Intake Design

The function of the intake, whether it be an open channel or a tunnel having 100 per cent wetted perimeter, is to supply an evenly distributed flow of water to the suction bell. An uneven distribution of flow, characterized by strong local currents, favors formation of vortices and with certain low values of submergence, will introduce air into the pump with reduction of capacity, accompanied by noise. Uneven distribution can also increase or decrease the power consumption with a change in total developed head. There can be vortices which do not appear on the surface, and these also may have adverse effects.

Uneven velocity distribution leads to rotation of portions of the mass of water about a center-line called vortex motion. This centerline may also be moving. Uneven distribution of flow is caused by the geometry of the intake and the manner in which water is introduced into the intake from the primary source.

Calculated low *average* velocity is not always a proper basis for judging the excellence of an intake. High *local* velocities in currents and in swirls may be present in intakes which have very low *average* velocity. Indeed, the uneven distribution which they represent occurs less in a higher velocity flow with sufficient turbulence to discourage the gradual build-up of a larger and larger vortex in any region. Numbers of small surface eddies may be present without causing any trouble.

The ideal approach is a straight channel coming directly to the pump. Turns and obstructions are detrimental since they may cause eddy currents and tend to initiate deep-cored vortices.

Water should not flow past one pump to reach the next if this can be avoided. If the pumps must be placed in line of flow, it may prove necessary to construct an open front cell around each pump or to put turning vanes under the pump to deflect the water upward.

All possible streamlining should be used to reduce the trail of alternating vortices in the wake of the pump or of other obstructions in the stream flow.

The amount of submergence for successful operation will depend greatly on the approaches to the intake and the size of the pump. While specific design is generally beyond the scope of the pump manufacturer's responsibility, he may comment while the intake layout is still preliminary if he is provided with the necessary intake drawings reflecting the physical limitations of the site.

Complete analysis of intake structures is best accomplished by scale model tests.

Subject to the qualifications of the foregoing statements, Figs. 81, 82, & 83 have been constructed for single and simple multiple pump arrangements to show suggestions for basic sump dimensions. They are for pumps normally operating in the capacity range of approximately 3,000 to 300,000 gpm. Since these values are composite averages from a great many pump types and cover the entire range of specific speeds, they must not be thought of as absolute values but rather as basic guides subject to some possible variations. For pumps normally operating at capacities below approximately 3,000 gpm, refer to Sump or Pit Designs (small pumps) page 129.

All of the dimensions in Figs. 81, 82 & 83 are based on the rated capacity of the pump at the design head. Any increase in capacity above these values should be momentary or very limited in time. If operation at an increased capacity is to be undertaken for considerable periods of time, the maximum capacity should be used for the design value in obtaining sump dimensions.

The Dimension C is an average, based on an analysis of many pumps. Its final value should be specified by the pump manufacturer.

Dimension B is a suggested maximum dimension which may be less depending on actual suction bell or bowl diameters in use by the pump manufacturer. The edge of the bell should be close to the back wall of the sump. When the position of the back wall is determined by the driving equipment or the discharge piping, Dimension B may become excessive and a "false" back wall should be installed.

Dimension S is a minimum for the sump width for a single pump installation. This dimension can be increased, but if it is to be made smaller, the manufacturer should be consulted or a sump model test should be run to determine its adequacy.

Dimension H is a minimum value based on the "normal low water level" at the pump suction bell, taking into consideration friction losses through the inlet screen and approach channel. This dimension can be considerably less momentarily or infrequently without damage to the pump. It should be remembered, however, that this does not represent "submergence." Submergence is normally quoted as dimension H minus C. This represents the physical height of water level above the bottom of the suction inlet. The actual submergence of the pump is something less than this, since the impeller eye is some distance above the bottom of the suction bell, the magnitude being a function of pump size and style. For the purposes of sump design in connec-

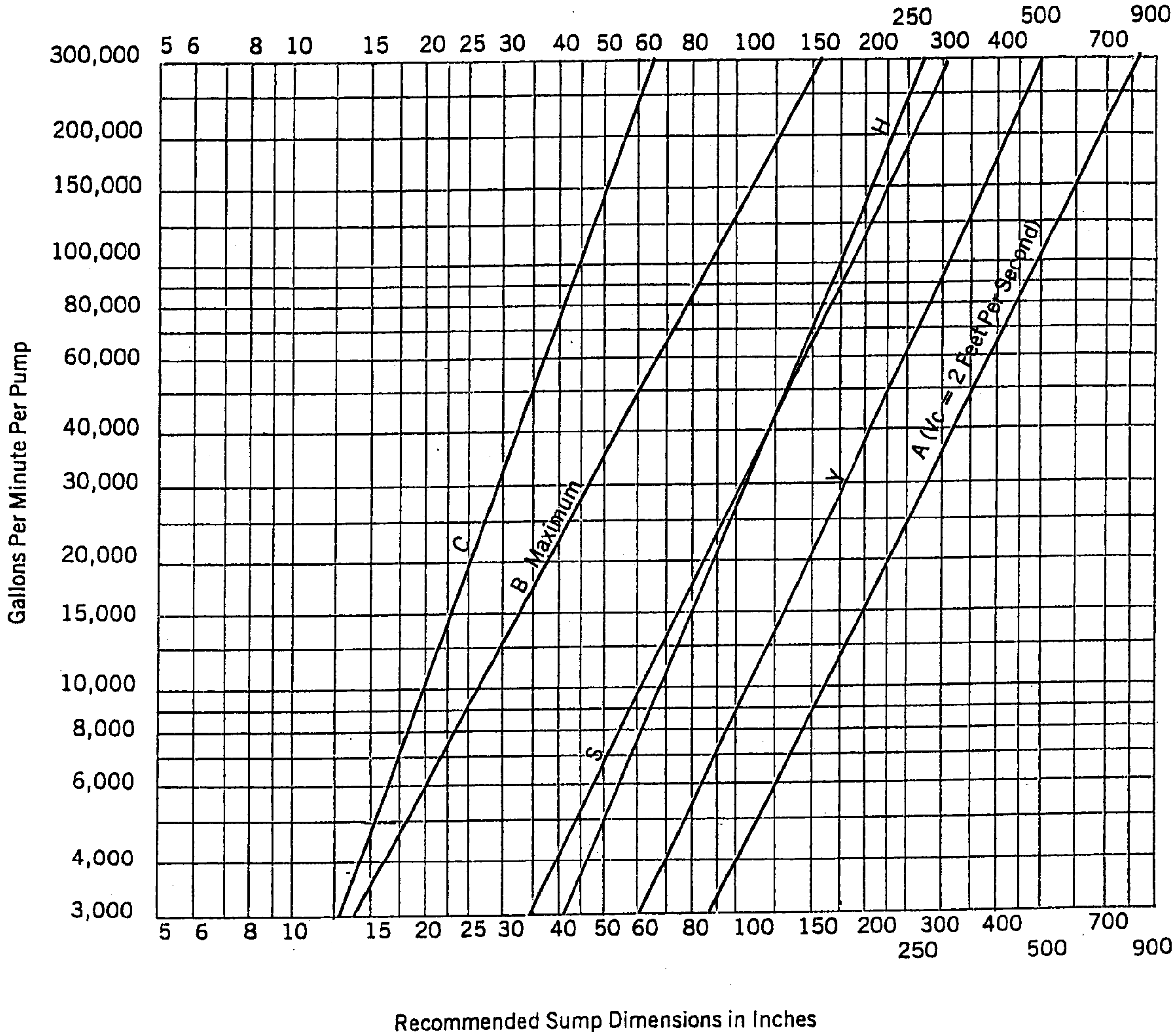


Fig. 81 SUMP DIMENSIONS VERSUS FLOW

Note: Recommended value of Y equals approximately 3D for most bell designs.

tion with this chart, it is understood that the pump has been selected in accordance with maximum speeds charts, Figs. 68 and 69, the submergence referred to herein having to do only with vortexing and eddy formations.

