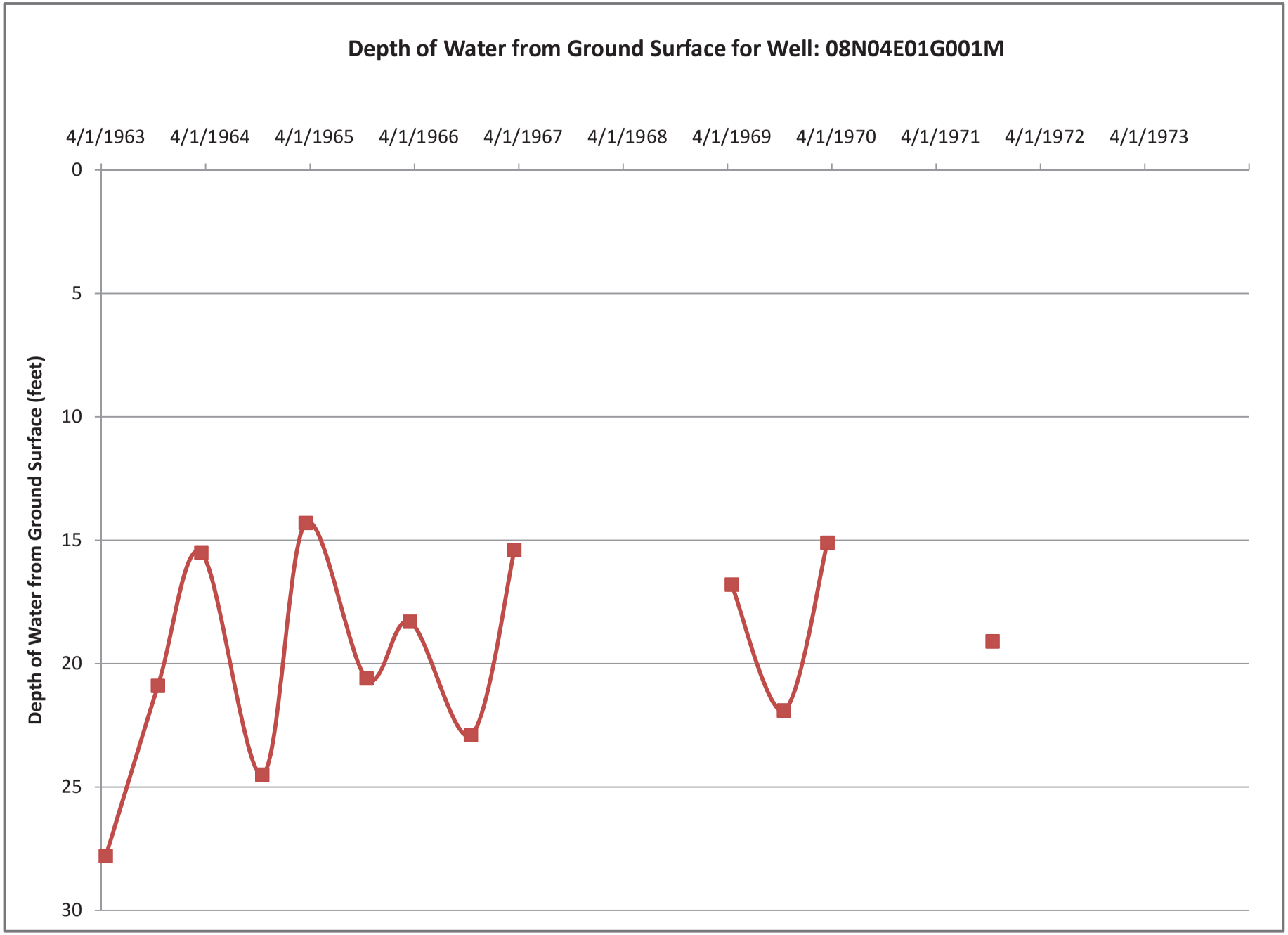
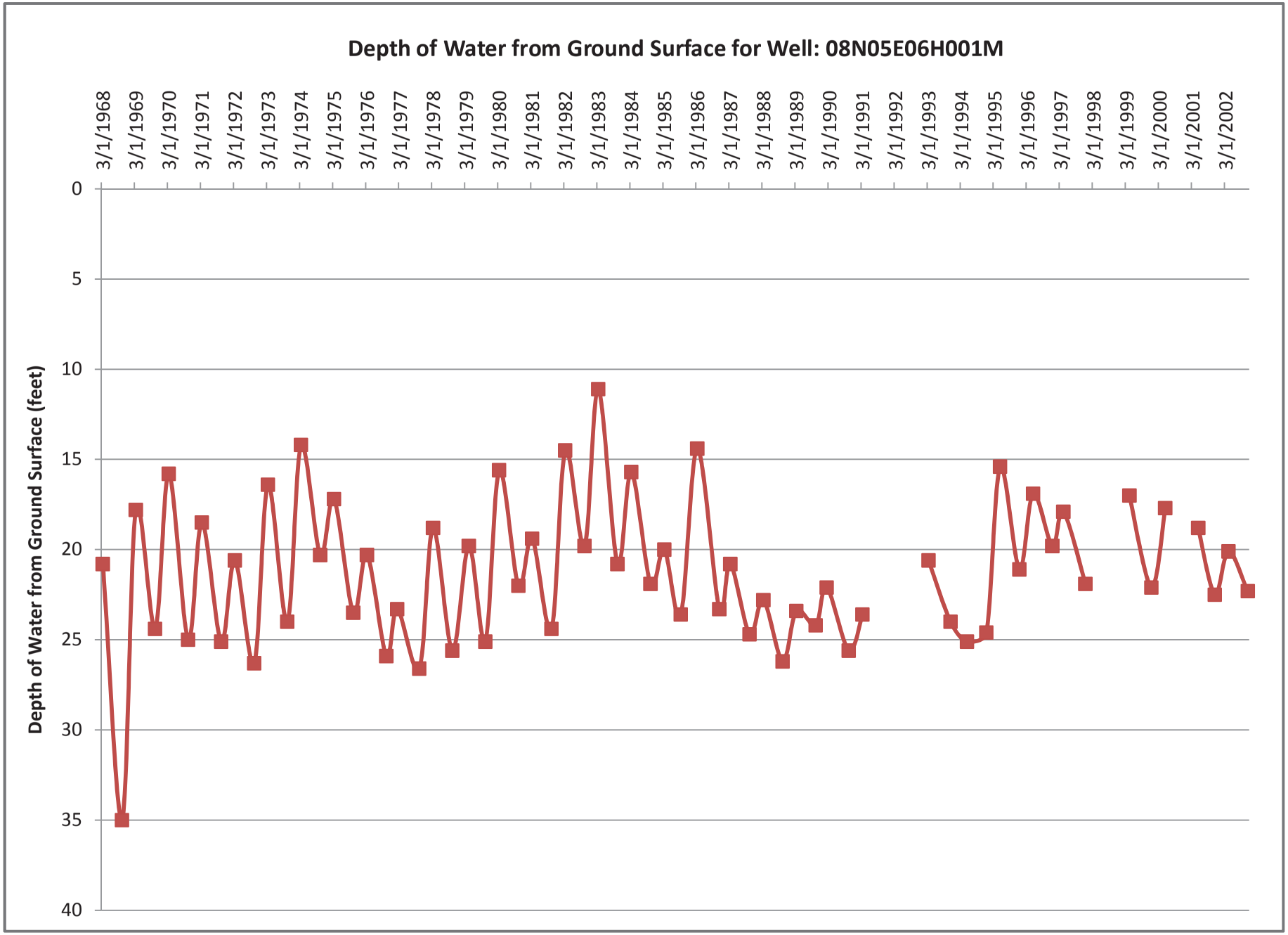
3/1/1995
3/1/1996
3/1/1997


Depth of Water from Ground Surface for Well: 08N04E01G001M
4/1/1963
4/1/1964
4/1/1965
4/1/1966
4/1/1967
4/1/1968
4/1/1969
4/1/1970
4/1/1971
4/1/1972
4/1/1973
0
5
10
15
20
25
30
Depth of Water from Ground Surface (feet)


Depth of Water from Ground Surface for Well: 08N05E06H001M
Depth of Water from Ground Surface (feet)
0
5
10
15
20
25
30
35
40
3/1/1968
3/1/1969
3/1/1970
3/1/1971
3/1/1972
3/1/1973
3/1/1974
3/1/1975
3/1/1976
3/1/1977
3/1/1978
3/1/1979
3/1/1980
3/1/1981
3/1/1982
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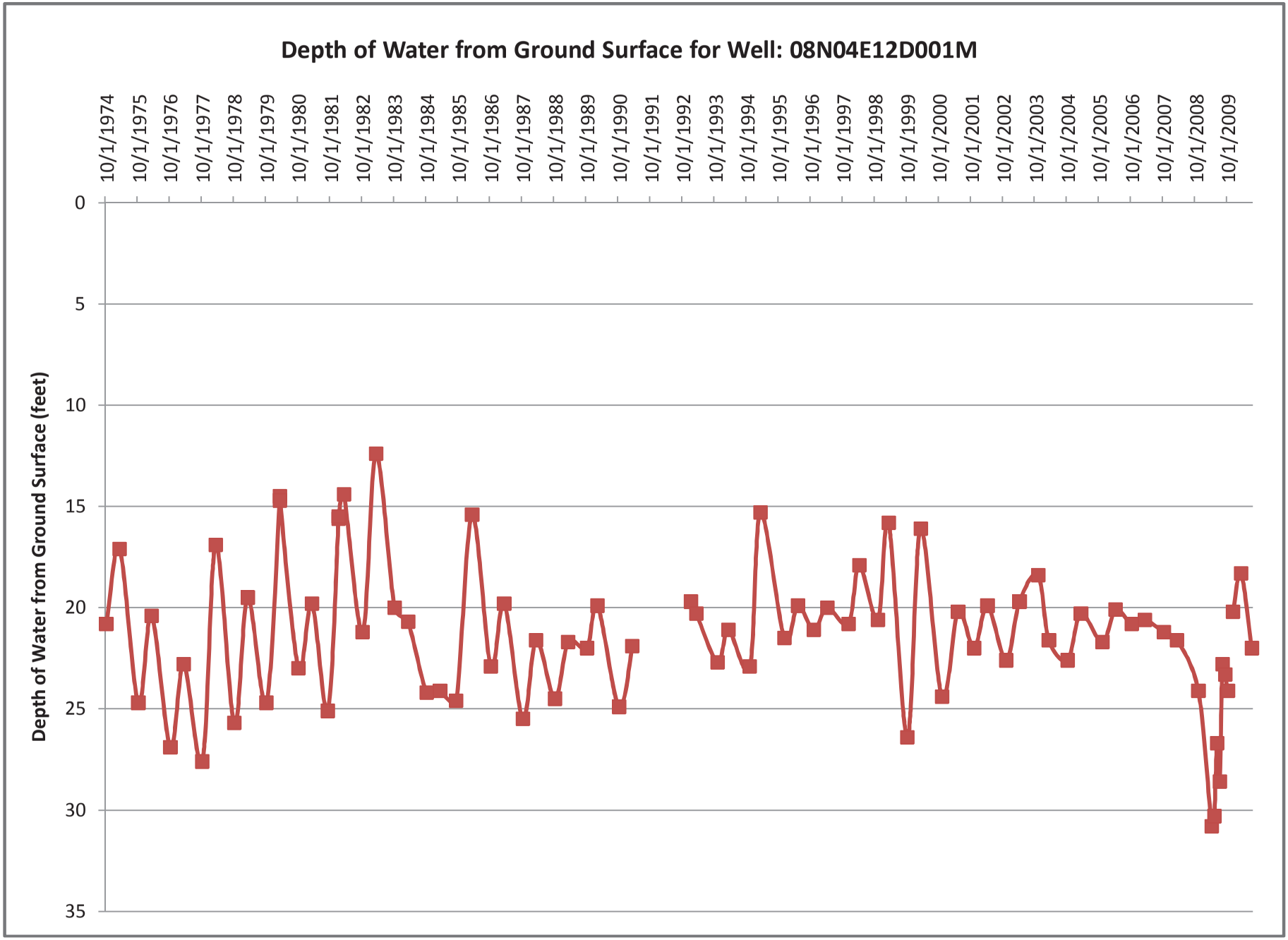
3/1/1996
3/1/1997
3/1/1998
3/1/1999
3/1/2000
3/1/2001
3/1/2002


Depth of Water from Ground Surface for Well: 08N04E12D001M
Depth of Water from Ground Surface (feet)
0
5
10
15
20
25
30
35
10/1/1974
10/1/1975
10/1/1976
10/1/1977
10/1/1978
10/1/1979
10/1/1980
10/1/1981
10/1/1982
10/1/1983
10/1/1984
10/1/1985
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10/1/2000
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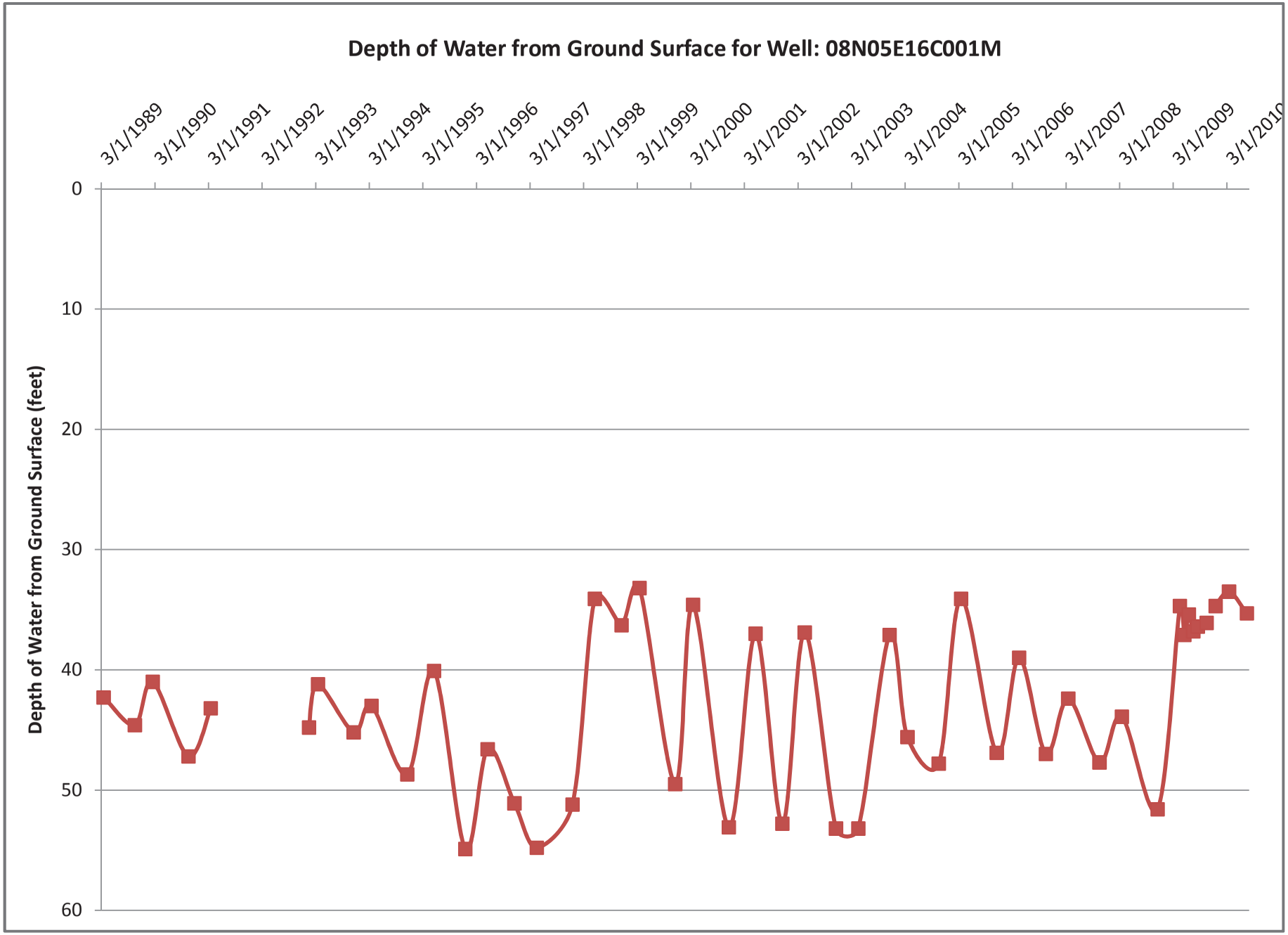
10/1/2008
10/1/2009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Depth of Water from Ground Surface (feet)
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20
30
40
50
60
3/1/1989
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3/1/1996
3/1/1997
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3/1/1999
3/1/2000
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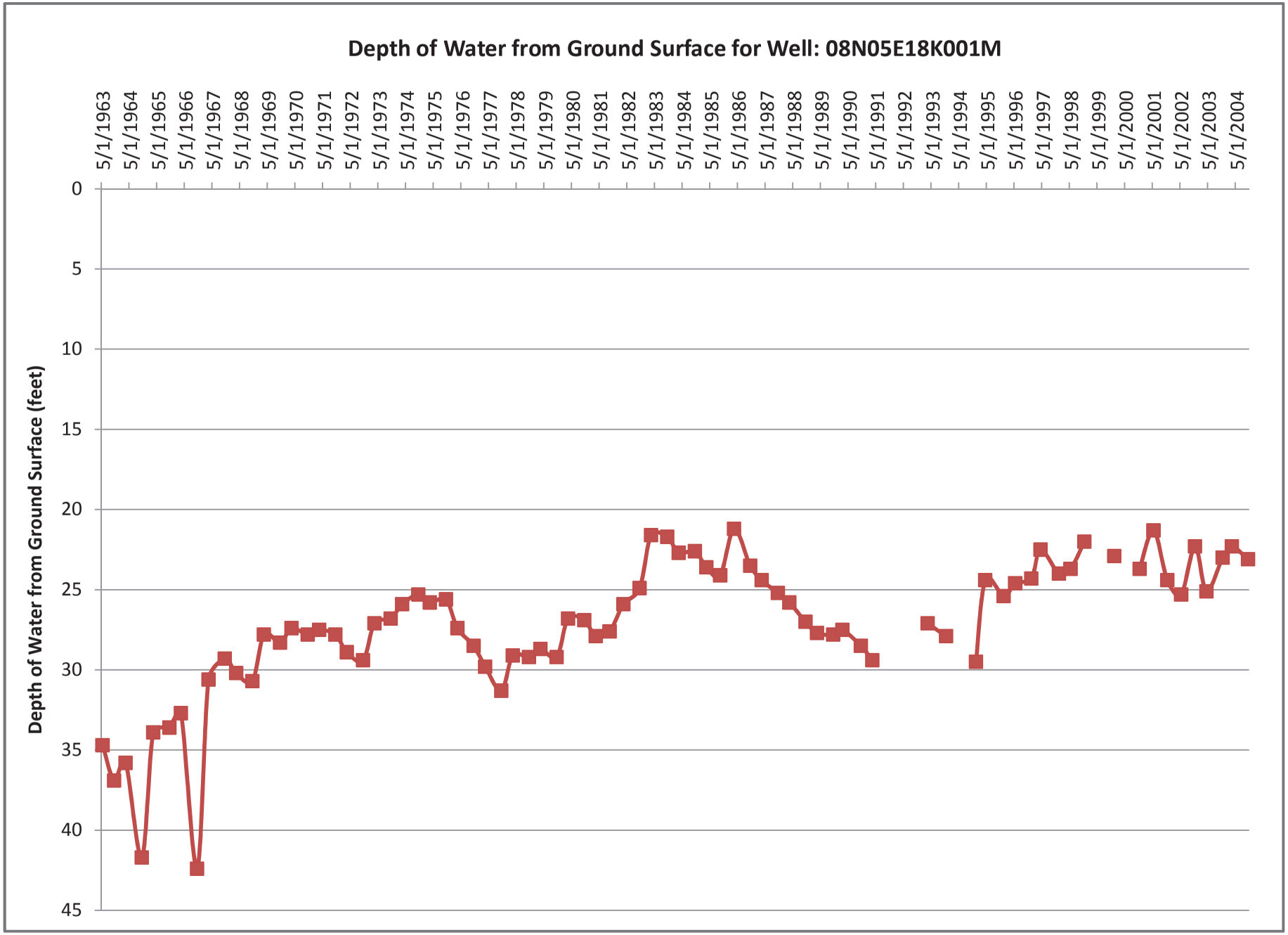
3/1/2009
3/1/2010


Depth of Water from Ground Surface for Well: 08N05E18K001M
Depth of Water from Ground Surface (feet)
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40
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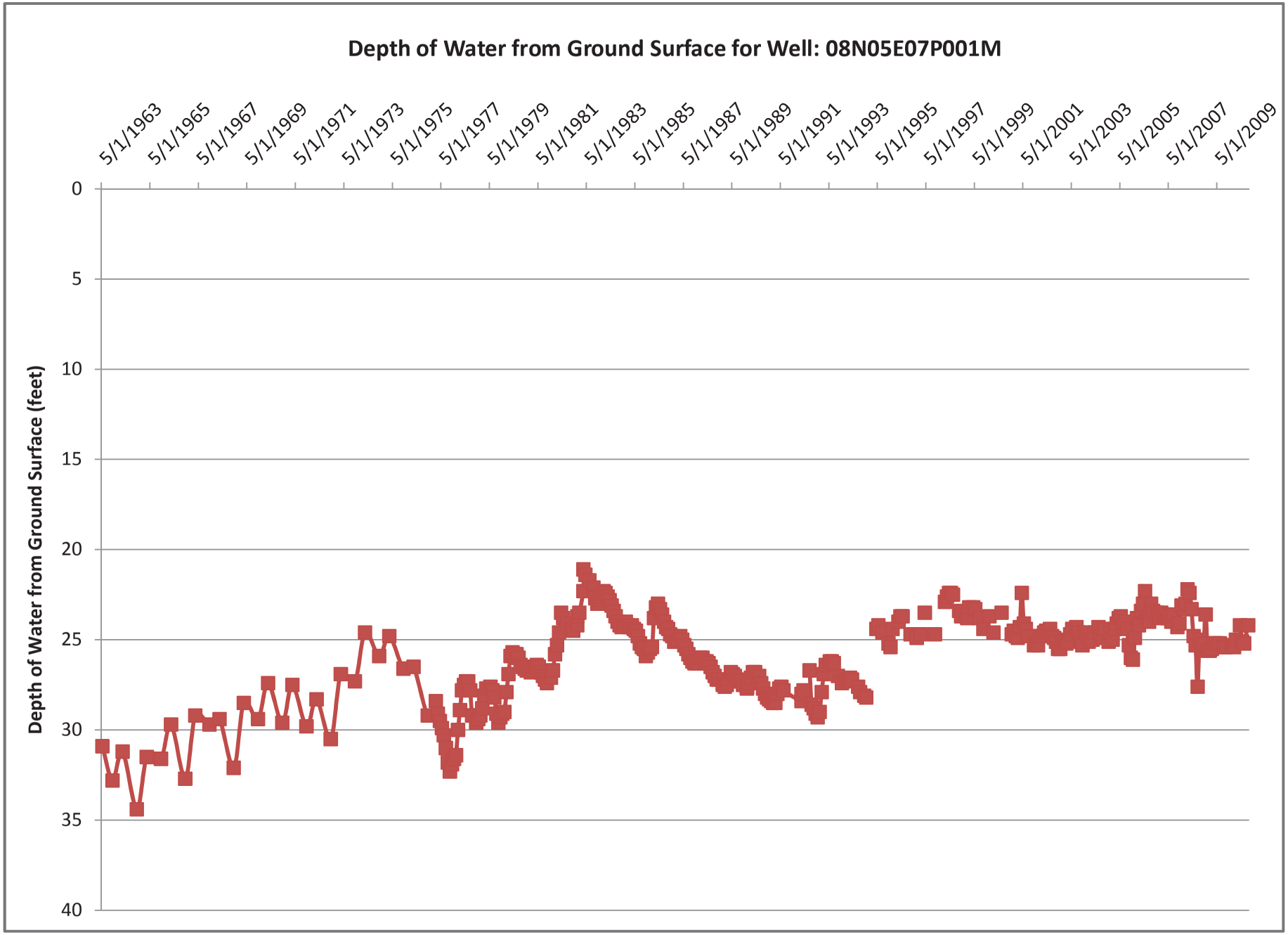
5/1/2004


Depth of Water from Ground Surface for Well: 08N05E07P001M
Depth of Water from Ground Surface (feet)
0
5
10
15
20
25
30
35
40
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5/1/1969
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5/1/1975
5/1/1977
5/1/1979
5/1/1981
5/1/1983
5/1/1985
5/1/1987
5/1/1989
5/1/1991
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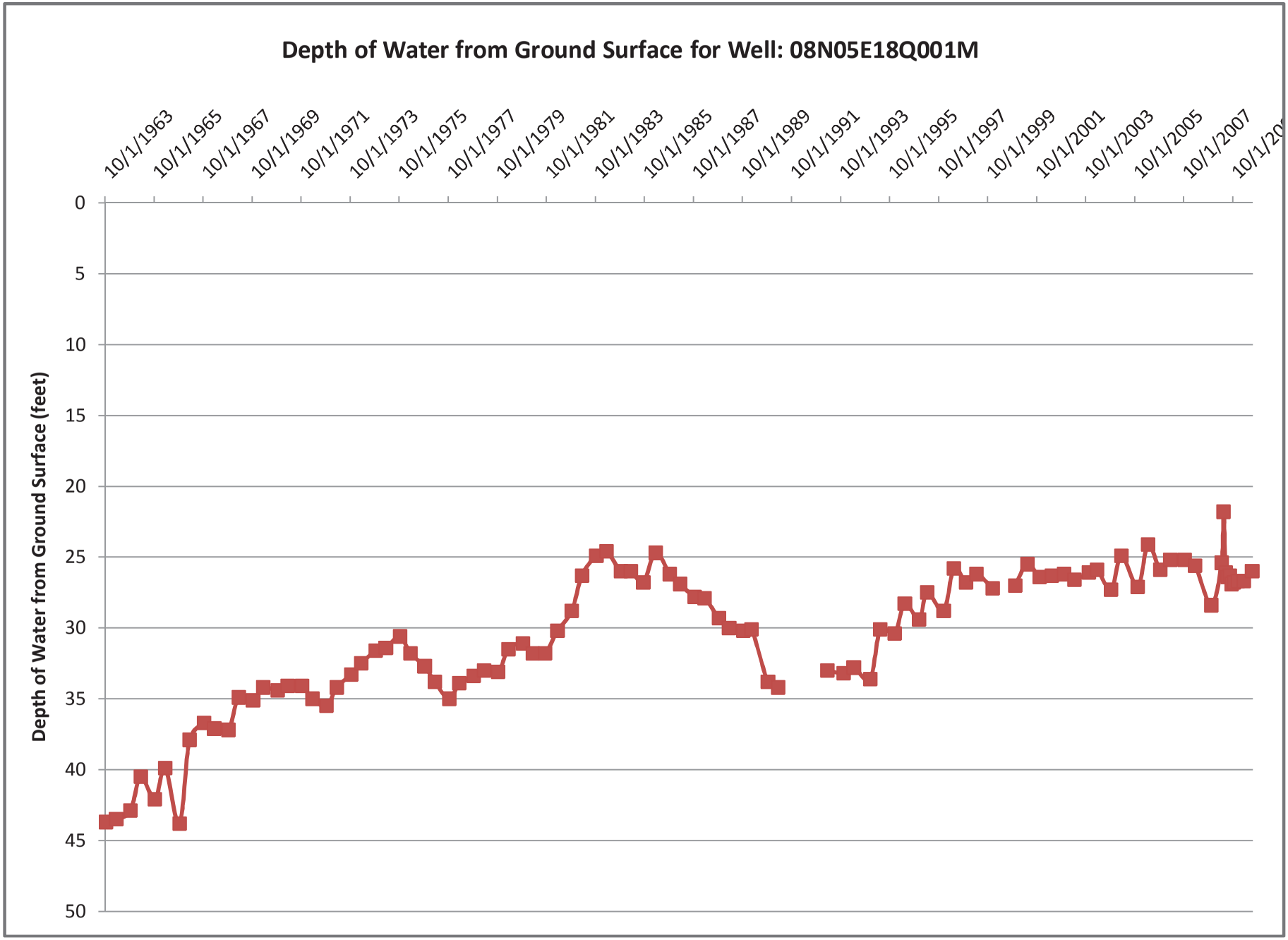
5/1/2007
5/1/2009


Depth of Water from Ground Surface for Well: 08N05E18Q001M
Depth of Water from Ground Surface (feet)
0
5
10
15
20
25
30
35
40
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10/1/1963
10/1/1965
10/1/1967
10/1/1969
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10/1/1973
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10/1/1981
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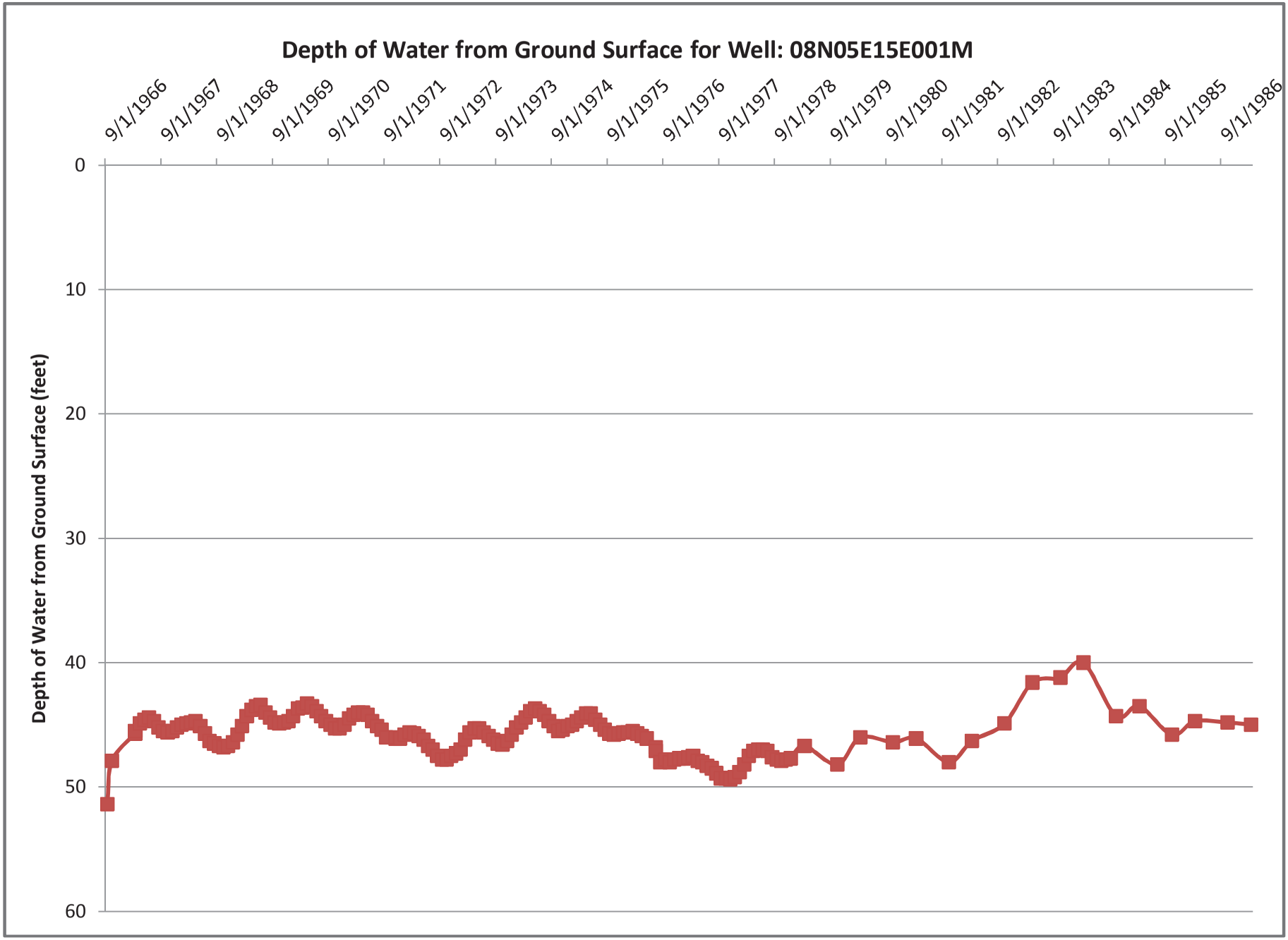
10/1/2005
10/1/2007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9/1/1967
9/1/1968
9/1/1969
9/1/1970
9/1/1971
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9/1/1974
9/1/1975
9/1/1976
9/1/1977
9/1/1978
9/1/1979
9/1/1980
9/1/1981
9/1/1982
9/1/1983
9/1/1984
9/1/1985
9/1/1986
Depth of Water from Ground Surface (feet)
0
10
20
30
40
50
60

City of SACRAMENTO
Department of Utilities

Combined Sewer System (CSS) Improvement Plan Update – Phase 2

Task 3 – Model Sensitivity Analysis Technical Memorandum

November 17, 2011

Version 1 (Final)

1.0 Introduction

Under the City of Sacramento Department of Utilities (DOU) Combined Sewer System (CSS) Improvement Plan Update – Phase 2 project, AECOM will refine and improve the collection system model. The improved model will be used to assess and refine the proposed improvement alternatives as well as recommend additional alternatives to meet the interim and final goals under the permit. Due to the significant capital investment for the proposed improvements, the project team deemed it would be helpful to do an analysis of the design storm rainfall and perform sensitivity checks to the input parameters used in the model simulations.

In evaluating the rainfall data used in model simulations, several issues were raised as potentially having an influence on planning level sizing and decision-making. This memorandum documents the impact of these assumptions on the selected rainfall data for analysis.

a. Volume and Intensity

The current DOU model uses 5-year and 10-year design storms that were developed by City and County Hydrology (CCH) in 1991. The 1991 CCH design storms have no spatial or temporal variation within the CSS study area. DOU staff has observed that in the last 10 years there have been more than 4-5 storms that have exceeded the 10-year CCH design storm volume. National Oceanic and Atmospheric Administration (NOAA) has revised the design storms for CSS study area in 2004, whereas DOU revised the design storms in 2010, both of which support a different design storm volume and peak intensity than currently used in the model. To support the anecdotal observation, DOU staff expressed interest in studying model sensitivity to recently developed design storms (2004 NOAA and 2010 DOU).

b. Recent Rainfall Events

In order to provide a qualitative assessment of the model performance, the project team, under the direction from DOU, decided to validate the model to known 1 or 2 flooding events in the last 10 years. System performance in these events could be compared with recent flooding evidence to determine if the model predicted data was found to be similar in magnitude and location to the observed flooding instances.

c. Historical Rainfall

A potential issue with the 6 hour design storm is the timing of the peak rainfall intensity. In the CCH design storms, the peak hour comes after 3 hours of significant rainfall. AECOM recommended the evaluation of historical rainfall data to determine how the design storm rainfall pattern compared to historical data. In this case, the approach was to compare rainfall preceding the design storm's peak intensity in a number of large historical events.

d. Spatial and Temporal Variability

DOU staff has anecdotal evidences of the storm fronts typically moving from the south-west corner of the CSS and ending at the center of CSS. Since the system outfall (treatment facilities) are in south-west corner of the city, flows from rain falling in this portion of the city are drained out first leaving capacity for the flows coming from farther ends of the system, with possibly fewer flooding instances. To support this anecdotal observation, DOU expressed interest in performing model sensitivity to a temporally varying design storm.

e. Pervious Contributions

In many instances, models are calibrated to smaller events where pervious contributions are non-existent or provide a very small component of contribution. In some cases, when these parameters are used in a larger event (5, year, 10 year, etc.) they may result in an over prediction of runoff from pervious area that may not actually be tributary to the collection system. An evaluation was recommended to examine the relative contribution of pervious areas in the model to ensure that they were not overly conservative in larger events, accounting for runoff that may or may not actually enter the system.

f. Swimming Pools

Lastly, CSS has several swimming pools within the study area, which have an approximately 2-3 feet freeboard. Rainfall occurring over these swimming pools may not make it to the collection system, and therefore should be part of initial losses. The team decided to do a quick check for one wet area to see if accounting for swimming pool freeboard makes any difference to the wet area's flooding volume.

To summarize, the purpose of this study was to do a qualitative check of the model input parameters (rainfall, runoff coefficients, initial losses, etc) to determine the overall impact on predicted flooding volumes. With the potentially large capital investment associated with these planning projects, the evaluation of these parameters was important to understand the sensitivity of the modeling assumptions used for the sizing of the infrastructure. A discussion of the six model sensitivity analyses outlined above is included in Section 2.0 below.

2.0 Sensitivity Analysis

As discussed in the introduction six different factors were identified with DOU for performing model sensitivity. By evaluating the impact of these issues, additional confidence could be gained that the design storm simulations in the model were likely to produce reasonable results for planning future infrastructure and that there was not a substantial bias incorporated into the analysis through the design storm assumptions or model development. The results of this analysis are listed below along with the results and conclusion of the overall study and potential impact on proposed infrastructure sizing.

a. **Perform sensitivity analysis for the current design storms**
The current model uses the 1991 Sacramento City and County Hydrologic (CCH) Manual design storm. DOU requested comparing the model output for the 1991 CCH design storm with the updated 2004 NOAA design storm and 2010 DOU generated design storm. Design storms for 10-year and 5-year return period were input to the model for the three different sources. **Table 1** below documents the model result metrics from the comparison.

Observations:
-From 10-year return period standpoint, the increase in flooding is not significant (less than 10%) with 2004 and 2010 design storms.
-From 5-year return period standpoint, there is a significant increase (~50%) in flooding with 2004 and 2010 design storms.

Table 1: Design storm comparison

Rainfall	10-year			5-year		
	Model (1991)	NOAA (2004)	DOU (2010)	Model (1991)	NOAA (2004)	DOU (2010)
Total Rainfall Depth (inches)	1.62	1.68	1.70	1.32	1.44	1.46
Duration (hours)	6	6	6	6	6	6
Average intensity (in/hr)	0.27	0.28	0.28	0.22	0.24	0.24
Maximum 5 minute Intensity (in/hr)	2.56	2.34	2.34	1.58	1.58	1.60
Model Result Metrics						
Number of Pipes Surcharged (Out of 2791)	2,563 (92%)	2,560 (92%)	2,588 (93%)	2,485 (89%)	2,502 (90%)	2,505 (90%)
Total Length of Pipes Surcharged (Out of 126 miles)	117.1 (93%)	116.8 (93%)	117.2 (93%)	113.2 (90%)	114.3 (91%)	114.2 (91%)
(All volumes are reported for a 48-hour simulation period)						
Total Volume of Flooding (MG) [1]	64.1	68.5	70.4	27.7	37.7	38.7
Total Outflow	15.8	17.5	18.2	4.8	8.5	8.8

Rainfall	10-year			5-year		
	Model (1991)	NOAA (2004)	DOU (2010)	Model (1991)	NOAA (2004)	DOU (2010)
Volume to Street from System (MG) [1]						
Total Street Ponding Volume (MG) due to Gully or System Capacity[1]	48.3	51.0	52.2	22.9	29.2	29.9
Total CSO Volume to River (MG)	83.5	91.3	93.8	50.1	63.0	64.6
Total Flows to Treatment (Sac Regional + CWTP + Pioneer) MG	142.0	145.7	147.2	124.2	131.1	132.3

Notes:
[1]There are two components of flooding:
-Outflows to Street: Combined sewage exiting to the streets from the collection system due to downstream constriction.
-Street Ponding: Storm water cannot enter the combined collection system because the drain inlet (i.e. gully in model) or the connecting sewer has limited capacity.

b. Validate model with recorded flooding evidence

DOU provided the NPDES permit[1] that reported outfall volumes at authorized discharge for storms between 2002 and 2008. DOU also provided rainfall data for 5-day storm that occurred from December 31st 2005 to January 4th 2006. For this storm event, the permit recorded flows at two discharge points (i.e. to the Sacramento River at 006 locations and CWTP at 002 location). The model was used to simulate this event, with the operational controls modified to make sure that rest of the other discharge points (004, 005 and 007) were not discharging to the river. Tables 2 and 3 below show the modeled and recorded flows and volume at the two discharge points respectively.

Observations:
The total volume of CSO for the entire event in the model is lower than that reported in the permit; however it is indicative of the lower rainfall volume used in the model. In general it predicts the peak flows at discharge point 006 closely for the wet days. The model did not produce the lag in flow at 006 for dry days in the event as reported in the permit, and the flows going to CWTP in the model do not match that with the permit. This is likely due to a different strategy taken by the Sump 2 operators for this event. The total CSO volume comparison

[1] ORDER NO: R5-2010-0004 NPDES NO: CA0079111

provides the qualitative assessment of the model required for this task. The total CSO volume comparison is acceptable and the model is validated to simulate wet weather events.

Table 2: Modeled peak flows and volume for Dec 2005 storm

Date	Discharge Point 002 - CWTP		Discharge Point 006 - Pioneer		Untreated Volume (MG)	Total Outfall Volume – 002 + 006 + Untreated (MG)	Total Rainfall (in)
	Peak Flow (MGD)	Treated Volume (MG)	Peak Flow (MGD)	Treated Volume (MG)			
12/30/2005	130	29	0	0	0	29	0.9
12/31/2005	130	107	482	191	0	298	2.3
1/1/2006	130	38	252	11	0	49	0.7
1/2/2006	130	57	289	32	0	89	1.0
1/3/2006	0	0	0	0	0	0	0.0
1/4/2006	0	0	0	0	0	0	0.0
	-	231	-	234	0	465	4.9

Table 3: Observed peak flows and volume for Dec 2005 storm as reported in the permit

Date	Discharge Point 002 - CWTP		Discharge Point 006 - Pioneer		Untreated Volume (MG)	Total Outfall Volume – 002 + 006 + Untreated (MG)	Total Rainfall (in)
	Peak Flow (MGD)	Treated Volume (MG)	Peak Flow (MGD)	Treated Volume (MG)			
12/30/2005	0	0	0	0	-	0	0
12/31/2005	130	65	500	193	61	319	4.1
1/1/2006	0	0	270	75	-	75	0.7
1/2/2006	130	24	382	117	-	141	1.0
1/3/2006	0	0	63	32	-	32	0.0
1/4/2006	0	0	35	7.7	-	7.7	0
	-	89	-	425	61	575	5.7

c. Evaluate design storm peak hourly rainfall with antecedent condition

The selected design storm event consists of a defined rainfall pattern; while the peak hourly intensity drives the majority of the flooding in the model, the rainfall preceding that peak hour generates a system that is already approaching capacity limitations. On a qualitative basis, it was important to understand whether the particular rainfall pattern in the design storm represented a reasonable scenario that was likely to occur and that it was not an overly conservative pattern of rainfall. To examine this issue, DOU provided historical hourly rainfall data for a 30-year period. The peak hourly intensity for the current model design storm (1991 CCH) and the antecedent 1-hour and 3-hours were compared with discrete storm events over the 30-year period. Table 4 below tabulates the comparison.

Observations:
In this listing of storm events over a 30 year period, it would be expected that the 10-year design storm would be similar to the 3rd largest actual event during the 30 year period. In this case, there were 4 events in the 30 year period that exceeded the design storm peak hourly intensity. Of these four events, two events were likely to generate a higher overall response for flooding (2/7/1983 and 1/9/1995) due to their much higher peak intensities. Of the other storms, the 1/12/1990 was similar to the design storm in terms of preceding 1 hour and 3 hour rainfall. The fourth event (1/26/1997), though slightly higher with the peak hourly intensity, had less rain overall in the preceding 3 hours. Though this is a qualitative analysis, it is reasonable that the design storm would likely fall somewhere between the 1/12/1990 and 1/26/1997 events, being between the 3rd and 4th largest events for flooding within a 30 year period. As such, it seems as if the design storm distribution is appropriate based on this comparison.

Table 4: Design Storm Peak Intensity and Antecedent Rainfall

Design vs. Historical	Duration (hours)	Average Intensity (inches/hour)	Peak hourly Intensity (inch/hr)	1-hour rainfall volume that precedes the peak (inches)	3-hour rainfall volume that precedes the peak (inches)
10-year Design Storm	6	0.27	0.66	0.36	0.74
2/7/1983 15:00	7	0.17	0.96	0.04	0.2
1/12/1990 18:00	15	0.17	0.71	0.5	0.68
1/9/1995 18:00	26	0.17	1.41	0.04	0.1
1/26/1997 2:00	6	0.25	0.74	0.21	0.43

d. Assess model results with temporal variation of design storm

During 2008 flow monitoring activities, DOU also received radar based rainfall data for the study area. Note that the radar based rainfall data was not used for model calibration since the storm intensities didn't match well with point rain gage intensities. However, this dataset was utilized to study typical storm front movement and the time period for the storm to move across the City and support the observations made by DOU. The list below provides five examples of storms observed during the 3-month period and the direction of the movement. The purpose of listing these historical temporally varying storms is to gauge the typical direction of the storm movement and the typical window within which the storm front moves in and out of the CSS. The storm volumes and intensities were not compared with the design storm.

- Example 1: 02/05/2008 16:15 - 02/05/2008 16:35 (moves from south-east to center in 20 minutes)
- Example 2: 02/05/2008 20:55 - 02/05/2008 21:30 (moves from south-east to center in 35 minutes)
- Example 3: 02/11/2008 13:40 - 02/11/2008 14:20 (moves from south-west to center in 40 minutes)

- Example 4: 02/15/2008 01:15 - 02/15/2008 01:35 (moves from south-west to center in 20 minutes)
- Example 5: 02/16/2008 18:30 - 02/16/2008 18:55 (moves from south-west to center in 25 minutes)

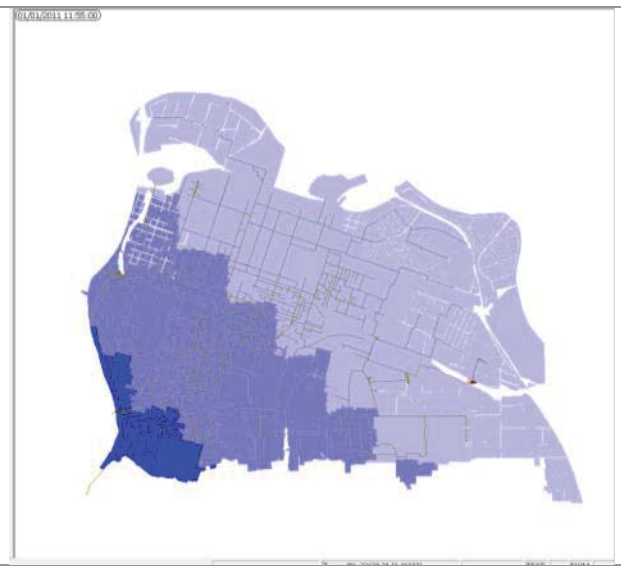
Using the same radar pixels, four different temporally varying 10-year design storms were synthesized with the peak intensity staggered at a 5-10 minute interval over a 30-60 minute window. **Figure 1** below presents an example of a temporal variation of the design storm that moves from south-west corner of the city to north-east within a 30 minute window.

Table 5 below tabulates the model result metrics for the four temporally varying storms and compares it with design storm with no temporal variation.

Observations: The impact of the temporal variation was apparent in the analysis. In the case of the movement from southwest to northeast, there was a reduction in the flooding volume and CSO volume; this was expected because an additional portion of the runoff from leading edge of the storm is able to be treated earlier in these scenarios. With the movement from the southeast to northwest, the volume of flooding and CSO volume actually increase. In both cases, however, the change in magnitude is minimal; the most likely reason for this is that the majority of the flooding is based on local capacity issues that are governed by the localized rainfall and not the downstream system capacity.

Time: 11:55 am

Intensity (in/hr)	Color
0-0.5	
0.5-1.0	
>1.0	

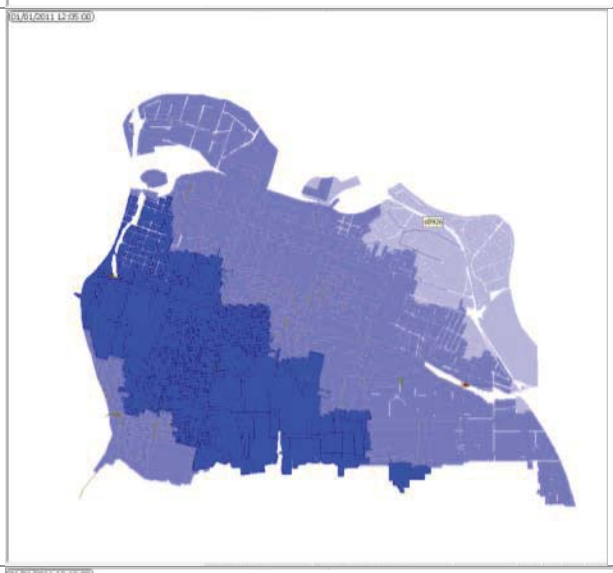


Time: 12:00 pm

Intensity (in/hr)	Color
0-0.5	
0.5-1.0	
>1.0	

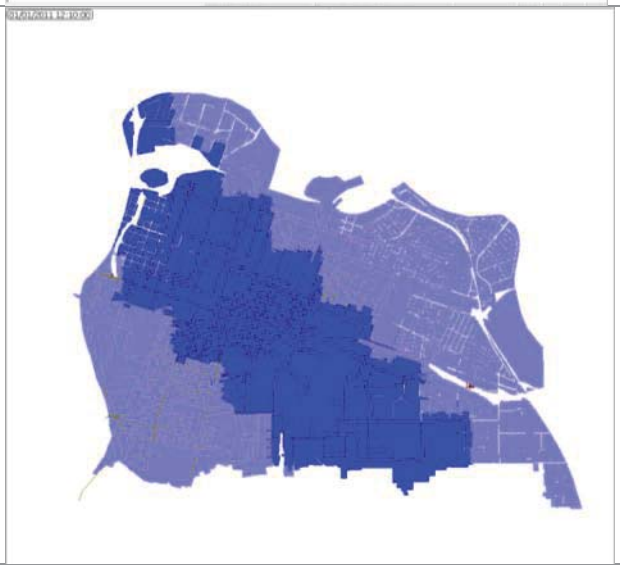
Time: 12:05 pm

Intensity (in/hr)	Color
0-0.5	
0.5-1.0	
>1.0	



Time: 12:10 pm

Intensity (in/hr)	Color
0-0.5	
0.5-1.0	
>1.0	



Time: 12:15 pm

Intensity (in/hr)	Color
0-0.5	
0.5-1.0	
>1.0	

Time: 12:20 pm

Intensity (in/hr)	Color
0-0.5	
0.5-1.0	
>1.0	

Time: 12:25 pm

Intensity (in/hr)	Color
0-0.5	
0.5-1.0	
>1.0	

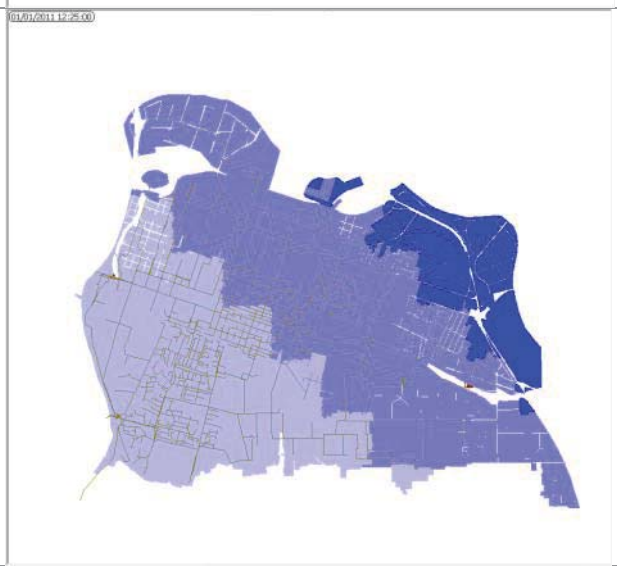


Figure 1: Example of temporal variation of 10-year design storm moving from south-west to north-east direction

Table 5: Model output metrics for temporally varying design storms

Model Result Metrics	10-year design storm (1991 CCH)				
	No Temporal Variation	Temporal Variation 1 (from SW to NE within 30 min)	Temporal Variation 2 (from SW to NE within 60 min)	Temporal Variation 3 (from SE to NW within 30 min)	Temporal Variation 4 (from SE to NW within 60 min)
Number of Pipes Surcharged	2,563	2,553	2,550	2,562	2,565
Total Length of Pipes Surcharged	117.1	116.5	116.5	116.9	117.1
(All volumes are reported for a 48-hour simulation period)					
Total Volume of Flooding (MG)[1]	64.1	63.6	63.0	64.4	64.6
Total Outflow Volume to Street from System (MG) [1]	15.8	15.6	15.4	15.8	15.8
Total Ponding Volume (MG) due to Gully or System Capacity [1]	48.3	48.0	47.6	48.6	48.8
Total CSO Volume to River (MG)	83.5	81.9	81.6	82.7	83.5
Total Flows to Treatment (Sac Regional + CWTP + Pioneer)	142.0	142.5	142.9	141.4	141.1

Notes:
[1]There are two components of flooding:
-Outflows to Street: combined sewage existing to the streets from the collection system due to downstream constriction.
-Street Ponding: storm water cannot enter the combined collection system because the drain inlet (i.e. gully in model) or the sewer/MH has limited capacity.

e. Determine % of runoff volume contribution

Five different design storms (2-year, 5-year, 10-year, 25-year and 50-year) for 6-hr duration were applied to the model to understand the ratio of total volume from pervious and impervious surfaces with increasing rainfall volume. Table 6 below tabulates the total volume from pervious and impervious surfaces for different sub-catchments of different size (i.e. acreage)

Observations: The ratio of pervious to impervious runoff volume is appropriate across various design storms and across subcatchments, of different sizes. The pervious contributions in the 5-year and 10-year design storms are only a small portion of the total runoff- this is a reasonable result that indicates the model is not likely over predicting pervious contributions to the collection system. An example of a scenario where this may be an issue is in the 50-year storm

for subcatchment 1142; in this case, the pervious contribution is almost 80% of the impervious. If a solution were being designed for a 50-year storm, it would be critical to determine if this large amount of runoff volume from the pervious area were able to enter the collection system. In this case, the levels of storm event in this evaluation do not show any cause for a potential issue in terms of unusually high pervious contributions.

Table 6: Impervious and Pervious runoff volume for design storms

Storm sub-catchment ID	Area (acres)	2-year			5-year			10-year			25-year			50-year		
		Perv	Impv	Ratio	Perv	Impv	Ratio	Perv	Impv	Ratio	Perv	Impv	Ratio	Perv	Impv	Ratio
		(MG)	(MG)	-	(MG)	(MG)	-	(MG)	(MG)	-	(MG)	(MG)	-	(MG)	(MG)	-
1602	0.2	0	0	0	0	0.01	0	0	0.01	0.00	0	0.01	0.00	0	0.01	0.00
827	0.6	0	0.01	0	0	0.02	0	0	0.02	0.00	0	0.02	0.05	0	0.03	0.12
233	1	0	0.01	0	0	0.02	0	0	0.02	0.04	0	0.03	0.11	0.01	0.03	0.19
811	5	0	0.06	0	0	0.08	0	0	0.09	0.03	0.02	0.11	0.15	0.03	0.13	0.24
834	10.4	0	0.11	0	0	0.14	0	0.01	0.17	0.03	0.04	0.2	0.18	0.07	0.23	0.28
1142	20.5	0	0.1	0	0	0.14	0	0.02	0.17	0.10	0.1	0.2	0.51	0.19	0.23	0.80
1334	51.8	0	0.86	0	0	1.11	0	0.02	1.39	0.01	0.12	1.65	0.07	0.21	1.89	0.11
All "storm" catchments	7,255	0.3	97.7	0.003	12.0	127.1	0.09	32.5	157.2	0.21	56.2	188.4	0.30	78.4	216.1	0.36

Ratio = Runoff from Pervious surface (MG)/Runoff from Impervious surface (MG)

f. Evaluate reduction in runoff due to swimming pool freeboard

There is a considerable freeboard (approx. 2') in swimming pools. The rain falling over the pools does not become runoff until the freeboard. Since the 10-year design storm is 1.7 inches, all the rainfall falling in the pool should be added to initial losses. Wet area 4 was identified for this analysis. **Figure 2** below shows the wet area 4 overlaid with an aerial in google earth. The swimming pools within this area were counted and the average size of the swimming pool was approximated.

Observations:
Total pools in wet area 4 = 38
Approximate average size of pool: 30 feet x 15 feet
Total square footage = 17,100 SF
10-year design storm (1991 CCH) = 1.7 inches = 0.1416 feet
Total volume of rainfall over the pools = 17,100 SF x 0.1416 feet = 2,422 CF = 0.018 MG

Assuming that in the current model swimming pools are part of the impervious surface in the model, all of the rainfall falling on the pools during design storm period ends up filling the pool and not contributing to the collection system. For the current 1991 design storm, the total flooding in Wet Area 4 is 2.9 MG. If initial losses to runoff due to freeboard in swimming pools are accounted, wet area 4 would see a reduction in flooding by 0.018 MG, a small fraction (0.6%) of total flooding to impact the flooding issues in Wet Area 4. It is not recommended to make any adjustments to model parameters to account for such a small reduction in flooding.

Figure 2: Wet Area 4 boundary

3.0 Conclusion

Out of the six sensitivity analyses performed so far, only the first analysis a (i.e. using an updated design storm) may require additional discussion of the model design storm. To summarize the analyses:

a. It is recommended to either use the updated DOU generated 2010 design storm or have a further discussion on the analysis. The updated 2010 design storms do cause higher flooding in the model, and therefore a more conservative estimate on flooding.

b. The analysis found the model performance was within acceptable range for the CSO occurences for a 5-day storm event. The analysis was only able to compare the magnitude of total CSO volume at all authorized discharge locations.

c. Given the historical evaluation, the distribution of rainfall in the design storm appears to be appropriate and would likely fall between the 3rd and 4th largest actual events from the past 30 years. In general, the shape and distribution of the rainfall is consistent with the expected level of service for a 10 year design storm.

d. Temporal and spatial variations are found to have a slight impact on predicted flooding volumes, but the impact is very minimal when compared to the overall global model predictions. Given the planning level analysis, the use of any spatial or temporal variation is not recommended due to the negligible impact on results.

e. The runoff coefficients for pervious surface areas in the model appear within acceptable range and are consistent over design storms in the 5 and 10 year frequency level. The model is not likely to show over prediction on contributions from pervious areas in these design storms given the analysis of subcatchments.

f. The freeboard in swimming pools does not provide any considerable reduction in runoff volume and does not require any change of model parameters.

City of SACRAMENTO
Department of Utilities

Combined Sewer System (CSS) Improvement Plan Update – Phase 2

Task 3 – Project Optimization Summary

November 21, 2011

Version 1 (Final)

1.0 Introduction

Under the City of Sacramento Department of Utilities (DOU) Combined Sewer System (CSS) Improvement Plan Update – Phase 2 project, AECOM will refine and improve the existing combined sewer hydrologic and hydraulic (H&H) model. The improved H&H model will be used to assess and refine proposed improvement alternatives as well as recommend additional alternatives to meet the interim and final goals under the permit. In addition to improvement alternatives, the project team evaluated existing projects to identify opportunities to optimize the project performance under the 10-year 6-hour design storm. The optimization would include maximizing available storage or conveyance capacity of the project, and thereby reducing flooding upstream or downstream of the project.

The purpose of this technical memorandum is to document the existing projects evaluated under Task 3 for optimization, and present the modeling results for the optimization scenarios. The recommendations for project optimization will be included in the baseline scenario for identifying additional alternatives for the Combined Sewer System Improvement Plan (CSSIP).

2.0 Project Optimization

Table 1 below presents a list of existing projects in the combined sewer collection system where an opportunity was identified by the project team to optimize the project. Each of the project as existing and optimized condition were simulated in the H&H model for the 10-year 6-hour design storm, and the model results (hydraulic grade line or flooding results) were summarized in the last column.

Table 1: List of Existing Projects and Potential Project Optimization

Project No.	Project Identifier (Name)	Location (e.g. Nearest Intersection)	Optimization Opportunity
1	42nd Street Drainage Area Storage and Pump Station Improvements	42nd & R. Street	Delay in pumping out has no benefit for the 10-year storm, but delay can benefit the water quality issue by sending the flows to Sac Regional [i.e. pumping out after the flows at Sump 2 < 60 mgd]
2	21st Street Local Storage - Broadway to 2nd Ave	On 21st Street, Betn Broadway and 2nd Ave.	Removing the constriction at the outlet does not benefit the flooding. Lowering the flow-splits does show some benefit system-wide, lowering the inverts of pipes at just 1 of 2 locations may be more cost effective. [Send model IWC to Rich for additional review]
3	LAND PARK COMB SEWER RELIEF PROJECT	On 12th, 13th and 14th Streets - From Landpark to Sump2	No further optimization
4	UCD MED CENTER REGIONAL STORAGE	49th and V Street	Delay in pumping out has no benefit for the 10-year storm, but delay can benefit the water quality issue by sending the flows to Sac Regional [i.e. pumping out after the flows at Sump 2 < 60 mgd]
5	Tahoe Park/Broadway Parallel Sewer Project	On Broadway Street -from 60th St. to La Solidad Way	Increasing the (e) sluice gate opening to 18” and adding a new sluice gate (or pipe for outlet control) of 15” provides

Project No.	Project Identifier (Name)	Location (e.g. Nearest Intersection)	Optimization Opportunity
			the best benefits for optimizing u/s facilities and d/s flooding. Added a overflow gutter (720" x 6") near (e) sluice gate to allow the sheet flow to overflow to the next manhole, and remove restriction on the pipe. This street flooding better represents the HGL at that location through discussions with Rich.
6	East End Project Relief Sewer	Along 14th and 15th Street, between L and S Streets	Tie-in at L Street using a 24" pipe, and upsizing the existing 12" pipe provides the best benefits for overflows. Removing the d/s flow constriction (21" pipe section) does not provide any benefit.
7	U&S St. In-Line Storage Project	U Street between 6th and Sump1, S Street between 5th and 7th St.	Three scenarios considered: Replacing weirs with automated sluice gates provide 0.7 MG system-wide flood reduction. Increasing both weir widths to 10 ft, and same invert provide 0.3 MG system-wide flood reduction. Increasing the weir width (on 84") to 10 ft and lowering the invert to 2.0 ft, provides 0.2 MG system-wide flood reduction. [Ideal construction feasibility wise] Cost-benefit for all 3 options should be considered.
8	7th Street H-K Sewer Replacement	On 7th Street, between H and K Streets.	No further optimization
9	N Street Sewer, 25th to 29th	On 29th Street - from Capitol to O Street. Check location with Rich, there does not appear to be a inline storage on N Street from 25th to 29th Streets.	No benefit found for system-wide or local overflows. No further optimization.
10	R Street CSS Storage Project	On R St between 11th and 13th Streets, and 12th St between R and S Streets.	Increasing the weir invert by 1.5 feet (to capture the peak flow and keep capacity for downstream interceptor available) does not affect the downstream flooding.
11	5th Street Sewer Upsizing, P St to U St	On 5th Street, between P and U	Same as Project #7.

Project No.	Project Identifier (Name)	Location (e.g. Nearest Intersection)	Optimization Opportunity
		Streets.	
12	S St. Brick Sewer Replacement - 7th to 14th	On S Street, between 7th and 14th Streets.	No further optimization
13	S St. Brick Sewer Replacement - 14th to 17th	On S Street, between 14th and 15th Streets.	No further optimization

For the individual projects that showed benefit from optimization were grouped into three optimization scenarios, and simulated in the H&H model for the 10-year 6-hour design storm to identify the total performance benefits. The model network (August 2011 -Wet Area 6) was used as the baseline and optimization runs. The following projects were included into the three optimization scenarios:

- Project #2: Lowered weirs at two pipe junctions (i.e total 4 inverts) [To reduce flooding]
- Project #5: Added an additional sluice gate and control. Added a new street overflow channel and reduced the (e) sluice gate size [To reduce flooding]
- Project #6: Tie-in at upstream end [To reduce flooding]
- Project #7: Three different weir configurations [To reduce flooding]
    - P7_1: Removed weirs in model [will be replaced by sluice gates]
    - P7_2: Widened weirs, same invert
    - P7_3: Widened weirs, lower invert by 1 ft.
- Project #11: Same as Project #7.

The projects #2, #5, #6 and #11 were included in all three optimization scenarios. Three different versions of project #7 differentiate the three optimization scenarios. Table 2 below presents H&H model results and statistics for baseline and optimization scenarios.

Table 2: H&H Model Results for Optimization Scenarios

Model Statistics	Existing Scenario (Baseline)	Optimization Scenario #1 P7_1	Optimization Scenario #2 P7_2	Optimization Scenario #3 P7_3
Number of Pipes Surcharged (Out of 2,793)	2,689 (96.2%)	2,689 (96.2%)	2,689 (96.2%)	2,689 (96.2%)
Total Length of Pipes Surcharged (miles)	112.2	112.2	112.2	112.2
Total Volume of Flooding (MG)[1]	65.0	63.8	64.1	64.0
Total Outflow Volume to Street from System (MG) [1]	15.7	14.8	14.9	14.9
Total Ponding Volume (MG) due to Gully or	49.3	49.0	49.2	49.1

System Capacity [1]				
Total CSO Volume to River (MG)	88.0	89.4	88.8	89.2
Total Flows to Treatment (Sac Regional + CWTP)	146.0	145.2	145.3	145.3

Notes:
H&H model simulation run for 2 days for the 10-yr 6-hr design storm
[1]There are two components of flooding:
-Outflows to Street: combined sewage existing to the streets from the collection system due to downstream constriction.
-Street Ponding: storm water cannot enter the combined collection system because the drain inlet (i.e. gully in model) or the sewer/MH has limited capacity.

3.0 Summary and Next Steps

The H&H Model simulation results indicate that all optimization scenarios provide some level of performance benefits (i.e. reduction in flooding) as compared to the baseline scenario. The optimization scenario P7_1 has best performance for flooding benefits but would likely have a higher cost for replacing the weirs with sluice gates. Scenario P7_3 has the next best performance for flooding benefits, and have a reasonable cost for widening and lower the weirs. Based on these results, the projects under the optimization scenario P7_3 was deemed most appropriate for optimization and will be included as part of the CSSIP.

City of SACRAMENTO
Department of Utilities

Combined Sewer System (CSS) Improvement Plan Update – Phase 2

Task 7 – CSS Discharge Analysis

May 28, 2013

Version 5 (Final)

Table of Contents

1. INTRODUCTION 4
   1.1. BACKGROUND 4
   1.1. PURPOSE OF THE STUDY 4
   1.2. STUDY AREA 4
2. MODEL CALIBRATION 6
   2.1. MODEL BACKGROUND & AVAILABLE DATA 6
   2.2. MODEL INPUT AND ASSUMPTIONS 7
   2.3. MODEL RESULTS 10
3. TYPICAL YEAR DEVELOPMENT 12
   3.1. AVAILABLE RAINFALL DATA 13
   3.2. RAINFALL ANALYSIS 15
4. DISCHARGE FREQUENCY ANALYSIS 23
   4.1. BASELINE CONDITIONS 23
   4.2. SCENARIO I 26
5. CONCLUSIONS 30

List of Figures

Figure 1: Overview of the CSS System 5
Figure 2: Combined Wastewater Treatment Plant (CWTP)....7
Figure 3: Rain Gauges for 03/14/11 Event....9
Figure 4: Map of long-term NOAA Rainfall Gauges.... 13
Figure 5: Annual Rainfall at NOAA Rainfall Gauges.... 14
Figure 6: Average Monthly Rainfall at Sacramento Downtown Gauge.... 14
Figure 7: Running 10-year averages for various annual statistics.... 17
Figure 8: Monthly Rainfall Depth for 2008-2009 compared to Annual Averages.... 19
Figure 9: Monthly Rainfall Depth for Typicalized 2008-2009 Compared to Annual Averages.... 20
Figure 10: Comparison of Original and Adjusted January 2009 Storms.... 21
Figure 11: S Street Project 26
Figure 12: P Street and 7th Street Projects 27
Figure 13: Oak Park Storage Project 28

List of Tables

Table 1: CSO Events 2010-2012 Prioritized for Modeling Output 6
Table 2: Rain Gauge Data Statistics for 03/14/11 Event 10
Table 3: Model Comparison for Four Storm Events 11
Table 4: 1-Year Depths for Varying Durations as per Atlas 14.... 15
Table 5: Example Scoring Calculation 18

Table 6: Calculated Scores for 2001-2010 18
Table 7: Comparison of Original and Modified 2008-2009 Rainfall Season 20
Table 8: Storms in Developed Typical Year 22
Table 9: Baseline Discharge Frequency and Volume in a Typical Year 25
Table 10: Baseline Sanitary Flow Component in a Typical Year 25
Table 11: Project Scenario I Discharge Frequency and Volume in a Typical Year 29
Table 12: Scenario 1 - Sanitary Flow Component in a Typical Year 29

1. INTRODUCTION

1.1. BACKGROUND

In June 1990, the Regional Water Quality Control Board (RWQCB) issued a Cease and Desist Order (CDO) requiring the City to reduce Combined Sewer Outflows (Outflows) from the Combined Sewer System (CSS) into City streets and properties. After exhaustive studies and rigorous benefit/cost analysis, the City Council adopted a Combined Sewer System Improvement Plan (CSSIP) in 1995. The CSSIP met the requirements of the CDO, the EPA's Combined Sewer Overflow Control Policy, and the City's adopted goals for the storm drainage system. That same year, the RWQCB approved the Improvement Plan, rescinded the CDO, and issued a National Pollutant Discharge Elimination System (NPDES) permit that mandated implementation of the Improvement Plan and protection of the water quality in the Sacramento River.

On 28 January 2010, the California RWQCB issued a revised NPDES permit that required the City to continue the implementation of the Long-Term Control Plan with interim goals to be met as progress is made towards the final goal of minimizing street flooding during a 10-year storm event and to prevent structure flooding during the 100-year storm event.

The intent of this study is to evaluate if CSS discharges from the City of Sacramento are compliant with the NPDES permit description of discharges. The study supports the NPDES water quality analysis by characterizing the frequency and volume of treated and untreated CSS discharges for the baseline conditions, and evaluating the impact of any proposed CSSIP updates on the discharges to the river.

1.1. PURPOSE OF THE STUDY

The purpose of this study is to analyze CSS discharges to the Sacramento River for the baseline and proposed conditions within the range of the LTCP using the latest hydrologic and hydraulic (H&H) numerical model representing the City of Sacramento's CSS. This study will validate the H&H model using monitoring data for CSS discharges in recent years, and calibrate the H&H model if necessary. Using long term historical rainfall records, this study developed a typical year rainfall dataset to establish the baseline conditions and compare the impact of proposed CSSIP projects with the baseline conditions. Furthermore, the study will evaluate the proposed CSS outflow and CSS discharge reduction performance of the CSSIP (i.e. projects that are already in design or planned), and work with City of Sacramento's Department of Utilities (DOU) on developing projects for meeting future goals for a range of LTCP conditions. The purpose of this technical memorandum is to document the approach, methodology and outcome of the study.

1.2. STUDY AREA

The City of Sacramento owns and operates a CSS that conveys residential and commercial wastewater and storm water runoff from 7,510 acres (approximately 334 miles of sewer pipe) in downtown Sacramento, East Sacramento, and Land Park areas. The CSS serves approximately 300,000 people. The City also owns and operates a separate sanitary sewer system that conveys

residential and commercial wastewater from 3,690 acres (approximately 566 miles of sewer pipe) from parts of the City surrounding the CSS to the north, east, and south. The CSS includes four key facilities to manage the collected flow: Sumps 1/1A, Sumps 2/2A, Pioneer Reservoir, and Combined Wastewater Treatment Plant (CWTP). Sumps 1/1A and 2/2A pump up to 60 mgd of flows to Sacramento Regional County Sanitation District's regional wastewater treatment plant (SRWTP) via the regional interceptor. Pioneer Reservoir and CWTP provide additional storage as well as primary treatment and disinfection of combined sewage prior to discharging to the river. The CSS has six discharge points along Sacramento River.

Figure 1 shows the model representation of the CSS, pumping and treatment facilities and six discharge points. Discharge points 002, 003 and 006 receive primary treatment and disinfection prior to discharge into Sacramento River and flows from these outfalls are referred as treated CSS discharges. Discharge points 004 and 005 are discharges of untreated flows from Sump 2. Discharge point 007 releases untreated flows from Sump 1 during emergency conditions. Flows from outfalls 004, 005, and 007 are referred to as untreated CSS discharges.

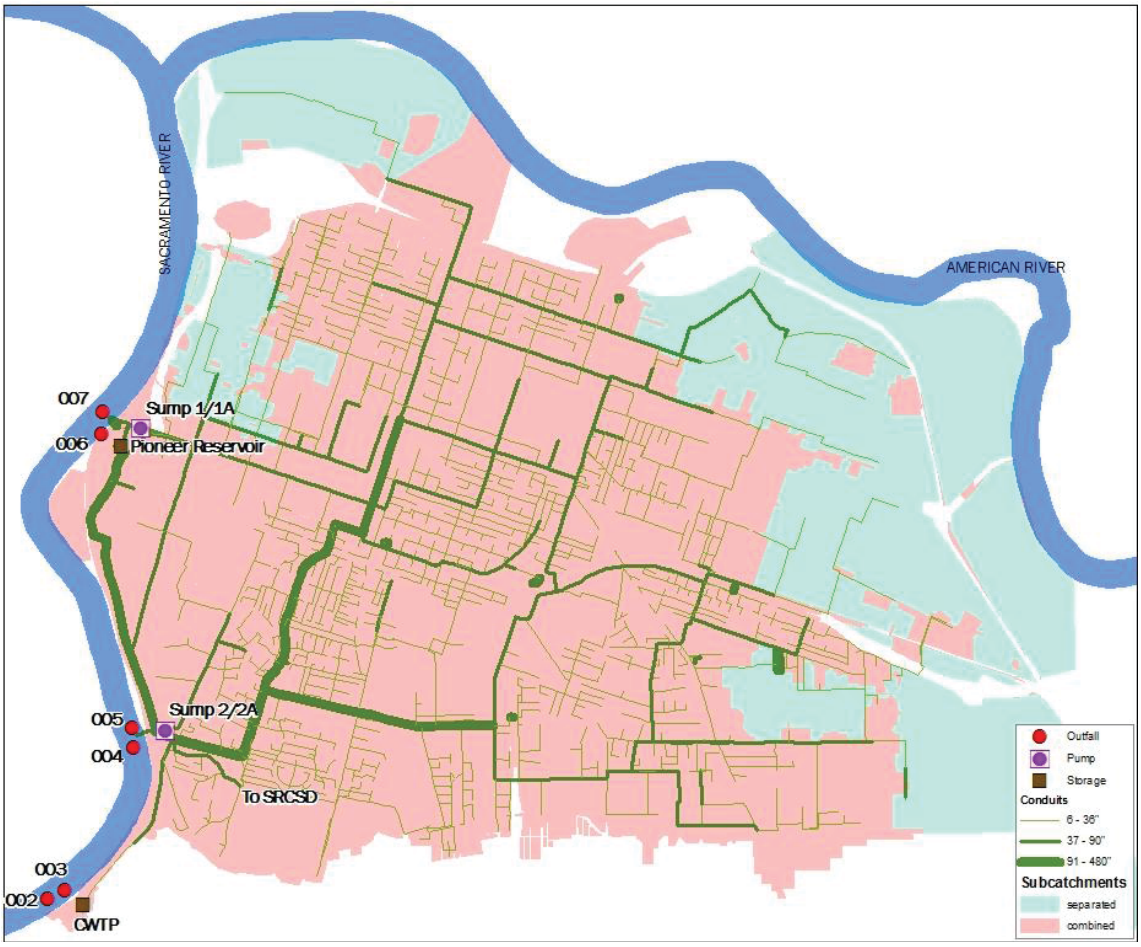


Figure 1: Overview of the CSS System

2. MODEL CALIBRATION

2.1. MODEL BACKGROUND & AVAILABLE DATA

In 2008, as part of Phase 1 of the CCSIP, the City of Sacramento DOU updated the hydrologic and hydraulic (H&H) model of its CSS in 2008. The model was developed using InfoWorks CS version 10.0 and calibrated using intensive flow and rainfall monitoring data from the 2008-2009 wet weather season. The H&H model includes all major large pipes within the CSS and a detailed representation of the facilities including pump stations, storage and CSS discharge locations. The model also includes the operational control logic for the hydraulic representation of the different treatment paths during a wet weather event. Since 2008, DOU has been refining the model to add details in the six wet areas of the CSS to simulate the flooding events and plan projects to address the flooding issues. The model hydrology and hydraulics was calibrated using 2008-2009 monitoring data at several in-system depth and velocity monitoring locations.

Recently, the DOU provided a summary of monitoring data collected at two locations in recent wet weather events. Table 1 documents monitoring details at two locations for four discharge events. The monitoring site Pioneer refers to outfall 006 and CWTP refers to outfall 002.

Table 1: Discharge Events 2010-2012 Prioritized for Modeling Output

Priority	Date	Site Name	24 hr Rain (in.)	48 hr Rain (in.)	Discharge Start	Discharge End	Discharge Volume (MG)	Note
High	12/19/10	Pioneer	0.97	1.86	8:07	13:25	57	Expanded sampling
High	3/14/11	CWTP	1.51	1.51	2:30	6:00	25	Expanded sampling
Medium	3/14/11	Pioneer	1.51	1.51	2:55	6:30	35	
High	1/21/12	Pioneer	1.33	1.48	0:40	3:05	24.69	Expanded sampling
High	1/23/12	Pioneer	1.02	1.02	9:20	11:27	16.35	
Note: N/A data not available for 10/24/10								

2.2. MODEL INPUT AND ASSUMPTIONS

Model Network: Since the development of the H&H model in InfoWorks CS in 2008, DOU has been continuously refining the model to add details (manholes, pipes and sub-catchments) to the model representation in the six wet areas. The purpose of this refinement is to enable better analysis of design storms for flooding analysis in the six wet areas, and develop improvement projects. The last model refinement released by DOU was in July 2012.

At the request of the DOU, the model was further updated for this analysis to incorporate the CWTP sedimentation basins, 84" interceptor from Sump 2 to the CWTP sedimentation basins, and CWTP's effluent pipe leading to outfall 002. The additional storage was added to the hydrologic and hydraulic model as this storage volume was important for model calibration of flows to outfall 002 and calculating CSS discharge frequency and volume for a typical year.

Figure 2 shows the location of the storage added to the model.

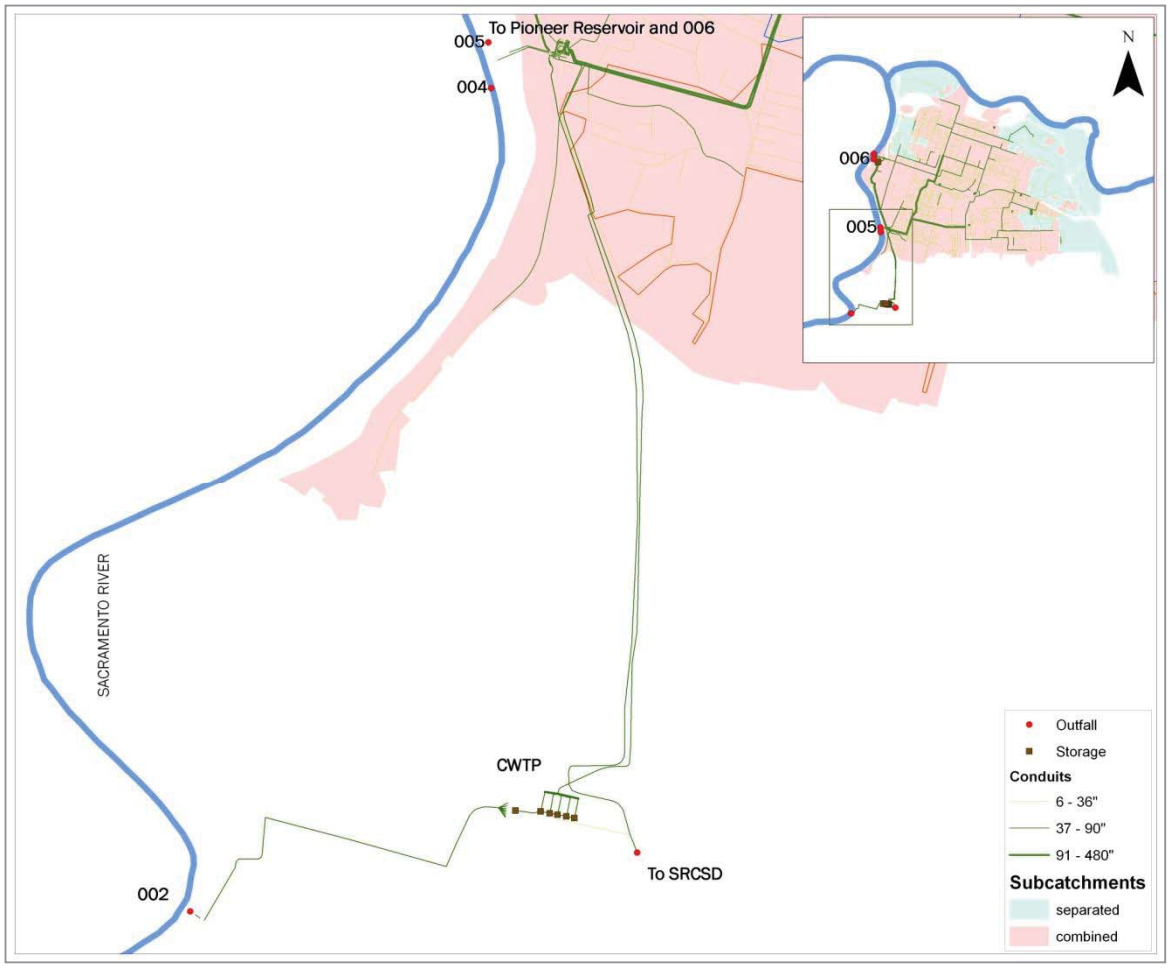


Figure 2: Combined Wastewater Treatment Plant (CWTP)

As-built drawings and pump information provided by DOU were used to add facilities to the modeled system. Flows are pumped from Sump 2 to CWTP via an 84" interceptor. CWTP itself is a primary

treatment facility that consists of five sedimentation basis with associated sludge lines and a sludge pump which sends flows to the Sacramento Regional County Sanitation District (SRCSD). The final facility added was the effluent line and pump station from CWTP to outfall 002. Outfall 003 was not added in the model as this is an emergency outfall and not used during regular operations.

CWTP sedimentation basins provide a total storage of 7.1 MG, in addition to 2.1 MG of storage provided by the CWTP interceptor. A sludge line from CWTP to SRCSD has a total capacity of 6 MGD, and is represented in the model using a single pump. The effluent pump station from CWTP to outfall 002 consists of 10 pumps with a design capacity of 130 MGD. The simplified pump representation was set in the model with on and off set points as provided by DOU.

Operational Controls and Logic: The InfoWorks CS model provides a numerical computation module called RTC (Real Time Control). This RTC module defines the operational controls and logic for operation of different facilities in the CSS. The model RTC module was developed in 2008 during model development and calibration, and updated in 2010 based on meetings with City of Sacramento's operations staff. This operational logic and control was further modified in this study to maximize the available storage in the system and mimic the operations as defined by the City's NPDES permit[1].

The City's NPDES permit describes the sequence of CSS flow routing paths differently than as setup in the 2008 version model which was based on input from City of Sacramento's operations staff. Upon adding CWTP and its outfall 002 to the model, it became necessary to modify the RTC to first maximize the storage at CWTP and Pioneer Reservoir before discharging at CSS outfalls. Once this storage was maximized, flows could continue to be sent to CWTP and Pioneer to be discharged through the associated outfalls (002 and 006 respectively). DOU stated that storage would be maximized first and in updating the RTC to do so, the RTC now matches the City's permit. The updated RTC module has the following steps:

- The first 60 MGD of CSS flows are pumped to SRCSD.
- The next 130 MGD of CSS flows are pumped to CWTP, until storage is fully utilized at CWTP and in the interceptor (9.2 MG), then flows to CWTP stop.
- Remaining flows are routed to the Pioneer Reservoir which has a storage capacity of 23 MG.
- After the storage capacity at Pioneer Reservoir is maximized, the CSS flows discharge into the Sacramento River at outfall 006.
- Once overflows at outfall 006 peak at 350 MGD, flows are sent to CWTP and CSS flows discharge into the Sacramento River at outfall 002.
- If flows at outfalls 006 and 002 are maximized (at 350 MGD and 130 MGD respectively) and the wet well at Sump 2 is above 5 feet, CSS flows discharge into the Sacramento River at outfall 004.

Rainfall Data: The historical rainfall data for the storms listed in **Table 1** above was an input to the H&H model simulation.

[1]ORDER NO. R5-2010-0004, NPDES NO. CA0079111

For the four selected storm events, DOU provided rainfall datasets at a 5-minute interval for eight rain gauges covering the CSS study area. **Figure 3** below shows the eight rain gauges recording rainfall data for these events, whereas **Table 2** tabulates the storm depth, peak intensity, and storm duration statistics for an example event (the 03/14/2011 event). The statistics on rain gauge data for this event shows a significant spatial variation in peak intensity and the total depth.

Accurate representation of rainfall, including spatial and temporal variation as an input for model simulation is important for verifying model performance to any monitoring data. All eight rain gauges are an invaluable input for model calibration and verification with sampling data.

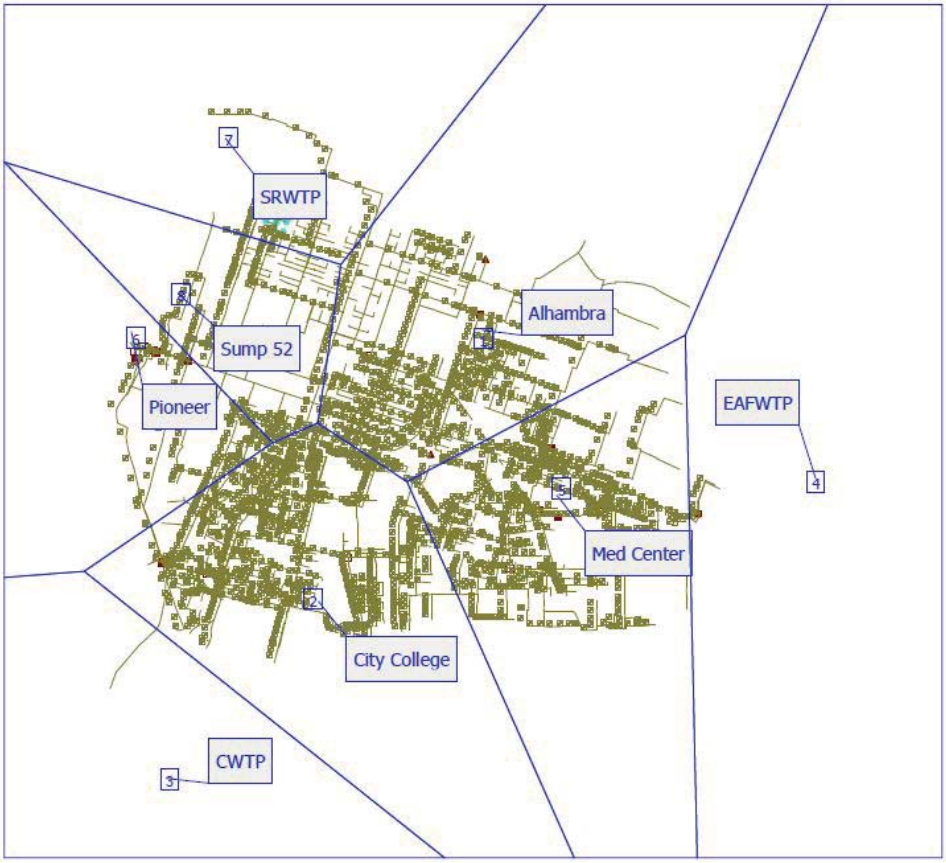


Figure 3: Rain Gauges for 03/14/11 Event

Table 2: Rain Gauge Data Statistics for 03/14/11 Event

Rain Gauge	Storm Depth (in.)	Peak Intensity (in/hr)	Storm Duration (hrs)
1 - Alhambra	1.32	1.68	15.4
2 – City College	1.46	1.68	15.4
3 – CWTP	1.45	1.80	15.1
4 – EAFWTP	1.58	2.04	15.2
5 – Med Center	1.57	1.44	15.3
6 – Pioneer	0.91	1.20	15.3
7 – SRWTP	0.90	0.60	15.4
8 – Sump 52	1.18	1.44	15.4

2.3. MODEL RESULTS

DOU also provided time series datasets of flows to key facilities, including recorded data at Sump 2 for flows pumped to SRCSD and CWTP and flows at the outfall weir at Pioneer Reservoir. These time series datasets enable refining the model to improve the correlation to observed data.

Using the model inputs and assumptions described in the section above, the model was run by modifying Sump 2 operations such that flows going to SRCSD and CWTP in the model match exactly as recorded in the SCADA system. Table 3 lists the four discharge events for which sampling data were collected by the City, and the modeled discharge at corresponding locations. Appendix A provides a graphical representation of simulated flows to key facilities during the storm event compared against observed flows. It should be noted from the observed data in Appendix A, that the Sump 2 flows to SRCSD and CWTP exceed the permit defined peak flows of 60 MGD and 130 MGD respectively for short periods of time.

Table 3: Model Comparison for Four Storm Events

Date	Site Name	Observed Discharge				Modeled Discharge			
		Start Time	End Time	Duration (hrs)	Volume (MG)	Start Time	End Time	Duration (hrs)	Volume (MG)
12/19/2010	CWTP (002)	-	-	0	0	7:10	9:10	2	0.9
	Sump 2 (004)	-	-	0	0	7:00	9:15	2.3	7.6
	Pioneer (006)	8:07	13:25	5.3	57	6:30	12:30	6	80.8
	Sump2 to SRCSD	0:00	23:59	48	115.3	0:00	23:59	48	108.0
	CWTP Sludge Line	-	-	-	-	0:00	23:59	48	11.0
TOTAL (w/o flows from CWTP Sludge Line to SRCSD)					172.3				197.4
3/14/2011	CWTP (002)	2:30	6:00	3.5	25	2:45	14:45	12	10.4
	Sump 2 (004)	-	-	0	0	–	–	0	0
	Pioneer (006)	2:55	6:30	3.6	35	2:10	16:00	13.5	48.9
	Sump2 to SRCSD	0:00	23:59	48	83.8	0:00	23:59	48	78.7
	CWTP Sludge Line	-	-	-	-	0:00	23:59	48	11.4
TOTAL (w/o flows from CWTP Sludge Line to SRCSD)					143.8				138
1/21/2012	CWTP (002)	-	-	0	0	–	–	0	0
	Sump 2 (004)	-	-	0	0	–	–	0	0
	Pioneer (006)	0:40	3:05	2.4	24.7	23:40	4:00	4.3	44.9
	Sump2 to SRCSD	0:00	23:59	48	95.6	0:00	23:59	48	88.3
	CWTP Sludge Line	-	-	-	-	0:00	23:59	48	6.9
TOTAL (w/o flows from CWTP Sludge Line to SRCSD)					120.3				133.2
1/23/2012	CWTP (002)	-	-	0	0	9:00	10:00	1	0.18
	Sump 2 (004)	-	-	0	0	–	–	0	0
	Pioneer (006)	9:20	11:27	2.1	16.3	5:00	16:40	11.7	33.2
	Sump2 to SRCSD	0:00	23:59	48	89.3	0:00	23:59	48	87.9
	CWTP Sludge Line	-	-	-	-	0:00	23:59	48	9.7
TOTAL (w/o flows from CWTP Sludge Line to SRCSD)					105.6				121.3

The model over predicts for 3 out of 4 events within 15% and under predicts for 1 out 4 events within 10% of the observed volume. The model calibration is within the calibration and verification criteria defined by the Wastewater Planning Users Group (WaPUG)[2]. The model calibration is considered good for simulating CSS performance and planning improvements.

System operations (i.e. how different facilities are operated to maximize storage) is a key factor in CSS discharges during wet weather events. Any operational strategy different than as defined in the NPDES permit and documented in the TM can provide different discharge frequency and volume at the outfalls. In order to provide data that can be used for continued model verification it is recommended that DOU collect in system level and velocity monitoring data every few years and perform model calibration and verification to maintain a high level of confidence in the model. Areas with most changes in land uses should be focused for model calibration as change in land use can affect resulting runoff from the area.

3. TYPICAL YEAR DEVELOPMENT

One tool that can be used to analyze a collection system is a "typical" rainfall year which, when used in conjunction with a model, can assess how a collection system will perform on an annual basis. A typical rainfall year can be used to estimate average annual overflow frequency and volume in Sacramento as well as provide other annual performance statistics.

This section explains the methodology used to select the 2008-2009 rainfall season as the period to analyze Sacramento's collection system. Analyses were conducted using the hourly rainfall recorded at the "Sacramento 5 ESE CA US" National Weather Service (NWS) rain gauge located in downtown Sacramento (also called the Downtown gauge). This gauge has a period of record from 1936 through present day. Rainfall statistics were developed for each storm event including total storm depth, peak intensity, and duration. A minimum inter-event time of 24 hours was used to define storm events. Annual rainfall statistics were developed including total yearly rainfall depth and the number of storms in specified intensity and depth ranges. Storm depth is important for CSO control measures that involve storage, while peak intensity controls the occurrence of individual CSO activations.

The procedure to develop the typical period involved selecting a year that closely resembles the long-term average in terms of number and distribution of storms. A scoring system was developed to rank years based on how well the year matched the average for the period of record. The most typical year that has occurred in the past 10 years is 2008. Though 2008 statistics are relatively close to annual averages, there were too few storms with large intensities and depths. A single storm was added to "typicalize" the rainfall and make the year match long term averages more closely.
The exact procedures used to select the typical rainfall season are explained in more detail in the following subsections.

[2] WaPUG Code of Practice for the Hydraulic Modeling of Sewer Systems Version 3.001

3.1. AVAILABLE RAINFALL DATA

The rainfall used for this analysis comes from records kept by NOAA (the National Oceanic and Atmospheric Administration). NOAA identifies two rain gauges within the city that contain long-term hourly rainfall data. The two gauges are:

- SACRAMENTO 5 ESE CA US (DOWNTOWN) – period of record from 1936-Current
- SACRAMENTO EXECUTIVE AIRPORT CA US – period of record from 1948-Current

The location of these two gauges can be found in **Figure 4**. Both gauges were moved around to different locations over the long term period; however the datasets correlate very well in terms of annual total basis. A graph showing annual rainfall depths can be found in **Figure 5**. For the purposes of the analysis presented in this report, a rainfall season has been defined to start on October 1st and end September 30th the following year. This is to coincide with reporting requirements for the city's NPDES Permit.

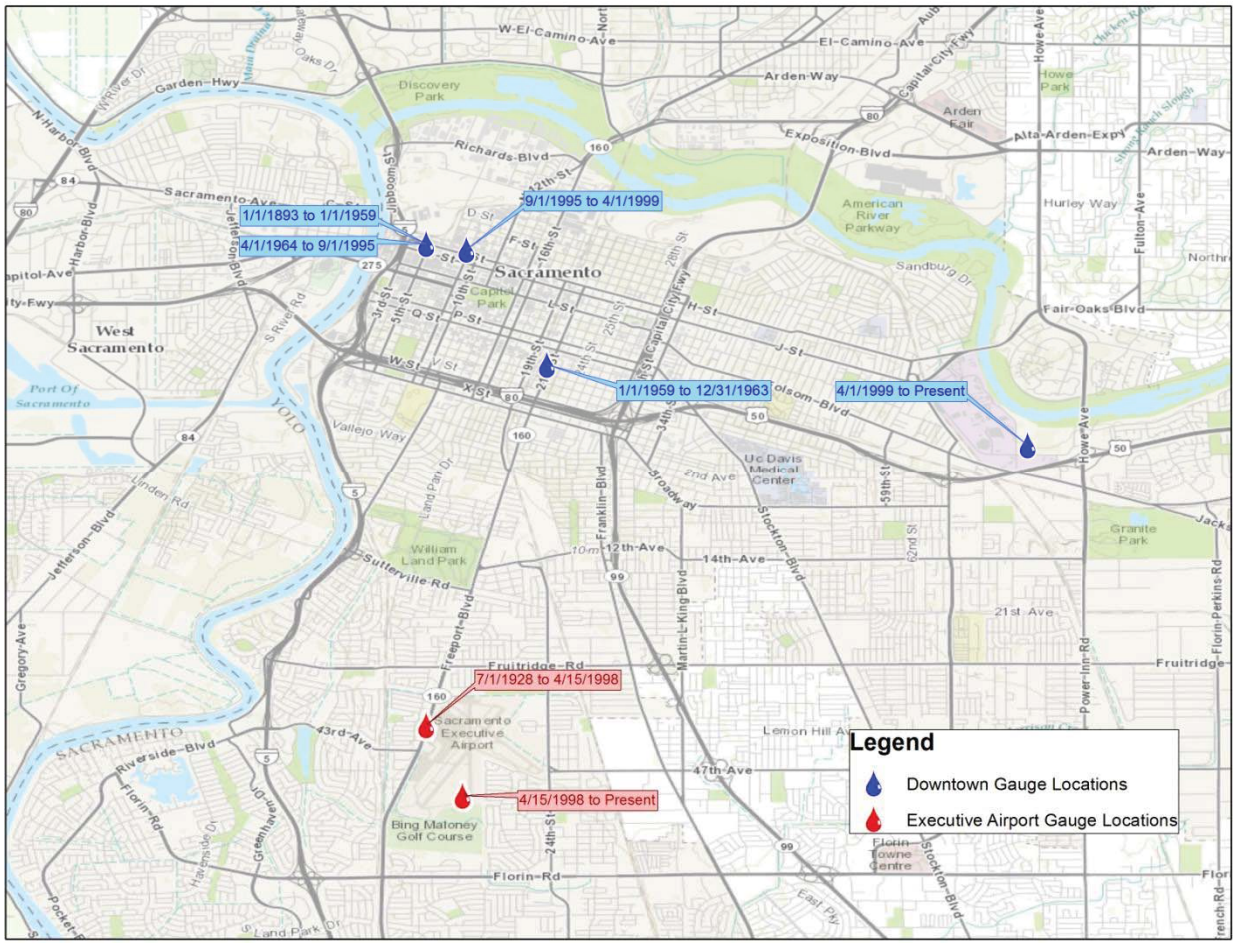


Figure 4: Map of long-term NOAA Rainfall Gauges

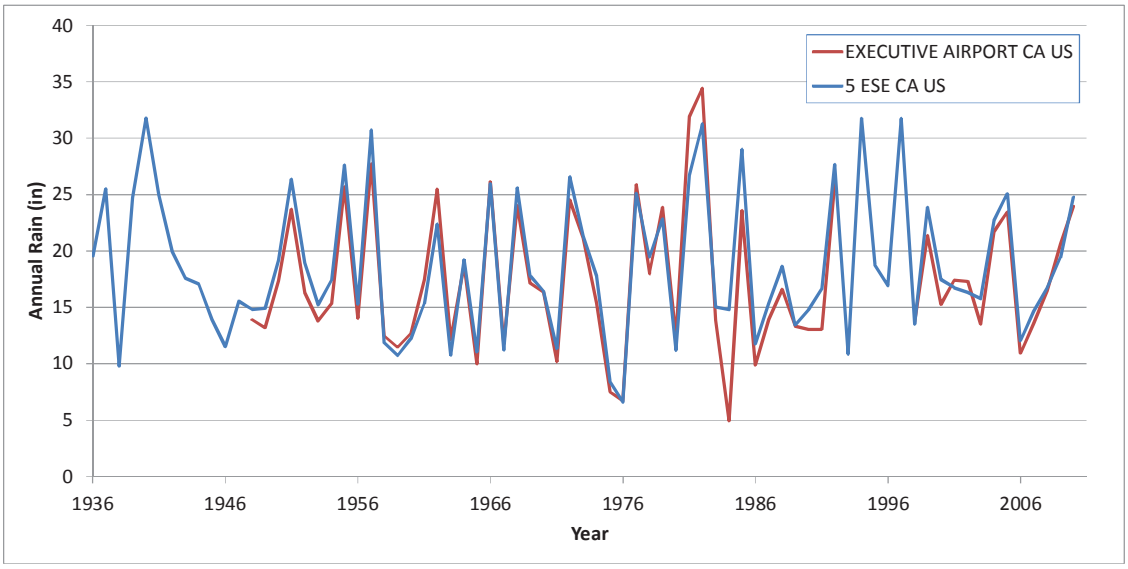


Figure 5: Annual Rainfall at NOAA Rainfall Gauges

Of the two gauges, the Downtown gauge has a longer record and is more complete. For years where both gauges contain data, the Executive Airport gauge shows annual rainfall totals very similar to that of the Downtown gauge. This indicates that even though rainfall may vary from storm to storm spatially, on an annual total basis spatial variation does not play a factor within the city. For this reason, the analysis presented in the rest of this section focuses exclusively on rainfall recorded at the Downtown gauge.

Rainfall in Sacramento shows strong seasonal characteristics, with most precipitation occurring during the winter months. **Figure 6** shows average monthly rainfall within Sacramento for the Downtown gauge.

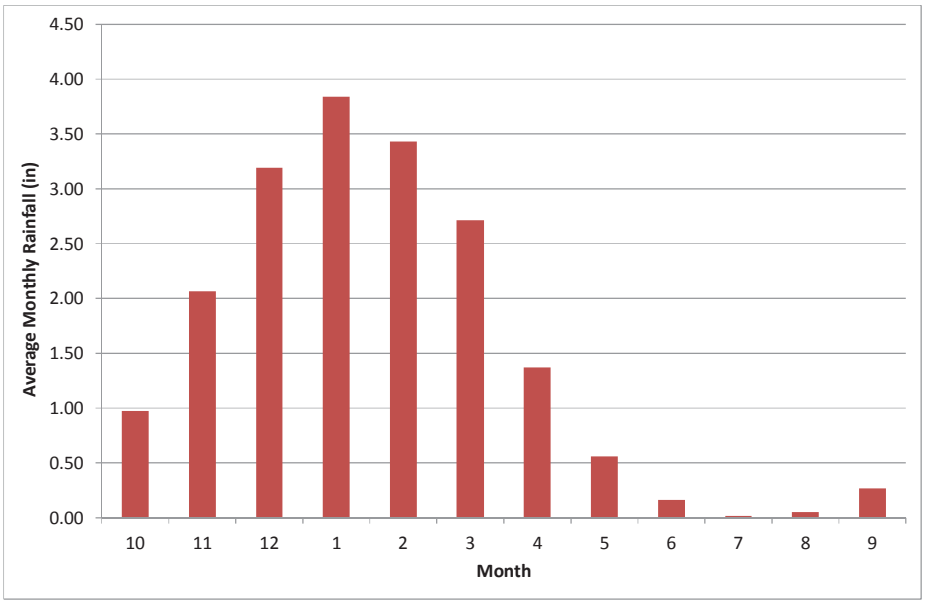


Figure 6: Average Monthly Rainfall at Sacramento Downtown Gauge

3.2. RAINFALL ANALYSIS

For each gage, rainfall events were identified assuming a minimum inter-event time of 24 hours. This means rainfall had to be separated by at least 24 hours to be considered a separate storm event. For each storm event total depth, duration, and peak intensity were calculated.
Each storm was categorized based on how the event compared to the 1-year rainfall intensity-duration-frequency (IDF) curve as cited by NOAA Atlas 14, Volume 6 (2012). Atlas 14 provides precipitation frequency point estimates for the state of California and is the successor to TP-40. Frequency estimates for 5-minute through 60-day durations are published in Atlas 14, at average recurrence intervals of 1-year through 1,000-year. The hourly peak intensity for each even was compared to the 1-year, 1-hour storm depth of 0.40 inches. The total depth for the storm was compared to the 1-year depth for the duration that was closest to the actual duration of the storm. For example, a 13 hour long storm would be compared to the 1-year, 12-hour depth of 1.3 inches.

Table 4 shows the rainfall depths for the 1-year IDF curve from Atlas 14.

Table 4: 1-Year Depths for Varying Durations as per Atlas 14

Duration	Depth (inches)
5 min	0.12
10 min	0.17
15 min	0.21
30 min	0.29
60 min	0.40
2 hr	0.60
3 hr	0.74
6 hr	1.00
12 hr	1.30
24 hr	1.69
48 hr	2.20
3 day	2.54
4 day	2.81

For each year, the following statistics were calculated

- Total number of storms
- Total annual rainfall depth
- Number of storms with intensities:
  - Larger than a 1 year storm
  - 75-100% of a 1 year storm
  - 50-75% of a 1 year storm
- Number of storms with total depths:
  - Larger than a 1 year storm
  - 75-100% of a 1 year storm
  - 50-75% of a 1 year storm

Smaller storm categories were not considered because most overflows do not occur during smaller storms according to Sacramento's overflow reporting. On average, 3.4 activations occurred each year between 2001 and 2007.

The annual average of each statistic over the period of record was calculated for the gauge. One consideration is the period of record that should be used for calculating annual averages. Climate change can change rainfall characteristics over a long period of time. Therefore, examining the entire rainfall record for an increasing or decreasing trend is important. If rainfall depths or intensities have been changing, then calculating an average for the entire period of record may not reflect the current average rainfall; calculating averages for just the past 20 or 30 years may be more appropriate in this case. Rainfall can vary from year to year substantially due to numerous factors, such as the occurrence of El Nino. For this reason, examining running averages is more useful for determining trends in data. **Figure 7** shows a running 10-year average for various annual statistics.

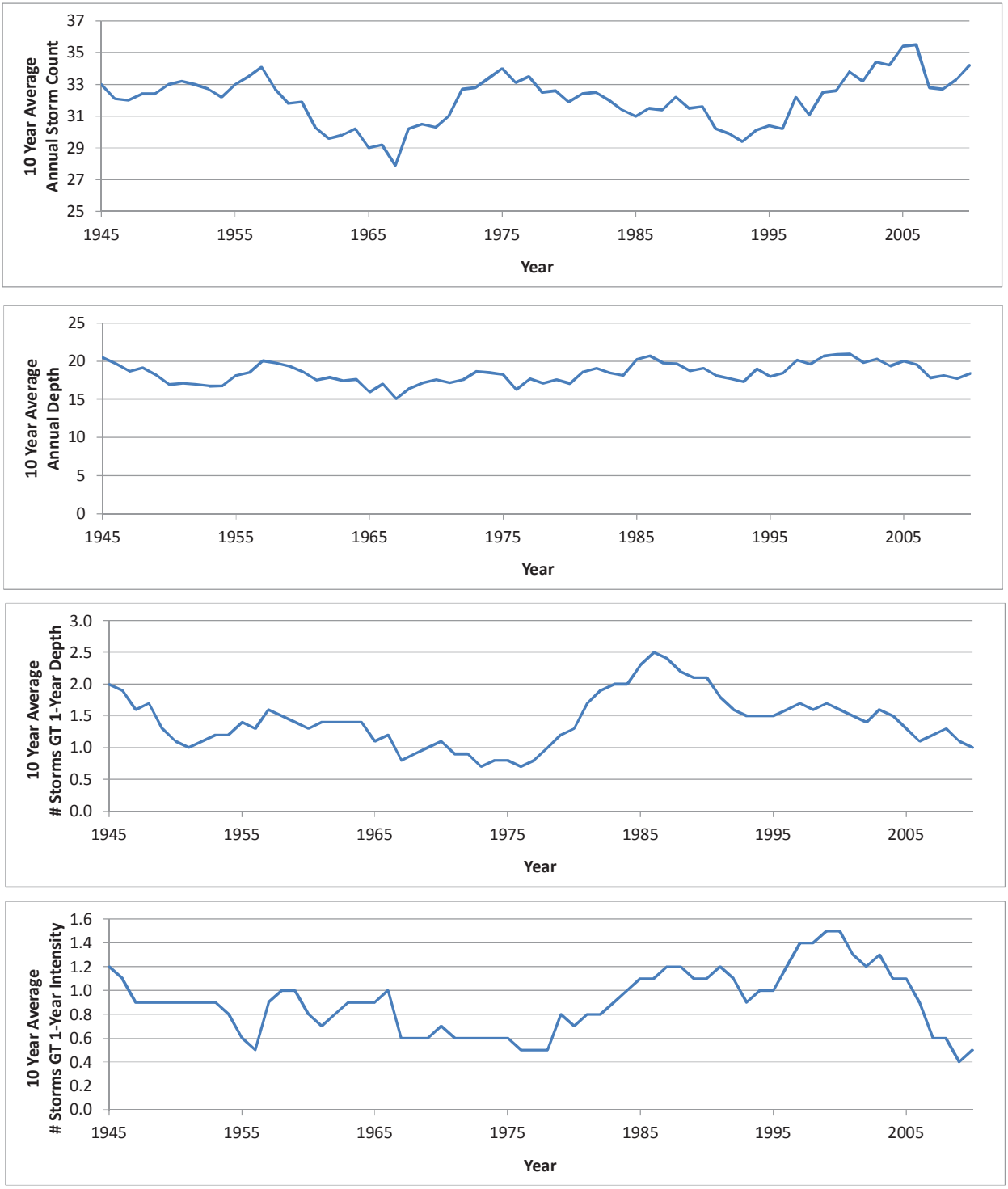


Figure 7: Running 10-year averages for various annual statistics

The graphs shown in **Figure 7** show a rainfall record that has been fairly stationary over the period of record. Therefore the entire period of record was used to calculate annual average rainfall statistics.

A scoring system was developed to characterize how close each year was to the overall annual average. The "score" for each year represents the total number of standard deviations away from average the year is in each category. Therefore a low score means the year is closer to the overall annual average compared to a year with a higher score. A table illustrating this scoring system for the year 2010 can be found in **Table 5**. The scores calculated for the past 10 years can be found in **Table 6**.

Table 5: Example Scoring Calculation

Statistic	# Storms	Total Depth	Peak intensity compared to 1-year storm			Total depth compared to 1-year storm		
			> 100%	75-100%	50-75%	> 100%	75-100%	50-75%
Annual Average (1936-2010)	32.1	18.6	0.9	1.6	3.9	1.4	1.8	4.0
Standard Deviation (1936-2010)	6.3	6.3	0.9	1.4	1.8	1.2	1.6	2.0
2010	37	24.8	1.0	3	5	0	6	3
# of Std Dev away from average	0.8	1.0	0.1	1.0	0.6	1.2	2.7	0.5
						TOTAL SCORE:		7.8

Table 6: Calculated Scores for 2001-2010

Year	# Storms	Total Depth	Peak intensity compared to 1-year storm			Total depth compared to 1-year storm			Total Scores
			> 100%	75-100%	50-75%	> 100%	75-100%	50-75%	
2001	33	16.8	0	1	5	0	3	5	5.0
2002	36	16.3	0	1	6	1	1	5	4.9
2003	37	15.8	1	1	6	2	2	1	5.1
2004	38	22.7	0	3	4	1	2	5	4.6
2005	42	25.1	1	0	8	1	3	5	7.8
2006	28	12.0	0	1	3	1	0	3	5.6
2007	25	14.7	0	3	2	2	1	2	6.7
2008	28	16.7	1	1	4	1	1	5	3.0
2009	38	19.4	1	0	5	1	3	4	4.1
2010	37	24.8	1	3	5	0	6	3	7.8
Annual Average (1936-2010)	32.15	18.6	0.89	1.65	3.95	1.45	1.78	3.96	--
Standard Deviation (1936-2010)	6.33	6.3	0.94	1.42	1.79	1.18	1.56	1.96	--

For the past 10 years, the 2008-2009 rainfall season is the most typical. The 2008-2009 rainfall season was the third most typical year for the entire period of record; only the years 1964 and 1942

provide a better match. A more recent year is considered preferable, as more records are available for more recent data. Based on engineering judgment, an additional storm event was added to the 2008-2009 rainfall to make it more typical.

Table 7 shows the 2008-2009 rainfall period contains too few storms in the 75%-100% range compared to the overall average. It is important to contain enough larger storms because these drive the occurrence of overflow activations.

To base where the storm should be inserted, a graph showing monthly rainfall compared to average monthly rainfall was examined. This graph is shown in Figure 8. The graph shows that there is less rain in December and January compared to the average for the period of record.

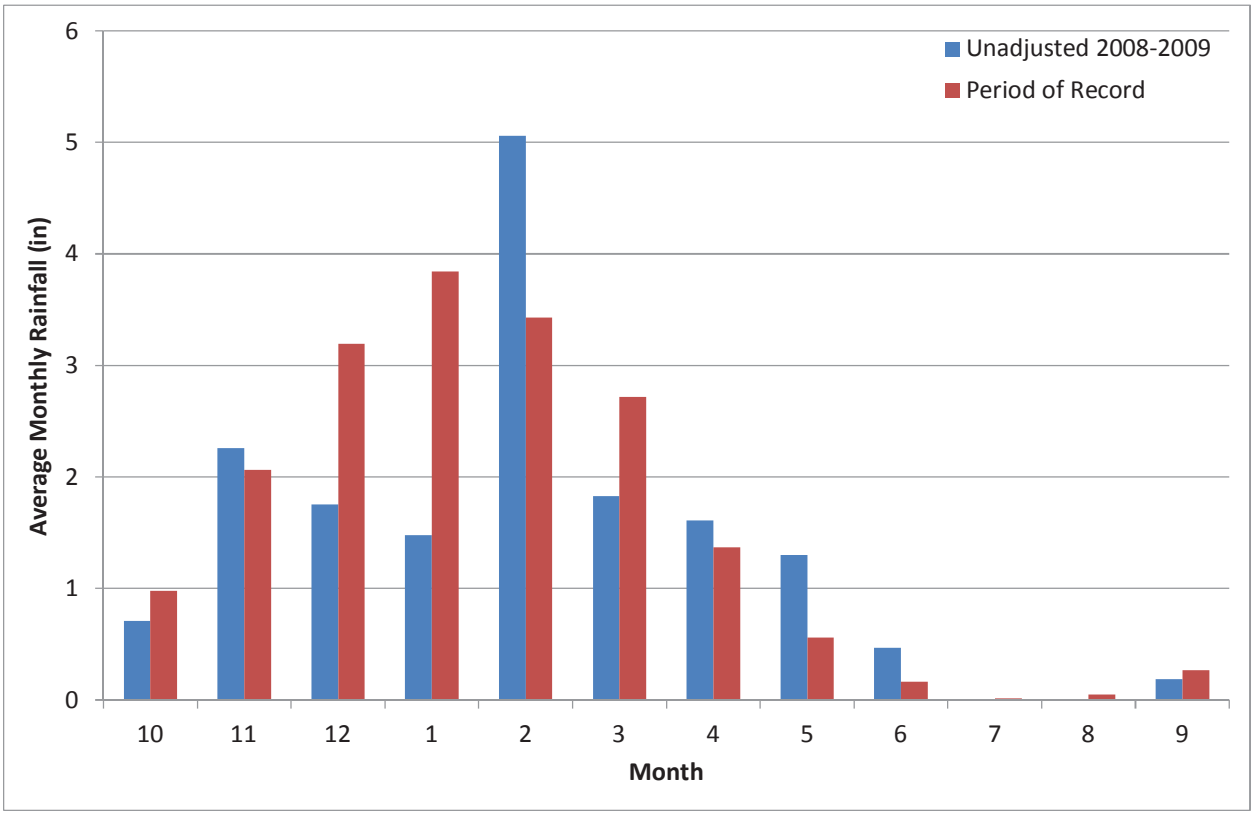


Figure 8: Monthly Rainfall Depth for 2008-2009 compared to Annual Averages

The graph shows that there is less rain in December and January than is typical for an average rainy season. The 04/12/2003 storm was selected to be inserted into the rainfall period during these months. The storm was selected because it is recent and has depth and intensity ranges in the 75%-100% category. A comparison of the statistics for the original and typicalized 2008 rainfall season can be found in Table 7.

Table 7: Comparison of Original and Modified 2008-2009 Rainfall Season

Year	# Storms	Total Depth	Peak intensity compared to 1-year storm			Total depth compared to 1-year storm			Total Scores
			> 100%	75-100%	50-75%	> 100%	75-100%	50-75%	
Original 2008	28	16.7	1	1	4	1	1	5	3.0
Typicalized 2008	29	18.1	1	2	4	1	2	5	2.0
Annual Average (1936-2010)	32.15	18.6	0.89	1.65	3.95	1.45	1.78	3.96	--
Standard Deviation (1936-2010)	6.33	6.3	0.94	1.42	1.79	1.18	1.56	1.96	--

The modified monthly rainfall for the 2008 period can be found in **Figure 9.**

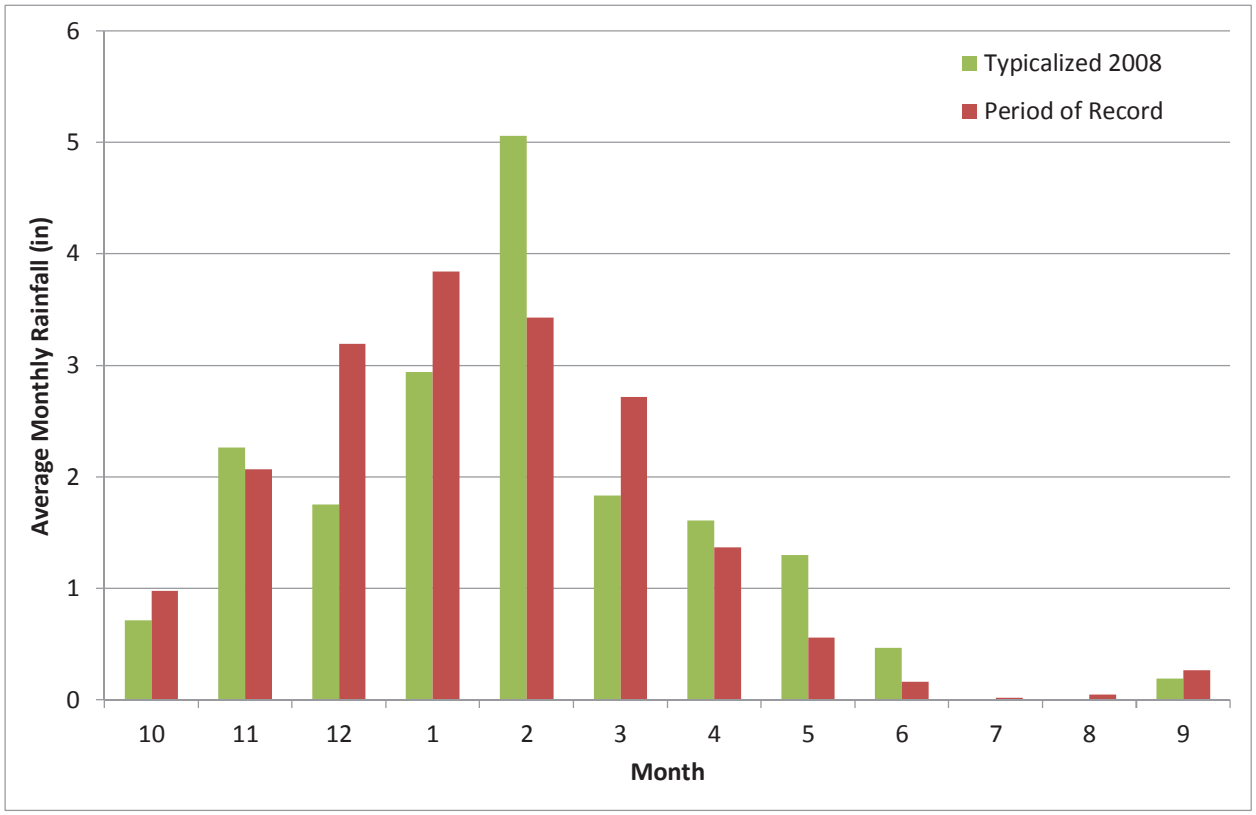


Figure 9: Monthly Rainfall Depth for Typicalized 2008-2009 Compared to Annual Averages

A comparison of the original and modified January 2009 rainfall is shown in **Figure 10** and rainfall statistics for the Typical Year can be seen in **Table 8**.

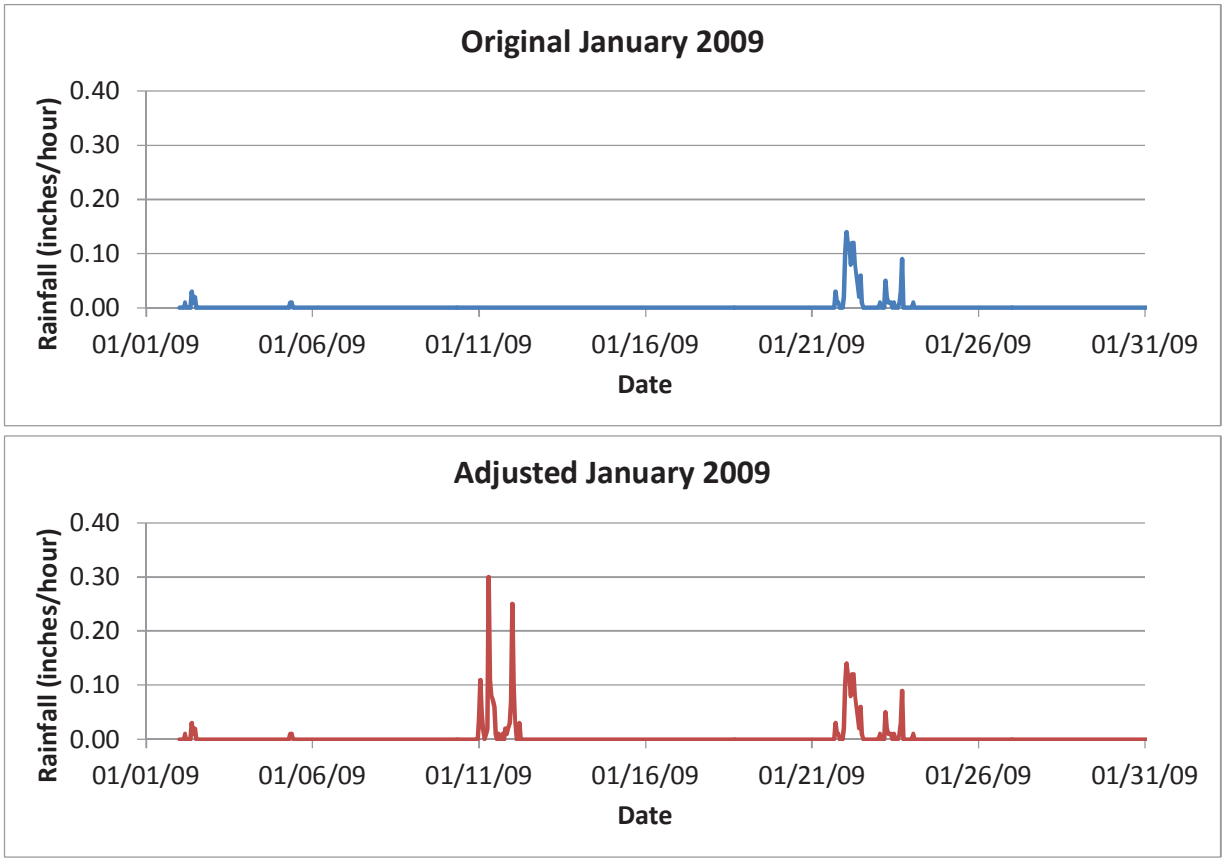


Figure 10: Comparison of Original and Adjusted January 2009 Storms

Table 8: Storms in Developed Typical Year

Start Date/Time	Duration (hours)	Depth (inches)	Max intensity (in/hr)	Avg Intensity (in/hr)	Peak intensity compared to 1-year storm			Total depth compared to 1-yr storm with closest duration		
					> 100 %	75-100%	50-75%	> 100%	75-100 %	50-75%
10/3/2008 23:00	7	0.18	0.07	0.026						
10/30/2008 16:00	58	2.36	0.29	0.041			1	1		
11/3/2008 15:00	5	0.27	0.11	0.054						
11/9/2008 1:00	3	0.07	0.04	0.023						
11/26/2008 11:00	12	0.09	0.02	0.008						
12/6/2008 1:00	1	0.01	0.01	0.010						
12/8/2008 1:00	6	0.02	0.01	0.003						
12/14/2008 11:00	49	0.58	0.08	0.012						
12/18/2008 19:00	11	0.06	0.01	0.005						
12/21/2008 2:00	29	0.38	0.08	0.013						
12/24/2008 1:00	29	0.7	0.15	0.024						
1/2/2009 4:00	8	0.07	0.03	0.009						
1/5/2009 8:00	2	0.02	0.01	0.010						
4/12/2003 6:00 INSERT TO START ON 01/11/2009	30	1.46	0.30	0.049		1			1	
1/21/2009 17:00	57	1.39	0.14	0.024						1
2/5/2009 8:00	37	0.29	0.09	0.008						
2/8/2009 20:00	4	0.08	0.04	0.020						
2/10/2009 22:00	79	1.3	0.27	0.016			1			
2/15/2009 11:00	68	1.92	0.36	0.028		1			1	
2/22/2009 2:00	38	1.33	0.14	0.035						1
2/26/2009 1:00	4	0.14	0.08	0.035						
3/1/2009 1:00	86	1.73	0.22	0.020			1			1
3/21/2009 16:00	19	0.1	0.08	0.005						
4/7/2009 13:00	71	1.57	0.47	0.022	1					1
4/24/2009 6:00	4	0.04	0.03	0.010						
5/1/2009 1:00	59	1.17	0.15	0.020						
5/5/2009 0:00	11	0.13	0.05	0.012						
6/4/2009 0:00	5	0.47	0.24	0.094			1			1
9/14/2009 0:00	6	0.19	0.07	0.032						

4. DISCHARGE FREQUENCY ANALYSIS

The discharge frequency analysis enables comparing current baseline conditions with future planning projects. This analysis will evaluate the effect of future projects on CSS performance during wet-weather events such as the typical rainfall period. The discharge frequency analysis for this study is divided into four sections:

- Baseline discharge frequency, volume, and percent wastewater
- Project Scenario I discharge frequency, volume, and percent wastewater for funded or planned projects since January 2010 and through January 2015.
- Project Scenario II discharge frequency, volume, and percent wastewater for projects necessary to protect from the 5-year storm in the six areas of worst flooding
- Project Scenario III discharge frequency and volume for projects necessary to protect from the 10-year storm throughout CSS

This preliminary technical memorandum only includes the baseline conditions and project scenario I, since the project scenarios II and III are dependent on evaluating the set of future projects. AECOM and DOU will be working on developing and refining these projects in the near-term, and will update the technical memorandum with the results soon after.

4.1. BASELINE CONDITIONS

As described in the section above, using the typical year rainfall developed in this study the H&H model simulated the typical year rainfall dataset. The model RTC module and calibration remain the same as described in the Section 2.

Table 9 tabulates the storms in a typical year rainfall with corresponding discharge volume at Outfalls 002, 004, and 006. An inter-event duration of 24 hours was used to identify discrete discharge events. Table 9 also presents percent capture for the system, which is calculated by dividing all treated flows by total system flows. CSS discharges from 002 and 006 are considered to receive primary treatment or equivalent, whereas discharges from 004 are untreated flows to the river. The NPDES permit requires the system to meet at least 85% by volume of percent capture. The baseline conditions exceed the criteria with a percent capture of 100% by volume on an average annual basis. Additionally, a water quality tracer was added to the wastewater flows during typical year simulation, and the mass of this tracer at SRCSD and three discharge points was tabulated. The tracer calculates the percentage derived from sanitary sources and storm sources at any location in the system. Table 10 presents the percentages at SRCSD and the three discharge points.

Table 9: Baseline Discharge Frequency and Volume in a Typical Year

No.	Storm Event Start Date	Storm Event End Date	Rain Depth (inches)	Event Maximum Intensity (in/hour)	Volume at 002 (MG)	Volume at 004 (MG)	Volume at 006 (MG)
1	10/31/2008 19:00	11/2/2008 1:00	1.93	0.29	7.0	--	99.5
2	1/11/2009 0:00	1/12/2009 5:00	1.46	0.30	0.17	--	33.7
3	1/21/2009 17:00	1/22/2009 12:00	1.13	0.14	--	--	33.0
4	2/16/2009 21:00	2/18/2009 6:00	1.58	0.36	6.2	--	61.6
5	4/9/2009 9:00	4/10/2009 4:00	0.94	0.47	--	--	7.7
6	5/1/2009 11:00	5/1/2009 11:00	0.87	0.15	--	--	5.9
Count of Discharge Events					3	0	6
Total Volume of Discharge (MG)					13.5	0	245.1
Total Volume to SRCSD (MG)					6780.8		
Percent Capture (%)					100%		

Table 10: Baseline Sanitary Flow Component in a Typical Year

No.	Storm Event Start Date	Storm Event End Date	Wastewater Component at SRCSD (%)	Wastewater Component at 002 (%)	Wastewater Component at 004 (%)	Wastewater Component at 006 (%)
1	10/31/2008 19:00	11/2/2008 1:00	42%	3%	0%	54%
2	1/11/2009 0:00	1/12/2009 5:00	76%	0%	0%	24%
3	1/21/2009 17:00	1/22/2009 12:00	80%	0%	0%	20%
4	2/16/2009 21:00	2/18/2009 6:00	76%	1%	0%	22%
5	4/9/2009 9:00	4/10/2009 4:00	95%	0%	0%	5%
6	5/1/2009 11:00	5/1/2009 11:00	94%	0%	0%	6%
AVERAGE FOR SIX EVENTS			77.3%	0.8%	0%	21.9%

Based on the modeling analysis as described in the sections above, the current system for a typical year analysis has two discharge events at outfall 002 and six discharge events at outfall 006. The current NPDES permit[3] lists annual observed discharge events for seven wet weather seasons from 10/2001-09/2008. Based on this observed discharge events, the average discharge events at outfall 002 is 2 (i.e. 14 activations in 7 years), at outfall 004 is 0.3 (i.e. 2 activations in 7 years), and at outfall 006 is 3.4 (i.e. 23 activations in 7 years). Comparing the modeled discharge activations with the observed activations, the modeled discharge activations are the same as observed for 002, lower than observed for 004 and higher than the observed events for 006. Referring to **Table 6** in Section 3, it should be noted that most years in this timeframe had annual rainfall lower than average typical year rainfall, and therefore the higher number of modeled discharge activations at 006 is within acceptable range for a typical year. The typical year was simulated using the

[3]ORDER NO. R5-2010-0004, NPDES NO. CA0079111

operational logic as defined in the NPDES permit and documented in this memorandum, whereas the facilities may have been operated differently during the actual storms in 2001-2008.

4.2. SCENARIO I

The project scenario I includes funded or planned projects from January 2010 through January 2015, and calculating discharge frequency, volume, and percent wastewater for a typical year rainfall period. The funded or planned projects were coded into the model using as-builts or design drawings. The projects included in scenario I are listed below:

- **Project 1 - S Street Brick Sewer Main Replacement, 14th to 17th Street:** This project was constructed in October 2012, and includes increasing conveyance capacity of the collection system. Figure 11 below highlights the project extent. The project components include:
    - Upsize existing 48 inch CSS main to 66 and 54 inch main.

Figure 11: S Street Project

- **Project 2 - 7th Street Sewer Replacement, P to K Street:** This project has been awarded and is planned for construction in 2013. The purpose of this project includes increasing

the conveyance capacity of the existing collection system. Figure 12 below highlights the project extent. The project components include:

- Upsize existing 24 and 36 inch CSS main on 7th Street from P Street to L Street to 72 inch pipe
- Upsize existing 24 inch combined sewer on 7th Street from L Street to K Street to 54 pipe
- Install 60 inch combined sewer on L Street to connect 7th Street and 9th Street

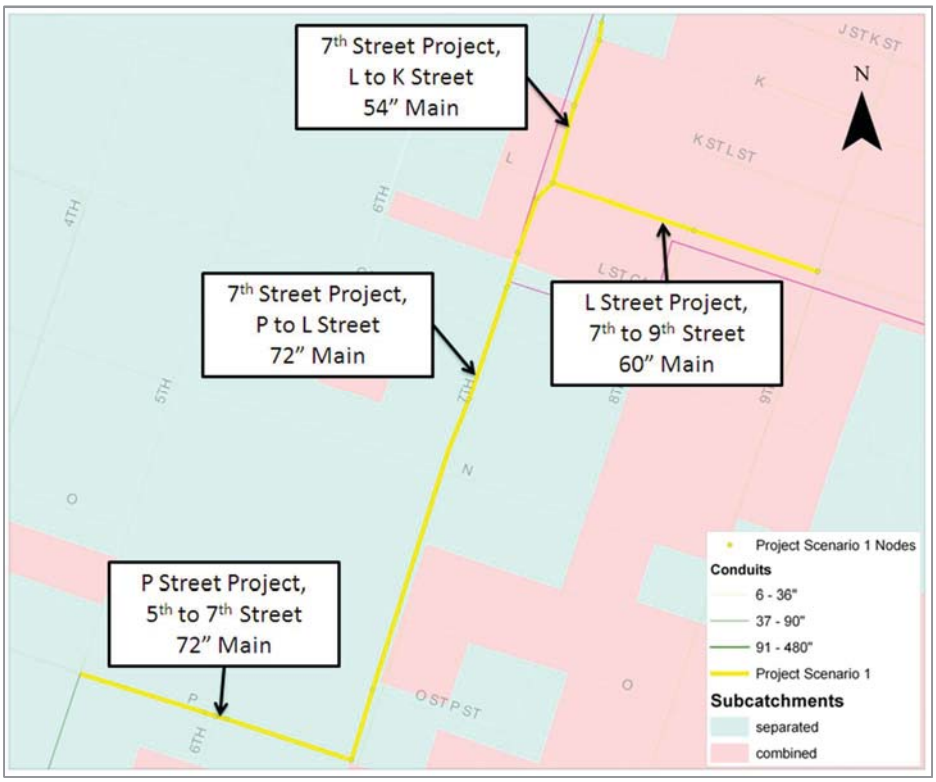


Figure 12: P Street and 7th Street Projects

- **Project 3 - P Street Sewer Improvements, between 5th and 7th Streets:** This project is out for bid and is planned to begin construction in 2013. This project provides additional flow connectivity and routing for better conveyance of upstream flows. Figure 12 above highlights the project extent. The project components include:
  - New 72 inch combined sewer pipe to connect existing 72 inch pipe on 5th Street and upsized 72 inch pipe on 7th Street.

- **Project 4 - Oak Park Regional Storage Project:** This project is under the planning process. This project provides additional offline storage of 4MG. Figure 13 below highlights the project extents. The project components include:
  - A series of large 120-inch pipes providing total storage of 4MG
  - A weir sized to allow peak flows enter the storage during 5-year and 10-year design storms

- Orifice and flap valve to allow draining the storage after the storm event.
- Additional pump station of 0.06 MGD capacity for additional draining capacity of the storage.
- The model RTC was updated so that the pump station would drain the storage after the storm event has ended.

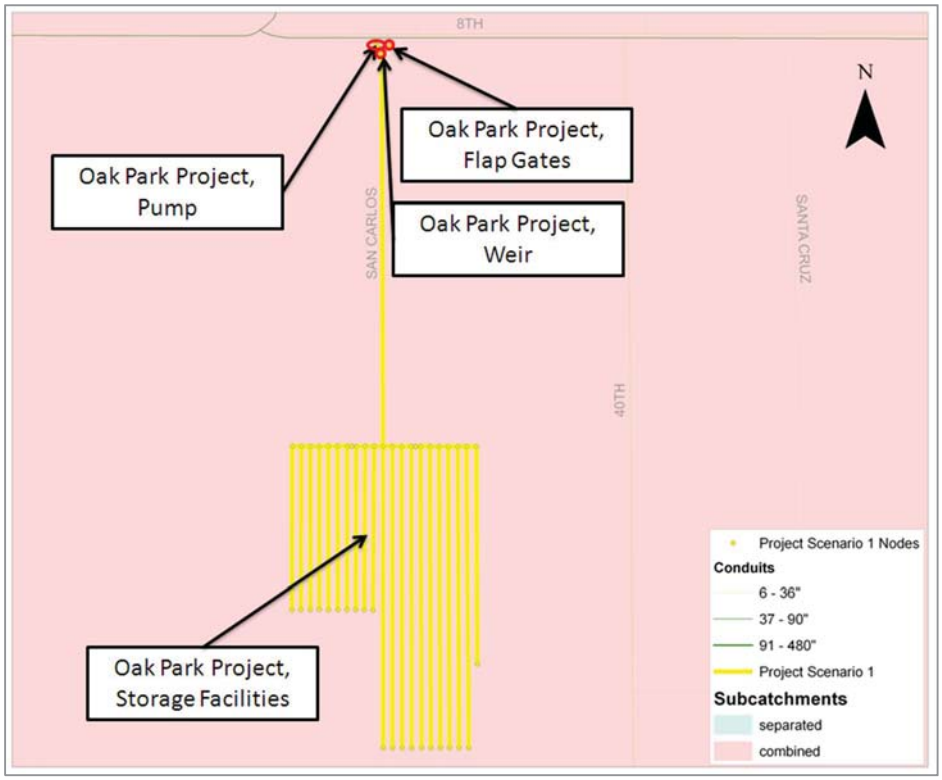


Figure 13: Oak Park Storage Project

Projects 1, 2 and 3 provide flooding benefit in Wet Area 1, whereas Project 4 provides flooding benefit in Wet Area 2. In Wet Area 1, there is 0.1 MG and 0.3 MG reduction in total flood volume for 5-year and 10-year design storms respectively with Scenario I projects. In Wet Area 2, there is a 0.4 MG and 1.7 MG reduction in total flood volume for 5-year and 10-year design storms respectively with Scenario I projects.

Table 11 below presents a discharge volume and activation summary for a typical year rainfall dataset for the Scenario I conditions. Table 12 below tabulates the wastewater component at SRCSD and the three discharge points. The typical year results show the Scenario I projects increase the CSS discharge activation at 002, and increases CSS volume by 1% at 002 and 006. However there is no impact on the CSO discharge activation or volume at 004, a discharge point with untreated flows to the river. The three conveyance projects added to relieve upstream flooding are moving the system flows to the Sump 2 and outfalls faster and thus result in an increase in CSS outflow discharge volume. In contrast, the Oak Park storage was designed to capture flows for 5-year and larger storms and was not completely utilized in the typical year storms, thus may not provide any benefit in reducing discharge volume or activation. To summarize, Scenario I projects provide the

flood reduction benefit in Wet Areas, cause a minor increase in treated CSS outflows to the river but do not cause any negative impacts by keeping untreated CSO activations to zero.

Table 11: Project Scenario I Discharge Frequency and Volume in a Typical Year

No.	Storm Event Start Date	Storm Event End Date	Rain Depth (inches)	Event Maximum Intensity (in/hour)	Volume at 002 (MG)	Volume at 004 (MG)	Volume at 006 (MG)
1	10/31/2008 19:00	11/2/2008 1:00	1.93	0.29	6.8	--	103.7
2	1/11/2009 0:00	1/12/2009 5:00	1.46	0.30	0.2	--	33.0
3	1/21/2009 17:00	1/22/2009 12:00	1.13	0.14	--	--	32.2
4	2/16/2009 21:00	2/18/2009 6:00	1.58	0.36	6.7	--	61.9
5	4/9/2009 9:00	4/10/2009 4:00	0.94	0.47	--	--	7.8
6	5/1/2009 11:00	5/1/2009 11:00	0.87	0.15	--	--	4.3
Count of Discharge Events					3	--	6
Total Volume of Discharge (MG)					13.7	0	242.9
Total Volume to SRCSD (MG)					6783.1		
Percent Capture (%)					100%		

Table 12: Scenario 1 - Sanitary Flow Component in a Typical Year

No.	Storm Event Start Date	Storm Event End Date	Wastewater Component at SRCSD (%)	Wastewater Component at 002 (%)	Wastewater Component at 004 (%)	Wastewater Component at 006 (%)
1	10/31/2008 19:00	11/2/2008 1:00	43%	4%	0%	53%
2	1/11/2009 0:00	1/12/2009 5:00	75%	0%	0%	25%
3	1/21/2009 17:00	1/22/2009 12:00	78%	0%	0%	22%
4	2/16/2009 21:00	2/18/2009 6:00	83%	1%	0%	16%
5	4/9/2009 9:00	4/10/2009 4:00	95%	0%	0%	5%
6	5/1/2009 11:00	5/1/2009 11:00	91%	0%	0%	9%
AVERAGE FOR SIX EVENTS			77.7%	0.8%	0%	21.6%

5. CONCLUSIONS

For the four events used for model calibration, the model over predicts 3 events and under predicts 1 event, and is within 15% of observed volume for all four events. The model calibration is considered good for planning purposes and can be used with good confidence for CSS outflows and CSS discharge analysis. In addition to model calibration of hydrology and hydraulics of the CSS, operation of pumping and storage facilities plays a key role in matching discharge volume and activations at the outfalls. An operational strategy different than as defined and documented in this memorandum can provide different discharge frequency and volume at the outfalls. Continuous verification of the model using additional observations and data is recommended to maintain a high level of confidence in the model. Areas with most changes in land uses should be focused on for model calibration.

CSS discharges using typical year rainfall for baseline conditions are slightly higher than the average reported activations from 2001-2008. The typical year was simulated using the operational logic as defined in the NPDES permit and documented in this memorandum, whereas the facilities may have been operated differently during the actual storms in 2001-2008. Additionally, average of annual rainfall for the 2001-2008 period is less than the typical year rainfall, and therefore a higher frequency in the typical year simulation is rational.

CSS discharges using the typical year rainfall for Scenario I conditions are higher in discharge volume by 1% than as compared to the Baseline conditions. The conveyance projects added to relieve upstream flooding in the Scenario I condition are moving system flows to Sump 2 and outfalls faster and thus increasing the CSS discharge volume at the outfalls by 1%. However, the Scenario I projects do not increase untreated CSO discharges to the river.

The typical year discharge volume and frequency for Scenario II and III conditions are forthcoming.

City of SACRAMENTO
Department of Utilities

FINAL
Combined Sewer System Improvement Plan Update Report

December 2015

Version 15

TABLE OF CONTENTS

1. EXECUTIVE SUMMARY 1-1

2. PURPOSE AND GOAL OF THE COMBINED SEWER SYSTEM IMPROVEMENT PLAN UPDATE 2-1

2.1 INTRODUCTION 2-1
2.2 BACKGROUND 2-2
2.3 PURPOSE 2-3
2.4 CONTENT 2-3

3. TOOLS FOR DEVELOPING THE CSSIP UPDATE 3-1

3.1 H&H MODEL DEVELOPMENT HISTORY AND BACKGROUND 3-1
3.2 DESIGN STORM CHARACTERISTICS 3-2
3.3 TYPICAL-YEAR RAINFALL CHARACTERISTICS 3-4

4. ASSUMPTIONS AND PERFORMANCE CRITERIA 4-1

4.1 MODELING ASSUMPTIONS 4-1
4.2 BASELINE SYSTEM PERFORMANCE 4-5

5. CSSIP PROJECTS 5-1

5.1 PROJECT 1-1 (ZAPATA PARK STORAGE) 5-3
5.2 PROJECT 1-2 (G ST. AND 9TH ST. PARKING LOT) 5-10
5.3 PROJECT 1-3 (9TH STREET FROM G TO L STREET) 5-15
5.4 PROJECT 1-4 (14TH STREET STORAGE) 5-22
5.5 PROJECT 1-5 (N STREET AND 22ND STREET) 5-29
5.6 PROJECT 1-6 (24TH STREET STORAGE) 5-35
5.7 PROJECT 1-7 (GRANT PARK) 5-41
5.8 PROJECT 2-1 (M STREET STORAGE) 5-47
5.9 PROJECT 2-2 (MCCLATCHY PARK) 5-53
5.10 PROJECT 2-3 (4TH AVENUE PARK) 5-59
5.11 PROJECT 3-1 (BIDWELL & FREEPORT) 5-65
5.12 PROJECT 3-2 (LAND PARK TRIBUTARY AREA UPSIZING) 5-71
5.13 PROJECT 3-3 (CURTIS PARK VILLAGE STORAGE) 5-76
5.14 PROJECT 3-4 (SIERRA SCHOOL PARK) 5-82

5.15 PROJECT 3-5 (BEVERLY WAY) 5-87
5.16 PROJECT 3-6 (17TH AND BIDWELL STORAGE) 5-92
5.17 PROJECT 3-7 (TARGET PARKING) 5-97
5.18 PROJECT 4-1 (MCKINLEY PARK STORAGE) 5-102
5.19 PROJECT 4-2 (SUTTER MIDDLE SCHOOL STORAGE) 5-107
5.20 PROJECT 4-3 (P STREET STORAGE) 5-112
5.21 PROJECT 4-4 (CASITA WAY) 5-117
5.22 PROJECT 5-1 (T AND 20TH STREET) 5-122
5.23 PROJECT 5-2 (T/U ALLEY AND 28TH STREET) 5-128
5.24 PROJECT 5-3 (W AND 25TH STREET) 5-133
5.25 PROJECT 6-1 (FREEPORT BOULEVARD UPSIZING) 5-140
5.26 PROJECT 6-2 (RIVERSIDE BOULEVARD UPSIZING) 5-147
5.27 PROJECT 6-3 (CURTIS PARK STORAGE) 5-154
5.28 PROJECT 6-4 (11TH AVENUE CONVEYANCE) 5-160
5.29 EXISTING SYSTEM OPTIMIZATION PROJECTS 5-166
5.30 PROGRAM 1 (RDII REDUCTION OPPORTUNITIES) 5-173
5.31 PROGRAM 2 (GI) 5-175
5.32 PROGRAM 3 (WATER CONSERVATION) 5-177

6. COST OF CSSIP PROJECTS 6-1

6.1 COST BASIS AND ASSUMPTIONS 6-1
6.2 LINEAR STORAGE 6-1
6.3 PARCEL STORAGE 6-2
6.4 PIPE UPSIZING AND NEW PIPE INSTALLATION 6-2
6.5 PUMP STATIONS 6-4
6.6 WEIR STRUCTURES 6-4
6.7 FLAP VALVES 6-6
6.8 GREEN INFRASTRUCTURE 6-6
6.9 RDII SOURCE CONTROL 6-6
6.10 WATER CONSERVATION 6-7
6.11 CONTINGENCIES TO ESTIMATE PROJECT CAPITAL COSTS 6-7
6.12 COST SUMMARY FOR ALL PROPOSED PROJECTS 6-8

7. BENEFITS OF CSSIP PROJECTS 7-1

7.1 FLOOD-REDUCTION BENEFITS 7-1
7.2 SACRAMENTO RIVER DISCHARGE BENEFITS 7-7
7.3 SUMMARY OF CSSIP BENEFITS 7-8

8. CSSIP PROJECT SEQUENCING 8-1

8.1 CONSIDERATIONS FOR DEVELOPING AND PRIORITIZING AN IMPLEMENTATION SCHEDULE 8-1
8.3 CSSIP PROGRAMS SCHEDULE 8-12

9. CONCLUSION 9-1

LIST OF FIGURES

Figure 1-1: CSSIP Recommendations 1-5
Figure 2-1: City of Sacramento CSS Overview 2-1
Figure 3-1: H&H Model History and Details 3-1
Figure 3-2: Design Storm Distribution 3-3
Figure 4-1: Extent of the Latest Building Footprint GIS Data Layer 4-3
Figure 4-2: Model Nodes Indicating Six Wet Areas 4-7
Figure 4-3: Model Baseline Results for 5-Year, 6-Hour Design Storm 4-9
Figure 4-4: Model Baseline Results for 10-Year, 6-Hour Design Storm 4-10
Figure 5-1: Baseline Flooding Near Zapata Park 5-3
Figure 5-2: Project Components for Zapata Park Storage 5-4
Figure 5-3: Pre-Project Performance at D/E Alley in 10-Year Storm 5-6
Figure 5-4: Post-Project Performance at D/E Alley in 10-Year Storm 5-6
Figure 5-5: Pre-Project Performance at 9th/10th Street in 10-Year Storm 5-7
Figure 5-6: Post-Project Performance at 9th/10th Street in 10-Year Storm 5-7
Figure 5-7: Pre-Project Performance at C/D Alley in 10-Year Storm 5-8
Figure 5-8: Post-Project Performance at C/D Alley in 10-Year Storm 5-8
Figure 5-9: Zapata Park Aerial View 5-9
Figure 5-10: Baseline Flooding at G & 9th Street Parking Lot 5-10
Figure 5-11: Project Components for G & 9th Street Parking Lot 5-11
Figure 5-12: Pre-Project Performance at 9th Street in 10-Year Storm 5-13
Figure 5-13: Post-Project Performance at 9th Street in 10-Year Storm 5-13
Figure 5-14: G and 9th Street Storage, Aerial View 5-14
Figure 5-15: Baseline Flooding near 9th Street Upsizing 5-15
Figure 5-16: Project Components for 9th Street Upsizing 5-16
Figure 5-17: Pre-Project Performance at G and 9th Streets in 10-Year Storm 5-18
Figure 5-18: Post-Project Performance at G and 9th Streets in 10-Year Storm 5-18
Figure 5-19: Pre-Project Performance on 9th from G to L Streets in 10-Year Storm 5-19
Figure 5-20: Post-Project Performance on 9th from G to L Streets in 10-Year Storm 5-19
Figure 5-21: 9th Street Aerial View 5-20
Figure 5-22: 9th Street View (Looking South from H and 9th Street) 5-21
Figure 5-23: Baseline Flooding near 14th Street 5-22

Figure 5-24: Project Components for 14th Street Storage....5-23
Figure 5-25: Pre-Project Performance on H/I Alley in 10-Year Storm 5-25
Figure 5-26: Post-Project Performance on H/I Alley in 10-Year Storm....5-25
Figure 5-27: Pre-Project Performance on 13th Street in 10-Year Storm 5-26
Figure 5-28: Post-Project Performance on 13th Street in 10-Year Storm....5-26
Figure 5-29: 14th Street Storage Aerial View 5-27
Figure 5-30: H and 14th Street View (Looking North) 5-28
Figure 5-31: Baseline Flooding Near N and 22nd Street 5-29
Figure 5-32: N and 22nd Street Project Components....5-30
Figure 5-33: Pre-Project Performance on N Street in 10-Year Storm....5-32
Figure 5-34: Post-Project Performance on N Street in 10-Year Storm 5-32
Figure 5-35: Pre-Project Performance on N & 22nd Street in 10-Year Storm 5-33
Figure 5-36: Post-Project Performance on N & 22nd Street in 10-Year Storm....5-33
Figure 5-37: N and 22nd Street Aerial View 5-34
Figure 5-38: N Street View (Looking East from N and 21st Street)....5-34
Figure 5-39: Baseline Flooding near 24th Street 5-35
Figure 5-40: 24th Street Storage Project Components....5-36
Figure 5-41: Pre-Project Performance on 25th Street in 10-Year Storm 5-38
Figure 5-42: Post-Project Performance on 25th Street in 10-Year Storm....5-38
Figure 5-43: 24th Street Storage Aerial View 5-39
Figure 5-44: 24th Street View (Looking North from K and 24th Streets)....5-40
Figure 5-45: Baseline Flooding near Grant Park 5-41
Figure 5-46: Grant Park Project Components....5-42
Figure 5-47: Pre-Project Performance at 25th Street in a 10-Year Storm 5-44
Figure 5-48: Post-Project Performance at 25th Street in a 10-Year Storm....5-44
Figure 5-49: Grant Park Project Aerial View....5-45
Figure 5-50: D Street View (Looking East from 22nd & D Street) 5-46
Figure 5-51: Baseline Flooding near M Street....5-47
Figure 5-52: M Street Storage Project Components....5-48
Figure 5-53: Pre-Project Performance on 41st Street in 10-Year Storm....5-50
Figure 5-54: Post-Project Performance on 41st Street in 10-Year Storm 5-50
Figure 5-55: Pre-Project Performance on Folsom Blvd. in 10-Year Storm 5-51
Figure 5-56: Post-Project Performance on Folsom Blvd. in 10-Year Storm....5-51

Figure 5-57: M Street Aerial View 5-52
Figure 5-58: M Street View (Looking East from M and 42nd Street) 5-52
Figure 5-59: Baseline Flooding near McClatchy Park.....5-53
Figure 5-60: McClatchy Park Storage Project Components 5-54
Figure 5-61: Pre-Project Performance on Broadway in 10-Year Storm 5-56
Figure 5-62: Post-Project Performance on Broadway in 10-Year Storm 5-56
Figure 5-63: Pre-Project Performance on Broadway in 10-Year Storm 5-57
Figure 5-64: Post-Project Performance on Broadway in 10-Year Storm 5-57
Figure 5-65: McClatchy Park Aerial View 5-58
Figure 5-66: Baseline Flooding near 4th Avenue Park 5-59
Figure 5-67: 4th Avenue Park Storage Project Components.....5-60
Figure 5-68: Pre-Project Performance on 4th Avenue in 10-Year Storm 5-62
Figure 5-69: Post-Project Performance on 4th Avenue in 10-Year Storm.....5-62
Figure 5-70: Pre-Project Performance on Broadway in 10-Year Storm 5-63
Figure 5-71: Post-Project Performance on Broadway in 10-Year Storm 5-63
Figure 5-72: 4th Avenue Park Aerial View 5-64
Figure 5-73: 4th Avenue Street View (Looking East from La Solidad Way and 4th Avenue) .. 5-64
Figure 5-74: Baseline Flooding near Bidwell Way and Freeport Boulevard 5-65
Figure 5-75: Freeport and Bidwell Connection Project Components 5-66
Figure 5-76: Pre-Project Performance on Freeport Blvd. in 10-Year Storm 5-68
Figure 5-77: Post-Project Performance on Freeport Blvd. in 10-Year Storm.....5-68
Figure 5-78: Pre-Project Performance in Donner Trunk in 10-Year Storm.....5-69
Figure 5-79: Post-Project Performance in Donner Trunk in 10-Year Storm 5-69
Figure 5-80: Freeport and Bidwell Aerial View.....5-70
Figure 5-81: Baseline Flooding near Land Park Drive 5-71
Figure 5-82: Land Park Tributary Area Upsizing Components.....5-72
Figure 5-83: Pre-Project Performance on 18th Street in 10-Year Storm 5-74
Figure 5-84: Post-Project Performance on 18th Street in 10-Year Storm.....5-74
Figure 5-85: Land Park Tributary Area Aerial View 5-75
Figure 5-86: Baseline Flooding near Curtis Park Village 5-76
Figure 5-87: Curtis Park Village Storage Project Components 5-77
Figure 5-88: Pre-Project Performance on 21st Street in 10-Year Storm.....5-79

Figure 5-89: Post-Project Performance on 21st Street in 10-Year Storm 5-79
Figure 5-90: Curtis Park Village Aerial View.....5-80
Figure 5-91: Markham and 21st Street View (Looking South).....5-81
Figure 5-92: Baseline Flooding near Sierra School Park.....5-82
Figure 5-93: Sierra School Park Storage Project Components.....5-83
Figure 5-94: Pre-Project Performance near Sierra School Park in 10-Year Storm.....5-85
Figure 5-95: Post-Project Performance near Sierra School Park in 10-Year Storm 5-85
Figure 5-96: Sierra School Park Storage Project Components.....5-86
Figure 5-97: Baseline Flooding near Beverly Way 5-87
Figure 5-98: Beverly Way Storage Project Components 5-88
Figure 5-99: Pre-Project Performance on 1st Avenue in 10-Year Storm.....5-90
Figure 5-100: Post-Project Performance on 1st Avenue in 10-Year Storm.....5-90
Figure 5-101: Beverly Way Aerial View 5-91
Figure 5-102: Beverly Way Street View (Looking West from Beverly Way and 13th Street) ... 5-91
Figure 5-103: Baseline Flooding near 17th Street and Bidwell Way.....5-92
Figure 5-104: 17th Street and Bidwell Way Storage Project Components 5-93
Figure 5-105: Pre-Project Performance at 17th and Bidwell in 10-Year Storm 5-95
Figure 5-106: Post-Project Performance at 17th and Bidwell in 10-Year Storm 5-95
Figure 5-107: 17th Street Bidwell Storage Project Components 5-96
Figure 5-108: Baseline Flooding near Target Store Parking Lot 5-97
Figure 5-109: Target Storage Project Components.....5-98
Figure 5-110: Pre-Project Performance at Broadway and 13th Street in 10-Year Storm 5-100
Figure 5-111: Post-Project Performance at Broadway and 13th Street in 10-Year Storm 5-100
Figure 5-112: Target Storage Aerial View 5-101
Figure 5-113: Baseline Flooding near McKinley Park 5-102
Figure 5-114: McKinley Park Storage Project Components 5-103
Figure 5-115: Pre-Project Performance on Park Way and 3rd Street in 10-Year Storm .. 5-105
Figure 5-116: Post-Project Performance on Park Way and 3rd Street in 10-Year Storm 5-105
Figure 5-117: McKinley Park Storage Project Aerial View 5-106
Figure 5-118: Baseline Flooding near Sutter Middle School..... 5-107
Figure 5-119: Sutter Middle School Storage Project Components 5-108

Figure 5-120: Pre-Project Performance on Alhambra Blvd. in 10-Year Storm 5-110
Figure 5-121: Post-Project Performance on Alhambra Blvd. in 10-Year Storm..... 5-110
Figure 5-122: Sutter Middle School Storage Project Aerial View 5-111
Figure 5-123: Baseline Flooding near P Street..... 5-112
Figure 5-124: P Street Storage Project Components 5-113
Figure 5-125: Pre-Project Performance on P Street in 10-Year Storm..... 5-115
Figure 5-126: Post-Project Performance on P Street in 10-Year Storm 5-115
Figure 5-127: P Street Aerial View 5-116
Figure 5-128: P Street View (Looking East from Santa Ynez Way and P Street)..... 5-116
Figure 5-129: Baseline Flooding near Casita Way..... 5-117
Figure 5-130: Casita Way Project Components..... 5-118
Figure 5-131: Pre-Project Performance on Casita Way in 10-Year Storm..... 5-120
Figure 5-132: Post-Project Performance on Casita Way in 10-Year Storm 5-120
Figure 5-133: Casita Way Aerial View 5-121
Figure 5-134: Casita Way Street View (Looking West from Casita and 32nd Street) 5-121
Figure 5-135: Baseline Flooding near T and 20th Streets 5-122
Figure 5-136: T and 20th Street Project Components 5-123
Figure 5-137: Pre-Project Performance on T Street at 20th in 10-Year Storm 5-125
Figure 5-138: Post-Project Performance on T Street at 20th in 10-Year Storm 5-125
Figure 5-139: T and 20th Street Aerial View 5-126
Figure 5-140: 20th Street View (Looking North from T and 20th Street)..... 5-127
Figure 5-141: Baseline Flooding near T/U Alley and 28th Street 5-128
Figure 5-142: T/U Alley and 28th Street Project Components..... 5-129
Figure 5-143: Pre-Project Performance at T Street and 28th in 10-Year Storm 5-131
Figure 5-144: Post-Project Performance at T Street and 28th in 10-Year Storm 5-131
Figure 5-145: T/U Alley and 28th Street Aerial View..... 5-132
Figure 5-146: T/U Alley Street View (Looking West from T/U and 29th Street)..... 5-132
Figure 5-147: Baseline Flooding near W and 25th Streets..... 5-133
Figure 5-148: W and 25th Storage Project Components 5-134
Figure 5-149: Pre-Project Performance on W/V Alley in 10-Year Storm 5-136
Figure 5-150: Post-Project Performance on W/V Alley in 10-Year Storm..... 5-136
Figure 5-151: Pre-Project Performance on 24th Street in 10-Year Storm 5-137
Figure 5-152: Post-Project Performance on 24th Street in 10-Year Storm 5-137

Figure 5-153: Pre-Project Performance at W and 25th Street in 10-Year Storm 5-138
Figure 5-154: Post-Project Performance at W and 25th Street in 10-Year Storm 5-138
Figure 5-155: W and 25th Storage Aerial View 5-139
Figure 5-156: V/W Alley Street View (Looking East from W/V Alley and 25th Street)..... 5-139
Figure 5-157: Baseline Flooding near Freeport Boulevard 5-140
Figure 5-158: Freeport Boulevard Upsizing Project Components 5-141
Figure 5-159: Pre-Project Performance on Freeport Blvd. in 10-Year Storm 5-143
Figure 5-160: Post-Project Performance on Freeport Blvd. in 10-Year Storm 5-143
Figure 5-161: Pre-Project Performance at Freeport Blvd. and 12th Avenue in 10-Year Storm 5-144
Figure 5-162: Post-Project Performance at Freeport Blvd. and 12th Avenue in 10-Year Storm 5-144
Figure 5-163: Freeport Boulevard Aerial View 5-145
Figure 5-164: Freeport Boulevard Street View (Looking North from Freeport and 12th Avenue) 5-146
Figure 5-165: Baseline Flooding near Riverside Boulevard 5-147
Figure 5-166: Riverside Boulevard Upsizing Project Components 5-148
Figure 5-167: Pre-Project Performance on Riverside Blvd. in 10-Year Storm 5-150
Figure 5-168: Post-Project Performance on Riverside Blvd. in 10-Year Storm 5-150
Figure 5-169: Pre-Project Performance on Beverly Way in 10-Year Storm 5-150
Figure 5-170: Post-Project Performance on Beverly Way in 10-Year Storm 5-151
Figure 5-171: Riverside Boulevard Aerial View 5-152
Figure 5-172: Riverside Boulevard Street View (Looking North from Riverside and Beverly Way) 5-153
Figure 5-173: Baseline Flooding near Curtis Park 5-154
Figure 5-174: Curtis Park Storage Project Components 5-155
Figure 5-175: Pre-Project Performance on 24th Street in 10-Year Storm 5-157
Figure 5-176: Post-Project Performance on 24th Street in 10-Year Storm 5-157
Figure 5-177: Pre-Project Performance on Franklin Street in 10-Year Storm 5-158
Figure 5-178: Post-Project Performance on Franklin Street in 10-Year Storm 5-158
Figure 5-179: Curtis Park Storage Aerial View 5-159
Figure 5-180: Baseline Flooding near 11th Avenue 5-160
Figure 5-181: 11th Avenue Installation Project Components 5-161
Figure 5-182: Pre-Project Performance on Freeport Blvd. in 10-Year Storm 5-163

Figure 5-183: Post-Project Performance on Freeport Blvd. in 10-Year Storm 5-163
Figure 5-184: Pre-Project Performance on Land Park Drive in 10-Year Storm 5-164
Figure 5-185: Post-Project Performance on Land Park Drive in 10-Year Storm 5-164
Figure 5-186: 11th Avenue Aerial View 5-165
Figure 5-187: 11th Avenue Street View (Looking West from Freeport and 11th Avenue)..... 5-165
Figure 5-188: 21st Street Local Storage Optimization 5-166
Figure 5-189:Tahoe Park/Broadway Parallel Sewer Project 5-169
Figure 5-190: East End Relief Sewer Tie-in Project 5-170
Figure 5-191: U St. Inline Storage Project 5-171
Figure 5-192: Separated Areas of the Combined Sewer System 5-173
Figure 5-193: Percent Impervious Surface of the Combined System 5-175
Figure 6-1: Parcel Storage Construction Costs 6-3
Figure 6-2: Pump Station Cost Curve 6-5
Figure 7-1: Results for 5-Year, 6-Hour Design Storm for 28 Projects 7-3
Figure 7-2: Results for 10-Year, 6-Hour Design Storm for 28 Projects 7-4
Figure 7-3: Results for 5-Year, 6-Hour Design Storm for 28 Projects and 3 Programs 7-5
Figure 7-4: Results for 10-Year, 6-Hour Design Storm for 28 Projects and 3 Programs 7-6
Figure 8-1: CSSIP Projects Implementation Sequence and Cumulative Costs 8-11
Figure 8-2: CSSIP Program Implementation Schedule 8-15
Figure 8-3: CSSIP Projects and Programs Implementation Schedule 8-16
Figure 9-1: CSSIP Recommendations 9-3
Figure 9-2: CSSIP Implementation Approach Using Adaptive Management 9-5
Figure 9-3: Adaptive Management Strategy 9-5

LIST OF TABLES

Table 1-1: Prioritized List of all Projects and Costs 1-8
Table 3-1: Design Storm Details 3-3
Table 4-1: H&H Modeling History 4-1
Table 4-2: Typical Year Results 4-5
Table 4-3: Baseline Results Summary for Design Storms 4-8
Table 5-1: Modeled Peak HGL for Node Near Zapata Park 5-5
Table 5-2: Modeled Peak HGL for Node Near G Street and 9th Street Parking Lot 5-12
Table 5-3: Modeled Peak HGL for Node Along 9th Street 5-17
Table 5-4: Modeled Peak HGL for Node Near 14th Street Storage 5-24
Table 5-5: Modeled Peak HGL for Node Near N & 22nd Street 5-31
Table 5-6: Modeled Peak HGL for Node Near 24th Street 5-37
Table 5-7: Modeled Peak HGL for Node Near Grant Park 5-43
Table 5-8: Modeled Peak HGL for Node Near M Street 5-49
Table 5-9: Modeled Peak HGL for Node Near McClatchy Park 5-55
Table 5-10: Modeled Peak HGL for Node Near 4th Avenue Park 5-61
Table 5-11: Modeled Peak HGL for Node Near Bidwell and Freeport Boulevard 5-67
Table 5-12: Modeled Peak HGL for Node Near Land Park Drive 5-73
Table 5-13: Modeled Peak HGL for Node Near Curtis Park Village 5-78
Table 5-14: Modeled Peak HGL for Node Near Sierra School 5-84
Table 5-15: Modeled Peak HGL for Node Near Beverly Way 5-89
Table 5-16: Modeled Peak HGL for Node Near 17th Street & Bidwell 5-94
Table 5-17: Modeled Peak HGL for Node Near Target Parking 5-99
Table 5-18: Modeled Peak HGL for Node Near McKinley Park 5-104
Table 5-19: Modeled Peak HGL for Node Near Sutter Middle School 5-109
Table 5-20: Modeled Peak HGL for Area Near P Street 5-114
Table 5-20: Modeled Peak HGL for Node Near Casita Way 5-119
Table 5-21: Modeled Peak HGL for Node Near T and 20th Street 5-124
Table 5-22: Modeled Peak HGL for Node Near T/U Alley and 28th Street 5-130
Table 5-23: Modeled Peak HGL for Node Near W and 25th Street 5-135
Table 5-24: Modeled Peak HGL for Node Near Freeport Boulevard 5-142
Table 5-25: Modeled Peak HGL for Node Near Riverside Boulevard 5-149
Table 5-26: Modeled Peak HGL for Node Near Curtis Park Storage 5-156

Table 5-27: Modeled Peak HGL for Node Near 11th Avenue Conveyance 5-162
Table 5-28: Modeled Peak HGL for Node Near 21st Street Local Storage Improvement 5-167
Table 5-29: Modeled Peak HGL for Node Near Tahoe Park/Broadway Parallel Sewer Project 5-169
Table 5-30: Modeled Peak HGL for Node Near East End Relief Sewer Tie-in Project..... 5-170
Table 5-31: Modeled Peak HGL for Node Near U St. Inline Storage Project 5-171
Table 5-32: System-Wide Benefits of RDII Reduction 5-174
Table 5-33: System-Wide Benefits of Green Infrastructure..... 5-176
Table 5-34: System-Wide Benefits of Water Conservation..... 5-177
Table 6-1: Planning-Level Capital Cost Estimate Summary..... 6-9
Table 7-1: Baseline vs. 28 Projects Results Summary for Design Storms 7-1
Table 7-2: Baseline vs. 28 Projects + Three Programs Results Summary for Design Storms 7-2
Table 7-3: Typical Year Results..... 7-7
Table 8-1: Weighing Factors for Prioritization Drivers 8-5
Table 8-2: Project Prioritization..... 8-6
Table 8-3: CSSIP Projects Prioritization Ranking..... 8-9
Table 8-4: RDII and Green Infrastructure Implementation.....8-14

ABBREVIATIONS/ACRONYM LIST

2D	two-dimensional
CCI	Construction Cost Index
CCTV	closed-circuit television
CDO	Cease and Desist Order
CIPP	cured-in-place pipe
City	City of Sacramento
CSS	combined sewer system
CSSIP	Combined Sewer System Improvement Plan
CSO	Combined Sewer Overflow
CWTP	Combined Wastewater Treatment Plant
DOU	Department of Utilities
ENR	Engineering News Record
EPA	Environmental Protection Agency
ESD	Equivalent Single-Family Dwelling
GI	Green Infrastructure
H&H	hydrologic and hydraulic
HGL	hydraulic grade line
LF	linear feet
LTCP	Long-Term Control Plan
mg	million gallons
mgd	million gallons per day
O&M	operation and maintenance
Outflows	Combined Sewer System Outflows
NPDES	National Pollutant Discharge Elimination System
RDII	rainfall derived infiltration and inflow
RWQCB	Regional Water Quality Control Board
SAC-SWMM	Sacramento Storm Water Management Model
SRWTP	Sacramento Regional County Sanitation District Regional Wastewater Treatment Plant
SWMM	Storm Water Management Model
TM	technical memorandum

GLOSSARY

CSS Discharge: Combined sewer system flows, treated or untreated, discharged to a receiving water body

CSS Outflow: Combined sewer system flows exiting system manholes or drain inlets onto streets and parcels during peak flow events

Flooding: Any combination of CSS Outflows and excess storm water runoff causing ponding on streets and property

Outfall 001: Sacramento Regional Water Treatment Plant (SRWTP) effluent discharge point to the Sacramento River; flows receive secondary treatment

Outfall 002: Combined Water Treatment Plant (CWTP) effluent discharge point to the Sacramento River; flows receive primary treatment and disinfection

Outfall 003: Emergency outfall only

Outfall 004: Sump 2 discharge point to the Sacramento River; flows are untreated

Outfall 005: Emergency outfall only

Outfall 006: Pioneer Reservoir effluent discharge point to the Sacramento River; flows receive primary treatment and disinfection

Outfall 007: Emergency outfall only

1. EXECUTIVE SUMMARY

The City of Sacramento (City) owns and operates a combined sewer system (CSS) that conveys residential and commercial wastewater and storm water runoff from approximately 11.7 square miles in downtown Sacramento, East Sacramento, Oak Park, and the Land Park area. There are 5.8 square miles of separated areas of the City north, east, and south of the CSS that contribute sanitary flows to the CSS. The City also includes approximately 76 square miles of separated areas that are not served by the CSS. The CSS serves approximately 205,000 people. The CSS includes four key facilities to manage the collected flow: Sumps 1/1A, Sumps 2/2A, Pioneer Reservoir, and the Combined Wastewater Treatment Plant (CWTP). Sumps 1/1A and 2/2A pump up to 60 million gallons per day (mgd) of flows to the Sacramento Regional County Sanitation District's Regional Wastewater Treatment Plant (SRWTP). Pioneer Reservoir and CWTP provide additional storage and, when needed, primary treatment, and disinfection of combined sewage prior to discharge to the Sacramento River. The CSS is regulated under the August 2015 National Pollutant Discharge Elimination System (NPDES) waste discharge permit, No. CA0079111. The permit allows for CSS discharge to the Sacramento River at six locations: two for primary treated (plus disinfection) effluent, and four that can discharge untreated combined sewage. The secondary treated effluent from SRWTP is discharged to the Sacramento River at a permitted location under a separate NPDES permit.

The City's Combined Sewer System Improvement Plan (CSSIP) Update is an ongoing multi-year program intended to evaluate and provide recommendations for projects to alleviate outflows and flooding in the CSS area during a 10-year storm event and to prevent structure flooding (as well as outflows) during the 100-year storm event. During large rain events such as the 10-year storm, a combination of stormwater and wastewater (combined sewage) ponds or moves across the land surface in some parts of the City. This combined sewage has the potential to be a public health risk (through human contact with constituents found in combined sewage) and safety risk (high depth and/or velocity of flow on the surface). Water on the surface in these instances is a combination of combined sewage outflowing from the collection system that has reached maximum capacity, and stormwater runoff that cannot be drained on account of the system being full. Phase 1 of the CSSIP Update collected flow monitoring data and developed and calibrated the hydrologic and hydraulic (H&H) model of the CSS area and separated sewer areas tributary to the CSS. Phase 2 uses the H&H model to evaluate and recommend capital improvement projects to achieve the City Department of Utilities (DOU) flood protection goals. Phase 2 also refined the H&H model with the latest planning information, refined the physical network representation in the six wet areas, and calibrated the operational controls for Sump 2/2A using recent wet-weather events. Additionally, the software was upgraded to enable use of a two-dimensional (2D) module to more precisely represent the overland surface routing of flows in the CSS area. This latest H&H model was used to evaluate prospective capital improvement projects to be included in the CSSIP Update.

The H&H model results for the 5-year and 10-year design storms matched observed flooding fairly well, providing confidence in the model's identification of flooding, and the solutions developed to address them. However, the H&H model is still a planning-level tool, and the datasets (e.g., ground model, building boundaries) used for the 2D module can be further refined in resolution. Additionally, the 2D module needs calibration and verification using historical surface flooding information. The H&H model, in combination with design rainfall datasets, was used to evaluate the system and develop projects for the CSSIP Update.

The CSSIP Update primarily addresses two NPDES permit requirements: The reduction of CSS discharges and in-system surface flooding and outflows. The NPDES requirements related to the impact of CSS discharges on Sacramento River receiving water quality and beneficial uses is addressed in the City's June 2013 Water Quality Assessment Report[1]. The report summarizes the most recent study to evaluate compliance with water quality objectives. It concluded that the infrequent and short duration CSS overflow discharges do not impact applicable receiving water beneficial uses.

> CSS Discharges: As mentioned previously, during wet-weather events, CSS flows can discharge to the Sacramento River at seven locations (i.e., outfalls), numbered 001 through 007. Outfall 001 is the SRWTP effluent discharge that receives secondary treatment. Discharges from the CWTP and Pioneer Reservoir (Outfalls 002 and 006) receive equivalent primary treatment and disinfection. Discharges from Sump 2 (Outfall 004) are untreated. Outfalls 003, 005, and 007 are only used for emergency bypass conditions, and, should they be activated, would discharge untreated flow. For a "typical" average annual rainfall dataset simulated in the H&H model for baseline conditions, all discharges to the Sacramento River are treated to, at a minimum, partial primary clarification or equivalent, solids and floatables disposal, and disinfection. There are no untreated discharges to the river for the "typical" average annual rainfall (all discharges are through one or more of outfalls 001, 002, and 006). The existing collection system is therefore in compliance with the NPDES permit's quantitative discharge requirements.
>
> Flooding and Combined Sewer Outflows: The City's NPDES permit requires implementation of the CSSIP to meet interim goals as progress is made toward the final goal of minimizing street flooding from combined sewage escaping the system as well as stormwater runoff not being drained by the system during a 10-year storm event and preventing structure flooding during the 100-year event. During large storm events, such as 5-year and 10-year storms, wet-weather flows in the CSS can exceed the capacity of the system, and can cause surface flooding. The NPDES permit references the six wet areas that have known flooding issues during larger

[1] City of Sacramento Combined Sewer System (CSS) Water Quality Assessment. June 25, 2013.

storm events. Prioritizing remediation and improvement in these six areas was a focus of the 1995 Long-Term Control Plan (LTCP), and continues to be a focus of the CSSIP Update. The 5- and 10-year design storms were simulated in the H&H model to create a baseline for the existing system and identify flooding locations within the six wet areas and system-wide. Model simulations of historical events were conducted, and results matched DOU's observations of associated flooding fairly well. This correlation gave confidence in the model's ability to identify deficiencies throughout the system. Based on the H&H model simulation of potential project alternatives, several capital improvement projects within the CSS and three programs were developed to address flooding in the six wet areas and system-wide. These improvements meet the interim goals of the CSSIP.

CSSIP Recommendations

For each flooding area identified using the H&H model for the design storms, different types of projects were identified that would either increase the conveyance capacity downstream of the flooding area or provide storage for the excess flows. The storage projects were developed as either parcel storage underneath an open park or vacant lot, or as linear storage within the City's right-of-way. The project sizing was taken into account to ensure that the project was the most cost-effective way to reduce or eliminate flooding in an area. Additionally, system-wide programs were identified that would reduce the sanitary flows and storm water runoff entering the collection system, thus providing a system-wide benefit to the flood-reduction goal. The programs are Green Infrastructure (GI) to detain or retain storm-water in the combined areas, Rainfall-Derived Infiltration and Inflow (RDII) source control to reduce wet-weather flows in the separated sanitary areas, and Water Conservation to reduce sanitary flows from the entire service area. Figure 1-1 presents the projects and programs that were developed and recommended as part of this CSSIP Update.

The Department is keenly interested in investigating the implementation of GI as a method to both contribute to CSSIP goals and to provide community benefits in the form of enhanced open and recreation space. GI can include surface retention as well as detention technologies, both mitigating the "flashy" urban watershed response by reducing or delaying surface runoff. Retention-based technologies include reduction in surface runoff through infiltration in areas of favorable soil conditions. Detention can be accomplished regardless of soil conditions through onsite storage and reuse systems. Detention-based improvements delay surface runoff by temporarily storing surface runoff and later discharging back into the system, thereby reducing peak flow rates. Examples of GI include bioretention (sidewalk planters, bulbouts, rain gardens, etc.), permeable pavement, engineered

routing of former creek flows or seeps along surface reaches, and rooftop improvements (“green” with plantings and “blue” with detention storage only).

Costs

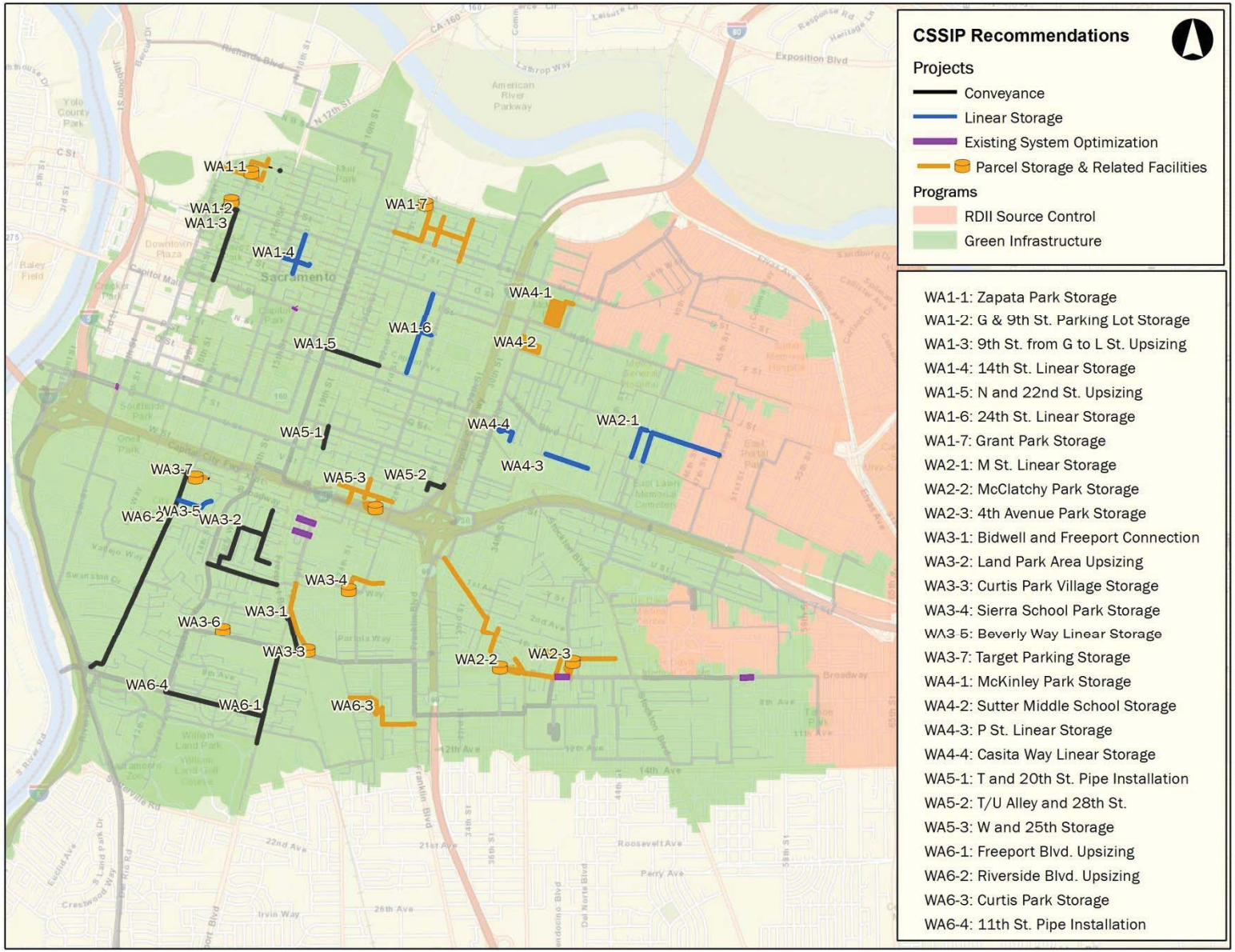
The costs estimates for the CSSIP projects were developed based on actual construction costs for similar facilities built by DOU, construction costs for similar facilities designed but not constructed, published industry standard cost data and curves, and data supplied by manufacturers. The contingency included all uncertainties or unanticipated construction costs, including engineering, legal, environmental review, inspection, and/or contract administration. All costs were adjusted to August 2013 dollars based on an Engineering News Record Construction Cost Index, which uses a 20-city national average. The index was used for planning-level cost estimates only, and will be updated as the projects become better defined.

For the GI program, DOU developed an initial program concept and cost estimate through internal discussions between the Engineering and Water Quality groups. The estimate and concepts are based on low-impact-development tools used by the Water Quality group, and reference materials made available by other utilities and developers nationwide.

For the RDII source control program, the unit costs for pipe relining were developed using recent sewer rehabilitation projects in the Northern California region. The costs vary considerably depending on the size and length of pipe to be lined and the type of lining. The cost estimate assumes all the lateral, manholes, and sewer mains will be rehabilitated, and is on the higher range for cost estimates. Pilot programs would help refine and reduce this cost once the actual cost and effectiveness of the pipe relining for different sizes, lengths, and types is developed for the City.

For the Water Conservation program, it is assumed that DOU sewer fund will not incur any programmatic costs for reduced sanitary flows due to residents adopting water-efficient devices and appliances that reduce water use. The cost of water conservation program will be incurred by DOU water fund that supports the water conservation program and provides rebates and water wise house calls.

Figure 1-1: CSSIP Recommendations



Project Benefits to the CSS

The individual benefits of the projects that provide localized or system-wide reduction to flooding vary significantly. A few projects completely eliminate flooding in the 10-year design storm, and several projects work in conjunction with other projects to minimize flooding. The programs integrated with the projects further enhance flood-reduction benefits at a system-wide level. None of the projects were found to increase untreated discharges to the Sacramento River. A few storage projects were found to provide a marginal (less than 2%) decrease in treated discharges. The programs provide a higher benefit (5–9%) in reducing treated discharges.

Prioritization

The projects were prioritized based on considerations such as flood-reduction benefits, cost-effectiveness, ensuring no increase in untreated discharges, sewer condition/age, cost-sharing opportunities, and City/community interests. The programs were identified and spatially distributed uniformly across the system. The costs and benefits of the programs are at a planning-level scale, and this CSSIP Update recommends developing pilot programs to refine the spatial application of programs within the system, and to refine the cost and performance of these programs.

Adaptive Management

The projects and programs recommended in this CSSIP Update do not completely achieve the final flood reduction goals of the NPDES permit LTCP to minimize street flooding for the 10-year design storm and to prevent structure flooding for the 100-year event. Modeling estimated that the total volume of CSS outflows, the quantitative metric used to measure flooding, decreased significantly (65%) across the entire City as a result of implementing all projects and programs. The estimated cost to address the remaining 35% of flooding, based on currently used planning-level tools and cost estimates, would be substantial. Rather than developing more projects to move closer to the final LTCP flood reduction goal, the CSSIP Update recommends an adaptive management strategy as part of CSSIP implementation.

The recommended adaptive management strategy starts with implementation of the top 20% of prioritized CSSIP projects, pilot testing of the programs, additional refinement of the H&H model, refinement of cost estimates, and refinement of any inputs with lower accuracy used to develop the CSSIP. Over the implementation period of the entire CSSIP, the adaptive management strategy would continuously refine the accuracy and confidence of the tools and information, and use the updated tools and information to further analyze, develop and refine projects until the final LTCP goal of minimizing street flooding for 10-year storm and preventing structure flooding for the 100-year event is met system-wide. The following

recommendations describe the specific improvements that can be incorporated as part of an adaptive management strategy:

1. Prepare a plan for implementation of the top 20% of capital improvement projects listed in this CSSIP Update, including any detailed feasibility studies followed by design and construction of these projects.

2. Develop a pilot program to evaluate different types of GI technologies and test their performance in different surface and sub-surface conditions, including soil type and slope; general public acceptability; maintenance issues; and other topics. Monitor the performance of the different technologies and establish the baseline performance of the different technologies. Update the cost estimates based on the pilot programs.

3. Develop a pilot program to evaluate the performance of RDII technologies, including detailed characterization of areas contributing to RDII by flow monitoring, smoke-testing, and sewer inspections. Develop an RDII reduction pilot program for source control by disconnecting storm water connections and sewer/manhole relining. Establish a baseline performance of the source control measures, and update the cost estimates based on pilot programs.

4. Refine the H&H model and relevant input datasets to further improve the accuracy of the evaluation tools. Develop a flow monitoring program for the next two wet weather seasons, and improve the H&H model calibration and validation using the flow monitoring data. Refine the surface elevation information in the H&H model using updated survey or LIDAR data. Use updated building and curb boundaries to improve the characterization of surface flow routing system-wide. Use surface flooding datasets (pictures, YouTube videos, etc.) of any recent historical flooding events to further calibrate and validate the 2D surface flooding results.

5. Confirm the performance of remaining projects in the CSSIP, and refine any projects or add new projects if necessary. Based on the pilot programs for Green Infrastructure and RDII technologies, develop a strategic longer-term program for implementation of these technologies to achieve a more cost-effective solution to local and system-wide flooding problems. Table 1-1 presents the prioritized list of CSSIP projects and costs; GI and RDII programs and their annual implementation costs for the entire implementation period of the adaptive management strategy. Figure 1-2 presents the cumulative costs of all the CSSIP projects, GI and RDII programs.

Table 1-1: Prioritized List of all Projects and Costs

Schedule	Project Name	Project Cost ($)[1]	Annual Cost ($)[1]	
			Green Infrastructure	RDII
	Existing System Optimization Project	$1,390,000	$250,000.00	$1,250,000
	3-1 Bidwell & Freeport	$1,470,000	$250,000.00	$1,250,000
	4-2 Sutter Middle School Storage	$19,340,000	$250,000.00	$1,250,000
	4-1 McKinley Park Storage	$21,030,000	$250,000.00	$1,250,000
	5-3 W and 25th Street Storage	$12,390,000	$250,000.00	$1,250,000
	1-6 24th Street Storage	$8,170,000	$500,000.00	$2,500,000
	1-3 9th Street from G to L Streets	$3,940,000	$500,000.00	$2,500,000
	1-4 14th Street Storage	$4,490,000	$500,000.00	$2,500,000
	1-7 Grant Park	$20,580,000	$500,000.00	$2,500,000
	3-4 Sierra School Park	$12,570,000	$500,000.00	$2,500,000
	2-3 4th Avenue Park Storage	$14,200,000	$1,000,000.00	$5,000,000
	2-1 M Street Storage	$11,359,000	$1,000,000.00	$5,000,000
	3-3 Curtis Park Village Storage	$18,390,000	$1,000,000.00	$5,000,000
	4-4 Casita Way	$1,080,000	$1,000,000.00	$5,000,000
	3-5 Beverly Way Storage	$2,260,000	$1,000,000.00	$5,000,000
	2-2 McClatchy Park Storage	$20,600,000	$1,000,000.00	$5,000,000
	6-3 Curtis Park Storage	$15,400,000	$1,000,000.00	$5,000,000
	6-2 Riverside Blvd. Upsizing	$16,620,000	$1,000,000.00	$5,000,000
	1-1 Zapata Park	$10,020,000	$1,000,000.00	$5,000,000
	1-2 G Street and 9th Street	$8,670,000	$1,000,000.00	$5,000,000
	6-1 Freeport Blvd. Upsizing	$2,860,000	$1,250,000.00	$6,250,000
	3-2 Land Park Tributary Area Upsizing	$7,550,000	$1,250,000.00	$6,250,000
	6-4 11th Street Installation	$5,140,000	$1,250,000.00	$6,250,000
	3-6 17th and Bidwell Storage	$7,400,000	$1,250,000.00	$6,250,000
	4-3 P Street Storage	$3,560,000	$1,250,000.00	$6,250,000
	3-7 Target Parking Storage	$8,970,000	$1,250,000.00	$6,250,000
	1-5 N & 22nd Street Upsizing	$2,580,000	$1,250,000.00	$6,250,000
	5-2 T Street and 28th	$510,000	$1,250,000.00	$6,250,000
	5-1 T Street and 20th	$670,000	$1,250,000.00	$6,250,000
TOTAL		$263,209,000	$25,000,000	$125,000,000

[1] In 2013 dollars

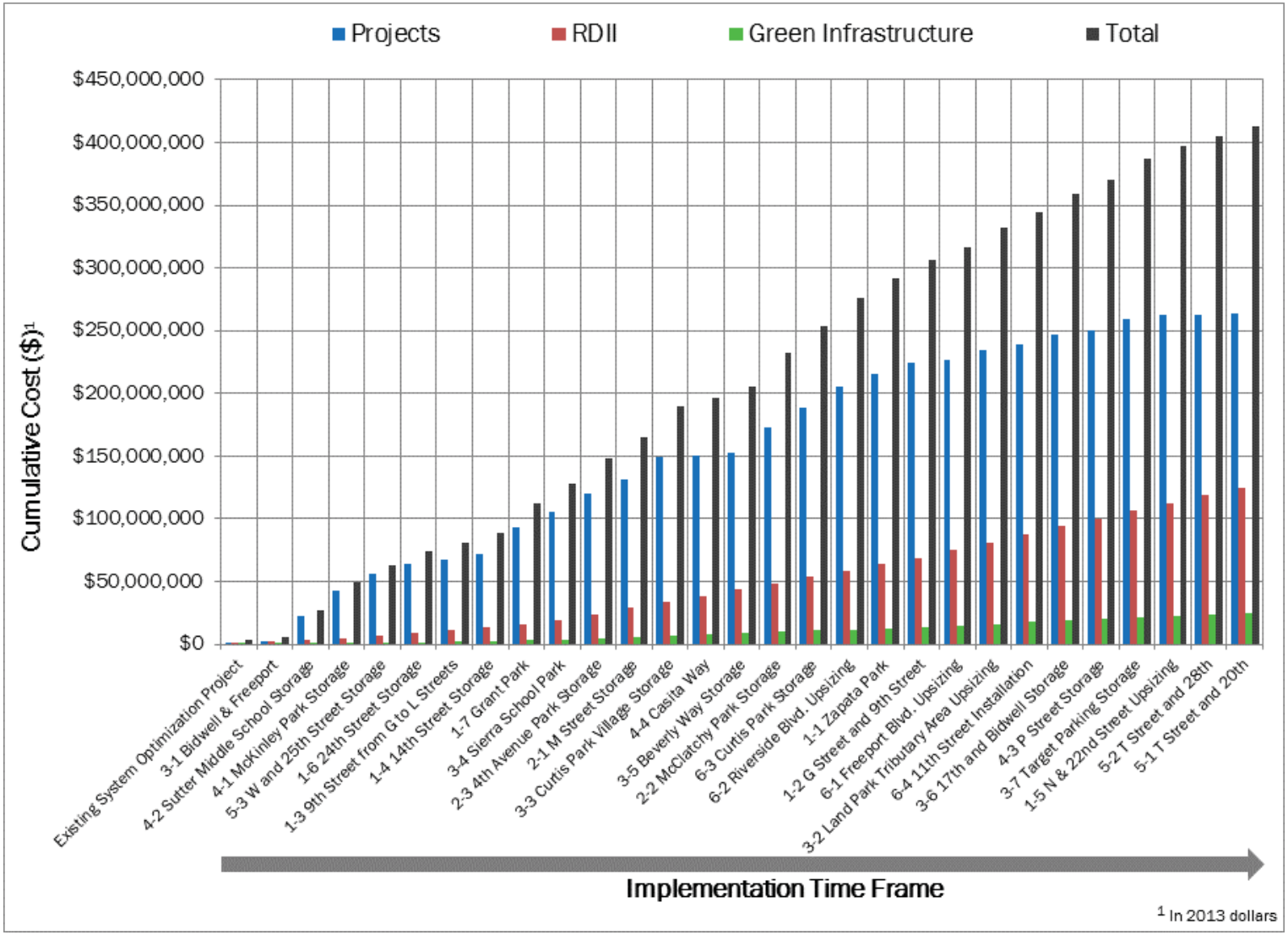


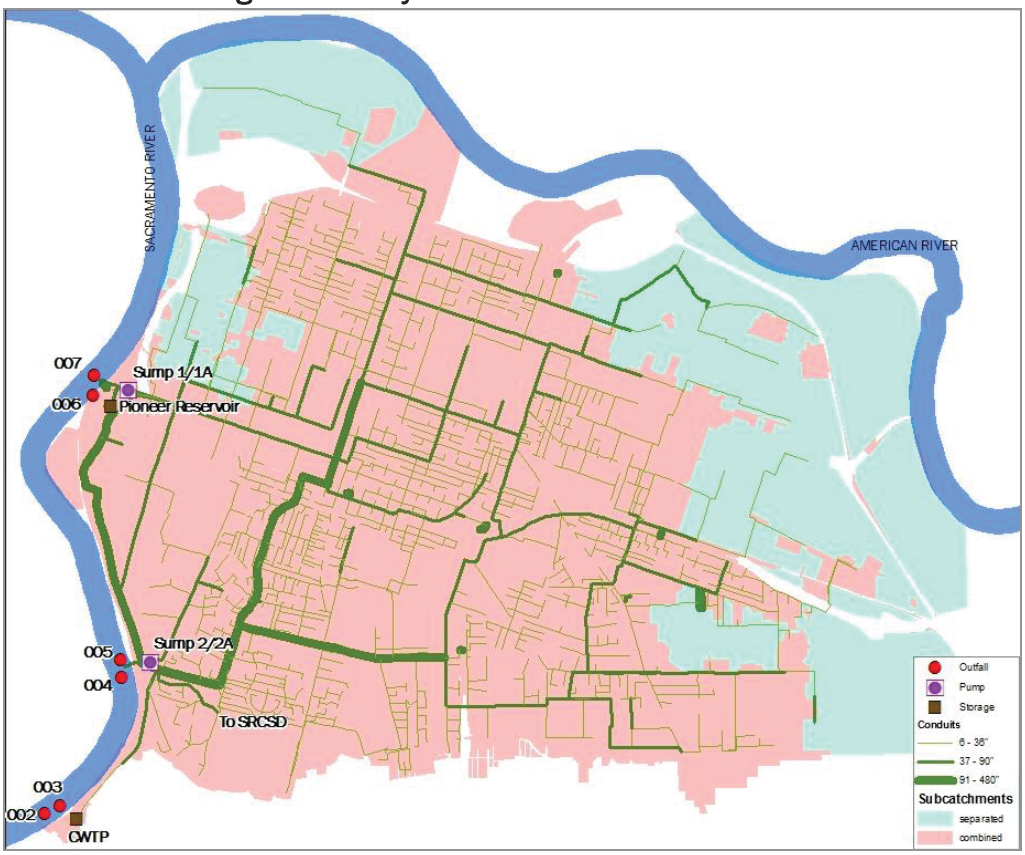
Figure 1-2: Cumulative Costs of all Projects

2. PURPOSE AND GOAL OF THE COMBINED SEWER SYSTEM IMPROVEMENT PLAN UPDATE

2.1 Introduction

The City of Sacramento (City) owns and operates a combined sewer system (CSS) that conveys residential and commercial wastewater and storm water runoff from approximately 11.7 square miles in downtown Sacramento, East Sacramento, Oak Park, and the Land Park area (Figure 2-1). There are 5.8 square miles of separated areas of the City north, east, and south of the CSS that contribute sanitary flows to the CSS. The City also includes approximately 76 square miles of separated areas that are not served by the CSS. The CSS serves approximately 205,000 people. The CSS includes four key facilities to manage the collected flow: Sumps 1/1A, Sumps 2/2A, Pioneer Reservoir, and the Combined Wastewater Treatment Plant (CWTP). Sumps 1/1A and 2/2A pump up to 60 million gallons per day (mgd) of flows to the Sacramento Regional County Sanitation District's Regional Wastewater Treatment Plant (SRWTP). Pioneer Reservoir and CWTP provide additional storage and, when needed, primary treatment, and disinfection of combined sewage prior to discharge to the Sacramento River. The CSS is regulated under the August 2015 National Pollutant Discharge Elimination System (NPDES) waste discharge permit, No. CA0079111. The permit allows for CSS discharge to the Sacramento River at six locations: two for primary treated (plus disinfection) effluent, and four that can discharge untreated combined sewage. The secondary treated effluent from SRWTP is discharged to the Sacramento River at a permitted location under a separate NPDES permit.

Figure 2-1: City of Sacramento CSS Overview



The Combined Sewer System Improvement Plan (CSSIP) Update is an ongoing multi-year project intended to evaluate and provide recommendations for projects to alleviate flooding in the CSS area during a 10-year storm event and to prevent structure flooding during the 100-year storm event. Phase 1 of the CSSIP Update collected flow monitoring data and developed and calibrated the Hydrologic and Hydraulic (H&H) of the CSS area and areas tributary to the CSS that contribute sanitary and infiltration/inflow-derived flows. Phase 2 uses the H&H model to evaluate and recommend projects to achieve the City Department of Utilities (DOU) flood protection goals.

2.2 Background

In June 1990, the California Regional Water Quality Control Board (RWQCB) issued a Cease and Desist Order (CDO) requiring the City to reduce Combined Sewer Outflows (Outflows) from the CSS into City streets and properties without increasing untreated Combined Sewer Overflows (CSOs) to the Sacramento River. In 1995, based on studies and benefit/cost analyses, the City Council adopted a LTCP titled the Combined Sewer System Improvement Plan. The CSSIP met the requirements of the CDO, the Environmental Protection Agency's (EPA) Combined Sewer Overflow Control Policy, and the City's adopted goals for the CSS. That same year, the RWQCB approved the CSSIP, rescinded the CDO, and issued a National Pollutant Discharge Elimination System (NPDES) permit that mandated implementation of the CSSIP and protection of the water quality in the Sacramento River.

The 1995 CSSIP recommended capacity expansion of Sump 1/1A and Sump 2, conversion of Pioneer Reservoir to a primary treatment facility, and construction of local storage projects in selected flooding/outflow locations. These projects have been completed and contributed significantly to reducing untreated CSO discharges. In March 2002 and May 2003, the City provided status updates of the CSSIP summarizing current projects and undertakings toward meeting the CSSIP goals that were outlined in the original 1995 CSSIP.

On January 28, 2010, the California RWQCB issued a revised NPDES permit that required the City to continue implementation of the CSSIP, with interim goals to be met as progress is made toward the final LTCP goal of minimizing street flooding during a 10-year storm event and to prevent structure flooding during the 100-year storm event.

The City's CSSIP satisfies the presumption approach of the Clean Water Act's Water Quality-Based Effluent Limitations requirements, regulated under EPA's 1994 Combined Sewer Overflow Policy. Computer model simulations of the CSS during a statistical "typical" year show (presume) that 100% of the City's CSS flows are captured for treatment, exceeding the requirements that 85% (by volume) be captured for treatment. Treatment, at a minimum, includes primary clarification or equivalent, solids and floatables disposal, and disinfection. Monitoring during the 2010/2011 and 2011/2012 wet seasons showed approximately 97% of CSS flows receiving secondary treatment and less than 0.1% of the annual CSS

volume discharged without treatment. Furthermore, the City's June 2013 Water Quality Assessment Report[2] that evaluates compliance with water quality objectives concluded that the infrequent and short duration CSS overflow discharges do not impact applicable receiving water beneficial uses.

2.3 Purpose

Current CSSIP activity seeks to achieve CSS intermediate LTCP goals without increasing untreated CSO discharges. Those sequential goals are summarized below, working toward a final goal of minimizing street flooding during a 10-year storm event and preventing structure flooding during the 100-year storm event.

- Obtaining protection from a 5-year storm in the six areas of worst flooding
- Obtaining protection from a 5-year storm throughout the CSS area
- Obtaining protection from a 10-year storm in the six areas of worst flooding
- Obtaining protection from a 10-year storm event throughout the CSS

This report presents a summary of the analytical process and basis for recommended projects to achieve flood protection goals.

2.4 Content

This report summarizes the history and background of the CSSIP and the analytical process, and recommends capital infrastructure improvements.

- Section 1 provides an executive summary of CSSIP Update.

- Section 2 presents the background and purpose of the CSSIP.

- Section 3 summarizes the tools used to develop the CSSIP Update.

- Section 4 describes the CSS improvement planning process that includes evaluating performance of the existing system, developing and implementing performance goals and metrics, and evaluating prospective projects relative to those metrics.

- Section 5 describes in detail each project recommended as part of the CSSIP, as well as general planning-level feasibility, purpose, and performance of each project. It also describes the programs proposed in addition to the projects.

- Section 6 presents planning-level cost estimates for the projects and programs, the cost assumptions, and factors used to develop the estimates.

[2] City of Sacramento Combined Sewer System (CSS) Water Quality Assessment. June 25, 2013.

- Section 7 describes the benefits of the recommended projects in terms of meeting the interim and final goals of the CSSIP. This section explains and illustrates how a certain grouping of projects can improve system performance to meet the interim and final goals.

- Section 8 identifies considerations for developing and prioritizing an implementation schedule and provides an implementation schedule for the recommended projects and the total planned cost of projects per the implementation schedule.

- Section 9 summarizes this report and any additional recommendations for continuous improvement of the CSS.

3. TOOLS FOR DEVELOPING THE CSSIP UPDATE

Since the early 1980s, the DOU has used an H&H numerical computer simulation model for its CSS. An H&H model provides a numerical representation of the different hydrologic and hydraulic processes in the CSS and allows simulation of different design conditions and scenarios for evaluating prospective improvements to the CSS.

3.1 H&H Model Development History and Background

The DOU's H&H model is a powerful tool to study the CSS, evaluate performance of the system under different design scenarios and storm events, and analyze benefits of any capital or operation and maintenance (O&M) improvement projects to achieve cost-effective solutions.

The DOU has used and maintained an H&H model for the CSS since the 1980s, continuously improving it in accordance with industry standards. Figure 3-1 summarizes the DOU H&H model development history and improvements.

Figure 3-1: H&H Model History and Details

Duration	1980s-2008	2008	2010
Software	EPA SWMM	InfoWorks CS	InfoWorks ICM
Network	400 nodes	1532 nodes	3860 nodes
Special Structures	Simple	Detailed	Detailed
Operational Controls	Simple	Detailed	Detailed & Calibrated
Features/Improvements		Gully Pot	Gully Pot 2D Module

1980–2008: In the early 1980s, the DOU used EPA's Storm Water Management Model (SWMM) as the H&H model, and customized the application to the City's CSS area. The H&H model was referred to as SAC-SWMM. The model was continuously updated, maintained, and used throughout the 1990s and early 2000s. SAC-SWMM was used to simulate and evaluate many of the alternatives considered for the CSSIP, adopted by the City in 1995, to reduce CSOs to the Sacramento River and the discharge of combined sewage to streets, surface areas, and properties (outflows).

2008–2010: In 2008, the DOU entered into a contract with AECOM to provide a CSSIP Update. Phase 1 of the CSSIP Update included refining the H&H modeling software to industry standards. InfoWorks CS 11.0 was selected as the H&H modeling software. The comprehensive effort included extending the physical network representation, updating dry- and wet-weather flow inputs, reviewing and refining model assumptions and techniques, and validating the model. The updated and extended model incorporated approximately three times as many nodes and conduits as the former model, and developed precise representation of all special structures and facilities such as Sumps 2/2A and 1/1A/1B, Pioneer Reservoir, and all diversions and sumps. Dry- and wet-weather flows were

represented based on monitoring data collected in 2009. Hydraulic parameters that influence the simulation of level, velocity, and flow were adjusted to produce a calibrated and verified model.

The InfoWorks CS "flood cone" method was used to represent flooding on the land surface in association with surface topography once the hydraulic grade line (HGL) exceeded the manhole rim elevation. This "flood cone" was customized to represent road and sidewalk topography. Additionally, a "gully pot" representation of manhole and inlets was setup in InfoWorks CS to distinguish and track CSS outflows from the system to the surface versus surface runoff that cannot enter a surcharged CSS. A "gully pot" node has two elements: an underground element connected to the conduits and an overland element connected to the sub-catchment. The two elements are connected to each other. When the HGL in the overland element is higher than in the underground element, storm water flows from the overland to the underground element. When the level in the underground element is higher, combined sewage flows from the underground to the overland element. A head-discharge curve specified by the modeler determines the flow rates between the two elements in relation to their relative levels.

2010–Present: In 2010, the DOU continued to build on the CSS modeling work conducted by AECOM and the DOU over the last 2 years. The model was augmented with the latest planning information, the physical network representation was developed in more detail (most manholes and pipes included) in the six wet areas, and the operational controls for Sump 2/2A that were developed in detail in Phase 1 were calibrated using recent wet-weather events. In 2013, the H&H model was converted from InfoWorks CS 11.0 to InfoWorks ICM 4.0 to enable use of a two-dimensional (2D) module to more precisely represent the surface flows in the CSS area. This latest H&H model in InfoWorks ICM 4.0 was used for evaluating prospective capital improvement projects to potentially be part of the CSSIP Update.

InfoWorks ICM's 2D module replaced the "flood cone" method with a surface mesh that represents surface topography. Excess flows on the surface can be routed based on open-channel hydraulics. This 2D module provides a useful, easily interpreted graphical representation of the extent of flooding and the surface routing of flows during storms.

Model Documentation References: The H&H model development and validation that has been ongoing since 2008 has been documented in a number of technical memoranda (TMs), listed here:

- CSSIP Phase 1 – Model Update and Validation, April 13, 2010
- CSSIP Phase 2 – Task 2, Sewer Flow Projection, July 11, 2011
- CSSIP Phase 2 – Task 3, Model Sensitivity Analysis, November 17, 2011
- CSSIP Phase 2 – Task 7, CSS Discharge Analysis, May 28, 2013

3.2 Design Storm Characteristics

After an H&H model is developed and calibrated, design storms can be used to evaluate the performance of the system and identify improvements. A design storm is a selected storm

event, of specific duration, described in terms of the probability of equaling or exceeding a certain magnitude in any given year. Design events are also commonly known by the reciprocal of probability of occurrence in any given year based on the average number of years between occurrences (e.g., a 1 in 5-year probability of occurrence, or "5-year storm"). A design storm is defined by the storm duration, total depth (volume), and peak intensity.

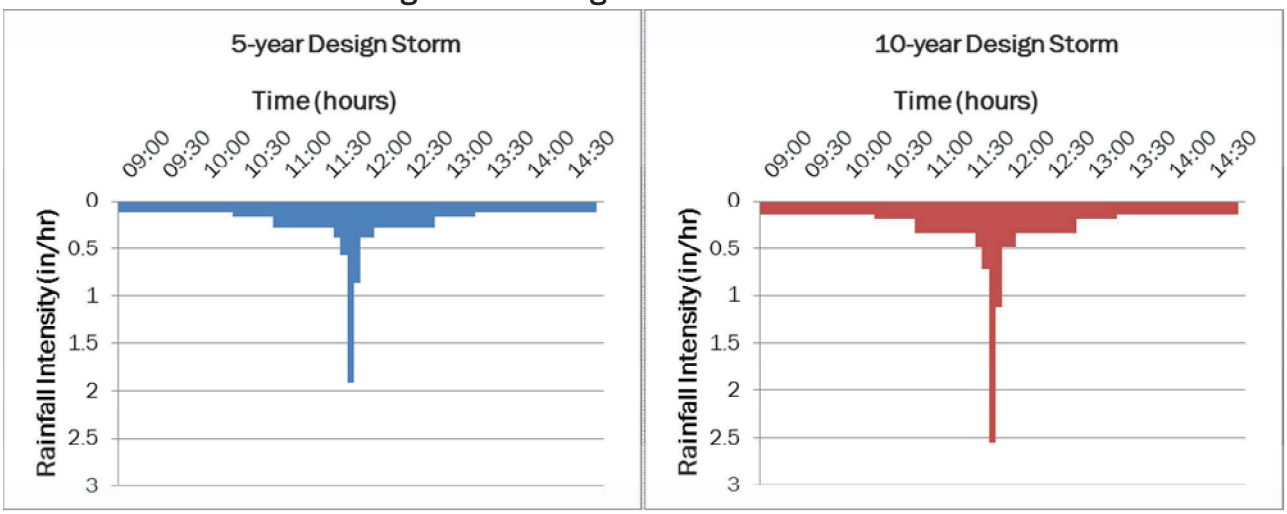
The City's 1995 CSSIP requires the City to progress toward minimizing flooding and obtaining protection for a 5-year design storm as an intermediate goal, with protection for a 10-year design storm and prevention of structure flooding during the 100-year storm event as final goals.

The City and County of Sacramento's Drainage Manual has depth (magnitude), intensity and duration recommendations for design storms. Based on the Drainage Manual, a 6-hour duration hyetograph was used for all design storm frequencies and applied equally across the entire CSS study area. Table 3-1 documents the duration, depth, and peak intensity for the 5-year and 10-year frequency design storms. Figure 3-2 presents the temporal distribution of the design storms.

Table 3-1: Design Storm Details

Design Storm	5-Year	10-Year
Duration (hours)	6	6
Depth (inches)	1.3	1.6
5-Minute Peak Intensity (inches per hour)	1.92	2.56

Figure 3-2: Design Storm Distribution



3.3 Typical-Year Rainfall Characteristics

Another useful rainfall time series used to analyze the CSS is a "typical" rainfall year, which, when used in conjunction with the H&H model, can assess how the CSS will perform on an average annual basis. Simulating a typical rainfall year provides an estimate of average annual discharge frequency and volume from the City's NPDES permitted outfalls as well as other annual CSS performance statistics.

Development of a "typical" rainfall year is documented in the Task 7 – CSS Discharge Analysis[3] Technical Memorandum. Based on that analysis, the typical rainfall year for the City includes 29 storms with a total annual rainfall depth of 18.1 inches.

[3] Task 7 – CSS Discharge Analysis of Combined Sewer System (CSS) Improvement Plan Update – Phase 2. May 28, 2013.