Dimensions Y and A are recommended minimum values. These dimensions can be as large as desired but should be limited to the restrictions indicated on the curve. If the design does not include a screen,

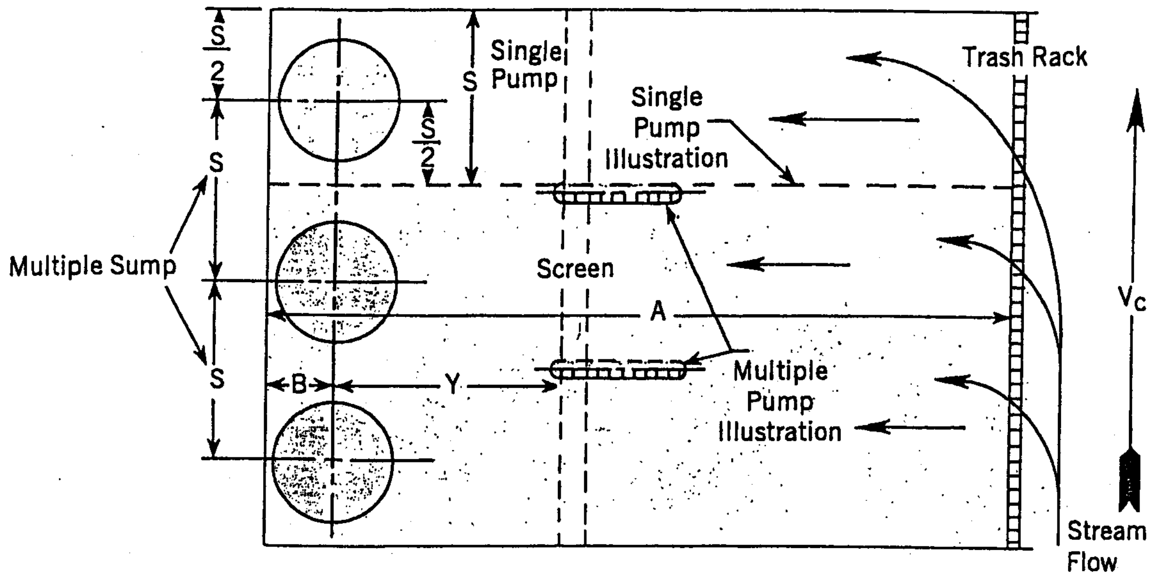


Fig. 82 SUMP DIMENSIONS PLAN VIEW

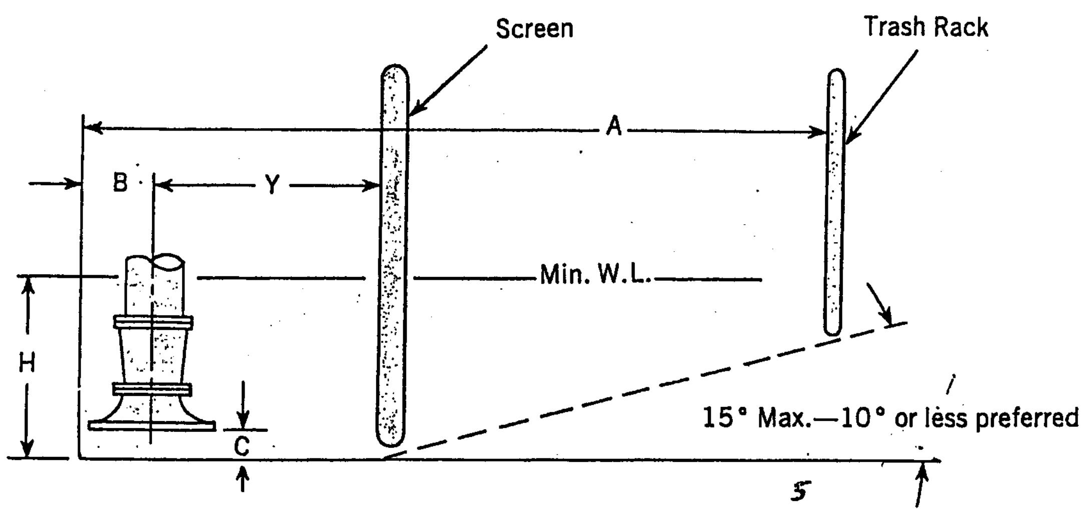


Fig. 83 SUMP DIMENSIONS ELEVATION VIEW

or if the channel has a sloping approach, dimension A should be considerably longer, even as much as twice the value shown. If the channel approach has a downslope, the angle should be not more than 15 degrees and preferably 10 degrees with the horizontal. The channel floor should be level for at least distance Y upstream before the slope begins. The screen or gate widths should not be substantially less than S, and heights should not be less than H. If the mainstream velocity is more than 2 feet per second, it may be necessary to construct straightening vanes in the approach channel, increase dimension A, conduct a sump model test of the installation, or work out some combination of these factors.

Dimension S becomes the width of an individual pump cell or the center-to-center distance of two pumps if no division walls are used.

On multiple pump installations, the recommended dimensions in Figs. 81, 82 and 83 also apply as noted above, and the following additional determinants should be considered.

Fig. 84A. Low velocity and straight-line flow to all units simultaneously is the first recommended style of pit. Velocities in pump area should be approximately one foot per second. Some pumps with velocities of 2 feet per second and higher have given good results. This is particularly true where the design resulted from a model study. Not recommended would be abrupt change in size of inlet pipe to sump or inlet from one side introducing eddying.

Fig. 84B. A number of pumps in the same sump will operate best without separating walls unless all pumps are always in operation at the same time, in which case the use of separating walls may be beneficial. If walls must be used for structural purposes, and pumps will operate intermittently, leave flow space behind each wall from the pit floor up to at least the minimum water level.

If walls are used, increase dimension S by the thickness of the wall for correct centerline spacing. Round or "ogive" ends of walls. NOT recommended is the placement of a number of pumps around the edge of a sump with or without dividing walls.

Fig. 84C. Abrupt changes in size from inlet pipe or channel to pump bay are not desirable. A relatively small pipe emptying into a large pump pit should connect to the pit with a gradually increasing taper section. The angle should be as small as possible, preferably not more than 15 degrees. With this arrangement, pit velocities much less than one foot, per second are desirable. Especially not recommended is a small pipe directly connected to a large pit with pumps close to the inlet. Flow will have excessive change of direction to get to most of the pumps. Centering pumps in the pit leaves large "vortex areas" behind the pumps which will cause operational trouble.

Fig. 84D. If the pit velocity can be kept low enough (approximately one foot per second), an abrupt change from inlet pipe to pit can be accommodated if the length equals or exceeds the values shown. As ratio W/P increases, the inlet velocity at p may be increased up to an allowed maximum of eight feet per second at $W/P = 10$. Pumps "in line" are not recommended unless the ratio of pit to pump size is quite large, and pumps are separated by a generous margin longitudinally. A pit can generally be constructed at much less cost by using a recommended design.

Fig. 84E. It is sometimes desirable to install pumps in tunnels or pipe lines. A drop pipe or false well to house the pump with vaned inlet elbow facing upstream will be satisfactory in flows up to eight feet per second. Without the inlet elbow, the pump section bell should be positioned at least two pipe (vertical) diameters above the top of the tunnel, not hung into the tunnel flow, especially with tunnel velocities of two feet per second or more. There should be no signs of air along the top of tunnel. It may be necessary to lower the scoop or insist on minimum water level in vertical well.

Note: The foregoing statements apply to sumps for clear liquid. For fluid-solids mixtures refer to the pump manufacturer.

CORRECTION OF EXISTING SUMPS

It is well established that vortexing in pump suction pits is harmful to pumps and intake structures. It is equally true that a very small force will actually begin generating a vortex. While this phenomenon can be avoided in a new design, for existing structures where problems are already apparent or where expansion is required, corrective measures may be necessary. Possible revisions to correct particular sump problems are shown in Fig. 85. In many cases, field modifications are expensive with no guarantee of success. It is recommended that a sump model test be considered to prove the effectiveness of the proposed changes.

Fig. 85A—Reduce inlet velocity by spreading the inflow over a larger area, or change the direction and velocity of inflow by suitable baffling. (The baffle may be floor mounted, extending above the minimum flow level, or may be hung from above, extending close to the floor.)

Fig. 85B—Change the location of pumps in relation to the inflow. A suitable baffle may be necessary in front of inlet.

Fig. 85C—A cone may be added to reduce the possibility of submerged vortex formation.

Fig. 85D—Provide break-thru to "no-flow" bays in multiple pump pits and round or "ogive" ends of separating walls, or

Fig. 85E—Eliminate separating walls.

Fig. 85F—Eliminate sharp corners at gates, screens, etc., by filling in for smooth flow contour (fairing).

Fig. 85G—Reduce the velocity of flow and eliminate vortexing by adding bell extension suction plate and splitter to pump bell. Splitter must be in line with the flow.

Fig. 85H—Use floating rafts around the pump column to prevent surface vortices.

Fig. 85I—Use large spheres to prevent surface vortices.

Fig. 85J—Reduce the clearance between the pump inlet and back wall. This will improve velocity pattern to the pump to reduce the possibility of vortex formation.

Fig. 85K—Change inlet flow direction gradually by means of parallel turning vanes.

In General:

Keep inlet flow below two ft per second.

Keep flow in pit below one ft per second.

Avoid changing direction of flow from inlet to pump, or

Change direction gradually, smoothly, independently.

In addition to the above, other modifications such as 1) Suction Cone; 2) Horizontal perforated plate, or grids; 3) Horizontal beam with bottom flange submerged to control water surface, and 4) other flow straighteners may be used to correct the existing sumps.

Any of these alterations, singly or in combination, may help to create a better flow pattern in the sump. If troubles persist, it may be necessary to limit the total flow or change pump size and speed.

Model Tests of Intakes

Often the analysis of a proposed intake design can only be made by use of a scale model of the intake. The engineers responsible for the design of the pumping station should consult with pump manufacturers to establish one or more intake arrangements. A sump model test can then be conducted by an independent laboratory or by the pump manufacturer. The sump model test may show modifications of structure or baffling arrangement to be necessary, and sometimes sump model tests show how considerable savings can be made in the intake structure. The model should be extensive enough to include all parts of the channel likely to affect the flow near the pump, including screens and gates.

Deviations may occur between model and prototype, since all considerations of similarity cannot be produced simultaneously. Consequently, the range of levels in velocities to be explored should be as broad as possible in order to disclose any markedly unfavorable tendencies which might only be incipient at mathematically analogous conditions.

Comparable flow in the model is generally considered to be obtained at equal Froude numbers.

On this basis,

$$V_m = V_p \times \sqrt{R}$$

where

V_m = Velocity of water in the model
V_p = Velocity of water in the prototype
R = Linear scale ratio of model to prototype

or

$$\frac{L_m}{L_p}$$

where

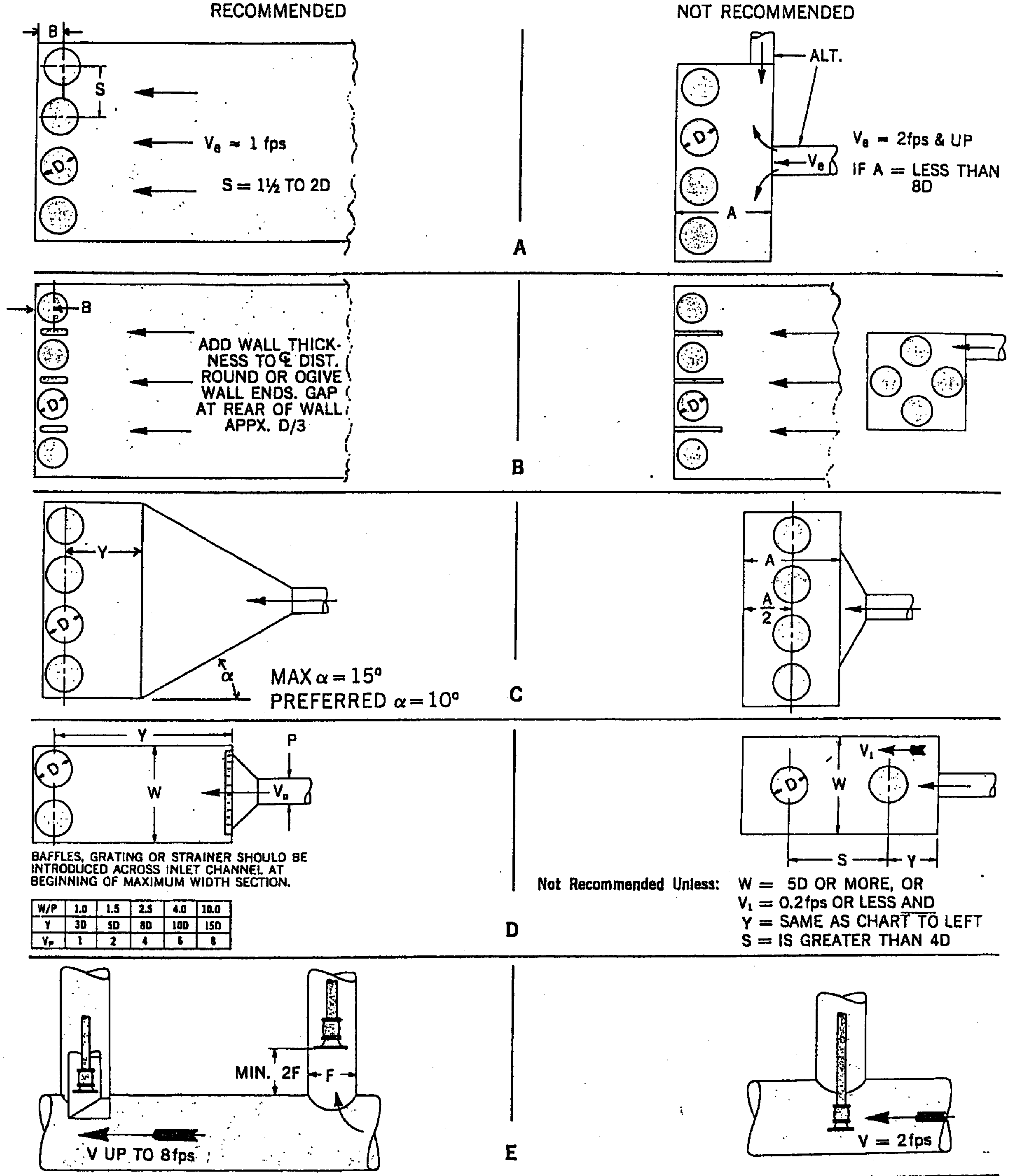
L_m = Any linear dimension of the model
L_p = The dimension on the prototype corresponding to any dimension L_m on the model.