4. ASSUMPTIONS AND PERFORMANCE CRITERIA

4.1 Modeling Assumptions

This section provides documentation of the H&H model inputs and assumptions used for the CSSIP analyses. The inputs and assumptions used for any H&H model simulation can affect the quality of the model results, and, therefore, it is important to use a consistent set of inputs and assumptions.

Model Version: The H&H model has been continuously updated and improved since the 1980s; advancements and refinements will continue. Table 4-1 presents a chronology of recent updates.

Table 4-1: H&H Modeling History

Updated Model Date	TMs Using the Model Version	Key Changes Since Last Version
June 2011	CSSIP Phase 2 – Task 2, Sewer Flow Projection, July 11, 2011 CSSIP Phase 2 – Task 3, Model Sensitivity Analysis, November 17, 2011	Updates to the sanitary flow assumptions Model refinements in Wet Areas 3, 4, and 5
Jan 2013	CSSIP Phase 2 – Task 7, CSS Discharge Analysis, May 28, 2013	Model refinements in Wet Areas 1, 2, and 6 Added CWTP Storage and Pumping, and 84-inch-diameter interceptor from Sump 2/2A to CWTP and outfall 002 at CWTP Calibration of operational controls for Sump 2/2A using monitoring data
June 2013	CSSIP Phase 2 – Task 6 Draft CSSIP, January 2014	Additional model refinements in Wet Area 1

Sanitary Flow Assumptions: Based on the Sewer Flow Projection[4] Study, the sub-catchments in the model represent sanitary flow loadings from an Equivalent Single-Family Dwelling (ESD). Two estimates for sanitary flows were developed to use in the H&H model for two different scenarios:

- 140 gpd/ESD for current baseline conditions
- 310 gpd/ESD for future planning conditions

[4] CSSIP Phase 2 – Task 2, Sewer Flow Projection, July 11, 2011

The baseline conditions model will use the 140 gpd/ESD estimates, and the future planning conditions, including CSSIP projects, will use the 310 gpd/ESD estimates. Increasing the ESD unit rate is a means of accounting for projected additional future population. However, CSSIP scenarios may include water conservation efforts that may be represented by using a lower sanitary flow estimate.

Base Flow Assumptions: Base flow (i.e., infiltration during dry weather) estimates were developed during CSSIP Phase 1, and no changes have been made to these estimates. Base flow in the model does not have a daily or seasonal diurnal variation. The groundwater infiltration base flow was set to 300 gpd/acre.

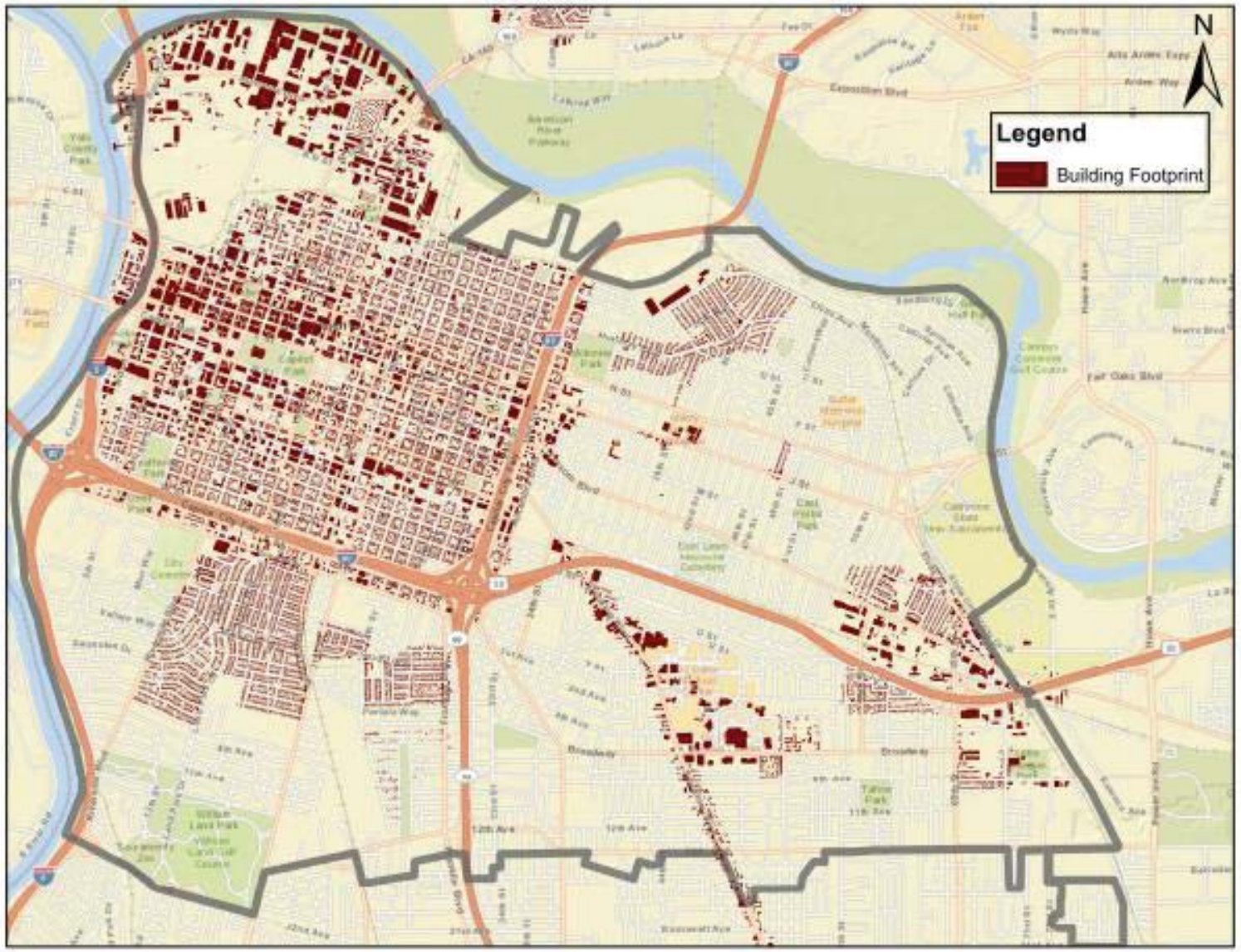
Downstream Boundary Conditions: The outfalls to the Sacramento River are currently set up as free discharge outfalls in the H&H model as boundary conditions. River levels are not known to create high tail water conditions that would affect the discharge capacity of the outfalls. Outfalls 003, 005, and 007 are only used for emergency bypass and therefore did not need to be represented in the model.

2D Topographic Mesh Setup: Two datasets are used for generating a 2D surface topography mesh that enables using the 2D module of InfoWorks ICM, and enables evaluating routing of CSS and storm flows on the surface. These two datasets are as follows:

> *Ground Model*: A ground elevation model covering the entire study area was provided by DOU. The elevation model for the study area is a digital terrain model that represents the bare ground surface without any objects such as plants and buildings. This digital terrain model has a 10- by 10-foot resolution with a high vertical resolution.
>
> *Buildings Footprint*: DOU is currently updating the buildings footprint using information from the water meter program. The coverage of the latest buildings footprint shapefile (November 2013) is presented in Figure 4-1.

The 2D mesh is generated using a two-dimensional triangle meshing functionality with a polygon that defines the boundary for the 2D analysis. Heights at the vertices of the generated mesh elements are calculated by interpolation from the ground model. The building footprints are used to create voids in the 2D mesh. Since building footprints are not available throughout the City, the 2D mesh is more detailed and accurate in the areas with building footprints. The results from 2D analysis in areas without building footprints will be reviewed carefully with aerial maps to ensure that the surface flows are routing along the streets and sidewalks and not entering buildings that are visible in the aerial maps.

Figure 4-1: Extent of the Latest Building Footprint GIS Data Layer



On-Going Projects: Four on-going (i.e., funded or planned projects from January 2010 through January 2015) were added to the baseline condition of the H&H model. These four projects are described in detail in the CSS Discharge Analysis TM,[5] and are summarized below:

- Project 1 – S. Street Brick Sewer Main Replacement, 14th to 17th Streets: This project was constructed in October 2012, and includes increasing conveyance capacity of the collection system.
    - Upsize existing 48-inch-diameter pipe to 66- and 54-inch-diameter (on S. Street between 14th and 17th Streets).

- Project 2 – 7th Street Sewer Replacement, P to K Streets: This project will be out to bid in early 2014 and is planned to complete construction in 2014. The purpose of this project includes increasing the conveyance capacity of the existing collection system. Project components include the following:
    - Upsize existing 24- and 36-inch-diameter pipe to 72-inch-diameter (on 7th Street from P to L Streets).
    - Upsize existing 24-inch-diameter pipe to 54-inch-diameter (on 7th Street from L Street to K Street).
    - Install new 60-inch-diameter pipe (on L Street to connect 7th Street and 9th Street).

- Project 3 – P Street Sewer Improvements, between 5th and 7th Streets: Construction for this project is substantially complete. The project punch list has been generated and all items completed apart from an as-built submittal. This project will provide additional flow connectivity and routing for better conveyance of upstream flows. The project components include the following:
    - Install new 72-inch-diameter pipe to connect existing 72-inch-diameter pipe on 5th Street.
    - Upsize pipe on P Street from 18-inch-diameter to 72-inch-diameter between 5th and 7th Streets.

- Project 4 – Oak Park Regional Storage Project: Construction of this project will be completed in 2014. This project provides additional storage of 4 million gallons. Project components include the following:
    - Install segments of 120-inch-diameter pipes in parallel to provide total storage of 4 million gallons.
    - A weir sized to allow peak flows to enter the storage facility during 5-year and 10-year design storms.
    - Orifice and flap valve to allow draining the storage facility after the storm event.
    - Additional pump station of 0.06 mgd capacity for additional draining capacity of the storage facility.

---

[5] Task 7 – CSS Discharge Analysis of Combined Sewer System (CSS) Improvement Plan Update – Phase. May 28, 2013.

- The model real-time control was updated so that the pump station would drain the storage facility after the storm event has ended.

4.2 Baseline System Performance

Typical Rainfall Year Results: A detailed comparison of model simulation results with recent storm events, development of "typical" rainfall year, and summary of "typical" year model results are presented in Task 7 – CSS Discharge Analysis.[6] Note that the results presented in this TM were based on the January 2013 H&H model.

The "typical" rainfall year dataset was simulated in the latest (June 2013) H&H model, and the discharge frequency and volume at each outfall to the river was tabulated. Table 4-2 presents the discharge frequency and volume at three of the six outfalls to the Sacramento River and total volume to the SRWTP. Outfall 001 is the SRWTP Effluent Discharge; Outfalls 003, 005, and 007 are only used for emergency bypass.

Table 4-2: Typical Year Results

No.	Storm Event Start Date	Storm Event End Date	Using June 2013 Model			
			Volume through CWTP (002) (MG)	Volume at Sump 2 (004) (MG)	Volume through Pioneer (006) (MG)	Peak Flow through Pioneer (006) >250 MGD
1	10/31/2008, 19:00	11/2/2008, 1:00	3.8	--	101.0	Yes
2	1/11/2009, 0:00	1/12/2009, 5:00	0.2	--	29.9	Yes
3	1/21/2009, 17:00	1/22/2009, 12:00	0.1	--	29.3	No
4	2/16/2009, 21:00	2/18/2009, 6:00	6.6	--	55.0	Yes
5	4/9/2009, 9:00	4/10/2009, 4:00	--	--	6.8	No
6	5/1/2009, 11:00	5/1/2009, 11:00	--	--	3.3	No
Number of Discrete Discharge Events			4	–	6	3
Total Volume of All Discharges (MG)			10.7	0	225.3	-
Total Volume to Secondary Treatment at SRWTP (MG)			6775.5			

MG = million gallons; MGD = million gallons per day; SRWTP = Sacramento Regional County Sanitation District Wastewater Treatment Plant

Note:

(1) Of the six events at Outfall 006, three events received full equivalent primary treatment (peak flow rates <250 mgd) and three events received partial primary treatment (peak flow rates >250 mgd)

CSO discharges from CWTP and Pioneer Reservoir (Outfalls 002 and 006) receive equivalent primary treatment and disinfection. Discharges from Pioneer Reservoir (Outfall 006) in

[6] Task 7 – CSS Discharge Analysis of Combined Sewer System (CSS) Improvement Plan Update – Phase. May 28, 2013.

excess of 250 mgd are considered to only receive partial primary treatment. Discharges from Sump 2 (Outfall 004) are untreated. Based on this modeling analysis, all discharges to the Sacramento River are treated to, at a minimum, partial primary clarification or equivalent, solids and floatables disposal, and disinfection. There are no untreated discharges to the river.

5-Year and 10-Year Design Storms: Both the 5- and 10-year, 6-hour design storms were simulated with the H&H model to evaluate flooding areas, including the six wet areas. The NPDES permit verbally describes the general extent of the six wet areas according to the following:

- Wet Area 1: Downtown, north of Capital Park
- Wet Area 2: U.C. Medical Center area
- Wet Area 3: Area immediately south of Highway 80 between Riverside and Freeport
- Wet Area 4: Area northeast of Highway 99 and Highway 80 interchange
- Wet Area 5: Area northwest of Highway 99 and Highway 80 interchange
- Wet Area 6: Land Park area

The exact boundaries of each wet area are not specified in the permit. The DOU has precisely defined the wet areas according to topography and surface drainage. Figure 4-2 presents the six wet areas as represented in the H&H model.

Baseline Performance Results: A 2D mesh was used to simulate the results of excess flows (i.e., outflows and storm water flows) on the surface, and identify and prioritize areas with flooding problems within each wet area. Table 4-3 summarizes the outflow volume for each of the wet areas and system-wide for the 5- and 10-year storm events. The outflow volume listed in Table 4-3 is the sum of outflows leaving the system and entering the 2D mesh from all model nodes. This metric, along with the maximum flood depth on the 2D mesh, will be useful for comparing the performance of potential CSSIP Update projects with the baseline and the performance goals defined for the CSSIP. The areas with high maximum flood depth on 2D mesh correlated well with DOU's anecdotal observations of flooding.

Figure 4-3 presents maximum flood depth in the 2D mesh for the 5-year, 6-hour design storm for the baseline model. The six wet areas are identified on the map. For the 2D results displayed on the map, the blue color indicates 2 to 6 inches of maximum flood depth, and red indicates greater than 6 inches of maximum flood depth.

Figure 4-2: Model Nodes Indicating Six Wet Areas

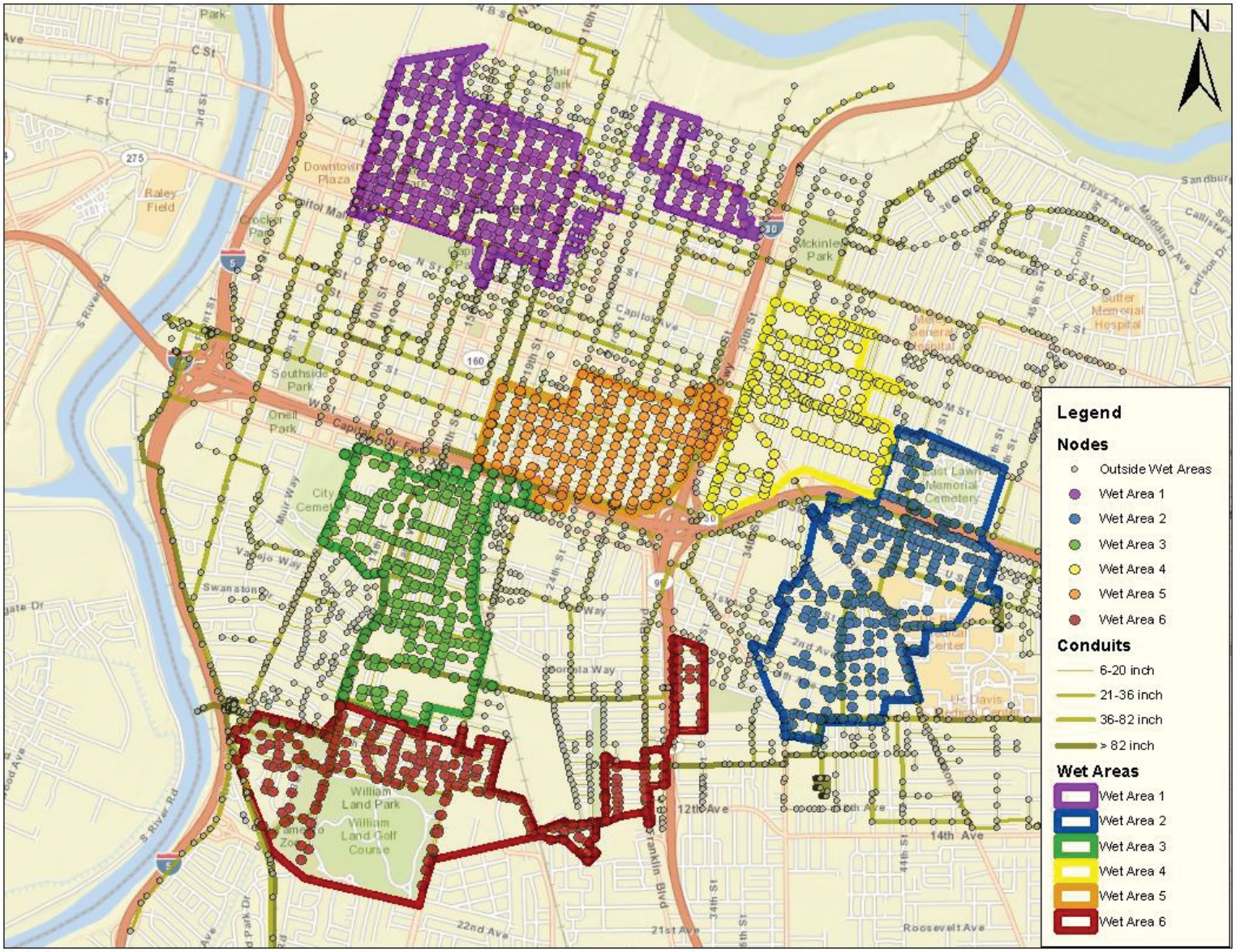


Table 4-3: Baseline Results Summary for Design Storms

Extent	5-Year 6-Hour Design Storm	10-Year 6-Hour Design Storm
	Outflow Volume (MG)	Outflow Volume (MG)
Outside Wet Area	9.1	24.9
Wet Area 1	2.6	6.1
Wet Area 2	2.3	6.4
Wet Area 3	2.5	6.4
Wet Area 4	0.4	1.4
Wet Area 5	1.0	2.5
Wet Area 6	1.0	2.9
System-Wide	18.9	50.6

MG = million gallons

Figure 4-4 presents maximum flood depth in the 2D mesh for the 10-year, 6-hour design storm for the baseline model.

Appendix A presents a large exhibit-style map of the study area for the baseline conditions results for the 5-year, 6-hour and 10-year, 6-hour design storms.

Proposed Project Performance Goals: The definition of flood protection to meet interim and final goals for 5-year and 10-year storms is interpreted as achieving 2 inches or less of flood depth on the surface at any time during the storm event.

Average Annual Rainfall Year Criteria: The suite of projects planned as part of the CSSIP cannot increase the untreated discharge frequency or volume of discharge to the Sacramento River, as that would violate the anti-back-sliding requirements of the NPDES permit. The combination of all CSSIP Update recommended projects is evaluated using the H&H model for the typical rainfall year to ensure that the overall plan does not increase the frequency or volume of untreated discharge to the river during the typical year.

Figure 4-3: Model Baseline Results for 5-Year, 6-Hour Design Storm

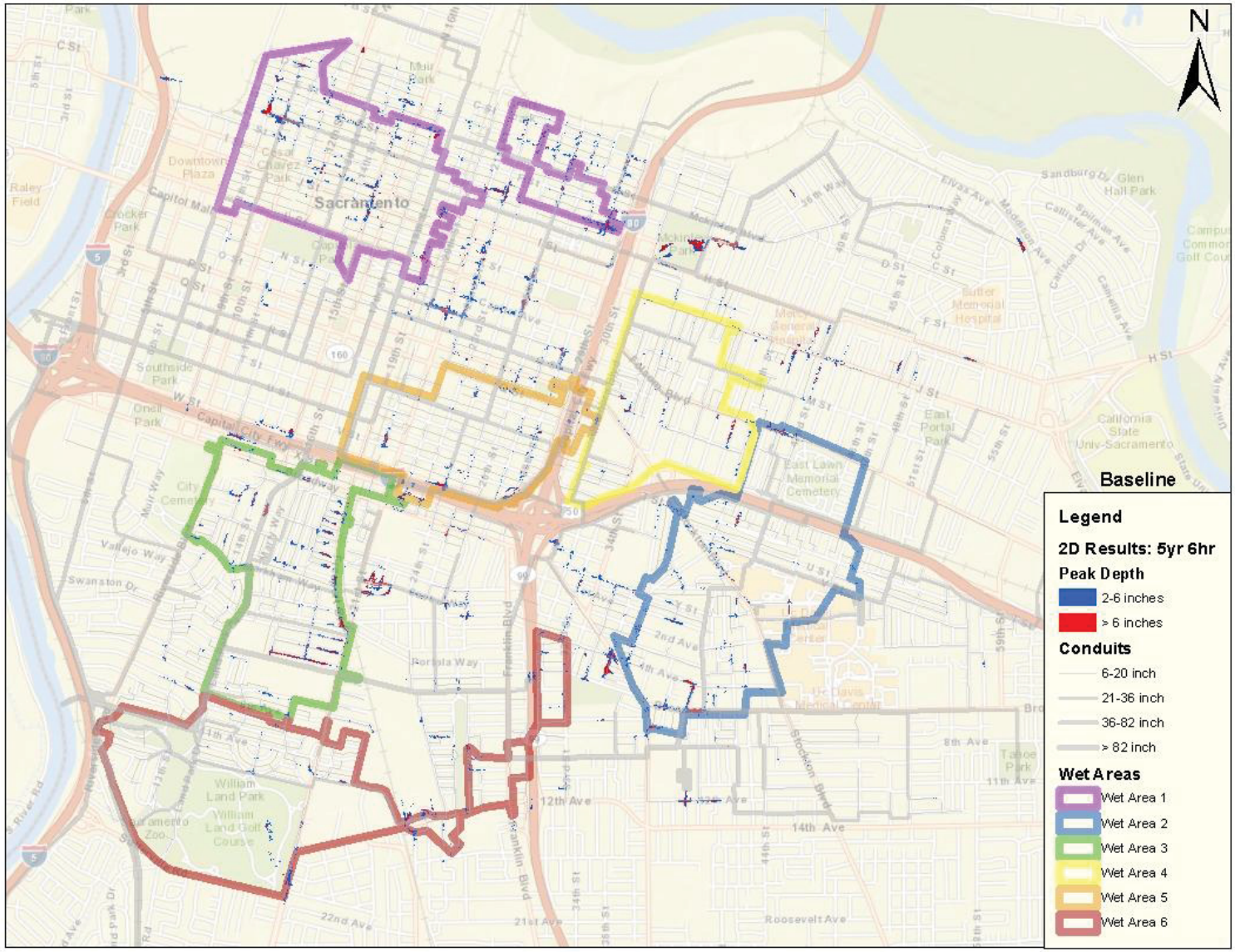
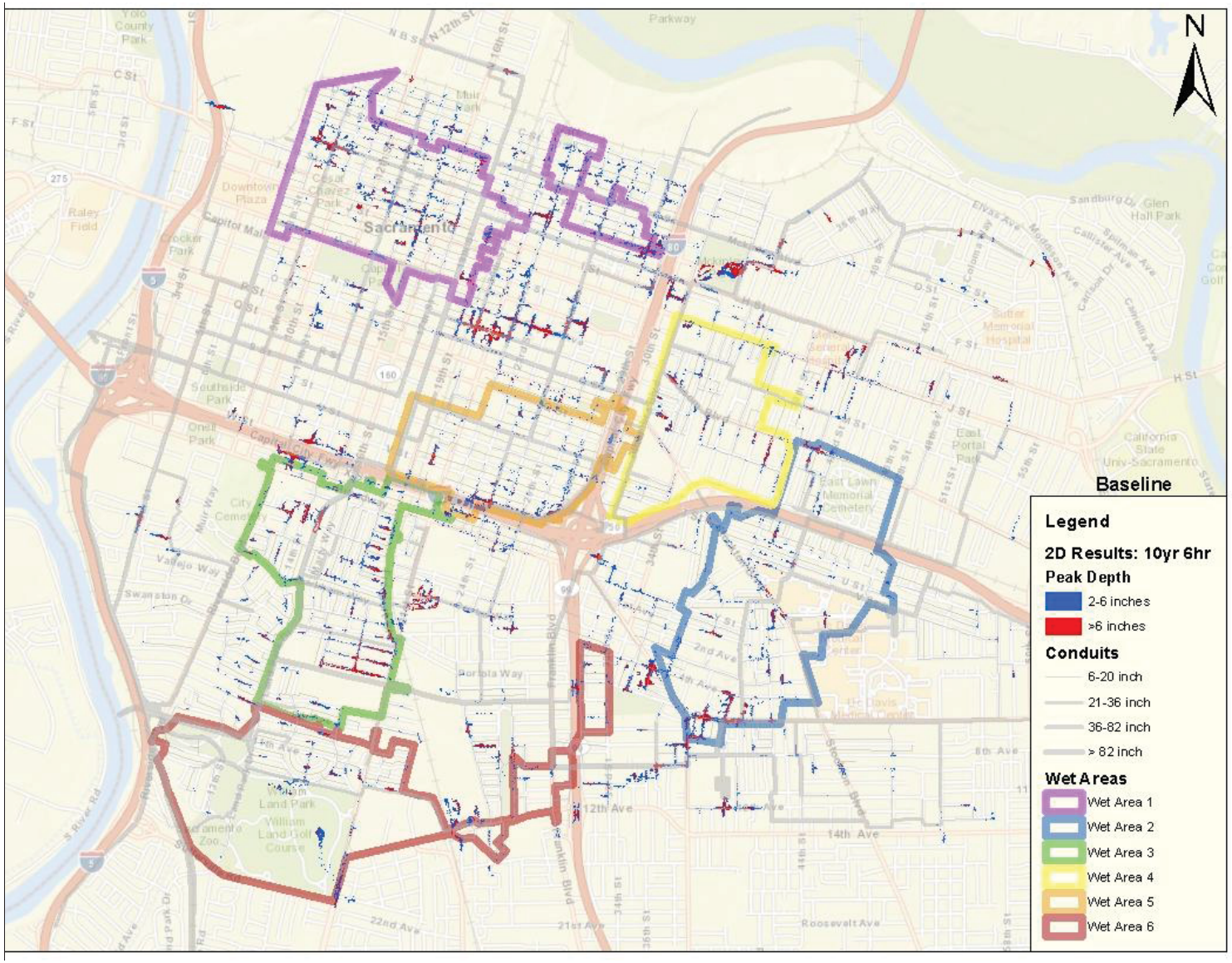


Figure 4-4: Model Baseline Results for 10-Year, 6-Hour Design Storm



5. CSSIP PROJECTS

H&H model results for baseline conditions identified where there was need for CSS improvement within the six wet areas and system-wide. Improvement opportunities were then developed and evaluated based on their ability to provide flood reduction. Criteria included maximum surface flood depth and peak HGL in the system during the 5-year and 10-year design storms. Projects were developed to specifically address localized flooding, but were further refined to additionally provide system-wide benefits downstream and reduce untreated CSO discharges to the river. Two categories of projects were developed and evaluated: storage and conveyance.

Storage: The storage projects are located upstream or downstream of local flooding areas, and are intended to detain flows until the CSS has re-generated capacity (i.e., peak of the storm has passed and HGL in the system has receded from peak conditions) and the storage facilities can be dewatered. The storage projects can be linear or parcel based.

- Linear storage facilities can likely be constructed in the City right-of-way. They can be configured as larger-diameter replacements of existing pipe or as new pipes of larger capacity with a controlled release to detain flows until the downstream system has regenerated its conveyance capacity. Linear storage facilities would be initially sized to maximize the flood reduction in the immediate area and lower the peak HGL below the ground in a 10-year design storm. They could be off-line or in-line. Off-line storage facility connections to the primary conveyance system would be set higher than peak dry weather depths to avoid detaining any dry weather flows in the storage. Factors such as ground cover requirements, right-of-way width, and existing system pipe invert elevations (to which linear storage facilities must connect) were factored into the storage configurations.

- Parcel storage facilities, usually off-line storage, are options for construction underneath parcel lots such as parks, parking lots, and open spaces. Similar to the linear storage, the initial parcel storage size was configured to reduce flood potential and lower the peak HGL below ground in a 10-year design storm. The inflow connection to the existing system would be above the peak dry weather depths to avoid detaining any dry weather flows in the storage. Factors such as maximum footprint of the parcel lot, limited ground cover for storage vault, and inverts of the existing pipes connecting the storage were considered in configuring the parcel storage. A shallow parcel storage that can drain passively by gravity is preferred over deep storage that requires pumping to de-water.

Conveyance: Conveyance projects would generally be located in proximity to or just downstream of localized flooding areas. Their objective would generally be to convey peak flows from and through the flood-prone areas to points downstream with greater capacity. The analysis carefully considered whether the increased conveyance had the potential to cause or exacerbate downstream flooding. If that was determined to be true, the conveyance project(s) were combined with upstream or downstream storage projects to mitigate the downstream flood exacerbation risk. Conveyance projects included upsizing

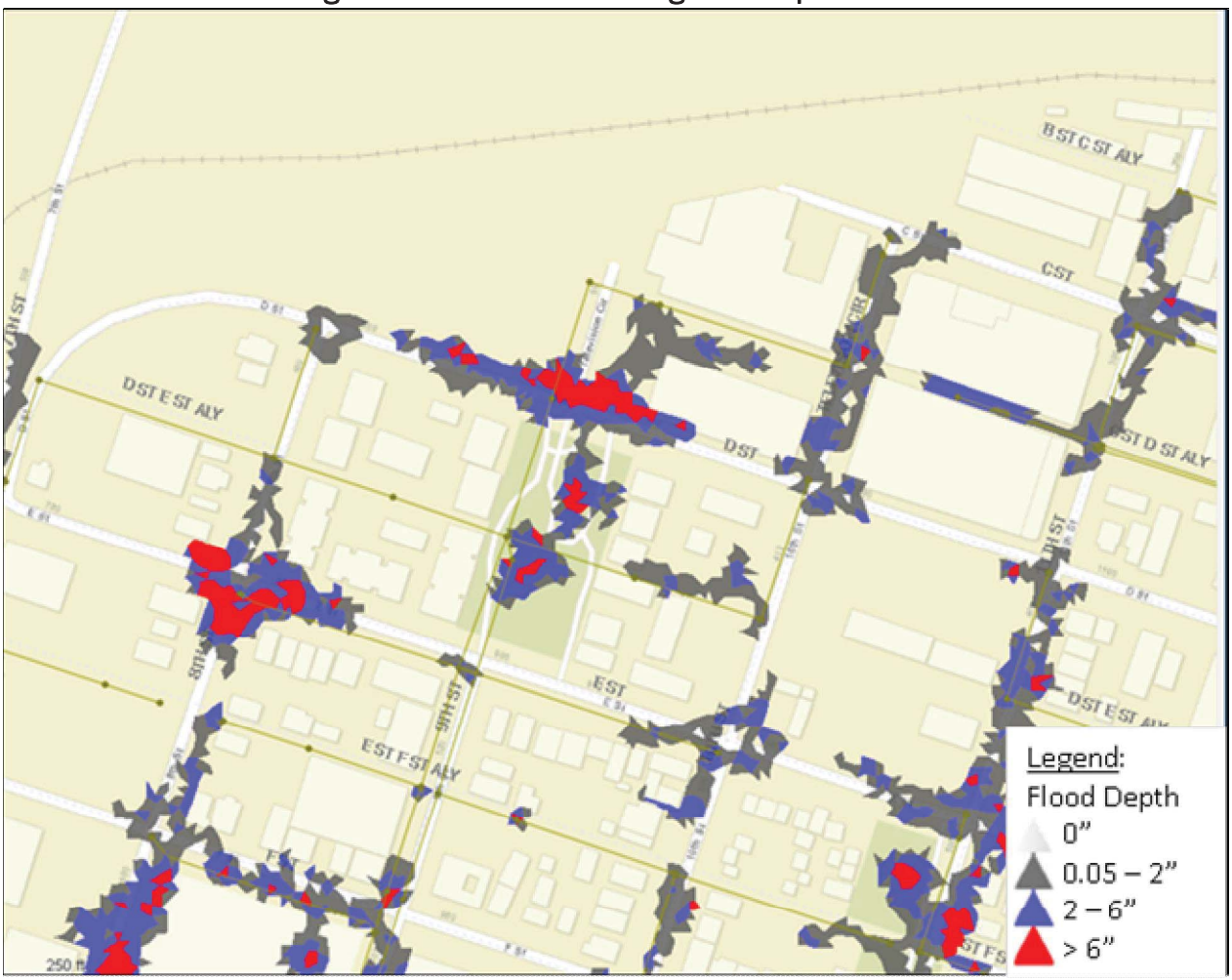
existing pipes or constructing new pipes. Where baseline flooding occurred in a location with no opportunities for storage, a new pipe was sized to convey the 10-year storm design peak flows to the downstream system. Factors such as ground cover requirements, right-of-way width, and existing system pipe invert elevations (to which linear storage facilities must connect) were factored into the storage configurations.

Some of the projects were most effective when grouped with upstream or downstream projects to meet the target flood reduction goals. Evaluations were performed on a system-wide basis to consider benefits associated with GI (runoff source control), rainfall derived infiltration and inflow (RDII) reduction in separate storm water areas, and water conservation. This section describes in detail the considered CSSIP projects, including their components, general feasibility/constructability, system-wide programs, and performance in 5-year and 10-year design storms.

5.1 Project 1-1 (Zapata Park Storage)

Figure 5-1 shows the baseline H&H model results for the area surrounding Zapata Park in a 10-year design storm. The primary flooding in the area occurs on D Street north of the park. The model shows limited conveyance capacity of pipes in the immediate vicinity and downstream of the area, causing flooding.

Figure 5-1: Baseline Flooding Near Zapata Park



This proposed project would involve providing parcel storage at Zapata Park, located on 9th Street between D and E Streets. Figure 5-2 presents the project components.

Figure 5-2: Project Components for Zapata Park Storage

Description: The project would include pipe upsizing and a storage facility. Pipe upsizing would include the following:

- Upsize 1,400 linear feet (LF) of 8-inch-diameter pipe to 16-inch-diameter on D/E Alley
- Upsize 650 LF of 8-inch-diameter pipe to 16-inch-diameter on 10th Street and D/E Alley
- Upsize 650 LF of 8-inch-diameter pipe to 24-inch-diameter on 10th Street and C/D Alley
- Upsize 420 LF of 12-inch-diameter pipe to 24-inch-diameter on 9th Street

The proposed storage facility would include a 0.5 million gallon (mg) combined sewer storage vault underneath Zapata Park. Two weir structures would connect the storage facility to the collection system to convey flows from the flooding areas to the storage facility. The first weir structure would be located at D/E Alley and 9th Street, and the second at C/D Alley and 10th Street. Weir invert elevations would be set above peak dry-weather depths in the primary system pipes. A 0.05 mgd pump would be required to de-water the storage facility.

Benefits: The project would reduce flooding in the immediate area. It would reduce the HGL below the surface for a majority of the manholes in the area. Table 5-1 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-3 through Figure 5-8 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm. Additional projects downstream of this area will further reduce flooding in the area as downstream conveyance capacity is increased.

Table 5-1: Modeled Peak HGL for Node Near Zapata Park

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
D Street and 9th Street	20.5	**21.1**	19.9	19.3	**21.3**	20.5	19.6

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-3: Pre-Project Performance at D/E Alley in 10-Year Storm

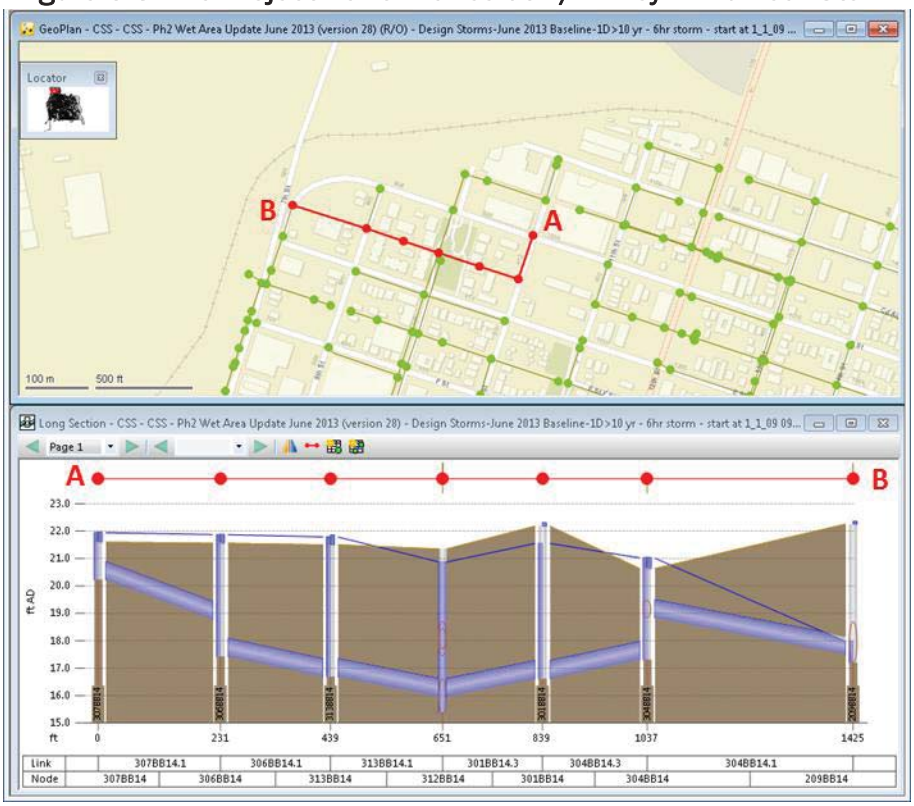


Figure 5-4: Post-Project Performance at D/E Alley in 10-Year Storm

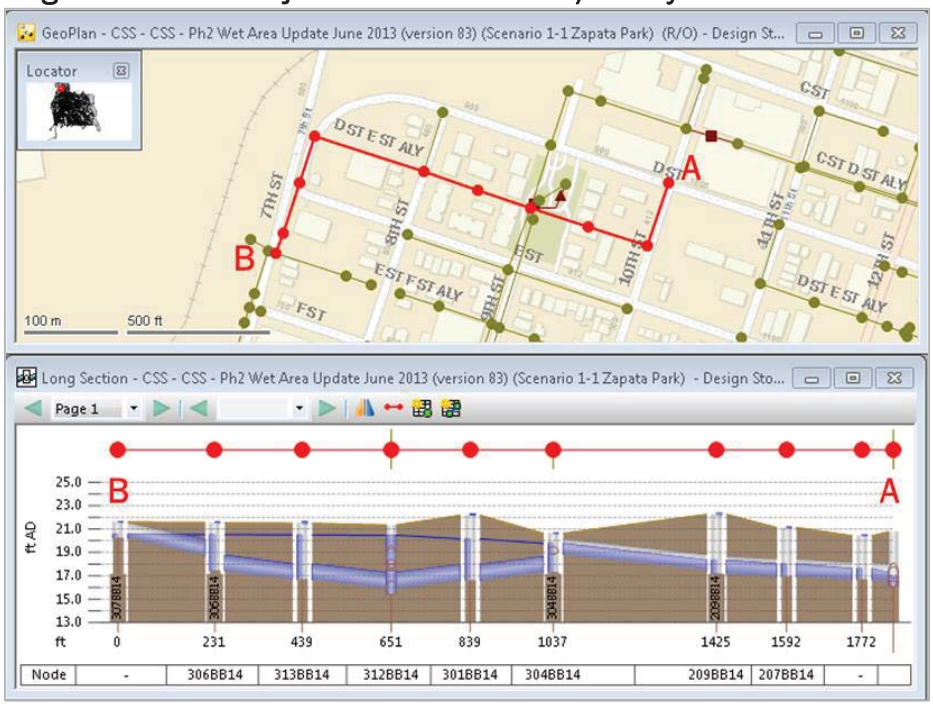


Figure 5-5: Pre-Project Performance at 9th/10th Street in 10-Year Storm

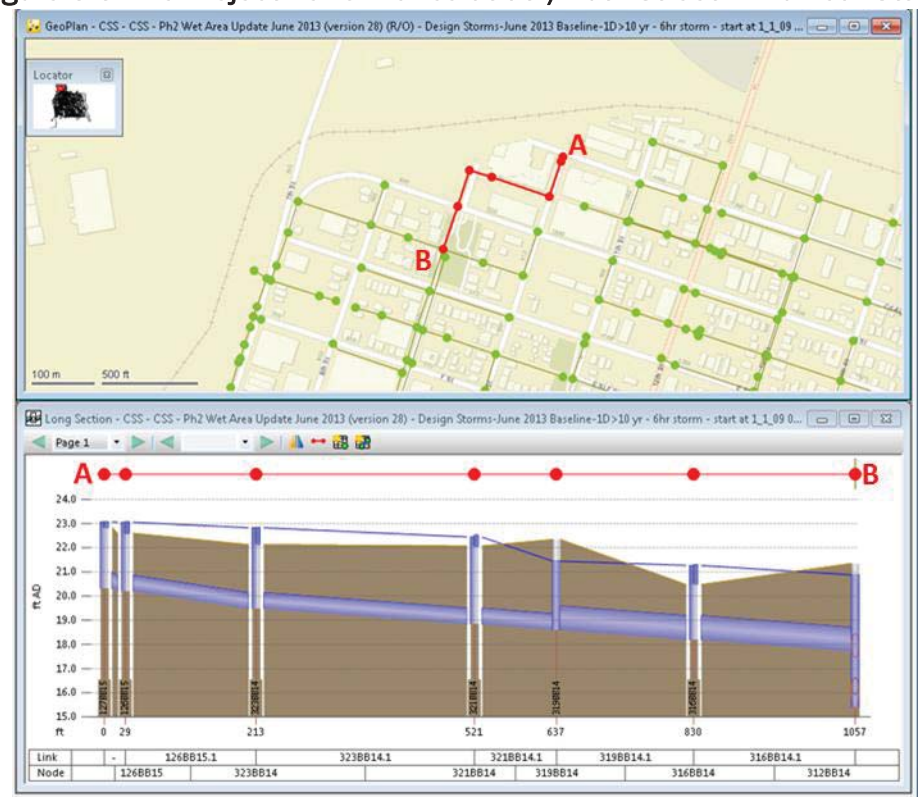


Figure 5-6: Post-Project Performance at 9th/10th Street in 10-Year Storm

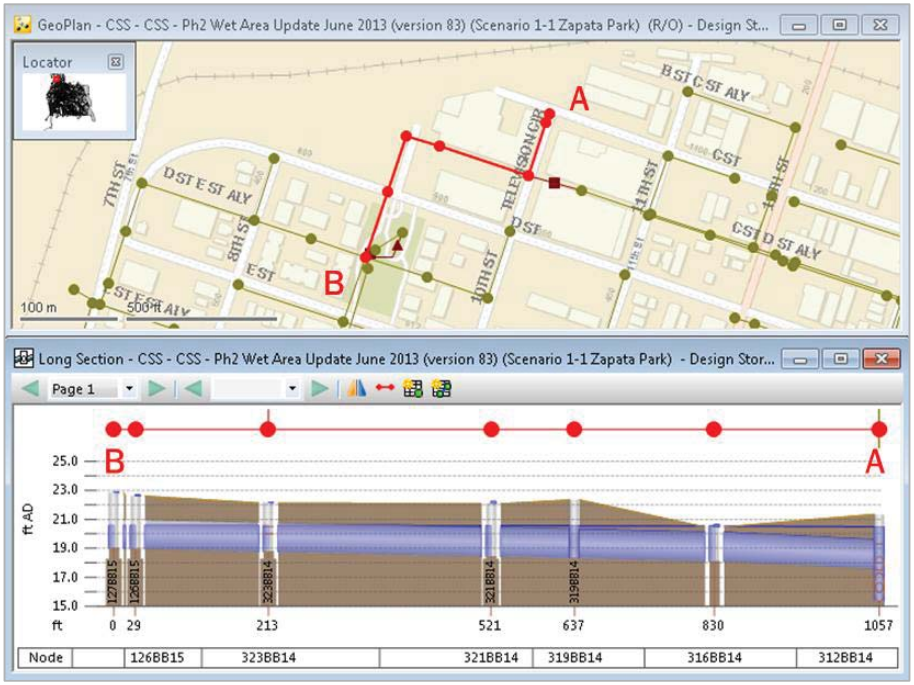


Figure 5-7: Pre-Project Performance at C/D Alley in 10-Year Storm

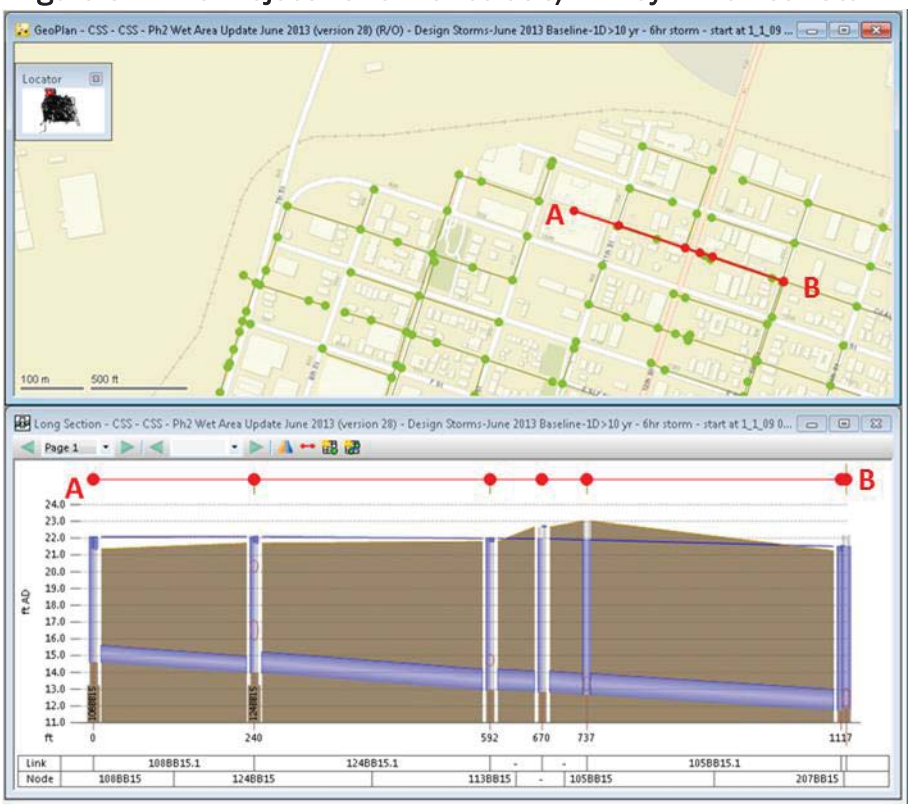
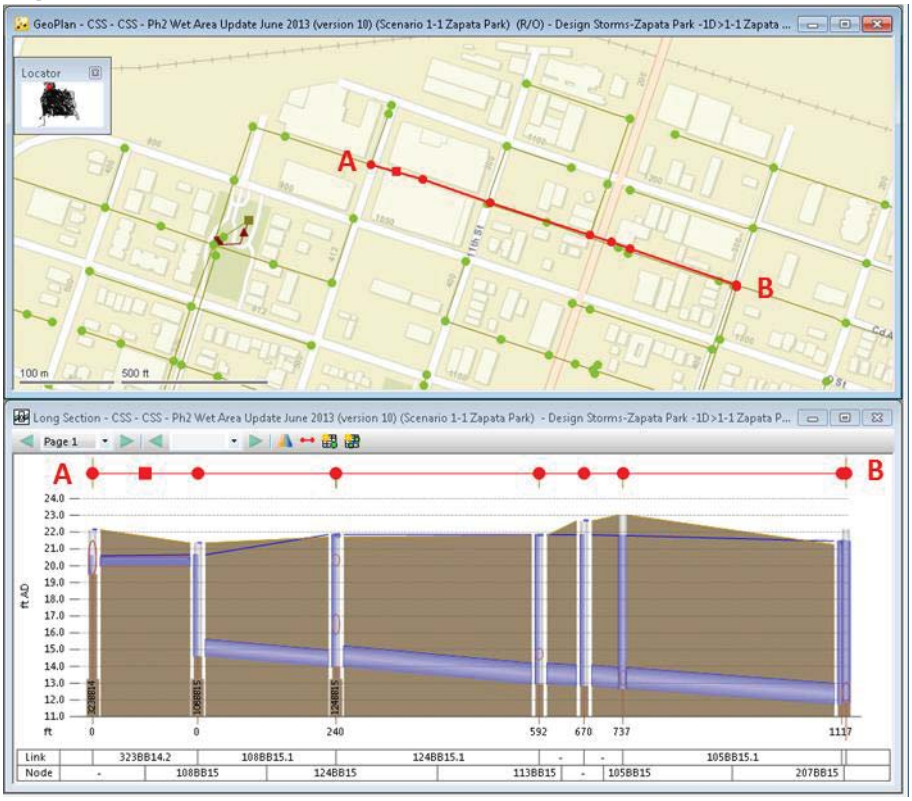


Figure 5-8: Post-Project Performance at C/D Alley in 10-Year Storm



Feasibility/Constructability: Zapata Park is City property, owned and maintained by the Department of Parks and Recreation. Figure 5-9 presents an aerial view of Zapata Park. Based on this aerial view, it appears that the park has a playground that has not been recently rehabilitated. Constructing a storage facility below this playground, and rebuilding the playground, may provide a synergistic opportunity to reduce costs and provide multiple benefits to the neighborhood.

Figure 5-9: Zapata Park Aerial View

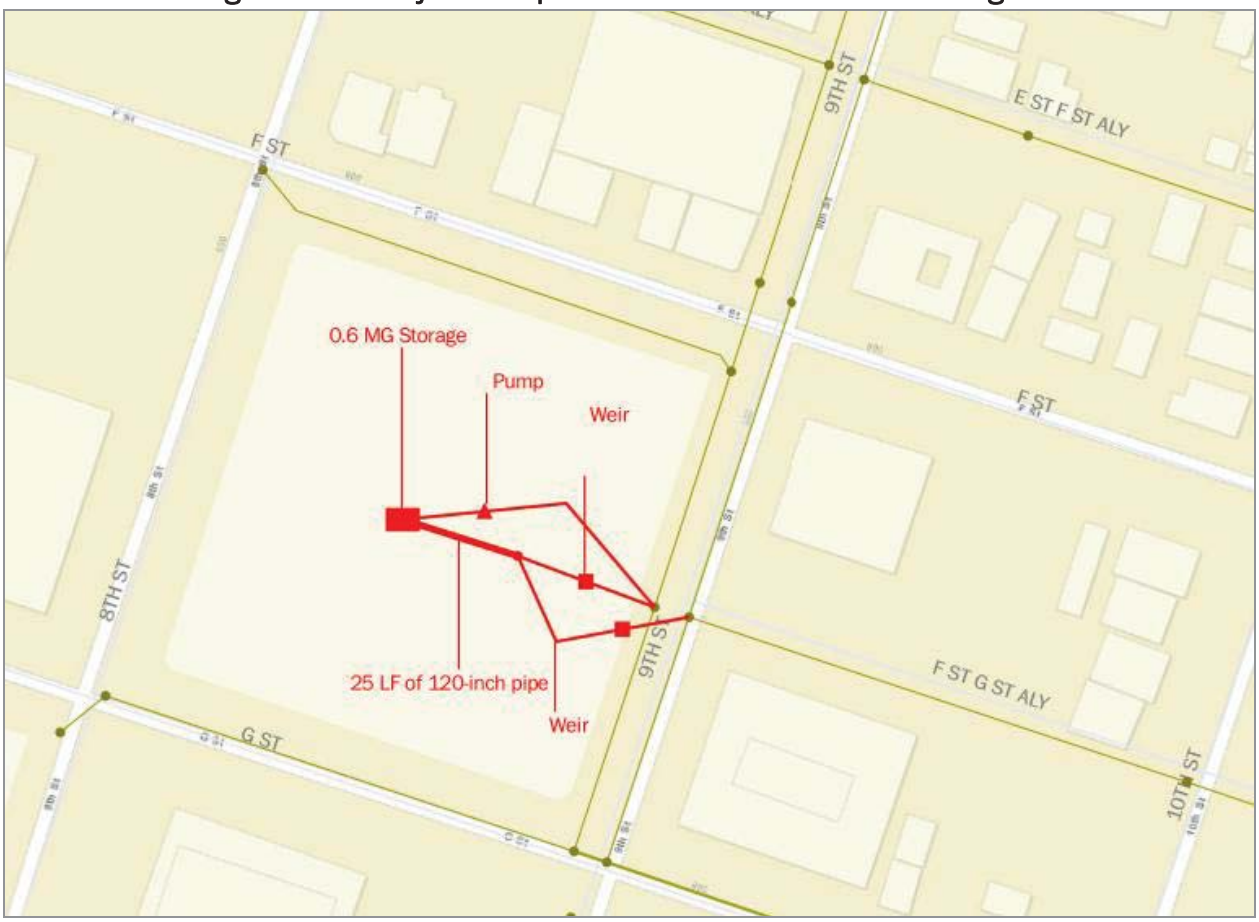
5.2 Project 1-2 (G St. and 9th St. Parking Lot)

The H&H model indicates flooding at the intersection of 9th Street and G Street for both 5- and 10-year design storms. Figure 5-10 shows the baseline H&H model results for the area surrounding G and 9th Street in a 10-year design storm. The pipes upstream and downstream of this intersection are completely surcharged, and combined sewage exits the manholes during the design storms.

Figure 5-10: Baseline Flooding at G & 9th Street Parking Lot

Based on suggestions from the DOU, a parcel storage facility was evaluated on a County of Sacramento-owned parking lot that is situated northwest of this intersection. Performance of this parcel storage facility was found to be effective in reducing flood volume in both design storms. Figure 5-11 presents the system components for this project.

Figure 5-11: Project Components for G & 9th Street Parking Lot



Description: The proposed parcel storage facility would include a 0.6 mg combined sewer storage vault underneath the parking lot. It would have two weir structures: one connecting an 18-inch-diameter pipe on 9th Street, and other connecting to a 16-inch-diameter pipe on 9th Street. The two weir structures would connect to 25 LF of 120-inch-diameter pipe that would convey the flows to the storage vault. The weir crest elevation would be set above the peak dry-weather depths in the connecting pipes. A 0.05 mgd fixed pump station was sized to drain the storage vault.

Benefits: This parcel storage facility would reduce the peak HGL in the immediate area, and provide a moderate level of benefits by reducing surface flooding. This project is needed to store combined flows in 5-year and larger storms. Storms in the average annual year are much smaller than the 5-year storm, and the existing system has capacity to convey flows for these smaller storms without any flooding issues. Although the benefits of this storage project are moderate, combined with other upstream or downstream projects, performance can be further improved. Table 5-2 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-12 and Figure 5-13 presents the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-2: Modeled Peak HGL for Node Near G Street and 9th Street Parking Lot

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
G Street and 9th Street	19.7	**20.3**	**19.8**	18.4	**20.5**	**20.3**	19.4

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-12: Pre-Project Performance at 9th Street in 10-Year Storm

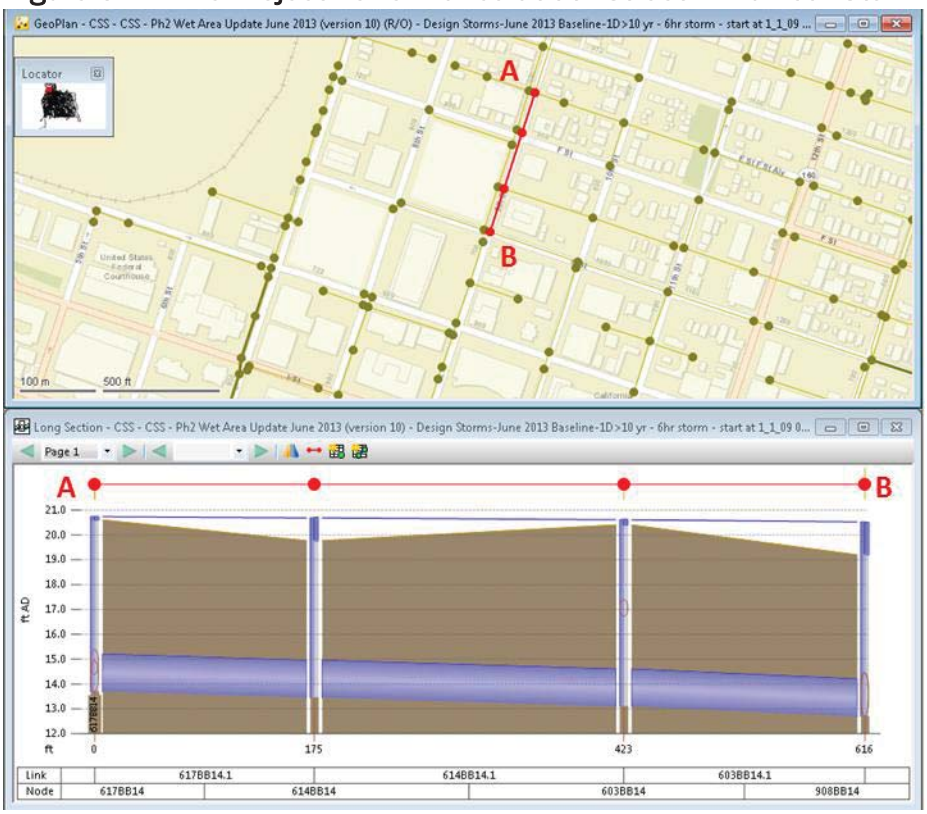
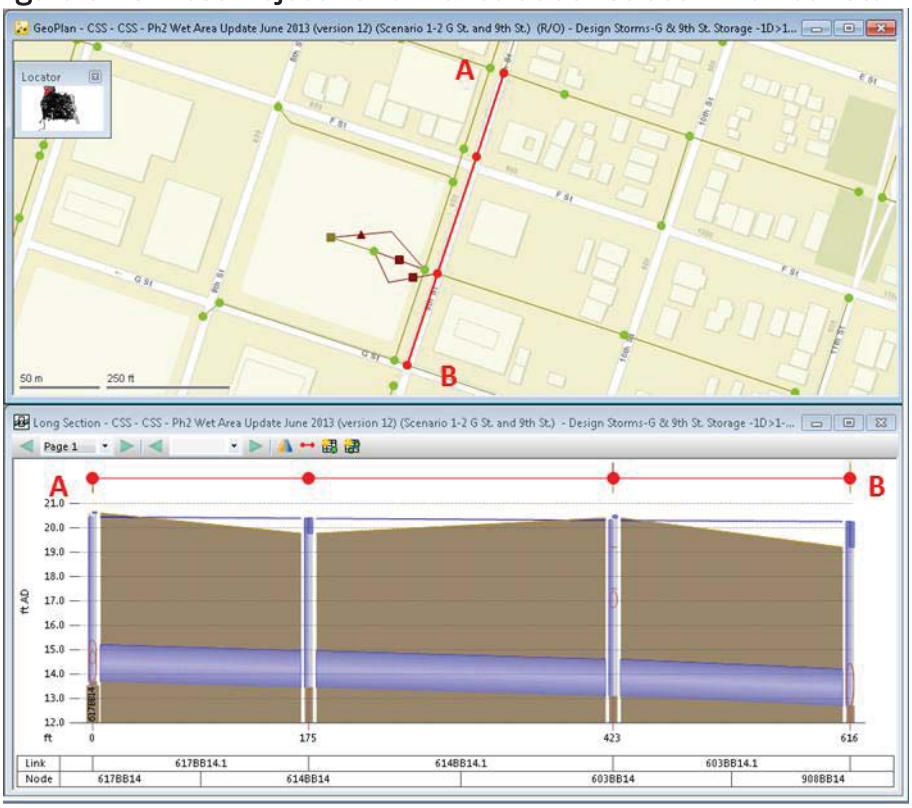


Figure 5-13: Post-Project Performance at 9th Street in 10-Year Storm



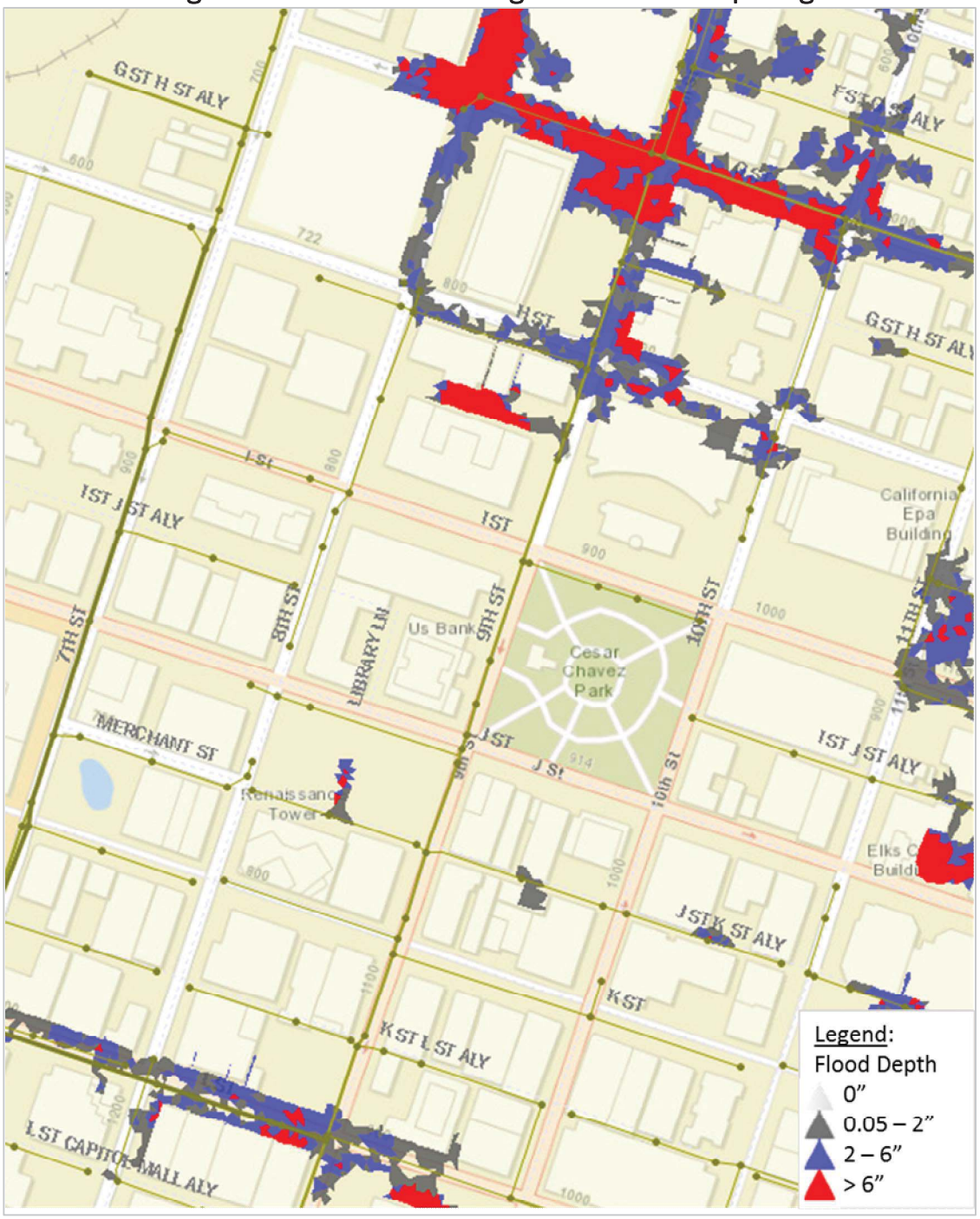
Feasibility/Constructability: Figure 5-14 presents an aerial view of the parking lot at G and 9th Streets. The lot is owned by the County of Sacramento, which would thus require partnering with an outside agency. The lot is, however, at-grade and would require only minimal demolition. Despite the external agency coordination necessitated, DOU deems this project site to be an ideal location for parcel storage construction.

Figure 5-14: G and 9th Street Storage, Aerial View

5.3 Project 1-3 (9th Street from G to L Street)

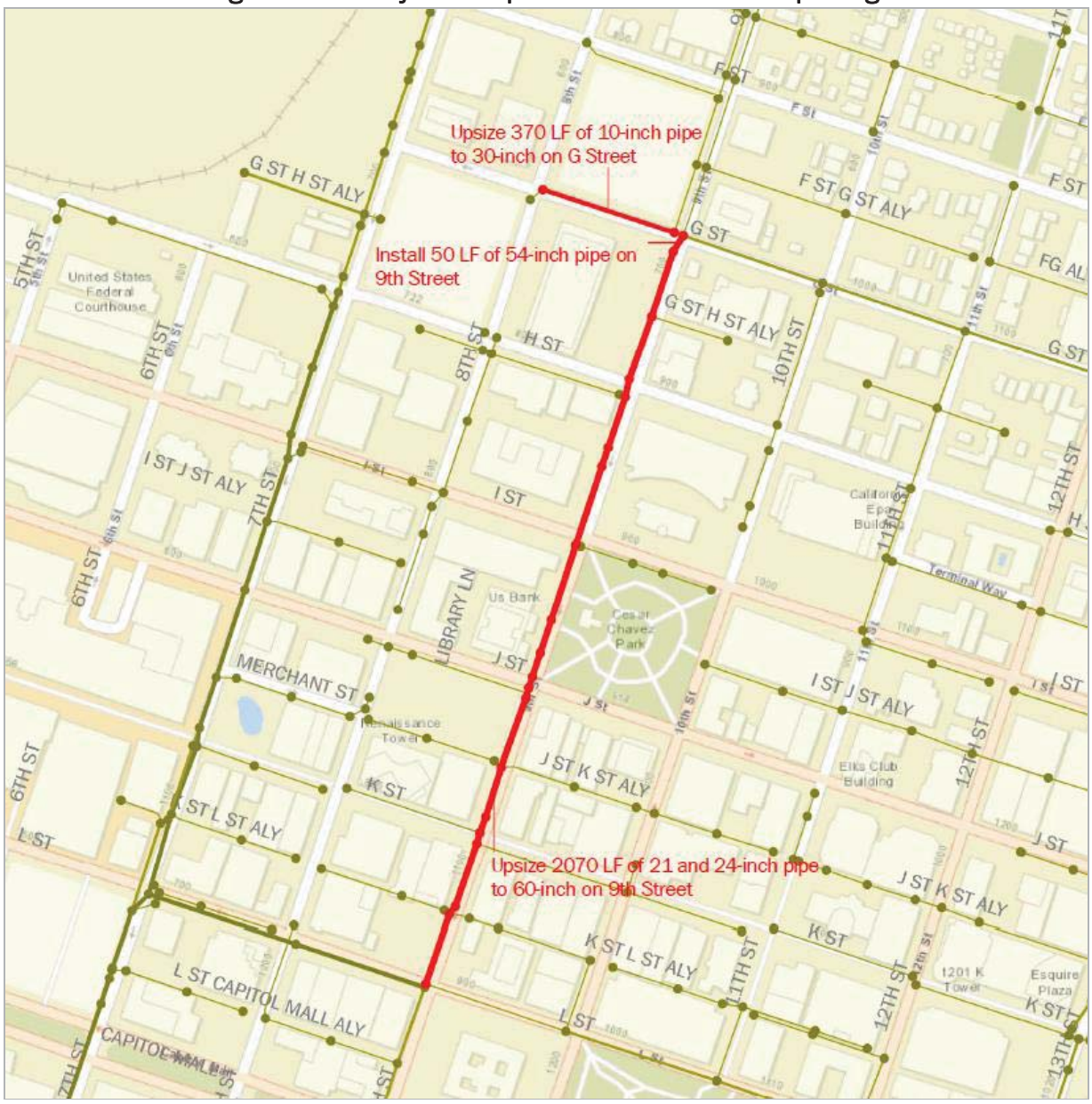
DOU is planning to upsize the existing 21- and 24-inch-diameter pipes to 60-inch-diameter pipes on 9th Street from G to L Street. Figure 5-15 shows the baseline H&H model results for the area near 9th Street in a 10-year design storm.

Figure 5-15: Baseline Flooding near 9th Street Upsizing



Part of this project is currently in design by West Yost Associates; modifications were developed in the CSSIP to further improve the performance of this project. The two sewer pipes north of 9th and G Street drain into the pipes on G Street. These two pipes do not connect to existing 21- and 24-inch-diameter pipes on 9th Street from G to L Street. The modeling analysis indicated that connecting the two pipes north of 9th and G Streets to the upsized 60-inch-diameter pipes on 9th Street would increase downstream conveyance capacity and relieve upstream surcharged pipes. Additionally, local upsizing on G Street could further increase the benefits of this project. Figure 5-16 presents the system components of this project.

Figure 5-16: Project Components for 9th Street Upsizing



Description: The proposed project has the following conveyance components:

- Upsize 2,070 LF of 21- and 24-inch-diameter pipe to 60-inch-diameter pipe on 9th Street from G to L Street
- Upsize 370 LF of 10-inch-diameter pipe to 30-inch-diameter pipe on G Street from 8th to 9th Street
- Install 50 LF of 50-inch-diameter pipe on 9th Street to connect pipes over G Street

The upsizing portion from G to L Street was modeled per existing pre-design drawings developed by DOU for 9th Street.

Benefits: The project would lower the peak HGL along 9th Street and provide a moderate level of benefits by reducing surface flooding. The project would be particularly effective for the areas north of 9th and G Streets. If combined with upstream detention projects, including Project 1-1 (Zapata Park) and Project 1-2 (G and 9th Street Storage Parking Lot), the benefits of this project would be further improved. Table 5-3 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-17 through Figure 5-20 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-3: Modeled Peak HGL for Node Along 9th Street

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
G Street and 9th Street	19.7	**20.3**	19.7	18.4	**20.5**	**20.0**	19.4

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-17: Pre-Project Performance at G and 9th Streets in 10-Year Storm

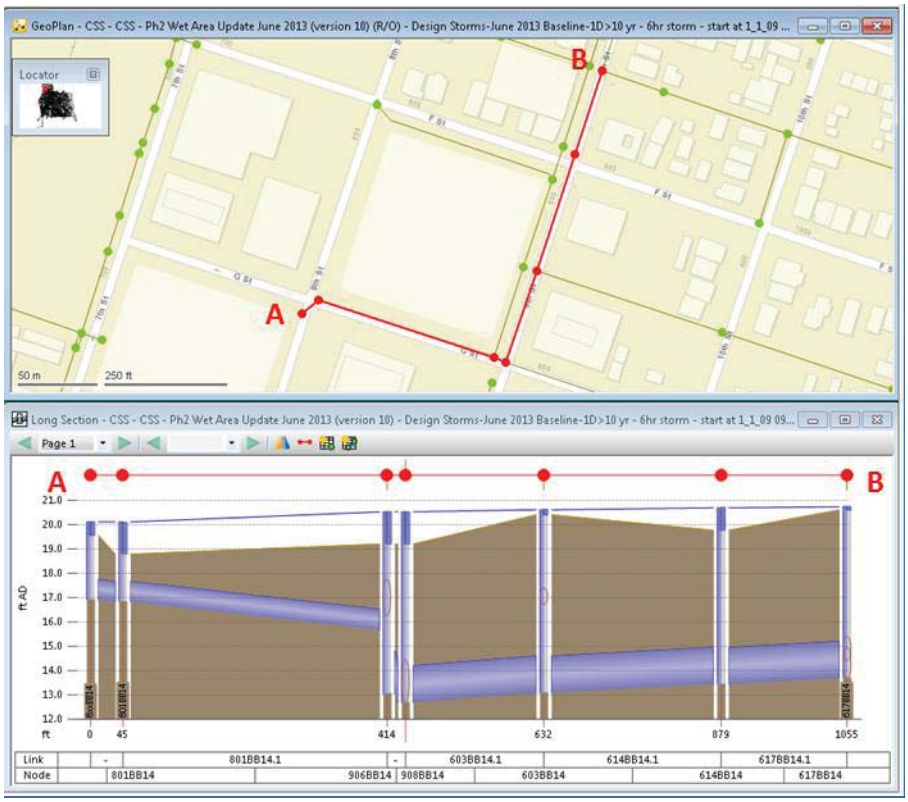


Figure 5-18: Post-Project Performance at G and 9th Streets in 10-Year Storm

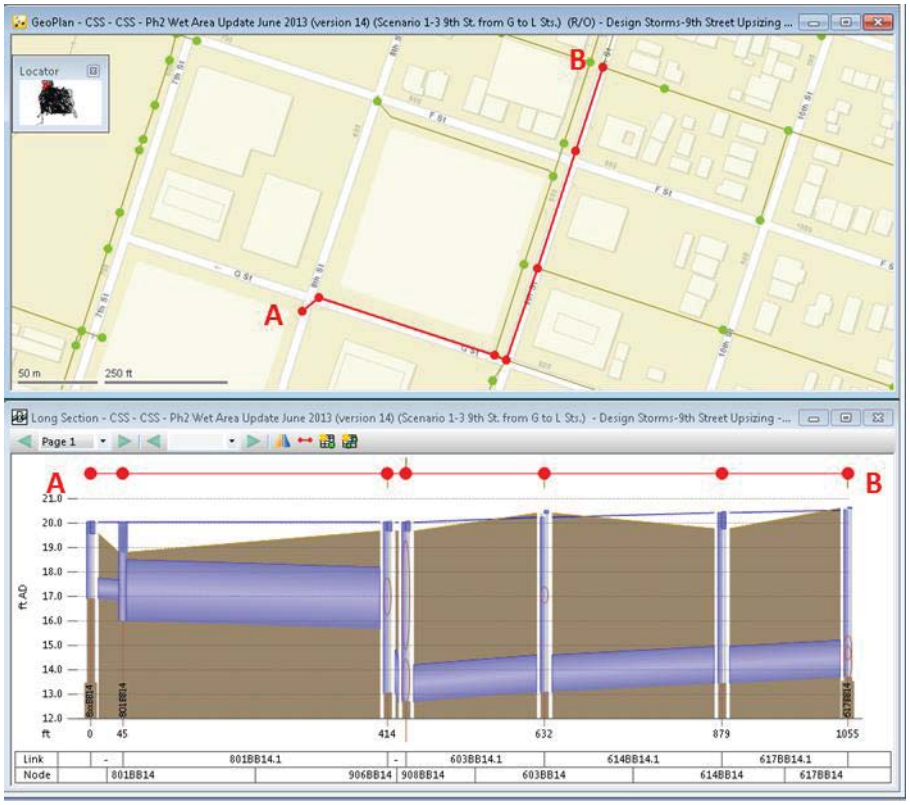
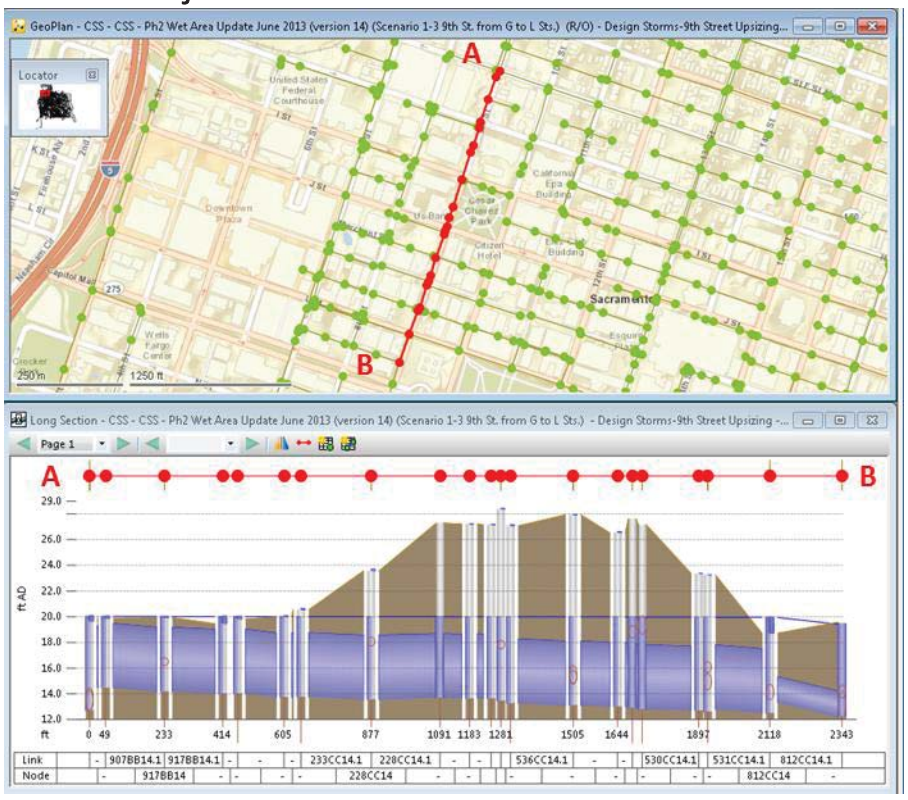


Figure 5-19: Pre-Project Performance on 9th from G to L Streets in 10-Year Storm

Figure 5-20: Post-Project Performance on 9th from G to L Streets in 10-Year Storm



Feasibility/Constructability: The 9th Street upsizing project is highly feasible. DOU developed the majority of the project through pre-design, and has drafted drawings of the G to L Street upsizing. The remaining two portions of the project, the connection across G Street and the small upsizing on G Street, would be minor additions to the previously developed project. Figure 5-21 and Figure 5-22 present aerial and street-level views of the project site.

Figure 5-21: 9th Street Aerial View

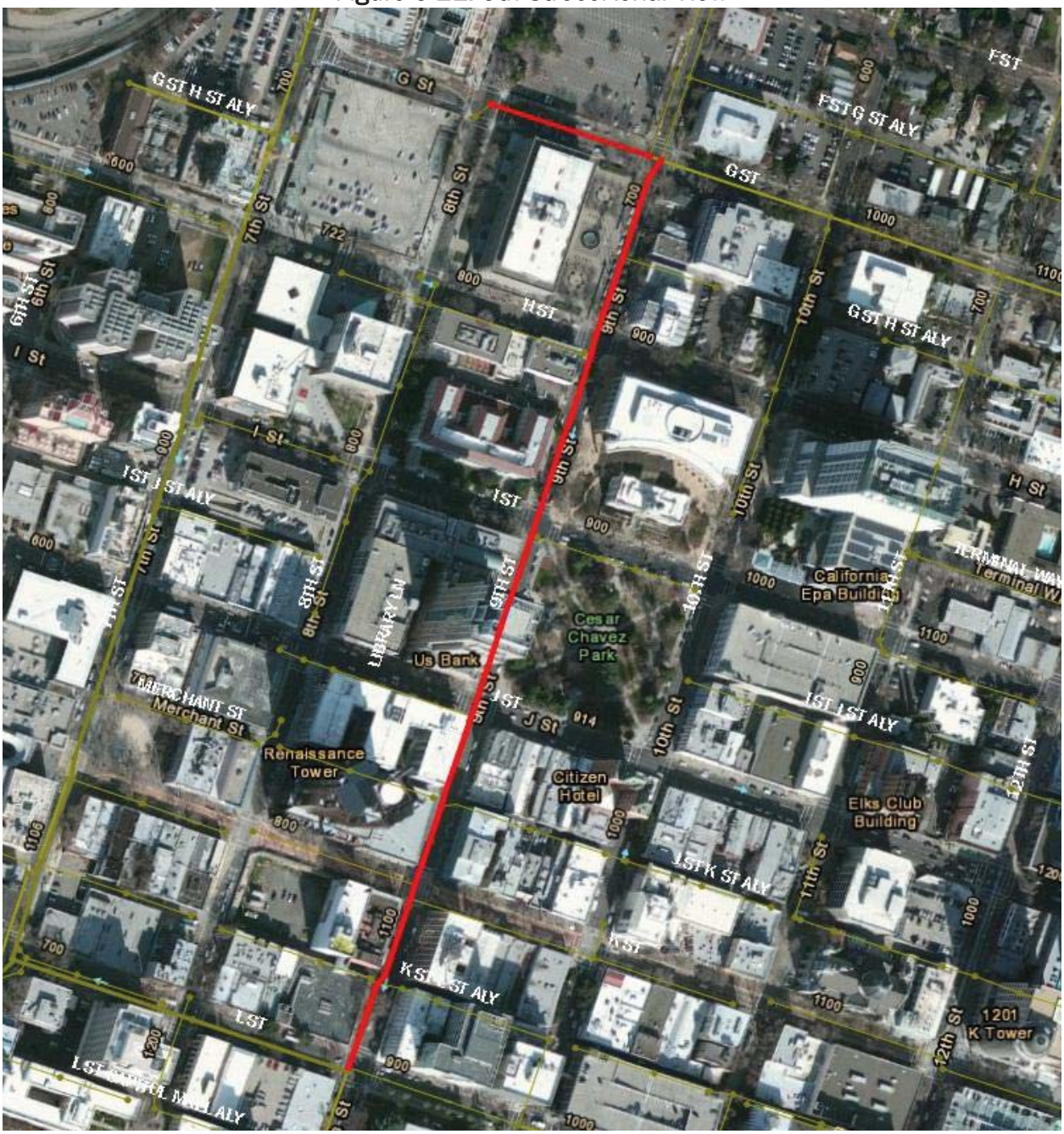


Figure 5-22: 9th Street View (Looking South from H and 9th Street)

5.4 Project 1-4 (14th Street Storage)

The H&H model indicates flooding at 13th Street between H and J Streets, as well as areas farther east on both H and J Streets. The flooding is largely the result of small pipes draining to under-capacity downstream pipes. A potential solution to this challenge is to detain the CSS flows to relieve downstream drainage pipes and attenuate peak flows. This project provides detention for 13th Street area flows. Figure 5-23 shows the baseline H&H model results for the area surrounding 14th Street in a 10-year design storm.

Figure 5-23: Baseline Flooding near 14th Street

No suitable parcels were available in the area; 14th Street appears to have less traffic than 13th Street, and, currently, 14th Street does not have a large sewer pipe. Given these constraints, a linear storage facility along 14th Street was developed to address localized flooding problems. Figure 5-24 presents the system components for the project.

Figure 5-24: Project Components for 14th Street Storage

Description: This project has two main components: pipe upsizing and installation of a linear storage facility. The pipe upsizing includes the following:

- Upsize 800 LF of 8-inch-diameter pipe to 24-inch-diameter on I Street

The linear storage facility would be 1,150 LF of 120-inch-diameter conduit from J and 14th Street to H and 14th Street. This linear storage facility would have a storage volume of

0.7 mg. Three weir structures would connect the storage facility to the collection system to collect flows from the flooding areas to the storage facility. The first weir structure would be located at 14th and I Streets, the second weir structure would be at 14th Street and H/I Alley, and the third weir structure would be at 14th and G Streets. The weir invert elevations are set above peak dry-weather depth in the connecting pipes. A 6-inch-diameter flap valve located at G Street drains the storage facility with a gravity system.

Draining by gravity constrains the design of this project. Gravity drainage requires less operation and maintenance than does a pumped system. To ensure that such drainage is possible, however, no part of the storage facility can be deeper than lowest point of the downstream collection system.

Benefits: Both the 5- and 10-year storms fully utilize the proposed storage volume. Detaining the volume lowers HGL on 13th Street and achieves a moderate reduction in flooding. Table 5-4 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-25 through Figure 5-28 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm. Additional projects downstream of this area will reduce the remnant flooding in the area further.

Table 5-4: Modeled Peak HGL for Node Near 14th Street Storage

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
I Street near 13th Street	18.4	**19.0**	**18.6**	17.4	**19.2**	**19.3**	**18.8**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-25: Pre-Project Performance on H/I Alley in 10-Year Storm

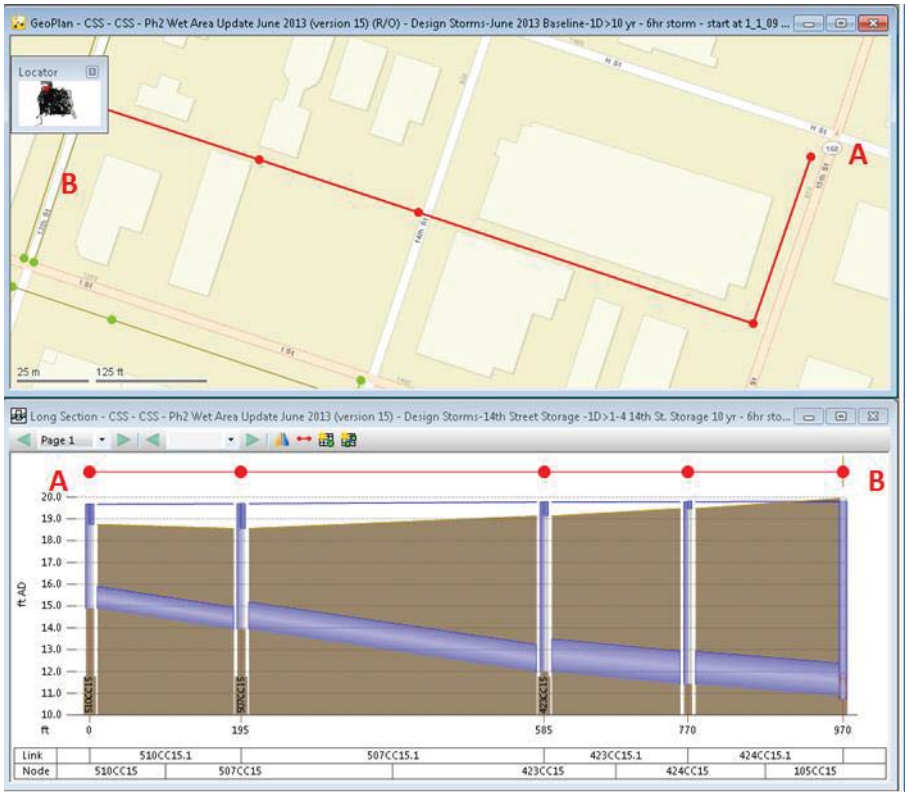


Figure 5-26: Post-Project Performance on H/I Alley in 10-Year Storm

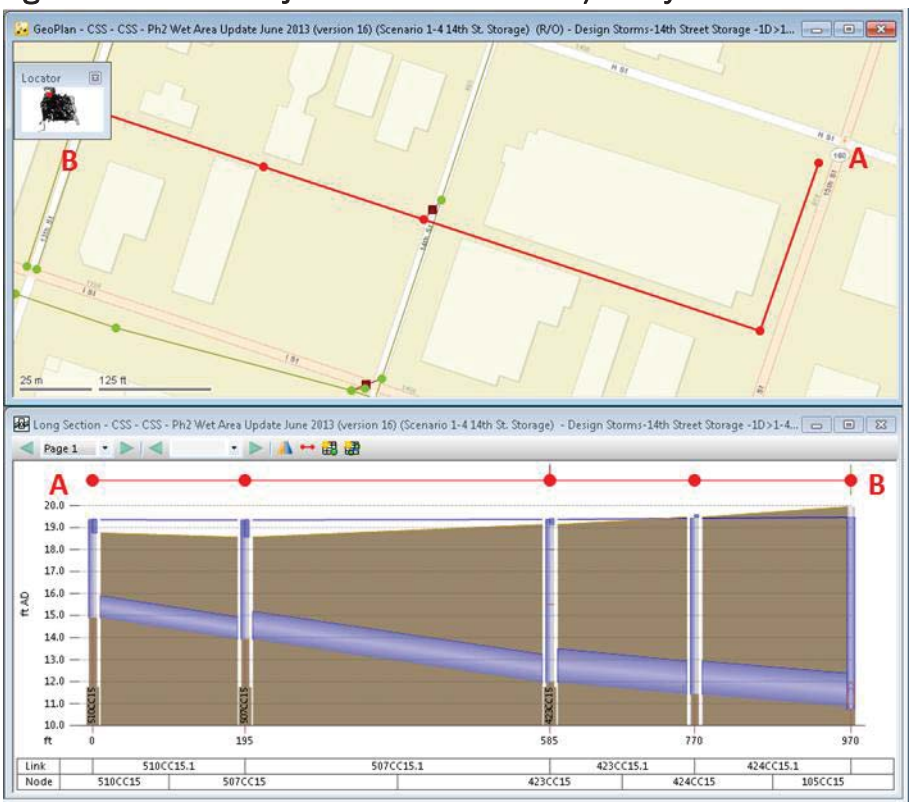


Figure 5-27: Pre-Project Performance on 13th Street in 10-Year Storm

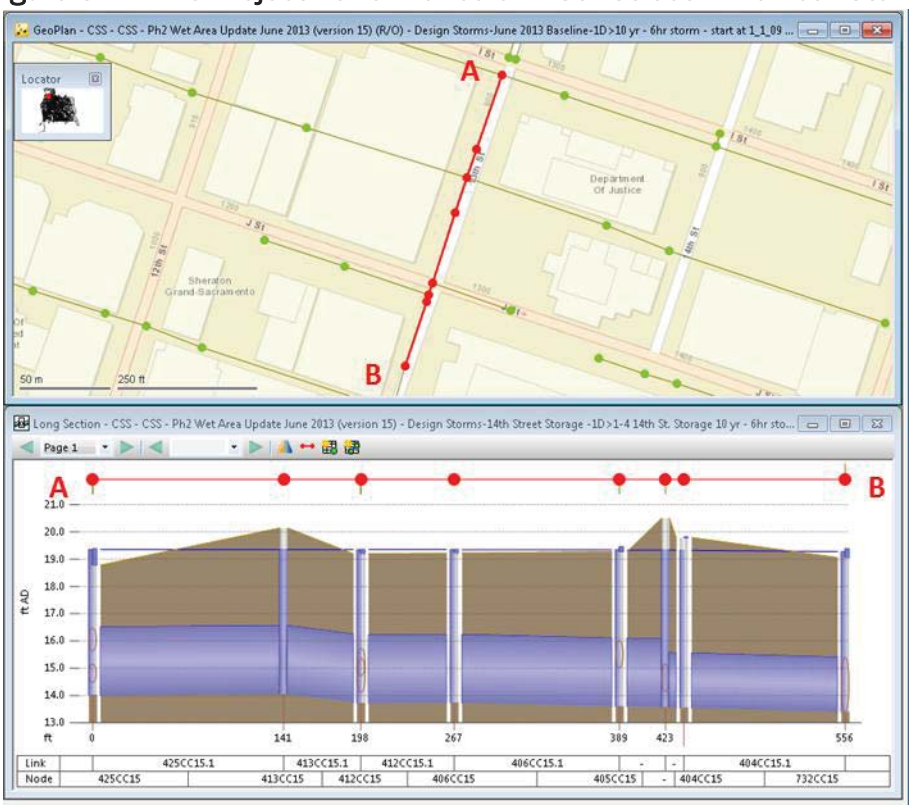
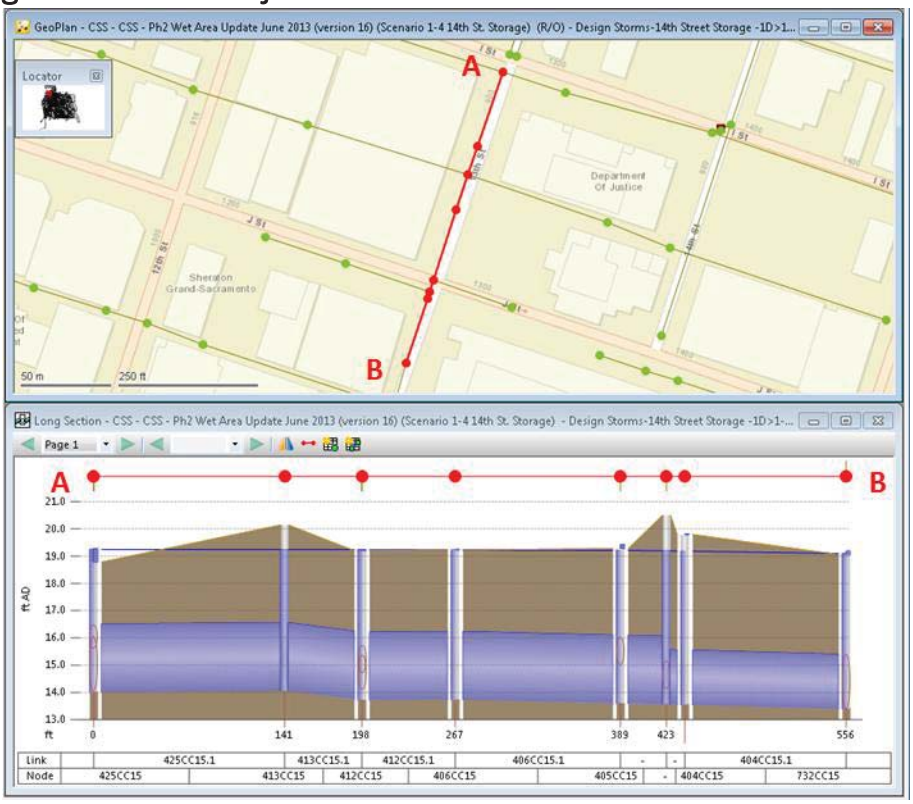


Figure 5-28: Post-Project Performance on 13th Street in 10-Year Storm



Feasibility/Constructability: The 14th street storage project appears to be the most feasible option for this area. As can be seen in Figure 5-29 and Figure 5-30, 14th Street is a wide street and presents a good opportunity for linear storage.

Figure 5-29: 14th Street Storage Aerial View

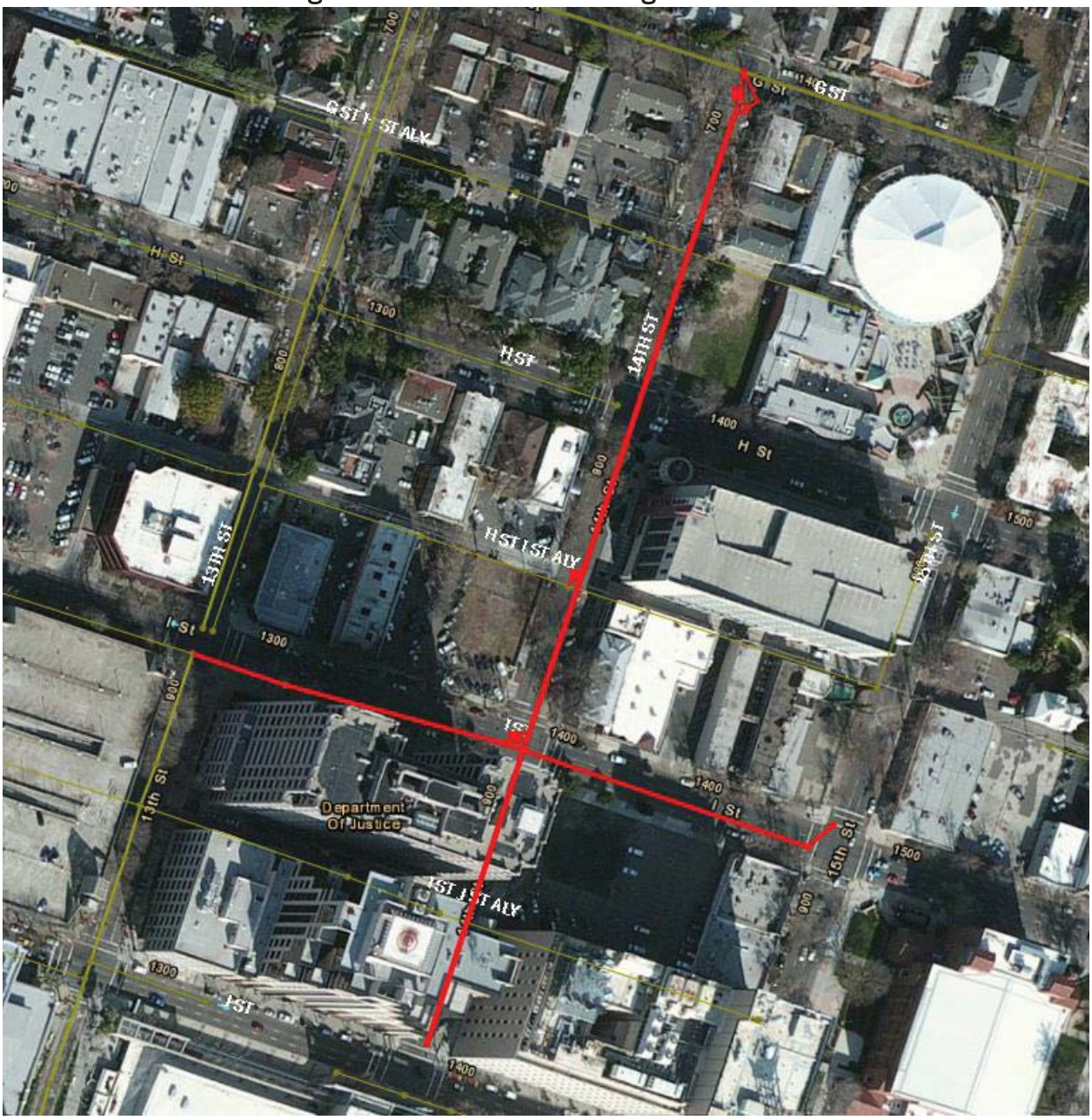
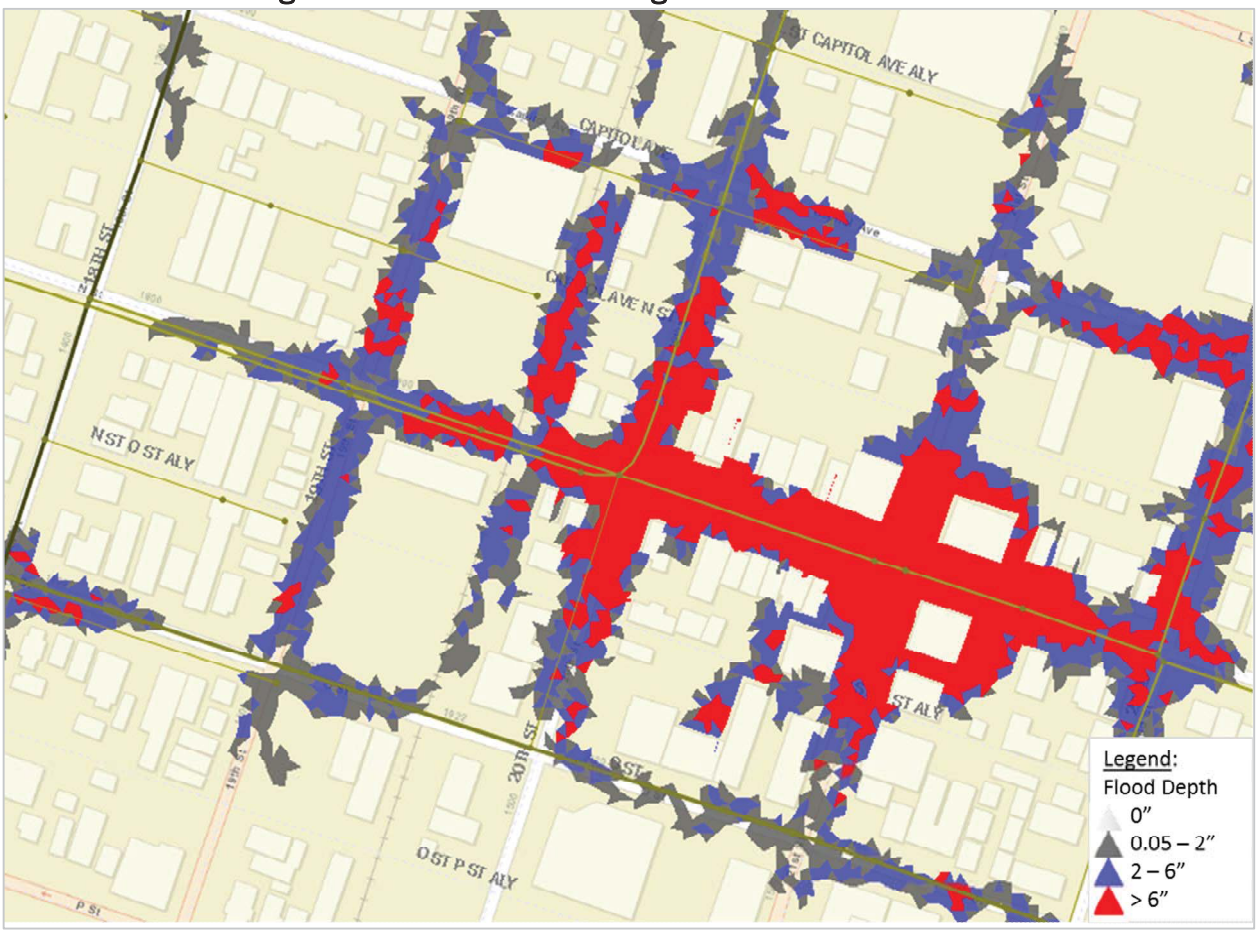


Figure 5-30: H and 14th Street View (Looking North)

5.5 Project 1-5 (N Street and 22nd Street)

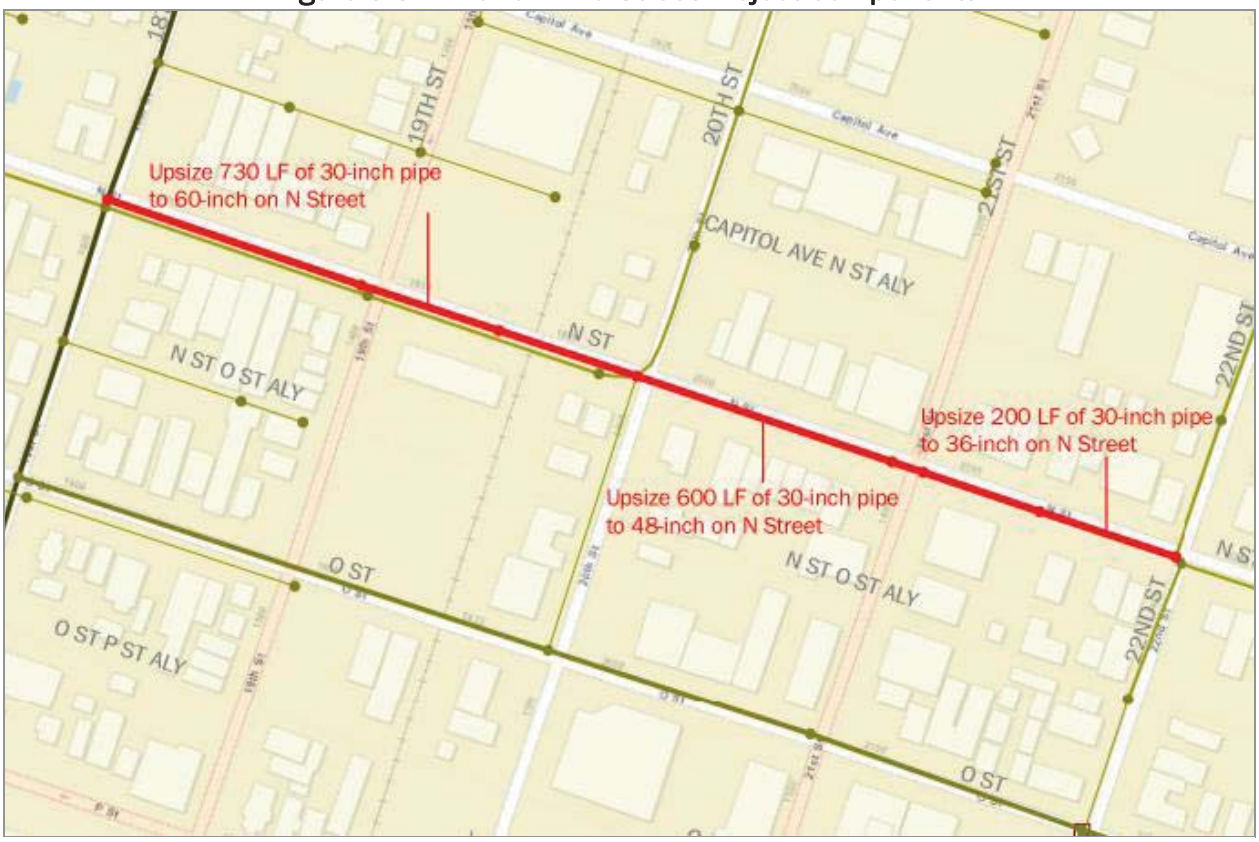
The H&H model indicates greater than 2 inches of flooding in the area surrounding the N and 22nd Street intersection during both the 5- and 10-year design storms. Figure 5-31 shows the baseline H&H model results for the area near the N and 22nd Street intersection in a 10-year design storm.

Figure 5-31: Baseline Flooding Near N and 22nd Street



In this densely developed area, appropriate storage parcels were not available. Instead, the goal of this project is to maximize conveyance in this key drainage area. This project, when combined with other projects, including Project 1-6 (24th Street Storage) and 1-7 (Grant Park), can further reduce flooding in this area. Figure 5-32 presents the system components of the project.

Figure 5-32: N and 22nd Street Project Components



Description: This project involves upsizing three portions of the 30-inch-diameter pipe on N Street:

- Upsize 730 LF of 30-inch-diameter pipe to 60-inch-diameter pipe on N Street from 18th to 20th Street (will require jack and bore beneath railroad tracks)
- Upsize 600 LF of 30-inch-diameter pipe to 48-inch-diameter pipe on N Street from 20th Street to halfway past 21st Street
- Upsize 200 LF of 30-inch-diameter pipe to 36-inch-diameter pipe on N Street from halfway past 21st Street to 22nd Street

The upsizing aims to remove a pipe constraint on N Street and convey flows faster downstream. The diameter of the new pipes would be constrained by the ground surface elevation and existing inverts.

Benefits: The project would result in a moderate reduction of the HGL in the surrounding area. The benefit is slight, however, and the project would perform optimally when paired with an upstream storage project(s). Table 5-5 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-33 through Figure 5-36 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-5: Modeled Peak HGL for Node Near N & 22nd Street

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
N Street and 22nd Street	15.4	**16.8**	**16.3**	14.0	**17.4**	**17.0**	**15.9**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-33: Pre-Project Performance on N Street in 10-Year Storm

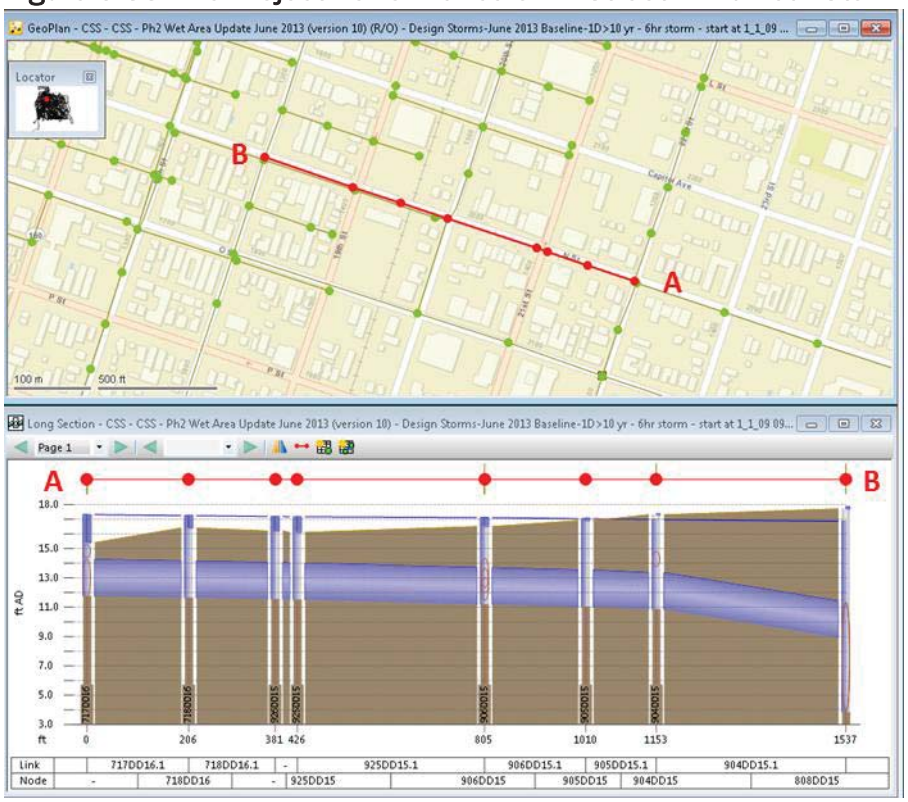


Figure 5-34: Post-Project Performance on N Street in 10-Year Storm

Figure 5-35: Pre-Project Performance on N & 22nd Street in 10-Year Storm

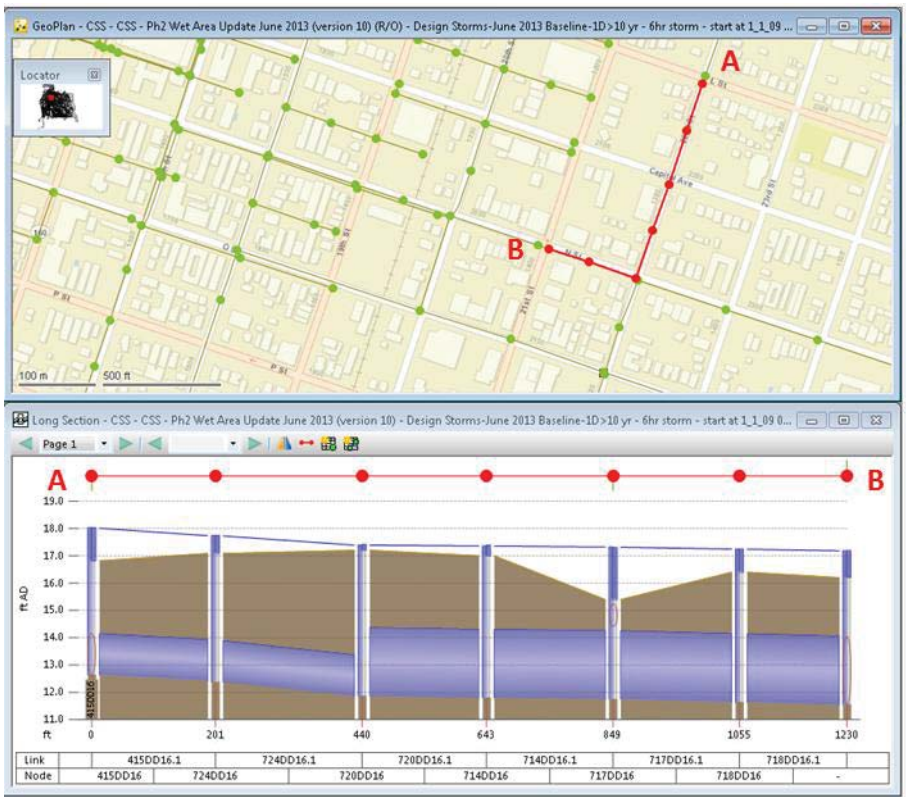


Figure 5-36: Post-Project Performance on N & 22nd Street in 10-Year Storm

Feasibility/Constructability: This project would be challenging to construct. N Street is a major transportation corridor, and construction may be disruptive to traffic flow. One section of upsizing will further require bore and jack under railroad tracks. Figure 5-37 and Figure 5-38 show aerial and a street views of the project area.

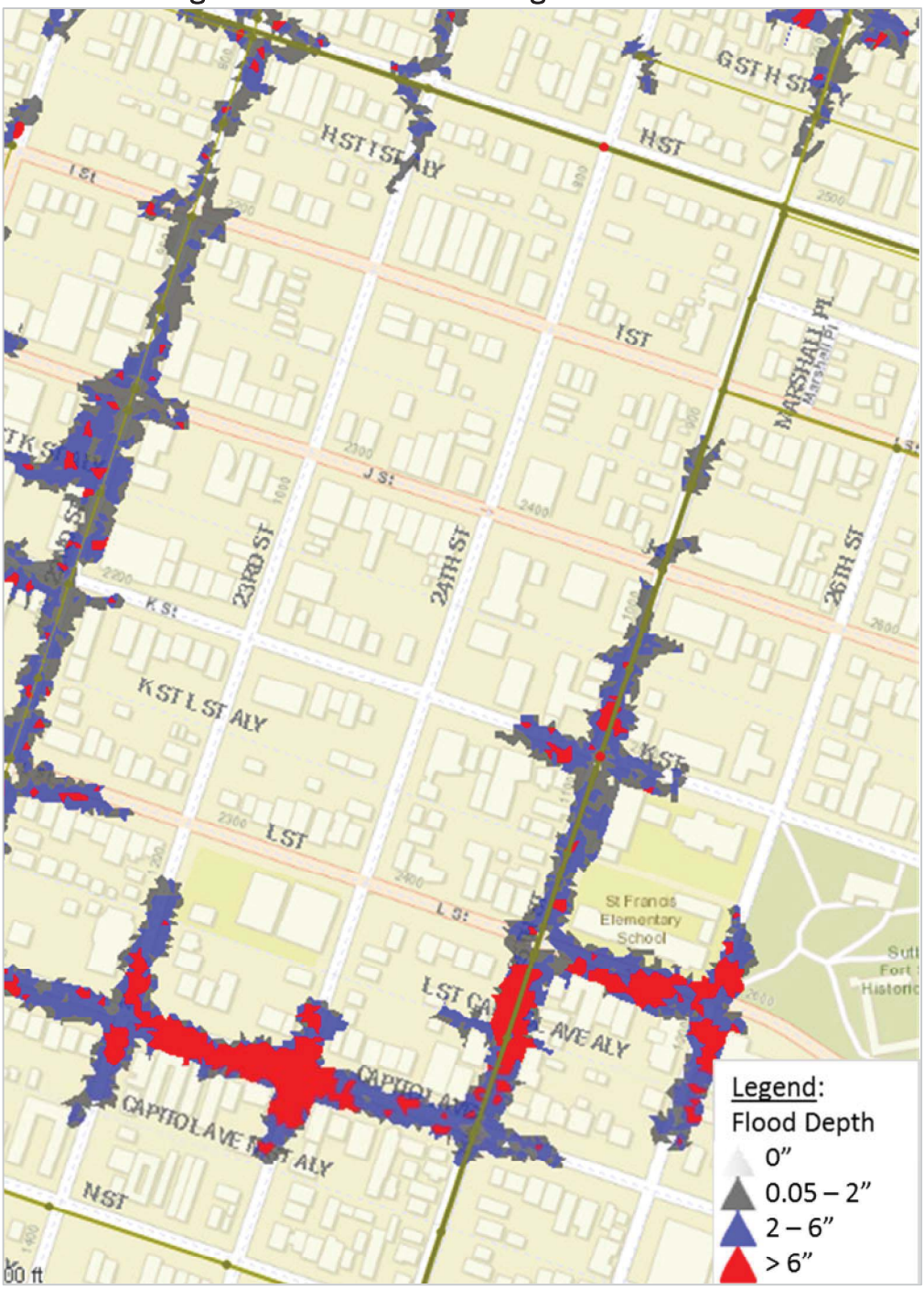
Figure 5-37: N and 22nd Street Aerial View

Figure 5-38: N Street View (Looking East from N and 21st Street)

5.6 Project 1-6 (24th Street Storage)

This area is located northeast of the N Street and 22nd Street intersection, and has flooding issues. Figure 5-39 shows the baseline H&H model results for the area surrounding 24th Street in a 10-year design storm.

Figure 5-39: Baseline Flooding near 24th Street



With a lack of appropriate parcels for storage in this area, 24th Street provides an opportunity to plan linear storage facility to reduce flooding. Currently, the street has no large sewer mains, but is close to the large sewer main on 25th Street. The pipe on 25th Street drains to the N Street and 22nd Street intersection, so detaining flows upstream of 25th Street would alleviate flooding downstream. Saint Francis School was considered for parcel storage, but was deemed infeasible by DOU. Figure 5-40 presents the system components for this project.

Figure 5-40: 24th Street Storage Project Components

Description: The linear storage facility would be a conduit from N Street and 24th Street to H Street and 24th Street, with a one-block section on K Street between 24th and 25th Streets. The linear storage would include the following:

- Install 2,325 LF of 108-inch-diameter pipe on 24th Street
- Install 325 LF of 108-inch-diameter pipe on K Street

A weir structure would be located at 25th and K Street and collect flow from the 25th Street pipe to the storage facility. Ultimately, the storage facility would drain to H Street via a 6-inch-diameter flap valve at H Street and 24th Street.

Benefits: This project would benefit the local area by lowering the HGL on 25th Street and reducing the amount of flow going toward the N Street and 22nd Street problem area. Table 5-6 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-41 and Figure 5-42 presents the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-6: Modeled Peak HGL for Node Near 24th Street

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
K Street and 25th Street	17.2	**17.9**	17.0	14.7	**18.3**	**18.1**	16.8

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-41: Pre-Project Performance on 25th Street in 10-Year Storm

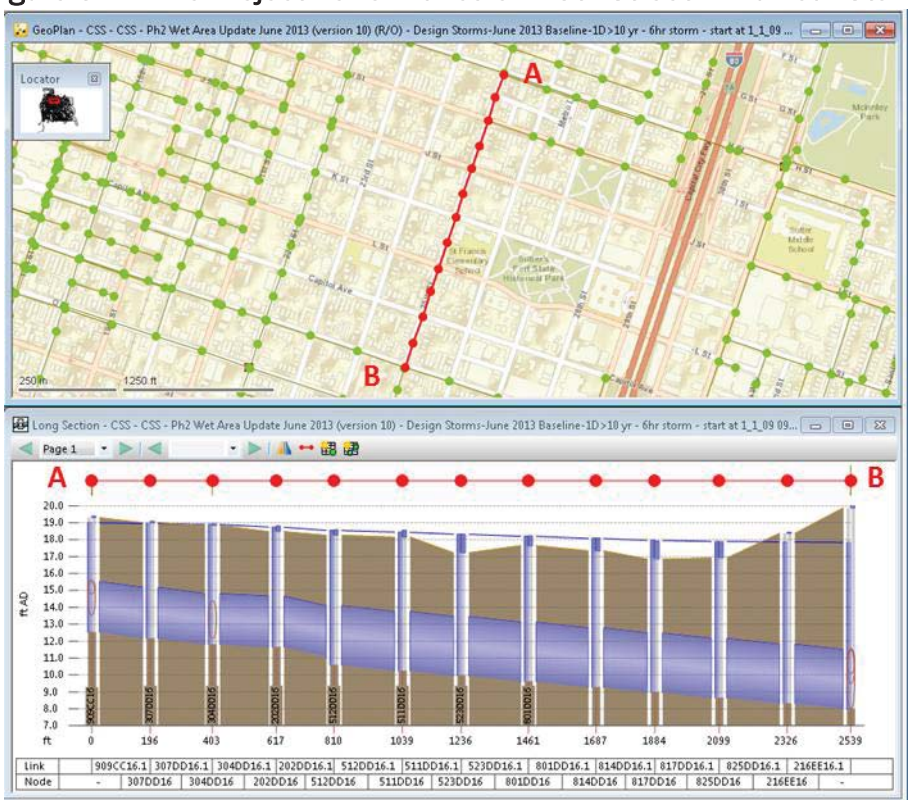
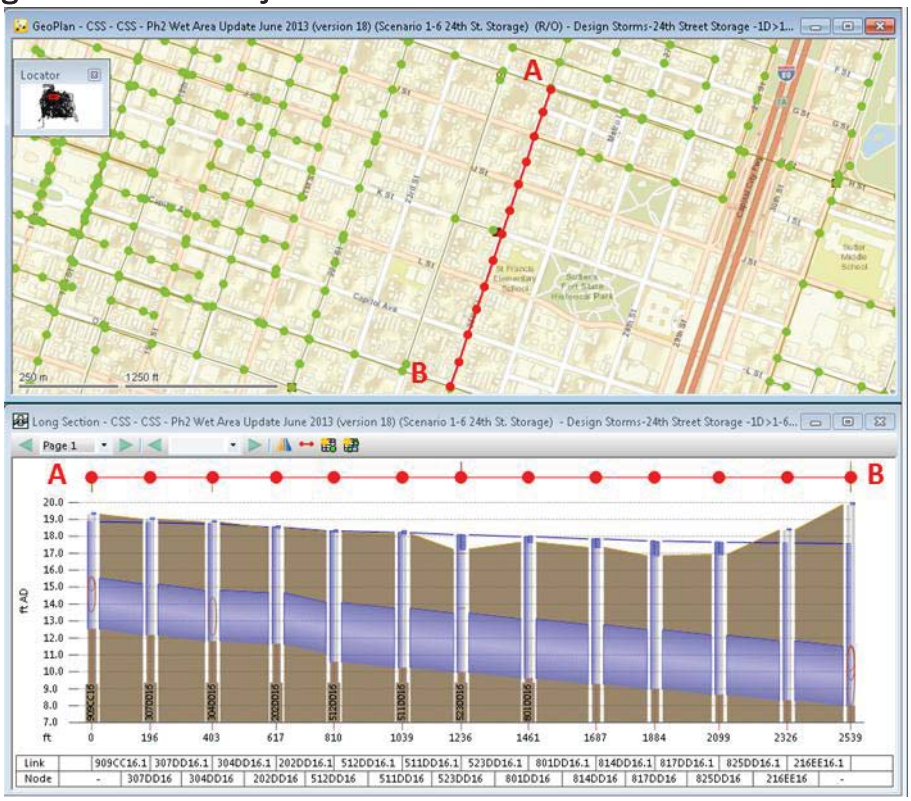


Figure 5-42: Post-Project Performance on 25th Street in 10-Year Storm



Feasibility/Constructability: Due to the lack of open spaces in this area, this linear storage project was deemed as a feasible detention option. This project is feasible as it is located close to surcharging pipes and has potentially fewer utility constraints, with no large sewer pipes in 24th Street. Figure 5-43 presents an aerial view and Figure 5-44 presents a street view of 24th Street.

Figure 5-43: 24th Street Storage Aerial View

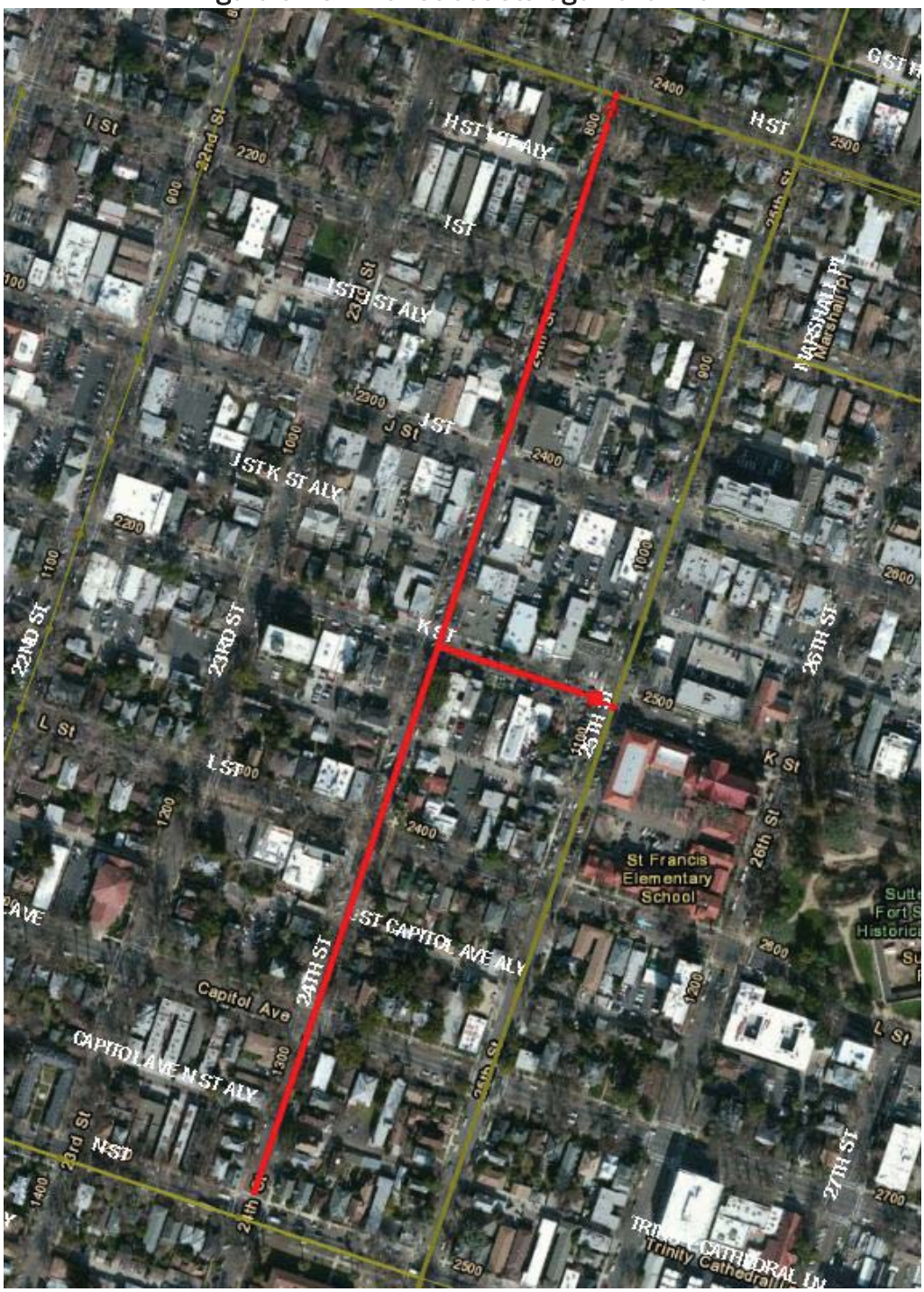


Figure 5-44: 24th Street View (Looking North from K and 24th Streets)

5.7 Project 1-7 (Grant Park)

The H&H model indicates flooding in the area surrounding and south of Grant Park. Excess flows in this area are due to undersized local pipes and surcharged downstream pipes. Figure 5-45 shows the baseline H&H model results for the area surrounding Grant Park in a 10-year design storm.

Figure 5-45: Baseline Flooding near Grant Park

A storage vault project at Grant Park would detain flow, lower the HGL in the immediate area, and relieve surcharged downstream pipes, including pipes near Project 1-5 (N and 22nd Street). Figure 5-46 shows project components.

Figure 5-46: Grant Park Project Components

Description: This project would contain both conveyance and storage elements. The conveyance portion would include the following:

- Upsize 850 LF of 8-inch-diameter pipe to 16-inch-diameter on 23rd Street
- Upsize 1,250 LF of 10-, 12-, and 14-inch-diameter pipe to 16-inch-diameter on 25th Street

The storage element of this project would consist of a storage vault and connected large-diameter storage pipes to convey flow into the storage facility. Specifically, the project would include the following:

- Install a 2.8 mg storage vault underneath Grant Park
- Install 1,100 LF of 60-inch-diameter pipe on 22nd Street from E Street to the storage vault

- Install 1,200 LF of 60-inch-diameter pipe on D Street from 25th Street to a 60-inch-diameter pipe on 22nd Street

In total, three weir structures placed at D and 23rd Streets, D and 25th Streets, and E and 22nd Streets would connect the storage facility to the collection system. The 60-inch-diameter pipe on D Street would collect flow from 23rd Street and 25th Street via two weir structures and convey flow to the storage facility in Grant Park. The 60-inch-diameter pipe on 22nd would collect flow from the surcharging main on E Street via a weir structure. The collective storage project would drains by gravity to a 6-inch-diameter flap valve at the E and 22nd Street intersection.

Benefits: The project would benefits the area surrounding Grant Park by significantly lowering the HGL and reducing flooding in the area. The project would also attenuate flows going down 22nd and 25th Streets. This would relieve the flooding area at N and 22nd Streets, and would work well with Project 1-5 (N Street and 22nd Street). Table 5-7 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-47 and Figure 5-48 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

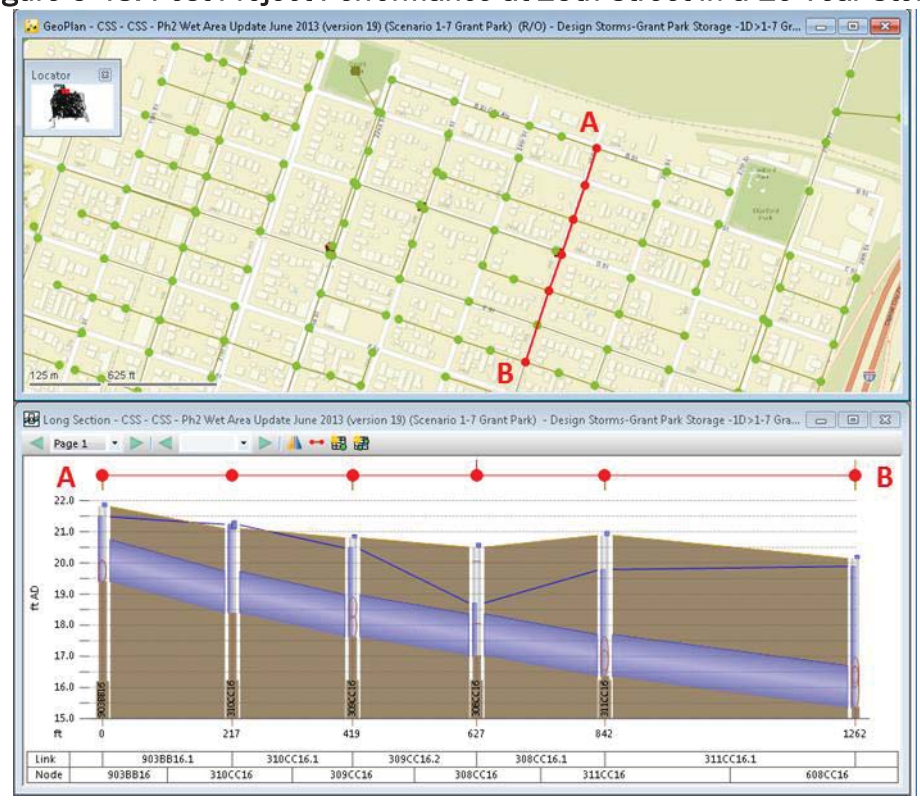
Table 5-7: Modeled Peak HGL for Node Near Grant Park

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
C/D Alley and 25th	20.8	**21.6**	20.3	20.3	**21.8**	20.5	20.5

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-47: Pre-Project Performance at 25th Street in a 10-Year Storm

Figure 5-48: Post-Project Performance at 25th Street in a 10-Year Storm



Feasibility/Constructability: Pipes on 23rd and 25th Streets are good candidates for replacement because they appear to be undersized and are overwhelmed by storm flows. The 60-inch-diameter storage conduits also appear to be feasible, due to the extra wide 22nd Street and the lack of existing large sewer mains on D Street. Grant Park is also feasible for a storage vault, as it is a park managed by the Department of Parks and Recreation with a similar layout to Oak Park. Figure 5-49 is an aerial view of the Grant Park area.

Figure 5-50 presents a street view at D and 22nd Street looking across 22nd Street and down D Street to the west.

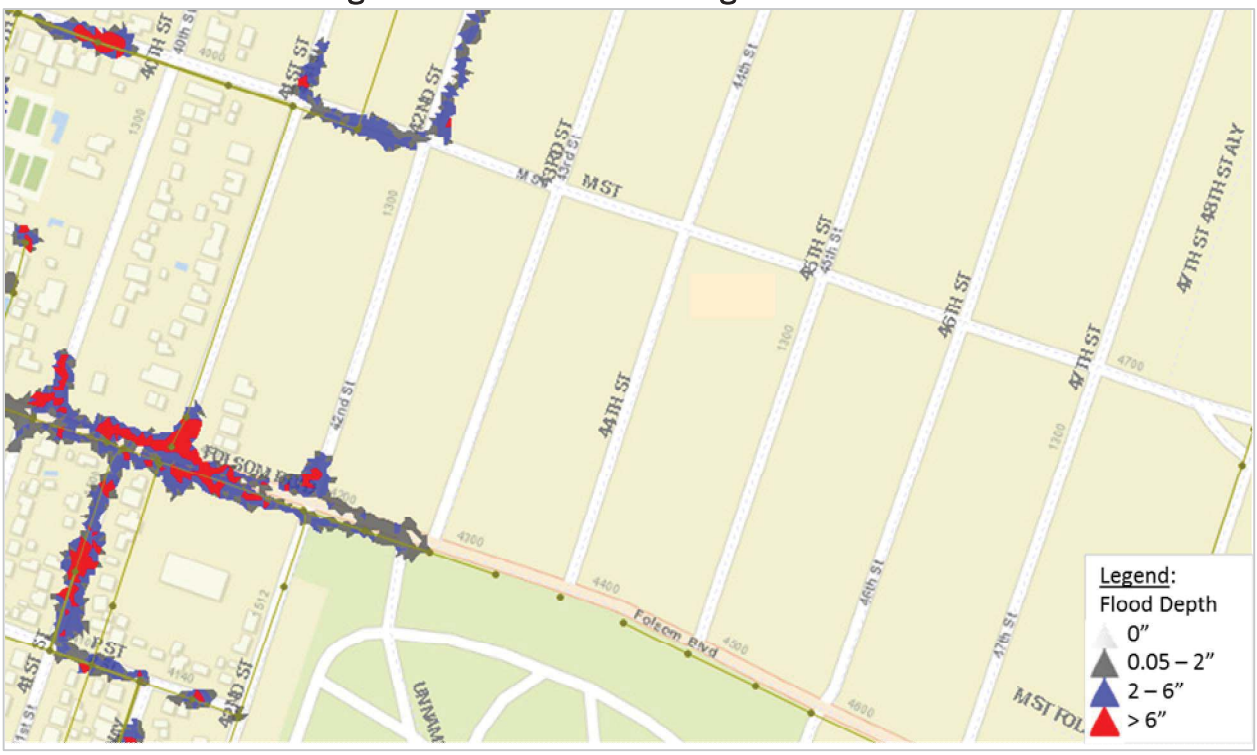
Figure 5-49: Grant Park Project Aerial View

Figure 5-50: D Street View (Looking East from 22nd & D Street)

5.8 Project 2-1 (M Street Storage)

The H&H model indicates flooding in the area near the Folsom Boulevard and 41st Street intersection during the design storms. Figure 5-51 shows the baseline H&H model results for the area surrounding M Street in a 10-year design storm.

Figure 5-51: Baseline Flooding near M Street



DOU recently completed work on the streets south of Folsom Boulevard, and prefers to avoid another construction project to limit neighborhood disturbance. For this reason, upstream detention projects were favored in this area. Appropriate parcels for storage are not available in the area, but linear storage options are available north of the main problem area, particularly on M Street. Figure 5-52 shows the project components.

Figure 5-52: M Street Storage Project Components

Description: The M Street storage project is primarily a detention project with one small conveyance improvement. Upsizing on 41st Street is included in the project to reduce flooding on 41st Street and convey flow more effectively to the M Street storage project. The improvement would involve the following:

- Upsize 860 LF of 15-inch-diameter pipe to 24-inch-diameter on 41st Street
- Upsize 160 LF of 14-inch-diameter pipe to 24-inch-diameter on M Street

The detention facility would be composed of a large linear storage facility, two weir structures to bring flow from M Street and Folsom Boulevard, and a pump to drain the storage facility at 48th Street. The linear storage facility would be a 3,000-LF 120-inch-diameter conduit on 42nd Street from Folsom Boulevard to M Street, and on M Street from 42nd to 48th Street. The first weir structure would be at Folsom Boulevard and 42nd Street, and the second weir structure would be at 42nd and M Street. A 0.05 mgd pump installed at M and 48th Street would be sized to drain the storage facility.

Benefits: The M Street storage project would lower the peak HGL on both 41st Street and Folsom Boulevard. The project would eliminate a major flood point on 41st Street and significantly reduce flooding on Folsom Boulevard. Table 5-8 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-53 through 556 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-8: Modeled Peak HGL for Node Near M Street

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Folsom Blvd. and 41st Street	21.5	**22.3**	20.7	20.6	**22.6**	21.4	21.1

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-53: Pre-Project Performance on 41st Street in 10-Year Storm

Figure 5-54: Post-Project Performance on 41st Street in 10-Year Storm

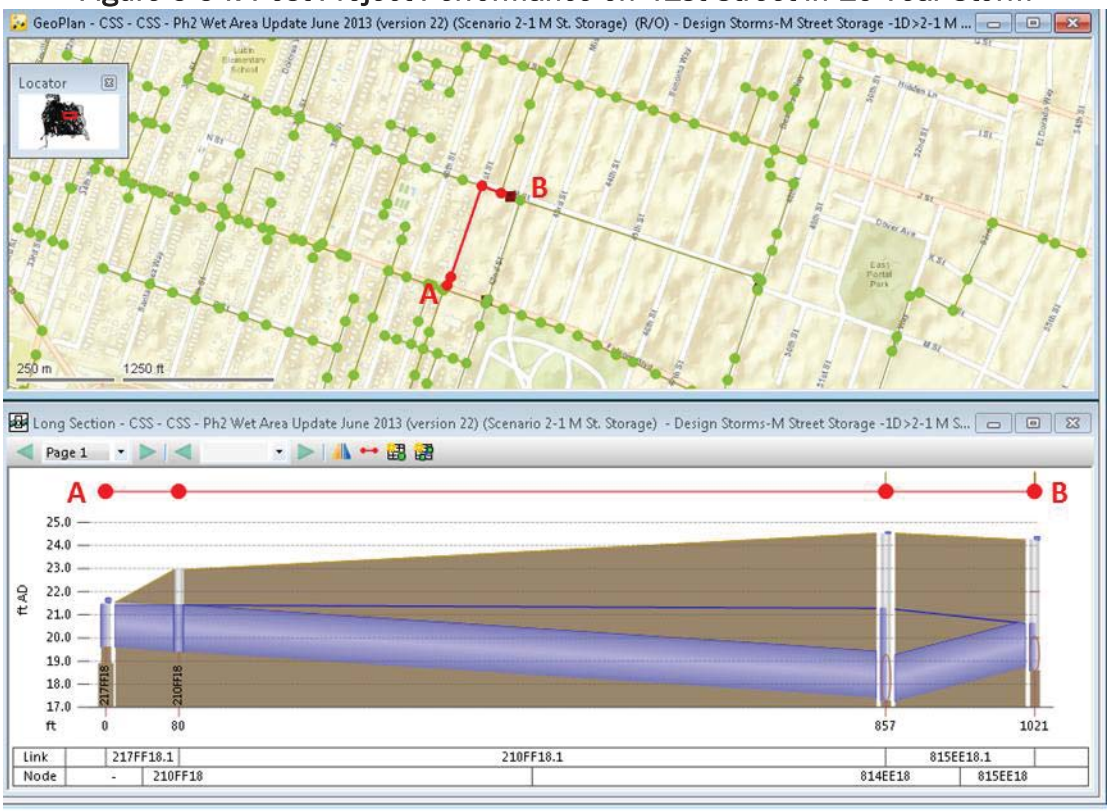


Figure 5-55: Pre-Project Performance on Folsom Blvd. in 10-Year Storm

Figure 5-56: Post-Project Performance on Folsom Blvd. in 10-Year Storm

Feasibility/Constructability: M Street is particularly feasible given the absence of a large sewer pipe and its proximity to both Folsom Boulevard and 41st Street—key flooding pipes in the area. M Street is also a fairly wide street, which will make construction easier.
Figure 5-57 presents an aerial view of the area and Figure 5-58 presents a street view.

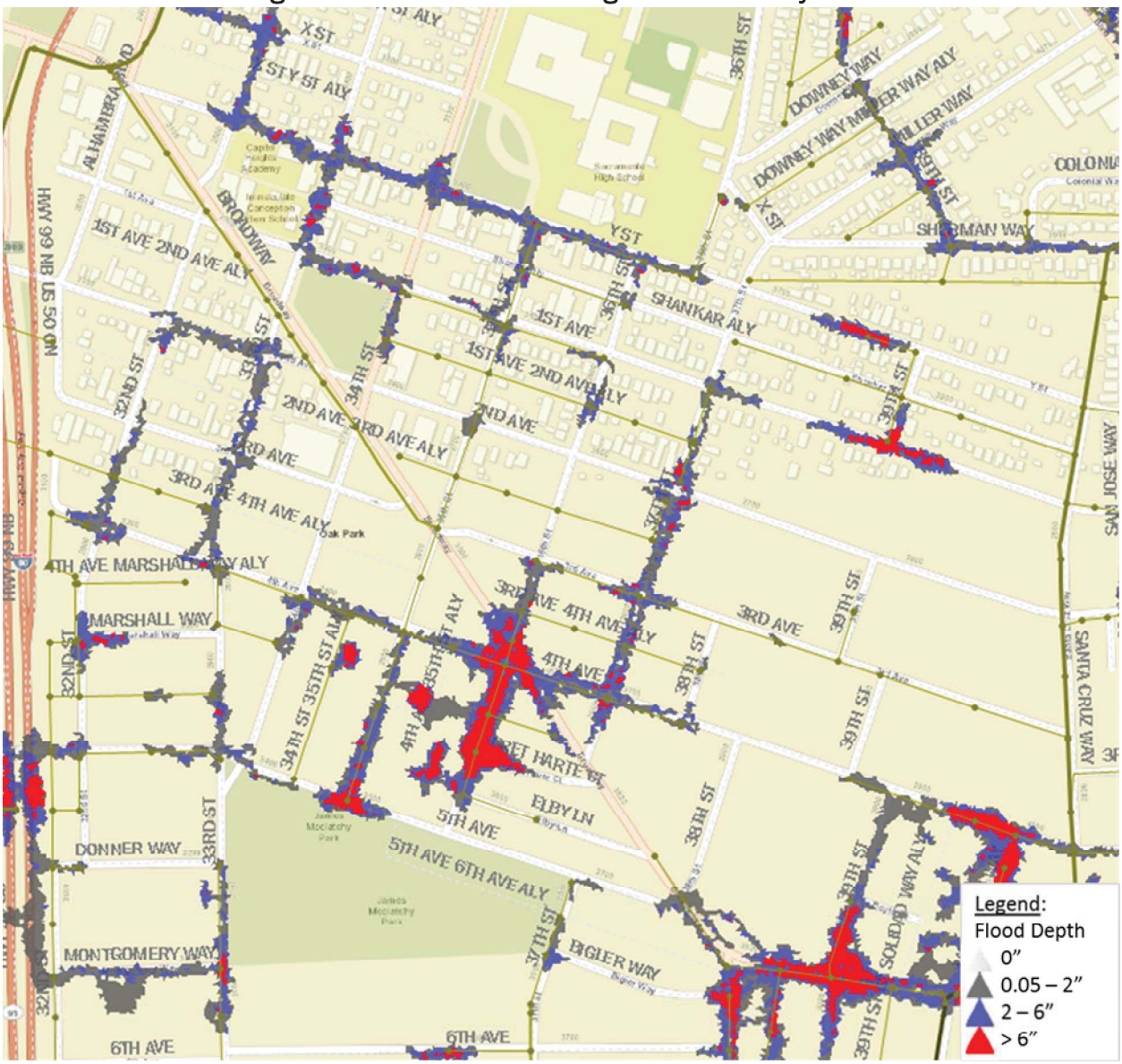
Figure 5-57: M Street Aerial View

Figure 5-58: M Street View (Looking East from M and 42nd Street)

5.9 Project 2-2 (McClatchy Park)

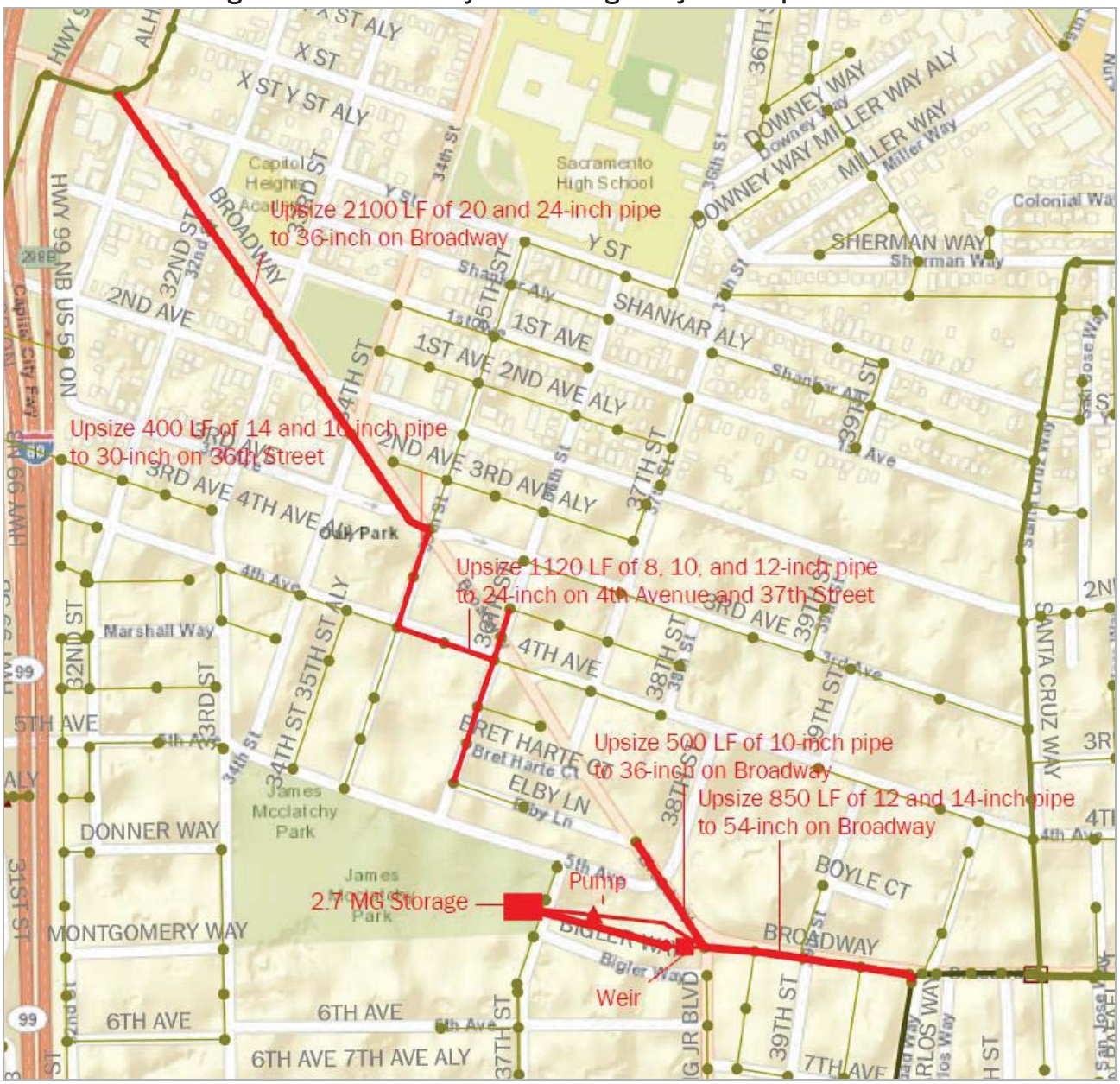
McClatchy Park is located on 5th Avenue adjacent to Wet Area 2 and Wet Area 6. The H&H model indicates flooding in the area surrounding the park. The flooding is centered on Broadway, but also affects small tributary pipes. Figure 5-59 shows the baseline H&H model results for the area surrounding McClatchy Park in a 10-year design storm.

Figure 5-59: Baseline Flooding near McClatchy Park



McClatchy Park offers an opportunity to detain flow and relieve nearby pipes. In this project, detention and increased conveyance together would work to reduce flooding. Figure 5-60 presents the project components.

Figure 5-60: McClatchy Park Storage Project Components



Description: The first goal of this project is to increase conveyance. Specifically, the project includes the following:

- Upsize 850 LF of 12- and 14-inch-diameter pipe to 54-inch-diameter on Broadway east of the park
- Upsize 500 LF of 10-inch-diameter pipe to 36-inch-diameter on Broadway northeast of the park

- Upsize 2,100 LF of 20- and 24-inch-diameter pipe to 36-inch-diameter on Broadway north of the park
- Upsize 400 LF of 14- and 16-inch-diameter pipe to 30-inch-diameter on 36th Street
- Upsize 1,120 LF of 8-, 10-, and 12-inch-diameter pipe to 24-inch-diameter on 4th Avenue and 37th Avenue

The second goal of the project is to provide detention to the area by installing a 2.7-mg storage vault at McClatchy Park. The storage facility would be connected to Broadway in the east via a weir structure. This location serves to draw down the peak HGL at Broadway, a primary flooding location. A 0.1-mgd pump was sized to drain the storage facility.

Benefits: The project would significantly lower the HGL on Broadway and the smaller pipes north of the park. The project would also successfully detain flow and relieve downstream pipes. Table 5-9 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-61 through Figure 5-64 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-9: Modeled Peak HGL for Node Near McClatchy Park

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Brett Harte Court and 36th Street	24.0	**24.9**	21.3	21.3	**25.0**	**24.1**	22.8

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-61: Pre-Project Performance on Broadway in 10-Year Storm

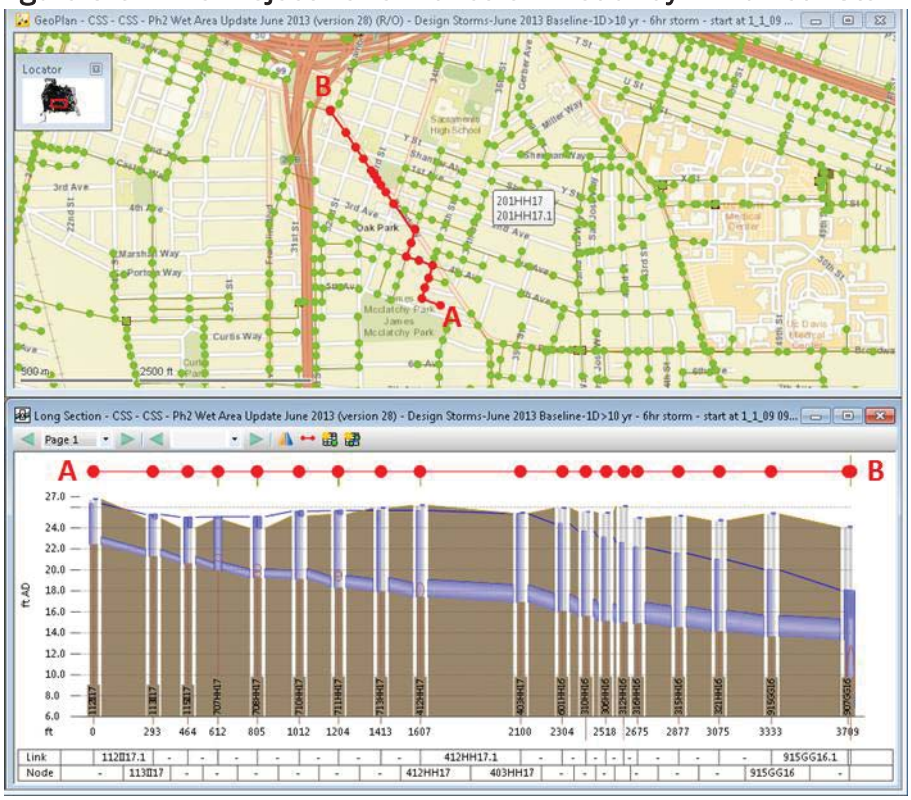


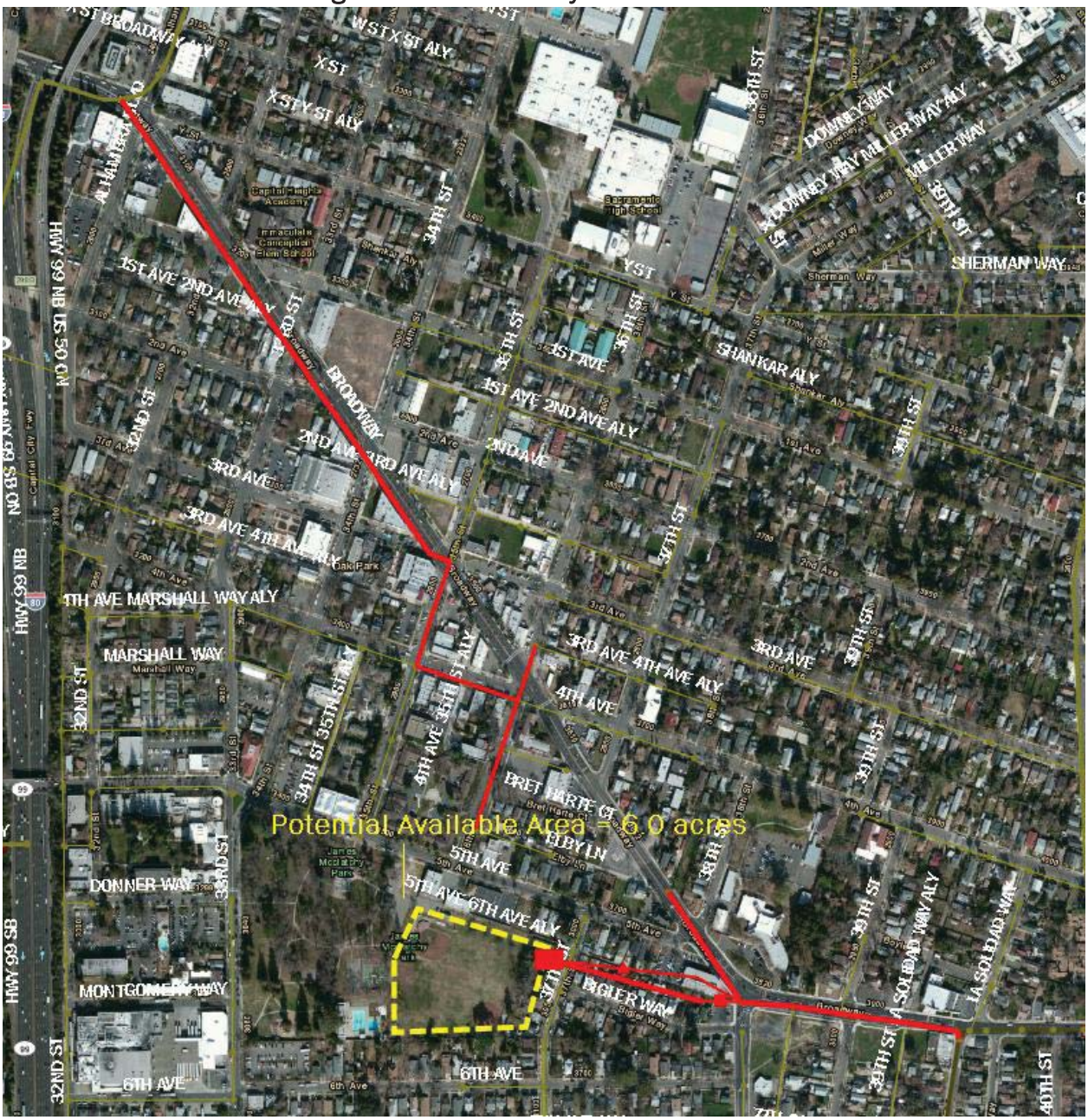
Figure 5-62: Post-Project Performance on Broadway in 10-Year Storm

Figure 5-63: Pre-Project Performance on Broadway in 10-Year Storm

Figure 5-64: Post-Project Performance on Broadway in 10-Year Storm

Feasibility/Constructability: McClatchy Park has been considered by DOU for a storage project before, and was found to be feasible. Broadway is a wide street where a wider pipe appears feasible. The project's upsizing would be extensive, but could potentially be phased. Figure 5-65 presents an aerial view of the project area.

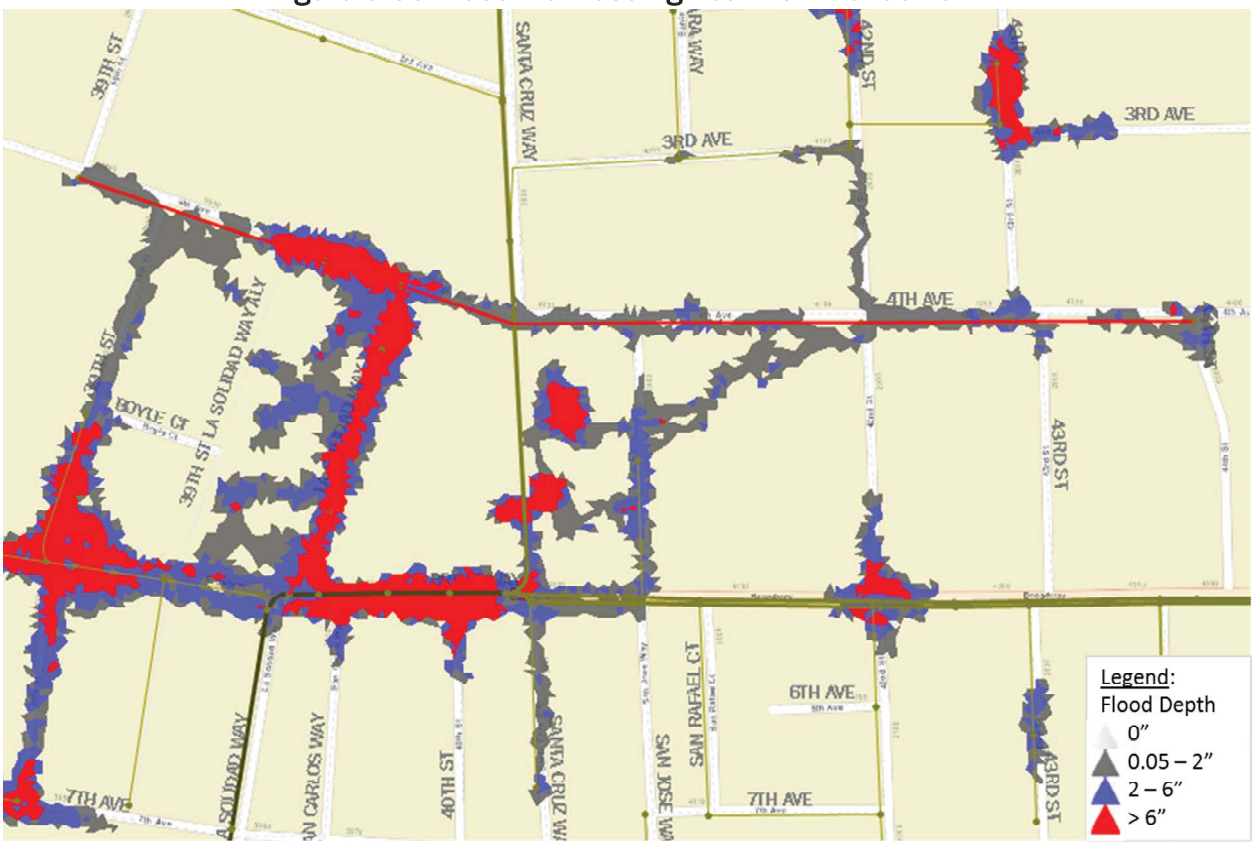
Figure 5-65: McClatchy Park Aerial View



5.10 Project 2-3 (4th Avenue Park)

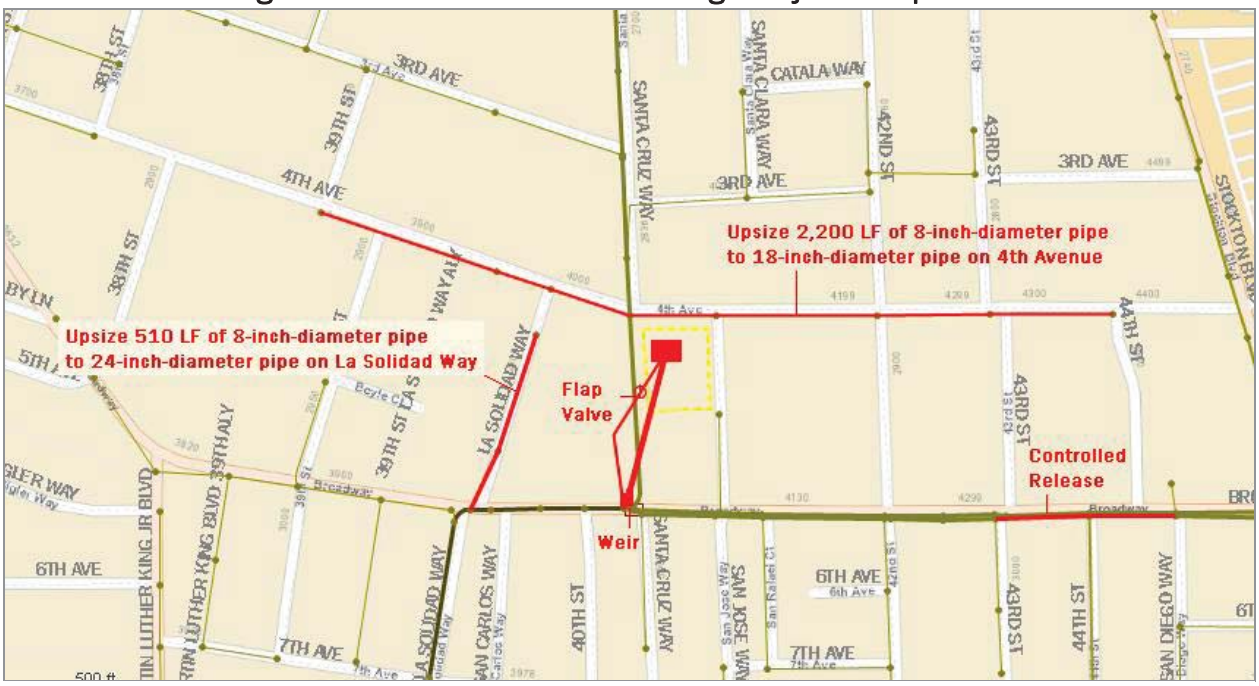
The 4th Avenue Park storage project, when combined with Project 2-2 (McClatchy Park), would further reduce flooding indicated in the model along Broadway and localized flooding along 4th Avenue. 4th Avenue is located to the east of McClatchy Park and one block north of Broadway. The model indicates that localized flooding on 4th Avenue is due to an undersized pipe along this street and downstream surcharged pipe on Broadway, as well as surcharge resulting from a gate at Broadway and Santa Cruz Way. Figure 5-66 shows the baseline H&H model results for the area surrounding 4th Avenue Park in a 10-year design storm.

Figure 5-66: Baseline Flooding near 4th Avenue Park



4th Avenue Park presents an opportunity for a storage project to detain upstream flows and reduce flooding. Figure 5-67 presents the components of the 4th Avenue storage project.

Figure 5-67: 4th Avenue Park Storage Project Components



Description: As with McClatchy Park, this project would be made up of both conveyance and storage elements. The first goal of this project is to increase conveyance on 4th Avenue and near Broadway with the following steps:

- Upsize 2,200 LF of 8-inch-diameter pipe to 18-inch-diameter pipe on 4th Avenue
- Upsize 510 LF of 8-inch-diameter pipe to 24-inch-diameter pipe on La Solidad Way

The project would also provide detention to the area by installing a 2-mg storage vault at 4th Avenue Park. The storage would be connected to Broadway in the south via a weir structure. The weir structure would be located at Broadway and Santa Cruz Way. The storage facility would drain by gravity via an 8-inch-diameter flap valve at Broadway and Santa Cruz Way.

Benefits: The 4th Avenue project would significantly lower the peak HGL in the local area and reduce flooding on 4th Avenue, Broadway, and La Solidad Way. The controlled release upstream on Broadway is effective at detaining flows and more fully utilizing capacity available in the upstream pipes. The controlled release does cause small instances of upstream flooding to worsen, but these instances are addressed by the programs proposed later in this report. Table 5-10 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-68 through Figure 5-71 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-10: Modeled Peak HGL for Node Near 4th Avenue Park

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
4th Avenue and La Solidad Way	19.7	**20.3**	18.8	18.4	**20.5**	**19.9**	**19.8**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-68: Pre-Project Performance on 4th Avenue in 10-Year Storm

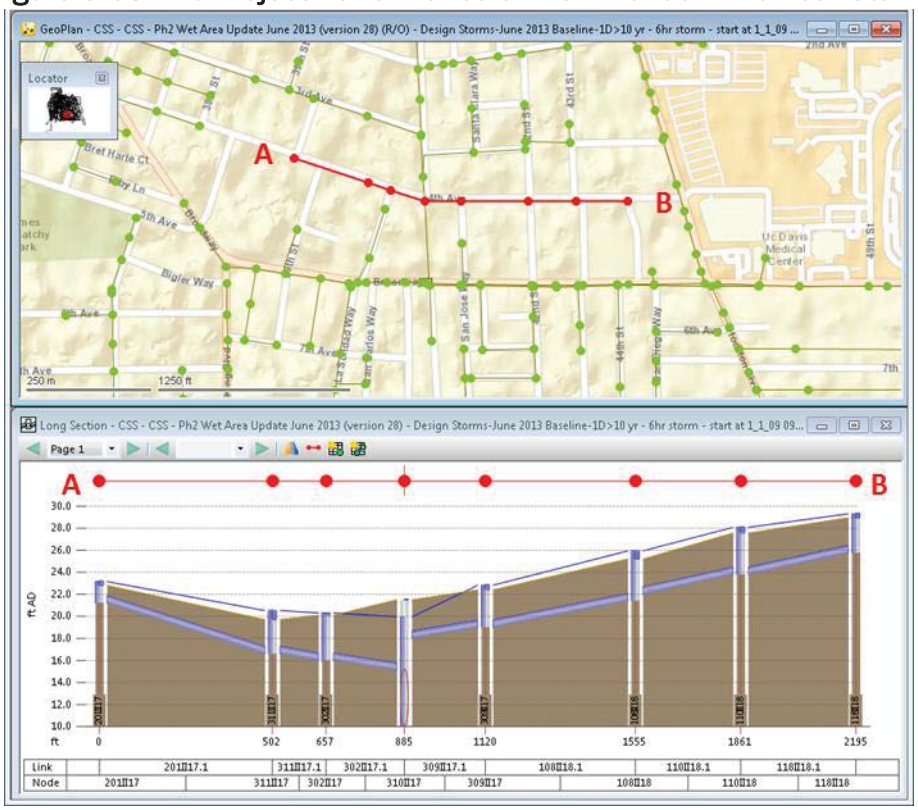


Figure 5-69: Post-Project Performance on 4th Avenue in 10-Year Storm

Figure 5-70: Pre-Project Performance on Broadway in 10-Year Storm

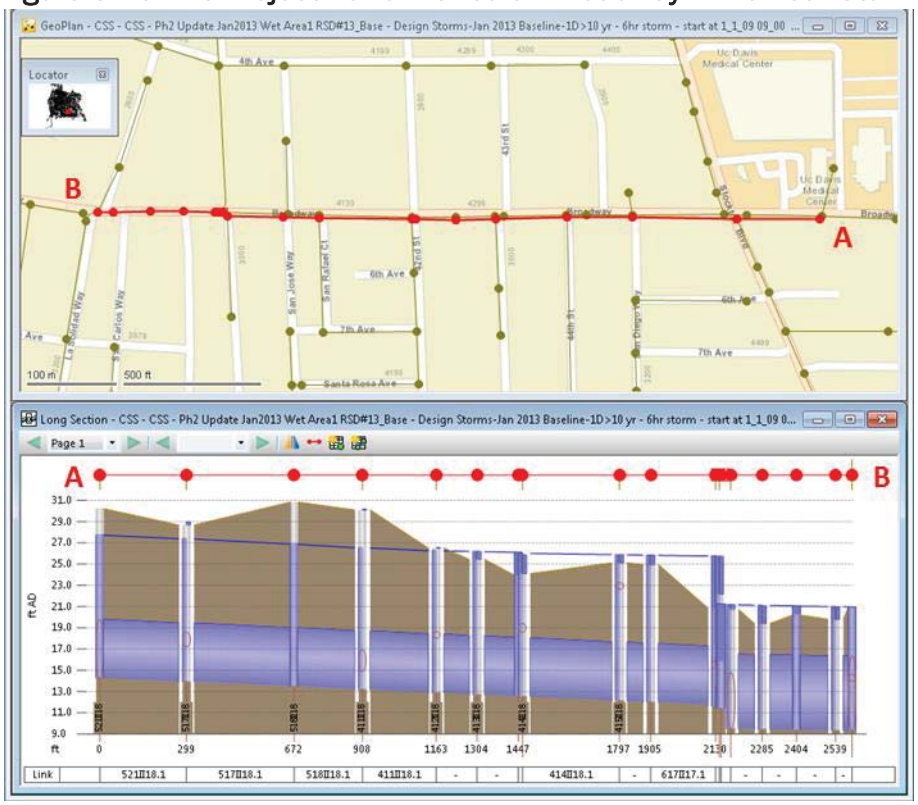
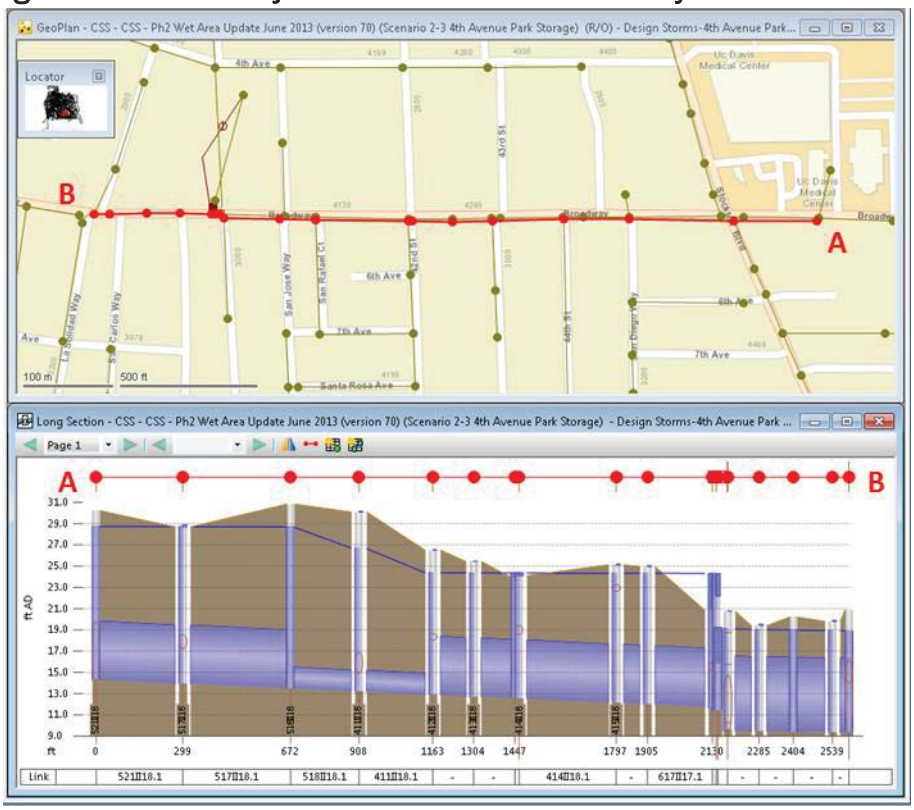


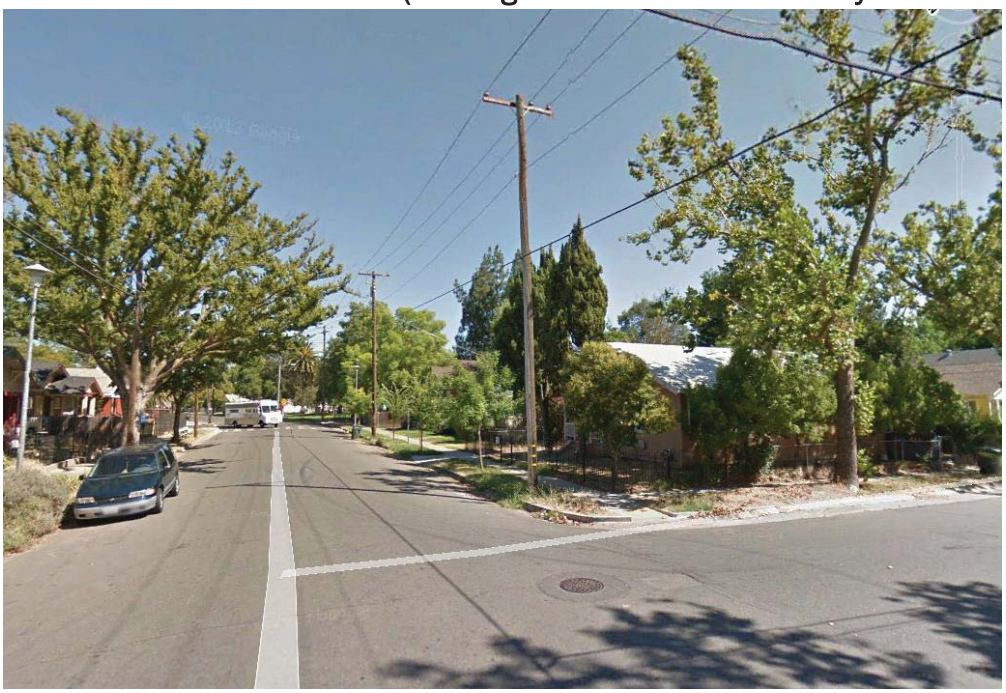
Figure 5-71: Post-Project Performance on Broadway in 10-Year Storm



Feasibility/Constructability: Although 4th Avenue Park has not been considered for a storage project previously, it appears to be a good candidate. This park has significant open area, and has a small play structure that may be in need of rehabilitation. 4th Avenue, La Solidad Way, and Broadway all appear to be sufficiently wide for larger-diameter pipes. The controlled release on Broadway could either be in the form of an orifice or other component which could be determined in a more detailed design process. Figure 5-72 presents an aerial view of the project area and Figure 5-73 presents a street view.

Figure 5-72: 4th Avenue Park Aerial View

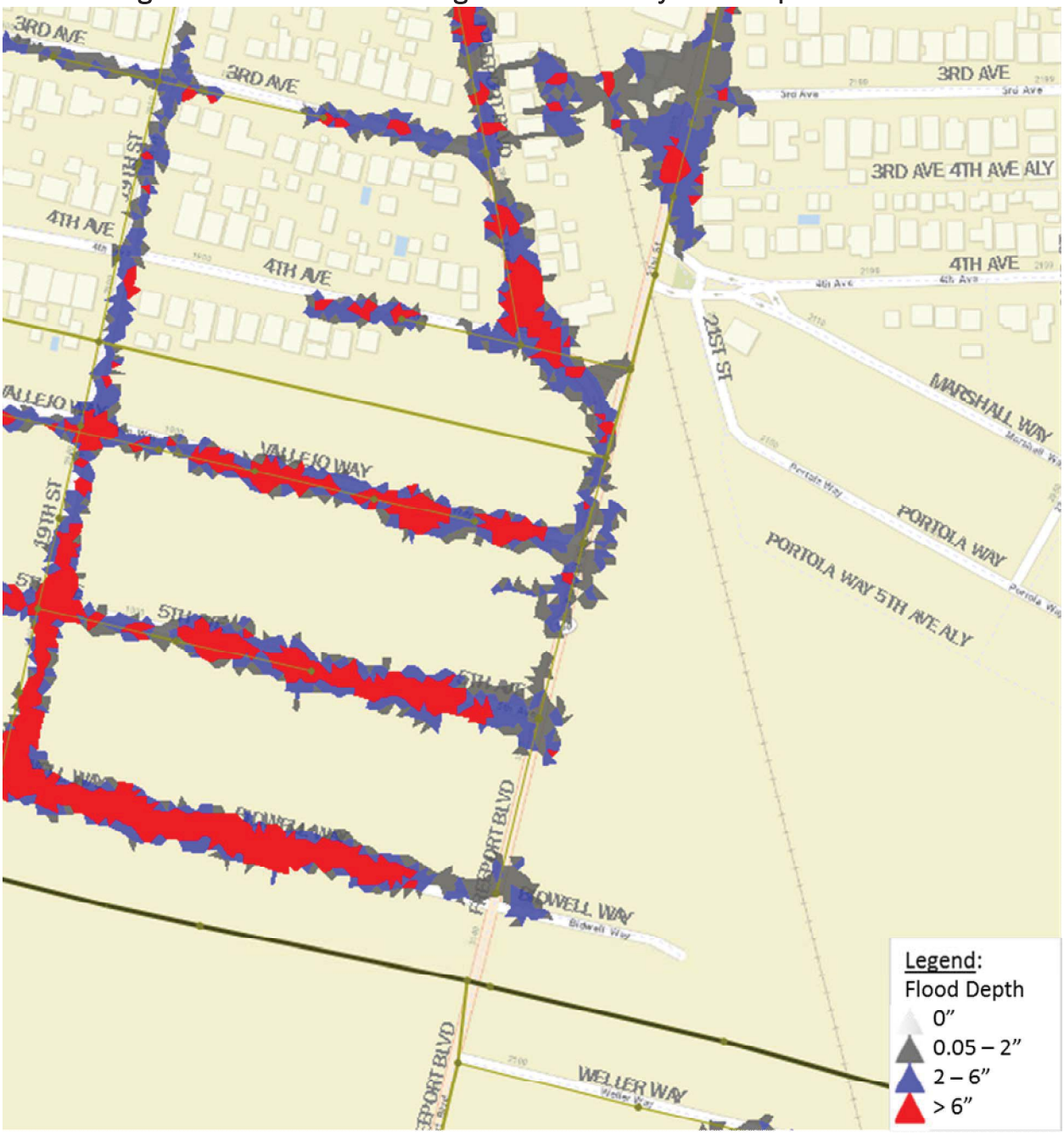
Figure 5-73: 4th Avenue Street View (Looking East from La Solidad Way and 4th Avenue)



5.11 Project 3-1 (Bidwell & Freeport)

The H&H model indicates that Freeport Boulevard experiences flooding, particularly near the intersections with Castro Street and Bidwell Street. The flooding is a result of limited conveyance both locally and downstream. Figure 5-74 shows the baseline H&H model results for the area surrounding the intersection of Bidwell Way and Freeport Boulevard in a 10-year design storm.

Figure 5-74: Baseline Flooding near Bidwell Way and Freeport Boulevard



The “Donner Trunk” of the CSS runs south of the Freeport Boulevard area and has available capacity. This project seeks to use the available capacity by connecting the pipes on Freeport Boulevard to the Donner Trunk and, thus, provide relief for the entire Freeport Boulevard area.

Figure 5-75 presents project components.

Figure 5-75: Freeport and Bidwell Connection Project Components

Description: This project would be made up of the following components:

- Install 1,070 LF of 48-inch-diameter pipe connecting Freeport Boulevard between Vallejo and 4th Avenues to a manhole adjacent to the Donner Trunk east of Freeport Boulevard
- Install 48-inch-diameter flap valve from manhole to Donner Trunk to limit flow direction

The new installation would divert only wet-weather flows to Donner Trunk; dry-weather flows would continue to follow the existing drainage path.

Benefits: The project would benefit Freeport Boulevard by lowering the peak HGL and providing moderate flood reduction in the area. Donner Trunk would not see a significant increase in peak HGL with additional flows from Freeport Boulevard. Table 5-11 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-76 through Figure 5-79 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-11: Modeled Peak HGL for Node Near Bidwell and Freeport Boulevard

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Vallejo Way and Freeport Blvd.	11.6	**12.4**	11.6	10.8	**12.6**	**12.3**	**11.8**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-76: Pre-Project Performance on Freeport Blvd. in 10-Year Storm

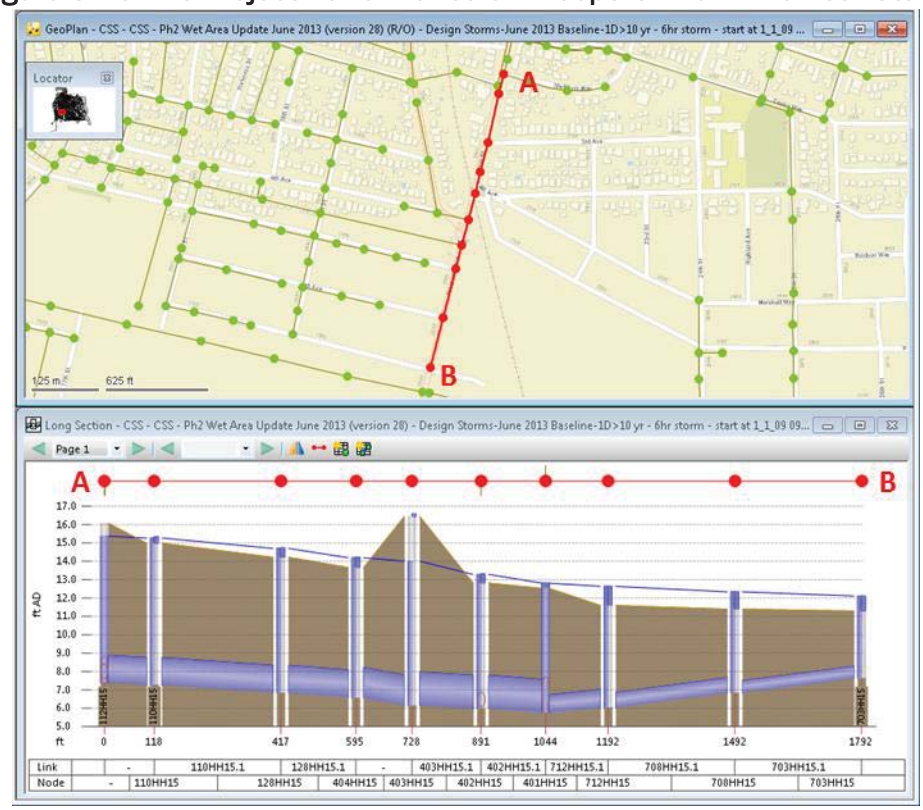


Figure 5-77: Post-Project Performance on Freeport Blvd. in 10-Year Storm

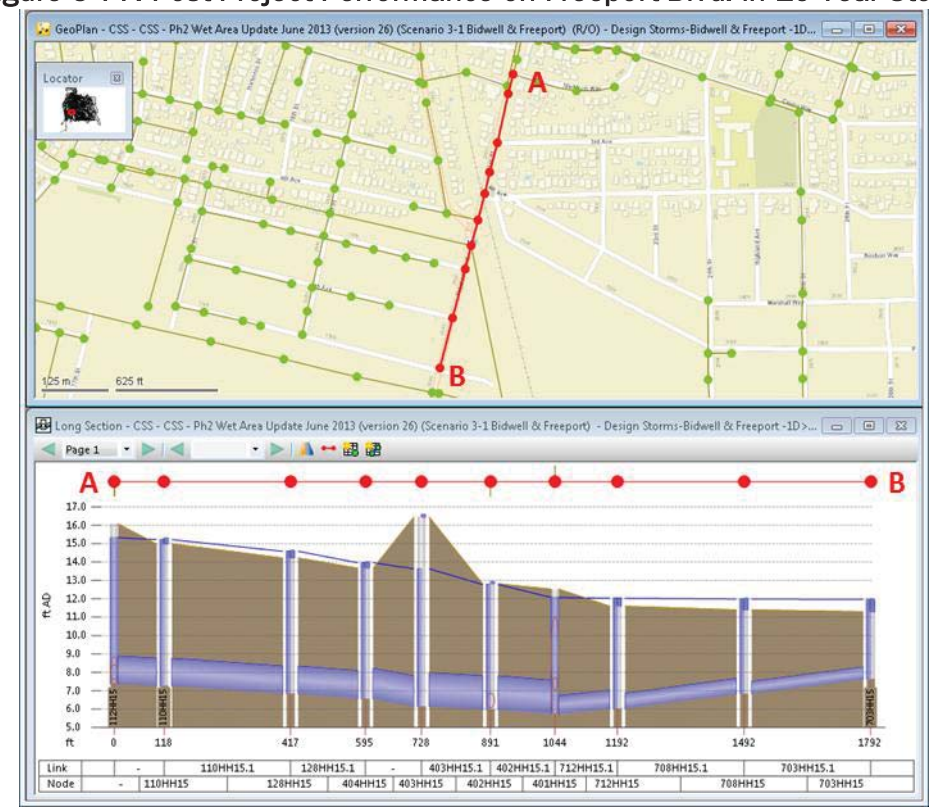


Figure 5-78: Pre-Project Performance in Donner Trunk in 10-Year Storm

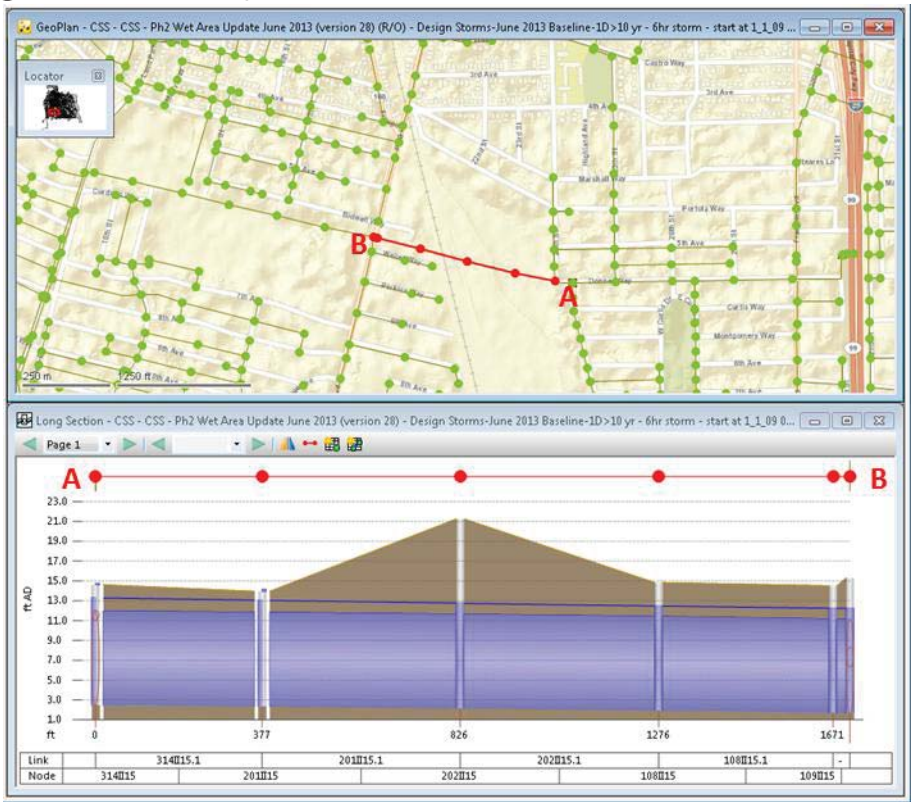
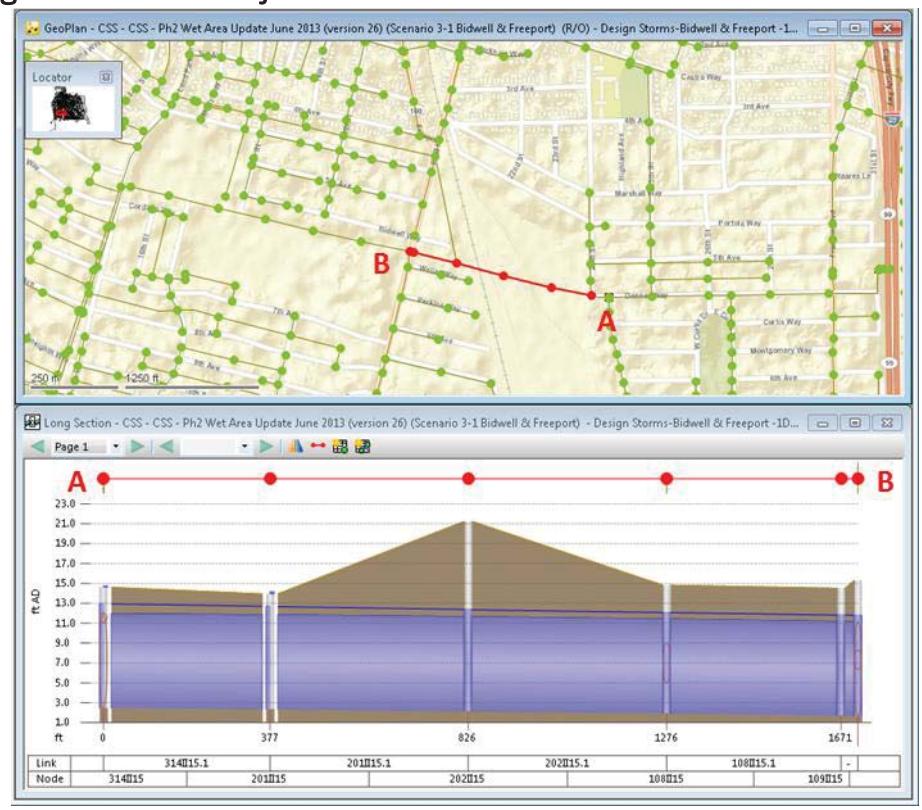


Figure 5-79: Post-Project Performance in Donner Trunk in 10-Year Storm



Feasibility/Constructability: This project requires constructing a new pipe parallel to a railway line. If installing a pipe parallel to the railway line is not feasible, the right-of-way along Freeport Boulevard can also be considered. Freeport Boulevard is a comparitively wide street, and multiple parallel sewer pipes can be constructed along the right-of-way. See Figure 5-80 for an aerial view of the project site.

Figure 5-80: Freeport and Bidwell Aerial View

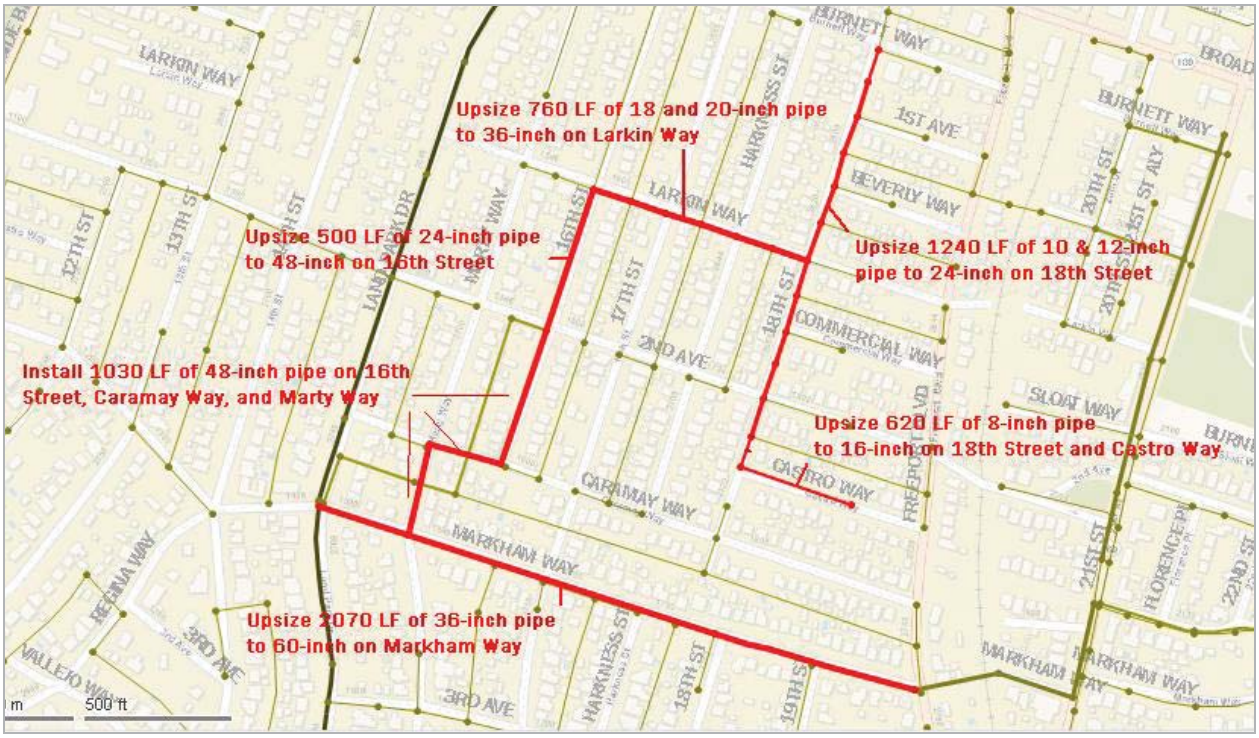
5.12 Project 3-2 (Land Park Tributary Area Upsizing)

The H&H model indicates that the tributary area draining to Land Park Drive has significant flooding. The primary reason for this flooding appears to be small pipes throughout, causing bottlenecks and subsequent flooding. Figure 5-81 shows the baseline H&H model results for the area surrounding Land Park Drive in a 10-year design storm.

Figure 5-81: Baseline Flooding near Land Park Drive

Widespread upsizing is proposed to reduce flooding in the area. Figure 5-82 presents the components of the upsizing project.

Figure 5-82: Land Park Tributary Area Upsizing Components



Description: The project includes upsizing several pipe sections and new pipe installations. Upsizing would consist of the following:

- Upsize 2,070 LF of 36-inch-diameter pipe to 60-inch-diameter on Markham Way
- Upsize 500 LF of 24-inch-diameter pipe to 48-inch-diameter on 16th Street
- Upsize 760 LF of 18- and 20-inch-diameter pipe to 36-inch-diameter on Larkin Way
- Upsize 1,240 LF of 10- and 12-inch-diameter pipe to 24-inch-diameter on 18th Street
- Upsize 620 LF of 8-inch-diameter pipe to 16-inch-diameter on 18th Street and Castro Way

The new pipe installations include the following:

- Install new 1,030 LF of 48-inch-diameter pipe between 2nd Avenue and Markham Way with pipe sections on 16th Street, Caramay Way, and Marty Way. This route was chosen in an effort to minimize construction in back yards.

Benefits: The project would significantly lower the peak HGL and reduce flooding in the area. Flooding would especially be reduced along 18th Street and along the pipes flowing to 18th Street. Table 5-12 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-83 and Figure 5-84 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-12: Modeled Peak HGL for Node Near Land Park Drive

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Castro Way between 18th Street and Freeport Blvd.	11.6	**12.4**	**11.8**	10.0	**12.6**	**12.0**	**11.9**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-83: Pre-Project Performance on 18th Street in 10-Year Storm

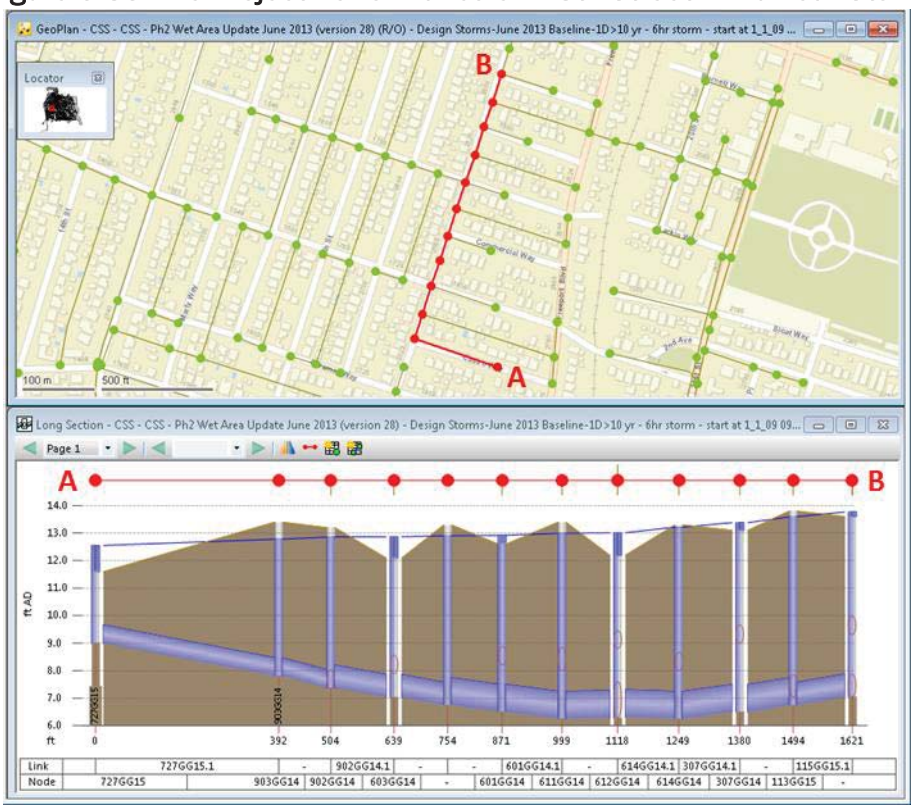


Figure 5-84: Post-Project Performance on 18th Street in 10-Year Storm

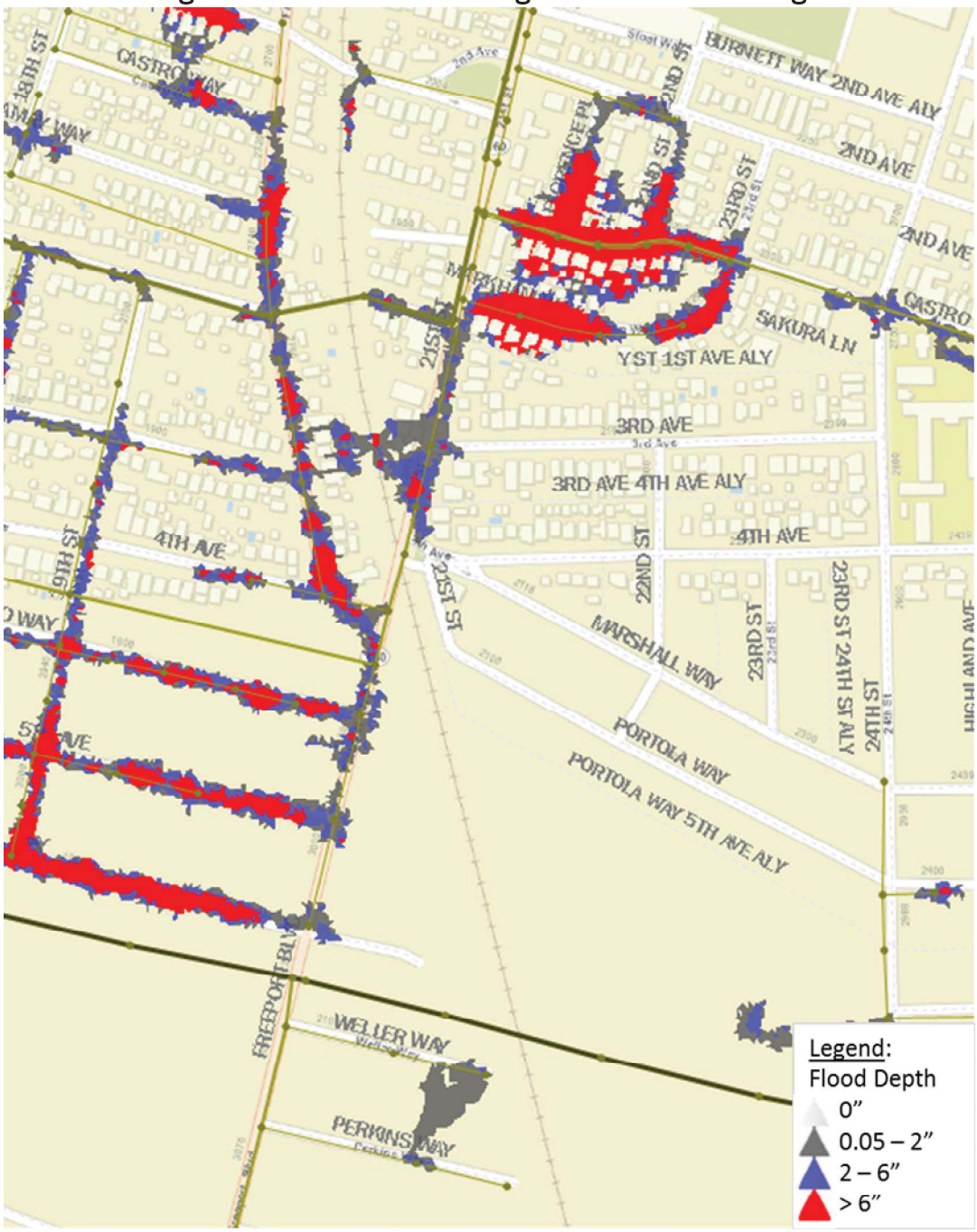
Feasibility/Constructability: This project would avoid upsizing or installing pipes in back yards to increase its feasibility. The magnitude of the upsizing for this project is large, but could be scheduled over several phases.
Figure 5-85 shows an aerial view of the project area for reference.

Figure 5-85: Land Park Tributary Area Aerial View

5.13 Project 3-3 (Curtis Park Village Storage)

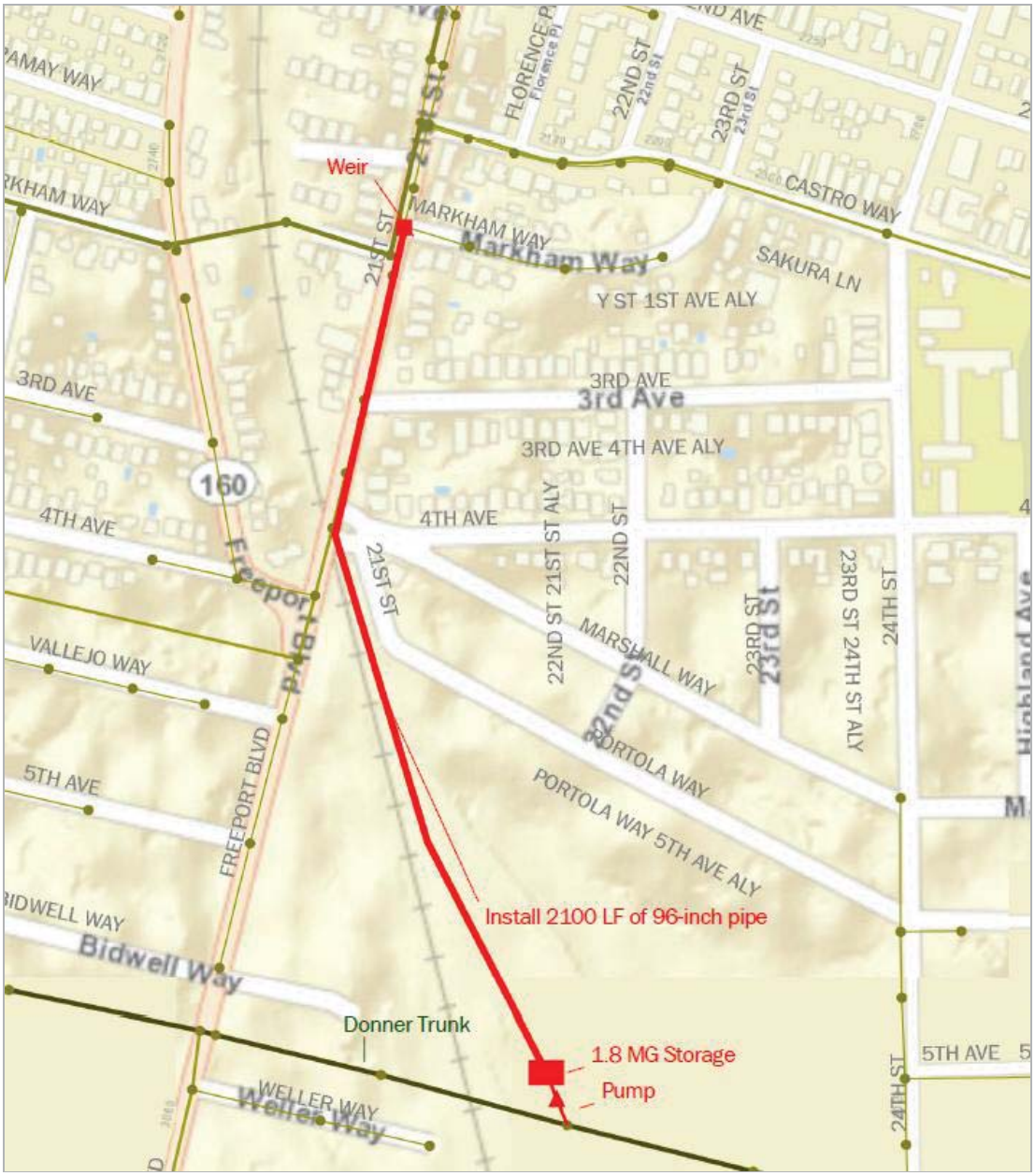
Curtis Park Village is a development being constructed east of Freeport Boulevard. As part of the development plans, DOU will gain right-of-way priviledges to an area parallel to the Donner Trunk. DOU has been considering installing a storage facility here to detain CSS flows in wet weather. Figure 5-86 shows the baseline H&H model results for the area surrounding Curtis Park Village in a 10-year design storm.

Figure 5-86: Baseline Flooding near Curtis Park Village



The project proposed at this site would collect flows from 21st Street and Markham Way. This intersection is upstream of the Freeport and Bidwell area, and downstream of Castro and 21st Streets; both areas have flooding indicated in the model. Detaining flows in a storage site would reduce flooding from both upstream and downstream areas. See Figure 5-87 for the components of this project.

Figure 5-87: Curtis Park Village Storage Project Components



Description: The would project involve the following pipe installation:

- Install 2,100 LF of 96-inch-diameter pipe from Markham and 21st Street to Donner Trunk

The storage facility includes a 1.8-mg combined sewer storage vault underneath the Curtis Park Village right-of-way along the Donner Trunk. One weir structure at Markham and 21st Street would convey flow to the storage facility at Curtis Park Village. The weir invert elevation would be set above peak dry-weather depth in the connecting pipe. A 0.25 mgd pump was sized to drain the storage facility.

Benefits: The project would significantly lower the peak HGL for the pipe on 21st Street. The model indicates that the project would eliminate flooding on the local section of 21st Street during a 10-year storm. Table 5-13 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-88 and Figure 5-89 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-13: Modeled Peak HGL for Node Near Curtis Park Village

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Castro Way between 22nd and 23rd Streets	14.6	**16.0**	**15.6**	14.0	**16.3**	**16.0**	14.4

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-88: Pre-Project Performance on 21st Street in 10-Year Storm

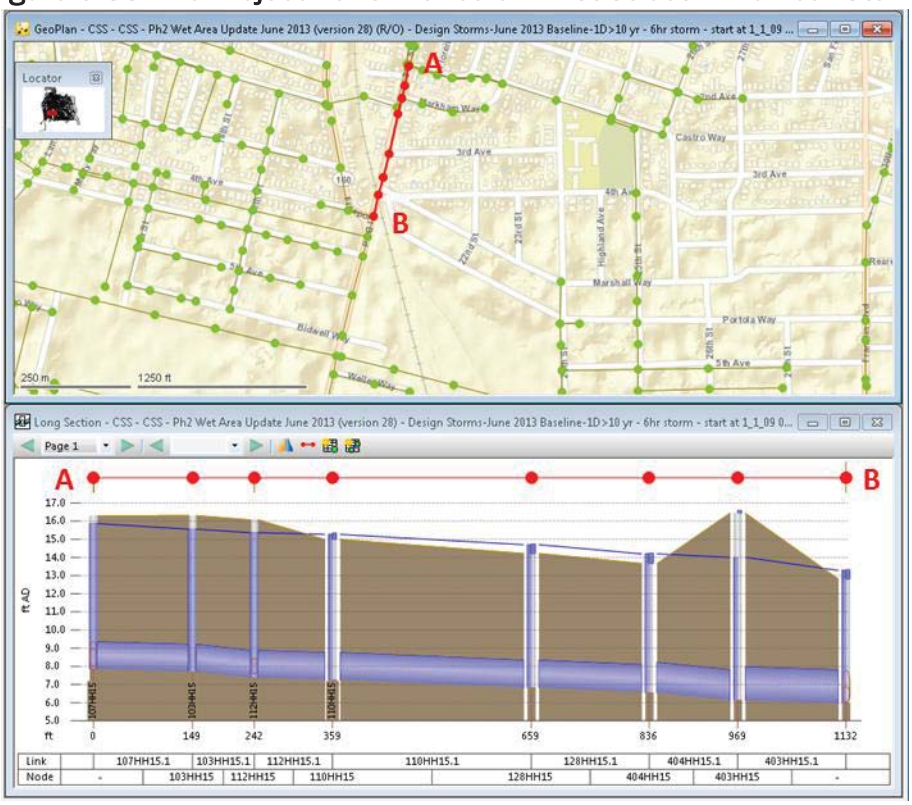
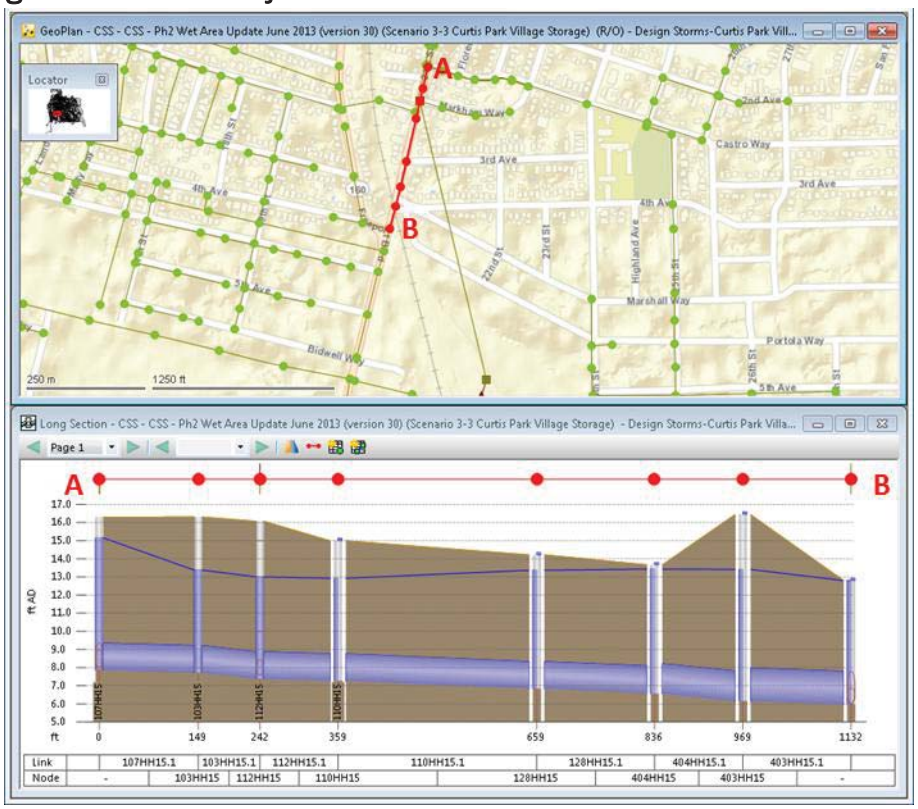


Figure 5-89: Post-Project Performance on 21st Street in 10-Year Storm



Feasibility/Constructability: The project would use the future right-of-way at the Curtis Park Village development along the Donner Trunk. DOU has previously considered storage here. The project is necessary due to large-scale flooding at the intersection of Castro and 21st Street. See Figure 5-90 for an aerial view of the project site.

Figure 5-91 shows a street view of Markham and 21st Streets.