Several investigators have found better agreement between model and prototype when velocities are equal than when velocities are in accord with the Froude number. In the present stage of the art, caution suggests that this entire range of velocities be explored in the model test.

Sump or Pit Design (Small Pumps)

The design of sumps for small pumps (less than approximately 3,000 gpm, normal discharge capacity per pump) should be guided by the same general principles as outlined.

However, since there is a large variety of geometric configurations for these small units, recommended



W/P	1.0	1.5	2.5	4.0	10.0
Y	3D	5D	8D	10D	15D
V_P	1	2	4	6	8

The Dimension D is generally the diameter of the suction bell measured at the inlet. This dimension may vary depending upon pump design. Refer to the pump manufacturer for specific dimensions.

Fig. 84 MULTIPLE PUMP PITS

Note: Figures apply to sumps for clear liquid. For fluid-solids mixtures refer to the pump manufacturer.

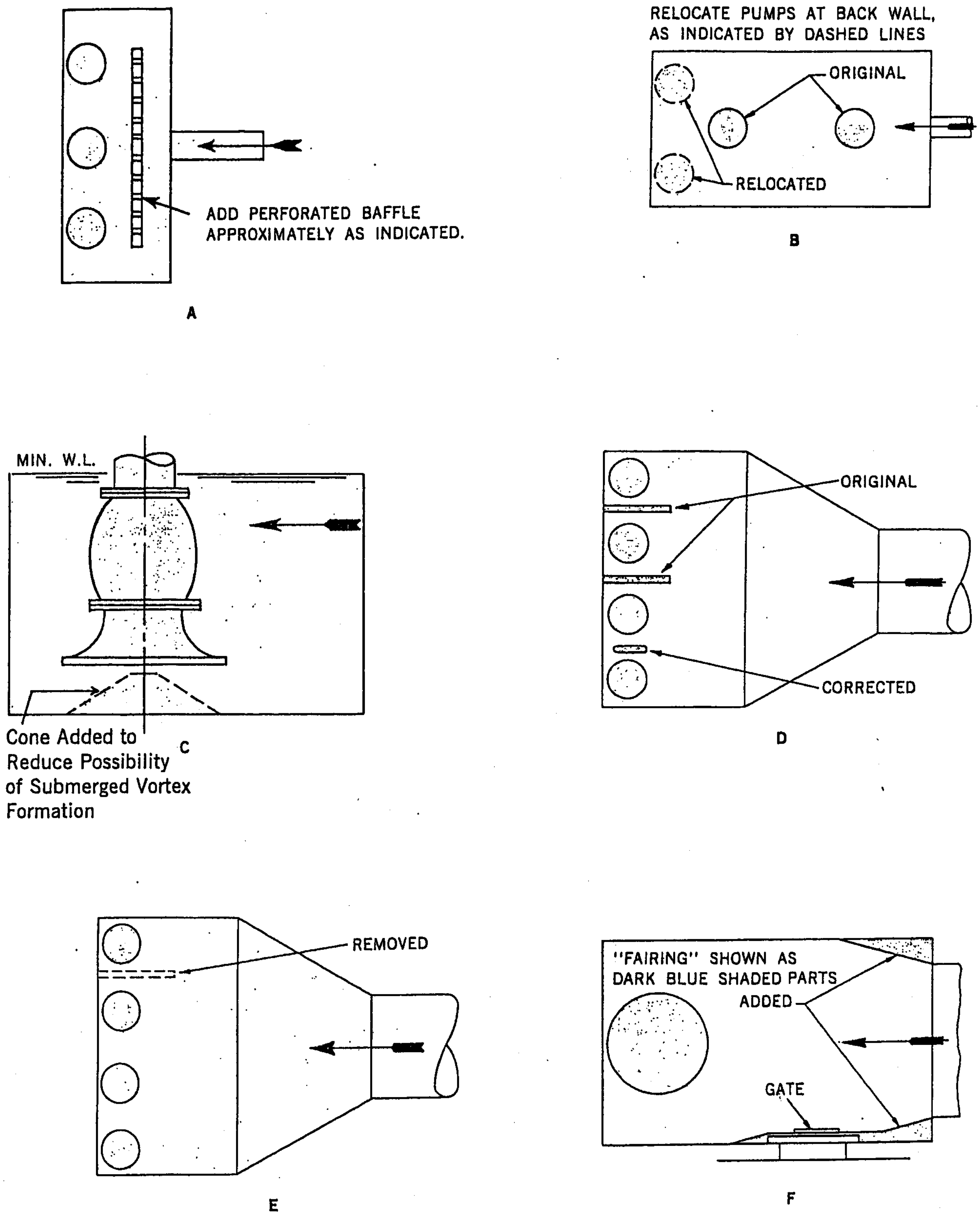
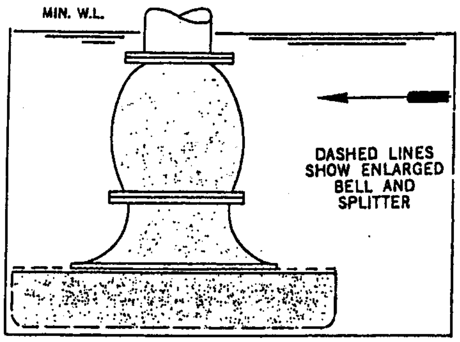


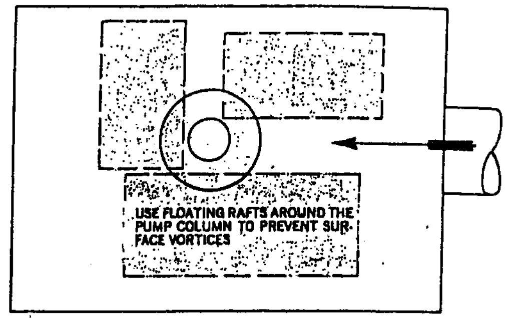
Fig. 85 CORRECTION OF EXISTING SUMPS (PART ONE)

Note: Figures apply to sumps for clear liquid. For fluid-solids mixtures refer to the pump manufacturer.