Currently, DOU has been unable to negotiate to construct a project within Curtis Park Village. The City is planning to have a siting study done to re-site this project Proposed Project 6-3 (Curtis Park Storage) is a potential site as alternative to this project. The CSSIP currently includes both the projects for flood reduction benefits. If this storage project is not feasible, further analysis will be required to either place a larger storage at Curtis Park Storage (Project 6-3), or new storage at the alternative site.

Figure 5-90: Curtis Park Village Aerial View

Figure 5-91: Markham and 21st Street View (Looking South)

5.14 Project 3-4 (Sierra School Park)

Sierra School Park offers a beneficial location for a detention project. The H&H model indicates that the area around the school, particularly 2nd Avenue to the north of the park, experiences localized flooding, and areas downstream of the park also experience flooding. Figure 5-92 shows the baseline H&H model results for the area surrounding Sierra School Park in a 10-year design storm.

Figure 5-92: Baseline Flooding near Sierra School Park

By detaining flows at a storage vault underneath Sierra School Park, localized flooding can be eliminated and downstream flooding can be reduced. Figure 5-93 presents the system components for this project.

Figure 5-93: Sierra School Park Storage Project Components

Description: The project would include both upsizing and detention. The upsizing portion involves the following:

- Upsize 350 LF of 16- and 20-inch-diameter pipe to 30-inch-diameter on 25th Street
- Upsize 850 LF of 12-inch-diameter pipe to 30-inch-diameter on 2nd Avenue

The detention facility would include a 1.6-mg storage vault underneath Sierra School Park. A weir structure at Castro and 25th Street would connect the storage facility to the collection system to convey flows from the flooding areas to the storage facility. The storage facility would drain by gravity via a 6-inch-diameter flap valve on Castro Way.

Benefits: The Sierra School Park Project would eliminate flooding in the immediate area around the park by significantly lowering the peak HGL. The project would also benefit downstream areas by reducing the amount of flow entering downstream pipes. Table 5-14 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-94 and Figure 5-95 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-14: Modeled Peak HGL for Node Near Sierra School

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Castro Way between 22nd and 23rd Streets	14.6	**16.0**	14.5	14.0	**16.3**	**15.0**	14.4

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-94: Pre-Project Performance near Sierra School Park in 10-Year Storm

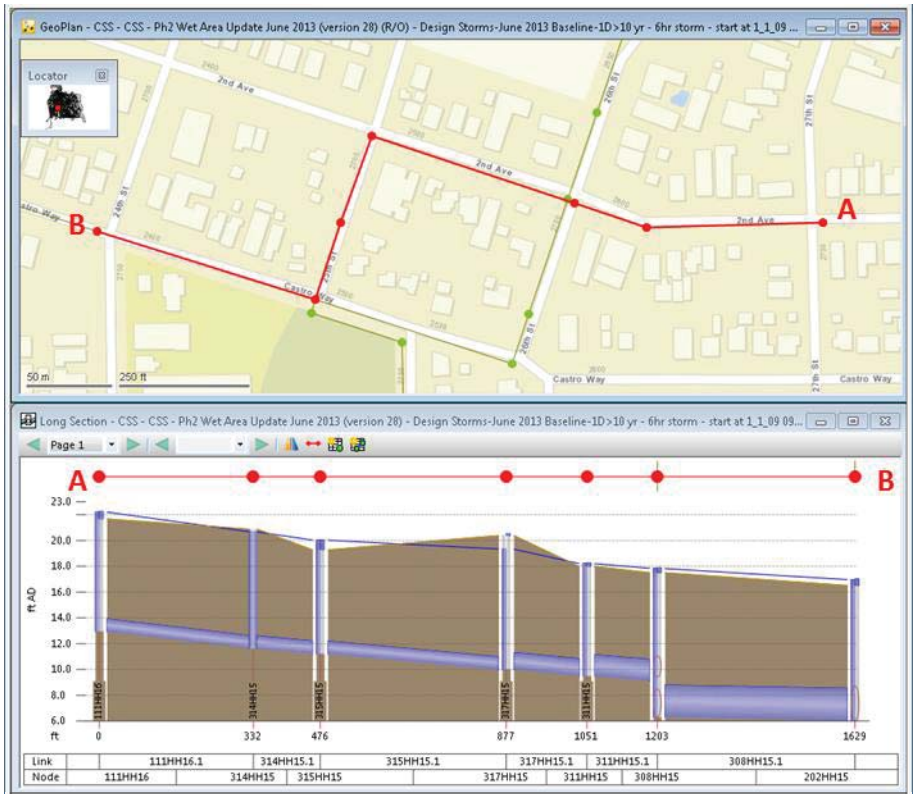
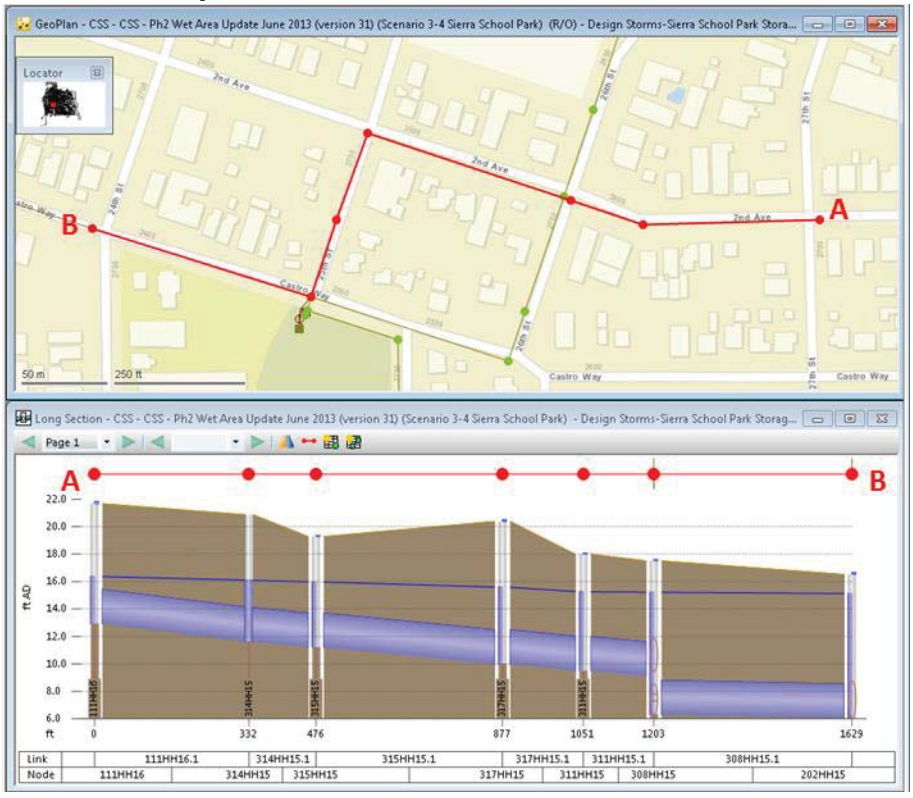


Figure 5-95: Post-Project Performance near Sierra School Park in 10-Year Storm



Feasibility/Constructability: The conveyance portion of the project appears to be feasible because all upsizing would involve wide streets. Storage projects in parks have been feasible in the past for DOU, and Sierra School Park appears to be feasible as well. Figure 5-96 displays an aerial of the project area.

Figure 5-96: Sierra School Park Storage Project Components

5.15 Project 3-5 (Beverly Way)

The H&H model indicates flooding in the area near the intersection of Beverly Way and 13th Street. Flooding in the area is due to small pipes draining to surcharged downstream pipes. Figure 5-97 shows the baseline H&H model results for the area surrounding Beverly Way in a 10-year design storm.

Figure 5-97: Baseline Flooding near Beverly Way

Parcel storage is not feasible near the intersection, and upsizing does not address downstream issues. In-line storage was found to be the best way to relieve flooding in the area and to reduce the load on downstream pipes. Figure 5-98 presents the system components for this project.

Figure 5-98: Beverly Way Storage Project Components

Description: The project would consist of pipe installed on Beverly Way and 13th Street. Specifically, the project would entail the following:

- Install 700 LF of 72-inch-diameter pipe on Beverly Way
- Install 400 LF of 72-inch-diameter pipe on 13th Street

Two weir structures would convey flow from the collection system to the storage facility. The first weir structure would be at Beverly and 13th Street, and the second weir structure would be at 1st and 13th Street. The storage facility would be gravity-drained via a 6-inch-diameter flap valve at 1st and 13th Street.

Benefits: The Beverly Way project would reduce the peak HGL in surrounding pipes. The project would achieve moderate success at reducing flooding, but is limited by the lack of downstream capacity. Ideally, this project would be combined with downstream conveyance projects for maximum benefit. Table 5-15 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-99 and Figure 5-100 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-15: Modeled Peak HGL for Node Near Beverly Way

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
13th Street between Beverly Way and 1st Avenue	11.0	**12.0**	11.0	9.6	**12.3**	**11.8**	**11.5**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-99: Pre-Project Performance on 1st Avenue in 10-Year Storm

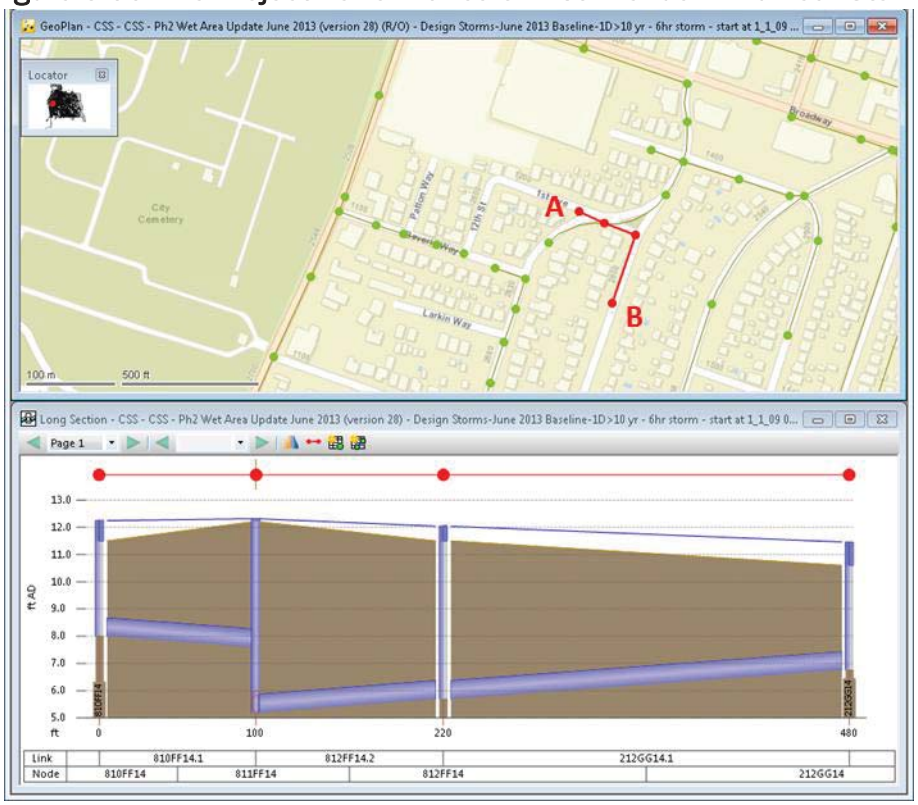
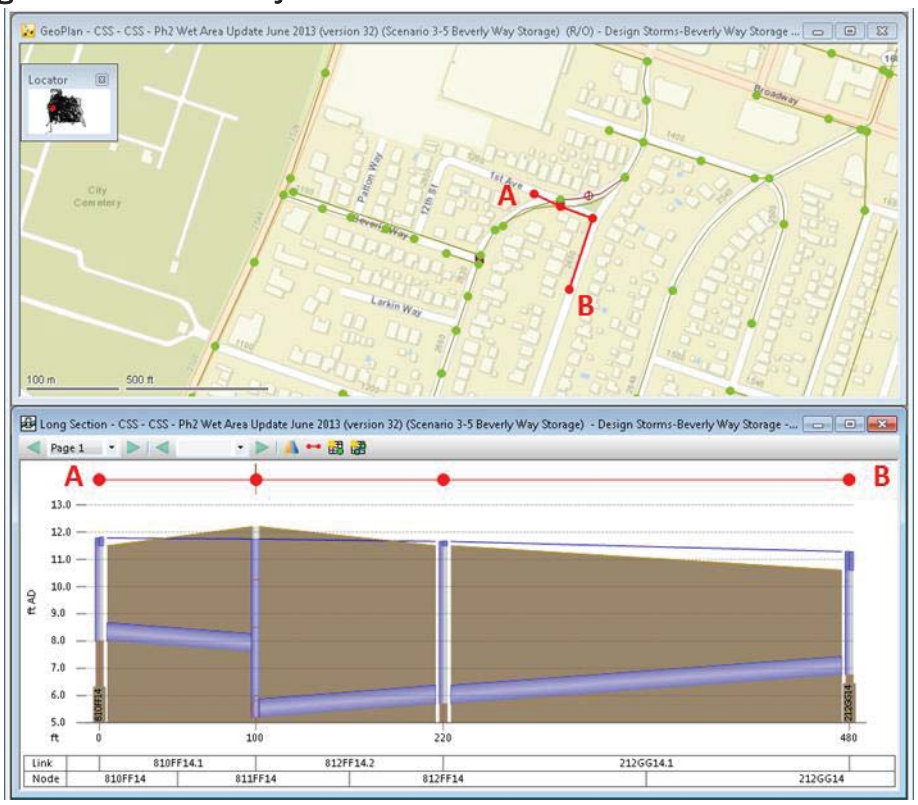


Figure 5-100: Post-Project Performance on 1st Avenue in 10-Year Storm



Feasibility/Constructability: Beverly Way is a medium-width residential street that may be able to hold a large storage pipe. Overall, the project appears to be feasible. Figure 5-101 presents an aerial view of the project site and
Figure 5-102 presents a street view of Beverly Way.

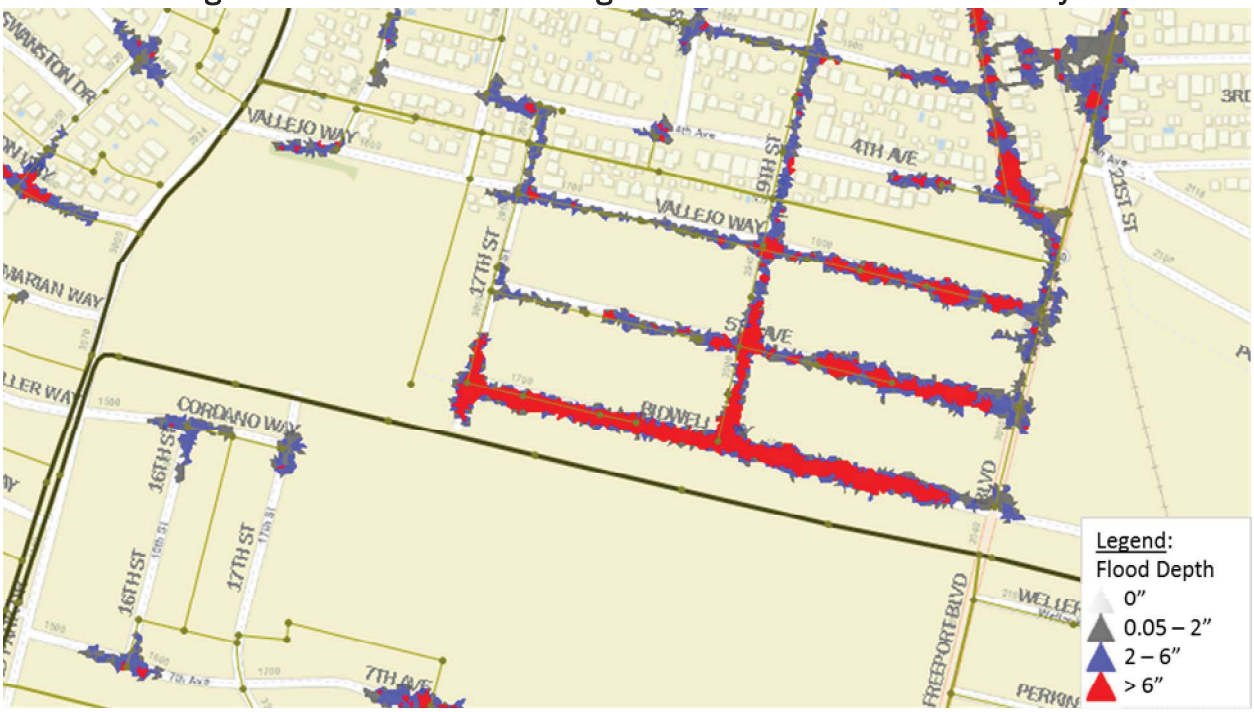
Figure 5-101: Beverly Way Aerial View

Figure 5-102: Beverly Way Street View (Looking West from Beverly Way and 13th Street)

5.16 Project 3-6 (17th and Bidwell Storage)

The H&H model indicates flooding at the intersection of 17th Street and Bidwell Way and in the area nearby. This is partially due to the limited capacity of the pipes here, but also the low elevation of the area. Furthermore, the larger pipes north of this area have a higher HGL, causing flows from the CSS to backflow into this area. Figure 5-103 shows the baseline H&H model results for the area surrounding 17th Street and Bidwell Way in a 10-year design storm.

Figure 5-103: Baseline Flooding near 17th Street and Bidwell Way



With the proximity of California Middle School and the Donner Trunk, isolating the area and storing excess flows in a detention facility appears to be the best solution. Figure 5-104 presents the system components for this project.

Figure 5-104: 17th Street and Bidwell Way Storage Project Components

Description: The 17th Street and Bidwell project would implement an 18-inch-diameter flap valve at Vallejo and 17th Street to prevent a backflow condition. A detention facility would be installed at 17th Street and Bidwell, possibly under the school parking lot or athletic field, with a storage volume of 0.3 mg. The weir structure connecting the storage facility to the existing system would be placed at 17th Street and Bidwell, and the weir invert would be set above the dry-weather peak level of the connected pipe. The storage facility would drain by gravity to the Donner Trunk via a 6-inch-diameter flap valve.

Benefits: The project would greatly benefit the 17th Street and Bidwell area by bringing the 10-year storm HGL below ground in the immediate area. Table 5-15 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-105 and Figure 5-106 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-16: Modeled Peak HGL for Node Near 17th Street & Bidwell

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Bidwell Way and 17th Street	9.4	**10.2**	6.7	6.7	**10.5**	6.8	6.8

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-105: Pre-Project Performance at 17th and Bidwell in 10-Year Storm

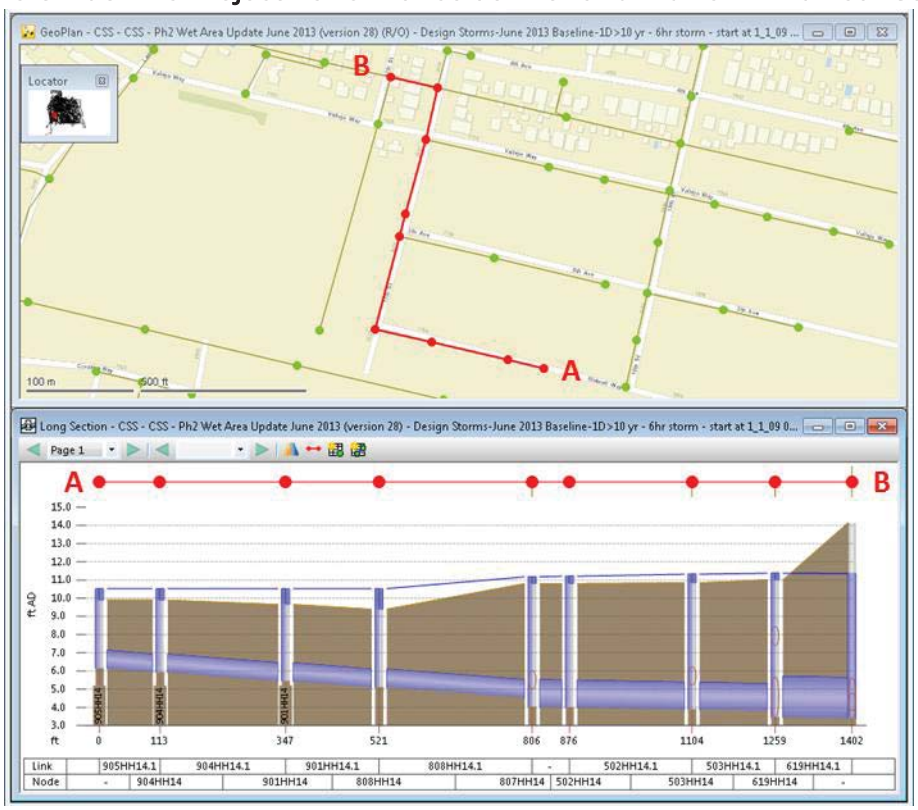
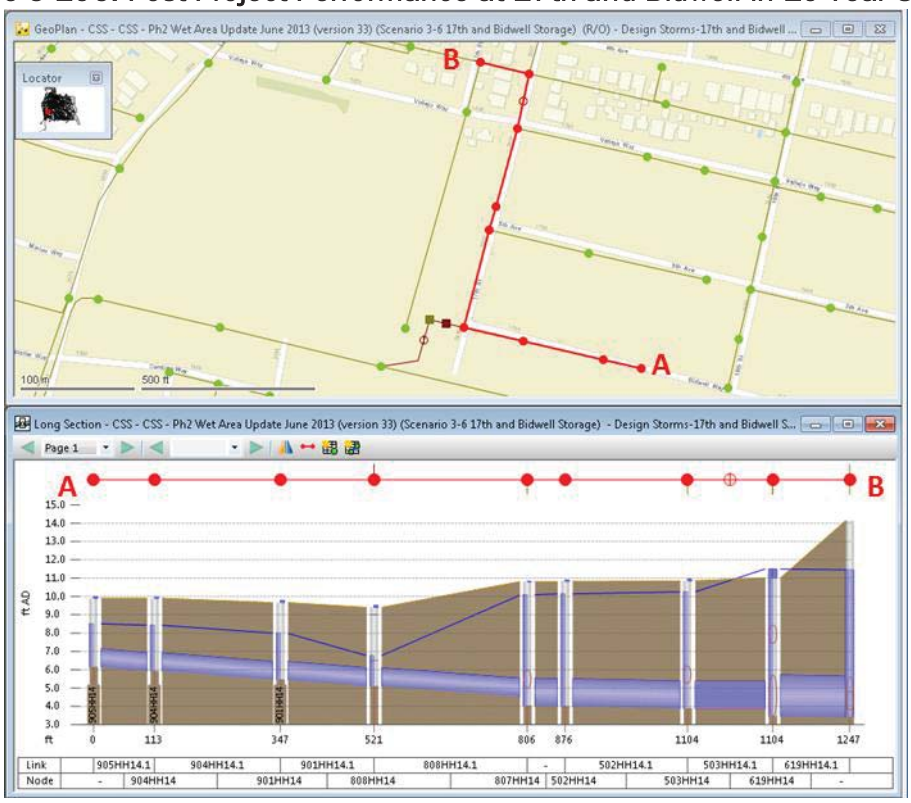


Figure 5-106: Post-Project Performance at 17th and Bidwell in 10-Year Storm



Feasibility/Constructability: The feasibility of this project largely depends on cooperation from California Middle School. If the storage facility can be installed there, the project has the potential to provide great benefit for the surrounding low-lying area.
Figure 5-107 provides an aerial view of the project site.

Figure 5-107: 17th Street Bidwell Storage Project Components

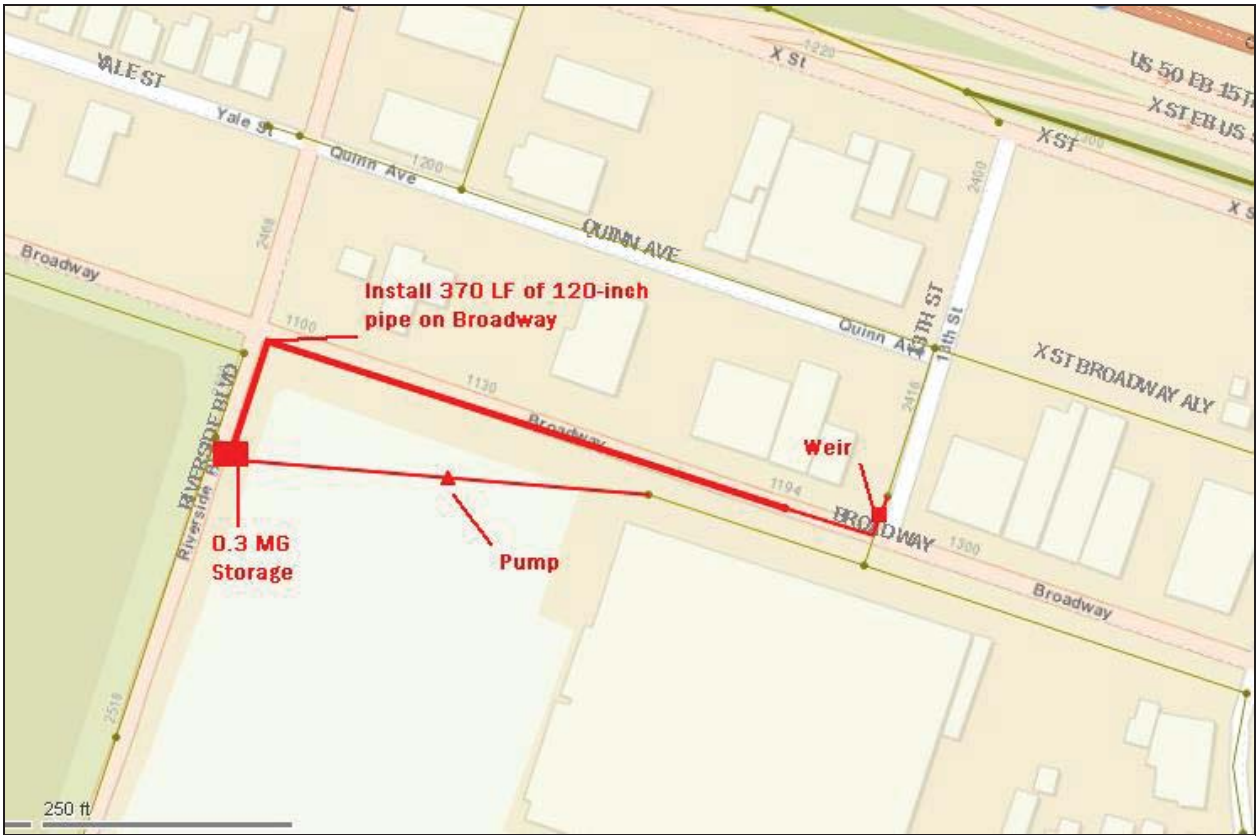
5.17 Project 3-7 (Target Parking)

Parcel storage opportunities in Wet Area 3 are fairly limited, and in an effort to increase the number of possible sites, DOU suggested the -of-way in the Target store parking lot at Riverside Boulevard and Broadway as a possibility. The right-of-way for Riverside Boulevard is 100 feet wide and extends 30 feet into the Target store parking lot. The H&H model indicates flooding in the area, especially north of the Target store, making this a highly desirable site for storage vault. Figure 5-108 shows the baseline H&H model results for the area surrounding the Target store parking lot in a 10-year design storm.

Figure 5-108: Baseline Flooding near Target Store Parking Lot

Figure 5-109 presents the system components for this project.

Figure 5-109: Target Storage Project Components



Description: The main component of the project is a 0.3 mg storage facility installed under the current Target store parking lot. A weir structure would connect the storage facility to the existing system at Broadway and 13th Street. A 120-inch-diameter conduit of 370 LF would bring flow from the weir structure to the storage facility. A 0.05 mgd pump was sized to drain the storage facility.

Benefits: The project would be effective at reducing flooding in the immediate area. This project could be combined with a downstream conveyance project, such as Project 6-2 Riverside Boulevard Upsizing, for maximum benefit. Table 5-17 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-110 and Figure 5-111 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-17: Modeled Peak HGL for Node Near Target Parking

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
X Street near Riverside Blvd.	11.9	**13.0**	**13.0**	**12.8**	**13.4**	**13.3**	**13.3**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-110: Pre-Project Performance at Broadway and 13th Street in 10-Year Storm

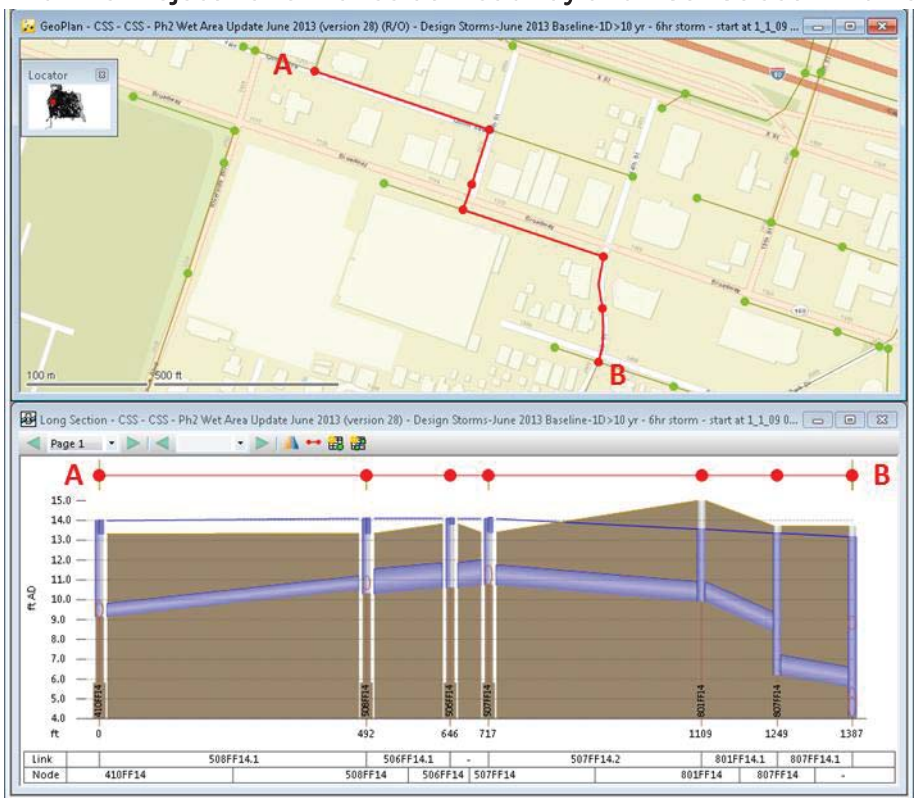
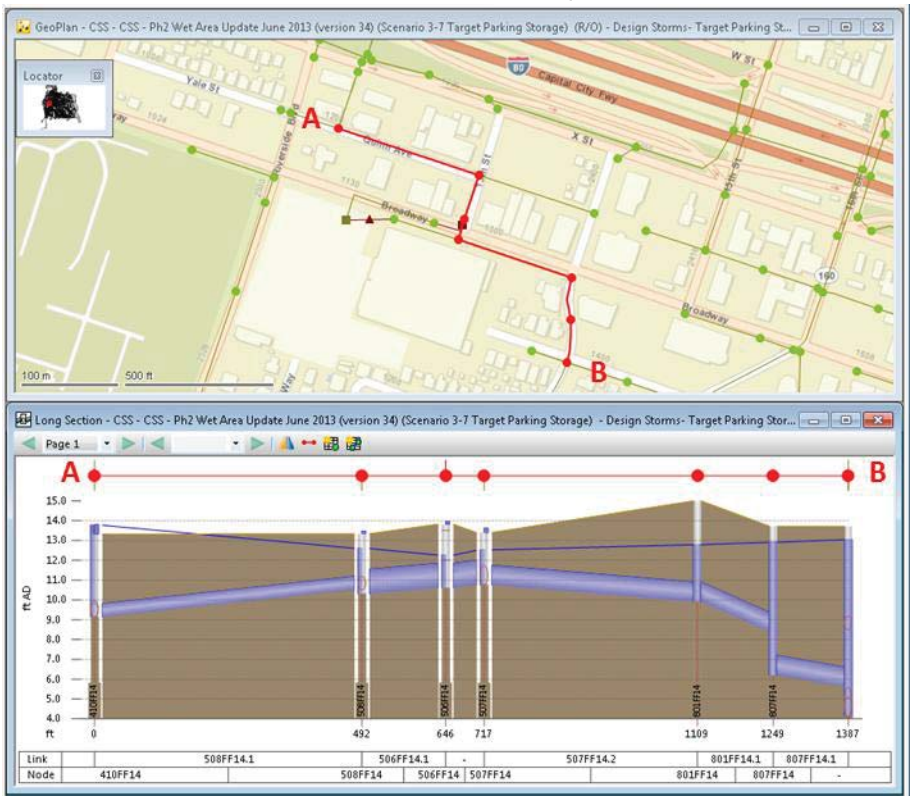


Figure 5-111: Post-Project Performance at Broadway and 13th Street in 10-Year Storm



Feasibility/Constructability: The Target parking lot is anecdotally known by DOU to flood in major storm events, and the store owner may be motivated to support and share construction costs for a storage facility that would eliminate flooding near the store. Constructing new infrastructure on Broadway appears feasible, as the road is wide and there are no large sewer pipes currently existing along the block. Figure 5-112 presents an aerial view of the site.

Figure 5-112: Target Storage Aerial View

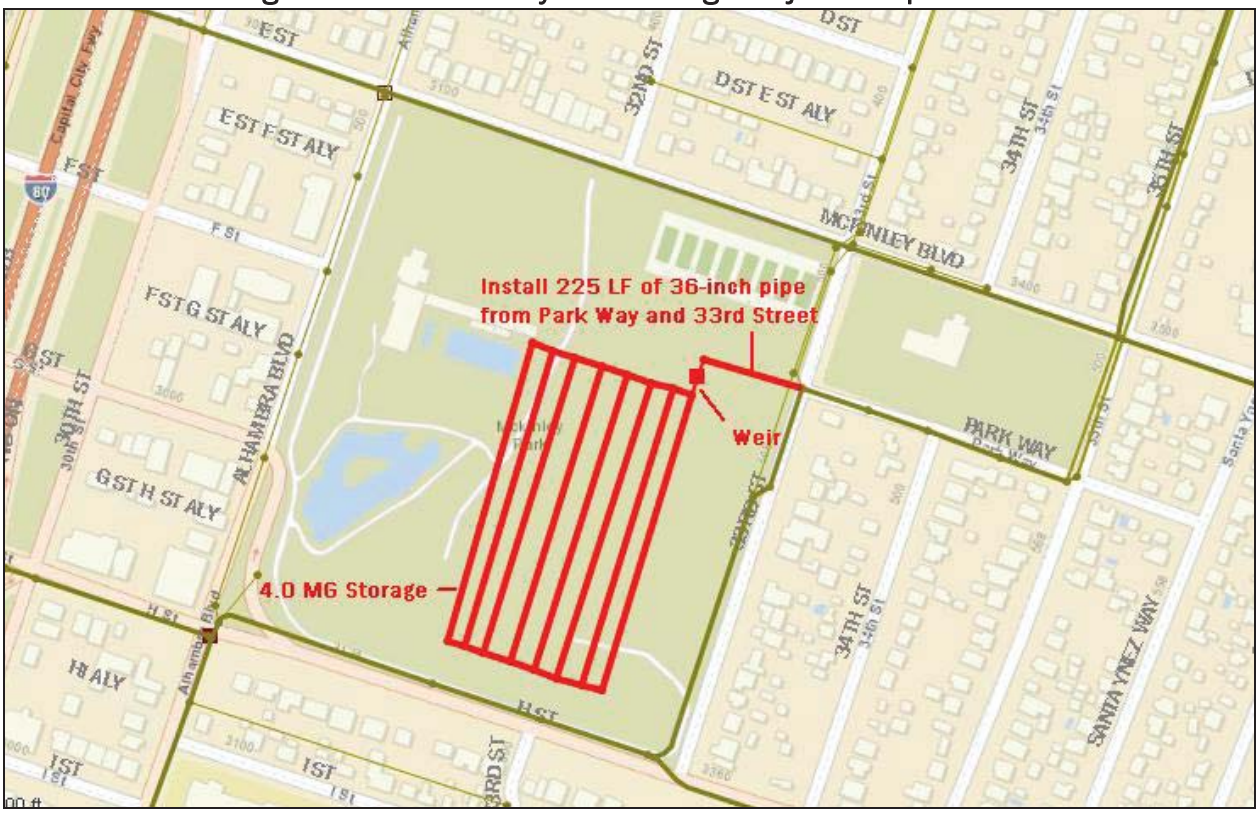
5.18 Project 4-1 (McKinley Park Storage)

The H&H model indicates that the area around McKinley Park in Wet Area 4 experiences flooding during the design storms. The pipe east of the park on Park Way and 3rd Street surcharges and causes flooding. Figure 5-113 shows the baseline H&H model results for the area surrounding McKinley Park in a 10-year design storm.

Figure 5-113: Baseline Flooding near McKinley Park

The location of McKinley Park is ideal to relieve this problem with a storage vault constructed underneath the park. Figure 5-114 presents the system components for this project.

Figure 5-114: McKinley Park Storage Project Components



Description: The main component of the project is a 4 mg storage facility installed underneath McKinley Park. There are several design approaches to build a storage facility; DOU's preference for the storage vault at this location is a set of large pipes laid out in parallel. The project also includes installing 225 LF of 36-inch-diameter pipe from Park Way and 33rd Street to a weir structure immediately before the storage facility at McKinley Park. A 4 mgd pump was sized to drain the storage facility.

Benefits: The project would significantly reduce flooding in the immediate area. Table 5-18 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-115 and Figure 5-116 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-18: Modeled Peak HGL for Node Near McKinley Park

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Park Way and 34th Street	16.9	**18.9**	**17.6**	15.3	**19.5**	**18.1**	16.5

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-115: Pre-Project Performance on Park Way and 3rd Street in 10-Year Storm

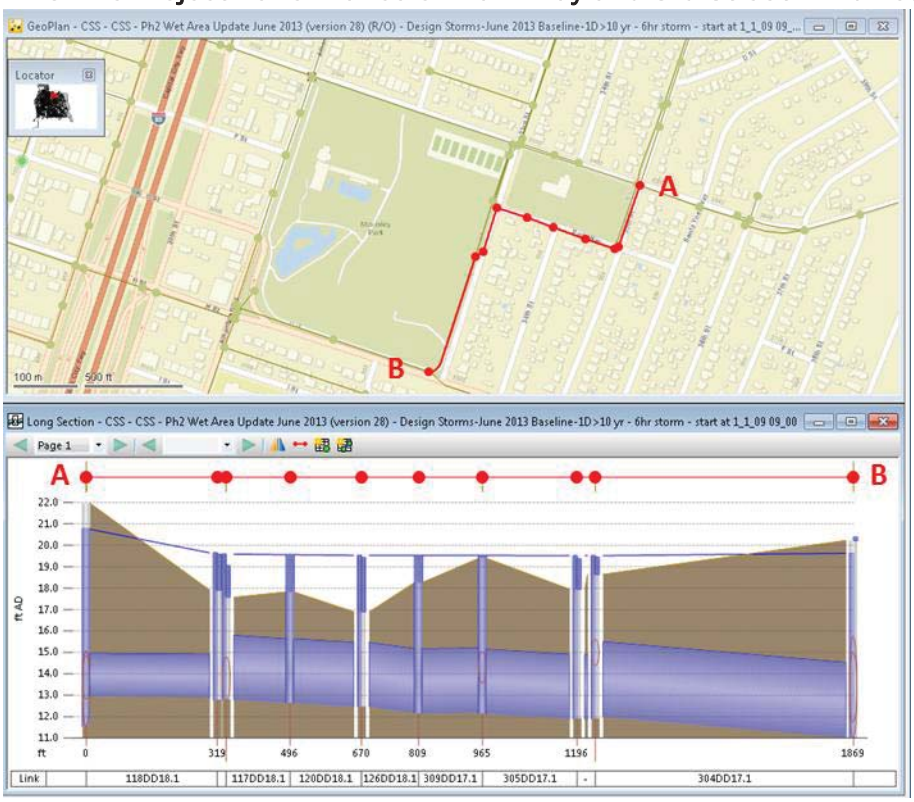


Figure 5-116: Post-Project Performance on Park Way and 3rd Street in 10-Year Storm

Feasibility/Constructability: DOU is currently considering a storage project in McKinley Park. A shallow storage facility is possible at this location to eliminate the flooding, thus reducing the cost of deeper trenching or excavation. The project appears feasible due to its many similarities to the currently under construction storage project at Oak Park. Figure 5-117 presents an aerial view of the site.

Figure 5-117: McKinley Park Storage Project Aerial View

5.19 Project 4-2 (Sutter Middle School Storage)

The H&H model indicates that Alhambra Boulevard in Wet Area 4 experiences flooding during the design storms. Figure 5-118 shows the baseline H&H model results for the area surrounding Sutter Middle School in a 10-year design storm.

Figure 5-118: Baseline Flooding near Sutter Middle School

Upsizing the pipe along Alhambra Boulevard would be expensive, as Alhambra Boulevard is a heavy traffic corridor and may also have utility conflicts. Parcel storage was explored as an alternative to upsizing pipes on Alhambra Boulevard, and Sutter Middle School provides an opportunity to detain flow and lower the HGL in the pipes along Alhambra Boulevard. Figure 5-119 presents the system components for this project.

Figure 5-119: Sutter Middle School Storage Project Components

The main component of the project would be a 4 mg storage facility installed underneath the Sutter Middle School parking lot and athletic field, which used to be a Cemetery. The storage facility would be similar to the one at McKinley Park: a set of large pipes laid out in parallel. A weir structure at Alhambra Boulevard and I Street would convey flow to the storage facility. The facility would be gravity drained via a 6-inch-diameter flap valve to Alhambra Boulevard and J Street.

Benefits: The project would significantly reduce the HGL along Alhambra Boulevard in the immediate area of the school and moderately reduces the HGL farther north on Alhambra Boulevard. Table 5-19 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-120 and Figure 5-121 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

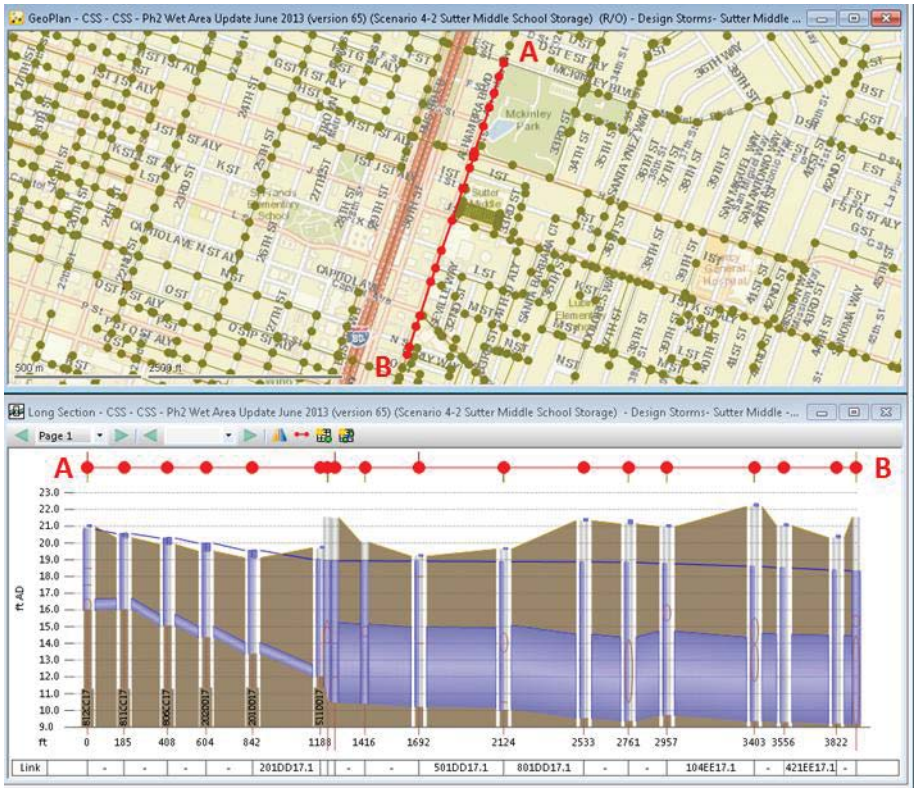
Table 5-19: Modeled Peak HGL for Node Near Sutter Middle School

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Alhambra Blvd. and G Street	19.1	**19.5**	19.1	19.1	**19.8**	**19.6**	**19.3**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-120: Pre-Project Performance on Alhambra Blvd. in 10-Year Storm

Figure 5-121: Post-Project Performance on Alhambra Blvd. in 10-Year Storm



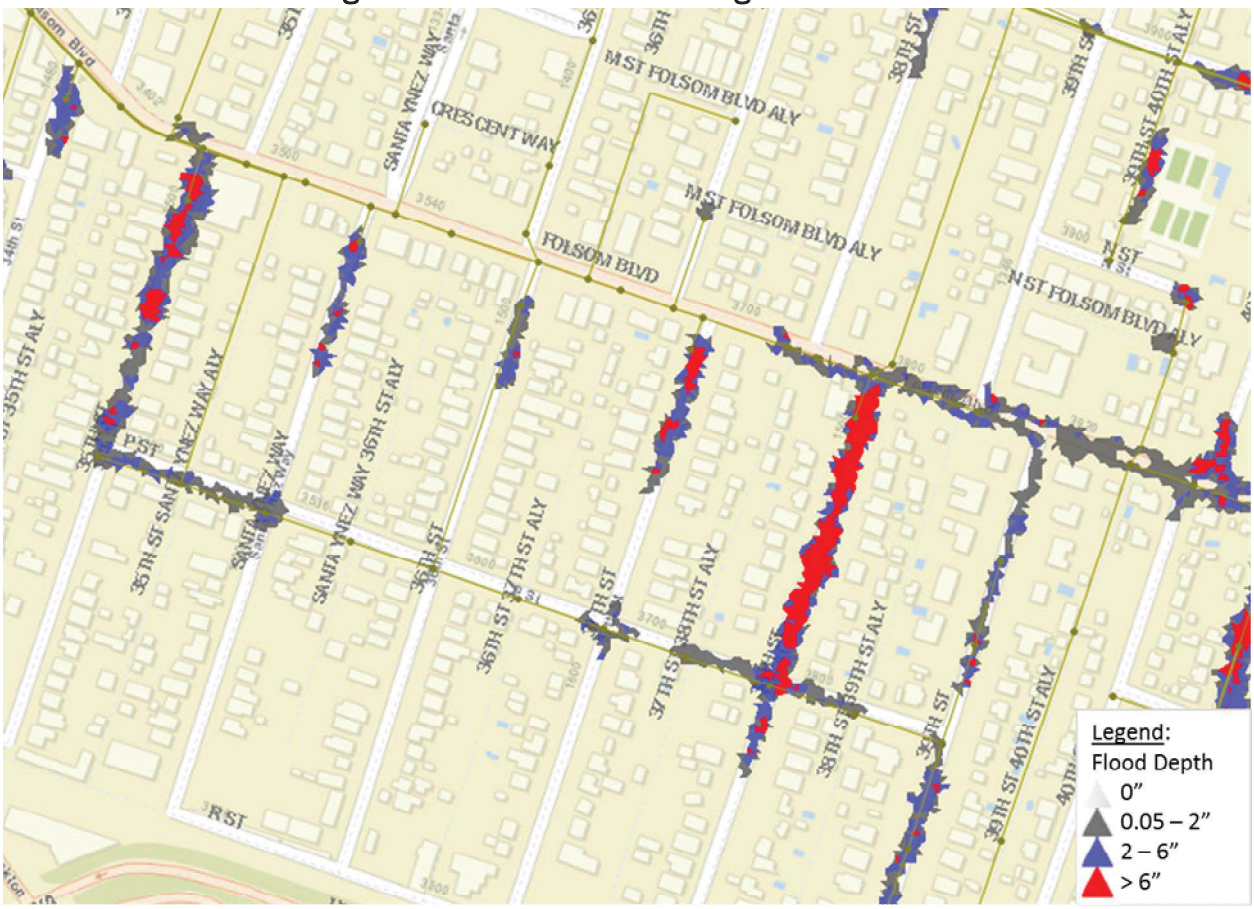
Feasibility/Constructability: DOU is currently considering a storage project at Sutter Middle School. A shallow storage facility is possible at this location to eliminate the flooding, thus reducing the cost of deeper trenching or excavation. The project appears feasible due to its many similarities to the currently under construction storage project at Oak Park. Cooperation from the school district and other local stakeholders would affect the feasibility of this project. Figure 5-122 presents an aerial view of the site.

Figure 5-122: Sutter Middle School Storage Project Aerial View

5.20 Project 4-3 (P Street Storage)

The H&H model indicates a flooding area at P Street and Folsom Boulevard from 40th Street to 35th Street. Limited downstream conveyance on Folsom Boulevard causes the P Street pipe to surcharge and flood onto 38th and 37th Streets. Figure 5-123 shows the baseline H&H model results for the area surrounding P Street in a 10-year design storm.

Figure 5-123: Baseline Flooding near P Street



Construction on downstream pipes on Folsom Boulevard is not desirable because the street is a busy traffic corridor. Parcel storage appears unavailable in the area; thus, in-line storage was the most feasible option. Figure 5-124 presents the system components for the project.

Figure 5-124: P Street Storage Project Components

Description: The P Street project would consist of a large storage pipe placed parallel to the pipe already installed on P Street. The pipe would be 1,300 LF of 96-inch-diameter pipe starting at 38th Street and ending at Santa Inez Way. The storage facility would draw flows from P Street with two weir structures placed on P Street at 38th Street and at 37th Street. The storage facility would be gravity-drained back to P Street via a 6-inch-diameter flap valve.

Benefits: The P Street Storage project would significantly lower the HGL along P Street and almost entirely eliminate flooding.

Table 5-20 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Table 5-20: Modeled Peak HGL for Area Near P Street

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
P Street and 38th Street	26.8	**27.3**	24.3	24.3	**27.5**	24.4	24.4

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-125 and Figure 5-126 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Figure 5-125: Pre-Project Performance on P Street in 10-Year Storm

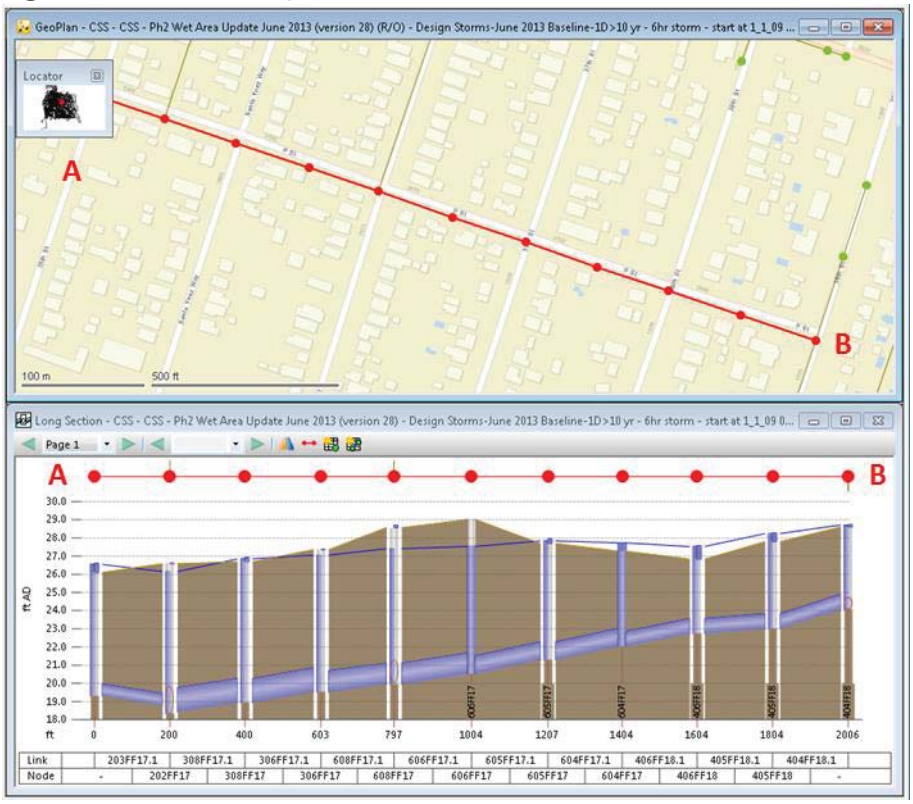
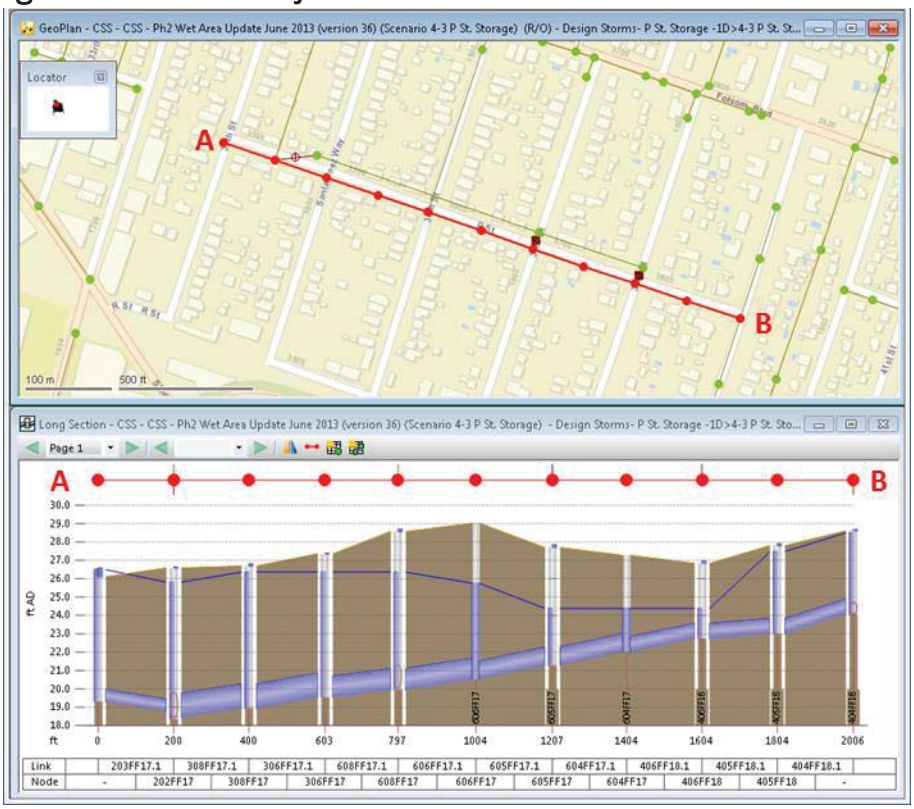


Figure 5-126: Post-Project Performance on P Street in 10-Year Storm



Feasibility/Constructability: In-line storage such as the P Street storage project is the most feasible option for solving the area's flooding problem. DOU proposed the P Street project, and the street itself appears to be a suitable location for an in-line storage project. Figure 5-127 gives an aerial view of the project area. Figure 5-128 provides a street view of P Street.

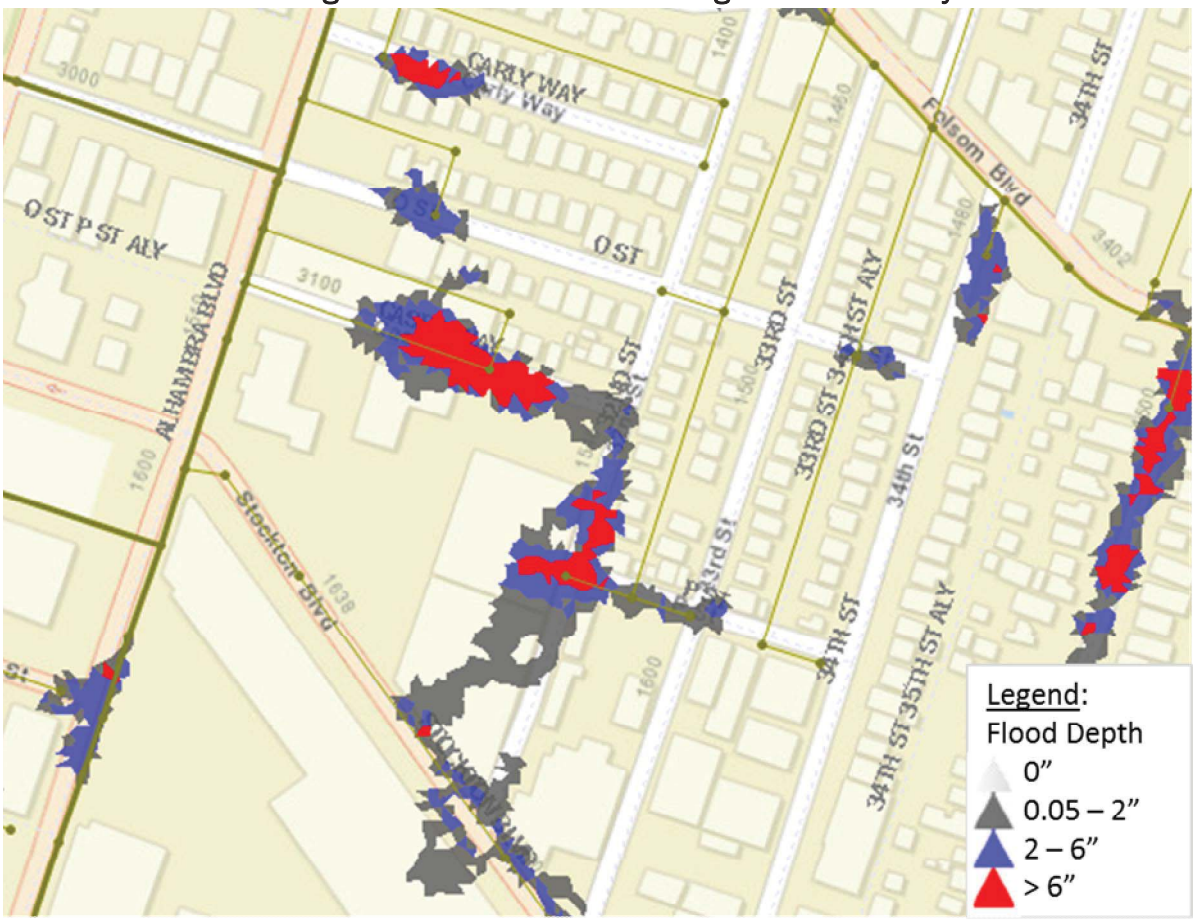
Figure 5-127: P Street Aerial View

Figure 5-128: P Street View (Looking East from Santa Ynez Way and P Street)

5.21 Project 4-4 (Casita Way)

Casita Way between Alhambra Boulevard and 32nd Avenue is known by DOU to flood during large storm events. DOU has received complaints of excess flow here following past storms, and is working to solve the problem. The H&H model results corroborate anecdotal reports of flooding on Casita Way. Figure 5-129 shows the baseline H&H model results for the area surrounding Casita Way in a 10-year design storm.

Figure 5-129: Baseline Flooding near Casita Way



Projects to alleviate flooding along Alhambra Boulevard (Project 4-1 and Project 4-2) would help lower the HGL on Alhambra Boulevard and Casita Way. An in-line storage project along Casita Way would reduce or eliminate flooding in this area completely when combined with upstream projects on Alhambra Boulevard. In the absence of increased conveyance and available parcels, an in-line storage facility was evaluated along Casita Way. Figure 5-130 presents the system components for this project.

Figure 5-130: Casita Way Project Components

Description: The Casita Way project is made up of the following components:

- Install 200 LF of 84-inch-diameter pipe on Casita Way
- Install 250 LF of 84-inch-diameter pipe on 32nd Street

The new in-line storage facility would collect flow from both the P Street and 32nd Street intersection and Casita Way. The in-line storage facility would drain via a 6-inch-diameter flap valve conveying flow from the Casita Way and 32nd Street intersection to Alhambra Boulevard.

Benefits: Peak HGL in this area would be significantly reduced, and flooding would be almost entirely eliminated in a 10–year storm. Table 5-20 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-131 and Figure 5-132 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-20: Modeled Peak HGL for Node Near Casita Way

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Casita Way between Alhambra Blvd. and 32nd Street	20.6	**21.0**	18.7	16.7	**21.2**	**20.8**	18.8
Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation. Model results are for individual project only using one-dimensional results.							

Figure 5-131: Pre-Project Performance on Casita Way in 10-Year Storm

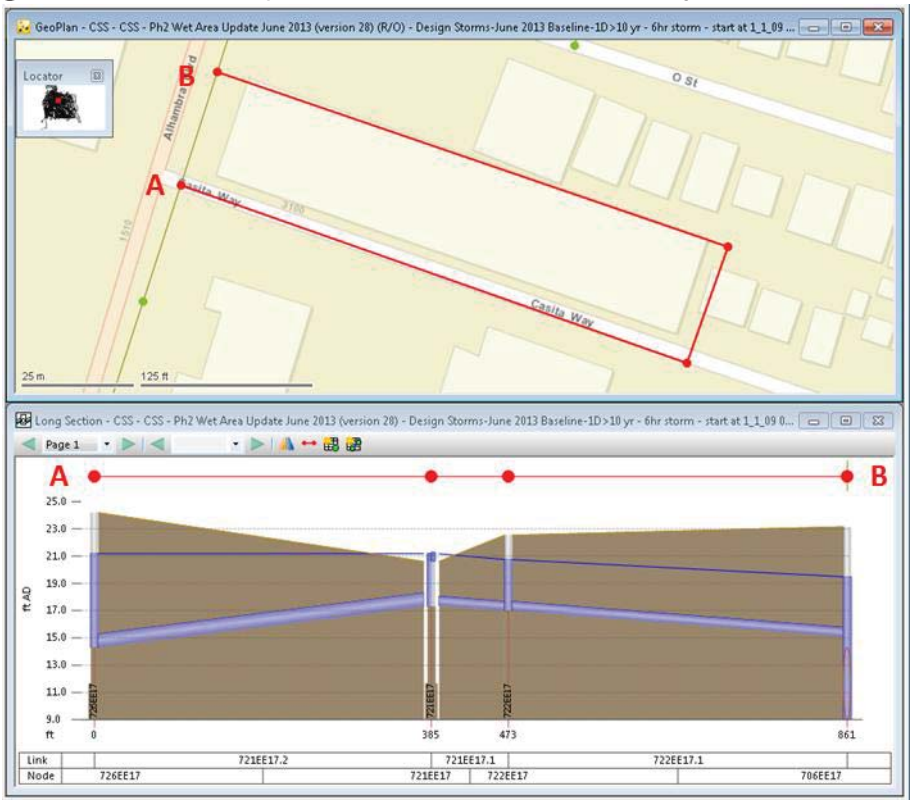
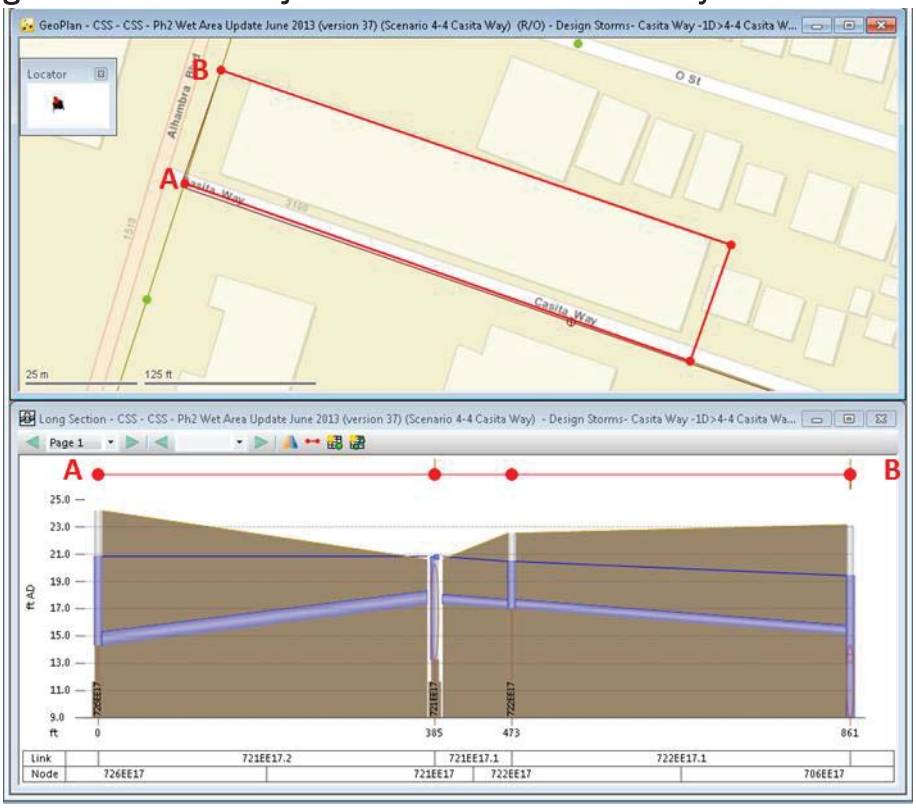


Figure 5-132: Post-Project Performance on Casita Way in 10-Year Storm



Feasibility/Constructability: The project would disturb very few street blocks and appears to be feasible. The project would address local complaints and work to appease neighbors. Figure 5-133 shows an aerial view of the area and Figure 5-134 presents a street view.

Figure 5-133: Casita Way Aerial View

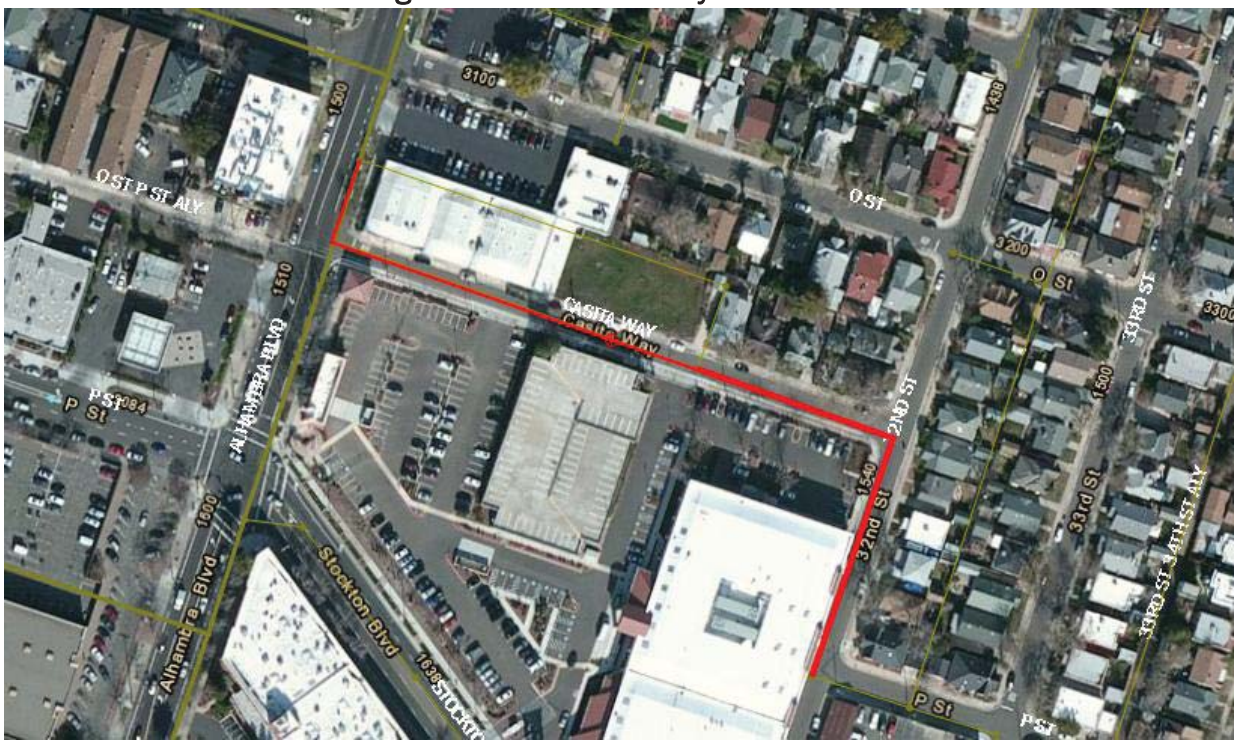
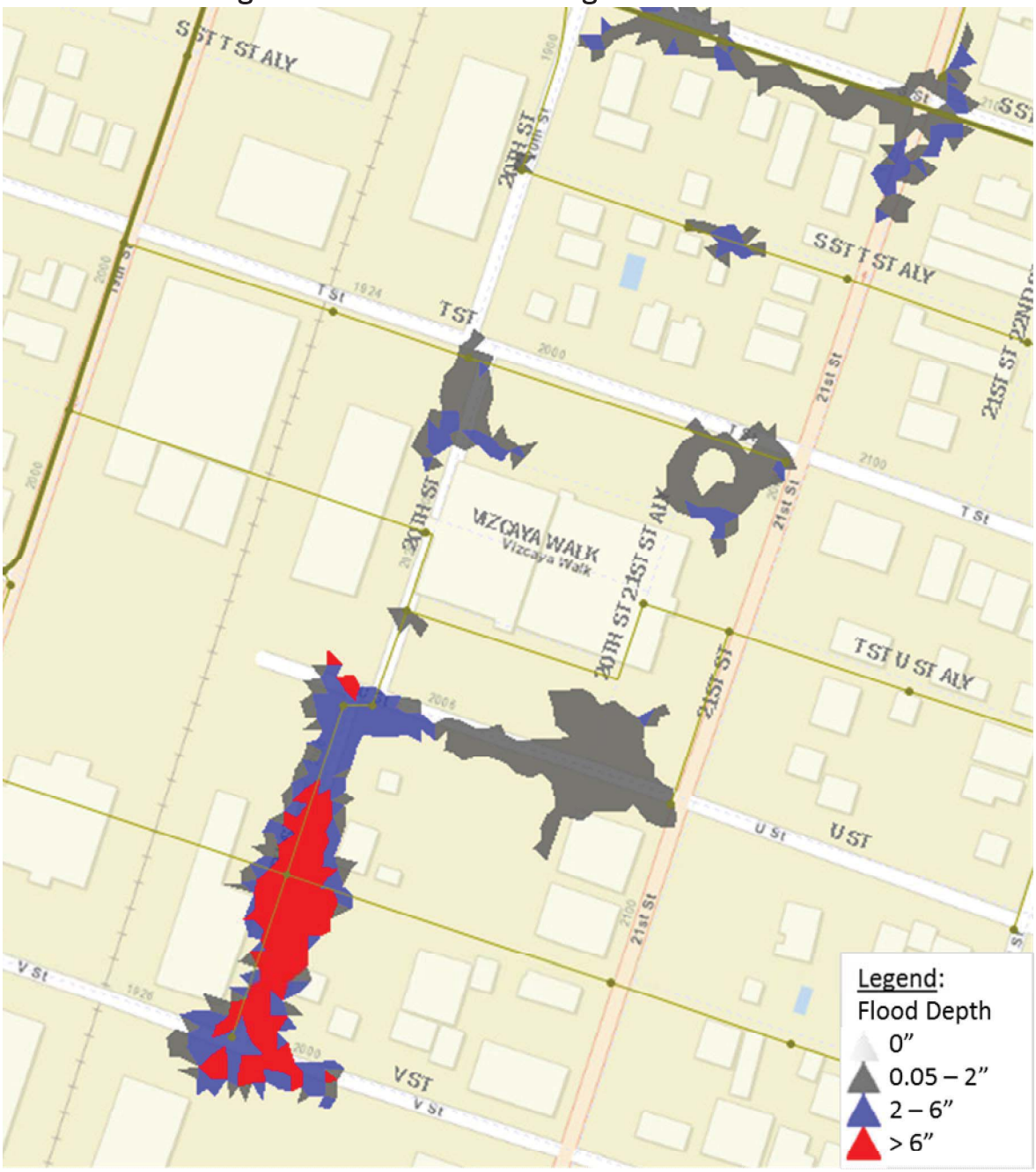


Figure 5-134: Casita Way Street View (Looking West from Casita and 32nd Street)

5.22 Project 5-1 (T and 20th Street)

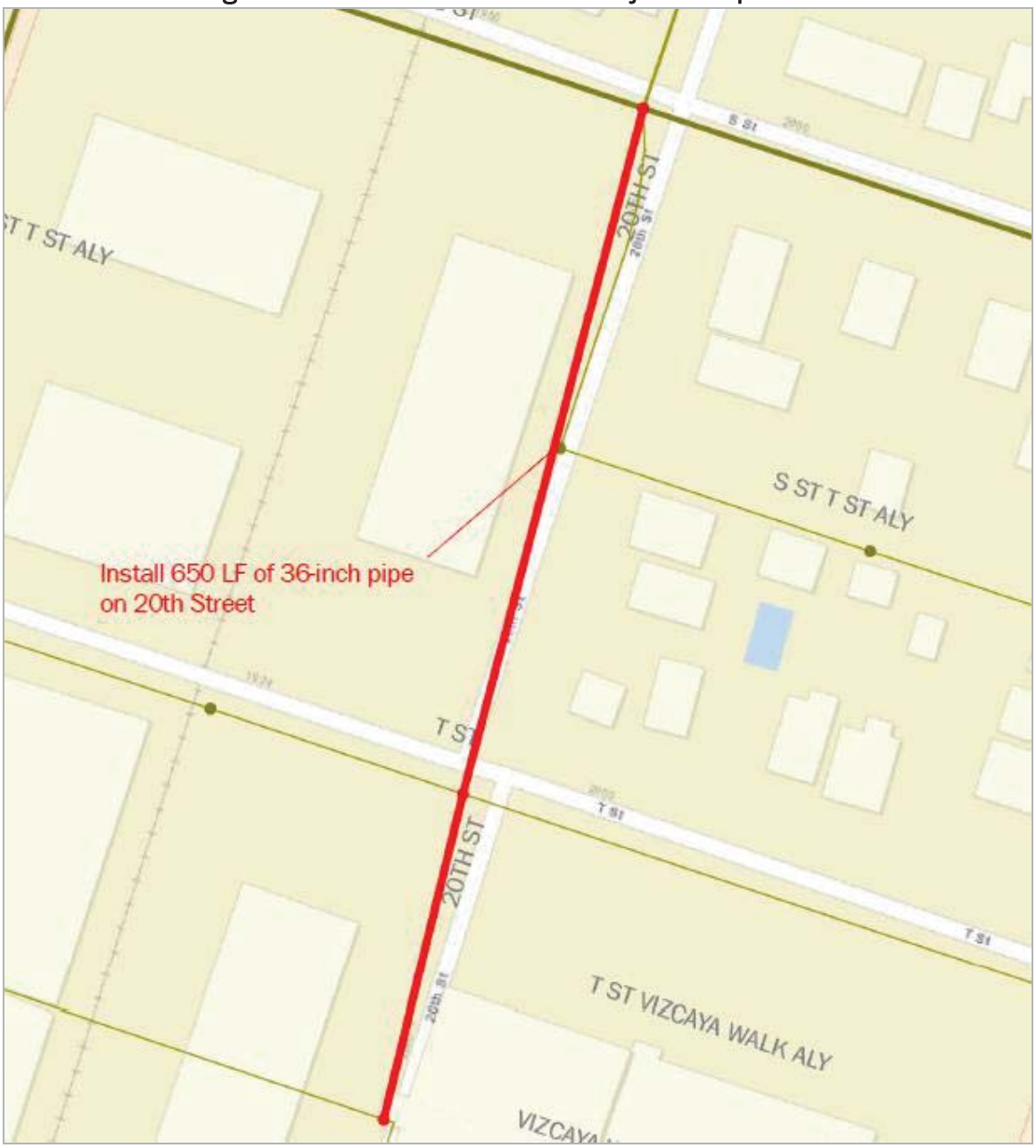
The intersection of T Street and 20th Street is a low-lying area, and the H&H model indicates that the area experiences flooding during design storms. Flooding is primarily due to limited downstream drainage paths and small-diameter pipes overwhelmed by wet-weather flows. Figure 5-135 shows the baseline H&H model results for the area surrounding the T Street and 20th Street intersection in a 10-year design storm.

Figure 5-135: Baseline Flooding near T and 20th Streets



After modeling various solutions, increasing conveyance by installing a new pipe appeared to be the best option. The installation would disturb a minimal number of blocks and would solve the issue of limited downstream capacity. Figure 5-136 presents project components.

Figure 5-136: T and 20th Street Project Components



Description: The project would consist of installing a new pipe to connect the small-diameter pipe at T/U Alley (also called T/Vizcaya Walk Alley) to an existing larger-diameter pipe on S Street. The new pipe would be 36 inches in diameter and approximately 650 LF in length.

Benefits: The project would have a significant effect on T Street during storm events. The new pipe would convey flow out of the area much faster, reduce surcharging, and eliminate

flooding on the street. Table 5-21 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-137 and Figure 5-138 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-21: Modeled Peak HGL for Node Near T and 20th Street

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
T/U Alley and 20th Street	17.3	**17.7**	15.5	13.2	**17.9**	16.5	15.3

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-137: Pre-Project Performance on T Street at 20th in 10-Year Storm

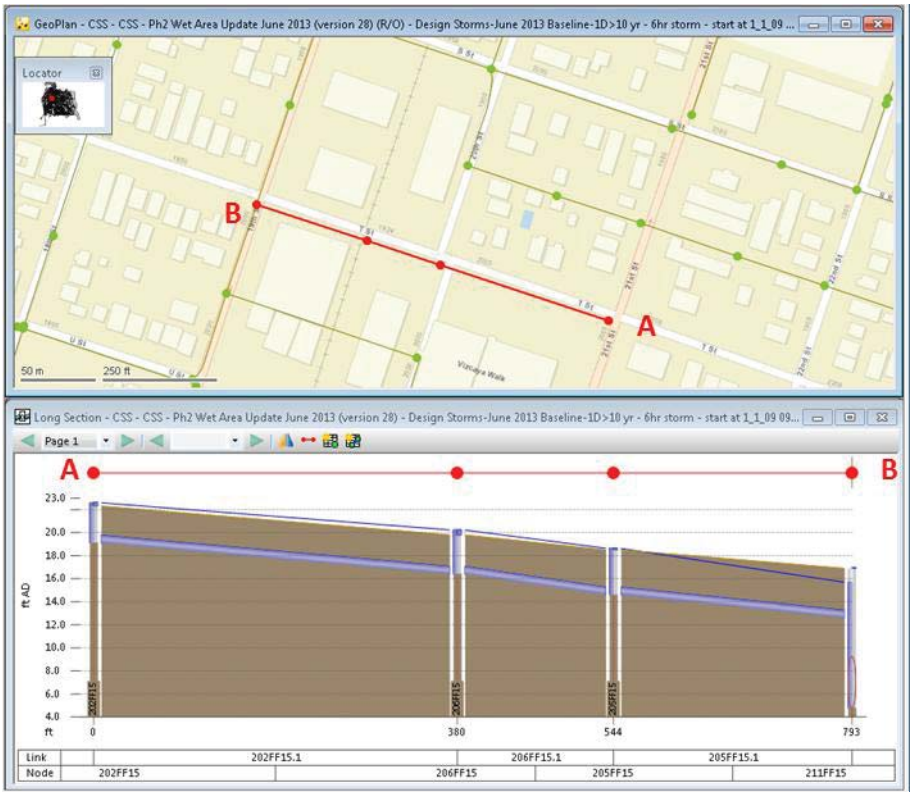
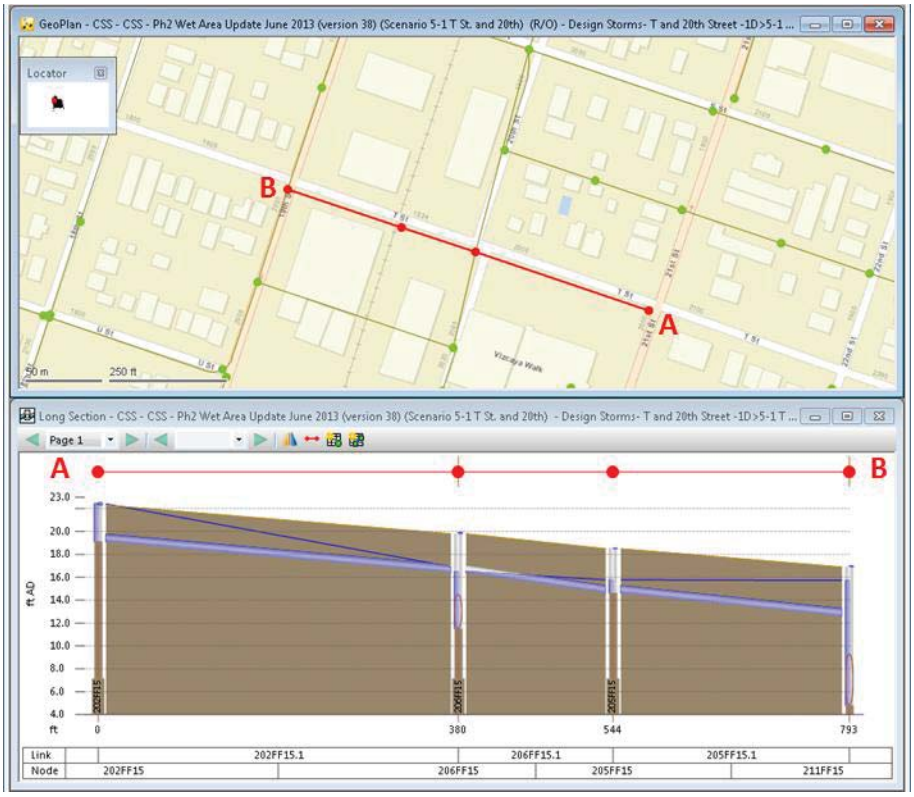


Figure 5-138: Post-Project Performance on T Street at 20th in 10-Year Storm



Feasibility/Constructability: DOU has recently done work in this area in coordination with a development project on 20th Street. As per DOU, the project may be difficult due to the narrow width of the alley south of T Street. Figure 5-139 presents an aerial view of the project area. Figure 5-140 presents a street view of the project area.

Additionally, due to a failed pipe on T Street under the rail tracks, DOU has awarded a contract to construct a 12-inch-diameter pipe on 20th Street from T Street to S Street. DOU is currently negotiating a change order to upsize the pipe to a 36-inch-diameter one.

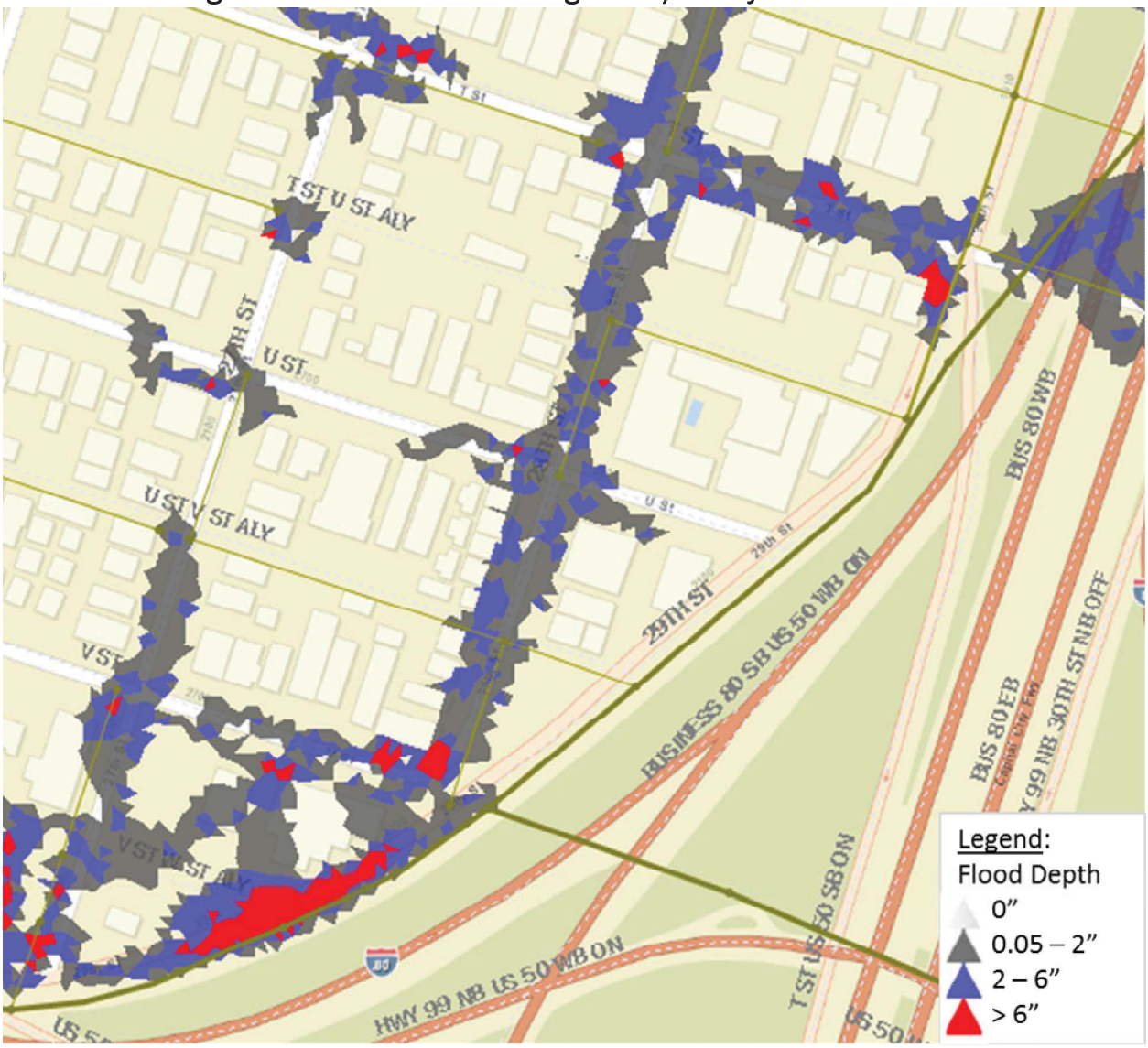
Figure 5-139: T and 20th Street Aerial View

Figure 5-140: 20th Street View (Looking North from T and 20th Street)

5.23 Project 5-2 (T/U Alley and 28th Street)

The H&H model indicates flooding at the intersection of T/U Alley and 28th Street. The intersection's ground level is lower than the surrounding area, the pipes in this area are small, and the pipes drain to surcharging pipes. Figure 5-141 shows the baseline H&H model results for the area surrounding the intersection of T/U Alley and 28th Street in a 10-year design storm.

Figure 5-141: Baseline Flooding near T/U Alley and 28th Street



In addition to upsizing the existing pipes, there is an opportunity to connect to the existing system for improved drainage during storm events. This project proposes this connection, as well as upsizing on T/U Alley and 28th Street. Figure 5-142 presents the system components for this project.

Figure 5-142: T/U Alley and 28th Street Project Components

Description: The project would involve the following changes:

- Upsize 200 LF of 8-inch-diameter pipe to 24-inch-diameter on 28th Street
- Upsize 400 LF of 8-inch-diameter pipe to 24-inch-diameter on T/U Alley
- Install 100 LF of 36-inch-diameter pipe near 29th Street to connect to the existing system

Benefits: The project would not only eliminate flooding at T/U Alley and 28th Street, but also downstream at T Street and 29th. The new pipe invert would be placed above dry-weather flows, and the connection would only be used during wet weather. Table 5-22 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-143 and Figure 5-144 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-22: Modeled Peak HGL for Node Near T/U Alley and 28th Street

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
U Street and 28th Street	18.0	**18.8**	16.8	16.7	**19.0**	18.0	17.0

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-143: Pre-Project Performance at T Street and 28th in 10-Year Storm

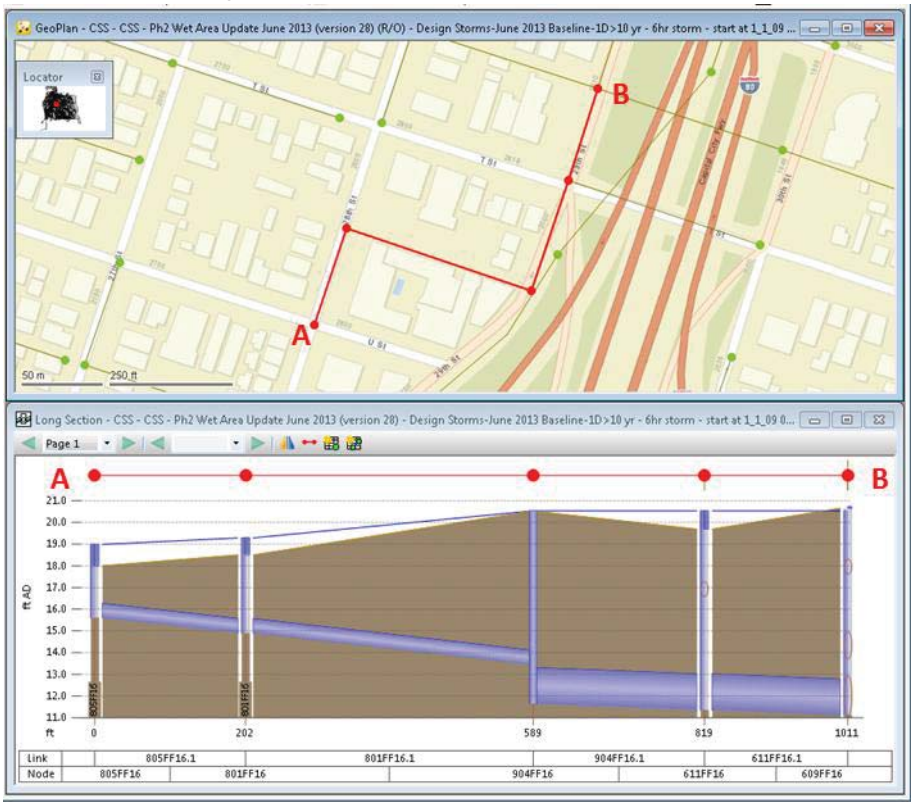
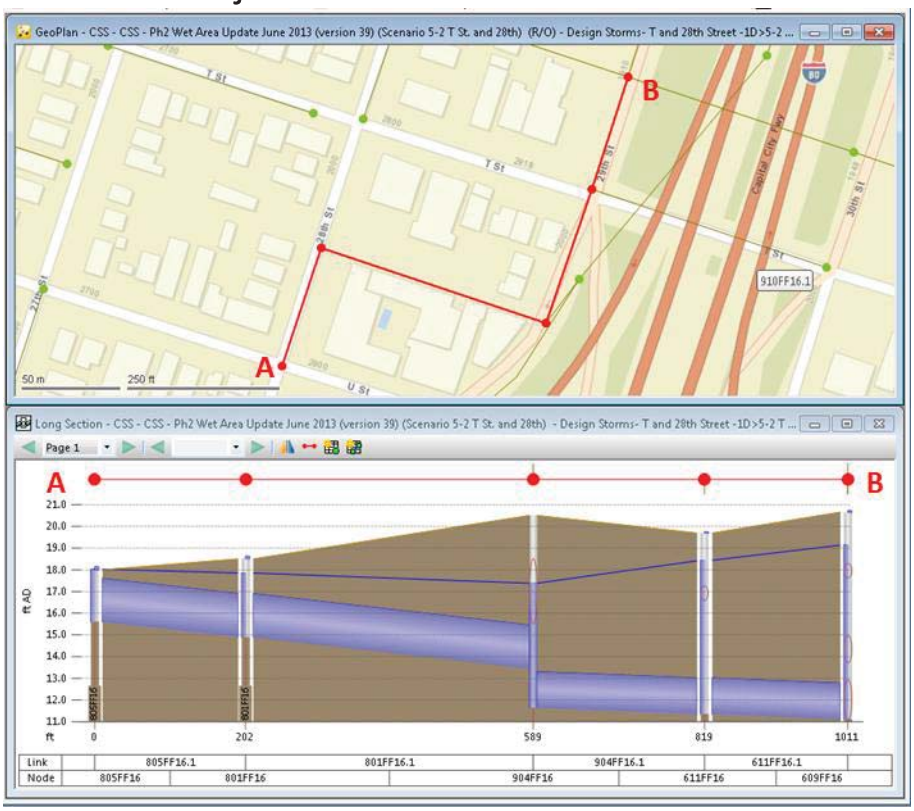


Figure 5-144: Post-Project Performance at T Street and 28th in 10-Year Storm



Feasibility/Constructability: The new connection pipe between the T/U Alley pipe and the freeway pipe appears to be feasible. The main concern for the project is the upsizing on T/U Alley due to the narrowness of the alley and the non-City-owned land along the pipe. The alley is a paved roadway rather than backyards, which is favorable. Figure 5-145 gives an aerial view of the project site. Figure 5-146 gives a street view of the project site.

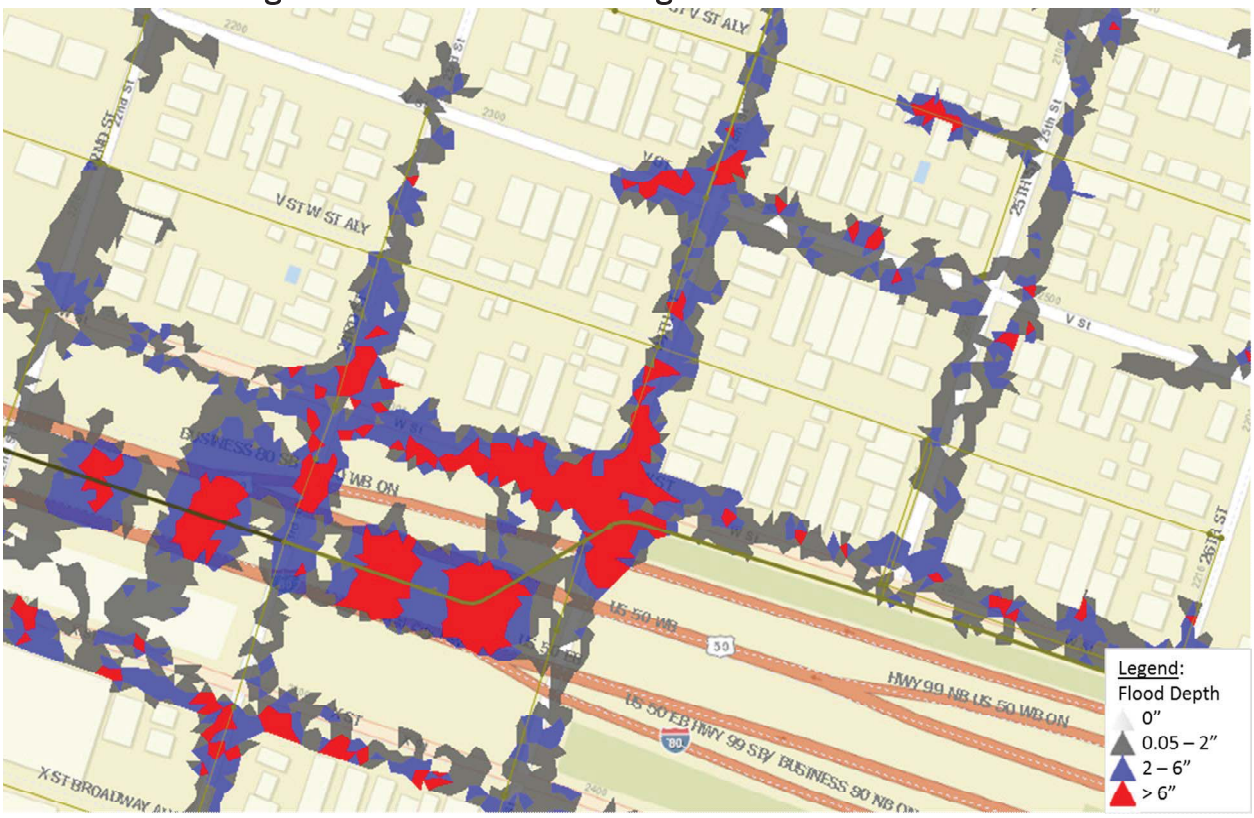
Figure 5-145: T/U Alley and 28th Street Aerial View

Figure 5-146: T/U Alley Street View (Looking West from T/U and 29th Street)

5.24 Project 5-3 (W and 25th Street)

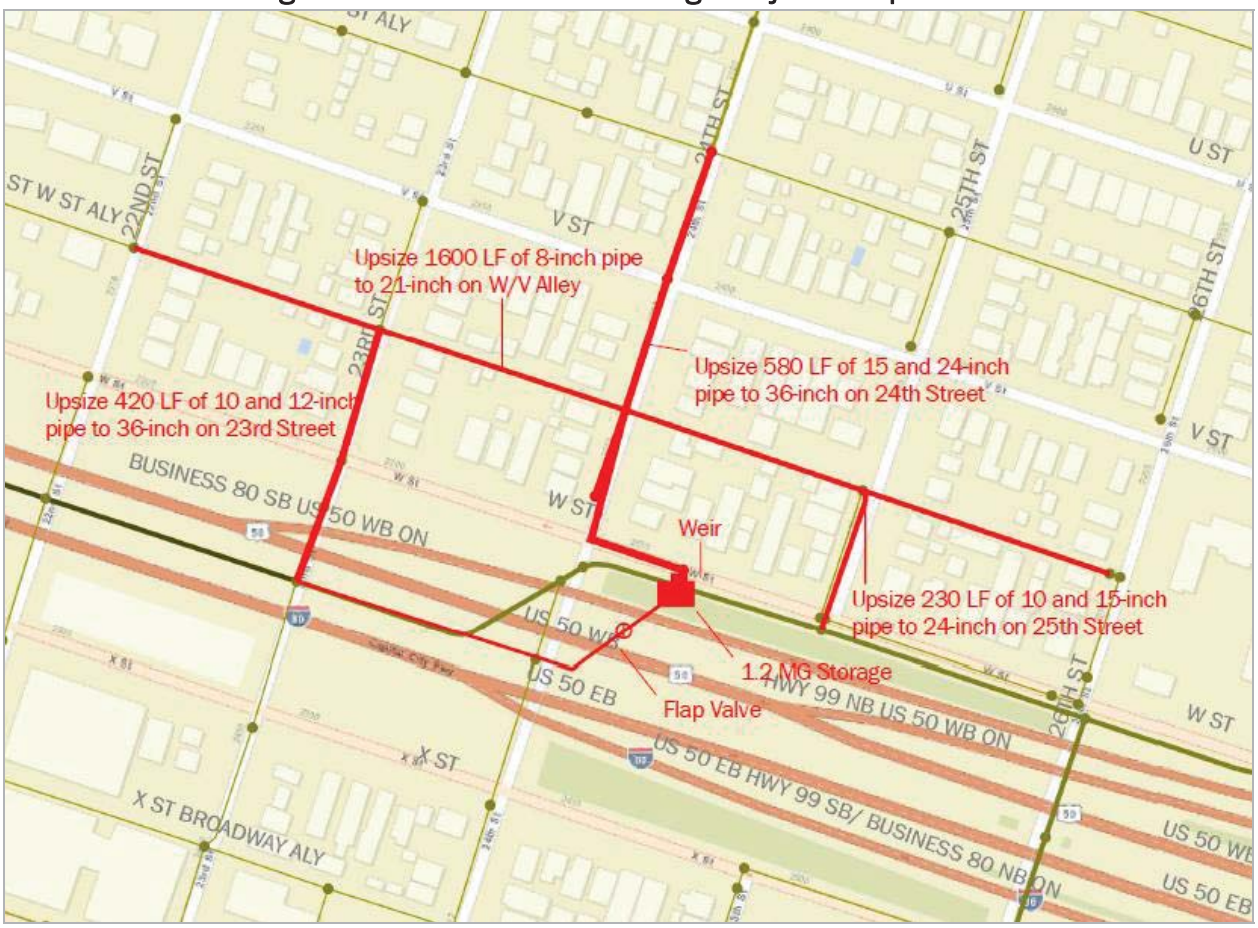
The area surrounding the intersection of W and 25th Streets is indicated as a flooding area in the H&H model. The area is part of a low-lying portion of the City, close to the major freeway corridors. Smaller-diameter pipes and limited downstream conveyance capacity cause flooding in the area. Figure 5-147 shows the baseline H&H model results for the area surrounding W and 25th Streets in a 10-year design storm.

Figure 5-147: Baseline Flooding near W and 25th Streets



The freeway corridor offers a potential location for detention. This project proposes using the wide medians and freeway rights-of-way to install a storage facility to detain flows until downstream pipes have capacity. Figure 5-148 presents the system components for this project.

Figure 5-148: W and 25th Storage Project Components



Description: This project would be made up of two components: increased conveyance and a storage facility. Increased conveyance consists of the following:

- Upsize 1,600 LF of 8-inch-diameter pipe to 21-inch-diameter on W/V Alley
- Upsize 420 LF of 10- and 12-inch-diameter pipe to 36-inch-diameter on 23rd Street
- Upsize 580 LF of 15- and 24-inch-diameter pipe to 36-inch-diameter on 24th Street
- Upsize 230 LF of 10- and 15-inch-diameter pipe to 24-inch-diameter on 25th Street

The storage facility would have a volume of 1.2 mg and be located under the freeway near W and 24th Street. The inlet weir structure for the storage facility would collect flows from the W and 24th Street intersection and bring it to the storage facility.

Benefits: The project would significantly reduce flooding in the area, and lower the HGL below ground surface in the area. Table 5-23 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-149 through Figure 5-154 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-23: Modeled Peak HGL for Node Near W and 25th Street

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
V Street and 24th Street	15.5	**16.1**	14.0	13.0	**16.6**	14.9	14.3

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-149: Pre-Project Performance on W/V Alley in 10-Year Storm

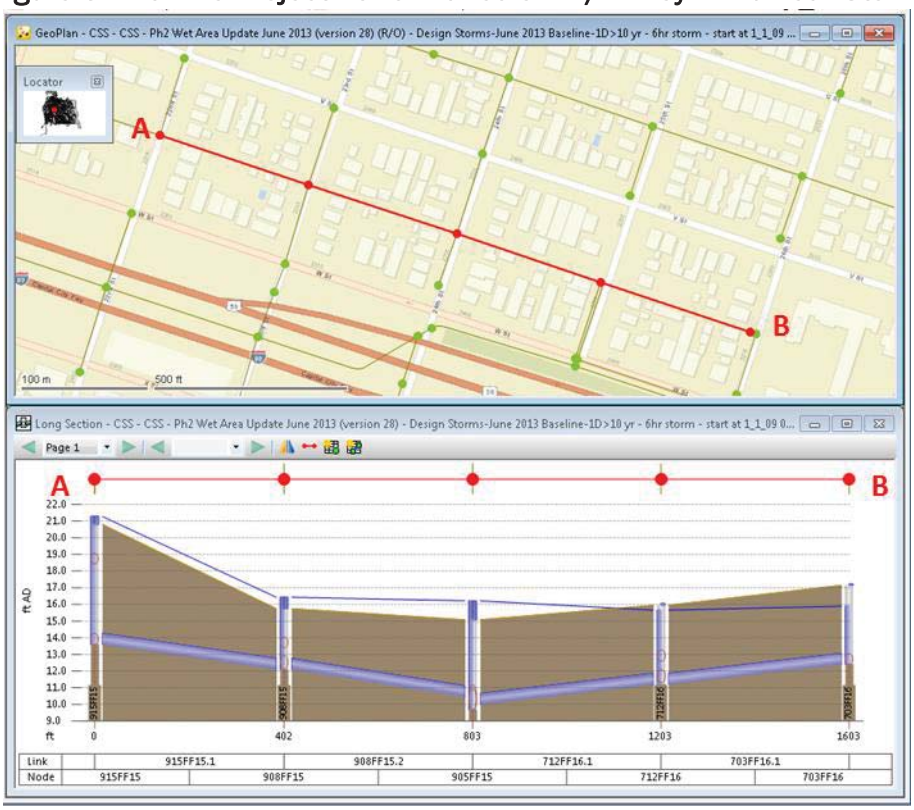


Figure 5-150: Post-Project Performance on W/V Alley in 10-Year Storm

Figure 5-151: Pre-Project Performance on 24th Street in 10-Year Storm

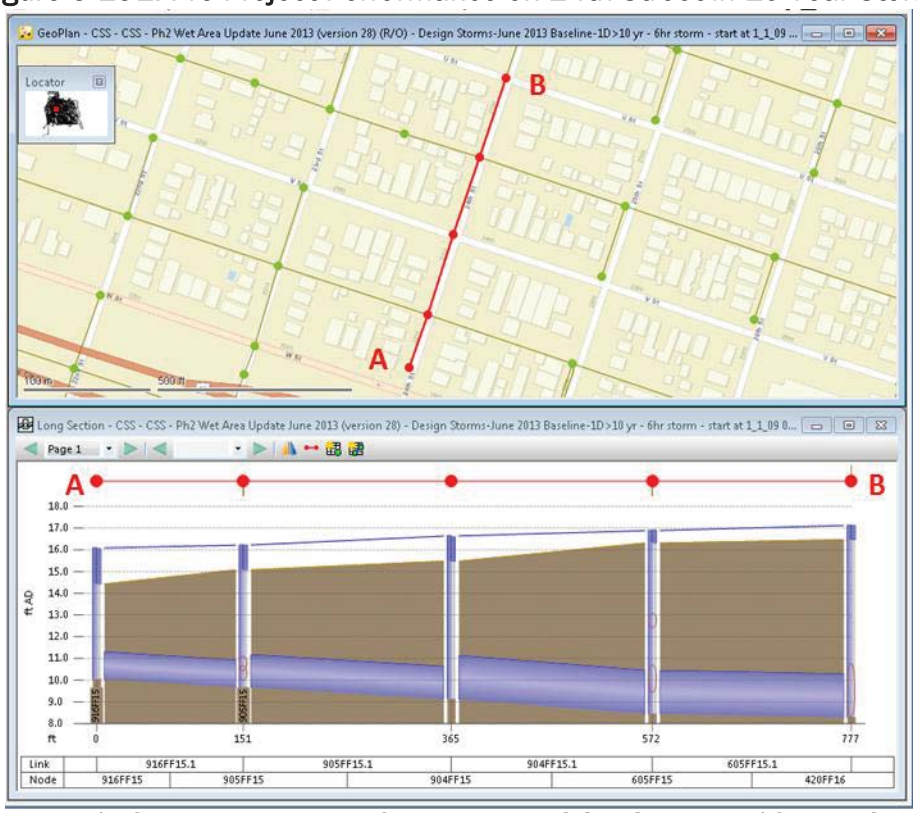


Figure 5-152: Post-Project Performance on 24th Street in 10-Year Storm

Figure 5-153: Pre-Project Performance at W and 25th Street in 10-Year Storm

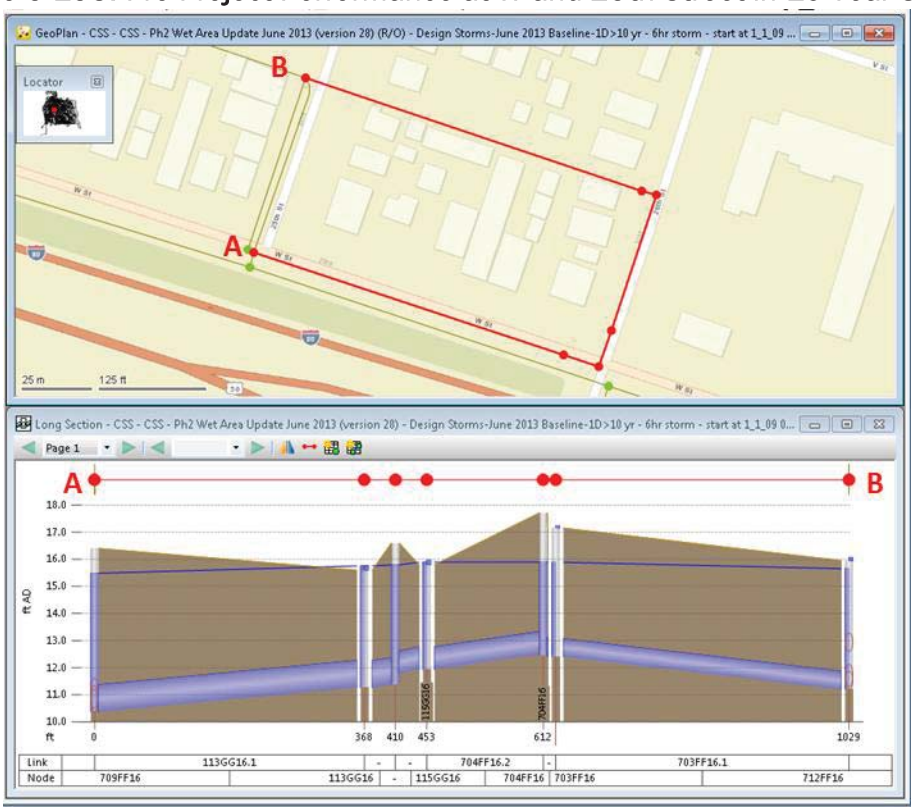
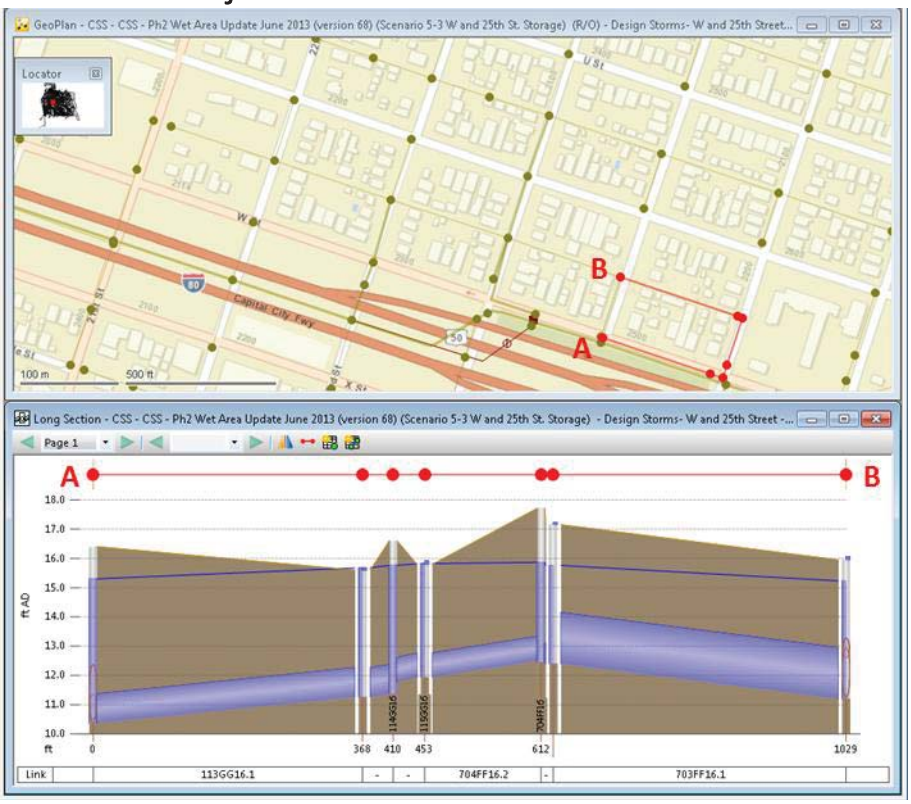


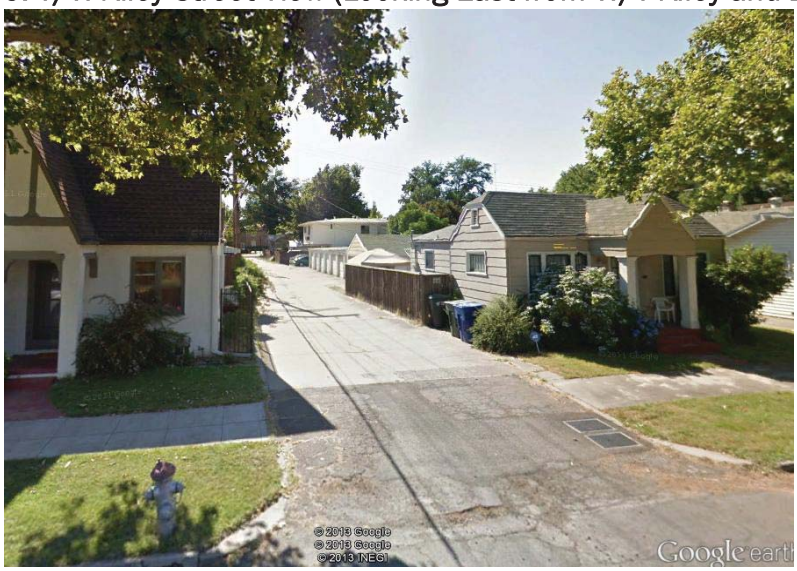
Figure 5-154: Post-Project Performance at W and 25th Street in 10-Year Storm



Feasibility/Constructability: All of the upsizing, except from upsizing along W/V Alley, would occur on wide streets where feasibility is higher. W/V Alley is a narrow alley, but is a paved road rather than private backyards, and could be feasible for upsizing. The feasibility of the storage facility would need to be investigated with input from the California Department of Transportation and any other interested stakeholders. Figure 5-155 presents an aerial view of the site. Figure 5-156 below presents a street view of the site.

Figure 5-155: W and 25th Storage Aerial View

Figure 5-156: V/W Alley Street View (Looking East from W/V Alley and 25th Street)



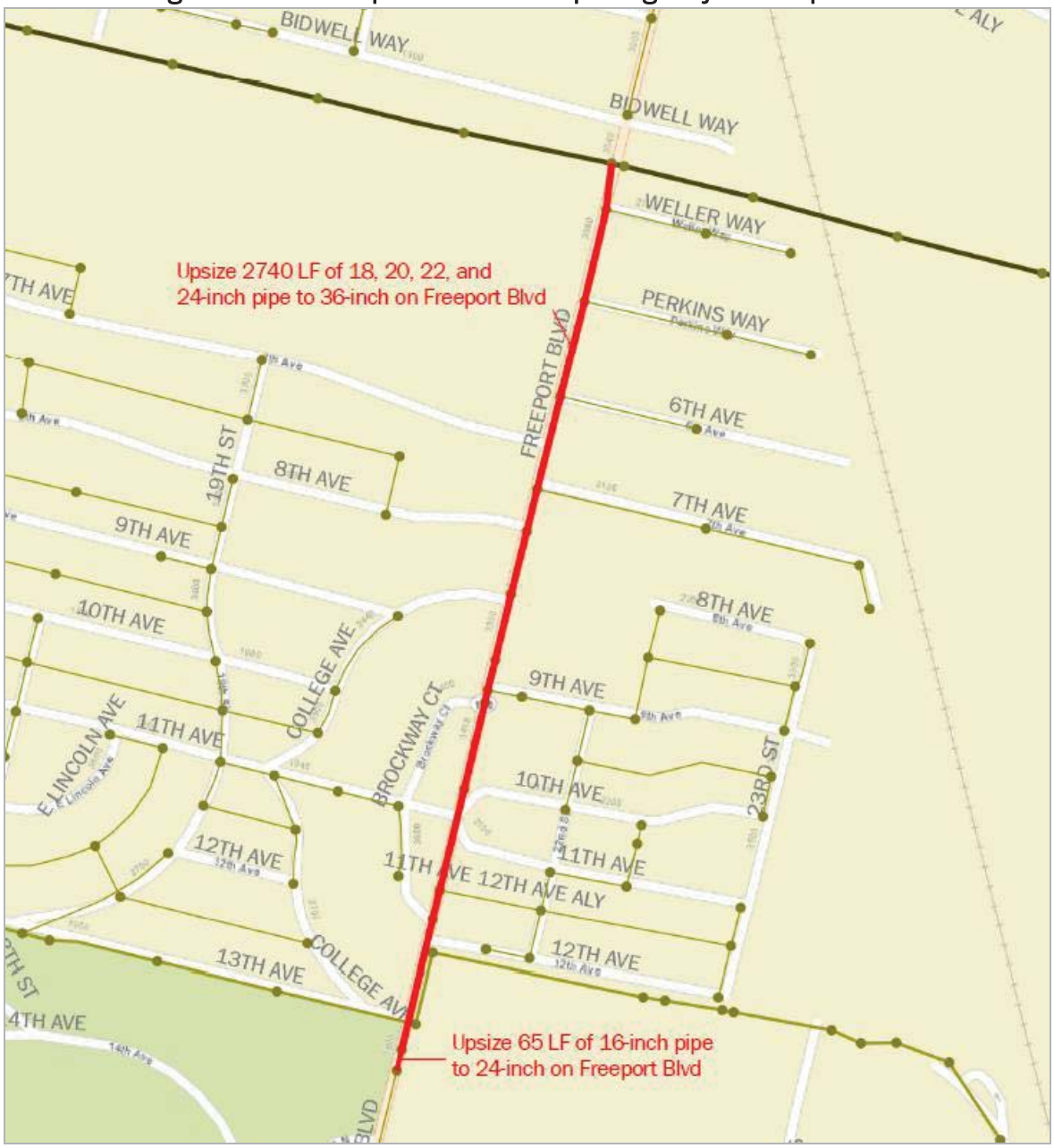
5.25 Project 6-1 (Freeport Boulevard Upsizing)

The pipe on Freeport Boulevard in Wet Area 6 is an important collector pipe and drainage pathway for the surrounding pipes. The H&H model shows that the pipe is overwhelmed and surcharges during design storms, causing flooding along Freeport Boulevard and along upstream pipes. Figure 5-157 shows the baseline H&H model results for the area surrounding Freeport Boulevard in a 10-year design storm.

Figure 5-157: Baseline Flooding near Freeport Boulevard

Increasing conveyance capacity in the pipe would not only relieve Freeport Boulevard, but would also work to reduce flooding upstream. Figure 5-158 presents the system components for the project.

Figure 5-158: Freeport Boulevard Upsizing Project Components



Description: This project would solely be a conveyance project. Upsizing would consist of the following:

- Upsize 65 LF of 16-inch-diameter pipe to 24-inch-diameter on Freeport Boulevard
- Upsize 2,740 LF of 18-, 20-, 22-, and 24-inch-diameter pipe to 36-inch-diameter on Freeport Boulevard

Benefits: The peak HGL along Freeport Boulevard would be lowered below the ground surface to eliminate flooding. Table 5-24 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-159 and Figure 5-160 presents the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-24: Modeled Peak HGL for Node Near Freeport Boulevard

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Freeport Blvd. below 13th Avenue	19.3	**19.4**	12.0	11.8	**19.7**	14.2	11.9

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-161 and Figure 5-162 also present the peak HGL profiles in the baseline and project scenarios for the pipe north of 12th Avenue near Freeport Boulevard for a 10-year storm. These present the impact that the upsized pipe on Freeport Boulevard would have on upstream pipes. The peak HGL would be lowered below the ground surface and flooding would be eliminated.

Figure 5-159: Pre-Project Performance on Freeport Blvd. in 10-Year Storm

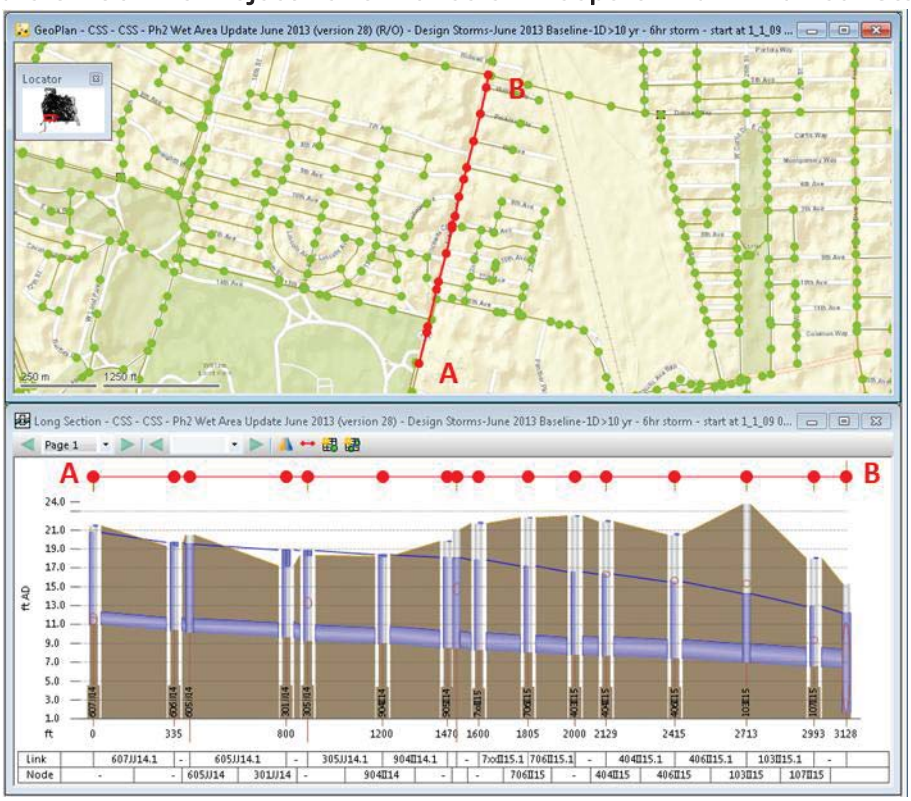


Figure 5-160: Post-Project Performance on Freeport Blvd. in 10-Year Storm

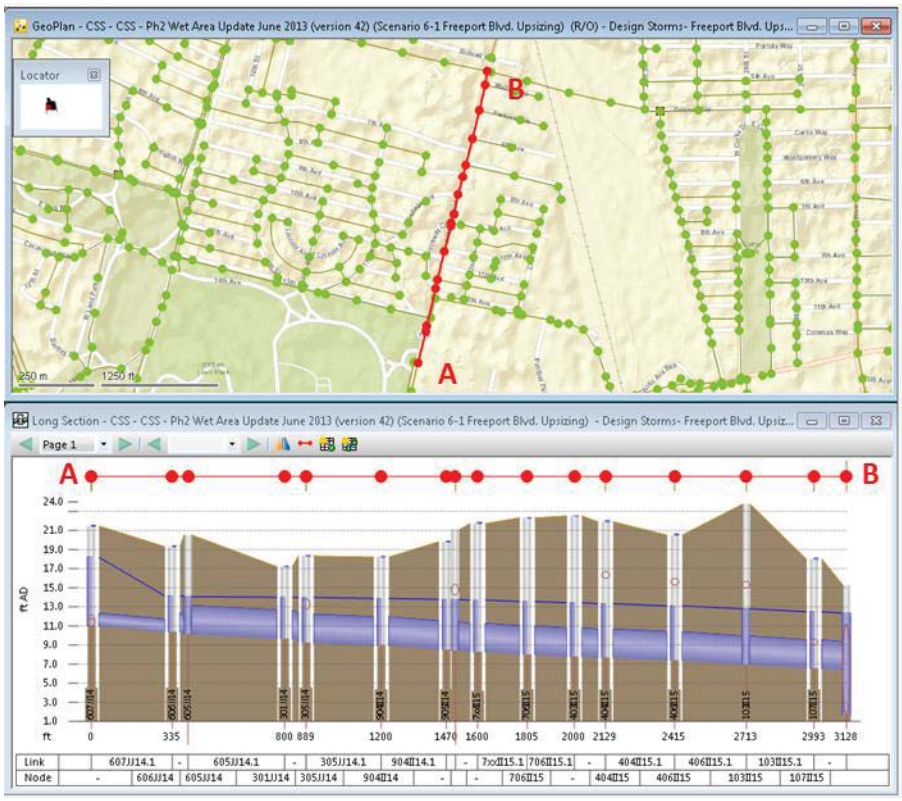


Figure 5-161: Pre-Project Performance at Freeport Blvd. and 12th Avenue in 10-Year Storm

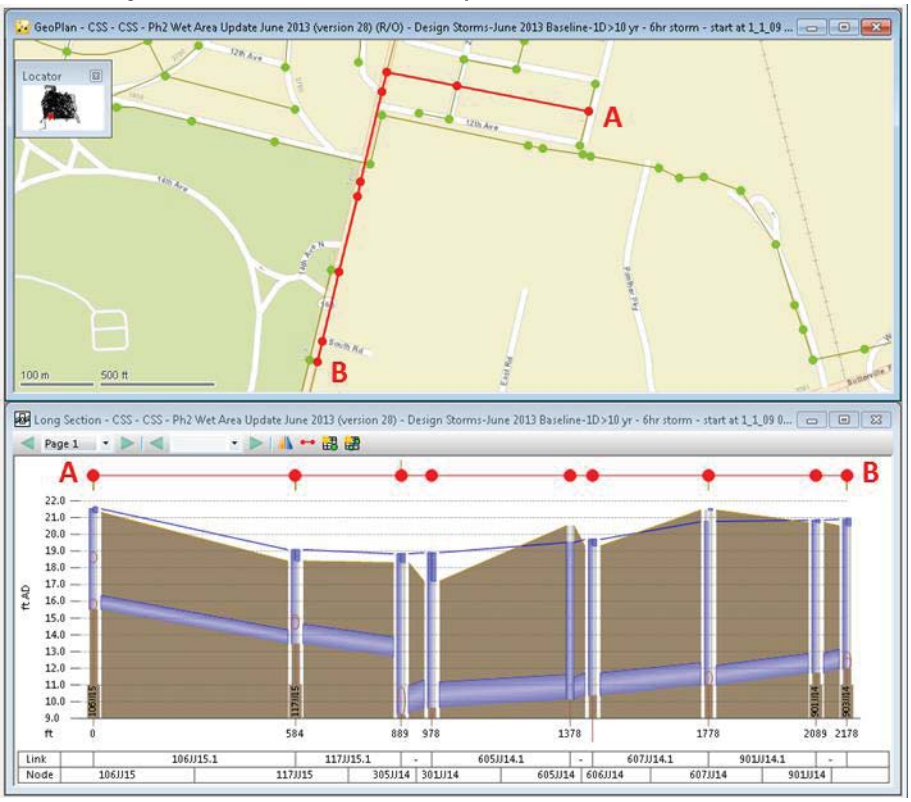
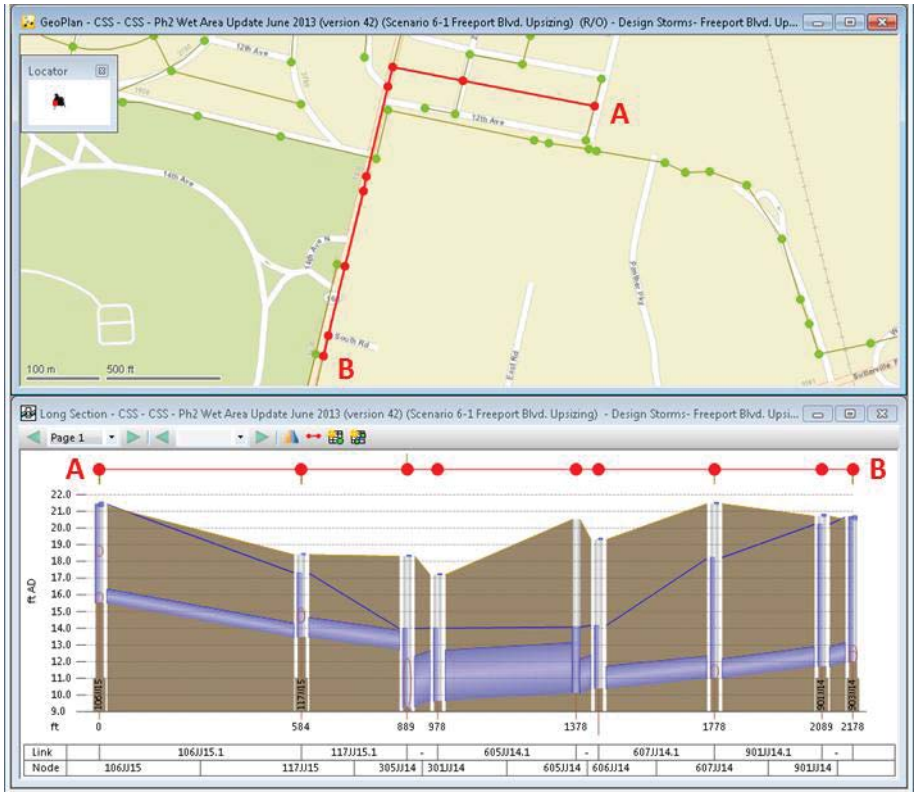


Figure 5-162: Post-Project Performance at Freeport Blvd. and 12th Avenue in 10-Year Storm



Feasibility/Constructability: Freeport Boulevard is a fairly wide roadway that appears to be suitable for a larger-diameter pipe. However, Figure 5-163 presents an aerial view of the area and Figure 5-164 presents a street view looking north on Freeport Boulevard at 12th Avenue. High traffic due to the nearby McClatchy High School and Sacramento City College may add difficulties to the construction of this project.

Figure 5-163: Freeport Boulevard Aerial View

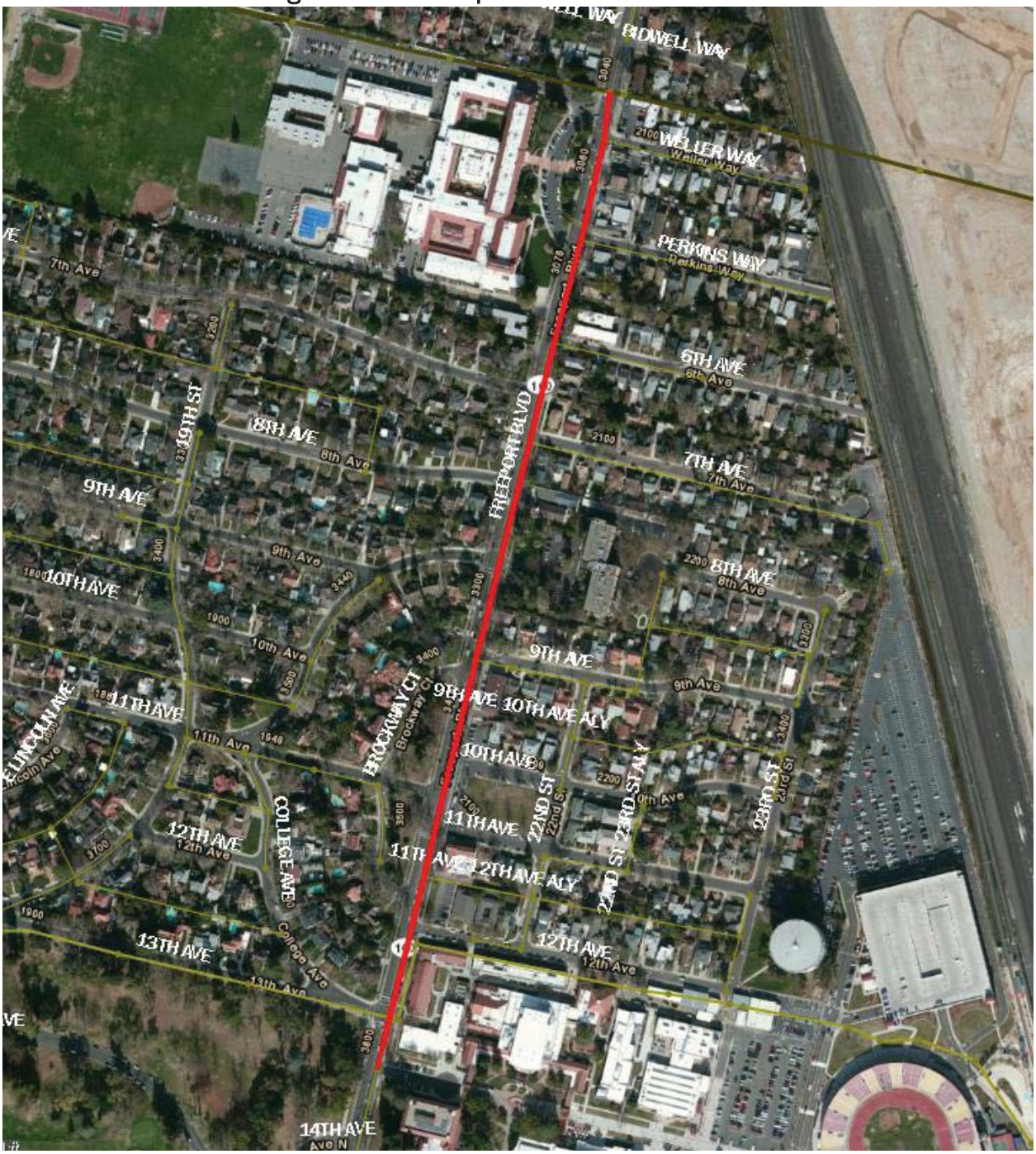
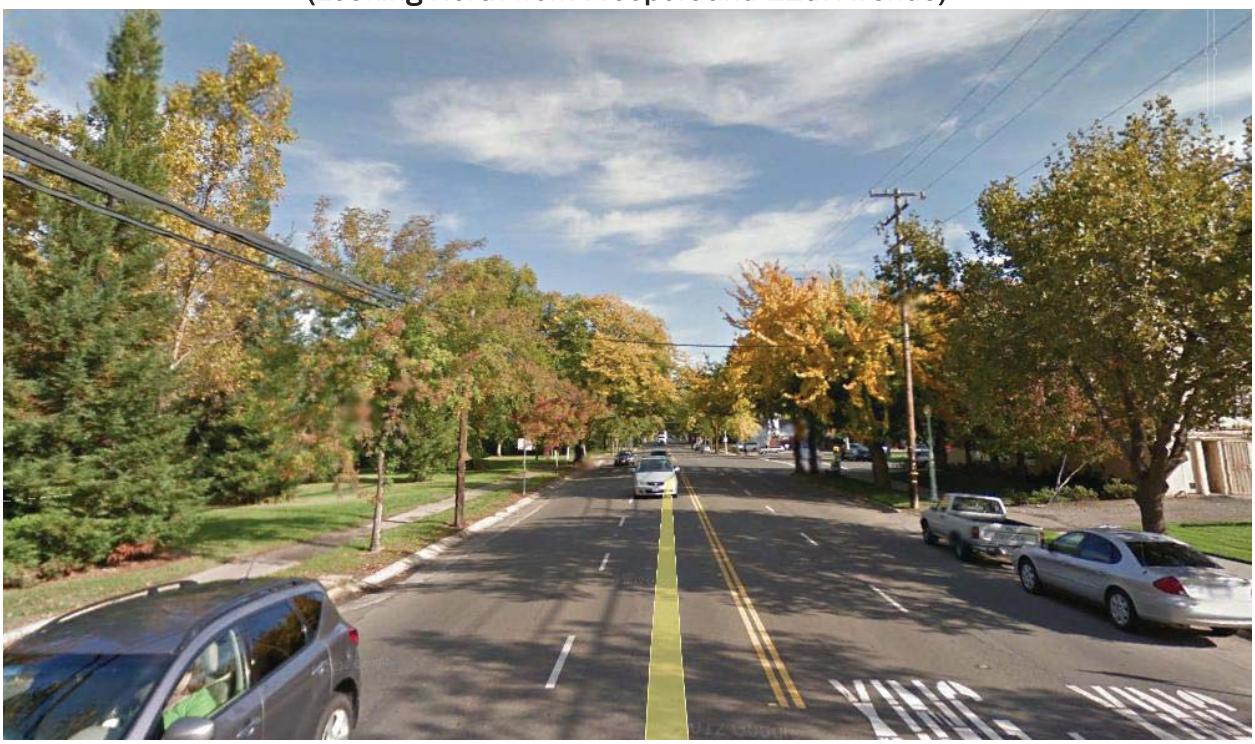


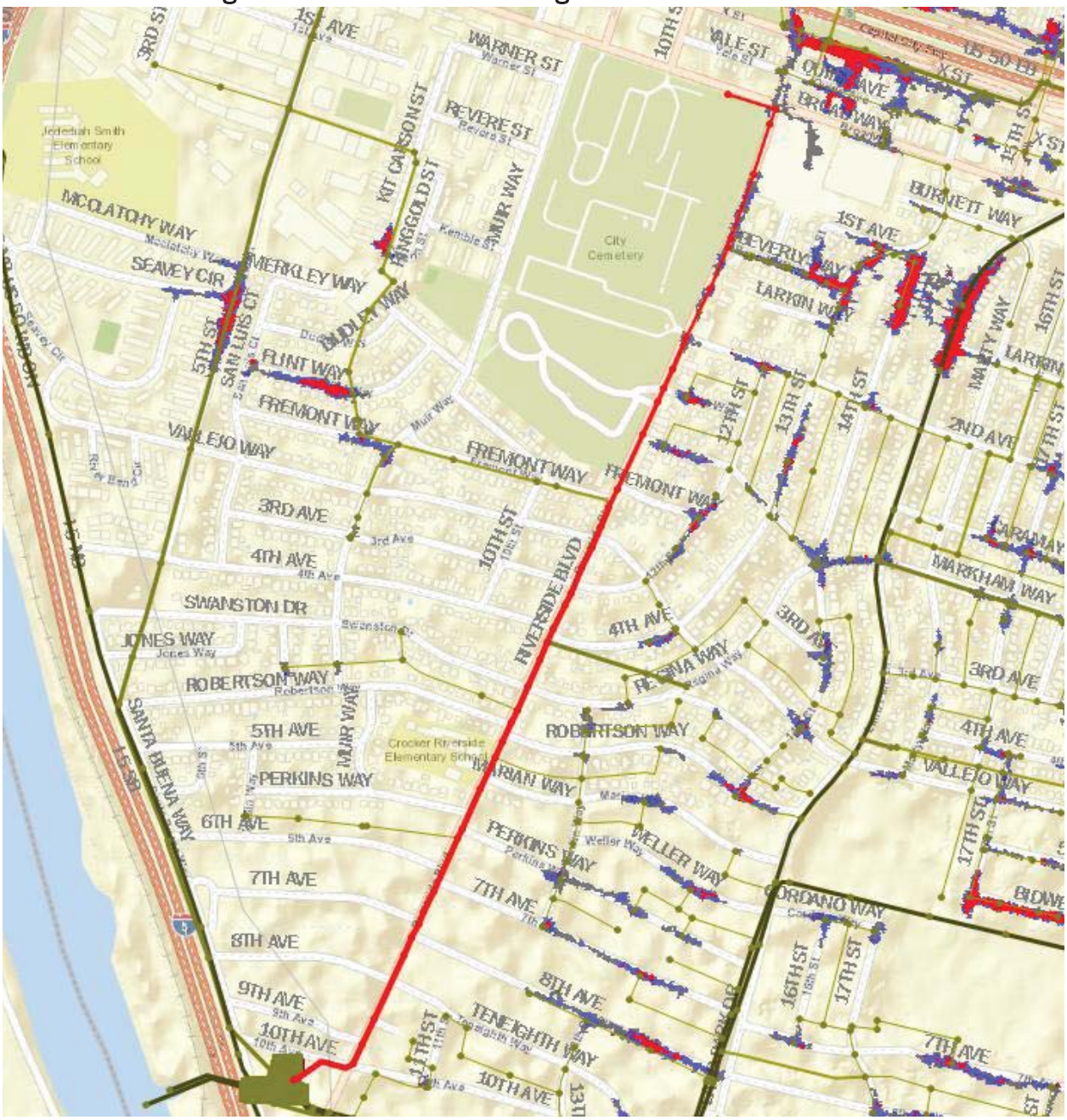
Figure 5-164: Freeport Boulevard Street View
(Looking North from Freeport and 12th Avenue)



5.26 Project 6-2 (Riverside Boulevard Upsizing)

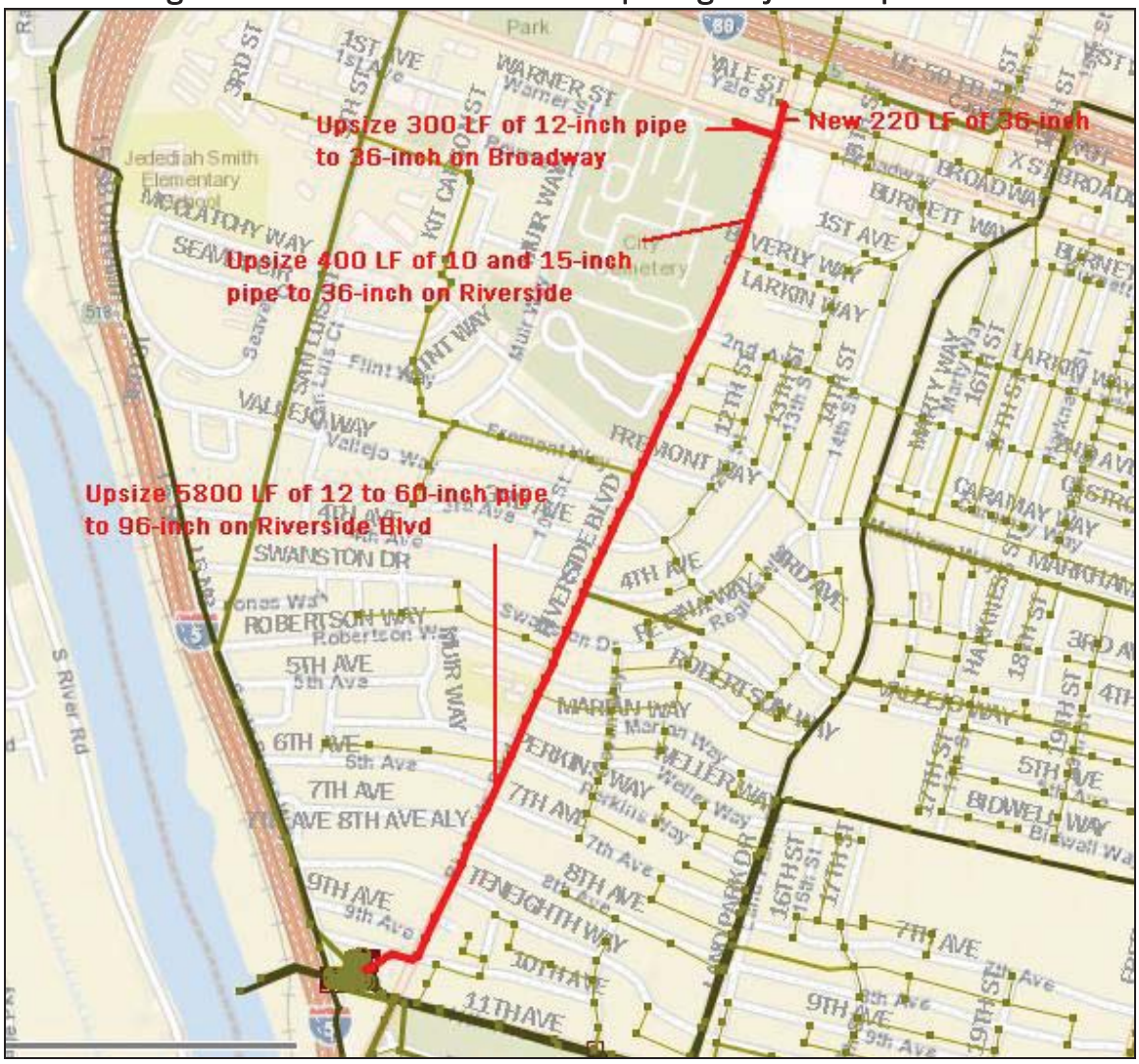
Riverside Boulevard was identified by DOU as containing an approximately 100-year-old pipe that is a good candidate for rehabilitation and upsizing. The H&H model indicated that upsizing the Riverside Boulevard pipe would relieve upstream flooding areas, such as Beverly Way, Quinn Ave. Figure 5-165 shows the baseline H&H model results for the area surrounding Riverside Boulevard in a 10-year design storm.

Figure 5-165: Baseline Flooding near Riverside Boulevard



Additionally, increased conveyance would also move wet-weather flows to Sump 2/2A faster, allowing it to pump more flows to the regional storage and treatment facilities, lowering the HGL of the entire system. Figure 5-166 presents the system components for this project.

Figure 5-166: Riverside Boulevard Upsizing Project Components



Description: The Riverside Boulevard project would involve the following three sections of pipe:

- Upsize 300 LF of 12-inch-diameter pipe to 36-inch-diameter on Broadway
- Upsize 400 LF of 10- and 15-inch-diameter pipe to 36-inch-diameter pipe on Riverside Boulevard
- Upsize 5,800 LF of 12- through 60-inch-diameter pipe to 96-inch-diameter pipe on Riverside Boulevard
- New 200 LF of 36-inch pipe to connect Quinn Ave to Riverside Blvd

Benefits: Table 5-25 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms. Figure 5-167 through Figure 5-170 present

the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-25: Modeled Peak HGL for Node Near Riverside Boulevard

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Beverly Way and 12th Street	10.8	**11.5**	9.2	9.0	**11.9**	**10.9**	**10.9**

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-167: Pre-Project Performance on Riverside Blvd. in 10-Year Storm

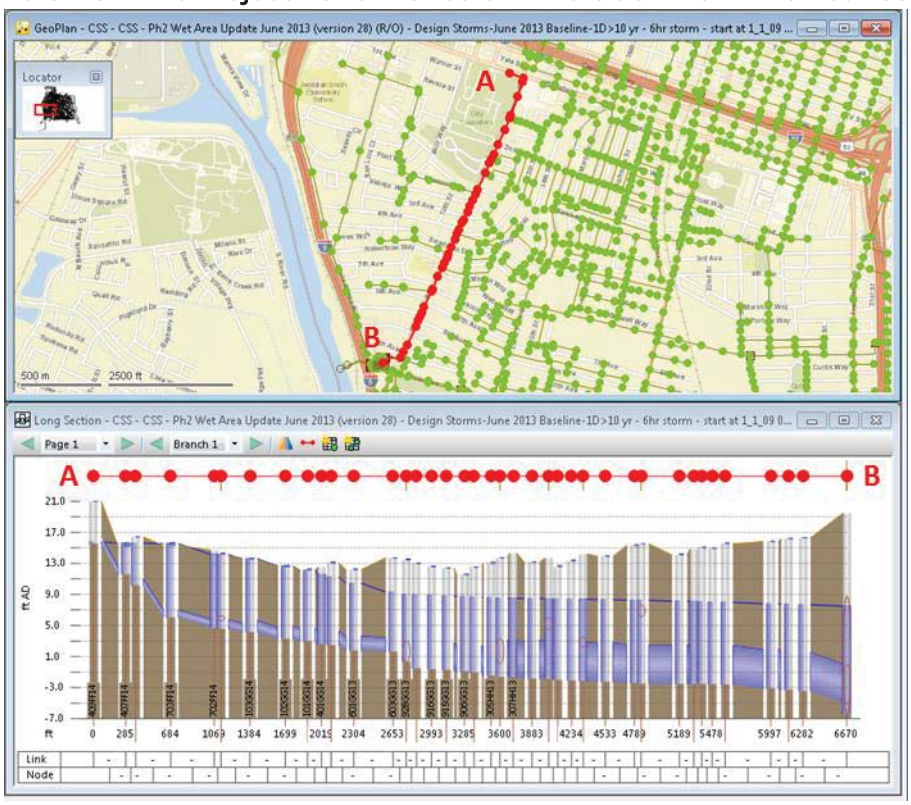


Figure 5-168: Post-Project Performance on Riverside Blvd. in 10-Year Storm

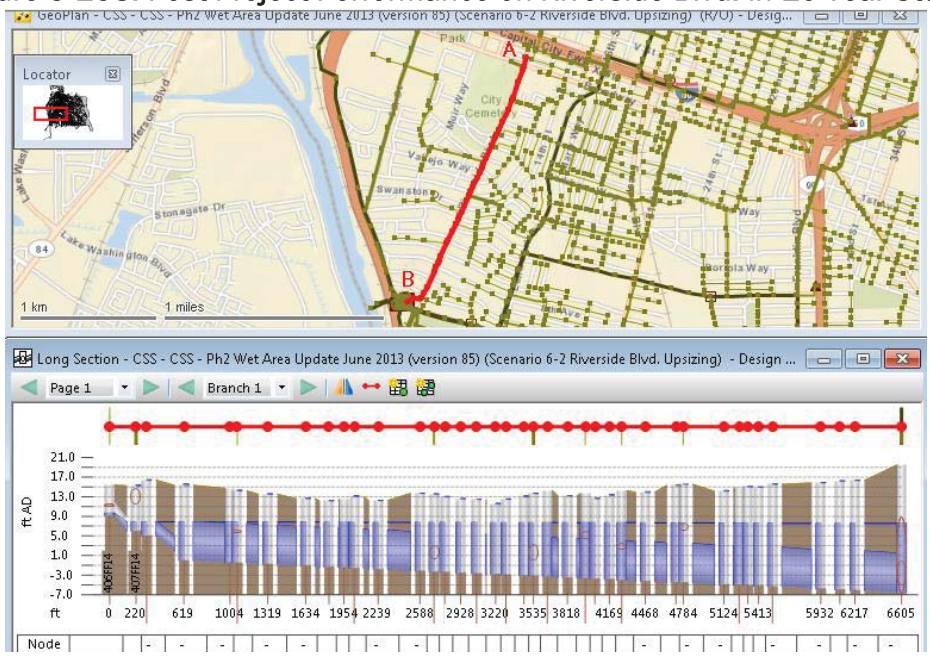


Figure 5-169: Pre-Project Performance on Beverly Way in 10-Year Storm

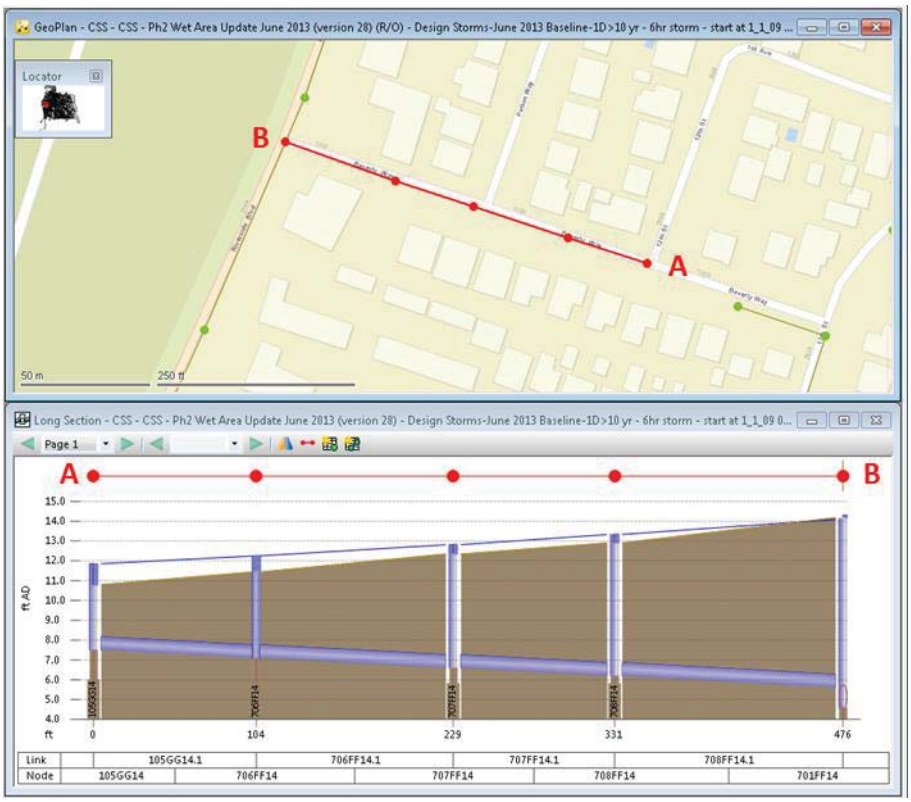
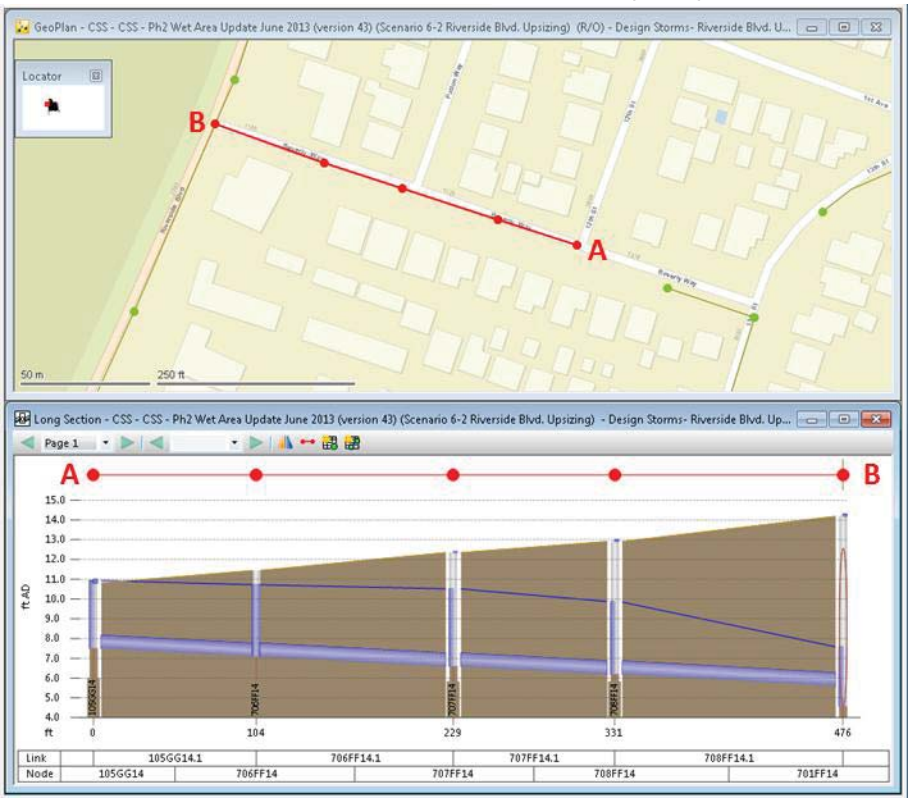


Figure 5-170: Post-Project Performance on Beverly Way in 10-Year Storm



Feasibility/Constructability: The project is considered important because the pipe on Riverside Boulevard is reaching the end of its useful life, and is a main artery of the collection system. The street is wide with a moderate level of traffic in general, with except of higher level of traffic near the elementary school. In general, it would be feasible to construct a larger pipe along the Riverside Boulevard corridor, with the exception of some construction difficulties near the elementary school. Figure 5-171 presents an aerial view of the project area and Figure 5-172 presents a street view of Riverside Boulevard.

Figure 5-171: Riverside Boulevard Aerial View

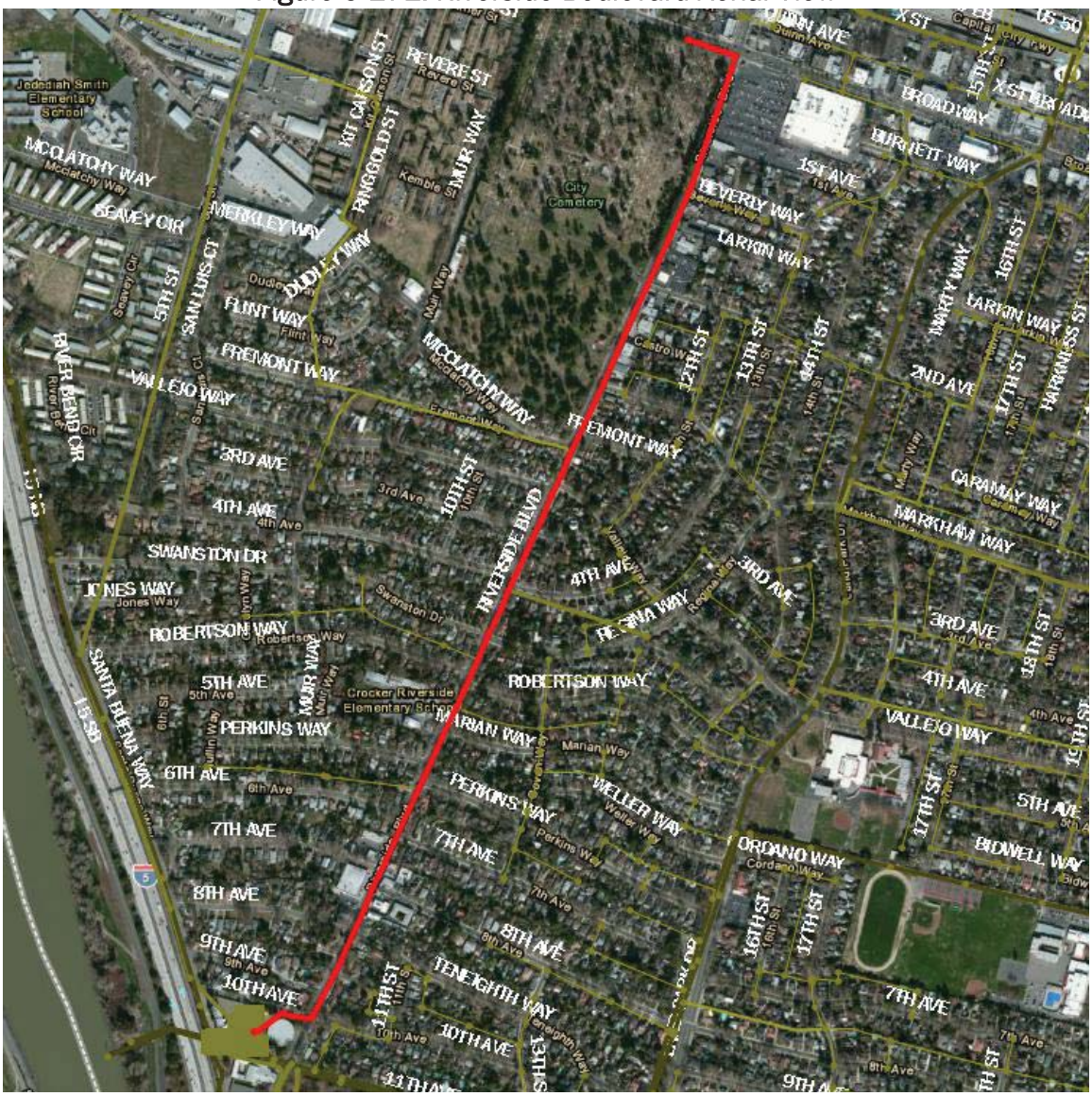


Figure 5-172: Riverside Boulevard Street View
(Looking North from Riverside and Beverly Way)

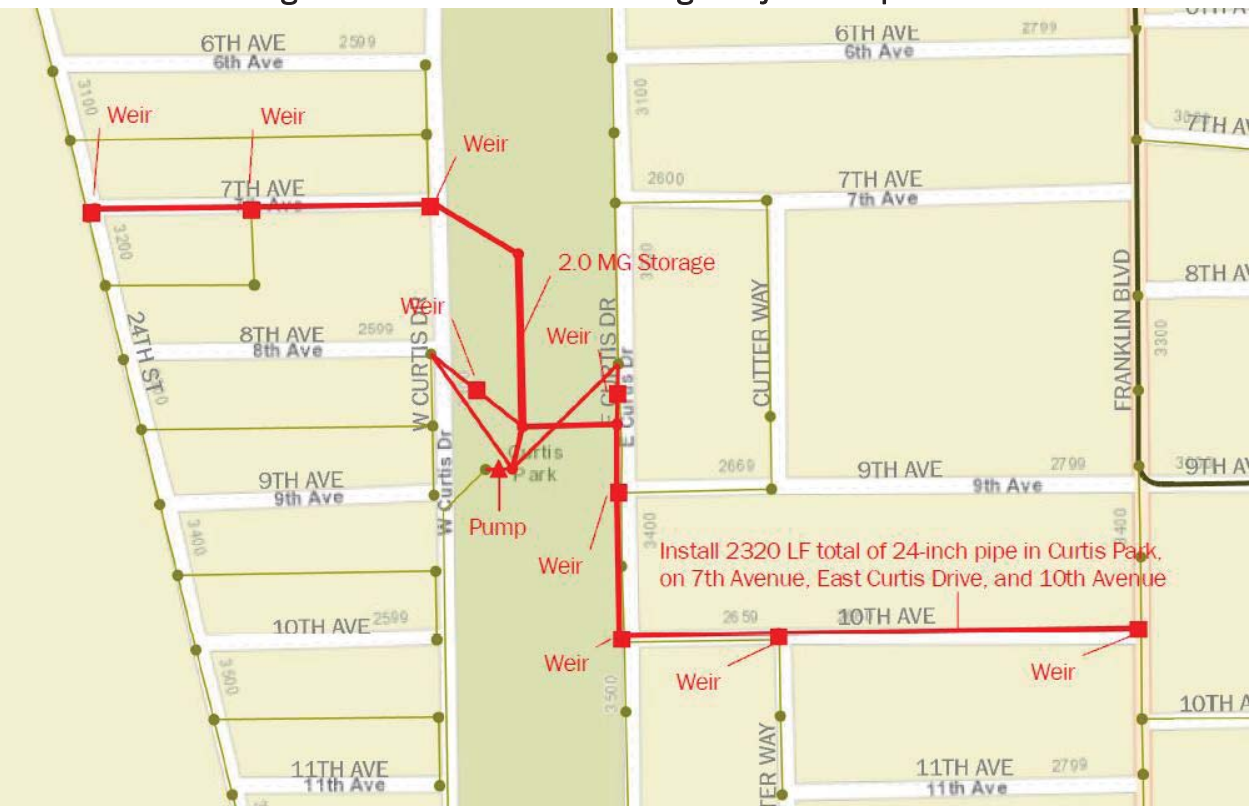
5.27 Project 6-3 (Curtis Park Storage)

As part of the CSSIP development process, DOU proposed a project to replace and redesign the old existing Sump 4 drainage pump station to drain a proposed storage facility placed at Curtis Park. Anecdotal data suggests that there is flooding in the area of Curtis Park, and the H&H model corroborates this. Figure 5-173 shows the baseline H&H model results for the area surrounding Curtis Park in a 10-year design storm.

Figure 5-173: Baseline Flooding near Curtis Park

The project would include a storage facility and an extended pipe system with multiple weir structures to collect flows from the smaller pipes in the area into the storage facility. Figure 5-174 presents the system components for this project.

Figure 5-174: Curtis Park Storage Project Components



Description: The project would be composed of a storage facility and pipe installations intended to bring flow to the facility. The pipe installation would include the following:

- Install 2,320 LF of 24-inch-diameter pipe in Curtis Park, 7th Avenue, East Curtis Drive, and 10th Avenue

The installed pipe would connect to the existing collection system via nine weir structures placed at the following locations:

- 7th Avenue and 24th Street
- 7th Avenue between 24th Street and West Curtis Drive
- 7th Avenue and West Curtis Drive
- 8th Avenue and West Curtis Drive
- East Curtis Drive between 7th and 9th Avenues
- 9th Avenue and East Curtis Drive
- 10th Avenue and East Curtis Drive
- 10th Avenue and Cutter Way
- 10th Avenue and Franklin Boulevard

The storage facility would be a 2 mg combined sewer storage vault built underneath Curtis Park. Sump 4 would be set to a pumping rate of 0.5 mg, lower than its current rate, to drain the storage facility. Sump 4 is currently larger than this and pumps during peak flows.

Benefits: This project would significantly lower peak HGL on 7th Avenue and Franklin Boulevard, and reduce flooding in the area. Table 5-26 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-175 through Figure 5-178 present the peak HGL profile in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-26: Modeled Peak HGL for Node Near Curtis Park Storage

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
E. Curtis Drive near 9th Avenue	13.7	**14.1**	12.9	12.9	**14.4**	13.1	13.1

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-175: Pre-Project Performance on 24th Street in 10-Year Storm

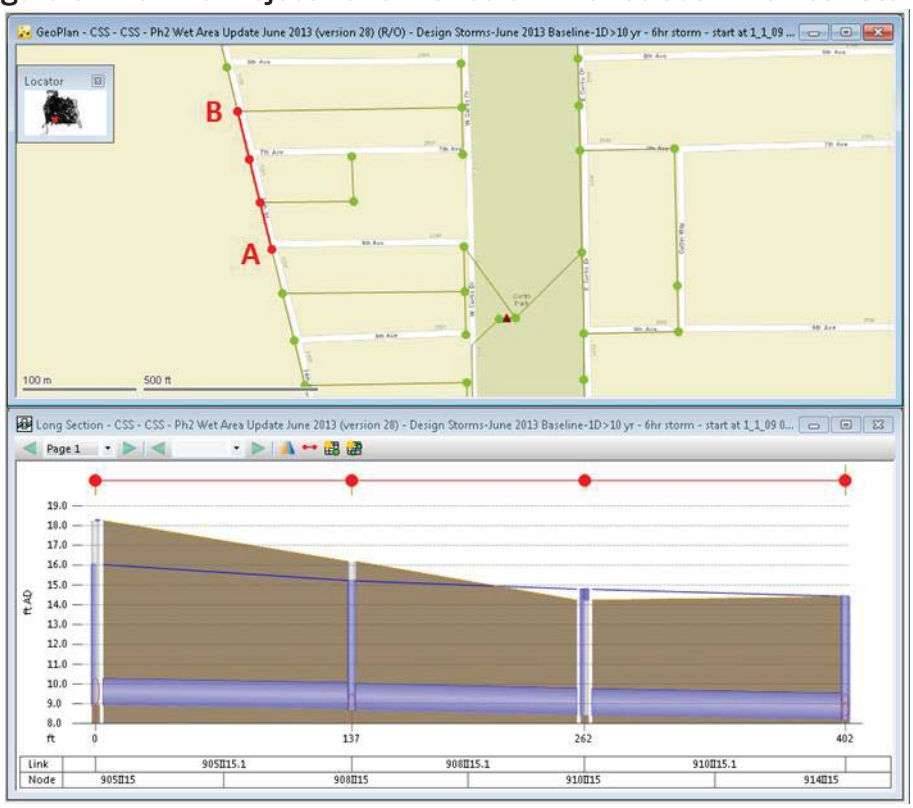


Figure 5-176: Post-Project Performance on 24th Street in 10-Year Storm

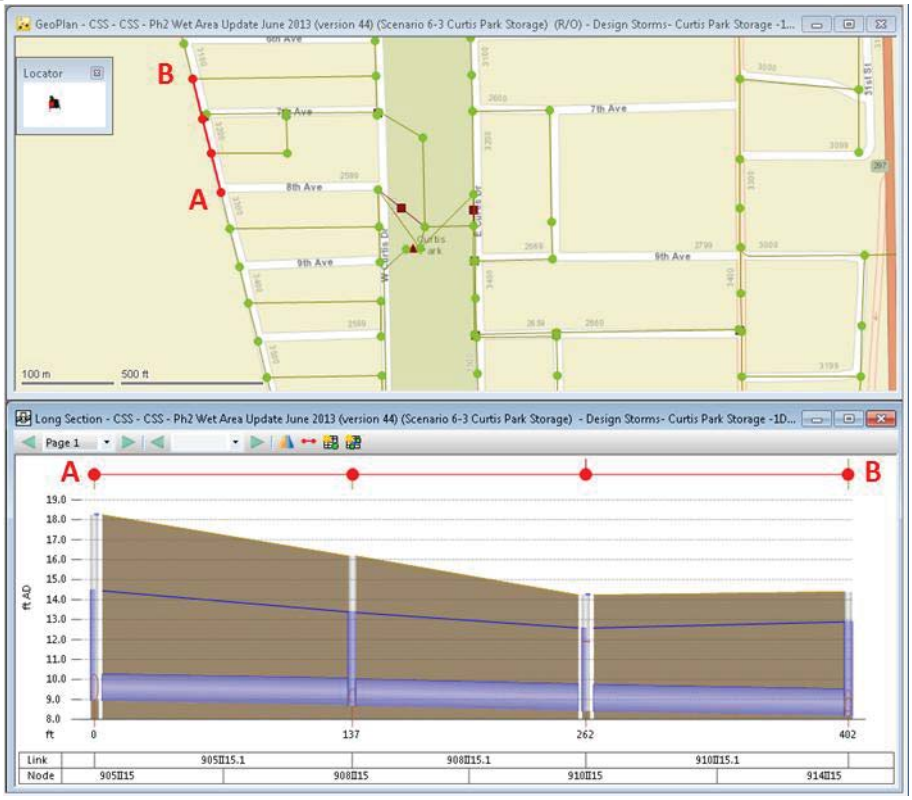


Figure 5-177: Pre-Project Performance on Franklin Street in 10-Year Storm

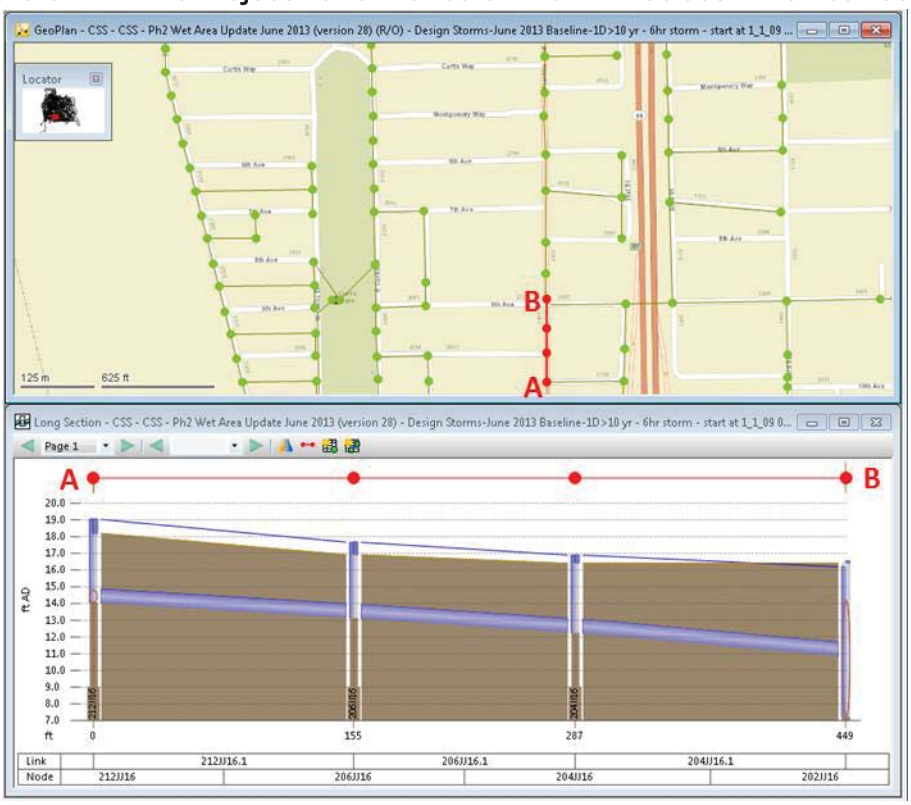
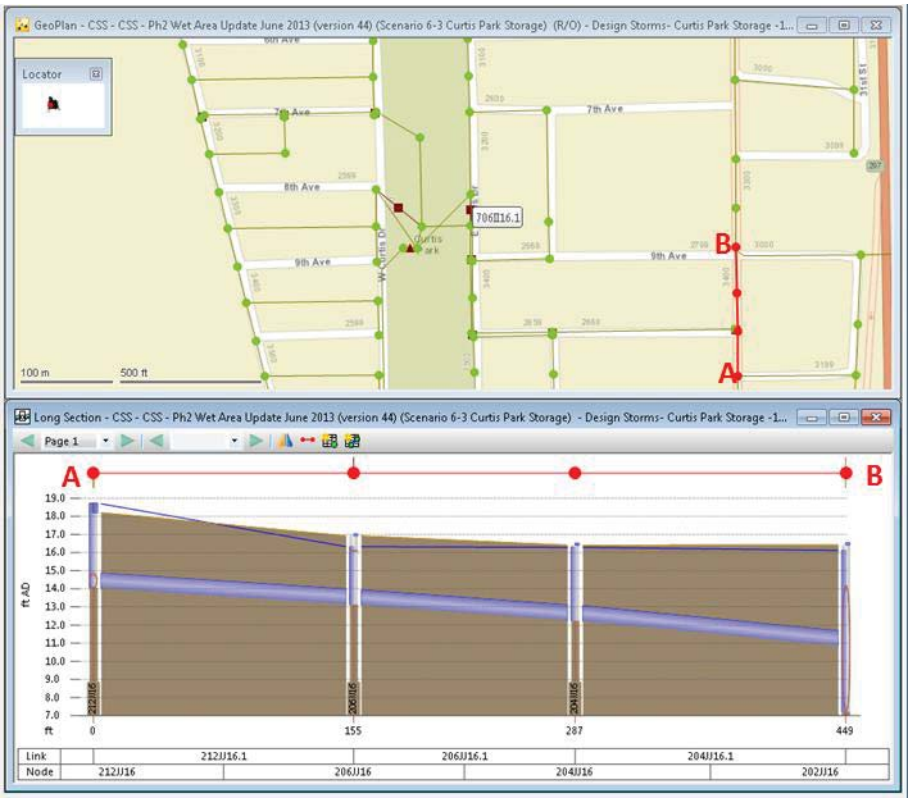
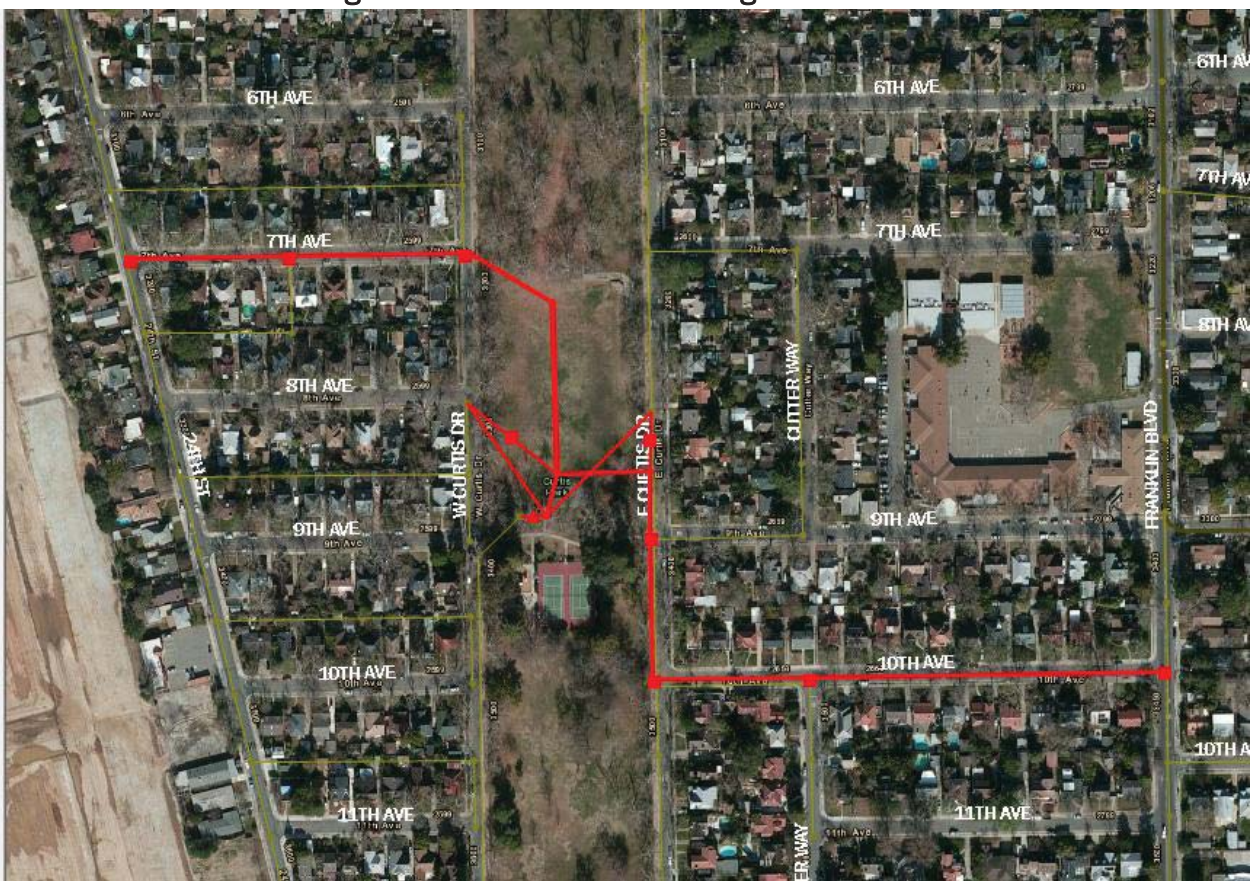


Figure 5-178: Post-Project Performance on Franklin Street in 10-Year Storm



Feasibility/Constructability: This project was suggested by DOU for two reasons: Sump 4 is an aging infrastructure and in need of replacement, and this project serves as an alternate site to the Curtis Park Village Storage (Project 3-3), due to delays in development of the storage project at the site. Figure 5-179 provides an aerial view of the project area.

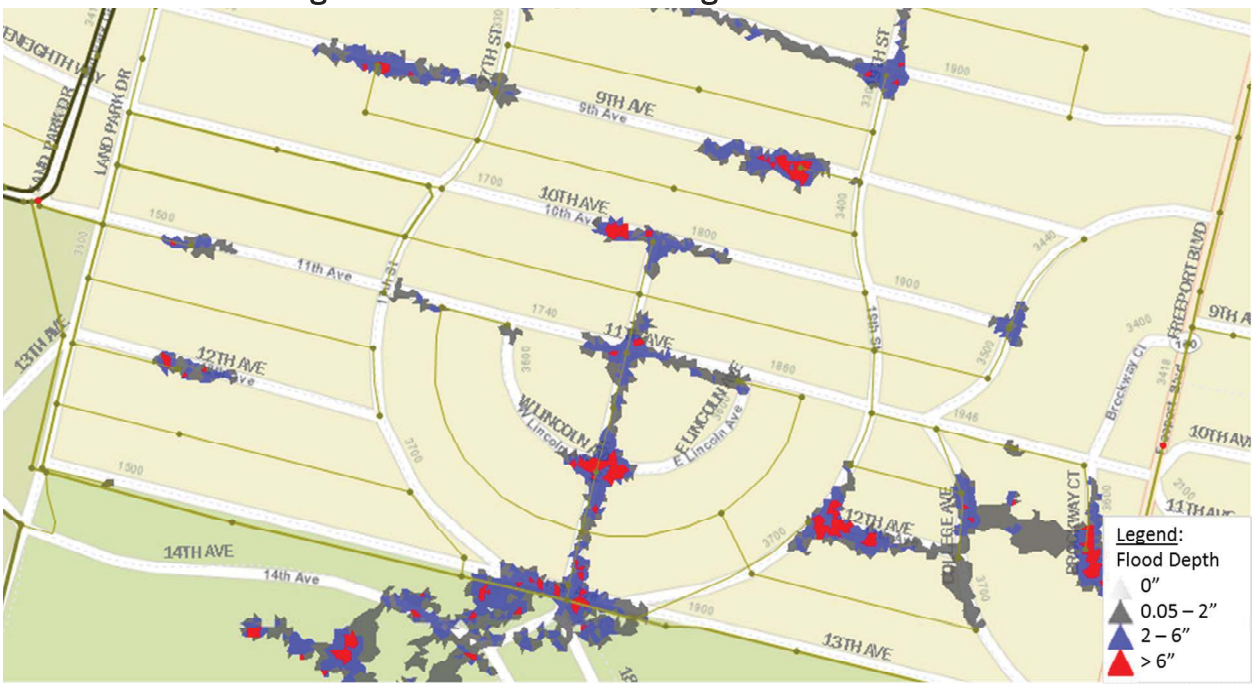
Figure 5-179: Curtis Park Storage Aerial View



5.28 Project 6-4 (11th Avenue Conveyance)

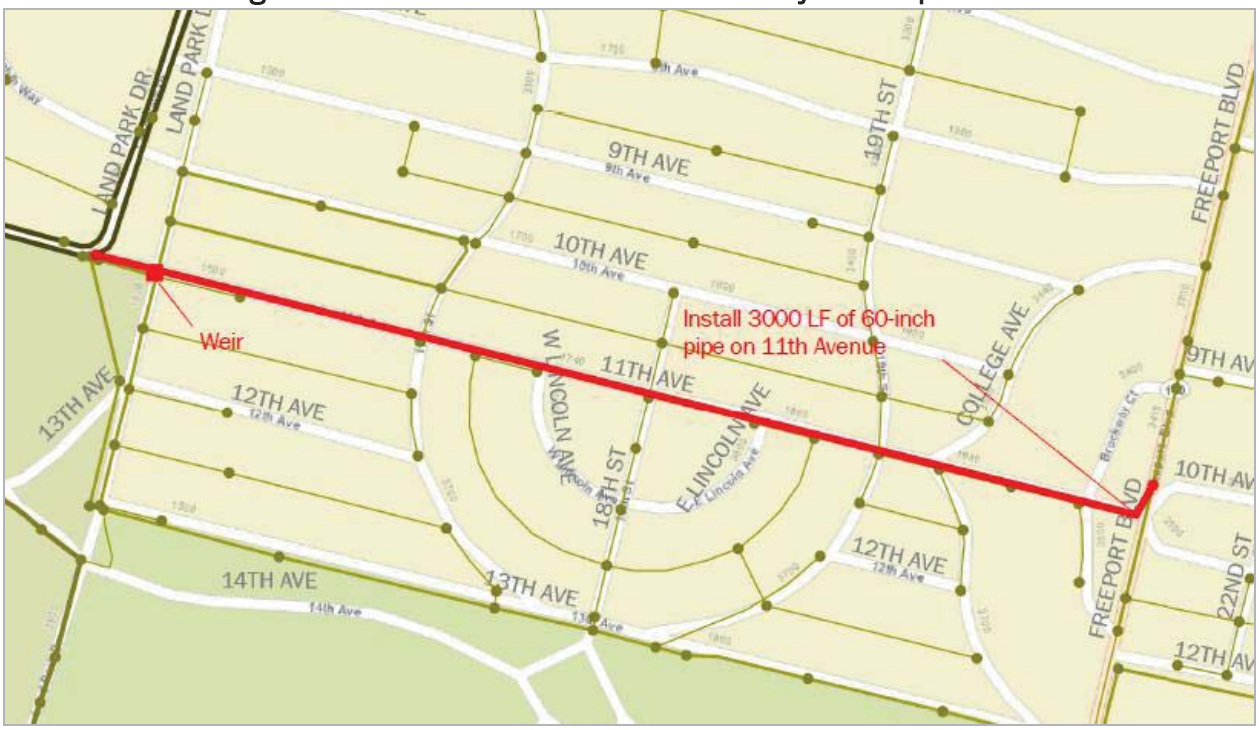
The H&H model indicates flooding in the area around Freeport Boulevard in Wet Area 6. In addition to Project 6-1 (Freeport Boulevard Upsizing), another project that can further reduce flooding in this area is to create alternate drainage paths to relieve surcharging pipes. Figure 5-180 shows the baseline H&H model results for the area surrounding 11th Avenue in a 10-year design storm.

Figure 5-180: Baseline Flooding near 11th Avenue



The 11th Avenue conveyance would install a new pipe along 11th Avenue to increase the conveyance of upstream flows. Figure 5-181 presents the system components for this project.

Figure 5-181: 11th Avenue Installation Project Components



Description: The project would install 3,000 LF of 60-inch-diameter pipe on 11th Avenue starting at Freeport Boulevard and ending at Land Park Drive. The new pipe would convey flow from Freeport Boulevard west and relieve Freeport Boulevard and other nearby drainage paths. A weir structure placed at 11th Avenue and Land Park Drive would collect peak flows from the pipe along Land Park Drive and further benefit the area along Land Park Drive.

Benefits: The project would significantly relieve Freeport Boulevard by lowering the HGL there, and provide moderate benefit to Land Park Drive. Table 5-27 presents the peak HGL at the node with highest flooding for baseline and project scenarios for the two design storms.

Figure 5-182 through Figure 5-185 present the peak HGL profiles in the baseline and project scenarios for the pipes in the area for the 10-year design storm.

Table 5-27: Modeled Peak HGL for Node Near 11th Avenue Conveyance

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
Freeport Blvd. and 10th	18.2	17.7	11.6	10.9	**18.4**	11.8	11.6

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

Figure 5-182: Pre-Project Performance on Freeport Blvd. in 10-Year Storm

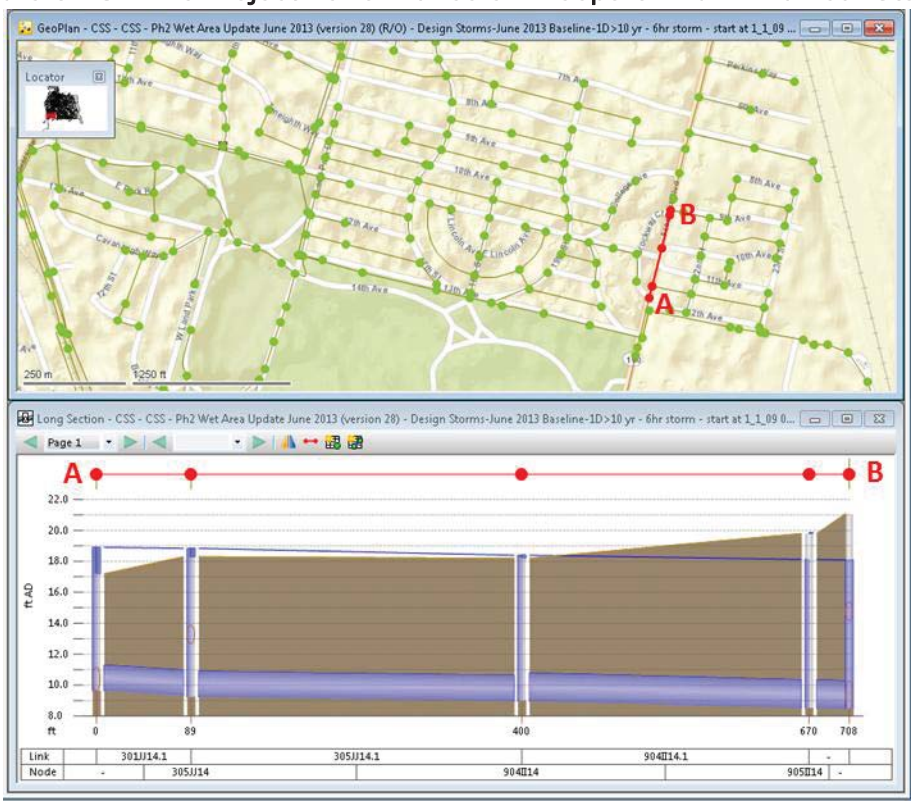


Figure 5-183: Post-Project Performance on Freeport Blvd. in 10-Year Storm

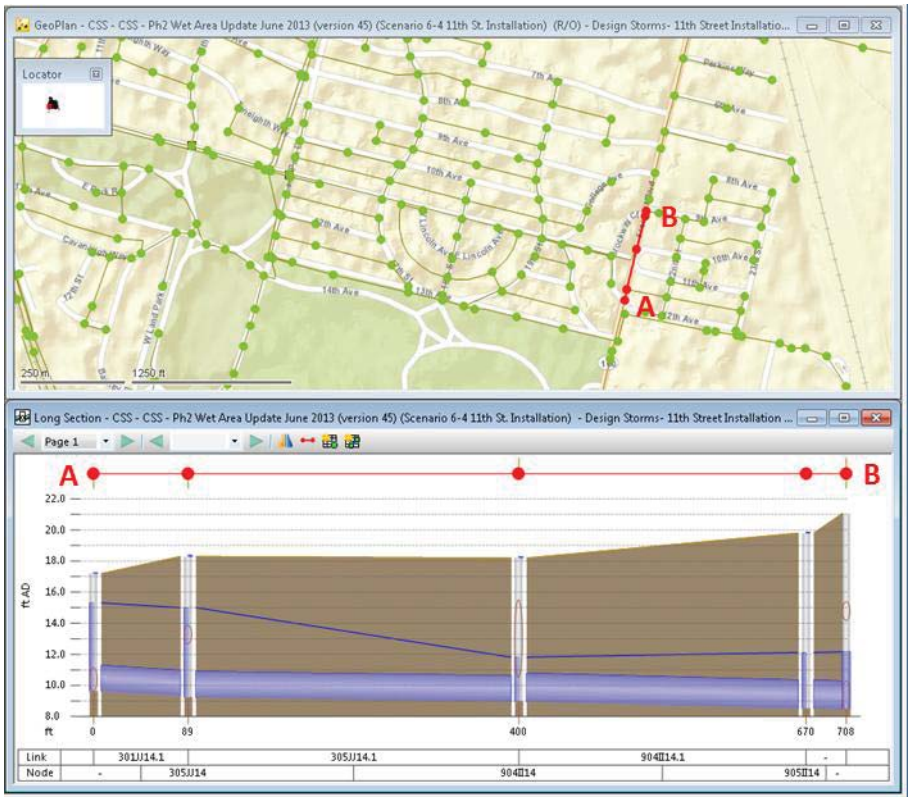


Figure 5-184: Pre-Project Performance on Land Park Drive in 10-Year Storm

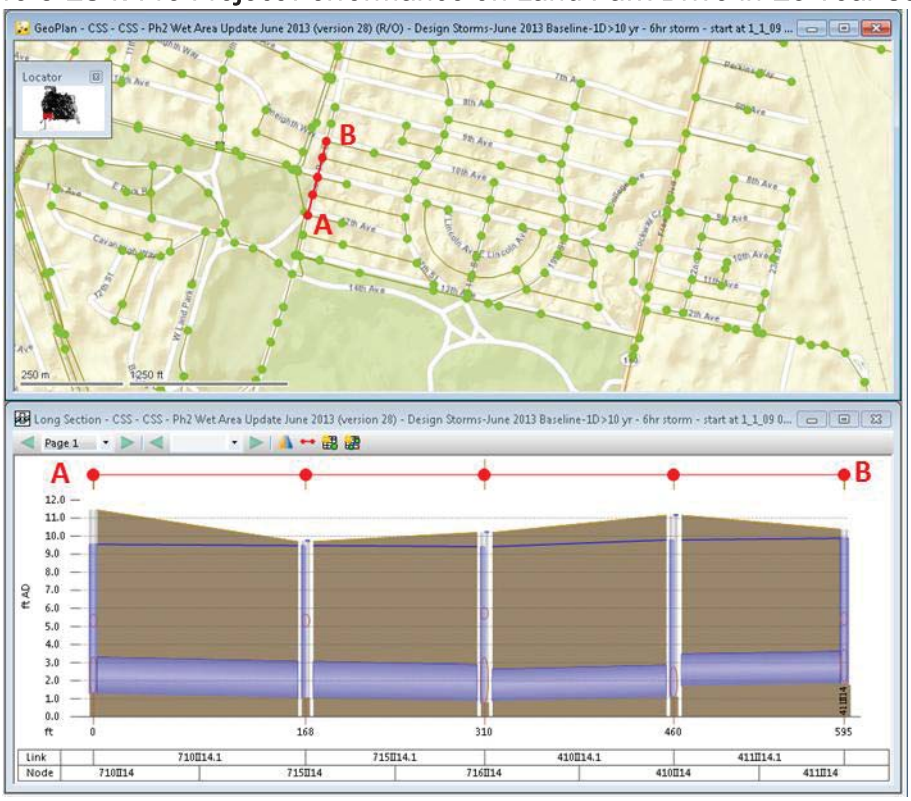


Figure 5-185: Post-Project Performance on Land Park Drive in 10-Year Storm

Feasibility/Constructability: This project appears to have high feasibility. 11th Avenue does not currently have any existing pipe, and appears to be wide enough to make construction of a new pipe feasible. Figure 5-186 shows and aerial view of the area and Figure 5-187 provides a street view of 11th Avenue at Freeport.

Figure 5-186: 11th Avenue Aerial View

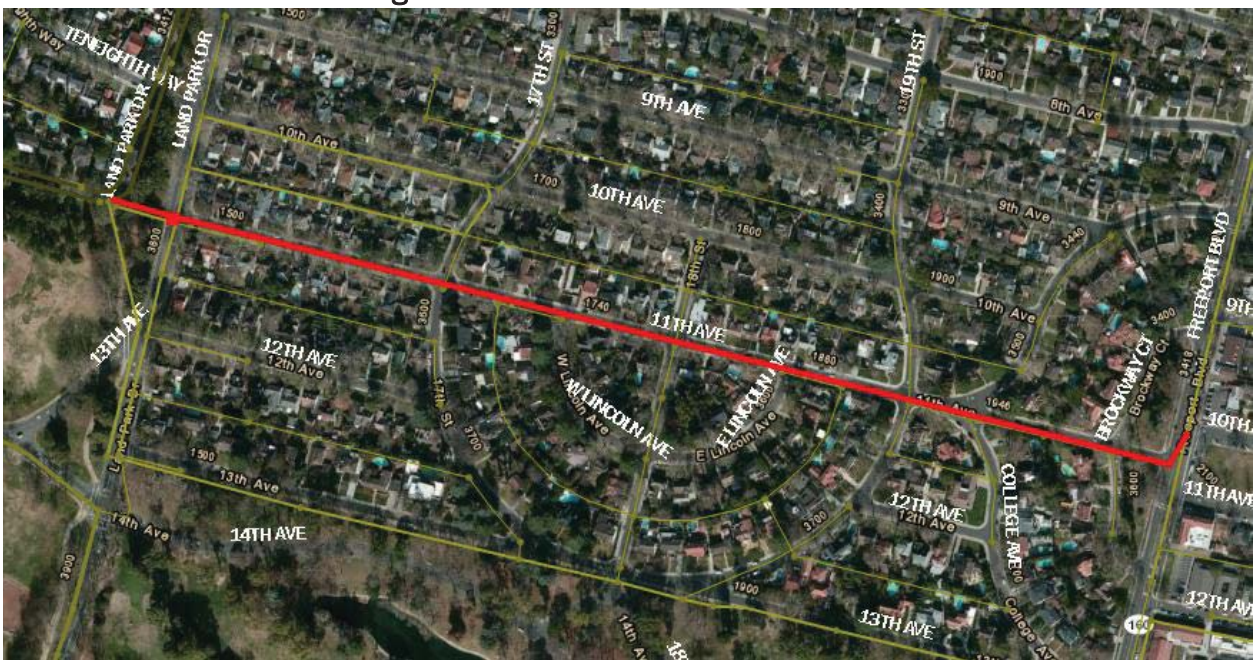
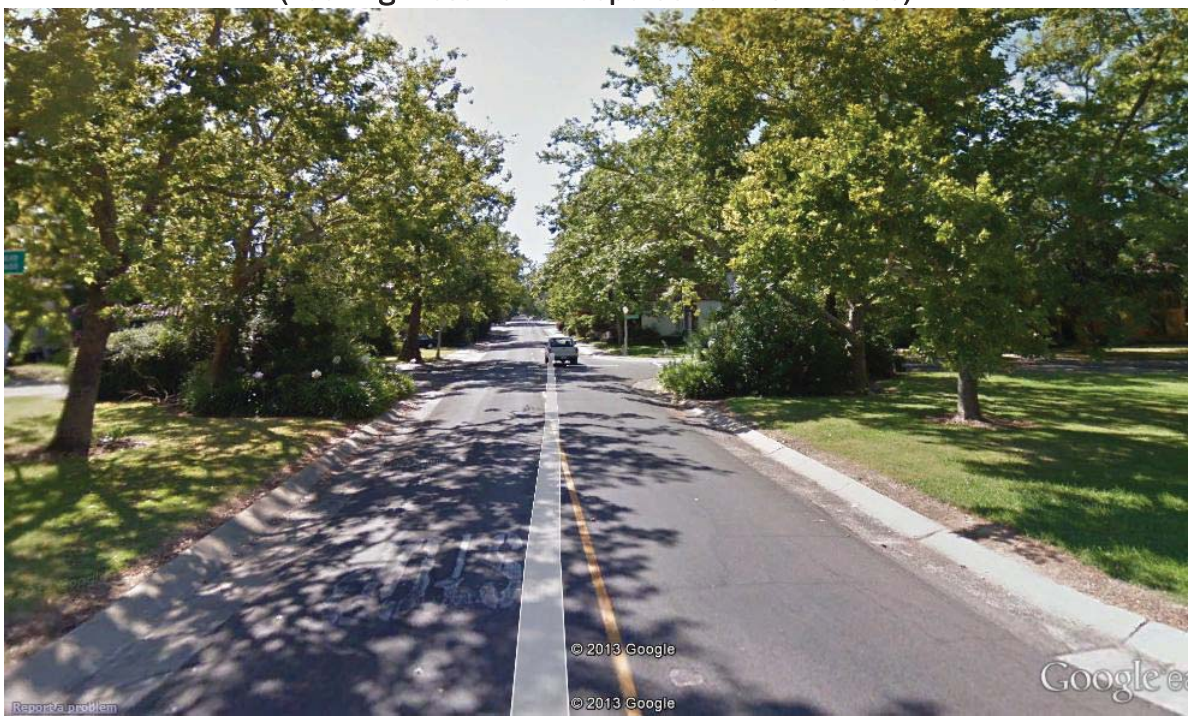


Figure 5-187: 11th Avenue Street View
(Looking West from Freeport and 11th Avenue)



5.29 Existing System Optimization Projects

Several existing linear and parcel storage projects throughout the system were evaluated for maximizing the storage capacity during the design storms and providing flood reduction benefits immediately upstream or downstream of the project. Four out of these projects were recommended for implementation for the existing system, and are included as part of the CSSIP projects. The four system optimization projects include:

i. 21st Street Local Storage Improvement

This existing system optimization project includes lowering the invert of sewer pipes on Burnett Way and 1st Ave, to allow more flows to enter the 21st Street linear storage and minimize flooding in the area west of the 21st St. Figure 5-188 below presents this optimization details. Table 5-28 presents the peak HGL at the node with highest flooding for baseline and project optimization scenarios.

Description: The 21st Street Local Storage project includes following components:

- Lower invert by 1.5 ft on upstream end of 150 LF of 24-inch pipe on Burnett Way draining to 21st St
- Lower invert by 1.5 ft on upstream end of 170 LF of 24-inch pipe on Burnett Way draining to 20th St
- Lower invert by 1.5 ft on upstream end of 150 LF of 30-inch pipe on 1st Ave draining to 21st St
- Lower invert by 1.5 ft on upstream end of 170 LF of 24-inch pipe on 1st Ave draining to 20th St

Figure 5-188: 21st Street Local Storage Optimization

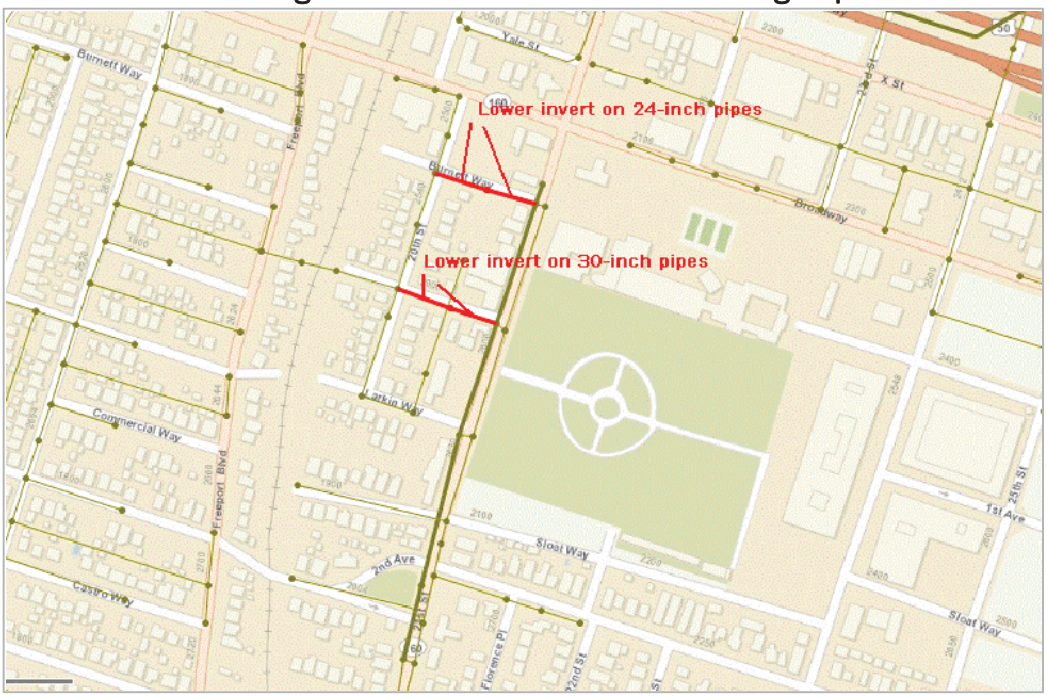


Table 5-28: Modeled Peak HGL for Node Near 21st Street Local Storage Improvement

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
18th St. and Beverly Way	13.10	12.89	12.86	9.79	**13.13**	13.06	11.66

Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation.
Model results are for individual project only using one-dimensional results.

ii. Tahoe Park/Broadway Parallel Sewer Project

This optimization project includes maximizing storage utilization in the 72”-90” interceptor on Broadway during design storms, to reduce peak flows during peak of the storm and minimizing flooding downstream of the interceptor. It includes modifying the control on existing sluice gate and add a new sluice gate for maximizing storage utilization. It also includes a 2’ deep x 4’ wide overland flow channel on Broadway between Santa Cruz Way and La Solidad Way to capture any overland flows and route the flows back into the collection system.

Figure 5-189 below presents the details of this optimization project. Table 5-29 presents the peak HGL at the node with highest flooding for baseline and project optimization scenarios.

Description: This project includes following components:

- Modify the controls of existing sluice gate at Broadway and Santa Cruz Way to reduce the sluice gate opening.
- Add a new sluice gate with controls on Broadway and 50th Street
  Add a 2' deep x 4' wide overland flow channel on Broadway between Santa Cruz Way and La Solidad Way to provide an overland flow path to any excess flows and route the flows back into the system.

Figure 5-189:Tahoe Park/Broadway Parallel Sewer Project

Table 5-29: Modeled Peak HGL for Node Near Tahoe Park/Broadway Parallel Sewer Project

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
8th Ave. and La Solidad Way	15.98	**17.94**	**17.92**	**17.38**	**18.88**	**18.83**	**18.06**
Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation. Model results are for individual project only using one-dimensional results.							

iii. East End Relief Sewer Tie-in Project

This optimization project includes improving the conveyance capacity of the L St. sewer by connecting the 30-inch sewer pipe on L St. to the 36-inch sewer on 15th St. This project provides a marginal benefit to flood reduction to the areas tributary upstream of the L St. Figure 5-190 below presents the details of this optimization project. Table 5-30 . presents the peak HGL at the node with highest flooding for baseline and project optimization scenarios.

Description: The East End Relief Sewer Tie-in includes following components:

- A New 20 LF of 30-inch pipe on 15th St. to tie-in with L St.sewer.
- Upsize 40-LF of existing 12-inch pipe to 30-inch on 15th St.

Figure 5-190: East End Relief Sewer Tie-in Project

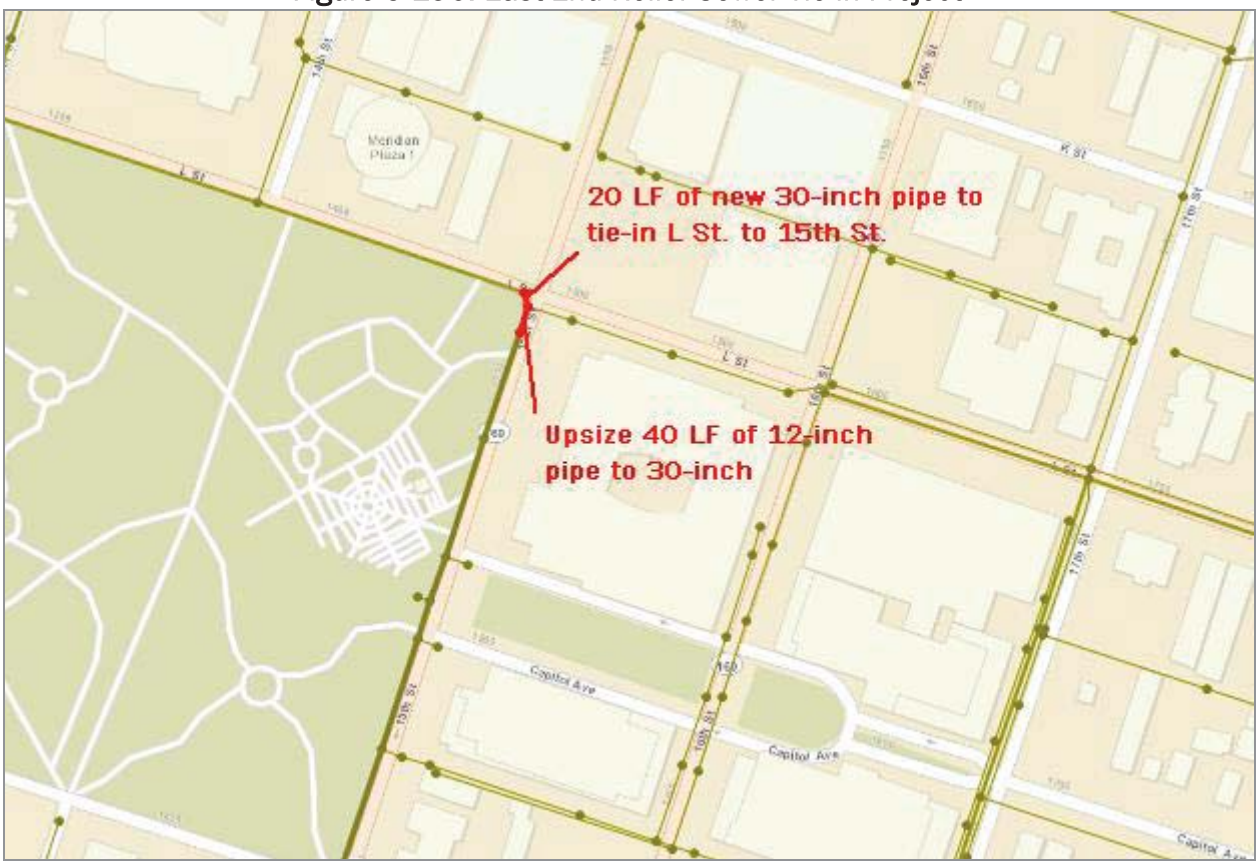


Table 5-30: Modeled Peak HGL for Node Near East End Relief Sewer Tie-in Project

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
L St. & 12th St.	17.83	17.33	16.38	14.65	**17.91**	17.77	17.04
Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation. Model results are for individual project only using one-dimensional results.							

iv. U St. Inline Storage Project

This optimization project includes lowering and expanding the weirs at U St. and 5th to increase the conveyance of flows to Sump 1, thereby reducing the hydraulic grade line in the system upstream of the U and 5th St and minimizing flooding. Figure 5-191 below presents

the details of this optimization project. Table 5-31 presents the peak HGL at the node with highest flooding for baseline and project optimization scenarios.

Description: The U St. Inline Storage Project includes following components:

- Lowering the weir on U St. (north side) by 1 ft and widening the weir from 5 ft. to 10 ft.
- Lowering the weir on U St. (south side) by 1 ft and widening the weir from 6 ft. to 10 ft.

Figure 5-191: U St. Inline Storage Project

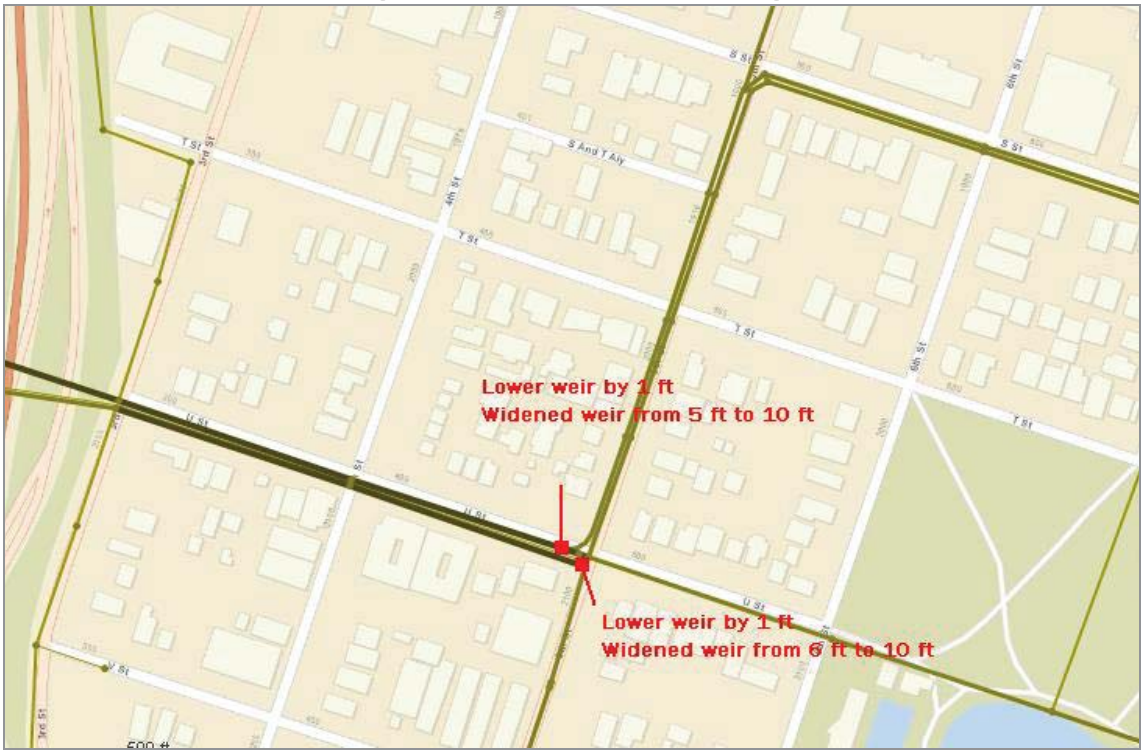


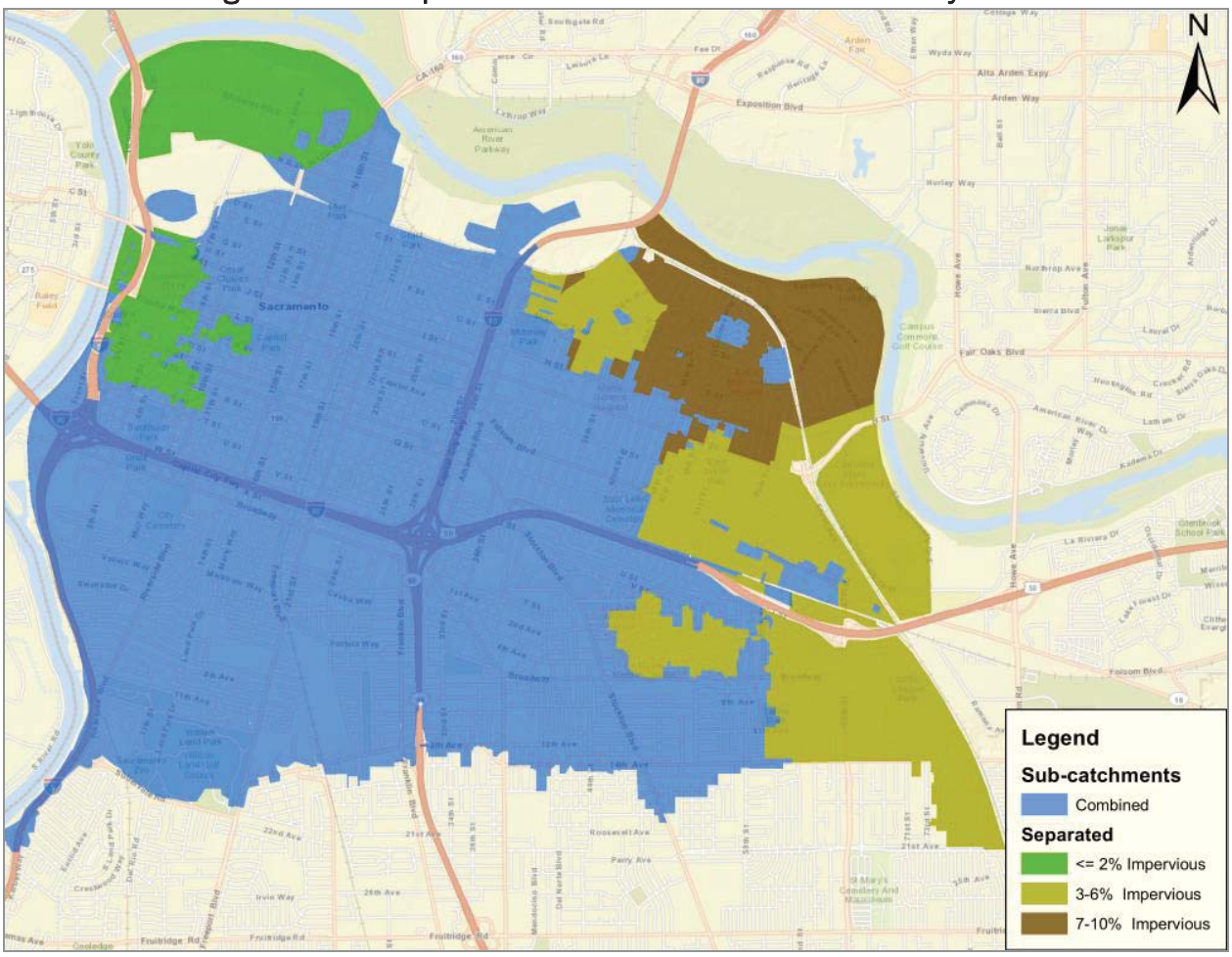
Table 5-31: Modeled Peak HGL for Node Near U St. Inline Storage Project

Model Intersection	Ground Elevation (feet)	Peak HGL in 5-Year 6-Hour Event			Peak HGL in 10-Year 6-Hour Event		
		Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)	Baseline (feet)	With Project (feet)	With All Projects and Programs (feet)
7th St. and S St.	13.34	10.26	10.03	9.10	11.86	11.81	11.16
Red bold text indicates peak hydraulic grade line (HGL) greater than ground elevation. Model results are for individual project only using one-dimensional results.							