G

SPLITTER MUST BE IN-LINE WITH FLOW.
SPLITTER IS TO PREVENT SUBMERGED VORTEXING



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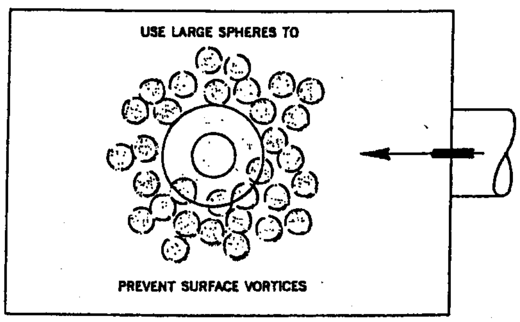

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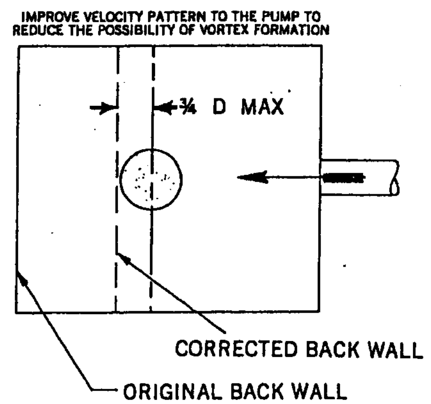

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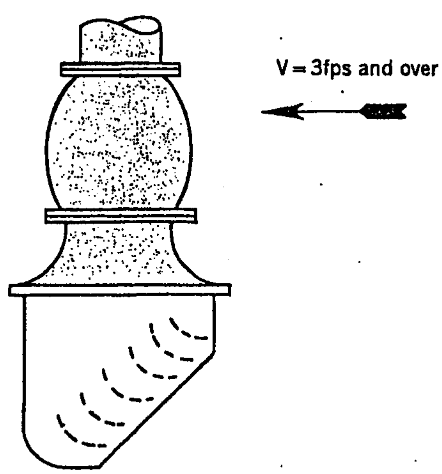

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Fig. 85 CORRECTION OF EXISTING SUMPS (PART TWO)

Note: Figures apply to sumps for clear liquid. For fluid-solids mixtures refer to the pump manufacturer.

limiting dimensions, such as shown in Fig. 81–83, cannot be sufficiently generalized and so presented. Where specific pit or sump dimensions are required, the manufacturer's recommendations should be requested.

In addition to the general design principles outlined, for single and multiple pump settings in large sump designs, the following factors are pertinent to the design of small sumps or pits:

Inlet Opening (Pit Type Sumps)

The sump inlet should be below the minimum liquid level, and as far away from the pump as the sump geometry will permit. The influent should not impinge against the pump, jet directly into the pump inlet, or enter the pit in such a way as to cause rotation of the liquid in the pit. Where required, a distribution nozzle can be used to prevent jetting, and baffling can be used to prevent rotation.

Sump Volume (Pit Type Sumps)

The usable pit volume should equal or exceed the maximum capacity to be pumped in two minutes. If units operate on float switch control, pit should be sized to allow no more than three or four starts per hour per pump. These guides generally insure pits of adequate size to dissipate the inflow turbulence and to assure reasonable life of the starting equipment.

Minimum Liquid Level

Minimum liquid level should be adequate to satisfy the particular pump design. The pump manufacturer's specific dimensions should be used.

SECTION B

FLOOD CONTROL PUMPING

Section B

Flood Control Pumping

B-1 Scope of Flood Control Pumping

The main flood control pumps are generally propeller, mixed flow and turbine pumps. Most pumps are in the low lift category and are in the specific speed range of from 5,000 to 13,000. For the purposes of this manual, single stage propeller pumps are used to about 20 feet of head, mixed flow pumps to about 60 feet of head and turbine pumps and multistage mixed flow pumps above 60 feet of head.

For the usual low-lift flood control application carrying debris and trash, the mixed flow or propeller pumps are the most efficient machines for the application. As long as an adequate trash rack is employed, the large open passageways through the bowls allow them to handle most trash.

The submerged propeller or mixed flow pump is always primed, while the driver is up at the motor room floor where it cannot be "flooded out". The vertical open suction method of installation requires minimum floor space.

B-2 Pumping Terminology

B-2.1 General

Table B-1 defines the symbols and units of various pumping and hydraulic terms which will be discussed in this manual.

B-2.2 Pump Capacity

The capacity of a pump is expressed in cubic feet per second or gallons per minute. One cubic foot per second is equivalent to 448.8 gallons per minute. The capacity is used to determine the brake horsepower required at the pump shaft. Refer to Section C-5.2.

B-2.3 Total Dynamic Head

The term Total Dynamic Head (TDH) consists of the sum of four items:

Static Head. This is the height that the water will be raised. The design static head will be the minimum value. If the pump discharges into a pressure line, the pressure at this point must be added to the "water-to-water" head.

B-2 Pumping Terminology continued.
B-2.3 Total Dynamic Head

Friction Head. The friction head includes all losses in the discharge line such as pipe friction, valve and bend losses, junction losses, transition losses and exit losses.

Velocity Head. This is the kinetic energy the water possesses due to its flow. It is equal to $V^2/2g$. The velocity is taken at the pump discharge elbow.

Column and Elbow Losses. The column loss can be calculated from a pipe friction formula while the elbow loss is found from pump manufacturer's charts.

The Total Dynamic Head defined in this manual is the same as the "Laboratory Total Dynamic Head" or "Bowl Assembly Head" shown on manufacturer's pump performance curves.

B-2.4 Pump Efficiency

The efficiency of a pump must be accounted for in the calculation of pump horsepower. Bowl efficiencies range from 65 to 90 percent in typical flood control applications.

TABLE B-I

Pumping Nomenclature

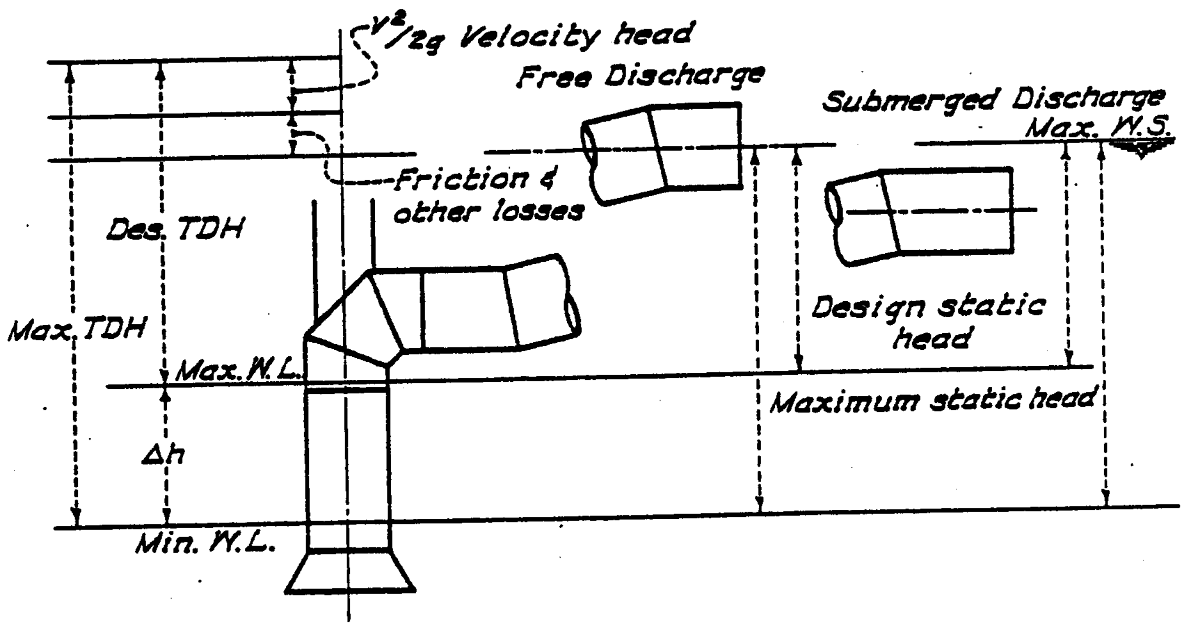
Symbol	Term	Units
TDH	total dynamic head	ft. of water
h_s	static head	ft. of water
h_f	friction head (losses)	ft. of water
h_v	velocity head (losses)	ft. of water
h_p	column and elbow loss	ft. of water
f	friction factor	-----
L	length of pipe	feet
D	inside diameter of pipe	feet
V	velocity in pipe	feet per second
g	acceleration of gravity	feet per second^2
ρ	weight density of water	lb. per ft^3
μ	absolute viscosity	centipoise
R_e	Reynold's number	-----
A	cross-sectional area of pipe	ft^2
Q	flow	cfs
R	hydraulic radius	feet
K	loss factor	-----
q	flow	gpm
L_e	equivalent length of pipe	feet
n	Manning friction factor	-----
η	pump efficiency	-----

B-3 Design Conditions

The inflow to the pump station will be determined by procedures dictated in the District's Hydrology Manual.

The total dynamic head at which a pump is rated should be calculated using the maximum water level for calculating static head.

A pump discharging into a manifold will have various friction heads depending on the number of pumps in operation. Each pump TDH should be calculated in a manner so that maximum TDH will be obtained.



B-4 System Head Capacity Curves

B-4.1 General

In order to best analyze a pumping condition it is advantageous to compare the system curve with various pump curves. In this manner a pump may be selected which will give best overall performance, considering capacity, efficiency and horsepower.

B-4 System Head Capacity Curves continued.

B-4.1 General

The head that the pump has to overcome for each particular pump flow is given by the Head-Capacity curve of the system, called the system curve. The head at each flow is the sum of the static head, column and elbow losses, and the friction and velocity head losses. Such a system curve should be plotted for both minimum and maximum static heads in order to check pump operation at all conditions.

Section B-2.3 gives the components of Total Dynamic Head. Thus the head at any pump condition can be written as

$$TDH = h_s + h_f + h_v + h_p$$

B-4.2 Maximum Static Head

The pump must be able to develop enough head so that the intersection of the pump curve and system curve based upon maximum static head (operating point) is within the normal operating range of the pump according to the manufacturer's published performance curve. This is the point where maximum power is required in most cases (for a propeller pump) and the driver should be sized for this operating condition.

B-4.3 Minimum Static Head

The capacity of the pump station is greatest at the minimum static head (maximum water level). The pump curve must intersect the system curve reflecting minimum static head at the rated capacity of the pump. Superimposing published pump performance curves over the system curve will define a pump which satisfies this requirement. At this point of minimum head, the horsepower required will be a minimum for propeller and mixed flow pumps, in most cases.

B-4.4 Friction Head

The friction losses in the discharge pipeline can be calculated by various methods. The Darcy equation for head loss due to friction is

$$h_f = f \frac{L}{D} \frac{V^2}{2g}$$

The terms are defined in Table B-1.

B-4 System Head Capacity Curves continued.
B-4.4 Friction Head

The friction factor can be obtained from the Moody diagram shown on Plate B-1. For laminar flow the friction factor is given by

$$f = 64/R_e$$

For turbulent flow (Re>2000) the relative roughness $\left(\frac{e}{D}\right)$ and Reynold's number will be needed to find the friction factor. The Reynold's number can be calculated from the following equation:

$$R_e = 50.6 \frac{q\rho}{d\mu}$$

Relative roughness values for various materials are found on Plate B-2.

The friction loss may also be computed from the Manning formula

$$h_f = L\left[\frac{Q \times n}{1.486\ AR^{2/3}}\right]^2$$

Some values of "n" are

Concrete pipe	n = .013
Steel pipe	n = .013
Corrugated metal pipe	See District's Hydraulic Design Manual

Bends and other irregularities in the discharge line add minor losses to the energy grade line. These minor losses are sometimes expressed as a multiple of velocity head ($KV^2/2g$). Losses calculated in this manner are added directly to other losses to determine the pumping head. Some typical pipeline losses are shown in Plate No. B-3. Flap gate losses are shown on Plate No. B-6. The District's Hydraulic Design Manual sets forth other losses.

Whenever the discharge lines from a pump station exceed 100 feet in length or are manifolded to a single line, the procedures set forth in the District's Hydraulic Design Manual (Manning formula) shall govern and must be used to determine the friction loss.

B-4 System Head Capacity Curves continued.

B-4.5 Velocity Head

The velocity head is the energy due to water flowing. This head must be developed by the pump so it must be added to other heads to obtain Total Dynamic Head. The velocity head is equal to $\frac{V^2}{2g}$.

B-4.6 Column and Elbow Losses

The normal range of velocity head is 0.5 to 1.5 feet. The elbow size which will yield a velocity head in this range can be found from Plate B-4.

The column and elbow losses are often small but should not be ignored. The loss for a 3-piece elbow can be found from Plate B-5. The column loss can be calculated in the same manner as the pipe friction loss, i.e., Darcy equation, but should be neglected for pump settings less than 25 feet.

B-5 System Curve Equation

The general equation for pump head may now be expressed as follows:

$$TDH = h_s + f\left[\frac{L_e}{D}\right]\left[\frac{V^2}{2g}\right] + \Sigma K \frac{V^2}{2g} + \frac{V^2}{2g} \quad + \text{minor losses}$$

ΣK is the total sum of all the K factors for each item in the system which produces a friction loss.

The above equation can be written in terms of Q as follows:

$$TDH = h_s + \frac{Q^2}{2gA^2}\left[f\left(\frac{L_e}{D}\right) + \Sigma K + 1\right] \quad + \text{minor losses}$$

For a given static head, the system curve is readily plotted by substituting arbitrary values of Q in this equation.

SECTION C

PUMP SYSTEM DESIGN

Section C

Pump System Design

C-1 Design Concept

Before the designer of the pump station commences his work, the District will determine and establish the design concept and criteria that he shall follow. The items that will be furnished by the District are as follows:

a. Design Capacity

The District will determine how much inflow (Q) can arrive at any proposed pump station from the contributory drainage area and storm drain system.

b. Standby Requirements

If the tributary drainage area of the pump station is subject to property damage or danger to life in the event of flooding, additional pumping capacity may be required. The District will analyze and determine if additional pumping units will be used.

c. Number of Pumps

The District will establish the number of the pumps in each station. In general, the minimum number of pumps required, excluding the cleanout pump (sump pump), will be three.

d. Types of Drivers

Economic, environmental, and safety factors will be considered by the District in establishing the type of drivers to be utilized to operate the pumps (i.e. engines or electric motors).