5.30 Program 1 (RDII Reduction Opportunities)

Separated areas of the system contribute sanitary flows to the CSS area, and, during wet weather, contribute flow due to RDII to the CSS area. The flows due to RDII were generated in the H&H model as a percentage of impervious area, and were calibrated using flow monitoring data in the CSSIP Phase 1. Figure 5-192 shows the separated areas and the percentage impervious surface used to represent the RDII generated from these areas. The eastern side of the CSS area has a higher contribution of RDII flows to the CSS, and these flows can be reduced by source control. Source control programs to reduce RDII can include disconnection of any identified storm water connections to eliminate inflow, lining of pipes and manholes to reduce infiltration, replacing cracked or old pipes, and other means. Reduction in RDII flows from these areas can augment CSSIP projects in reducing flooding in areas downstream of the separated system.

Figure 5-192: Separated Areas of the Combined Sewer System



Benefits: To evaluate the benefits of RDII reduction, a scenario was set up with reduced percent impervious with the following reductions:

- 7–10% impervious areas adjusted to 2% impervious areas
- 3–6% impervious areas adjusted to 1% impervious areas

The two design storms were simulated using the model scenario with RDII reduction. The system-wide benefits are tabulated in the Table 5-32: System-Wide Benefits of RDII Reduction. RDII reduction was applied in separated areas upstream of Wet Area 2.

Table 5-32: System-Wide Benefits of RDII Reduction

Extent	Outflow Volume (mg) for 5-Year 6-Hour		Outflow Volume (mg) for 10-Year 6-Hour	
	Baseline	With RDII Reduction	Baseline	With RDII Reduction
Outside Wet Area	9.1	8.1	24.9	22.5
Wet Area 1	2.6	2.5	6.1	6.0
Wet Area 2	2.3	2.0	6.4	5.2
Wet Area 3	2.5	2.4	6.4	6.3
Wet Area 4	0.4	0.4	1.4	1.4
Wet Area 5	1.0	1.0	2.5	2.5
Wet Area 6	1.0	1.0	2.9	2.8
System-Wide	**18.9**	**17.4**	**50.6**	**46.8**

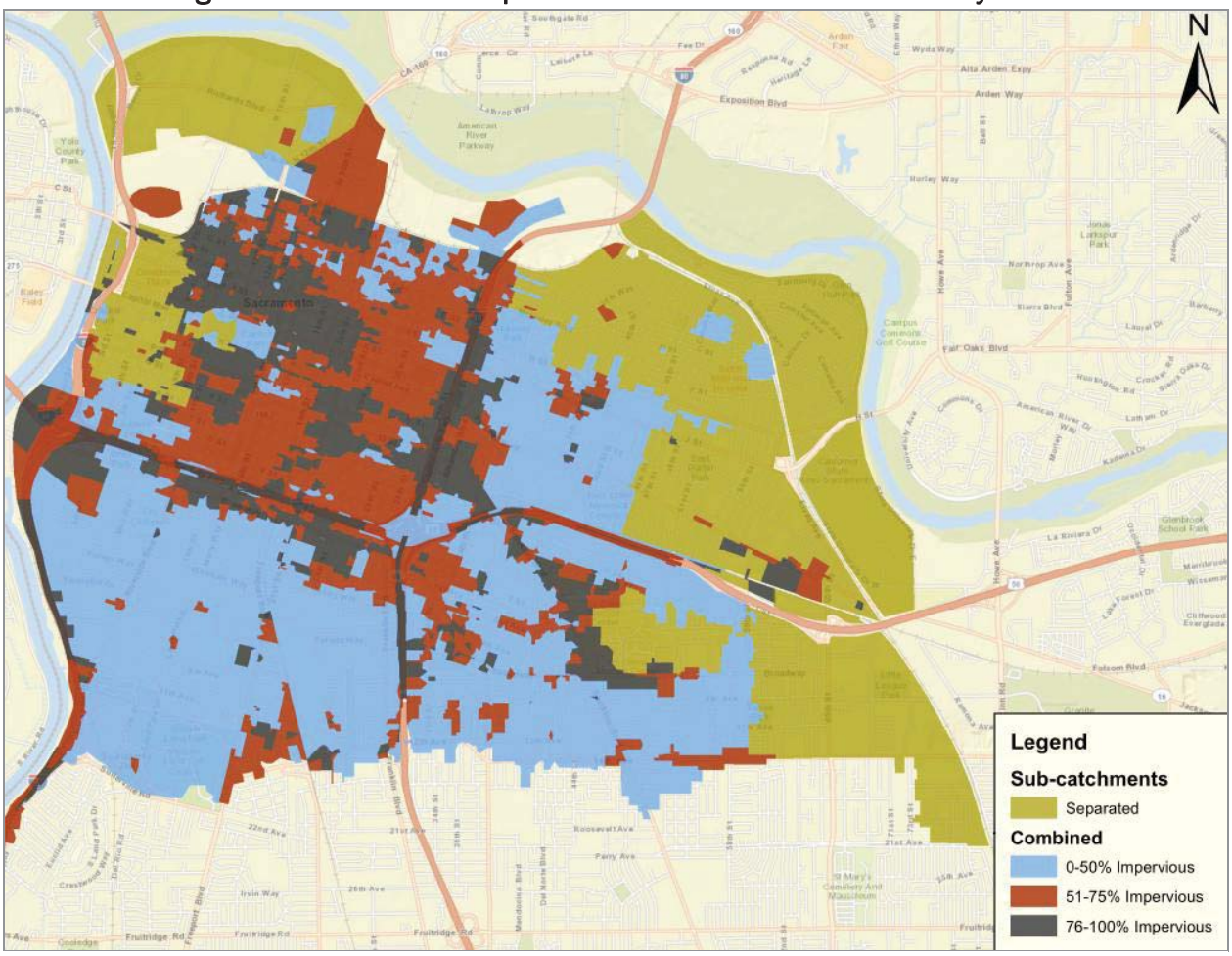
Notes: Separated areas are modeled as percent impervious of the contributing sub-catchment to generate RDII flows during wet weather. The baseline percent impervious for separated areas ranges from 1-10 % of the contributing area. To model benefits of RDII reduction:

- Areas with 7-10 % impervious were reduced to 2% impervious
- Areas with 3-6 % impervious were reduced to 1% impervious

5.31 Program 2 (GI)

GI technologies, also known as low-impact development, can augment the benefit of capital projects to the CSS by reducing runoff volume and potentially attenuating the peak flows entering the system. GI technologies can modify the hydrograph by providing retention or detention of flows, and increasing infiltration and evaporation losses from the surface. Figure 5-193 presents the percent impervious of the CSS. The northwest part of the City is highly impervious and is prone to flooding in several locations.

Figure 5-193: Percent Impervious Surface of the Combined System



Benefits: To evaluate the benefits of GI to the combined system, 5% of impervious area within the combined system was converted to pervious surfaces. Table 5-33 presents the system-wide benefits of GI, with almost all wet-weather areas having a moderate level of flood-reduction benefits.

City-wide application of GI would provide reasonable benefits, and application of such technologies strategically placed upstream or near flooding areas would further improve the performance of the system in conjunction with the other CSSIP projects. Site-specific issues with GI technologies may include public acceptability, potentially higher maintenance costs, and acceptable performance in larger storms such as 5-year and 10-year. The GI program should initiate with smaller pilot programs to evaluate the acceptability and performance of the technologies prior to launching a City-wide program.

Table 5-33: System-Wide Benefits of Green Infrastructure

Extent	Outflow Volume (mg) for 5-Year 6-Hour		Outflow Volume (mg) for 10-Year 6-Hour	
	Baseline	With Green Infrastructure	Baseline	With Green Infrastructure
Outside Wet Area	9.1	7.6	24.9	21.8
Wet Area 1	2.6	2.2	6.1	5.5
Wet Area 2	2.3	2.0	6.4	5.5
Wet Area 3	2.5	2.0	6.4	5.6
Wet Area 4	0.4	0.3	1.4	1.2
Wet Area 5	1.0	0.8	2.5	2.3
Wet Area 6	1.0	0.8	2.9	2.5
System-Wide	**18.9**	**15.7**	**50.6**	**44.4**

Notes: To model benefits of GI, 5% of the impervious area within the combined system was converted to the pervious area.

5.32 Program 3 (Water Conservation)

The sewer flows generated in the baseline conditions are approximately 140 gpd/ESD (See Chapter 4 for a discussion of ESDs). The City estimates increased growth of the Sacramento region, leading to higher residential and commercial population within the DOU sewer service area. The increase in sewer flows for future planning conditions is projected to be 310 gpd/ESD. However, water conservation plays a major role in sustainable development in this region and DOU plans to promote water conservation within its service area. The sewer flows for future planning conditions with water conservation are estimated to be 256 gpd/ESD. Table 5-34 presents the system-wide benefits of water conservation. The benefits from water conservation are marginal for flood reduction, but conservation may provide additional benefits by reducing energy costs at Sump 2/2A. It must be noted that reducing sanitary flows in large pipes in flat areas may cause or exacerbate odor and/or sediment deposition issues in the area.

Table 5-34: System-Wide Benefits of Water Conservation

Extent	Outflow Volume (mg) for 5-Year 6-Hour		Outflow Volume (mg) for 10-Year 6-Hour	
	Baseline	Water Conservation	Baseline	Water Conservation
Outside Wet Area	9.1	8.8	24.9	24.3
Wet Area 1	2.6	2.5	6.1	6.0
Wet Area 2	2.3	2.3	6.4	6.3
Wet Area 3	2.5	2.4	6.4	6.3
Wet Area 4	0.4	0.4	1.4	1.4
Wet Area 5	1.0	1.0	2.5	2.5
Wet Area 6	1.0	1.0	2.9	2.8
System-Wide	18.9	18.5	50.6	49.6
Notes: To model benefits of Water Conservation, the future sanitary flow generation of 310 gpd/ESD was reduced to 256 gpd/ESD.				

6. COST OF CSSIP PROJECTS

6.1 Cost Basis and Assumptions

West Yost Associates developed the estimates for construction costs for the CSSIP projects described in Section 5. Section 6 provides the process and assumptions used to generate the cost estimates. The costs were developed based on actual construction costs for similar facilities built by DOU, construction costs previously estimated by West Yost Associates for similar facilities designed but not constructed, published industry standard cost data and curves, and data supplied by manufacturers.

These construction cost estimates do not include uncertainties or unanticipated construction costs (e.g., variations in final quantities, utility relocations), or cost estimates for engineering, legal, environmental review, inspection and/or contract administration. These additional cost items are referred to as "construction contingency costs," and are added to the project construction cost to estimate the capital cost of the project.

All costs were adjusted to August 2013 dollars based on an Engineering News Record (ENR) Construction Cost Index (CCI) of 9967. The ENR CCI uses a 20-city national average; this analysis also used San Francisco CCIs. These indices were used for conceptual cost estimates only, and will be updated as the projects become better defined. Construction costs presented in this section are not intended to represent the lowest prices in the industry for each type of construction; rather, they are representative of average or typical construction costs. The planning-level cost estimates were prepared for guidance in evaluating various options, and are intended for comparative purposes only.

Historical project costs were updated to August 2013 dollars based on City procedures of averaging the ENR CCIs of the 20 cities and San Francisco to represent inflation escalation in the Sacramento area. Miscellaneous pipe costs, or costs associated with the project other than the major pipe item, were estimated using the historical cost of pipe-only projects and comparing that to other storage pipes. The costs for the miscellaneous pipe materials were approximately 70% of the construction costs of the pipes. Project costs were developed based on the City's Project Cost Summary table, received by West Yost Associates on December 4, 2013. The various components of the proposed CSSIP projects considered are discussed below and consist of linear storage, parcel storage, pipe upsizing, new pipe installation, pump stations, weir structures, flap valves, GI, and RDII source control.

6.2 Linear Storage

Linear storage projects are the construction of large-diameter pipes typically parallel to existing combined system pipes, with dry weather flows remaining in the existing pipe and high flows diverted to the new, large pipe at an upstream manhole or junction box. The high flows are then temporarily stored in and routed through the large pipe to the downstream end, where the flows are discharged back to the existing combined system pipes by gravity or pumped out to a downstream pipe once the downstream combined system capacity is restored.

Unit construction costs for linear storage are based on an estimate of $12 per-inch diameter per linear foot of pipe. The pipe unit cost was estimated using costs from previous linear storage projects constructed within the City, and other larger-diameter pipe project cost estimates developed by the City and West Yost. These costs include pipe materials, trenching, placing and jointing pipes, and backfill. An additional amount of 70% was added to the storage pipe costs to estimate the costs of associated items and activities needed to construct linear storage facilities, including manholes, junction structures, miscellaneous pipe materials, sheeting and shoring, dewatering, traffic control, service connections, asphalt pavement replacement, and relocation of other utilities such as gas and electric.

6.3 Parcel Storage

Parcel storage facilities are typically off-line facilities that allow high flows to be diverted to large storage vault facilities. The storage facility is typically located within a park or parking lot, and is a cast-in-place structure or made of precast pipes to accommodate the needed volume.

The lump sum costs for parcel storage were estimated using updated construction and engineering cost estimates from previous CSS storage projects within the City. The previous projects' construction costs and the constructed storage volume were plotted to develop a relationship between storage volume and total project construction cost for parcel storage projects.

The following previous projects were used in the estimate of parcel storage construction costs:

- 42nd Street Drainage Area Storage and Pump Station Improvement Project
- UCD Medical Center Regional Storage Project
- Oak Park Storage Project

Figure 6-1 illustrates the relationship between parcel storage volume and updated construction costs.

The construction cost estimates for the CSSIP parcel storage projects do not include the cost associated with land acquisition.

6.4 Pipe Upsizing and New Pipe Installation

Pipe upsizing and new pipe installation projects provide additional conveyance capacity and some in-line storage. They consist of constructing larger-diameter pipes within City streets and either replacing the existing pipe or abandoning the smaller pipe.

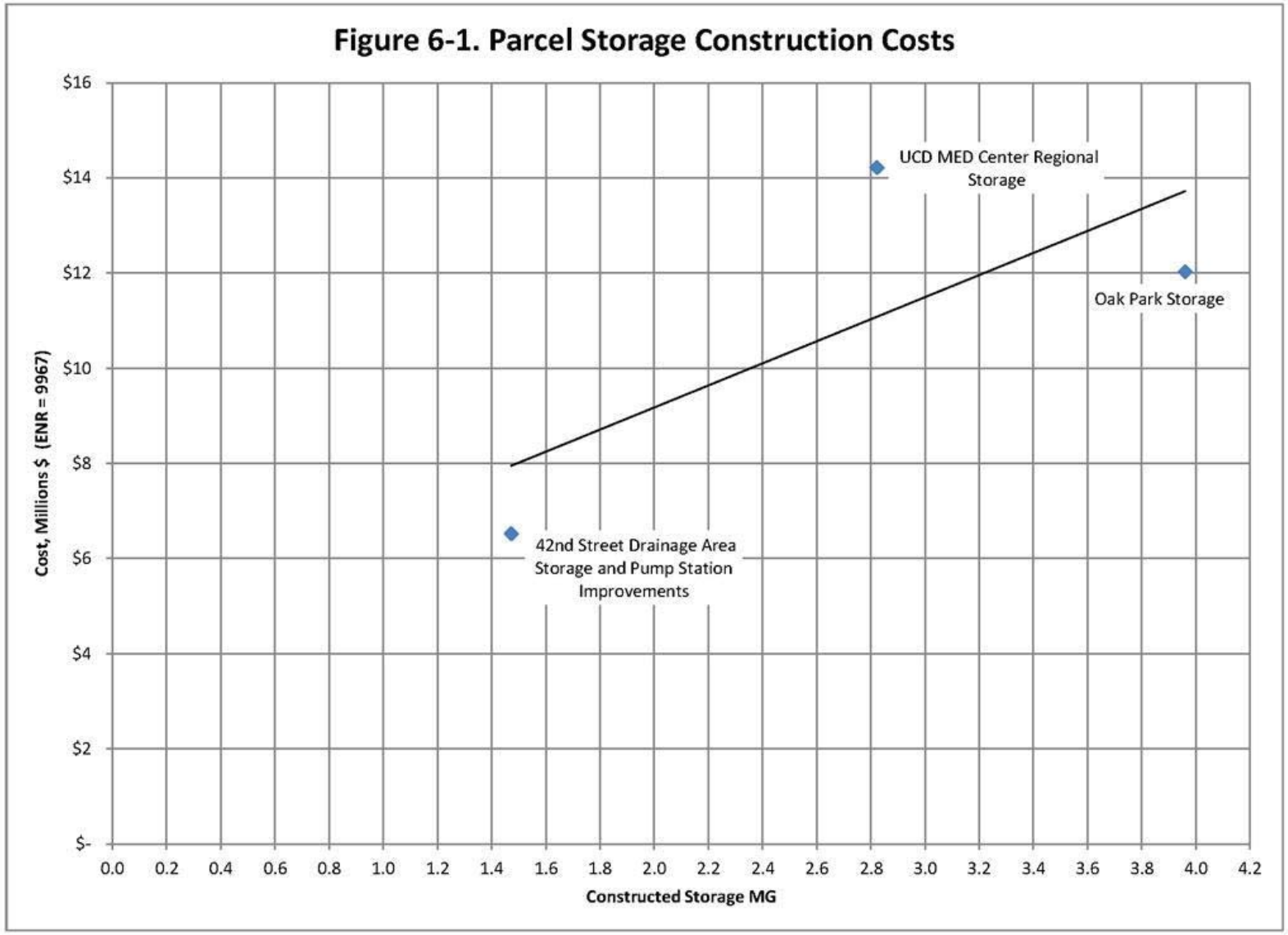


Figure 6-1. Parcel Storage Construction Costs
$16
$14
$12
$10
$8
$6
$4
$2
$-
Cost, Millions $ (ENR = 9967)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
4.2
Constructed Storage MG
UCD MED Center Regional Storage
Oak Park Storage
42nd Street Drainage Area Storage and Pump Station Improvements

New pipe construction and pipe upsizing costs were estimated using $12 per-inch diameter per linear foot of pipe unit cost. The pipe unit cost was estimated using previous linear storage projects constructed within the City, and other larger-diameter pipe project cost estimates developed by the City and West Yost. These costs include pipe materials, trenching, placing and jointing pipes, and backfill. An additional amount of 70% was added to the storage pipe costs to estimate the costs of associated items and activities needed to construct linear storage facilities, including manholes, junction structures, miscellaneous pipe materials, sheeting and shoring, dewatering, traffic control, relocation of other utilities such as gas and electric, service connections, and asphalt pavement replacement.

6.5 Pump Stations

CSS pump stations are typically associated with off-line storage facilities. The pump station provides a controlled release of stored water within the storage facility where gravity outflow is not achievable.

Pump station costs were estimated using previous pump station project cost information from the City and West Yost, and the 2013 Heavy Construction RSMeansOnline project database. Pump station costs vary considerably, depending on such factors as architectural design, pumping head, and station capacity. A planning estimate was used for the proposed CSSIP projects, and should be used for comparative purposes only.

Figure 6-2 illustrates the relationship developed between pump station capacity and updated construction costs.

6.6 Weir Structures

Weir structures are needed to divert flow from an existing pipe to a larger on- or off-line pipe or storage facility. They are typically constructed within a manhole or junction structure.

Factors that affect the cost of a weir structure include size, depth, and location. Other facilities and activities related to the cost of constructing a weir structure are excavation, backfill, shoring, dewatering, concrete structure box with access lids, and surface restoration. A planning-purposes unit cost of a weir structure is approximately $75,000. The costs of weir structures for off-line storage facilities have been incorporated in the additional 70% cost applied to the pipe or parcel storage cost installation.

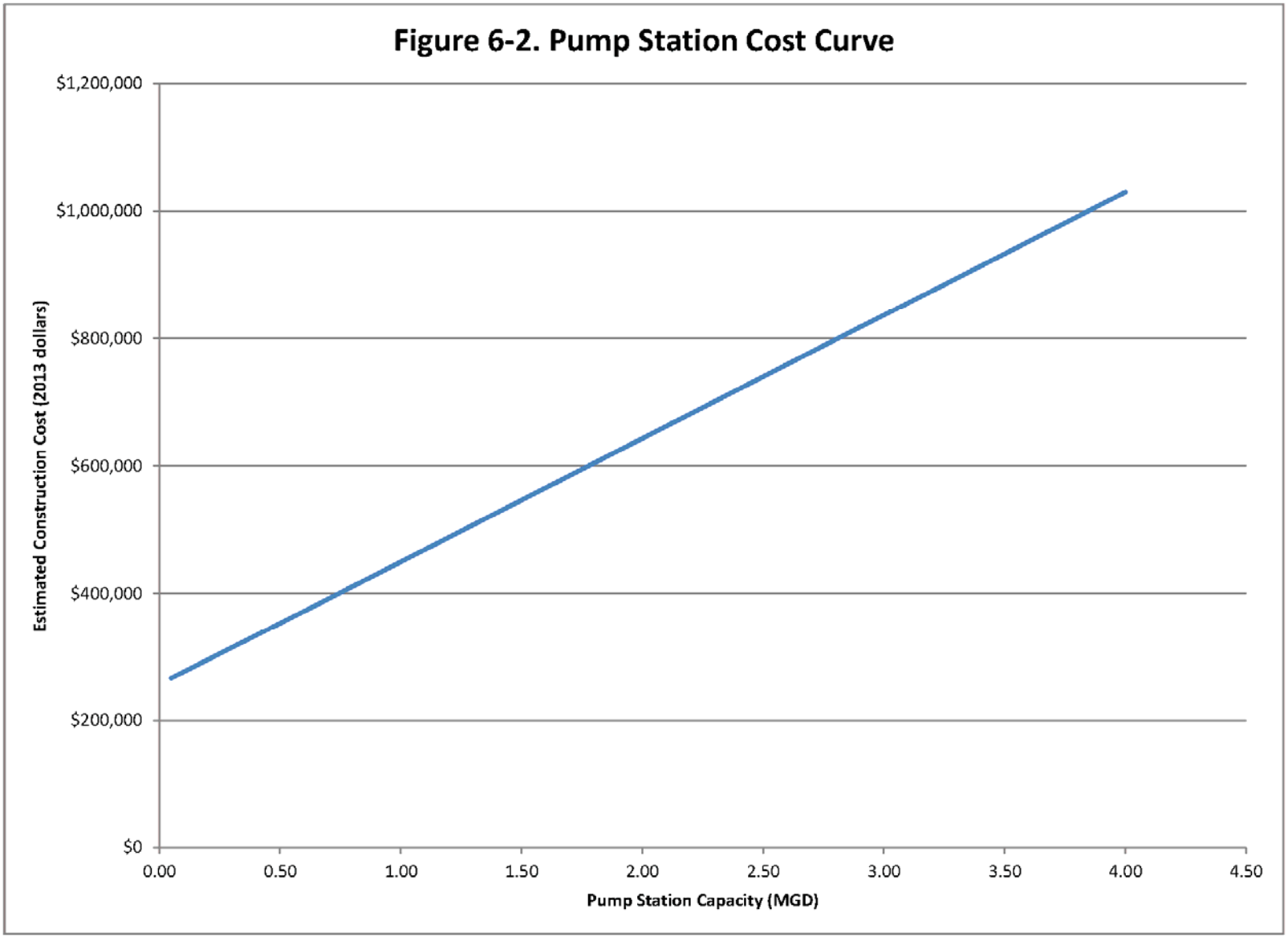


Figure 6-2. Pump Station Cost Curve
$1,200,000
$1,000,000
$800,000
$600,000
$400,000
$200,000
$0
Estimated Construction Cost (2013 dollars)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
Pump Station Capacity (MGD)

6.7 Flap Valves

Flap valves are typically added to the ends of pipes within existing manholes or junction structures to keep flow from entering the pipe from the downstream direction.

The costs of flap valves have been incorporated in the 70% contingency applied to the cost of the pipe upsizing and new pipe installation. A budgetary cost estimate of a stainless steel heavy-duty flap gate is $1,300 for a 6-inch-diameter flap valve and $1,600 for an 8-inch-diameter flap valve, as manufactured by Fresno Valves & Castings and HydroGate. For this study, it was assumed that the installation of a flap valve and pipes less than 12 inches in diameter would be $5,000.

6.8 Green Infrastructure

DOU developed an initial program concept and cost estimate for a GI program through internal discussions between the Engineering and Water Quality groups. The estimate and concepts are based on low-impact-development tools used by the Water Quality group, and reference materials made available by other utilities and developers nationwide. The program consists of rebate incentives and projects executed with partner agencies. The full suite of project types is as follows:

- Bioretention bulb-outs at street intersections
- Porous pavement on streets
- Interceptor tree partnership with Urban Forestry
- Rain barrel residential rebate program
- Disconnected roof drains residential rebate program
- Alternative driveway design residential rebate program
- Impervious area management residential rebate program
- Alternative pavement design commercial rebate program

GI improvements can vary significantly depending on site-specific conditions. The initial program concept and cost estimate are intended to be used as a starting point from which to launch a pilot study. The pilot study would identify the most viable and effective project types and target suitable locations via small-scale pilot projects, pre- and post-implementation monitoring, and benefits analysis. The initial concept and cost estimate would then be adaptively updated based on the findings of the pilot study to develop the most appropriate program for the CSS.

6.9 RDII Source Control

Costs for source control programs to reduce RDII can vary considerably depending on the length of pipe to be lined and the type of lining. For small-diameter projects (6-inch to 15-inch pipe), the typical rehabilitation project usually consists of a mix of open cut replacement, pipe-bursting with a new HDPE pipe, or lining using cured-in-place pipe (CIPP). For larger-diameter projects (18-inch and larger pipe), the typical rehabilitation project usually consist of lining with CIPP.

Unit costs for RDII source control projects were based on review of the following recent sewer rehabilitation projects:

Sausalito Marin City Sanitary District, Marin City Sewer Rehabilitation Project (2012). The project included a mix of open-cut replacement, pipe bursting, and CIPP for 11,000 feet of pipe ranging in diameter from 6 to 24 inches. The work included closed-circuit television (CCTV) inspections, spot repairs, sewer bypass, cleaning, mobilization, and re-connecting laterals. The total construction cost was approximately $1.2 million, and the unit cost was approximately $12.07 per diameter-inch per foot.

Union Sanitary District, Lower Hetch-Hetchy Corridor Sewer Rehabilitation Project (2008). The project included CIPP rehabilitation of 6,000 feet of 27-inch- and 30-inch-diameter sewer pipe. The total construction cost, including CCTV, sewer bypass, cleaning, and mobilization, was approximately $1.8 million. The unit cost was approximately $11.76 per diameter-inch per foot.

Sacramento Area Sewer District, Central Trunk Sewer Rehabilitation Project (2008). The project included 8 miles of large-diameter sewer rehabilitation using CIPP for pipe sizes ranging from 42 to 60 inches in diameter. The total construction cost was $22 million, and included CCTV, sewer bypass, cleaning, and mobilization. The unit cost was approximately $9.64 per diameter-inch per foot.

Based on the projects above, as well as other construction projects, unit costs for RDII source control can be estimated using the following:

- Manhole replacement at $10,000 each
- Manhole rehabilitation using epoxy linings at $6,000 each
- Lateral replacement using pipe bursting or open-cut at $7,500 each[7]
- Pipe rehabilitation at $12 per diameter-inch per foot.

The above construction costs include mobilization, CCTV inspection, sewer bypass, and site restoration. Although the new pipe construction costs presented in Section 6.4 require an additional 70% be added to account for other costs such as shoring, paving, and utility relocation, the unit costs presented above for RDII source control already include these costs and do not need to be adjusted.

6.10 Water Conservation

A water conservation program was assumed by DOU to have no capital improvement cost impacts. The program is designed to provide incentives for ratepayers to conserve water, and cost impacts would not be realized by the capital improvement program.

6.11 Contingencies to Estimate Project Capital Costs

A 40% total project estimating contingency was added to account for unknown conditions, mobilization, engineering, administration, inspection, and other costs. This contingency was

[7] This was changed from West Yost Associates' original estimate of $5,000 based on additional, recent AECOM experience in Columbus, Ohio.

added to the estimated construction cost to develop the estimated capital costs of the project for budgetary purposes.

6.12 Cost Summary for all proposed projects

The planning-level cost estimates are summarized in Table 6-1, and are to be used for comparison purposes only. Capital cost estimates for the CSSIP projects range from $510,000 to $21,030,00 (in 2013 dollars). Detailed information on the items included for each project is summarized in Appendix B.

The full initial GI program and estimate can be found in Appendix C. The RDII cost estimate can be found in Appendix D.

Table 6-1: Planning-Level Capital Cost Estimate Summary

Project #	Project Name	Total Estimated Capital Cost (Rounded ENR=9967)
WA1-1	Zapata Park	$10,020,000
WA1-2	G Street and 9th Street	$8,670,000
WA1-3	9th Street from G to L Street	$3,940,000
WA1-4	14th Street Storage	$4,490,000
WA1-5	N Street & 22nd Street	$2,580,000
WA1-6	24th Street Storage	$8,170,000
WA1-7	Grant Park	$20,580,000
WA2-1	M Street Storage	$11,359,000
WA2-2	McClatchy Park	$20,600,000
WA2-3	4th Avenue Park	$14,200,000
WA3-1	Bidwell & Freeport	$1,470,000
WA3-2	Land Park Tributary Area Upsizing	$7,550,000
WA3-3	Curtis Park Village Storage	$18,390,000
WA3-4	Sierra School Park	$12,570,000
WA3-5	Beverly Way	$2,260,000
WA3-6	17th and Bidwell Storage	$7,400,000
WA3-7	Target Parking	$8,970,000
WA4-1	McKinley Park Storage	$21,030,000
WA4-2	Sutter Middle School Storage	$19,340,000
WA4-3	P Street Storage	$3,560,000
WA4-4	Casita Way	$1,080,000
WA5-1	T Street or U Street at 20th	$670,000
WA5-2	28th and T/U Alley	$510,000
WA5-3	Storage under freeway at W and 25th	$12,390,000
WA6-1	Freeport Blvd. Upsizing	$2,860,000
WA6-2	Riverside Blvd.	$16,620,000
WA6-3	Curtis Park Storage	$15,400,000
WA6-4	11th Street Main Installation	$5,140,000
System	Existing System Optimization Projects	$1,390,000
Program 1	Green Infrastructure	$24,970,177
Program 2	RDII	$124,560,000
Program 3	Water Conservation	$0

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7. BENEFITS OF CSSIP PROJECTS

7.1 Flood-Reduction Benefits

Sections 5 and 6 described the performance benefits and costs of each of the 28 projects and three programs of the CSSIP individually; this section describes the system-wide benefits from the combined CSSIP projects and programs. Two CSSIP scenarios were developed, and their performances were evaluated for the 5-year and 10-year design storms:

Scenario 1: 28 Projects

- Projects 1-1 through 6-4
- Existing System Optimization Projects

Scenario 2: 28 Projects augmented by 3 programs

- Projects 1-1 through 6-4
- Existing System Optimization Projects
- Program 1: RDII
- Program 2: Green Infrastructure
- Program 3: Water Conservation

Table 7-1 and Table 7-2 present the performance of the two scenarios for 5-year and 10-year 6-hour design storms. The first scenario, consisting of the 28 projects, reduces flooding in wet areas and system-wide by 63% in 5-year storms, and by 56% in 10-year storms. The second scenario, consisting of the 28 projects and 3 programs, achieves additional flood reduction, primarily due to the RDII and GI programs. The combined 28 projects and 3 programs reduce flooding in the 5-year storm by approximately 74% in the wet areas and 71% system-wide. During the 10-year storm, this scenario reduces flooding by approximately 65% in the wet areas and 68% system-wide.

Table 7-1: Baseline vs. 28 Projects Results Summary for Design Storms

Extent	5-Year 6-Hour Design Storm		Percent Reduction (%)	10-Year 6-Hour Design Storm		Percent Reduction (%)
	Outflow Volume for Baseline (MG)	Outflow Volume with 28 Projects (MG)		Outflow Volume for Baseline (MG)	Outflow Volume with 28 Projects (MG)	
Outside Wet Area	9.1	3.3	64%	24.9	10.0	60%
Wet Area 1	2.6	0.7	73%	6.1	1.8	69%
Wet Area 2	2.3	1.5	33%	6.4	4.2	35%
Wet Area 3	2.5	0.5	80%	6.4	2.5	61%
Wet Area 4	0.4	0.1	75%	1.4	0.5	64%
Wet Area 5	1.0	0.4	60%	2.5	0.9	63%
Wet Area 6	1.0	0.4	60%	2.9	1.4	52%
Wet Areas	**9.8**	**3.6**	**63%**	**25.7**	**11.2**	**56%**

System-Wide	18.9	7.0	63%	50.6	21.2	58%

MG = million gallons

Table 7-2: Baseline vs. 28 Projects + Three Programs Results Summary for Design Storms

Extent	5-Year 6-Hour Design Storm		Percent Reduction (%)	10-Year 6-Hour Design Storm		Percent Reduction (%)
	Outflow Volume for Baseline (MG)	Outflow Volume with 28 Projects + 3 Programs (MG)		Outflow Volume for Baseline (MG)	Outflow Volume with 28 Projects + 3 Programs (MG)	
Outside Wet Area	9.1	2.6	71%	24.9	7.4	70%
Wet Area 1	2.6	0.6	77%	6.1	1.6	74%
Wet Area 2	2.3	1.0	56%	6.4	2.9	55%
Wet Area 3	2.5	0.4	84%	6.4	2.1	67%
Wet Area 4	0.4	0.1	75%	1.4	0.4	71%
Wet Area 5	1.0	0.4	60%	2.5	0.8	68%
Wet Area 6	1.0	0.3	70%	2.9	1.2	59%
Wet Areas	**9.8**	**2.5**	**74%**	**25.7**	**9.0**	**65%**
System-Wide	**18.9**	**5.5**	**71%**	**50.6**	**16.4**	**68%**

MG = million gallons

Figure 7-1 and Figure 7-2 present maximum flood depth on the land surface (using the model 2D mesh representation) for the 5-year and 10-year 6-hour design storms for the 28 projects. The six wet areas are identified on the maps. For the 2D results, the blue color indicates 2 to 6 inches of maximum flood depth, and red indicates greater than 6 inches of maximum flood depth.

Figure 7-3 and Figure 7-4 present maximum flood depth on the land surface for the 5-year and 10-year, 6-hour design storms for the 28 projects plus 3 programs. Appendix E presents a large exhibit-style map of the study area for the 5-year, 6-hour and 10-year, 6-hour design storms for the two scenarios.

Neither of the two scenarios fully eliminates flooding and outflows in the wet areas or system-wide for the 5-year and 10-year design storms. Based on the current model, additional projects to achieve a total reduction in flooding would be very costly. The 28 recommended projects and 3 programs provide a strong progression towards the final goal of minimizing flooding for a 10-year storm and preventing structure flooding for the 100-year storm. As these projects and programs are implemented their performance will be evaluated and the model will be continually refined. Future updates will incorporate a better understanding of actual performance as well as factor in changes to the City and the CSS. Furthermore, a future analysis can include a risk-based approach to evaluate any further need for CSSIP projects. The risk-based approach would use the model output of peak depth on the land surface and risk to the adjacent properties and human health and safety.

Figure 7-1: Results for 5-Year, 6-Hour Design Storm for 28 Projects

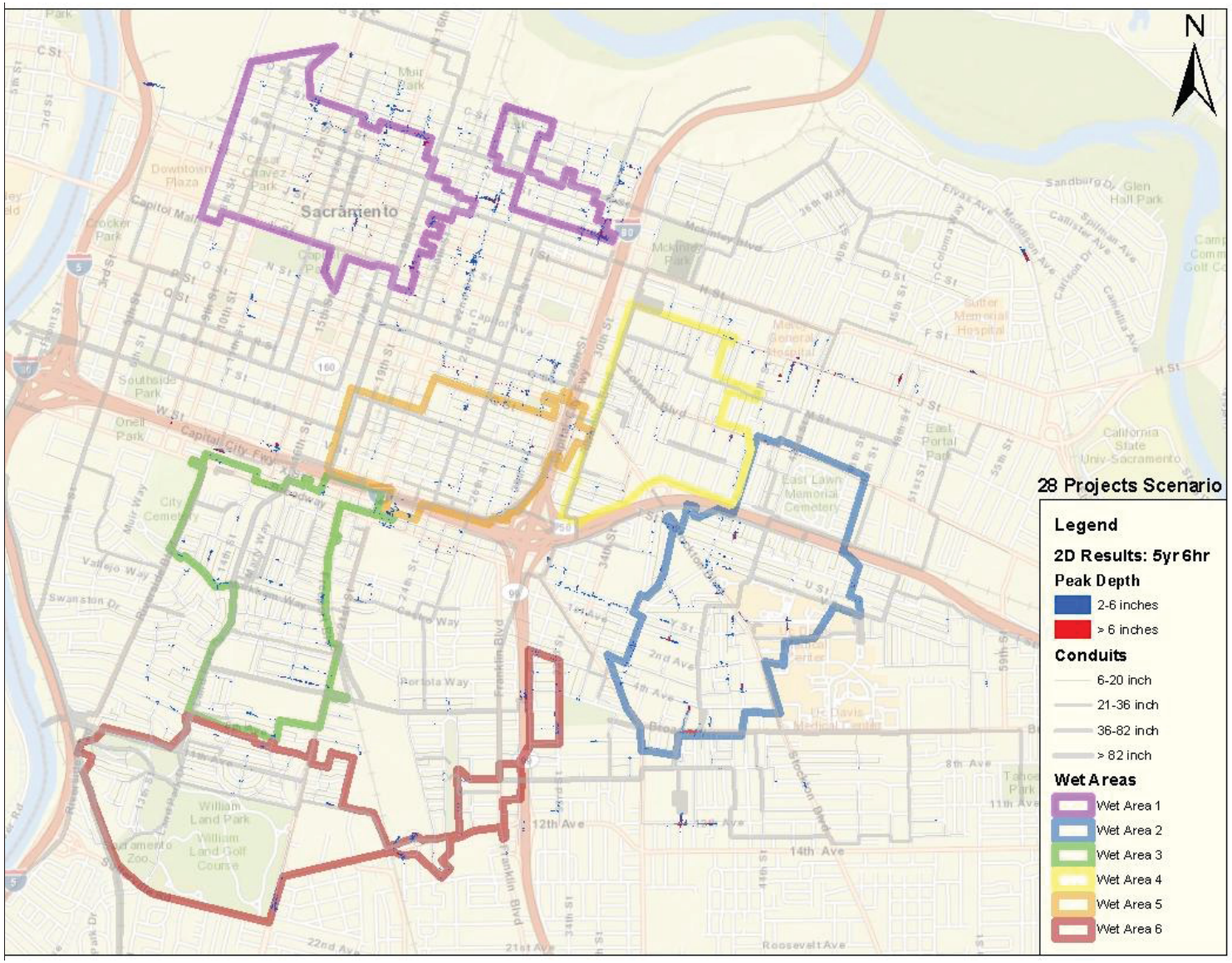


Figure 7-2: Results for 10-Year, 6-Hour Design Storm for 28 Projects

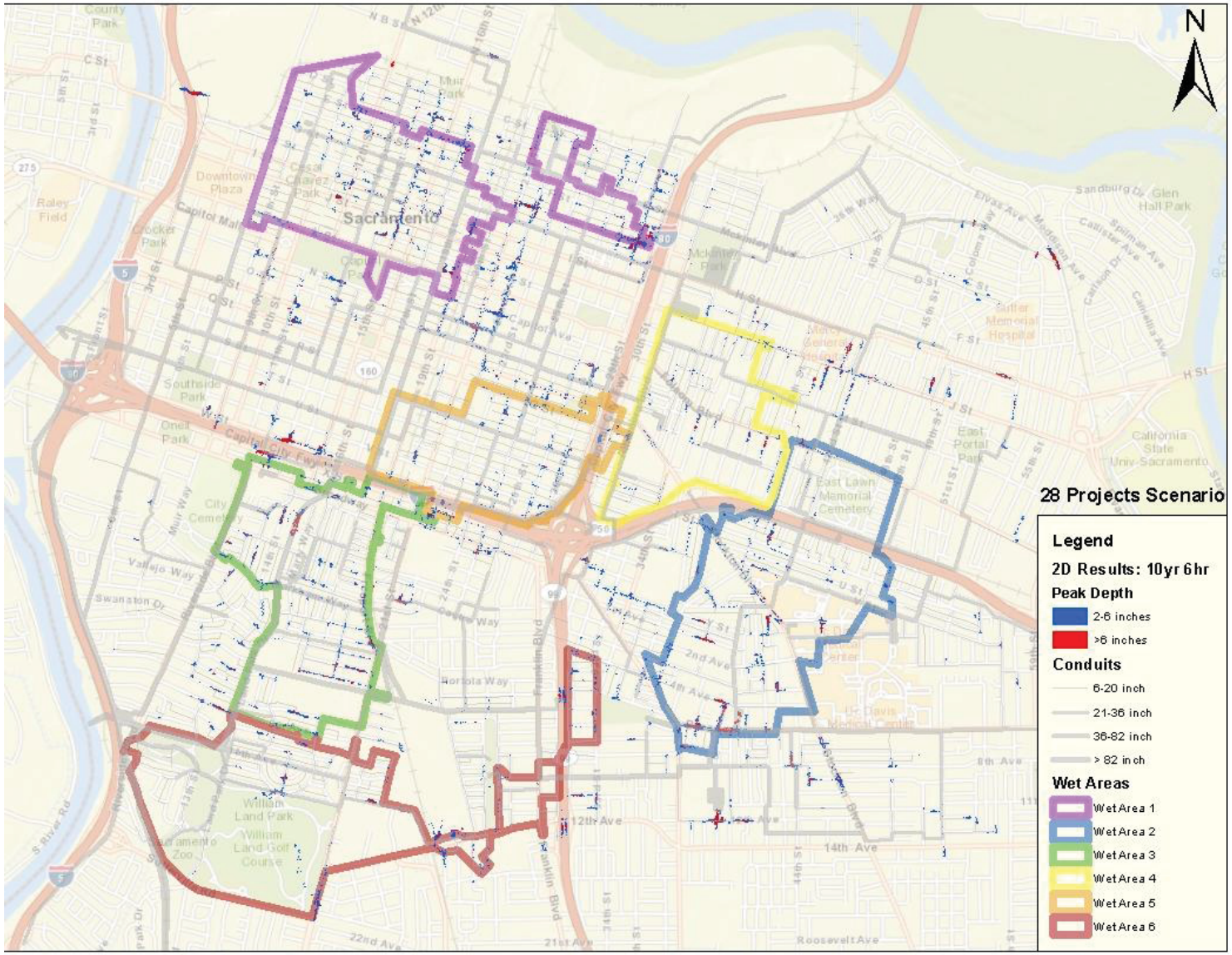


Figure 7-3: Results for 5-Year, 6-Hour Design Storm for 28 Projects and 3 Programs

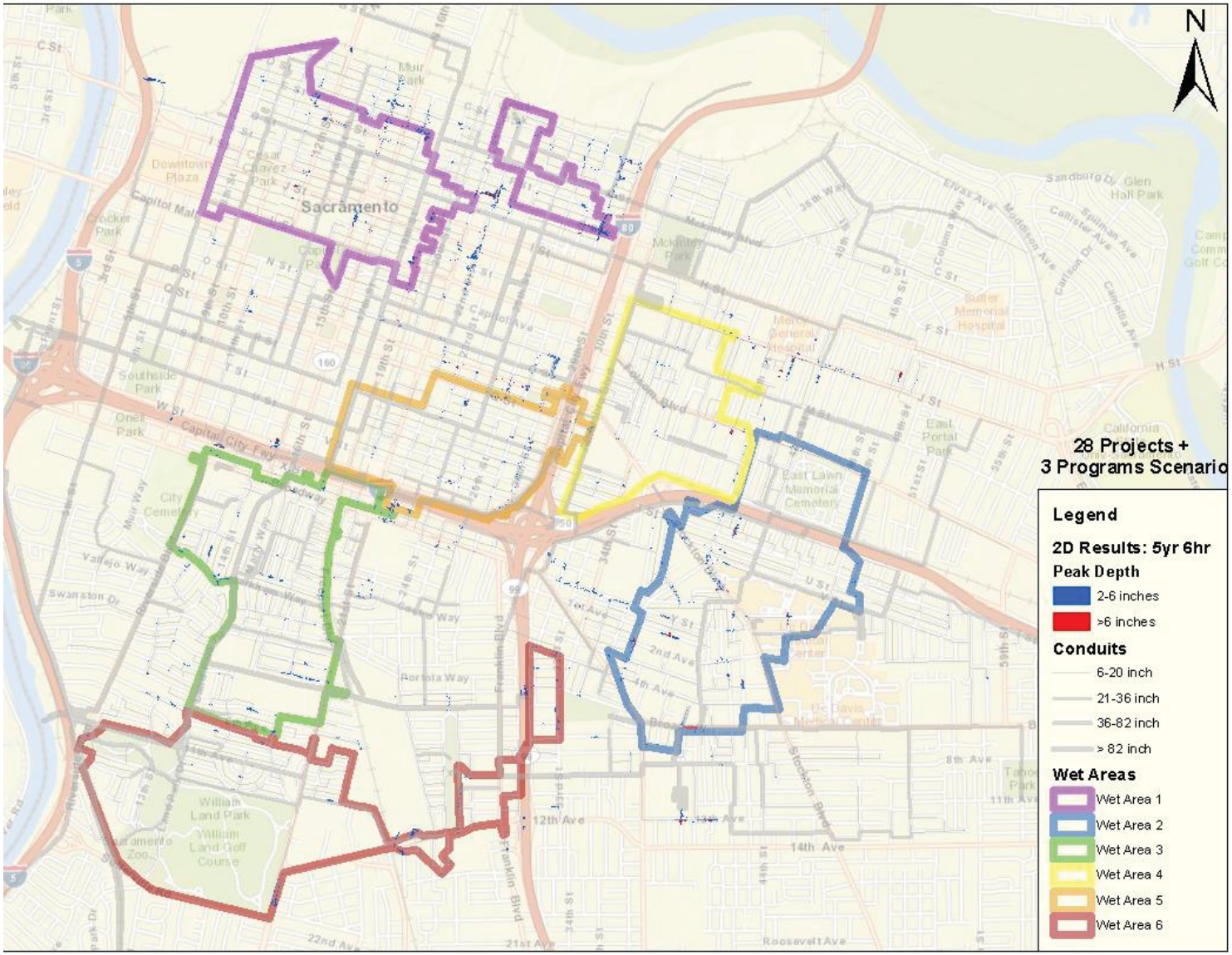
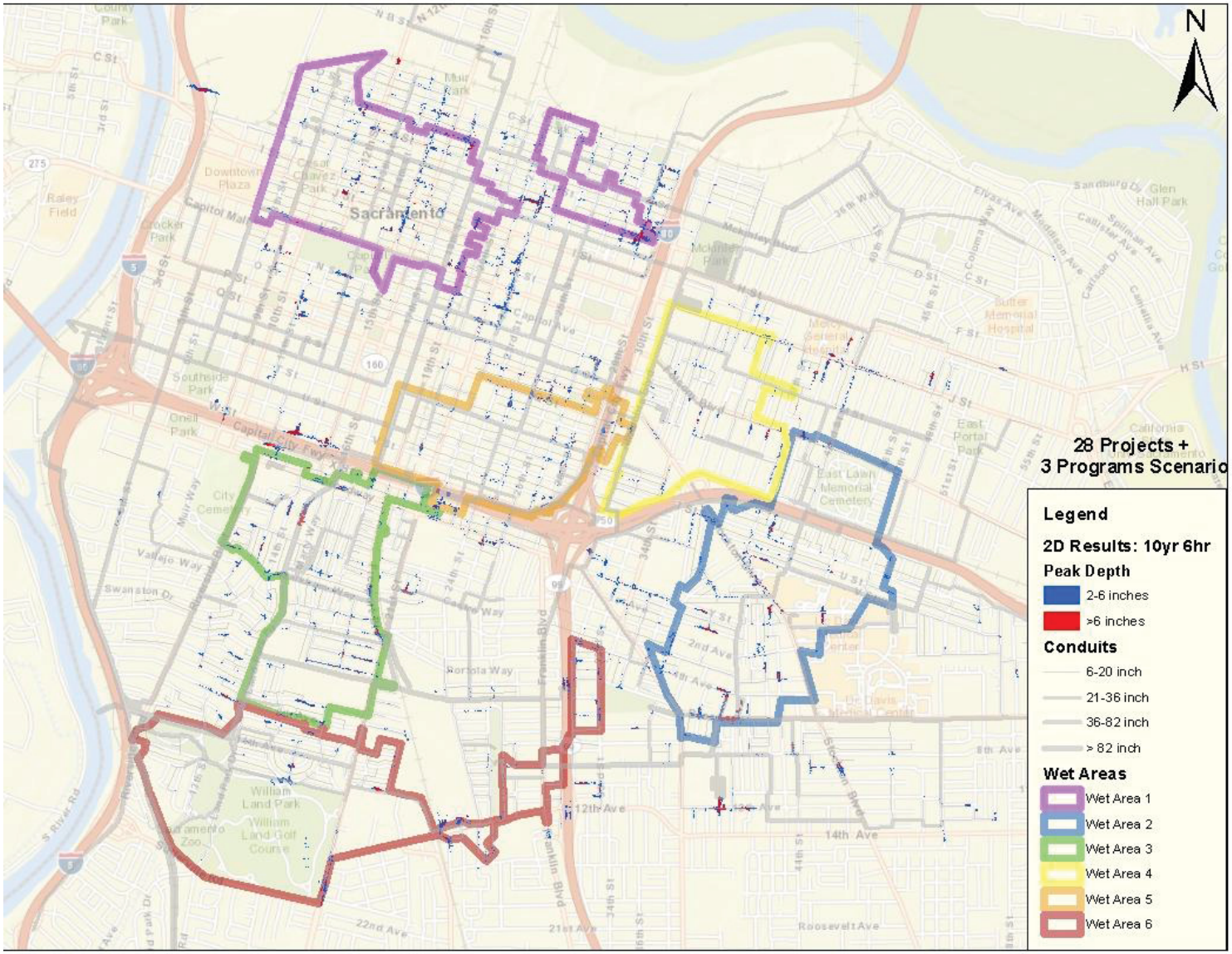


Figure 7-4: Results for 10-Year, 6-Hour Design Storm for 28 Projects and 3 Programs



7.2 Sacramento River Discharge Benefits

The CSSIP projects were targeted to address flooding in wet areas and system-wide. The projects can also contribute to reducing the volume and frequency of treated and untreated discharges to the Sacramento River across a range of storm event magnitudes. Furthermore, the three programs will reduce the total dry and wet weather flow entering the system, serving to reduce pumping and treatment costs (both at CWTP and SRWTP). A typical year rainfall dataset was simulated for the two final scenarios to evaluate the impact of CSSIP projects and programs. Table 7-3 presents the discharge frequency and volume at three outfalls to the Sacramento River and the total volume transmitted to the SRWTP for both baseline and the two scenarios.

Table 7-3: Typical Year Results

Location	Baseline		28 Projects		28 Projects + 3 Programs	
	Volume (MG)	Events	Volume (MG)	Events	Volume (MG)	Events
To SRWTP (Outfall 001)	6,775.5	-	6,771.9	-	6,731.3	-
CWTP (Outfall 002)	10.7	4	9.7	2	7.3	2
Sump 2 (Outfall 004)	0	0	0	0	0	0
Pioneer (Outfall 006)	225.3	6[2]	224.8	8[3]	184.0	6[4]
TOTAL	7,011	-	7,006	-	6,922	-

MG = million gallons; SRWTP = Sacramento Regional County Sanitation District Wastewater Treatment Plant

Notes:

(1) Outfalls 003, 005, and 007 are only used for emergency bypass

(2) Of the six discharge events at Outfall 006, three events received full equivalent primary treatment (peak flow rates <250 million gallons per day [mgd]) and three events received partial primary treatment (peak flow rates >250 mgd)

(3) Of the eight discharge events at Outfall 006, five events received full equivalent primary treatment (peak flow rates <250 million gallons per day [mgd]) and three events received partial primary treatment (peak flow rates >250 mgd)

(4) Of the six discharge events at Outfall 006, three events received full equivalent primary treatment (peak flow rates <250 million gallons per day [mgd]) and three events received partial primary treatment (peak flow rates >250 mgd)

For a typical year, the 28-project scenario slightly reduces the total volume of treated discharge to the river. The scenario reduces total volume of flow sent to CWTP and reduces the number of discharge events at the facility from four to two. The number of discharge events increases at Pioneer Reservoir from six to eight, but the total volume of discharge at

Pioneer decreases. The number of events increases because the conveyance projects enable higher peak flows to reach the facility. These higher flows previously contributed to surface flooding. The overall discharge volume decreases at Pioneer as a result of the greater volume of upstream detention storage provided by the 28 projects.

The 28 projects and three programs together reduce wet weather flows across a broad range of storm events. They also reduce dry weather flows via water conservation and RDII reduction (e.g., infiltration of irrigation flows). The number of discharge events and the total discharge volume to the river is reduced. The volume of flows sent to SRWTP for secondary treatment and discharge also decreases.

7.3 Summary of CSSIP Benefits

The 28 projects reduce flooding in the wet areas and system-wide for both design storms. In total, the 28 projects significantly reduce the outflow volume and depth of flooding on the surface.

The RDII, GI, and water conservation programs were modeled very generally to estimate the potential benefits of each. RDII was applied to separate sewersheds in the eastern part of the CSS because the eastern side is known to have a higher contribution of RDII flows to the CSS than the western separated area. RDII reduction technologies would likely yield a more cost-effective result in the eastern areas. GI and water conservation were applied system-wide, without targeting specific flooding areas. However, these programs can be targeted more specifically for flood reduction, as follows:

An RDII program can identify sources of high inflow and infiltration, such as storm water connections to the CSS in the separated areas, and leaky or cracked pipes and manholes. Flow monitoring can support identifying such areas, and allow for a planned RDII reduction program. Furthermore, defining sensitivity of flood reduction benefits from discrete areas would allow for prioritizing RDII reduction efforts within the separated areas, and would be more cost-beneficial. Overall, the RDII reduction program would be planned so that targeted flood reduction benefits to the CSS could be further refined through adaptive management.

The GI program would also be more effective if the spatial application of GI technologies is targeted toward flood reduction. Sensitivity analysis of different technologies and spatial placement of these technologies would support a strategic GI program. Furthermore, pilot programs for different technologies can be setup to monitor the performance and evaluate the benefits during design storms to further refine the application of GI. Refinement of the program to identify spatially effective locations and technologies would likely lead to improved benefits to the CSS.

The water conservation program was theoretically applied system-wide and demonstrated marginal benefits to flood reduction during the design storms. Although water conservation programs can be generally beneficial, they are typically not relevant for flood reduction.

8. CSSIP PROJECT SEQUENCING

Section 8 describes the methodology used to develop an implementation sequence for the CSSIP projects and programs. Implementation of CSSIP projects was prioritized based on several considerations. This section describes these considerations, provides quantitative metrics, and recommends weighting factors to represent importance. Unlike individual projects, the system-wide programs require additional information on specific technologies and performance prior to moving forward with application. Therefore, the sequencing of programs is structured differently than the projects to allow for pilot and early performance feedback to factor into the program progression.

8.1 Considerations for Developing and Prioritizing an Implementation Schedule

Considerations for prioritizing implementation of the CSSIP projects were developed through discussions with DOU staff. They are described below.

NPDES Permit – LTCP[8]: The interim goals described in the LTCP for the CSS are the primary drivers for project prioritization. Interim goals of the LTCP are as follows:

- Obtain protection from a 5-year storm in the six areas of worst flooding
- Obtain protection from a 5-year storm throughout the CSS area
- Obtain protection from a 10-year storm in the six areas of worst flooding
- Obtain protection from a 10-year storm event throughout the CSS

The final goals of the LTCP are to minimize street flooding during a 10-year storm event and prevent structure flooding during the 100-year storm event.

The CSS outflow reduction benefits of each project for the 5-year and 10-year design storms in the six wet areas and system-wide quantify each project's contribution to achieving LTCP goals. These CSS outflow reductions were used to prioritize projects. Projects with a larger CSS outflow reduction benefit received a higher priority.

Cost-Effectiveness: The cost-effectiveness of the projects to meet the system-wide CSS outflow reduction for 5- and 10-year design storms is a key consideration for prioritizing projects. Cost effectiveness is quantified by determining the unit volume of CSS outflow reduction achieved per unit construction cost. Projects with high cost-effectiveness for the 10-year storm and 5-year storm were assigned higher priority than projects with lower cost-effectiveness.

NPDES Permit[9]**- CSO Discharge:** As tabulated and described in Section 7, in association with a typical year rainfall, the CSSIP projects and/or programs do not cause any increase in

[8] http://www.waterboards.ca.gov/centralvalley/board_decisions/tentative_orders/1001/sacramento_css/2_sac_css_npdes.pdf (page 27)

[9] http://www.waterboards.ca.gov/centralvalley/board_decisions/tentative_orders/1001/sacramento_css/2_sac_css_npdes.pdf

untreated discharges to the Sacramento River and lead to a small decrease in treated discharge volume to the river. The CSSIP projects and programs are in compliance with the NPDES permit. However, because implementation of the projects will be sequenced, it is important as well to consider the effect of each individual project on the discharge to the Sacramento River. Simulation of the typical 1-year rainfall dataset is computationally intensive. Therefore, the largest event in a typical year rainfall dataset, simulated discretely, was used to evaluate the effect of each of the 28 CSSIP projects on the treated and/or untreated discharge volume to the Sacramento River as input to developing project prioritization.

Sewer Replacement: Several CSSIP projects include replacing an existing sewer pipe for increased conveyance or in-line storage. The age and condition of the existing sewer pipe were two considerations that factored into prioritizing CSSIP projects.

> **Age of Existing Sewer Pipes**: The age of an existing sewer pipe to be replaced in association with a CSSIP project was considered in project prioritization. For example, if a pipe in a proposed CSSIP project is at or is nearing the end of its useful life, the proposed project is assigned a higher priority. The age of the sewer pipe was calculated using installation-year information available in DOU's sewer geographic information systems (GIS) database.
>
> **Condition Assessment Drivers**: The asset management group at DOU conducts condition assessment of all large sewer pipes in the system and prepares plans to replace or rehabilitate them based on condition. The assessment includes several factors such as pipe age, pipe inspection reports, and consequence of the pipe's failure. The asset management group provided a priority ranking of the proposed CSSIP projects based on the condition and criticality of the asset involved. This information contributed to prioritizing the CSSIP projects. Asset management rankings were reflected by a numeric score of 1 or 2 for importance, with 1 for low, 2 for high, and non-priority projects receiving a score of 0.

Cost Sharing: Opportunities for cost sharing with other departments (Water, Streets, Park and Recreation) factored into prioritizing the CSSIP projects. DOU staff coordinated with the following three City departments to identify opportunities for cost-sharing:

> **Water:** The list of CSSIP projects was compared with Water Transmission Main Asset Management data and overlap with the CSSIP projects was identified. The CSSIP projects were also cross-referenced against the Water Department's on-going water meter replacement program. Overlaps between CSSIP projects and water mains scheduled for replacement as part of the water meter program were noted. Cost-sharing opportunities with Water Department projects were thus factored into CSSIP project prioritization.
>
> **Streets:** Combining street paving plans with sewer construction projects provides an opportunity to minimize public disturbance and provide cost savings synergies. DOU staff received a shapefile from the Streets Department for repaving projects that will start in 2015. Only a few CSSIP projects overlap with street repaving plans; these

were factored into project prioritization. Although projects may not have any significant overlap, the GI program can significantly leverage the street repaving plan for future cost sharing.

Parks and Recreation: DOU staff shared the list of CSSIP projects with the Parks and Recreation Department to identify any CSSIP storage projects within the footprint of parks slated for upgrade or major maintenance. DOU is awaiting input from Parks and Recreation on any cost-sharing opportunities for CSSIP storage projects.

City or Community Input: The CSSIP schedule also considers City and community input and concerns for prioritizing projects. The community may provide feedback on historical flooding or odor problems, recommend types of projects or synergy with other development ambitions, or provide political or economic reasons to improve the local community area. Community input quantification metrics consisted of a numeric score of 1, 2, or 3 for importance, with 1 for low, 2 for medium, and 3 for high importance.

8.2 Weighting Factors for Project Sequencing

The prioritizing considerations discussed above were all considered. Some were deemed more important than others.

Table 8-1 lists the considerations, an associated quantitative metric, and the relative importance for prioritizing projects. A weighting factor was developed for each based on discussions with DOU on the importance of the metric. For each project, a sum product of metrics and weighing factors was calculated. This sum product was the basis for ranking (Table 8-2) with implementation sequenced in descending priority.

Table 8-1: Weighing Factors for Prioritization Drivers

Prioritizing Considerations(s)	Importance	Metrics for Prioritizing	Weighing Factor (1–10)
NPDES Permit – CSS Outflows	Medium	Permit – CSS outflow Reduction in 5-Year Storm in all Six Wet Areas (MG)	3
		Permit – CSS outflow Reduction in 10-Year Storm in all Six Wet Areas (MG)	3
		Permit – CSS outflow Reduction in 5-Year Storm System-Wide (MG)	3
		Permit – CSS outflow Reduction in 10-Year Storm System-Wide (MG)	3
Cost-Effectiveness	High	Cost-Effectiveness of CSS outflow Reduction in 5-Year Storm System-Wide (G/$100)	8
		Cost-Effectiveness of CSS outflow Reduction in 10-Year Storm System-Wide (G/$100)	10
NPDES Permit – CSO Discharge	High	Increase in Untreated Discharge to the Sacramento River (Yes/No)	-8
Sewer Replacement	Medium	Sewer Age, Normalized (-)	3
		Asset Management for Replacement (2-High, 1-Low)	5
Cost Sharing	Low	Water Department (Yes/No)	2
		Streets Department (Yes/No)	2
		Parks and Recreation Dept. (Yes/No)	2
City/Community	High	City/Community/Neighborhood Interests (3-High, 2-Medium, 1-Low)	2

MG = million gallons
G/$ = gallons per $100
NPDES = National Pollutant Discharge Elimination System

Table 8-2: Project Prioritization

Project Name		Considerations for Project Scheduling													Total Weight
		NPDES Permit - CSS Outflow Reduction				Cost-Effectiveness		NPDES Permit - CSO Discharge	Sewer Replacement		Cost Sharing			City/ Community Input (3-High, 2-Medium, 1-Low)	
		In 5-Year Storm in all Six Wet Areas (MG)	In 10-Year Storm in all Six Wet Areas (MG)	In 5-Year Storm System-Wide (MG)	In 10-Year Storm System-Wide (MG)	In 5-Year Storm System-wide (G/$100)	In 10-Year Storm System-wide (G/$100)	Increase in Treated Discharge to the Sac. River (Yes/No)[1]	Sewer Age, Normalized	Asset Management for Replacement (2-High, 1-Low)	Water Dept. (Yes/No)	Streets Dept. (Yes/No)	Parks and Recreation Dept. (Yes/No)		
	Weighing Factor (1-10)	3	3	3	3	8	10	-8	3	5	2	2	2	2	-
	Cost (2013 $)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1-1 Zapata Park	$10,020,000	0.21	0.22	0.22	0.23	2.23	2.27	0.0	1.1	1	0	0	0	0	51
1-2 G Street and 9th Street	$8,670,000	0.23	0.24	0.23	0.23	2.65	2.68	0.0	0.0	0	0	0	0	0	51
1-3 9th Street from G to L Streets	$3,940,000	0.18	0.15	0.30	0.22	7.72	5.46	0.0	0.2	0	1	0	0	3	128
1-4 14th Street Storage	$4,490,000	0.15	0.28	0.19	0.35	4.16	7.89	0.0	0.0	0	1	1	0	0	119
1-5 N & 22nd Street Upsizing	$2,580,000	-0.03	0.09	-0.07	0.03	-2.69	1.25	0.0	0.2	0	0	0	0	0	-8
1-6 24th Street Storage	$8,170,000	0.18	0.71	0.21	0.83	2.52	10.14	0.0	1.1	0	0	0	0	0	131
1-7 Grant Park	$20,580,000	0.23	1.01	0.42	1.51	2.03	7.36	0.0	1.0	1	1	0	1	0	111
2-1 M Street Storage	$11,359,000	0.06	0.25	0.20	0.64	1.75	5.61	0.0	1.0	1	1	0	0	0	84
2-2 McClatchy Park Storage	$20,600,000	0.07	0.13	0.38	0.50	1.83	2.41	0.0	1.0	2	1	1	2	0	63
2-3 4th Avenue Park Storage	$14,200,000	0.16	0.17	0.63	0.59	4.47	4.13	0.0	1.0	0	0	1	1	0	89
3-1 Bidwell & Freeport	$1,470,000	0.18	0.21	0.20	0.20	13.30	13.68	0.0	0.0	0	1	0	0	0	248
3-2 Land Park Tributary Area Upsizing	$7,550,000	0.08	0.09	0.14	0.10	1.90	1.34	0.0	0.9	2	1	1	0	0	46
3-3 Curtis Park Village Storage	$18,390,000	0.23	0.55	0.40	0.88	2.15	4.79	0.0	1.0	0	1	0	0	1	78
3-4 Sierra School Park	$12,570,000	0.13	0.61	0.23	0.79	1.79	6.29	0.0	1.0	1	1	0	1	0	95

Project Name		Considerations for Project Scheduling													Total Weight
		NPDES Permit - CSS Outflow Reduction				Cost-Effectiveness		NPDES Permit - CSO Discharge	Sewer Replacement		Cost Sharing			City/ Community Input (3-High, 2-Medium, 1-Low)	
		In 5-Year Storm in all Six Wet Areas (MG)	In 10-Year Storm in all Six Wet Areas (MG)	In 5-Year Storm System-Wide (MG)	In 10-Year Storm System-Wide (MG)	In 5-Year Storm System-wide (G/$100)	In 10-Year Storm System-wide (G/$100)	Increase in Treated Discharge to the Sac. River (Yes/No)[1]	Sewer Age, Normalized	Asset Management for Replacement (2-High, 1-Low)	Water Dept. (Yes/No)	Streets Dept. (Yes/No)	Parks and Recreation Dept. (Yes/No)		
	Weighing Factor (1-10)	3	3	3	3	8	10	-8	3	5	2	2	2	2	-
	Cost (2013 $)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
3-5 Beverly Way Storage	$2,260,000	0.07	0.08	0.07	0.10	2.90	4.22	0.0	0.0	0	1	0	0	0	68
3-6 17th and Bidwell Storage	$7,400,000	0.07	0.08	0.16	0.16	2.15	2.22	0.0	0.0	0	1	0	0	0	43
3-7 Target Parking Storage	$8,970,000	0.01	0.03	0.05	0.10	0.54	1.11	0.0	0.0	0	0	0	0	0	16
4-1 McKinley Park Storage	$21,030,000	0.09	1.35	0.15	2.55	0.70	12.14	0.0	0.0	0	1	0	2	3	151
4-2 Sutter Middle School Storage	$19,340,000	0.33	1.65	0.55	2.51	2.83	12.99	0.0	0.0	0	1	0	0	0	170
4-3 P Street Storage	$3,560,000	0.02	0.05	0.06	0.07	1.66	2.04	0.0	0.0	0	1	0	0	0	36
4-4 Casita Way	$1,080,000	0.02	0.02	0.04	0.05	3.62	4.34	0.0	0.0	0	1	0	0	0	75
5-1 T Street and 20th	$670,000	0.00	-0.03	0.00	-0.03	0.43	-4.04	0.0	0.0	0	0	0	0	3	-31
5-2 T Street and 28th	$510,000	0.03	-0.02	0.06	-0.06	11.30	-11.84	0.0	1.0	1	0	1	0	0	-18
5-3 W and 25th Street Storage	$12,390,000	0.23	0.37	0.69	0.97	5.53	7.85	0.0	1.0	2	0	0	0	0	143
6-1 Freeport Blvd. Upsizing	$2,860,000	0.06	0.05	0.10	0.01	3.35	0.26	0.0	0.9	2	1	0	0	2	49
6-2 Riverside Blvd. Upsizing	$16,620,000	0.07	0.11	0.17	0.50	1.04	3.00	0.0	1.0	2	1	0	0	0	56
6-3 Curtis Park Storage	$15,400,000	0.15	0.19	0.35	0.53	2.29	3.47	0.0	0.0	0	1	1	0	0	61
6-4 11th Street Installation	$5,140,000	0.06	0.07	0.12	0.11	2.25	2.06	0.0	0.0	0	1	1	0	0	44

Project Name		Considerations for Project Scheduling													Total Weight
		NPDES Permit - CSS Outflow Reduction				Cost-Effectiveness		NPDES Permit - CSO Discharge	Sewer Replacement		Cost Sharing			City/ Community Input (3-High, 2-Medium, 1-Low)	
		In 5-Year Storm in all Six Wet Areas (MG)	In 10-Year Storm in all Six Wet Areas (MG)	In 5-Year Storm System-Wide (MG)	In 10-Year Storm System-Wide (MG)	In 5-Year Storm System-wide (G/$100)	In 10-Year Storm System-wide (G/$100)	Increase in Treated Discharge to the Sac. River (Yes/No)[1]	Sewer Age, Normalized	Asset Management for Replacement (2-High, 1-Low)	Water Dept. (Yes/No)	Streets Dept. (Yes/No)	Parks and Recreation Dept. (Yes/No)		
	Weighing Factor (1-10)	3	3	3	3	8	10	-8	3	5	2	2	2	2	-
	Cost (2013 $)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Existing System Optimization Project	$1,390,000	0.14	0.33	0.29	0.61	20.52	43.59	0.0	0.0	0	0	0	0	0	604
RDII Program	$124,560,000	0.18	0.61	0.64	1.50	0.51	1.20	0.0	0.0	0	0	0	0	0	25
GI Program	$24,970,177	0.45	0.99	0.89	1.88	3.57	7.52	0.0	0.0	0	0	0	0	0	116
Water Conservation Program	$0	0.06	0.16	0.15	0.37	100.00	100.00	0.0	0.0	0	0	0	0	0	1802

NA - Not applicable as the project may be parcel storage, or a new pipe serving as conveyance or linear storage
*Certain projects have an increase in CSS outflow (for 5-year or 10-year design storm) when implemented independently and therefore have a negative weighted score. These projects are a lower priority and when constructed after other higher priority CSSIP projects will provide a decrease in CSS outflow and system-wide flooding benefit.

Color Key for Model Results for 10-Year Storm	
	High CSS Outflow Reduction
	Moderate CSS Outflow Reduction
	Low CSS Outflow Reduction

MG = millions of gallons
G/$ = gallons per $100
NPDES = National Pollutant Discharge Elimination System

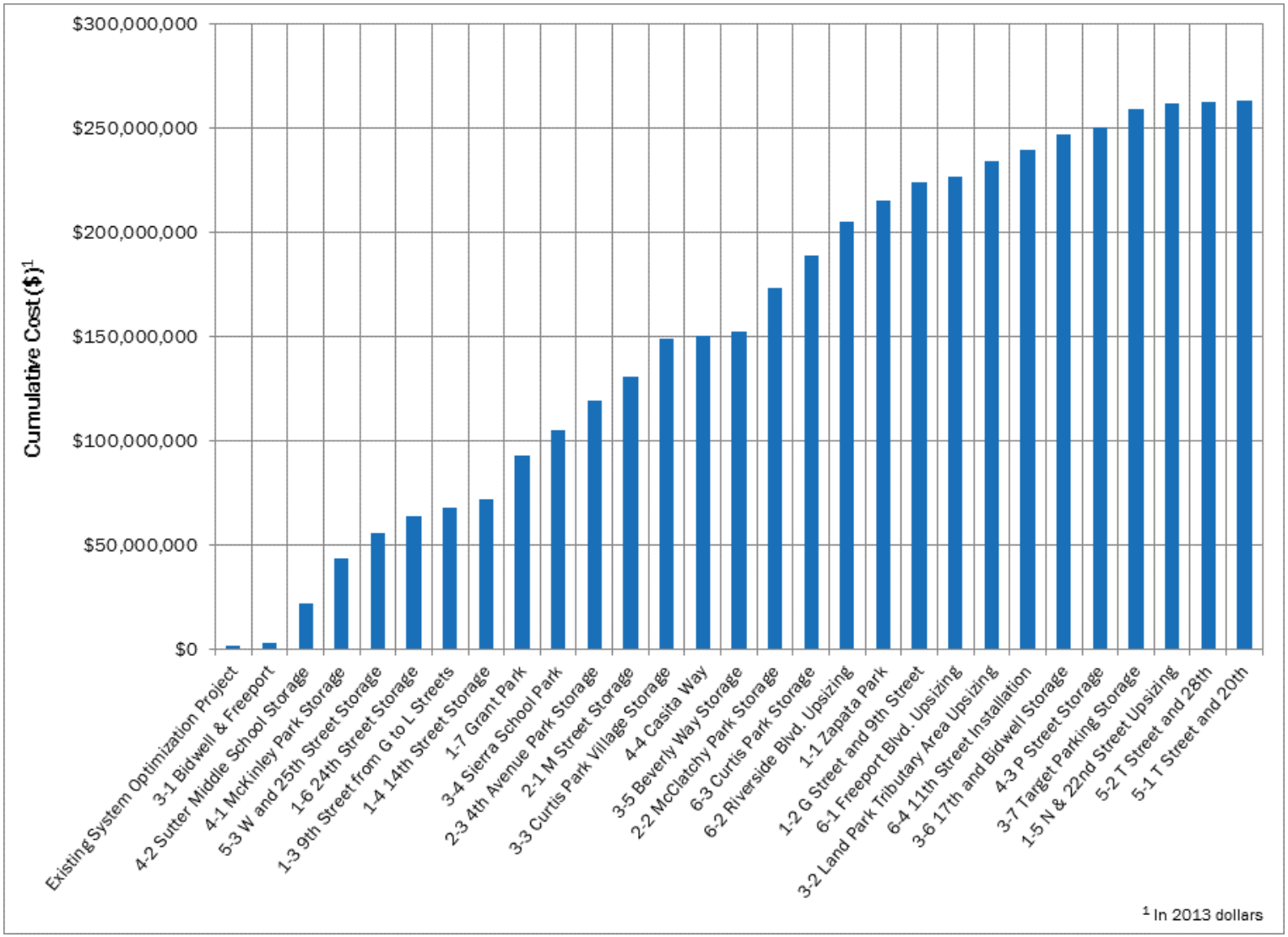
The projects as prioritized for implementation are listed in Table 8-3 (based on the prioritization approach described above and listed from high to low order of the total weighted factor). Figure 8-1 presents the implementation sequence for the 28 CSSIP projects. The y-axis in the figure presents the total cumulative cost[10] to implement all 28 CSSIP projects.

Table 8-3: CSSIP Projects Prioritization Ranking

Schedule	Project Name	Cost ($)[10]	Total Weighted Factor
	Existing System Optimization Project	$ 1,390,000	604
	3-1 Bidwell & Freeport	$ 1,470,000	248
	4-2 Sutter Middle School Storage	$ 19,340,000	170
	4-1 McKinley Park Storage	$ 21,030,000	151
	5-3 W and 25th Street Storage	$ 12,390,000	143
	1-6 24th Street Storage	$ 8,170,000	131
	1-3 9th Street from G to L Streets	$ 3,940,000	128
	1-4 14th Street Storage	$ 4,490,000	119
	1-7 Grant Park	$ 20,580,000	111
	3-4 Sierra School Park	$ 12,570,000	95
	2-3 4th Avenue Park Storage	$ 14,200,000	89
	2-1 M Street Storage	$ 11,359,000	84
	3-3 Curtis Park Village Storage	$ 18,390,000	78
	4-4 Casita Way	$ 1,080,000	75
	3-5 Beverly Way Storage	$ 2,260,000	68
	2-2 McClatchy Park Storage	$ 20,600,000	63
	6-3 Curtis Park Storage	$ 15,400,000	61
	6-2 Riverside Blvd. Upsizing	$ 16,620,000	56
	1-1 Zapata Park	$ 10,020,000	51
	1-2 G Street and 9th Street	$ 8,670,000	51
	6-1 Freeport Blvd. Upsizing	$ 2,860,000	49
	3-2 Land Park Tributary Area Upsizing	$ 7,550,000	46
	6-4 11th Street Installation	$ 5,140,000	44
	3-6 17th and Bidwell Storage	$ 7,400,000	43
	4-3 P Street Storage	$ 3,560,000	36
	3-7 Target Parking Storage	$ 8,970,000	16
	1-5 N & 22nd Street Upsizing	$ 2,580,000	-8
	5-2 T Street and 28th	$ 510,000	-18
	5-1 T Street and 20th	$ 670,000	-31

[10] 2013 dollars

Figure 8-1: CSSIP Projects Implementation Sequence and Cumulative Costs



8.3 CSSIP Programs Schedule

The three considered CSSIP programs provide system-wide benefits. Water conservation and GI can be applied throughout the CSS. RDII reduction can be applied in the separate sewer areas. Each program covers a different set of technologies with limited data on their performance. Therefore, the programs require different implementation strategies. An implementation strategy for each is proposed below.

Water Conservation: There is no cost estimate for the DOU sewer fund to implement this program, but a City-wide public outreach and education initiative under the DOU water fund could spur residents to conserve water and reduce sanitary flows to the CSS. Water conservation initiatives are currently underway by the DOU water department, and could be ramped up at the beginning of CSSIP implementation, and continue until water conservation goals are achieved.

RDII: The RDII program would be applicable only in the separated areas of the CSS. Because the eastern part of the City's separated sewer service area contributes more RDII flows than the western separated area, the RDII program is only applied there. A programmatic application of the RDII reduction and source-control program would require developing pilot studies to include monitoring and performance benchmarking before full-scale implementation. The proposed implementation schedule recommends an annual implementation rate of 1% of total RDII cost for the first 5 years, 2% for years 6 through 10, and a ramp-up on implementation after that. Table 8-4 presents a potential implementation rate for the RDII program. The 100% costs assume full rehabilitation or replacement of every pipe, lateral, and manhole. It is very likely that the pilot implementation would identify a prioritization scheme such that acceptable reductions can be achieved without full expenditure. Therefore, the 100% costs should be considered top of the scale book-end numbers.

Green Infrastructure: Planning-level analysis presented in this CSSIP show that cost-benefit of GI is competitive with Grey to provide flood reduction benefits. Additionally, GI would reduce the volume of treated discharges to the river. Future studies will further refine the modeling and site selection process for GI to select potential pilot programs. The GI program would be applicable in the entire CSS, but GI technologies to be applied would vary and would need to be customized for soil properties, surface slope, community interest, and other issues. The City has limited experience in constructing and maintaining such technologies, so a pilot program would occur before full-scale implementation of the program. The pilot program would allow for testing all viable technologies at different locations within the City's service area and developing performance benchmarks based on the pilot program.

This would allow for strategic City-wide implementation after the pilot program. The proposed implementation schedule recommends an annual implementation rate of 1% of total GI cost for the first 5 years, 2% for years 6 through 10, and a ramp-up on implementation after that. Table 8-4 presents the costs of implementation of the RDII and GI programs. Similar to RDII piloting, GI piloting would provide information that

would guide implementation such that acceptable source flow reductions can be achieved without full expenditure of the 100% cost estimate.

Figure 8-2 presents the total cumulative upper book end cost for implementation of the RDII and GI programs. Both these total cumulative costs are expected to be lower once piloting results are utilized to fine-tune the implementation. Figure 8-3 presents the total cost for all of the CSSIP projects and programs.

Table 8-4: RDII and Green Infrastructure Implementation

Schedule	Percentage of Total Cost	Annual Implementation Costs[11]	
		Green Infrastructure	RDII
Year 1	1.0%	$250,000.0	$1,250,000
Year 2	1.0%	$250,000.0	$1,250,000
Year 3	1.0%	$250,000.0	$1,250,000
Year 4	1.0%	$250,000.0	$1,250,000
Year 5	1.0%	$250,000.0	$1,250,000
Year 6	2.0%	$500,000.0	$2,500,000
Year 7	2.0%	$500,000.0	$2,500,000
Year 8	2.0%	$500,000.0	$2,500,000
Year 9	2.0%	$500,000.0	$2,500,000
Year 10	2.0%	$500,000.0	$2,500,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	4.0%	$1,000,000.0	$5,000,000
	5.0%	$1,250,000.0	$6,250,000
	5.0%	$1,250,000.0	$6,250,000
	5.0%	$1,250,000.0	$6,250,000
	5.0%	$1,250,000.0	$6,250,000
	5.0%	$1,250,000.0	$6,250,000
	5.0%	$1,250,000.0	$6,250,000
	5.0%	$1,250,000.0	$6,250,000
	5.0%	$1,250,000.0	$6,250,000
	5.0%	$1,250,000.0	$6,250,000
TOTAL	100%	$25,000,000	$125,000,000

Note: Total costs assume (a) full rehabilitation and replacement of pipes, laterals, and manholes in the eastern separate sewer areas; and, (b) full implementation of GI.

[11] 2013 dollars

Figure 8-2: CSSIP Program Implementation Schedule

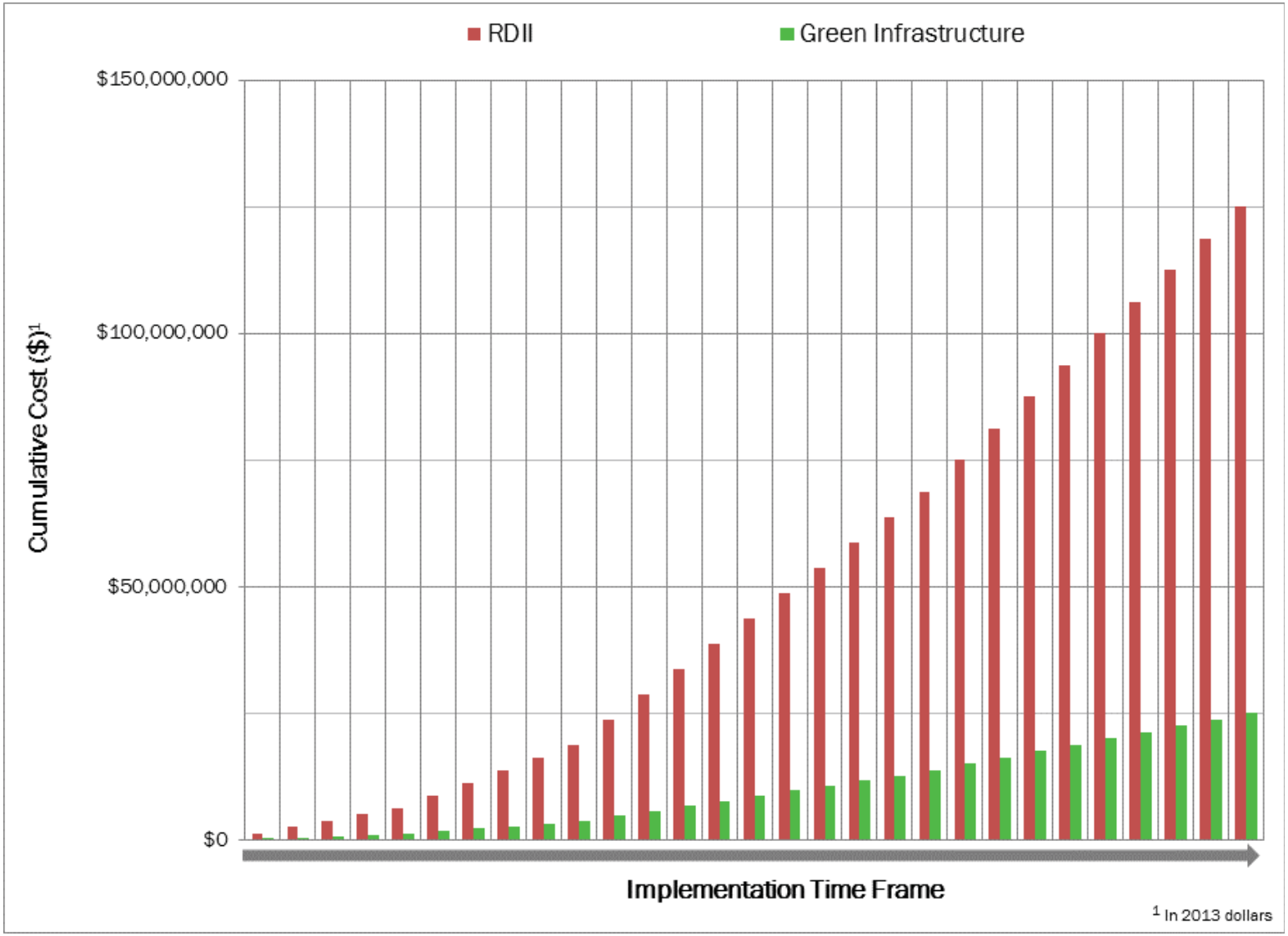
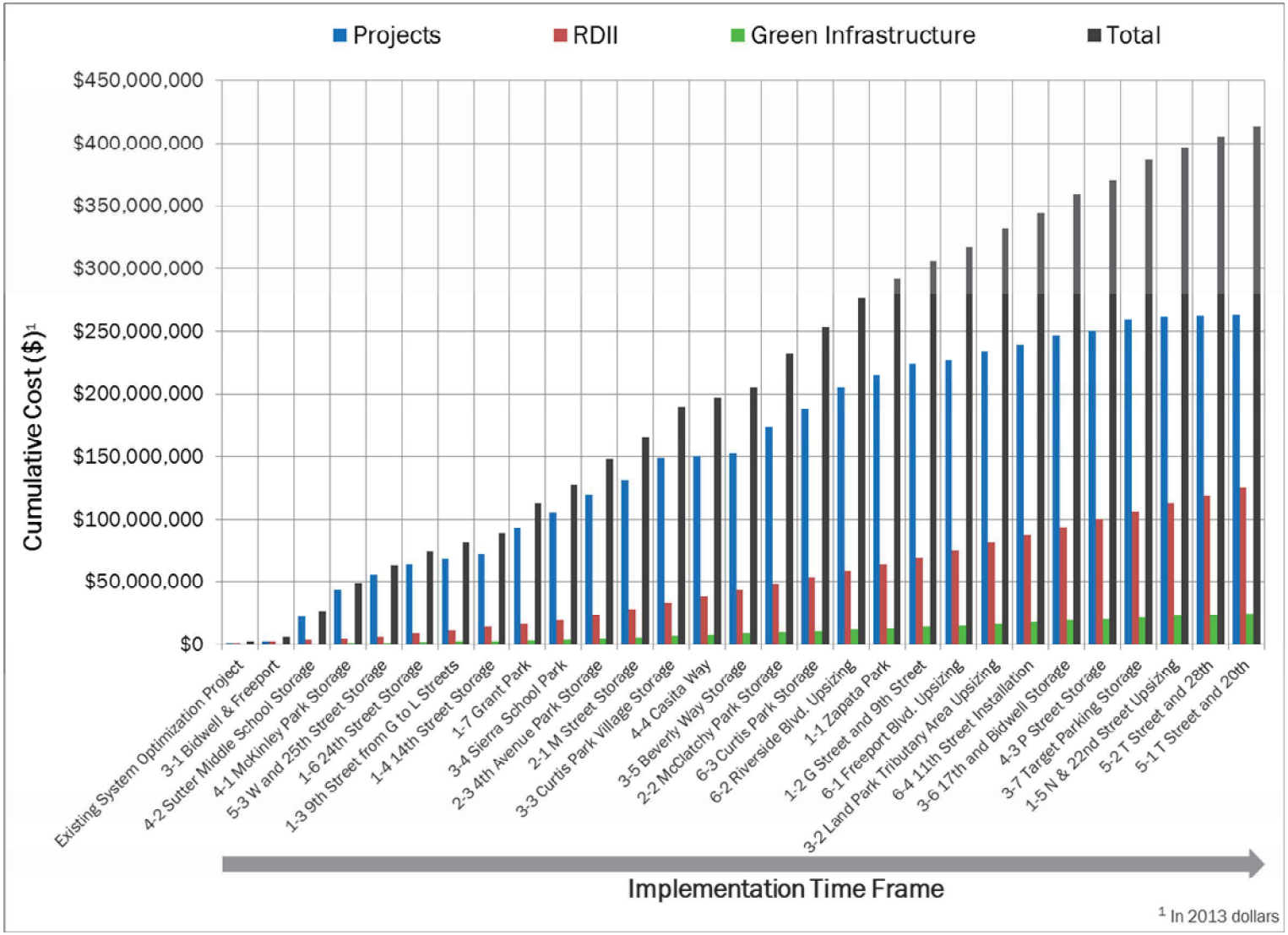


Figure 8-3: CSSIP Projects and Programs Implementation Schedule



9. CONCLUSION

This section summarizes the findings of the CSSIP Update, and provides the recommendations for implementation.

The analyses used to develop this CSSIP Update relied on a powerful H&H simulation tool for the collection system that helped to identify deficiencies and develop solutions to eliminate or reduce flooding challenges. The H&H model used to study the CSS was validated using monitoring data from 2008 and 2009. Model simulation results for various historical rainfall events correlate well with anecdotal observations by DOU staff, thus providing good confidence in the model's ability to aid in identifying flooding problems and potential solutions to address them. The H&H model is a planning-level tool and is subject to some inherent uncertainties. For example, the datasets (e.g., ground models, building boundaries, sub-catchment properties) used by the model are only accurate to within certain confidence levels, and may not be appropriate for more detailed design-level purposes.

Prior to developing projects and programs that comprise the CSSIP Update, the CSS was evaluated using the H&H model for different storm events to create a baseline of the current system's performance for regulatory requirements. The two regulatory-driven requirements are Sacramento River discharges and in-system flooding.

During wet-weather events, CSS flows generated in the collection system can be discharged to the Sacramento River through seven permitted outfalls, numbered 001 through 007. Outfall 001 discharges SRWTP effluent that has received secondary treatment. Discharges from CWTP and Pioneer Reservoir (Outfalls 002 and 006) have received equivalent primary treatment and disinfection. Discharges from Sump 2 (Outfall 004) are untreated. Outfalls 003, 005, and 007 discharge untreated flow only in emergency bypass situations. A "typical" average annual rainfall dataset was simulated by the H&H model to create a baseline of the existing system and characterize the discharge frequency and volume at each outfall. The analysis shows that all discharges to the Sacramento River are treated to, at a minimum, partial primary clarification or equivalent, solids and floatables disposal, and disinfection. There are no untreated discharges to the river for the "typical" average annual rainfall. Therefore, the existing CSS is in compliance with the NPDES permit discharge requirements, and the CSSIP does not include projects to specifically address discharges to the Sacramento River.

The second permit-addressed challenge is in-system flooding. During larger storm events, wet-weather CSS flows can exceed the storage and conveyance capacity of the system, resulting in surface flooding. The flooding can be caused either by combined sewage exiting the CSS as outflows, or surface storm water runoff unable to enter the collection system because it is already at capacity. The NPDES permit references six known areas particularly prone to flooding during larger storm events. The 5-year and 10-year return period design storms were simulated using the H&H model to create a baseline of the existing system for these conditions, and to identify outflow and flooding challenges within the six wet areas and system-wide. The model results for the baseline matched DOU's observations of historical flooding fairly well, and, thus, provided confidence in the model's representation of

challenges throughout the system. The CSSIP Update describes and prioritizes projects and programs that will progress the City toward the NPDES permit's ultimate LTCP goals for flood protection.

Using the H&H model, the collection system was simulated for the 5-year and 10-year recurrence interval design storms. Flooding areas were identified and validated by DOU staff. To address each flooding area identified in the H&H model for the design storms, different types of projects were identified that would either increase the conveyance capacity downstream of the flooding area or provide localized storage for the excess flows. The storage projects were developed as either parcel storage underneath an open park or vacant lot, or a linear storage within the City's right-of-way. Project types and sizes were developed to be most performance and cost-effective. The H&H model was used to ensure that no recommended projects, individual or in combination, increased untreated discharges to the Sacramento River during a typical rainfall year.

In addition to capital projects, system-wide programs were identified to augment the identified projects and further reduce flooding by reducing the sanitary flows and storm water runoff entering the collection system. The programs consist of localized GI projects to detain or retain storm water in the combined areas, RDII source controls to reduce wet-weather flows in the separated sanitary areas, and Water Conservation to reduce sanitary flows from the entire service area. The programs also provide some benefit (5–9%) in reducing treated CSS discharges during the typical rainfall year. Figure 9-1 presents the projects and programs that were developed and recommended as part of this CSSIP Update.

The CSSIP Update-recommended projects were prioritized based on several considerations: flood-reduction benefits, cost-effectiveness, sewer condition/age, cost-sharing opportunities, and City/community interests. The programs were identified and applied uniformly across the system, with early pilot programs to help inform and refine implementation in an adaptive manner. RDII program was applied only to the eastern separate sewer area.

The projects and programs recommended in this CSSIP Update do not completely eliminate total system flooding. The quantitative metric used to measure flooding, CSS outflows, decreased significantly (65%) as a result of the comprehensive set of projects and programs. The cost to completely address remnant flooding would be substantial. Instead, the CSSIP Update recommends an adaptive management strategy as part of CSSIP Update implementation.

Figure 9-1: CSSIP Recommendations

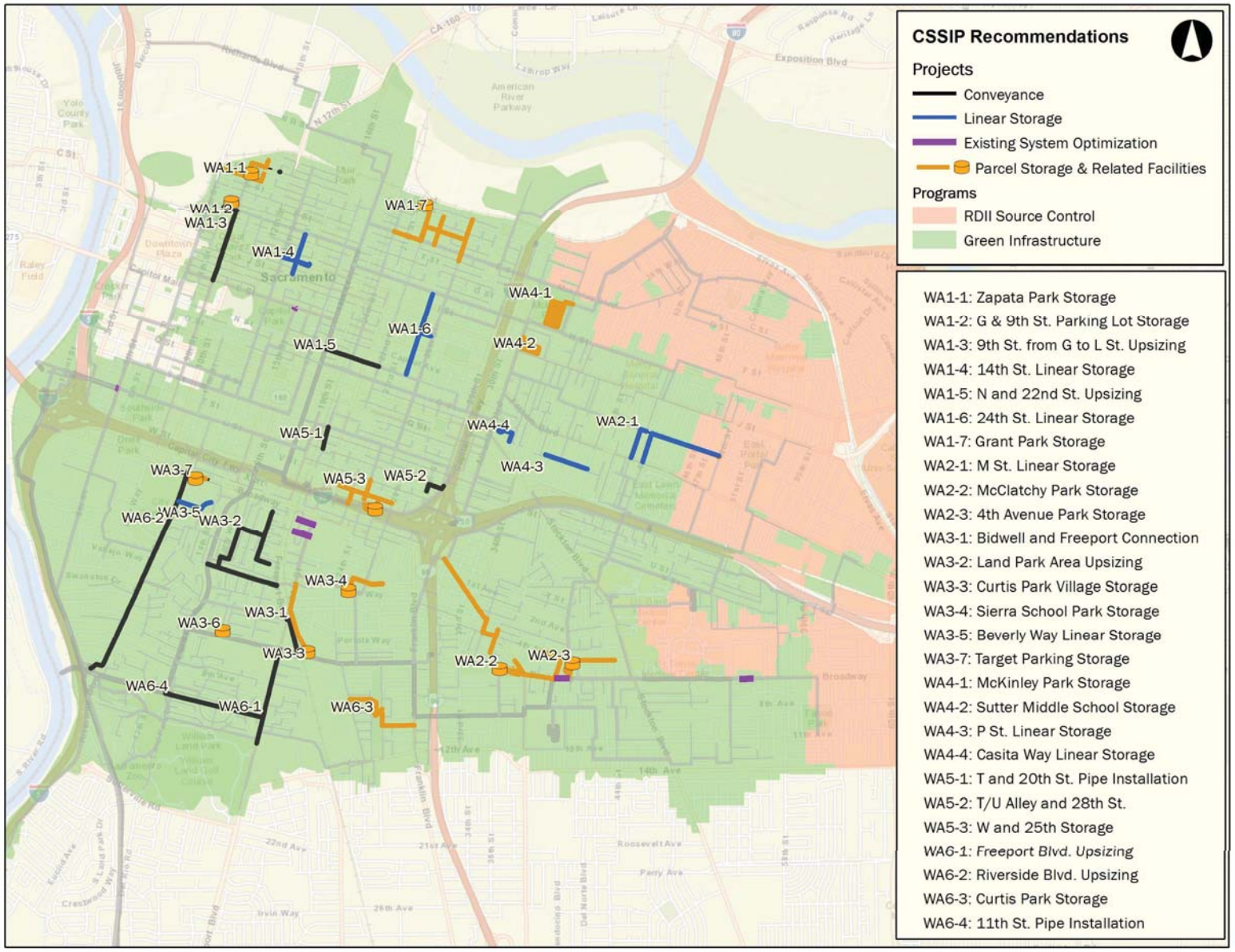


Figure 9-2 illustrates the benefits of using an adaptive management strategy in association with implementing the projects and programs recommended in this CSSIP and addressing remnant flooding. The solid red line represents an estimate of the total implementation cost for all projects and programs recommended in this CSSIP. The dashed red line shows an estimated cost for developing and implementing additional projects to address the remnant flooding, using the planning-level H&H model as an evaluation tool and planning-level cost and performance estimates. The green line shows the likely cost for the CSSIP implementation using an adaptive management strategy. The additional cost of refining the H&H model as an evaluation tool and implementing pilot programs would be significantly less than developing and implementing additional projects beyond the recommended projects and programs in this CSSIP to reduce remnant flooding. The likely costs shown in this figure are for illustration purposes, and not based on any studies or analysis.

Figure 9-3 shows an illustration of an adaptive management strategy for implementing the CSSIP. An adaptive management strategy involves implementing the top 20% of prioritized CSSIP projects, pilot testing the programs, additional refinement of the H&H model, refinement of cost estimates, and refinement of any inputs with lower accuracy used to develop the CSSIP. This adaptive management strategy can be iterated several times over the implementation of entire CSSIP to continuously refine the accuracy and confidence of the tools and information to ultimately meet the final goal of 100% reduction of CSS outflows, flood protection for the 10-year storm system-wide, and protection of structures in the 100-year event. The recommendations describe the specific improvements that can be incorporated as part of an adaptive management strategy.

Recommendations: The recommendations of the CSSIP Update report are as follows:

1. Prepare a plan for implementation of the top 20% of capital improvement projects listed in this CSSIP Update, including any detailed feasibility studies followed by design and construction of these projects.

2. Develop a pilot program to evaluate different types of GI technologies and test their performance in different surface and sub-surface conditions, including soil type and slope; general public acceptability; maintenance issues; and other topics. Monitor the performance of the different technologies and establish the baseline performance of the different technologies. Update the cost estimates based on the pilot programs.

3. Develop a pilot program to evaluate the performance of RDII technologies, including detailed characterization of areas contributing to RDII by flow monitoring, smoke-testing, and sewer inspections. Develop an RDII reduction pilot program for source control by disconnecting storm water connections and sewer/manhole relining. Establish a baseline performance of the source control measures, and update the cost estimates based on pilot programs.