C-2 Capacities of Pumps

C-2.1 Main Pumps

With the size of the pump station (capacity) and the minimum number of pumps known, the pump sizes can be determined.

It is desirable to have identical pumps so that maximum use of each pump can be obtained. For this case the station size would be a minimum. The available storage must be studied to see if this scheme can be employed.

For a pump station draining a large open retention basin, equal capacity, engine-driven pumps can be employed. It is not necessary to provide a lesser capacity pump for low flows since the basin is allowed to fill until adequate storage is available for the first pump.

For an underground storm drain system discharging into the station, the available storage is not as great as an open retention basin. If equal capacity pumps are to be employed, the capability to pump low flows must be checked. The pump handling low flows will usually be electric motor driven for ease of automation and the cycling criteria of Section C-2.2 must be followed. If the required operating range for this pump governs the sump depth, it may be desirable to lower the pump capacity to obtain a more economical sump.

C-2.2 Pump Cycling

One important condition to consider occurs at a pump station with no appreciable storage (no open retention basin) with electric motor pump drives in automatic operation. When the water level rises to the first main pump start point, pumping will begin until the stop elevation is reached. With only sump storage considered, the pump "on" time could only be a few seconds. If the inflow fills the sump again in a few minutes the pump will start again and will thus cycle several times an hour. The resultant motor overheating will seriously affect motor winding insulation and shorten motor life.

C-2 Capacities of Pumps continued.

C-2.2 Pump Cycling

The limiting factor is the number of times per hour the electric motor can be started without overheating. The time interval between two consecutive starts of a motor will be designated as ΔT.

For Code F induction motors the following times are recommended:

Motor HP	Cycling Time (ΔT), minutes
0-200	15
250-300	18
350-500	20

A shorter cycling time may require a special motor be utilized.

The relationship between inflow (Q_I) and pump outflow (Q_P) is given by

$$Q_P = \frac{Q_I^2}{Q_I - \frac{A \cdot \Delta h + V}{\Delta T \cdot 60}} \qquad \text{Equation C-1}$$

where: Q_I = inflow, cfs
Q_P = pump capacity, cfs
A = sump area, ft^2
Δh = difference between pump start and stop points, feet.
ΔT = time between starts, minutes
V = usable storage in inlet pipe, ft^3

The amount of usable storage (V) in an inlet line depends on line size, slope, pumping rate. The usable storage is the volume of water in the line above the depth of flow for an inflow of $Q_I = Q_P/2$.

C-2 Capacities of Pumps continued.

C-2.2 Pump Cycling

By differentiating Equation C-1 with respect to Q_i the maximum value for Q_p is found to be

$$Q_p = 2\,Qi \qquad \text{or} \qquad Qi = Q_p/2$$

This means that for a given pump with a capacity of Q_p, cycling will be a maximum (least time between starts) when the inflow is one-half the pump capacity. It is therefore necessary to provide enough pumping range at this critical inflow rate to limit cycling to the times previously recommended in this paragraph.

At the maximum cycling condition of $Q_i = Q_p/2$ the pumping range is given by

$$\Delta h = \frac{(15\ Q_p \cdot \Delta T) - V}{A}$$

The required operating range can be calculated from this equation when all other terms are known. This operating range must be provided for the electric-driven pumps operating during the low inflow periods of a storm.

The low flow periods of a storm can be defined by studying an inflow hydrograph which represents the inflow in c.f.s. during the fourth day of a 50-year frequency storm.

The inflow hydrograph for the Los Angeles Basin in most cases will be as shown in Plate No. C-1.

From the hydrograph three distinct phases are recognizable.

Phase I lasts approximately 90 percent of the time and has a maximum inflow of 12 percent of the storm peak.

Phase II lasts approximately 7 percent of the time and has a maximum inflow of 30 percent of the peak inflow.

Phase III lasts only 3 percent of the time but has a maximum inflow of 100 percent of the peak.

C-2 Capacities of Pumps continued.

C-2.2 Pump Cycling

The pumping system must be designed so that the pumps handling the flows during Phases I and II do not cycle more frequently than recommended. In other words, 30 per cent of the 50-year peak inflow must be pumped without excessive cycling of the motors. This requirement may dictate an increase in the number of pumps if the necessary operating range and resultant sump depth becomes excessive.

Most pumping stations will require at least two pumps to handle the flows during Phases I and II.

C-2.3 Sump Pump

The sizing of the sump pump is done after the sump has been designed. Once the sump area is known, the volume of water in the sump below the lowest pump suction bell can be calculated. The sump pump should be capable of pumping this volume in approximately one-half hour. Usual sump pump capacities range from 500 gpm to 1000 gpm.

Since it is desirable to limit the sump pump motor size to about 30 HP, the one-half hour limit may be exceeded on extremely large sumps. Where the pumping time exceeds one and one-half hours, more than one pump may be required.

C-3 Pump Selection

After the pump sizes have been established, pump selection is made by superimposing the system curve for each pump over manufacturer's published pump curves. In most cases only one pump will best be suited for a specific application.

When the design point falls between two pump curves, this indicates that the standard impeller will have to be trimmed to meet the design conditions. A line drawn parallel to the family of curves through the design point will indicate the trimmed impeller curve.

C-4 Sequence of Operation

After the pump sizes have been established the sequence of pump operation can be established. Pumps should come into operation in order of increasing capacity. The effect of the sump pump can be neglected as a high-level cutoff will stop this pump when the need for increasing main pump operation is apparent.

C-4 Sequence of Operation continued.

The last pump should be started when the maximum water surface level is reached. The "on" points of the pumping units should be at least one foot apart. The "off" points should be at least 6 inches apart.

The distance, Δh, between the "on" and "off" points of a given pump must be such that the cycling limitations in Section C-2.2 are met for pumps handling 30 per cent of the 50-year peak inflow. The operating range of the remaining pumps should not be less than two feet.

Sample calculations in Section S illustrate the procedure for determining pumping ranges and operating sequences.

C-5 Driver Selection

C-5.1 Type

The type of drivers (engine vs. electric motors) selected by the District will reflect the importance of the station in terms of protection against loss of life and damage to property. The greater initial cost of engines is negated by yearly standby costs for electrical power so the main consideration should be the effect of loss of pumping due to a local power failure. Most pumping stations will have both types of drivers, the electric driven pumps operating the smaller pumps in an automatic mode with the larger units engine driven and started manually. In some installations where the station may not be manned during a storm, the engines may be operated automatically also.

The District will be the final authority on driver selection for any station design under its jurisdiction.

C-5.2 Size

The horsepower required to drive the pump unit is calculated as follows:

$$BHP = \frac{GPM \times TDH}{3960 \times \eta}$$

where BHP = brake horsepower required
GPM = pump capacity at given TDH
TDH = total discharge head, ft.
η = bowl efficiency

The brake horsepower should be calculated at the maximum, design, and minimum TDH and a curve plotted if necessary to find the point of maximum horsepower.