Figure 9-2: CSSIP Implementation Approach Using Adaptive Management

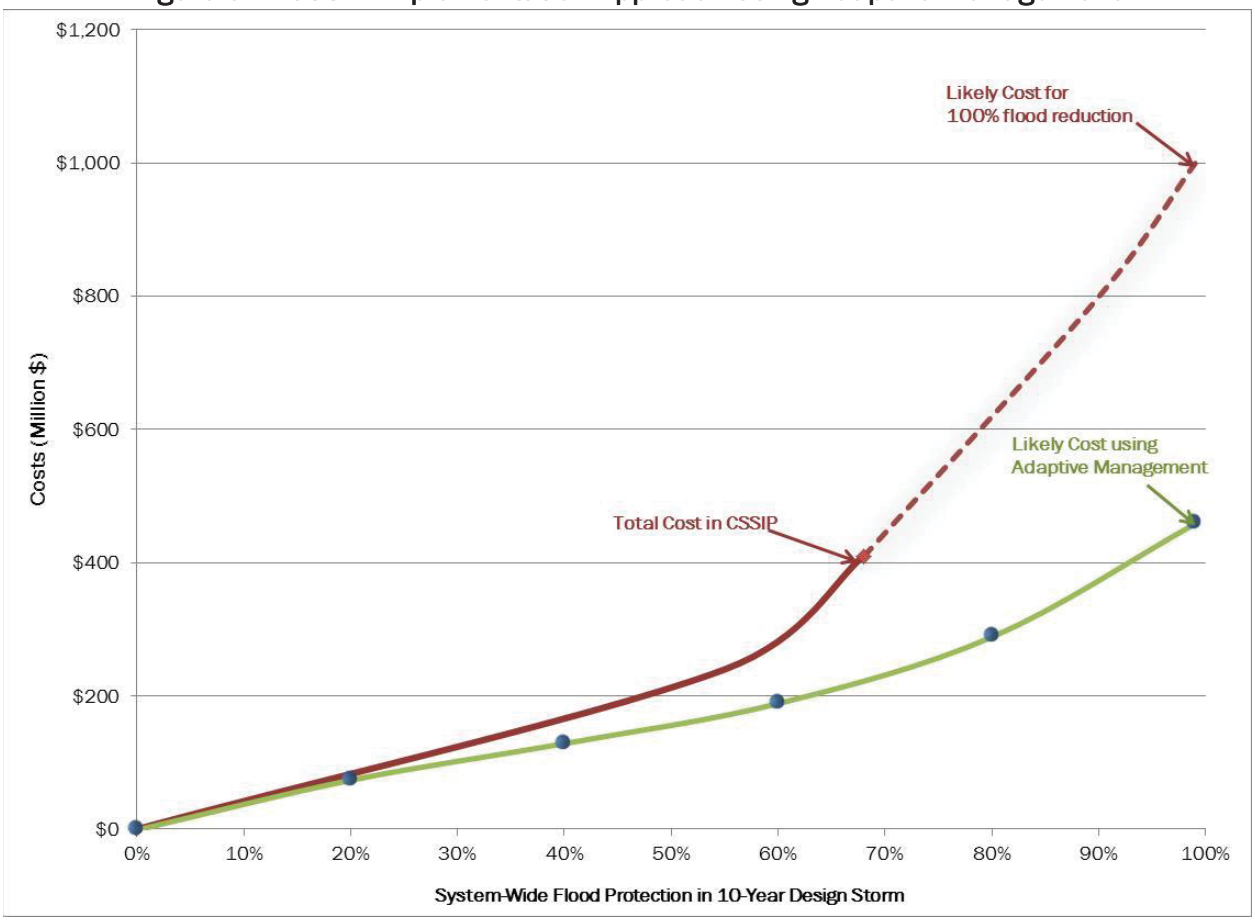
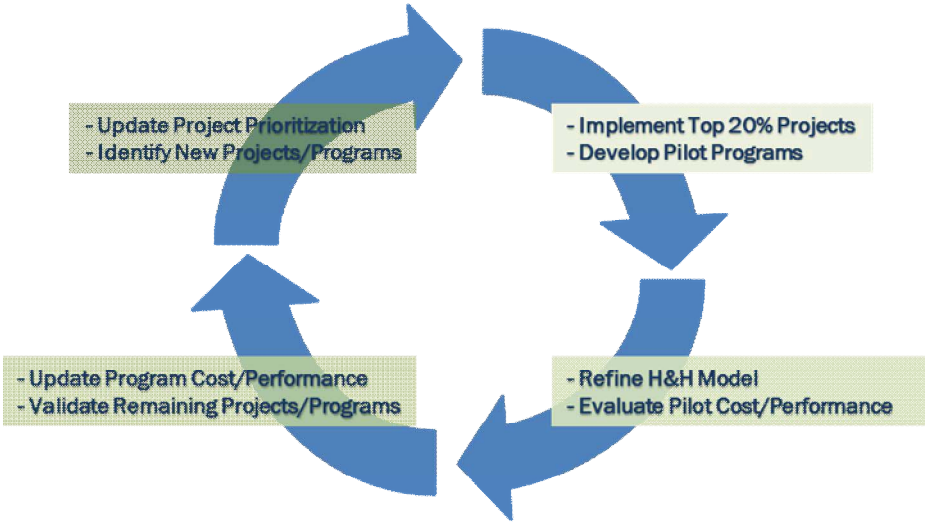


Figure 9-3: Adaptive Management Strategy

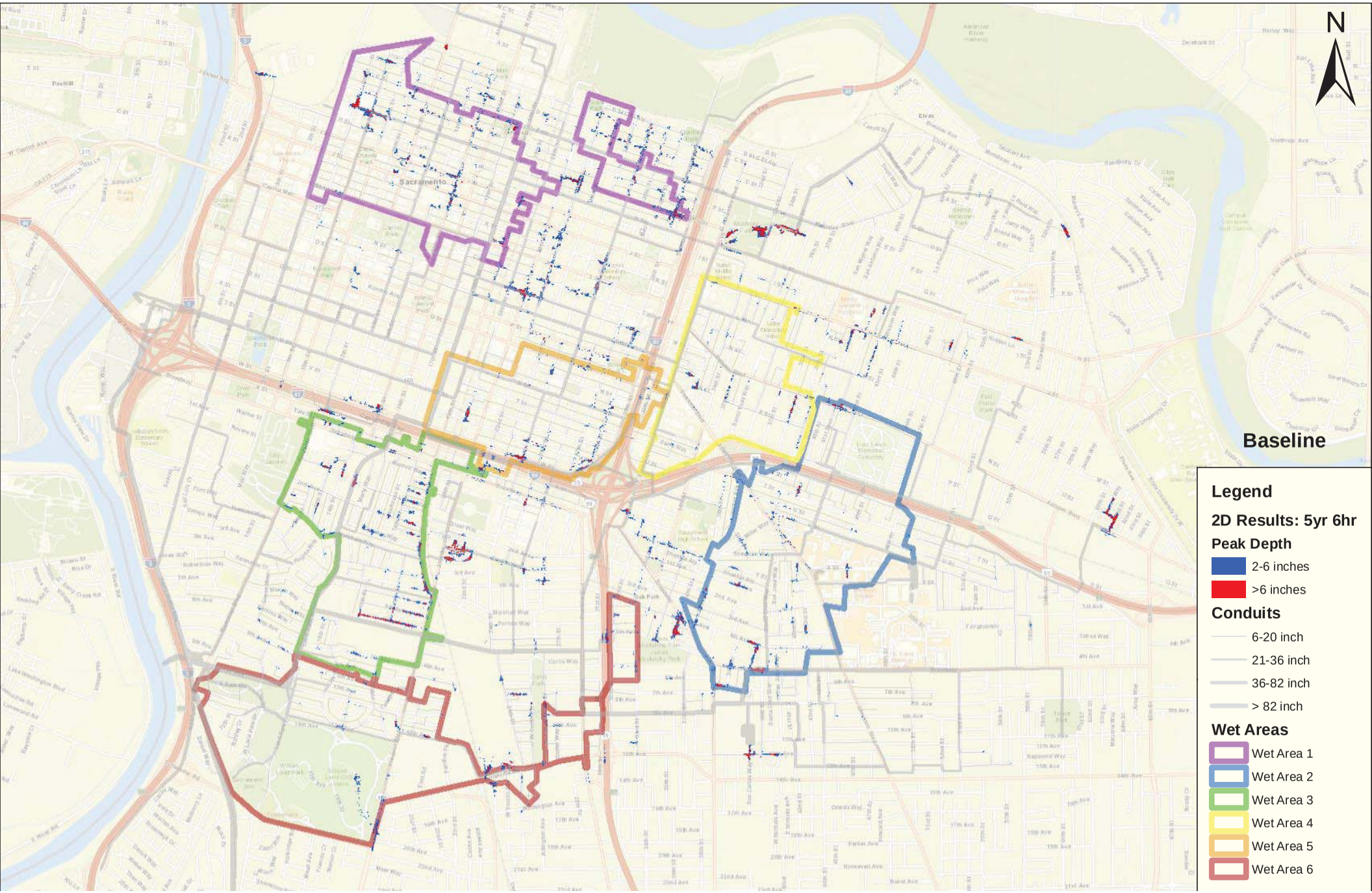


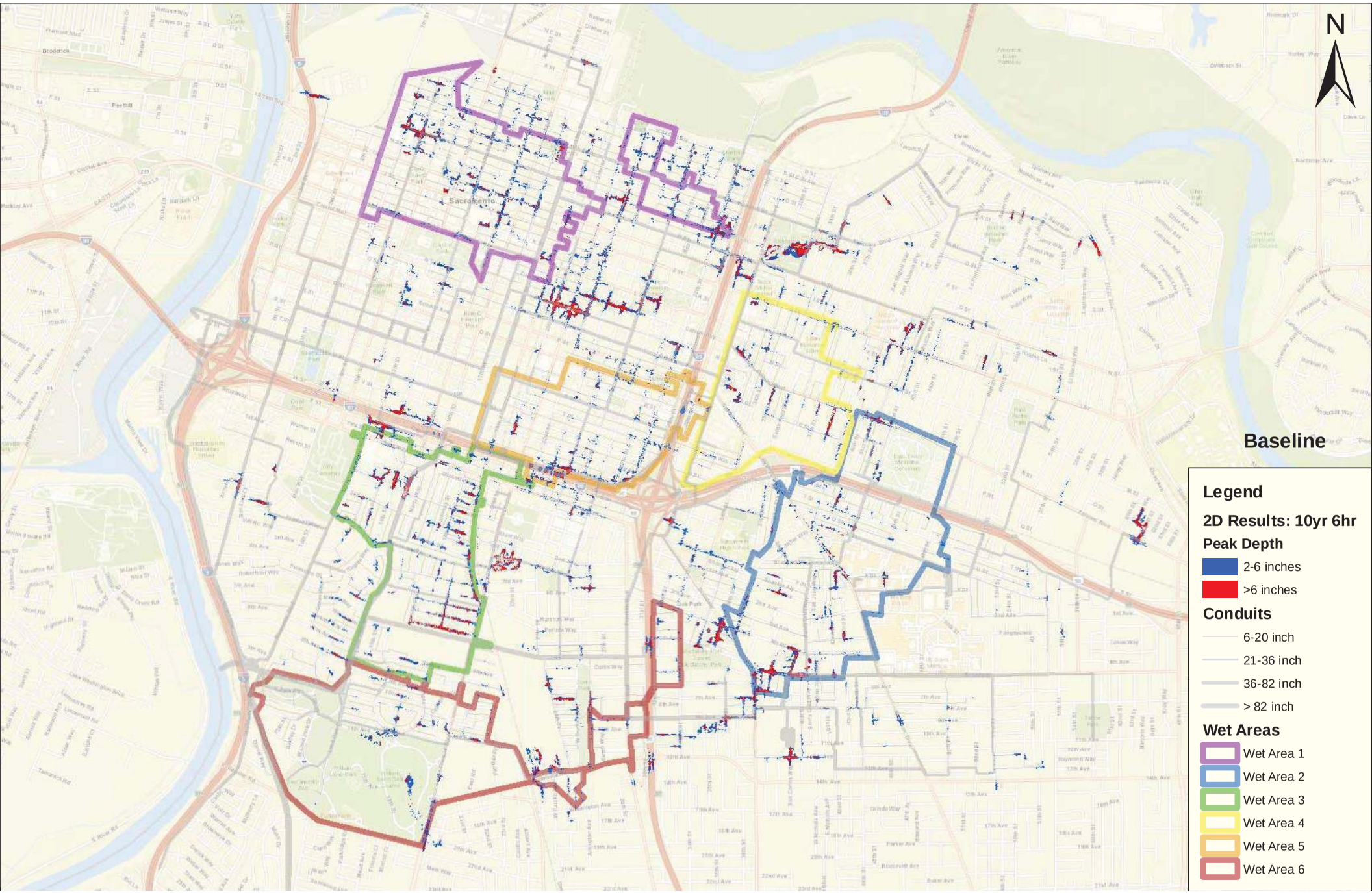
4. Refine the H&H model and relevant input datasets to further improve the accuracy of the evaluation tools. Develop a flow monitoring program for the next two wet weather seasons, and improve the H&H model calibration and validation using the flow monitoring data. Refine the surface elevation information in the H&H model using updated survey or LIDAR data. Use updated building and curb boundaries to improve the characterization of surface flow routing system-wide. Use surface flooding datasets (pictures, YouTube videos, etc.) of any recent historical flooding events to further calibrate and validate the 2D surface flooding results.

5. Confirm the performance of remaining projects in the CSSIP, and refine any projects or add new projects if necessary. Based on the pilot programs for Green Infrastructure and RDII technologies, develop a strategic longer-term program for implementation of these technologies to achieve a more cost-effective solution to local and system-wide flooding problems.

APPENDIX A

BASELINE RESULTS FOR 5-YEAR AND 10-YEAR DESIGN STORMS


N
Baseline
Legend
2D Results: 5yr 6hr
Peak Depth
2-6 inches
>6 inches
Conduits
6-20 inch
21-36 inch
36-82 inch
> 82 inch
Wet Areas
Wet Area 1
Wet Area 2
Wet Area 3
Wet Area 4
Wet Area 5
Wet Area 6


N
Baseline
Legend
2D Results: 10yr 6hr
Peak Depth
2-6 inches
>6 inches
Conduits
6-20 inch
21-36 inch
36-82 inch
> 82 inch
Wet Areas
Wet Area 1
Wet Area 2
Wet Area 3
Wet Area 4
Wet Area 5
Wet Area 6

APPENDIX B

COST ESTIMATES BY PROJECT

1. Project Cost Estimate for Project 1-1 (Zapata Park Storage)

Project # 1-1 Zapata Park			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
16	2,050	192	$393,600
24	1,070	288	$308,160
Pipeline Construction Cost Subtotal			$701,760
Misc. Pipeline Items (70%)			$491,232
Pipeline Construction Cost Estimate			$1,192,992
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	0.5	LS	$5,693,800
Pump Station			
Item	Capacity (mgd)	Unit Cost	Construction Cost
Pump Station, Fixed Pump	0.05	LS	$270,000
Estimated Project Construction Cost			$7,156,792
Construction Contingency (40%)			$2,862,717
Total Estimated Capital Cost (Rounded ENR=9967)			$10,020,000

2. Project Cost Estimate for Project 1-2 (G St. and 9th St. Parking Lot)

Project # 1-2 G St. and 9th St.			
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	0.6	LS	$5,925,820
Pump Station			
Item	Capacity (mgd)	Unit Cost	Construction Cost
Pump Station, Fixed Pump	$270,000	LS	$270,000
Estimated Project Construction Cost			$6,195,820
Construction Contingency (40%)			$2,478,328
Total Estimated Capital Cost (Rounded ENR=9967)			$8,670,000

3. Project Cost Estimate for Project 1-3 (9th Street from G to L Street)

Project # 1-3 9th St. from G to L St.			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
30	370	360	$133,200
54	50	648	$32,400
60	2,070	720	$1,490,400
Pipeline Construction Cost Subtotal			**$1,656,000**
Misc. Pipeline Items (70%)			$1,159,200
Estimated Project Construction Cost			**$2,815,200**
Construction Contingency (40%)			$1,126,080
Total Estimated Capital Cost (Rounded ENR=9967)			**$3,940,000**

4. Project Cost Estimate for Project 1-4 (14th Street Storage)

Project # 1-4 14th St. Storage				
Pipe Upsize/New Pipe Installation				
Diameter (inches)		Length (lf)	Unit Cost ($/lf)	Construction Cost
24		800	288	$230,400
Pipeline Construction Cost Subtotal				**$230,400**
Misc. Pipeline Items (70%)				$161,280
Pipeline Construction Cost Estimate				**$391,680**
Offline Linear Storage				
Diameter (inches)	Volume (MG)	Length (lf)	Unit Cost ($/lf)	Construction Cost
120	0.7	1,150	1,440	$1,656,000
Misc. Pipeline Items (70%)				$1,159,200
Storage Subtotal				**$2,815,200**
Estimated Project Construction Cost				**$3,206,880**
Construction Contingency (40%)				$1,282,752
Total Estimated Capital Cost (Rounded ENR=9967)				**$4,490,000**

5. Project Cost Estimate for Project 1-5 (N Street and 22nd Street)

Project # 1-5 N St. & 22nd St.			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
36	200	432	$86,400
48	600	576	$345,600
60	730	720	$525,600
84 (casing)	100	1,260	$126,000
Pipeline Construction Cost Subtotal			**$1,083,600**
Misc. Pipeline Items (70%)			$758,520
Estimated Project Construction Cost			**$1,842,120**
Construction Contingency (40%)			$736,848
Total Estimated Capital Cost (Rounded ENR=9967)			**$2,580,000**

6. Project Cost Estimate for Project 1-6 (24th Street Storage)

Project # 1-6 24th St. Storage				
Offline Linear Storage				
Diameter (inches)	Volume (MG)	Length (lf)	Unit Cost ($/lf)	Construction Cost
108	1.33	2,650	1,296	$3,434,400
Misc. Pipeline Items (70%)				$2,404,080
Estimated Project Construction Cost				**$5,838,480**
Construction Contingency (40%)				$2,335,392
Total Estimated Capital Cost (Rounded ENR=9967)				**$8,170,000**

7. Project Cost Estimate for Project 1-7 (Grant Park)

Project # 1-7 Grant Park			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
16	2,100	192	$403,200
20	420	240	$100,800
60	2,300	720	$1,656,000
Pipeline Construction Cost Subtotal			**$2,160,000**
Misc. Pipeline Items (70%)			$1,512,000
Pipeline Construction Cost Estimate			**$3,672,000**
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	2.8	LS	**$11,030,260**
Estimated Project Construction Cost			**$14,702,260**
Construction Contingency (40%)			$5,880,904
Total Estimated Capital Cost (Rounded ENR=9967)			**$20,580,000**

8. Project Cost Estimate for Project 2-1 (M St. Storage)

Project # 2-1 M St. Storage				
Pipe Upsize/New Pipe Installation				
Diameter (inches)		Length (lf)	Unit Cost ($/lf)	Construction Cost
24		1,020	288	$293,760
Pipeline Construction Cost Subtotal				$293,760
Misc. Pipeline Items (70%)				$205,632
Pipeline Construction Cost Estimate				$499,392
Offline Linear Storage				
Diameter (inches)	Volume (MG)	Length (lf)	Unit Cost ($/lf)	Construction Cost
120	1.1	3,000	1,440	$4,320,000
Misc. Pipeline Items (70%)				$3,024,000
Storage Subtotal				$7,344,000
Pump Station				
Item		Capacity (mgd)	Unit Cost	Construction Cost
Pump Station, Fixed Pump		0.05	LS	$270,000
Estimated Project Construction Cost				$8,113,392
Construction Contingency (40%)				$3,245,357
Total Estimated Capital Cost (Rounded ENR=9967)				$11,359,000

9. Project Cost Estimate for Project 2-2 (McClatchy Park)

Project # 2-2 McClatchy Park			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
24	1,120	288	$322,560
30	400	360	$144,000
36	2,600	432	$1,123,200
54	850	648	$550,800
Pipeline Construction Cost Subtotal			$2,140,560
Misc. Pipeline Items (70%)			$1,498,392
Pipeline Construction Cost Estimate			$3,638,952
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	2.7	LS	$10,798,240
Pump Station			
Item	Capacity (mgd)	Unit Cost	Construction Cost
Pump Station, Fixed Pump	0.1	LS	$280,000
Estimated Project Construction Cost			$14,717,192
Construction Contingency (40%)			$5,886,877
Total Estimated Capital Cost (Rounded ENR=9967)			$20,600,000

10. Project Cost Estimate for Project 2-3 (4th Avenue Park)

Project # 2-3 4th Avenue Park			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
16	2,200	192	$422,400
24	510	288	$146,880
Pipeline Construction Cost Subtotal			$569,280
Misc. Pipeline Items (70%)			$398,496
Pipeline Construction Cost Estimate			$967,776
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	2.0	LS	$9,174,100
Estimated Project Construction Cost			$10,141,876
Construction Contingency (40%)			$4,056,750
Total Estimated Capital Cost (Rounded ENR=9967)			$14,200,000

11. Project Cost Estimate for Project 3-1 (Bidwell & Freeport)

Project # 3-1 Bidwell & Freeport			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
48	1,070	576	$616,320
Pipeline Construction Cost Subtotal			$616,320
Misc. Pipeline Items (70%)			$431,424
Estimated Project Construction Cost			$1,047,744
Construction Contingency (40%)			$419,098
Total Estimated Capital Cost (Rounded ENR=9967)			$1,470,000

12. Project Cost Estimate for Project 3-2 (Land Park Tributary Area Upsizing)

Project # 3-2 Land Park Tributary Area Upsizing			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
16	619	192	$118,848
24	1,235	288	$355,680
36	756	432	$326,592
48	1,522	576	$876,672
60	2,074	720	$1,493,280
Pipeline Construction Cost Subtotal			$3,171,072
Misc. Pipeline Items (70%)			$2,219,750
Estimated Project Construction Cost			$5,390,822
Construction Contingency (40%)			$2,156,329
Total Estimated Capital Cost (Rounded ENR=9967)			$7,550,000

13. Project Cost Estimate for Project 3-3 (Curtis Park Village Storage)

Project # 3-3 Curtis Park Village Storage			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
96	2,100	1,152	$2,419,200
Pipeline Construction Cost Subtotal			$2,419,200
Misc. Pipeline Items (70%)			$1,693,440
Pipeline Construction Cost Estimate			$4,112,640
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	1.8	LS	8,710,060
Pump Station			
Item	Capacity (mgd)	Unit Cost	Construction Cost
Pump Station, Fixed Pump	0.25	LS	$310,000
Estimated Project Construction Cost			$13,132,700
Construction Contingency (40%)			$5,253,080
Total Estimated Capital Cost (Rounded ENR=9967)			$18,390,000

14. Project Cost Estimate for Project 3-4 (Sierra School Park)

Project # 3-4 Sierra School Park			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
30	1,200	360	$432,000
Pipeline Construction Costs Subtotal			$432,000
Misc. Pipeline Items (70%)			$302,400
Pipeline Construction Cost Estimate			$734,400
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	1.6	LS	$8,246,020
Estimated Project Construction Cost			$8,980,420
Construction Contingency (40%)			$3,592,168
Total Estimated Capital Cost (Rounded ENR=9967)			$12,570,000

15. Project Cost Estimate for Project 3-5 (Beverly Way)

Project # 3-5 Beverly Way				
Offline Linear Storage				
Diameter (inches)	Volume (MG)	Length (lf)	Unit Cost ($/lf)	Construction Cost
72	0.3	1,100	864	$950,400
Misc. Pipeline Items (70%)				$665,280
Project Subtotal				**$1,615,680**
Construction Contingency (40%)				$646,272
Total Estimated Construction Cost (Rounded ENR=9967)				**$2,260,000**

16. Project Cost Estimate for Project 3-6 (17th and Bidwell Storage)

Project # 3-6 17th and Bidwell Storage			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
18	150	216	$32,400
Pipeline Construction Cost Subtotal			$32,400
Misc. Pipeline Items (70%)			$22,680
Pipeline Construction Cost Estimate			$55,080
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	0.3	LS	$5,229,760
Estimated Project Construction Cost			$5,284,840
Construction Contingency (40%)			$2,113,936
Total Estimated Capital Cost (Rounded ENR=9967)			$7,400,000

17. Project Cost Estimate for Project 3-7 (Target Parking)

Project # 3-7 Target Parking			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
120	370	1,440	$532,800
Pipeline Construction Cost Subtotal			$532,800
Misc. Pipeline Items (70%)			$372,960
Pipeline Construction Cost Estimate			$905,760
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	0.3	LS	$5,229,760
Pump Station			
Item	Capacity (mgd)	Unit Cost	Construction Cost
Pump Stations, Fixed Pump	0.05	LS	$270,000
Estimated Project Construction Cost			$6,405,520
Construction Contingency (40%)			$2,562,208
Total Estimated Capital Cost (Rounded ENR=9967)			$8,970,000

18. Project Cost Estimate for Project 4-1 (McKinley Park Storage)

Project # 4-1 McKinley Park Storage			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
36	225	432	$97,200
Pipeline Construction Cost Subtotal			$97,200
Misc. Pipeline Items (70%)			$68,040
Pipeline Construction Cost Estimate			$165,240
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	4	LS	$13,814,500
Pump Station			
Item	Capacity (mgd)	Unit Cost	Construction Cost
Pump Station, Fixed Pump	4	LS	$1,040,000
Estimated Project Construction Cost			$15,019,740
Construction Contingency (40%)			$6,007,896
Total Estimated Capital Cost (Rounded ENR=9967)			$21,030,000

19. Project Cost Estimate for Project 4-2 (Sutter Middle School Storage)

Project # 4-2 Sutter Middle School Storage			
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	4	LS	$13,814,500
Estimated Project Construction Cost			$13,814,500
Construction Contingency (40%)			$5,525,800
Total Estimated Capital Cost (Rounded ENR=9967)			$19,340,000

20. Project Cost Estimate for Project 4-3 (P St. Storage)

Project # 4-3 P St. Storage				
Offline Linear Storage				
Diameter (inches)	Volume (MG)	Length (lf)	Unit Cost ($/lf)	Construction Cost
96	0.4	1,300	1,152	$1,497,600
Misc. Pipeline Items (70%)				$1,048,320
Estimated Project Construction Cost				**$2,545,920**
Construction Contingency (40%)				$1,018,368
Total Estimated Capital Cost (Rounded ENR=9967)				**$3,560,000**

21. Project Cost Estimate for Project 4-4 (Casita Way)

Project # 4-4 Casita Way			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
84	450	1,008	$453,600
Pipeline Construction Cost Subtotal			**$453,600**
Misc. Pipeline Items (70%)			$317,520
Estimated Project Construction Cost			**$771,120**
Construction Contingency (40%)			$308,448
Total Estimated Capital Cost (Rounded ENR=9967)			**$1,080,000**

22. Project Cost Estimate for Project 5-1 (T and 20th Street)

Project # 5-1 T St. or U St. at 20th			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
36	650	432	$280,800
Pipeline Construction Cost Subtotal			**$280,800**
Misc. Pipeline Items (70%)			$196,560
Estimated Project Construction Cost			**$477,360**
Construction Contingency (40%)			$190,944
Total Estimated Capital Cost (Rounded ENR=9967)			**$670,000**

23. Project Cost Estimate for Project 5-2 (T/U Alley and 28th Street)

Project # 5-2 28th and T/U Alley			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
24	600	288	$172,800
36	100	432	$43,200
Pipeline Construction Cost Subtotal			**$216,000**
Misc. Pipeline Items (70%)			$151,200
Estimated Project Construction Cost			**$367,200**
Construction Contingency (40%)			$146,880
Total Estimated Capital Cost (Rounded ENR=9967)			**$510,000**

24. Project Cost Estimate for Project 5-3 (W and 25th Street)

Project # 5-3 Storage under freeway at W and 25th			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
21	1,600	252	$403,200
24	230	288	$66,240
36	1,000	432	$432,000
Pipeline Construction Cost Subtotal			$901,440
Misc. Pipeline Items (70%)			$631,008
Pipeline Construction Cost Estimate			$1,532,448
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	1.2	LS	$7,317,940
Estimated Project Construction Cost			$8,850,388
Construction Contingency (40%)			$3,540,155
Total Estimated Capital Cost (Rounded ENR=9967)			$12,390,000

25. Project Cost Estimate for Project 6-1 (Freeport Blvd Upsizing)

Project # 6-1 Freeport Blvd. Upsizing			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
24	65	288	$18,720
36	2,740	432	$1,183,680
Pipeline Construction Cost Subtotal			$1,202,400
Misc. Pipeline Items (70%)			$841,680
Estimated Project Construction Cost			$2,044,080
Construction Contingency (40%)			$817,632
Total Estimated Capital Cost (Rounded ENR=9967)			$2,860,000

26. Project Cost Estimate for Project 6-2 (Riverside Blvd Upsizing)

Project # 6-2 Riverside Blvd.			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
36	700	432	$302,400
96	5,800	1,152	$6,681,600
Pipeline Construction Cost Subtotal			$6,984,000
Misc. Pipeline Items (70%)			$4,888,800
Estimated Project Construction Cost			$11,872,800
Construction Contingency (40%)			$4,749,120
Total Estimated Capital Cost (Rounded ENR=9967)			$16,620,000

27. Project Cost Estimate for Project 6-3 (Curtis Park Storage)

Project # 6-3 Curtis Park Storage			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
24	2,320	288	$668,160
Pipeline Construction Cost Subtotal			$668,160
Misc. Pipeline Items (70%)			$467,712
Pipeline Construction Cost Estimate			$1,135,872
Storage			
Item	Volume (MG)	Unit Cost	Construction Cost
Parcel Storage	2.0	LS	$9,174,100
Pump Station			
Item	Capacity (mgd)	Unit Cost	Construction Cost
Pump Station	2.2	LS	$690,000
Estimated Project Construction Cost			$10,999,972
Construction Contingency (40%)			$4,399,989
Total Estimated Capital Cost (Rounded ENR=9967)			$15,400,000

28. Project Cost Estimate for Project 6-4 (11th Avenue Conveyance)

Project # 6-4 11th St. Main Installation			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
60	3,000	720	$2,160,000
Pipeline Construction Cost Subtotal			$2,160,000
Misc. Pipeline Items (70%)			$1,512,000
Estimated Project Construction Cost			$3,672,000
Construction Contingency (40%)			$1,468,800
Total Estimated Capital Cost (Rounded ENR=9967)			$5,140,000

29. Project Cost Estimate for Existing System Optimization

21st Street Local Storage Improvement Project			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
24	490	288	$141,120
30	150	360	$54,000
Pipeline Construction Cost Subtotal			**$195,120**
Misc. Pipeline Items (70%)			$136,584
Estimated Project Construction Cost			**$331,704**
Small Project Allowance (40%)			$132,682
Construction Contingency (40%)			$132,682
Total Estimated Capital Cost (Rounded ENR=9967)			**$600,000**

Tahoe Park/Broadway Parallel Sewer Project		
Item	Unit Cost	Construction Cost
Modify Controls on Existing Sluice Gate	LS	$20,000
New Sluice gate with Controls and Structure	LS	$40,000
2 ft. deep x 4 ft. wide flow channel (500 lf @ $520/lf)	LS	$260,000
Estimated Project Construction Cost		**$320,000**
Small Project Allowance (40%)		$128,000
Construction Contingency (40%)		$128,000
Total Estimated Capital Cost (Rounded ENR=9967)		**$580,000**

East End Relief Sewer Tie-in Project			
Pipe Upsize/New Pipe Installation			
Diameter (inches)	Length (lf)	Unit Cost ($/lf)	Construction Cost
30	60	360	$21,600
Pipeline Construction Cost Subtotal			**$21,600**
Misc. Pipeline Items (70%)			$15,120
Estimated Project Construction Cost			**$36,720**
Small Project Allowance (40%)			$14,688
Construction Contingency (40%)			$14,688
Total Estimated Capital Cost (Rounded ENR=9967)			**$70,000**

U St. Inline Storage Project		
Item	Unit Cost	Construction Cost
Lowering the weir and widening on U St. (north side)	LS	$40,000
Lowering the weir and widening on U St. (south side)	LS	$40,000
Estimated Project Construction Cost		**$80,000**
Small Project Allowance (40%)		$32,000
Construction Contingency (40%)		$32,000
Total Estimated Capital Cost (Rounded ENR=9967)		**$140,000**

APPENDIX C

GREEN INFRASTRUCTURE PROGRAM COST ESTIMATE

Potential CSS Green Program

Green Infrastructure BMP	Area Reduction (acres)	Cost	Percent of CSS	Cost Benefit	Comments
Rain barrels	1.58	$ 706,100	25%	$ 448,317.46	Residential Rebate program
Bioretention Infiltration bulbouts	17.98	$ 5,149,022	5%	$ 286,374.94	Green Streets
Disconnected Pavement - Porous Pavement	23.14	$ 8,064,000	5%	$ 348,480.00	Green Streets
Interceptor Trees	4.42	$ 96,220	5%	$ 21,769.23	Replace dead city trees with evergre
Residential - Disconnected Roof Drains	45.84	$ 962,200	25%	$ 20,990.40	Residential Rebate program
Residential - Alternative Driveway Design	4.81	$ 977,980	5%	$ 203,322.26	Residential Rebate program
Residential -Impervious Area Managed by Amended Soil or Mulch Beds	26.51	$ 1,010,310	15%	$ 38,110.52	Residential Rebate program
Residential - Bio Retention	61.77	$ 1,010,310	15%	$ 16,356.00	Residential Rebate program
Commercial - Alternative Pavement Design	9.84	$ 2,000,000	?	$ 203,280.00	Commercial Rebate program
Disconnected Pavement - Divided Sidewalks	-	$ -			Retrofitting exisitng areas would be
Total	195.88	$ 19,976,142		$ 101,979.37	
25% Design/Contingency/Rebate Program Oversite		$ 4,994,035			
Total		**$ 24,970,177**			

Rain barrels

CSS & Contributing Sewer Accounts		26,646	
CSS Drainage Accounts		19,244	72%
Rain Barrels per Residence		4	
Total Rain Barrels		76,976	
Assume 25% interested		19,244	

Cost

Give 5000 away @ $70ea	$	350,000	http://www.sandiego.gov/thinkblue/pdf/rainbarrelflyer.pdf
Rebate the rest @ $25ea	$	356,100	
	$	706,100	

Runoff Reduction Credit from LID worksheet 1.58 acres

Bioretention

Bulbouts per intersection	4	
Area per bulb out 7 x 15	105	SF
Total Area per intersection	420	SF
Estimated number of Intersection is CSS	4,000	
Install @ 5% of intersections	200	
Total SF	84,000	
Cost per SF	$ 61	
Cost per intersection	$ 25,745	
Total Cost	$ 5,149,022	
Runoff Reduction Credit	17.98	acres

green street
http://www.missouribotanicalgarden.org/Portals/0/Shaw%20Nature%20Reserve/PDFs/horticulture/MSD/2012%20Stormwater/20120619Wilson.pdf

Hi Dalia,

Below are the construction cost estimate assumptions used for the Prop 84 concept proposal for the main LID devices. Unit costs for the devices are in dollars per square feet (SF) of device. The design, construction management, and vegetation establishment costs were a constant 15%, 15%, and 3% of the construction costs, respectively. Let me know if you have further questions. Thanks.
Kevin

Bioswales: $35/SF
Rain Gardens (with infiltration sump beneath media): $45/SF
Rain Gardens (without infiltration sump beneath media): $35/SF
Bioretention Areas (with infiltration sump beneath media): $45/SF

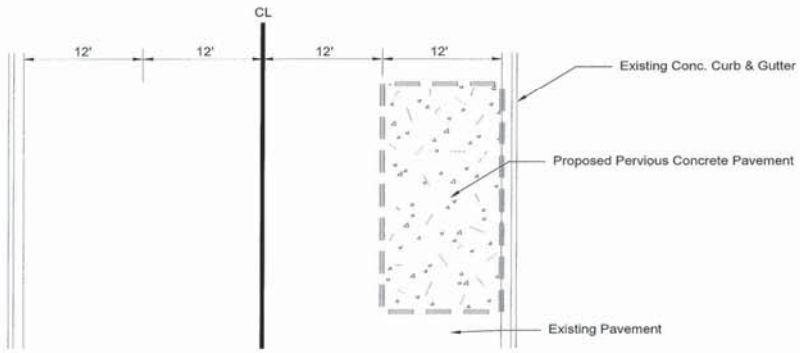
Disconnected Pavement - Porous Pavement for sidewalks and parking areas

porous pavement per city block

Length of street	320	feet
With of parking 8' each side	16	feet
With of sidewalk 6' each side	12	feet
Total Area per Block	8960	SF
Estimated blocks in CSS	3,000	
Install @ 5% of Blocks	150	
Total SF	1,344,000	
Cost per SF	$ 6	
Cost per Block	$ 53,760	
Total Cost	$ 8,064,000	
Runoff Reduction Credit	23.14	acres

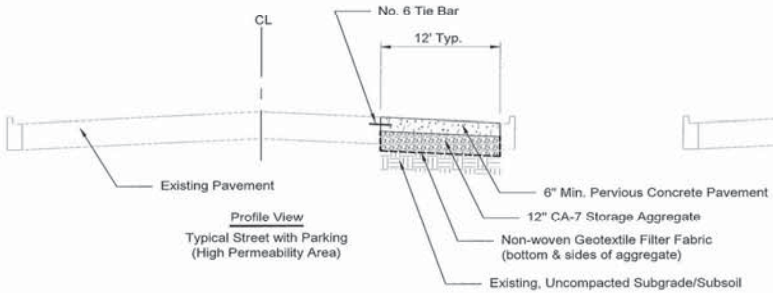
green street

http://www.missouribotanicalgarden.org/Portals/0/Shaw%20Nature%20Reserve/PDFs/h

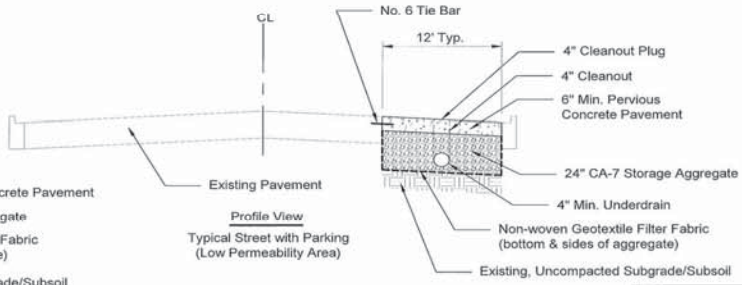


Plan View

Notes:
1. Base preparation is critical and should be done per manufacturer's recommendation and specification. Undulations and grade changes in the rock base will be reflected in the final outcome of the pervious concrete pavement.
2. Sub-grade soil infiltration rates should be confirmed prior to start of work.
3. The bottom of the stone should be a minimum of three feet above the seasonally high water table.
4. Bedding, base, and sub-base material shall be open-graded with no fines meeting the latest edition of the Illinois Standard Specifications For Road and Bridge Construction.
5. Avoid over compacting or contaminating the natural sub-grade soils.
6. Underdrain should be used if low permeability sub-grade soil is present.
7. The concrete shall be jointed and compacted using the methods listed, or alternatives, as demonstrated and approved by the Engineer.
8. Pervious concrete pavement shall not be placed when the ambient temperature is predicted for the jobsite to be 40°F or lower during the seven days following placement.
9. Pervious concrete pavement shall not be placed when the ambient temperature is predicted for the jobsite to rise above 90°F during the seven days following placement.
10. Pervious concrete pavement shall not be placed on frozen sub-grade.
11. If required by City of Evanston, the contractor shall furnish a proposed mixture design, with proportions of materials, or if mixture proportions are proprietary, a written submittal from the concrete supplier, prior to commencement of work.
12. Minimum thickness for all applications (excluding heavy vehicle or high traffic loads) shall be a single coarse placement with a minimum thickness of 6 inches unless otherwised specified.
13. Pavements frequently used by vehicles heavier than single axle service/delivery trucks shall have a minimum thickness of 8 inches.
14. Care must be taken to prevent closing the void structure of pervious concrete. Finishing operations are not allowed and internal vibration shall not be permitted.
15. Filter fabric shall be nonwoven, needle-punched polypropylene geotextile fabric. Fabric shall have a minimum weight of 3.5 ounces per square yard (ASTM D 3776), minimum wet grab tensile strength of 100 pounds (ASTM D 4632), and a minimum flow rate of 75 gallons/minute/square foot (ASTM D 4491).
16. No. 6 tie bars at 30 inch on center shall be placed at mid-depth of the slab and perpendicular to the longitudinal joint.
17. 4 inch PVC SDR 26 cleanouts shall be placed every 50 feet and connected to the underdrain system in low permeability areas.
18. The pervious concrete pavement shall be poured around the cleanout and the plug of the cleanout shall be flush with the surface.



Profile View
Typical Street with Parking
(High Permeability Area)



Profile View
Typical Street with Parking
(Low Permeability Area)

City of Evanston
Pervious Concrete Pavement
Detail
(Not To Scale)

Interceptor Trees		
CSS & Contributing Sewer Accounts		26,646
CSS Drainage Accounts		19,244
Existing City trees per Residence		2
Total existing trees		38,488
Replace Dead or Dieing Deciduous Trees with Conifers		
5% of existing trees over next 20years		1,924
Cost		
$50 per tree	$	96,220
Runoff Reduction Credit from LID worksheet		4.42

Disconnected Roof Drains				
CSS & Contributing Sewer Accounts	26,646			
CSS Drainage Accounts	19,244			
Existing City downspouts per Residence	4			
Total existing downspouts	76,976			
25% disconnected over next 20years	19,244			
Cost				
$50 per downspout	$ 962,200		$25	http://www.cityofpa.us/PDFs/Stormwater/RainGardenRebateApplication.pdf
			$50	http://www.foresthillsconnection.com/home-front/downspout-initiative-50-rebate-program/
Runoff Reduction Credit from LID worksheet	45.84	6du/ac	100% 5-10ft	

Too high did not use. LID calculator gives too much credit. Needs to be combined with rain graden/muclh bed

Alternative Driveway Design			
CSS & Contributing Sewer Accounts		26,646	
CSS Drainage Accounts		19,244	
4% of lot per calculator		290	
Total driveway available		5,588,458	sf
5% houses		962	
5% porous		279,423	sf
Cost			
$3.50 per sf rebate	$	977,980	
Runoff Reduction Credit from LID worksheet		4.81	

Impervious Area Managed by BioRetention	
CSS & Contributing Sewer Accounts	26,646
CSS Drainage Accounts	19,244
4' x 25'	100
Total mulch bio retention area available	1,924,400
15% houses	2,887
15% bioretention area	288,660
Cost	
$3.50 per sf rebate	$ 1,010,310
Runoff Reduction Credit from LID worksheet	61.77

Impervious Area Managed by Amended Soil or Mulch Beds	
CSS & Contributing Sewer Accounts	26,646
CSS Drainage Accounts	19,244
4' x 25'	100
Total mulch bed area available	1,924,400
15% houses	2,887
15% mulch bed area	288,660
Cost	
$3.50 per sf rebate	$ 1,010,310
Runoff Reduction Credit from LID worksheet	26.51

Alternative parking lot Design

porous pavement	571,429	sf

Cost

$3.50 per sf rebate	$ 2,000,000

Runoff Reduction Credit from LID worksheet 9.84

APPENDIX D

RDII PROGRAM COST ESTIMATE

	Actual #	Cost ($/item)	Total Estimated Cost ($)
Acres:	2,545	-	-
Manholes:	2,916	$ 6,000.00	$ 17,496,000.00
Laterals:	2,175	$ 7,500.00	$ 16,312,500.00
Total (w/mains)			$ 124,560,000.00

Diameter/Length Breakdown:

Diameter	Length (ft)	Cost ($/foot)	Estimated Cost ($)
Unknown	50,079	Unknown[1]	$ 7,000,000.00
4	5,393	$ 48.00	$ 258,873.18
6	112,639	$ 72.00	$ 8,109,976.02
8	188,616	$ 96.00	$ 18,107,145.60
9	425	$ 108.00	$ 45,871.88
10	58,398	$ 120.00	$ 7,007,778.69
12	85,832	$ 144.00	$ 12,359,850.10
14	3,624	$ 168.00	$ 608,858.56
15	39,040	$ 180.00	$ 7,027,138.86
16	2,486	$ 192.00	$ 477,293.26
18	24,478	$ 216.00	$ 5,287,250.35
20	3,194	$ 240.00	$ 766,657.41
21	14,378	$ 252.00	$ 3,623,193.77
22	2,499	$ 264.00	$ 659,687.92
24	25,015	$ 288.00	$ 7,204,241.77
26	868	$ 312.00	$ 270,838.80
27	7,156	$ 324.00	$ 2,318,405.61
30	18,186	$ 360.00	$ 6,547,099.15
33	1,131	$ 396.00	$ 447,918.71
36	6,073	$ 432.00	$ 2,623,481.62
39	3,998	N/A	N/A
42	7,117	N/A	N/A
45	1,815	N/A	N/A
48	15,273	N/A	N/A
51	252	N/A	N/A
54	5,021	N/A	N/A
60	7,308	N/A	N/A
63	1,508	N/A	N/A
66	10,052	N/A	N/A
72	766	N/A	N/A
78	2,881	N/A	N/A
84	311	N/A	N/A
90	374	N/A	N/A
Total			$ 90,751,561.27

[1] It is likely that the majority of these pipes are 36” and smaller. Suggestion would be to use the percentage of the total of the 36” and smaller and apply that same percentage to get a cost.

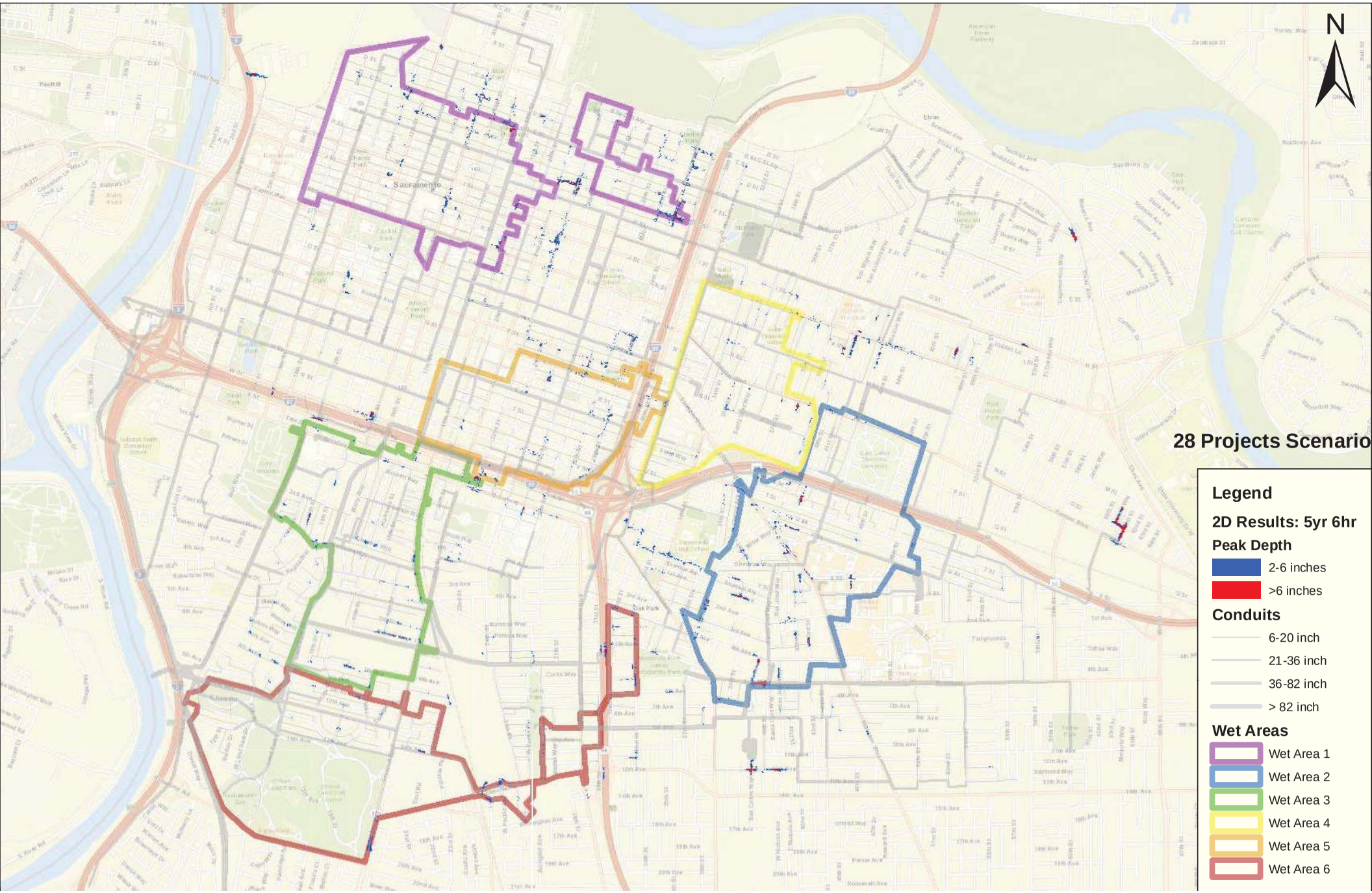
a. 50,079 ft is 8.3% of total length of pipe 4”-36”

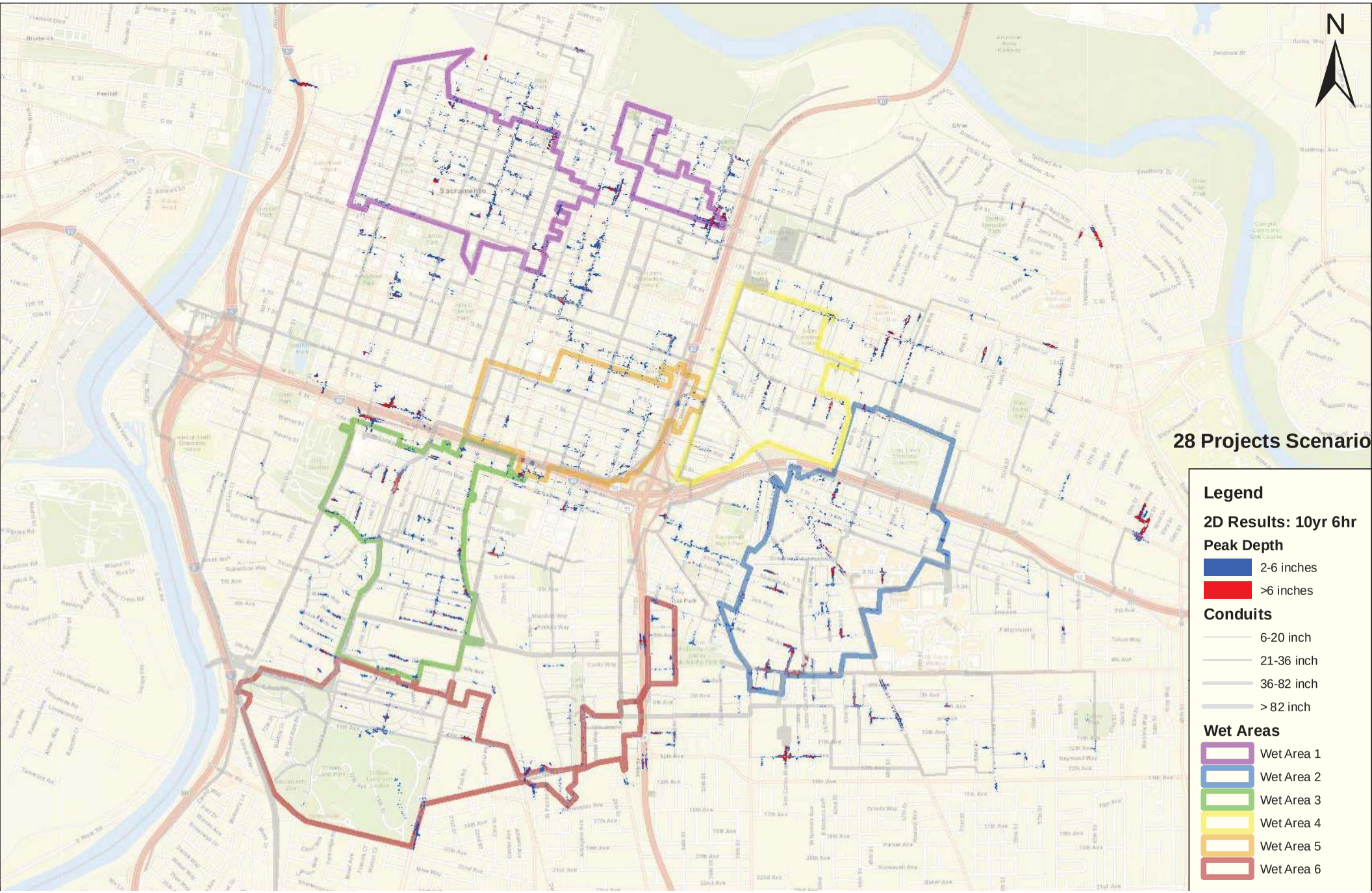
b. 8.3% of $83.75M is approximately $7.0M

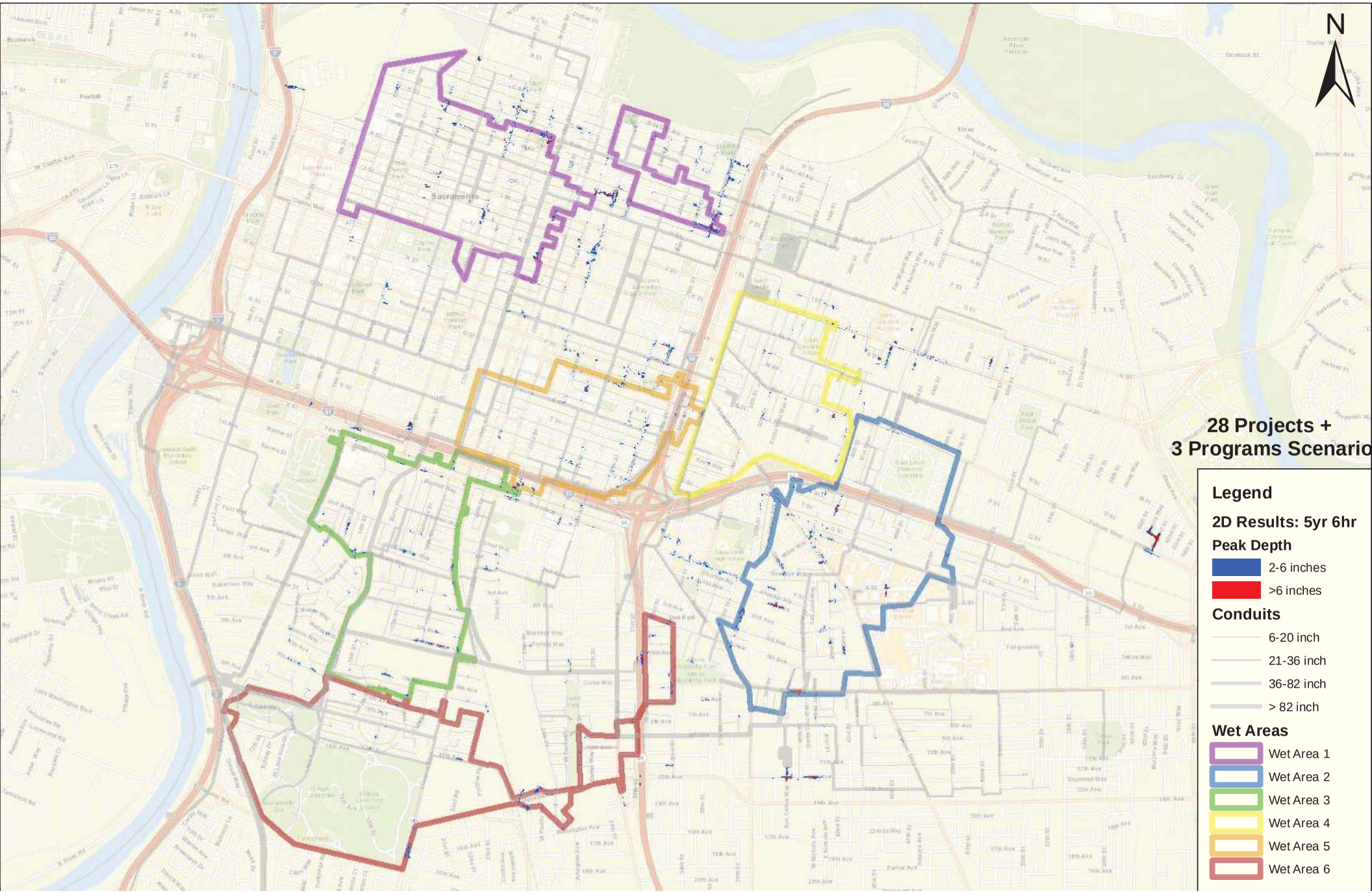
c. This would make the total for the mains approximately $90.75M

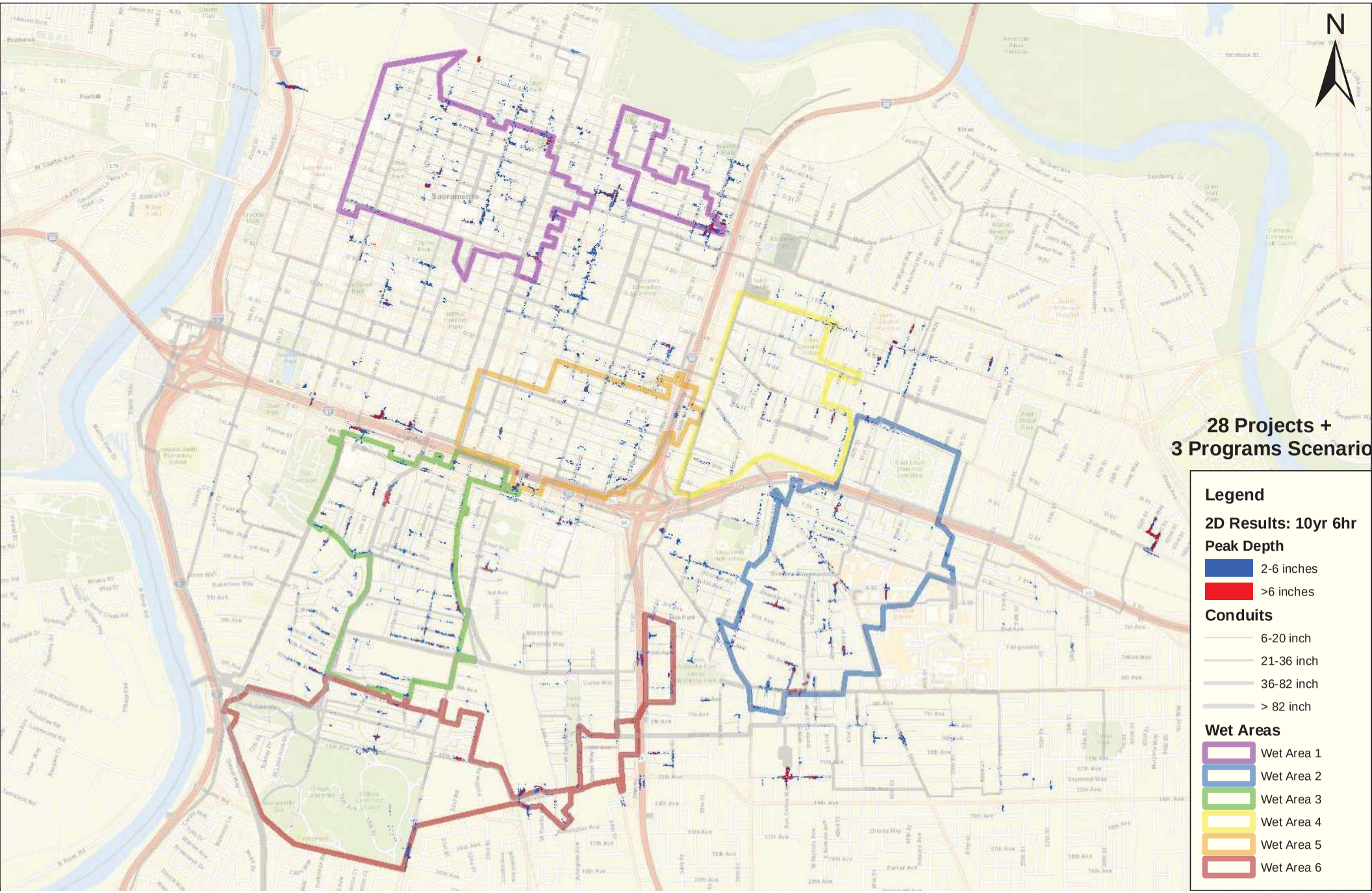
APPENDIX E

SCENARIO RESULTS FOR 5-YEAR AND 10-YEAR DESIGN STORMS


N
28 Projects Scenario
Legend
2D Results: 5yr 6hr
Peak Depth
2-6 inches
>6 inches
Conduits
6-20 inch
21-36 inch
36-82 inch
> 82 inch
Wet Areas
Wet Area 1
Wet Area 2
Wet Area 3
Wet Area 4
Wet Area 5
Wet Area 6


N
28 Projects Scenario
Legend
2D Results: 10yr 6hr
Peak Depth
2-6 inches
>6 inches
Conduits
6-20 inch
21-36 inch
36-82 inch
> 82 inch
Wet Areas
Wet Area 1
Wet Area 2
Wet Area 3
Wet Area 4
Wet Area 5
Wet Area 6


N
28 Projects +
3 Programs Scenario
Legend
2D Results: 5yr 6hr
Peak Depth
2-6 inches
>6 inches
Conduits
6-20 inch
21-36 inch
36-82 inch
> 82 inch
Wet Areas
Wet Area 1
Wet Area 2
Wet Area 3
Wet Area 4
Wet Area 5
Wet Area 6


N
28 Projects +
3 Programs Scenario
Legend
2D Results: 10yr 6hr
Peak Depth
2-6 inches
>6 inches
Conduits
6-20 inch
21-36 inch
36-82 inch
> 82 inch
Wet Areas
Wet Area 1
Wet Area 2
Wet Area 3
Wet Area 4
Wet Area 5
Wet Area 6

City of SACRAMENTO
Department of Utilities

Combined Sewer System (CSS) Improvement Plan Update – Phase 2

Technical Memorandum on Alhambra Alternatives

April, 2015

Version 4 (FINAL)

1. Introduction

The City of Sacramento regularly experiences flooding in the areas near Alhambra Blvd and McKinley Park. The hydrology and hydrodynamic (H&H) model of the City of Sacramento's combined sewer system corroborates historical observations of excess flow in these areas during storm events. Figure 1-1 below shows flooding results for a 10-year 6-hour design storm using the H&H model.

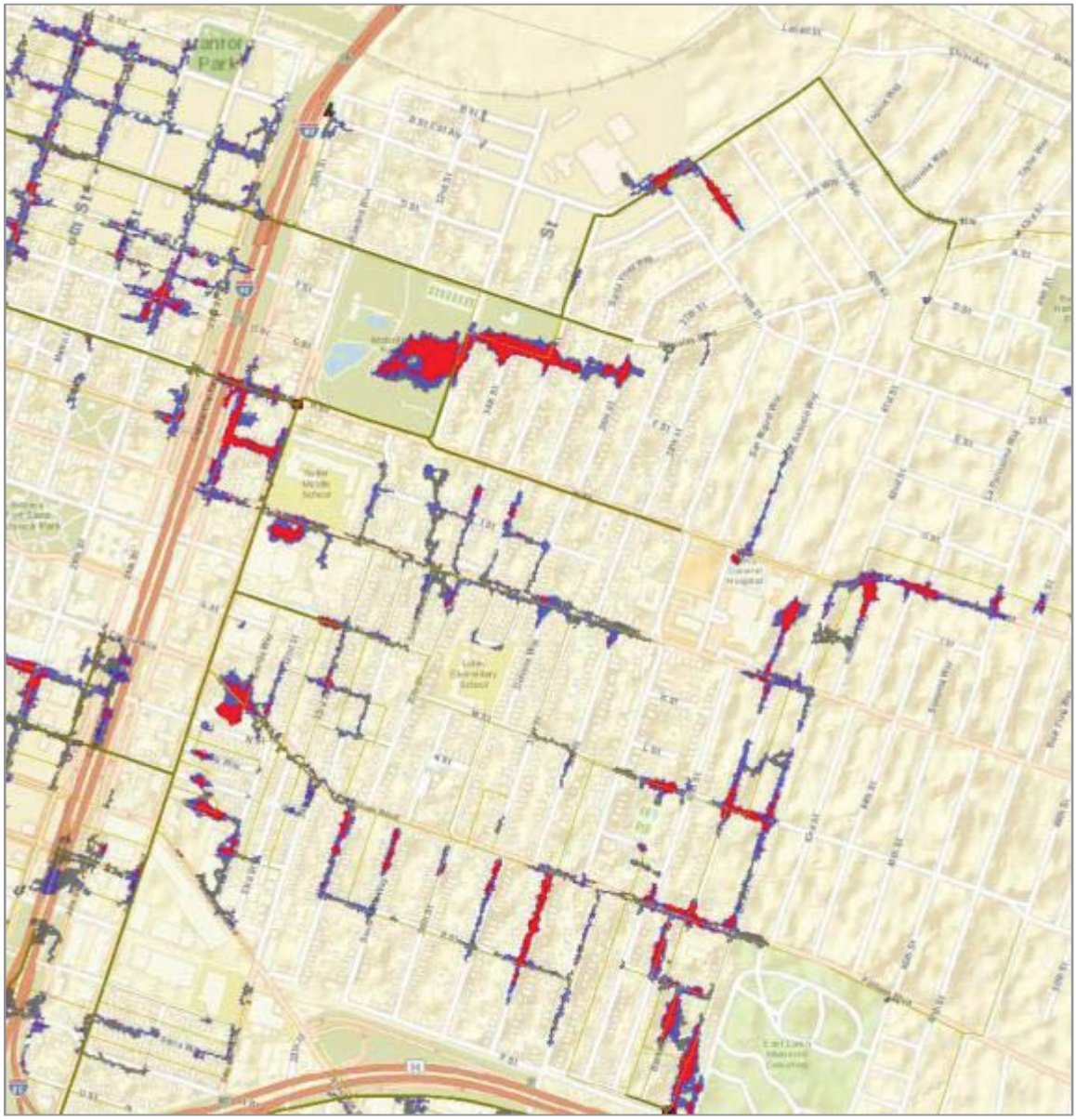


Figure 1-1: Flooding Results for a 10-year 6-hour design storm in the H&H Model

A proposed development at McKinley Park Village north of Alhambra Blvd and open spaces including McKinley Park, Sutter Middle School and Stanford Park provide opportunities to

accomplish flood volume reduction in this area. The McKinley Park Village developer and Sacramento Department of Utilities (DOU) have discussed designating a portion of the development area to be set aside for DOU to build a combined sewer storage facility.

The CSS Phase 1 study, as developed by DOU and AECOM, evaluated a deep tunnel and storage project to synergize with the McKinley Park Village development for construction of the storage project. The proposed project involved a deep tunnel from the O St on Alhambra Blvd to the McKinley Park Village. At the northern end of the tunnel, a storage vault would be built in the McKinley Park Village development area to store the combined sewer flows.

The purpose of this analysis was to develop a cost estimate for this project as defined in CSS Phase 1, and identify any additional alternatives that would provide comparable benefits at similar or lower cost. The analysis involved modeling the tunnel and storage project and all additional alternatives to quantify the flood reduction benefits and develop planning level cost estimates.

This Technical Memorandum summarizes the findings of this study. Section 2 presents the conceptual level details on tunnel and storage as developed in CSS Phase 1 Study, and alternatives that involve different tunnel and storage dimensions. Section 3 presents conceptual level details on alternatives that involve storage at other locations including McKinley Park, Sutter Middle School and Stanford Park. Section 4 presents the flood reduction benefits of all alternatives; Section 5 presents the cost estimates for the alternatives. Section 6 presents the summary and conclusions.

2. Alhambra Tunnel and Storage

2.1 CSS Phase 1 – Alhambra Tunnel and Storage Project

The proposed Alhambra Blvd tunnel and storage facility project consists of a new 8' to 10' deep tunnel along Alhambra Blvd and an underground storage facility at the north end of Alhambra Blvd, between the railroad levee and highway I-80. The purpose of this storage facility is to provide additional storage of combined and separated sanitary sewer flows during wet weather events from north-east part of the City of Sacramento. The area tributary to this storage facility includes approximately 591 acres of combined sewer area and 990 acres of separated sanitary sewer area. Figure 2-1 below shows a plan view of tunnel along Alhambra Blvd, storage facility at the end of Alhambra Blvd, and three inflow weirs that allow combined and separated sanitary sewer flows to enter the tunnel and storage facility during wet weather. This original tunnel and storage project is referred as "Scenario A0" in this technical memorandum.

Figure 2-1 : Plan View of Alhambra Blvd Storage Facility

A conceptual drawing of the proposed McKinley Park Village storage layout can be found in Appendix A. Figure 2-2 below shows a peak hydraulic grade line (HGL) in the tunnel and storage facility for a 5-year design storm.

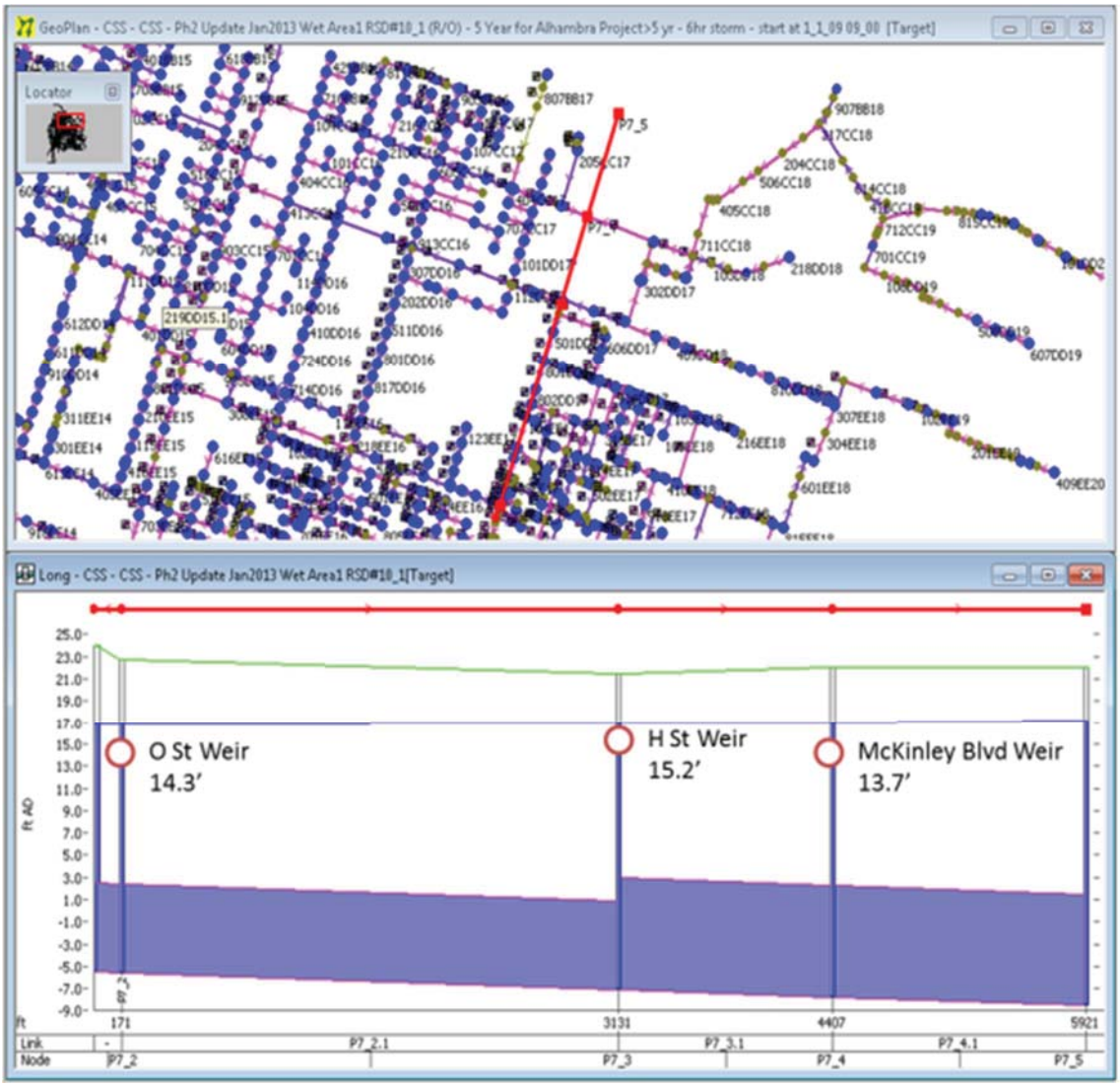


Figure 2-2: Peak HGL in Alhambra Blvd Tunnel and Storage in 5-yr design storm

2.2 Alternatives of Alhambra Tunnel and Storage with Deep Tunnel

Two deep tunneling technologies were investigated in this study for the purposes of developing cost estimates:

- First tunneling technology uses the tunneling machine to drill a tunnel larger than the intended carrier pipe. This technology allows the machine to tunnel as far as needed to create this larger casing. The carrier pipe is then inserted into the casing.

- Second tunneling technology involves excavating a pit approximately every 1000 feet and using the tunneling machine to direct bore and jack the carrier pipe. This option can mean less costly equipment and less excavation costs but does involve higher costs for digging many pits.

The costs were compared for both tunneling technologies, and ultimately are very similar.

The total cost of the project also does not vary significantly by the size of the tunnel diameter. The tunneling machines to use for either technology needs to be manufactured, shipped, and assembled on site for the project. The total cost of purchasing and setting up the tunneling machine at the site represents a large portion of the project cost, than the cost of a varying size of the tunnel diameter.

2.3 Alternatives of Alhambra Tunnel and Storage with Shallow Tunnel

As the cost of deep tunnel of large diameters being high; smaller diameter pipes constructed using open cut technologies were considered. Three, shallow smaller size pipe variations of the originally proposed Alhambra project were evaluated. All three modifications involve shallow depth, open cut tunnel installations. Open cut trenching is a much more commonplace and less expensive construction method, expected to bring down project costs significantly.

Three tunnel and storage configuration alternatives were considered as part of shallow tunnel alternatives:

i. First alternative, referred as “Scenario A1” includes:
   - 2750 linear feet (LF) of 4 foot diameter open cut pipe on Alhambra Blvd.
   - 1 weir only, at H St.
   - Storage at McKinley Park Village remains the same at 530,000 CF

ii. The second alternative, referred as “Scenario A2” includes:
   - 1500 LF of 4 foot diameter open cut pipe on Alhambra Blvd.
   - 1 weir only, at E St.
   - Storage at McKinley Park Village remains the same at 530,000 CF

iii. The third alternative, referred as “Scenario A3” includes:
   - 2750 LF of 4 foot diameter open cut pipe on Alhambra Blvd.
   - 2 weirs, 1 at E St. and 1 at H St.
   - Storage at McKinley Park Village remains the same at 530,000 CF

The flood reduction benefits of these alternatives will be discussed in detail in Section 4. To summarize, the shallow and smaller tunnel did not achieve the same magnitude of flooding reduction as did the originally planned deep tunnel and storage project in CSS Phase1. This result was expected due to a smaller diameter tunnel and maintaining the storage volume in McKinley Park Village.

3. Other Storage Alternatives to Alhambra Tunnel and Storage

Due to the high cost of large diameter deep tunnel and the limited capacity of smaller diameter shallow pipe to convey the flows to storage located in McKinley Park Village, other storage alternatives were considered and evaluated. Three alternative storage location options were evaluated, specifically McKinley Park, Sutter Middle School and Stanford Park. Constructing an underground storage at each of the sites was deemed feasible by DOU.

The storage in these three alternatives was modeled after the Oak Park project, a DOU wet weather storage facility that was recently designed and implemented. The Oak Park project consists of parallel pipe cells placed to form a pipe manifold which provides storage underneath a City Park. An inlet allows combined sewer flows to enter the manifold during wet weather events and a pump drains flows out after the storm is over and downstream HGL has receded. DOU found this storage construction to be time and cost-effective, providing the same flood reduction benefits as compared to a cast-in-place storage vault.

The three storage alternatives are discussed below:

3.1 McKinley Park Storage

The first location considered for an underground storage facility was McKinley Park. Figure 3-1 below shows the McKinley Park proposed storage facility. The project is referred as "Scenario B" and includes the following storage elements:

- Approximately 3.5 acres of available footprint to construct a storage facility
- Approximately 530,000 CF of storage with parallel pipes. A deep storage would be required to eliminate flooding in nearby low-lying areas
- 225 LF of 3 foot connecting pipe from existing CSS to storage facility
- Inlet weir on Park Way
- Pump station at 7.4 CF per second (cfs) to drain the storage facility[1]

[1] Based on a 24 hour draw down period

Figure 3-1: McKinley Park Storage

3.2 Sutter Middle School Storage

This alternative includes an underground storage facility at Sutter Middle School. Figure 3-2 below shows the extent of the proposed project. The project referred as "Scenario C", includes:

- Approximately 4.2 acres of available footprint to construct a storage facility
- Weirs along Alhambra Blvd to allow inflow
- Approximately 530,000 CF of storage with parallel pipes. A shallow storage is possible at this location to eliminate the flooding
- A flap valve to allow a drain the storage via gravity

Figure 3-2: Sutter Middle School Storage

3.3 Stanford Park Storage

This alternative includes an underground storage facility was Stanford Park, northwest of McKinley Park. Figure 3-3 presents a visual representation of the project. The project referred as "Scenario D", consists of the following:

- Approximately 2.3 acres available for storage
- 900 LF of 9 foot connecting pipe from existing CSS to storage vault
- Inlet weir on B St
- Approximately 530,000 CF of storage with parallel pipes. A deep storage would be required to eliminate flooding in nearby low-lying areas
- Pump station at 7.4 cfs to drain the storage facility[2]

[2] Based on a 24 hour draw down period

Figure 3-3: Stanford Park Storage

Each storage project alternative was evaluated using the H&H model individually; the performance of the project alternative for reducing flooding in the area was quantified for a 10-year 6-hour design storm. Section 4 below presents the performance of different project alternatives.

4 Performance of Project Alternatives

4.1 Performance of Individual Alternatives

This section documents the performance of project alternatives discussed above to reduce flooding in the area. Table 4-1 below tabulates all of the scenarios detailed previously, including the original proposed project, Scenario A0; the three shallow-depth modifications, Scenarios A1, A2, and A3; and the three alternative storage projects, Scenarios B, C, and D. The table summarizes the projects, including descriptions, pipe dimensions, storage volume; drain out method, and the total flooding in the area with the project. The table also tabulates the reduction in flood volume as compared to the existing conditions.

Note that the method to drain out the storage is important for both costing and operations. Pump stations are more expensive to install than gravity systems, and carry a higher life cycle cost for operations and maintenance. Out of the scenarios considered in this analysis, only Sutter Middle School Storage, “Scenario C”, uses a flap valve to drain out storage via gravity.

Overall, among the single project alternatives, the original deep tunnel and storage project as planned in CSS Phase 1 provides the greatest reduction in flood volume. McKinley Park and Sutter Middle School storages achieve high flood reduction, 530,000 and 480,000 CF respectively, while Stanford Park storage achieves the least at 390,000 CF reductions. The three shallow tunnel modifications achieve medium to low flood reduction.

As a measure of efficacy, Table 4-1 introduces the metric of flood volume reduced over storage volume added. The higher the value of this ratio, the more effective the constructed storage project is during the design storm. A ratio of 1 indicates that the storage added was fully utilized during the storm event, a desirable occurrence. Sutter Middle School storage has a ratio of 1, the highest of all the single project scenarios. McKinley Park Storage has the next best ratio of 0.9, with Stanford Park being least effective with a ratio of 0.6. The original project proposal achieved a ratio of 0.7.

4.2 Performance of Combined Alternatives

Since none of the projects completely eliminated the flooding in the area for a 10-year design storm, DOU also wanted to evaluate the performance of select project alternative combinations. Table 4-1 presents six scenarios, “Scenario E through J” that combined two or more individual project alternatives.

- “Scenario E” includes combining “Scenario A3”, the shallow tunnel with two weirs and proposed storage at McKinley Park Village; “Scenario B”, McKinley Park Storage and “Scenario C”; the Sutter Middle School Storage. In this combination scenario, the amount of storage volume increased significantly and the scenario provides a higher flood reduction benefit than any individual project. However the flood volume to storage ratio indicates very low efficacy for this combined scenario.

- “Scenario F” includes combining; “Scenario B”, McKinley Park Storage, “Scenario C”, Sutter Middle School Storage, and “Scenario D”, Stanford Park Storage. This scenario performed similarly to the previous combination, greatly increasing the available storage and flood reduction benefits, but lowering the efficiency.

- “Scenario G” includes combining two projects: Scenario B”, McKinley Park Storage and “Scenario C”, Sutter Middle School Storage. This scenario is most effective with a higher storage to flood reduction benefits ratio.

- “Scenario H” includes combining two projects: “Scenario A3”, the shallow tunnel with two weirs and proposed storage at McKinley Park Village and “Scenario B”, McKinley Park Storage.

- “Scenario I” includes combining two projects: “Scenario A3”, the shallow tunnel with two weirs and proposed storage at McKinley Park Village and “Scenario C”; the Sutter Middle School Storage.

- “Scenario J” includes combining only projects: Scenario B”, McKinley Park Storage and “Scenario D”, Stanford Park Storage.

“Scenario K” is a modified version of “Scenario B”, with multiple storage facilities placed underneath the McKinley Park footprint. This alternative considers a larger underground storage underneath the McKinley Park. **Figure 4-1** below shows the layout of the proposed Larger Size McKinley Park storage facility. The project is referred as “Scenario K” and includes the following storage elements:

- Approximately 3.5 acres of available footprint to construct a storage facility
- Approximately 1,062,000 CF of total storage under the baseball and soccer field
- Majority of the storage will be built under the baseball field by double stacking box culverts, with soccer field available as a back-up location for any storage not built under the baseball field footprint.
- 225 LF of 3 foot connecting pipe from existing CSS to storage facility
- Up to three inlet weirs on Park Way, Alhambra Blvd and H St, and McKinley and Alhambra Blvd.
- Pump station at 12 CF per second (cfs) to drain the storage facility[3], with the two storage facilities will be deep storages to eliminate flooding in the nearby low-lying areas. The two storage facilities would be constructed with large pipes set in parallel.

[3] Based on a 24 hour draw down period

Figure 4-1: Larger McKinley Park Storage

Table 4-1: Analysis Results by Scenario

Scenario	Description	Pipe Connection	New Pipe Storage (MG)	Storage Vault Size (MG)	Total Storage (MG)	Drain out	Local Flooding on 2D Mesh for 10-yr (MG)[1]	Total CSS Surcharge to Surface (MG)[2]	Reduction in Local Flooding on 2D Mesh in 10-yr (MG)	Reduction in CSS Surcharge to Surface (MG)	Ratio of Reduction in Local Flooding in 2D Mesh to Storage Volume
Baseline[3]	-	-	-	-	-	-	8.3	18.0	0.0	-	0.0
A0	530,000 CF of McKinley Village Dev. Storage	8 feet of 3000 LF, 10 feet of 2750 LF of tunnel (deep pipe) on Alhambra with weirs on E St, H. St and O St.	2.8	4	6.8	Pump Station	3.5	12.9	4.8	5.1	0.7
A1	530,000 CF of McKinley Village Dev. Storage	4 feet of 2750 LF open cut pipe (shallow depth) on Alhambra with weir on H St. only	0.3	4	4.3	Pump Station	5.4	15.7	2.9	2.3	0.7
A2	530,000 CF of McKinley Village Dev. Storage	4 feet of 1500 LF open cut pipe (shallow depth) on Alhambra with weir on E St.only	0.1	4	4.1	Pump Station	5.6	16.1	2.7	1.9	0.7
A3	530,000 CF of McKinley Village Dev. Storage	4 feet of 2750 LF open cut pipe (shallow depth) on Alhambra with weirs on E St. and H St	0.3	4	4.3	Pump Station	5.0	15.3	3.4	2.7	0.8
B	530,000 CF of McKinley Park Storage	3 feet pipe of 225 LF connecting from Park Way to the storage. Approx. 3.5 acres available for storage, deep storage required to eliminate low point on Park Way	0.0	4	4.0	Pump Station	4.7	15.0	3.6	3.0	0.9
C	530,000 CF of Sutter Middle School Storage	Weirs from Alhambra Blvd to fill the storage. Approx. 4.2 acres of footprint available for a shallow storage.	0.0	4	4.0	Flap valve (Gravity storage)	4.3	14.9	4.0	3.1	1.0
D	530,000 CF of Stanford Park Storage	9 feet pipe of 900 LF connecting E St. to storage in Stanford Park area. The storage has to be deep enough to capture flows from low point on E. Street.	0.5	4	4.5	Pump Station	5.4	15.9	2.9	2.1	0.6
E	Combo A3 + B + C	Combo A3 + B + C	0.3	12	12.3	2 Pump Station + Gravity	2.8	12.2	5.5	5.8	0.4
F	Combo B + C + D	Combo B + C + D	0.5	12	12.5	2 Pump Station + Gravity	2.8	12.0	5.5	6.0	0.4

G	Combo B + C	Combo B + C	0.0	8	8.0	1 Pump Station + Gravity	3.0	12.7	5.3	5.3	0.7
H	Combo A3 & B	Combo A3 & B	0.3	8	8.3	2 Pump Stations	3.7	13.5	4.6	4.5	0.6
I	Combo A3 & C	Combo A3 & C	0.3	8	8.3	1 Pump Station + Gravity	3.2	12.9	5.1	5.1	0.6
J	Combo B & D	Combo B & D	0.0	8	8.0	2 Pump Stations	4.1	13.8	4.2	4.2	0.5
K	1,062,000 CF storage facilities at McKinley Park	Majority of the storage will be built under the baseball field, with soccer field available for any storage not built under the baseball field footprint.	0.0	8	8.0	Combined Pump Station	3.0	12.4	5.3	5.6	0.7

[1] 2D Mesh for the Alhambra Area was used for evaluation, with the local flood extent polygon used to calculate total flooding on the surface.
[2] Total CSS Surcharge to Surface includes the flows from the entire combined sewer system going to the surface. In 1D area, this includes negative gully flows. In 2D area, this includes negative Flows to 2D mesh.
[3] Baseline includes approximately 25 downstream CSSIP Projects.

5 Cost of Project Alternatives

Planning level cost estimates were developed for all alternatives discussed in Section 4. Appendix B presents the planning level cost estimates for the deep and shallow tunnel alternatives on Alhambra Blvd (A0, A1, A2 and A3). Cost estimate for the storage alternatives at McKinley Park, Sutter Middle School, and Stanford Park (B,C, and D) were developed based on the Oak Park cost estimated provided by DOU. Appendix B includes the cost estimate developed for the Oak Park storage.

Table 5-1 below presents the total project cost, broken down by material and construction cost, misc. site work and design/contingency costs. This table also compares the total project cost with the flood reduction benefit (CF) provided by each alternative. The total cost for the combination of the key alternatives (E, F, G, H, I, J and K) is a sum of the costs for each alternative.

Figure 5-1 below compares the total project cost with the flood reduction benefit (CF). If a curve is drawn to connect the points, the alternatives G and K would likely be on the "knee" of the curve, i.e. provide the best performance in terms of flood reduction for the cost.

Figure 5-1: Cost Benefit Curve

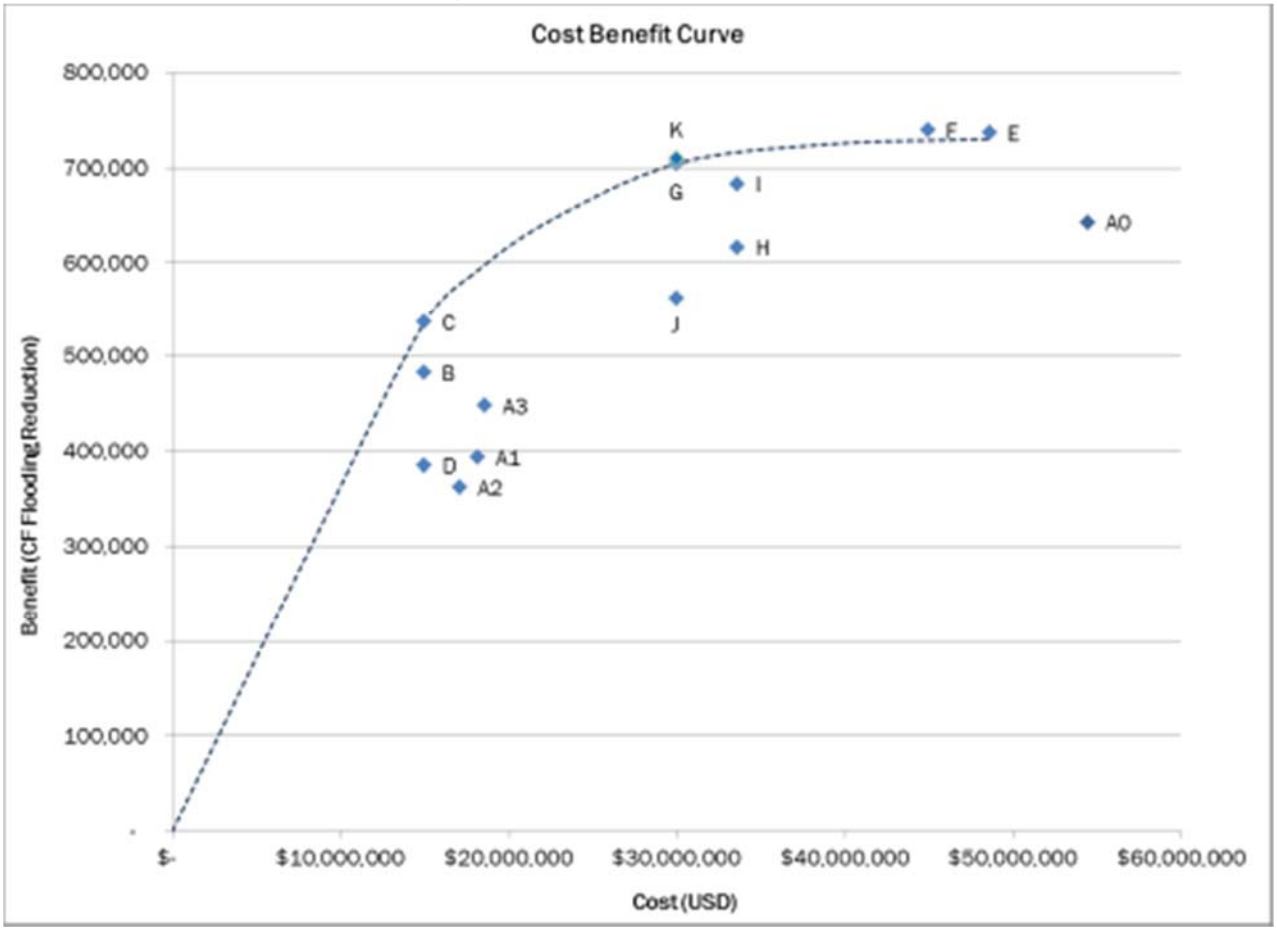


Table 5-1: Planning Level Costs for Alternatives

Scenario	Description	Pipe Connection	Materials and Construction Costs (USD)	Miscellaneous Site Work (USD)	Design, Contingency, and Other Costs (USD)	Total Project Cost (USD)	Flooding Reduction (CF)
A0	530,000 CF of McKinley Village Dev. Storage	8 feet of 3000 LF, 10 feet of 2750 LF of tunnel (deep pipe) on Alhambra with weirs on E St, H St and O St	$ 43,070,000	$ 500,000	$ 10,892,500	$ 54,462,500	643,048
A1	530,000 CF of McKinley Village Dev. Storage	4 feet of 2750 LF open cut pipe (shallow depth) on Alhambra with weir on H St only	$ 14,400,000	$ 150,000	$ 3,637,500	$ 18,187,500	393,048
A2	530,000 CF of McKinley Village Dev. Storage	4 feet of 1500 LF open cut pipe (shallow depth) on Alhambra with weir on E Stonly	$ 13,550,000	$ 150,000	$ 3,425,000	$ 17,125,000	362,299
A3	530,000 CF of McKinley Village Dev. Storage	4 feet of 2750 LF open cut pipe (shallow depth) on Alhambra with weirs on E St and H St	$ 14,740,000	$ 150,000	$ 3,722,500	$ 18,612,500	447,861
B	530,000 CF of McKinley Park Storage	3 feet pipe of 225 LF connecting from Park Way to the storage. Approx. 3.5 acres available for storage, deep storage required to eliminate low point on Park Way	$ 11,993,900		$ 2,998,475	$ 14,992,375	482,620
C	530,000 CF of Sutter Middle School Storage	Weirs from Alhambra Blvd to fill the storage. Approx. 4.2 acres of footprint available for a shallow storage	$ 11,993,900		$ 2,998,475	$ 14,992,375	536,096
D	530,000 CF of Stanford Park Storage	9 feet pipe of 900 LF connecting E St to a storage in Stanford Park area. The storage has to be deep enough to capture flows from low point on E St, approx. 2.3 acres available for storage	$ 11,993,900		$ 2,998,475	$ 14,992,375	385,027
E	Combo A3 + B + C	Combo A3 + B + C				$ 48,597,250	736,631
F	Combo B + C + D	Combo B + C + D				$ 44,977,125	739,305
G	Combo B + C	Combo B + C				$ 29,984,750	704,545
H	Combo A3 + B	Combo A3 + B				$ 33,604,875	616,310
I	Combo A3 + C	Combo A3 + C				$ 33,604,875	683,155
J	Combo B + D	Combo B + D				$ 29,984,750	562,834
K	1,062,000 CF of McKinley Park Storage	Majority of the storage will be built under the baseball field, with soccer field available for any storage not built under the baseball field footprint.	$ 23,987,800		$ 5,996,950	$ 29,984,750	709,893

*B, C, and D cost estimates based on Oak Park project of $22.63/CF with 25% contingency added

6 Summary and Conclusion

The planning-level cost estimates do not account for major relocation of other utilities (sewer, power, water, communications, etc), environmental impacts, pump station operating costs, etc. The purpose of the planning level costs is to allow comparison of different alternatives and select the best alternative(s) for further discussion on project feasibility and develop detailed design and cost estimates.

To summarize this analyses, the total planning-level project costs of the deep and shallow tunnel on Alhambra Blvd. are relatively higher than other storage projects. Among the project alternatives evaluated in this study; Alternative K – 1,062,000 CF storage facility at McKinley Park under the baseball and soccer fields is recommended. Alternative K provides the best flood reduction benefits for the total project cost compared to other storage projects in the area.

City of SACRAMENTO
Department of Utilities

Alhambra Alternatives

McKinley Park Village Storage Layout

Appendix A

Figure A-1: McKinley Park Village Storage, Aerial View

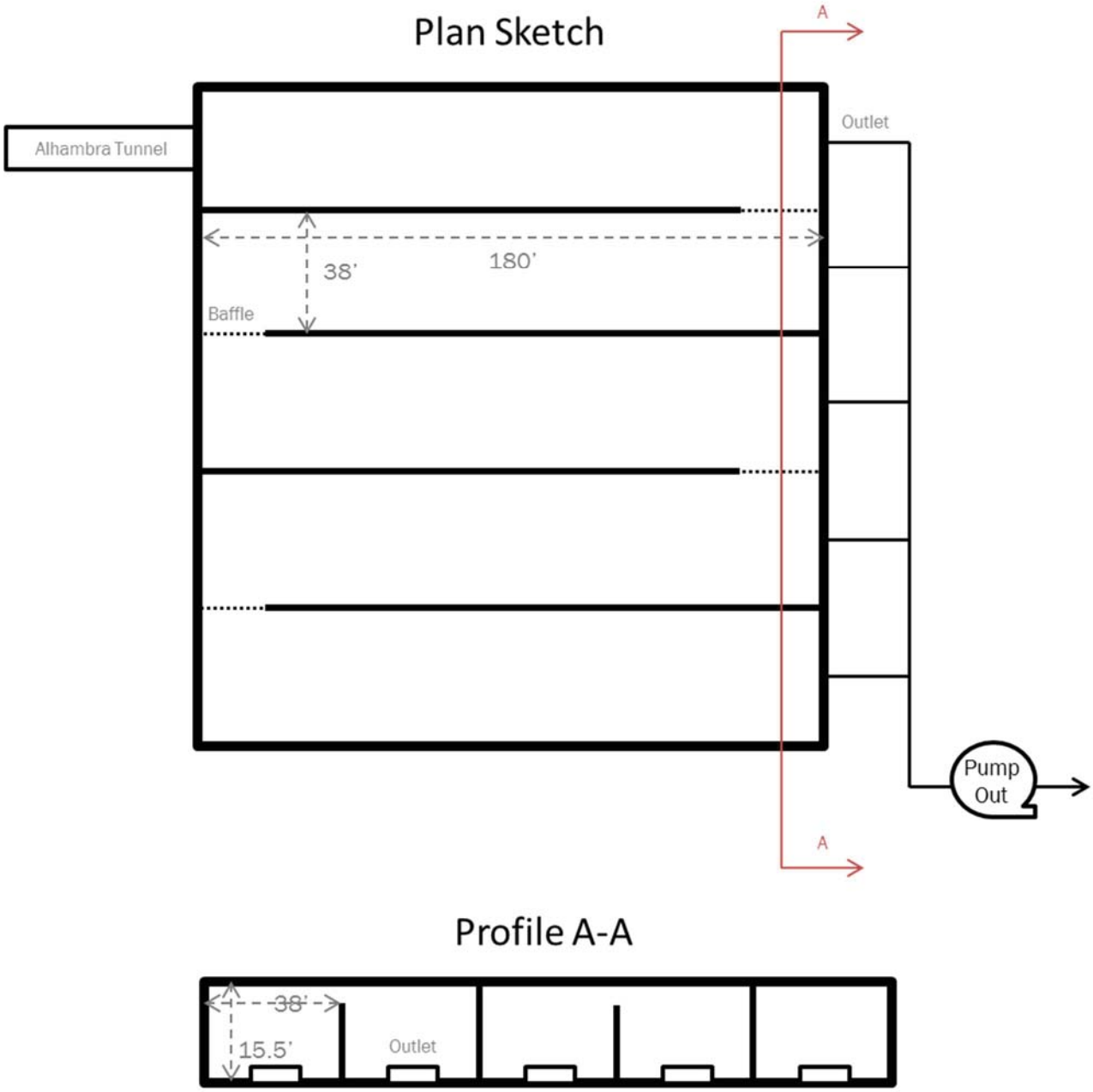


Figure A-2: McKinley Park Village Storage, Conceptual Layout

City of SACRAMENTO
Department of Utilities

Alhambra Alternatives

Cost Estimates

Appendix B

CITY OF SACRAMENTO
ALHAMBRA BLVD - CSO TUNNEL AND STORAGE PROJECT
CONCEPTUAL LEVEL CONSTRUCTION COST OPINION

DIRECT JACK RCP WITH EPBM

Date: 07/11/13
By: SH
Checked:

ITEM	DESCRIPTION	QTY.	UNIT	UNIT PRICE	TOTAL COST
1	Reinforced Concrete Storage Structure	530,000	CF	$ 23	$ 12,190,000
2	8 Foot Diameter RCP Tunnel	3000	LF	$ 4,500	$ 13,500,000
3	10 Foot Diameter RCP Tunnel	2800	LF	$ 4,800	$ 13,440,000
4	Boring and Jacking Pits (at approx 1000 foot spacing)	7	EA	$ 400,000	$ 2,800,000
5	Weir Structure and Connect to Tunnel	3	EA	$ 380,000	$ 1,140,000
6	Misc Site Work (utility relocation, paving, traffic control, etc)	LS	LS	$ 500,000	$ 500,000
	SUBTOTAL CONSTRUCTION COST				$ 43,570,000
	Contingency	25%			$ 10,892,500
	SUBTOTAL				$ 54,462,500
	Scale of Economy Factor	0%			$ -
	Market Factor	0%			$ -
	Escalation to Bid	0%			
	TOTAL CONSTRUCTION COST OPINION				$ 54,462,500

CITY OF SACRAMENTO
ALHAMBRA BLVD - CSO TUNNEL AND STORAGE PROJECT
CONCEPTUAL LEVEL CONSTRUCTION COST OPINION

Scenario A1 - 48 inch Dia, RCP Tunnel Work

Date: 09/06/13
By: SH
Checked:

ITEM	DESCRIPTION	QTY.	UNIT	UNIT PRICE	TOTAL COST
1	Reinforced Concrete Storage Structure	530,000	CF	$ 23	$ 12,190,000
2	48-Inch Diameter RCP (with 5 feet of earth cover)	2,750	LF	$ 450	$ 1,237,500
3	Dewatering (pipe trench)	2,750	LF	$ 50	$ 137,500
4	Traffic Control	2,750	LF	$ 30	$ 82,500
5	Asphalt Paving Repair	2,750	LF	$ 150	$ 412,500
6	Weir Structure at H Street (with shoring and dewatering systems)	1	EA	$ 340,000	$ 340,000
7	Misc Site Work (utility relocation, etc)	LS	LS	$ 150,000	$ 150,000
	SUBTOTAL CONSTRUCTION COST				**$ 14,550,000**
	Contingency	25%			$ 3,637,500
	SUBTOTAL				**$ 18,187,500**
	Scale of Economy Factor	0%			$ -
	Market Factor	0%			$ -
	Escalation to Bid	0%			
	TOTAL CONSTRUCTION COST OPINION				**$ 18,187,500**

CITY OF SACRAMENTO
ALHAMBRA BLVD - CSO TUNNEL AND STORAGE PROJECT
CONCEPTUAL LEVEL CONSTRUCTION COST OPINION

Scenario A2 - 48 inch Dia, RCP Tunnel Work

Date: 09/06/13
By: SH
Checked:

ITEM	DESCRIPTION	QTY.	UNIT	UNIT PRICE	TOTAL COST
1	Reinforced Concrete Storage Structure	530,000	CF	$ 23	$ 12,190,000
2	48-Inch Diameter RCP (with 5 feet of earth cover)	1,500	LF	$ 450	$ 675,000
3	Dewatering (pipe trench)	1,500	LF	$ 50	$ 75,000
4	Traffic Control	1,500	LF	$ 30	$ 45,000
5	Asphalt Paving Repair	1,500	LF	$ 150	$ 225,000
6	Weir Structure at E Street (with shoring and dewatering systems	1	EA	$ 340,000	$ 340,000
7	Misc Site Work (utility relocation, etc)	LS	LS	$ 150,000	$ 150,000
	SUBTOTAL CONSTRUCTION COST				$ 13,700,000
	Contingency	25%			$ 3,425,000
	SUBTOTAL				$ 17,125,000
	Scale of Economy Factor	0%			$ -
	Market Factor	0%			$ -
	Escalation to Bid	0%			
	TOTAL CONSTRUCTION COST OPINION				$ 17,125,000

CITY OF SACRAMENTO
ALHAMBRA BLVD - CSO TUNNEL AND STORAGE PROJECT
CONCEPTUAL LEVEL CONSTRUCTION COST OPINION

Scenario A3 - 48 inch Dia, RCP Tunnel Work

Date: 09/06/13
By: SH
Checked:

ITEM	DESCRIPTION	QTY.	UNIT	UNIT PRICE	TOTAL COST
1	Reinforced Concrete Storage Structure	530,000	CF	$ 23	$ 12,190,000
2	48-Inch Diameter RCP (with 5 feet of earth cover)	2,750	LF	$ 450	$ 1,237,500
3	Dewatering (pipe trench)	2,750	LF	$ 50	$ 137,500
4	Traffic Control	2,750	LF	$ 30	$ 82,500
5	Asphalt Paving Repair	2,750	LF	$ 150	$ 412,500
6	Weir Structures at E & H Street (w/shoring and dewatering systems)	2	EA	$ 340,000	$ 680,000
7	Misc Site Work (utility relocation, etc)	LS	LS	$ 250,000	$ 150,000
	SUBTOTAL CONSTRUCTION COST				$ 14,890,000
	Contingency	25%			$ 3,722,500
	SUBTOTAL				$ 18,612,500
	Scale of Economy Factor	0%			$ -
	Market Factor	0%			$ -
	Escalation to Bid	0%			
	TOTAL CONSTRUCTION COST OPINION				$ 18,612,500

	Project Name	Project #	Engr Cost Est	Final Budget	Year	Constructed Storage	$/CF	Effective Increase in Storage	$/CF	Benefit	Benefit/CF	$/Benefit
1	42nd Street Drainage Area Storage and Pump Station Impro	XM01		$ 4,005,011.00	1996	196,875	$ 20.34	196,875	$ 20.34	296,182	1.50	$ 13.52
2	21st Street Local Storage - Broadway to 2nd Ave	XM21	$ 1,247,275.00	$ 1,150,191.00	1997	22,816	$ 50.41	22,816	$ 50.41			
3	LAND PARK COMB SEWER RELIEF PROJECT	XM35	$ 2,435,027.07	$ 2,571,364.00	2000	60,132	$ 42.76	57,327	$ 44.85	see below	see below	see below
4	UCD MED CENTER REGIONAL STORAGE	XM44		$ 9,104,109.00	1998	377,280	$ 24.13	377,280	$ 24.13	see below	see below	see below
3&4	Land Park & Med Center			$ 11,675,473.00		437,412	$ 26.69	434,607	$ 26.86	426,990	0.98	$ 27.34
5	Tahoe Park/Broadway Parallel Sewer Project	XM46	$ 5,684,481.61	$ 6,452,997.00	2001	241,181	$ 26.76	201,302	$ 32.06	see below	see below	see below
6	East End Project Relief Sewer	XM63		$ 1,669,383.00	2001	39,600	$ 42.16	39,600	$ 42.16	see below	see below	see below
5&6	Tahoe Park/ Broadway & East End Project			$ 8,122,380.00		280,781	$ 28.93	240,902	$ 33.72	466,168	1.66	$ 17.42
7	U&S St. In-Line Storage Project	XM79		$ 5,865,068.00	2004	54,131	$ 108.35	54,131	$ 108.35	230,050	4.25	$ 25.49
8	7th Street H-K Sewer Replacement	XM97	$ 377,491.10	$ 884,117.00	2006	16,324	$ 54.16	12,243	$ 72.22			
9	N Street Sewer, 25th to 29th	XN05	$ 1,508,984.29	$ 1,508,348.00	2005	27,144	$ 55.57	27,144	$ 55.57			
10	R Street CSS Storage Project	XN27	$ 1,086,499.64	$ 1,152,039.00	2006	12,137	$ 94.92	12,137	$ 94.92			
11	Project 1 - 5th Street Sewer Upsizing, P St to U St	X14010032	$ 2,414,120.79	$ 2,399,342.94	2010	52,869	$ 45.38	48,816	$ 49.15	22,230	0.42	$ 107.93
12	S St. Brick Sewer Replacement - 7th to 14th	X14010013	$ 2,450,524.89	$ 2,507,438.67	2010	84,170	$ 29.79	51,995	$ 48.22	see below	see below	see below
13	S St. Brick Sewer Replacement - 14th to 17th	X14010036	$ 1,368,196.11	$ 1,027,129.63	2012	21,432	$ 47.93	6,955	$ 147.67	see below	see below	see below
12&13	Project 2 - S St. Combined			$ 3,534,568.30		105,601	$ 33.47	58,951	$ 59.96	112,579	1.07	$ 31.40
14	P Street Sewer Upsizing - 5th to 7th Street	X14010064	$ 1,129,873.11		2013	22,981	$ 49.17	22,981	$ 49.17	see below	see below	see below
15	7th Street from K to P Street	X14010063	$ 3,727,934.38		Design	69,332	$ 53.77	62,571	$ 59.58	see below	see below	see below
14&15	Project 3 - 7th St K to P and P St 5th to 7th		$ 4,857,807.49	$ -		92,313	$ 52.62	85,552	$ 56.78	64,363	0.70	$ 75.48
16	Project 5 - 3rd Street from I to R Street	X14010033	*$ 6,450,000.00*		prelim	60,118	$ 107.29	60,118	$ 107.29	10,236	0.17	$ 630.13
17	Project 6 - Oak Park Storage	X14010009	$ 11,978,775.85		2013	529,422	$ 22.63	529,422	$ 22.63	498,430	0.94	$ 24.03
18	Project 9 - 9th Street Sewer Replacement G to L Street	X14010042	$ 3,315,804.58		Design	41,822	$ 79.28	35,131	$ 94.38			
19	Project 4 - Curtis Park Regional Storage	X14010001	*$ 10,000,000.00*		Planning	325,000	$ 30.77	325,000	$ 30.77	208,731	0.70	$ 47.91
20	Project 7 - Alhambra Blvd. Storage		*$ 50,000,000.00*		prelim	906,507	$ 55.16	906,507	$ 55.16	1,070,841	1.18	$ 46.69
21	Project 8 - Downtown Sewer Upsizing - G & F Streets		*$ 2,000,000.00*		prelim	23,373	$ 85.57	23,373	$ 85.57	88,179	3.77	$ 22.68
	Average Cost			**$ 53,405,187.20**		**1,758,492**	**$ 30.37**	**1,661,024**	**$ 32.15**			