C-5 Driver Selection continued.

C-5.2 Size

The brake horsepower required by the pump must be adjusted when selecting driver size to account for losses in the pumping system. Some factors affecting pumping systems are given in detail in Section G-2.3. Sections G and J contain criteria for sizing engine and electric motor drives.

The shutoff horsepower must be calculated and used as the maximum horsepower required where the pumps operate against a closed valve or a check valve in a manifold system. This horsepower is developed as the pump slowly accelerates a long line full of water.

C-6 Discharge Configurations

The discharge line conveys pumped water from the pump discharge elbow to a channel or ocean discharge. The simplest discharge configuration occurs when each pump discharges into a nearby channel. The discharge line terminates in flap gate to prevent flowback into the sump if the invert of the discharge line is below the maximum water surface elevation in the channel.

When the discharge line terminates at a great distance from the pump station it is advantageous to manifold the pump discharge lines into one large line which is sized to carry the total station capacity. Check valves are needed in each discharge line to prevent recirculation. Plate C-2 shows a typical manifold system.

A special problem occurs when a portion of the discharge line of a manifold system design has an elevation above the pump discharge elbow. When pumps are shut down the flow reversal toward the pumps will cause dangerous water hammer surges if regular check valves are used to stop the flow. The discharge line check valves should have a slow-closing feature to prevent this from happening. The pump will operate at shutoff head for a few seconds on start-up when water is in the line. The shutoff horsepower must be considered when sizing the drivers for such an installation.

SECTION D

SUMP DESIGN

Section D

Sump Design

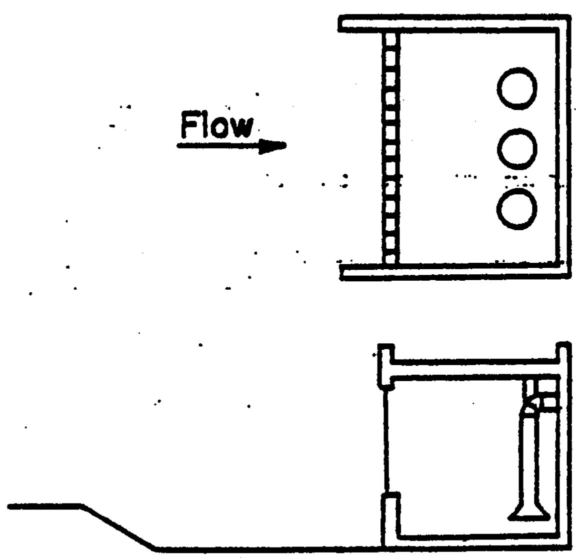
D-1 Intake Criteria

D-1.1 Open Retention Basin

The topography of the drainage area may allow the use of a large open sump or retention basin in the station design. When such basin storage is considered, the design capacity of the pump station will be reduced by the District from the Capital flood "Q".

The retention basin design will be dependent on the intended use of the basin (i.e. public park). Thus the establishment of design criteria for the basin is beyond the scope of this manual and will be determined for each case.

The flow to a station draining an open retention basin is shown below.

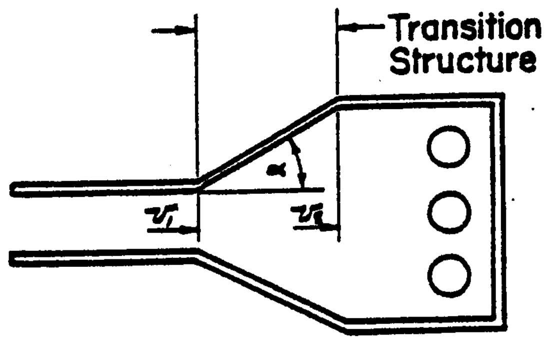


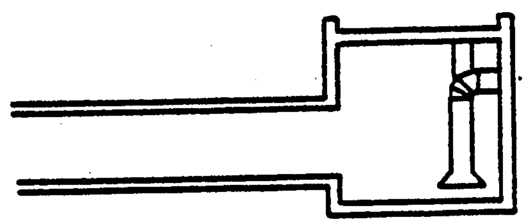
The station sump should have a wall to keep low flows and any mud out of the sump. The wall should be at the minimum water surface elevation. A valve must be provided to drain the low flows into the station sump. See Section R-5 and Plate No. R-1.

D-1 Intake Criteria continued.

D-1.2 Storm Drain Inlet

When the inlet to the sump is from a storm drain pipe or box the flow must be directed so that it is straight, parallel to the side walls and directed to each pump. This can be accomplished by a properly designed transition structure as shown below.





The angle of the diverging wall with the centerline, α, should be such that

$$\tan \alpha = \frac{1}{3F}$$

where F is the Froude number given by

$$F = \frac{v}{gd}$$

where v = average velocity in transition $\frac{V_1 + V_2}{2}$, fps

d = average depth in transition, ft.

g = gravitational constant

D-1 Intake Criteria continued.

D-1.2 Storm Drain Inlet

Plate D-1 gives the divergence angle, α , for known values of depth and velocity of flow. The maximum divergence angle should not exceed 45 degrees.

The transition structure should be designed using velocities and depths of flow corresponding to the design capacity of the pump station.

D-2 Trash Rack Design

A properly designed trash rack will screen out objects large enough to damage a pump but will allow smaller debris to be pumped. Strainers on the pump suction bell are not suitable for flood control pumping since clogging will occur, cutting off the flow, decreasing the suction head and eventually damaging the pump, either by cavitation or overload, or both.

A basic trash rack consists of rectangular steel bars with the long axis parallel to the flow. The rack should be vertical, or slightly inclined to facilitate cleaning. The top of the rack should terminate above the maximum water surface elevation.

The spacing between the bars should be not more than the maximum sphere size which can be handled by the smallest pump. If pumped trash must be kept to a minimum, when pumping into a marina, for instance, the spacing should be less than the maximum sphere size. Bar spacing will usually be 2 to 4 inches.

Stations draining an open retention basin employ a floating trashrack in addition to the vertical rack. This allows trash to be raked during storm periods as the rack is floating and still provides a barrier for submerged objects. See Section R-2 and Plate No. R-2.

Trash racks employed in flood control pumping applications will not create a significant head loss due to the low sump velocities. The head loss is a function of the angle between the flow and rack, bar shape and dimensions, bar spacing, and the velocity of the water.

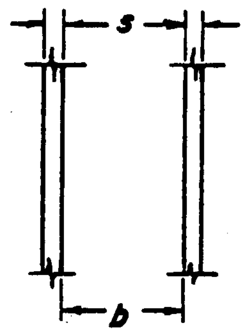
D-2 Trash Rack Design continued.

If the coefficient of resistance of the rack is designated as δ, then the rack loss becomes

$$H_L = \delta \frac{V^2}{2g}$$

where: H_L = loss through rack, ft.
V = velocity of flow, fps.
δ = coefficient of resistance

The amount of obstruction caused by a rack can be indicated by the ratio of the net rack area and the total area.



From the sketch

$$\frac{\text{net flow area}}{\text{total area}} = \frac{b}{s+b} = \epsilon$$

where: ϵ = flow ratio
b = width between bars
s = thickness of bar

With ϵ calculated from the trash rack dimensions, the coefficient of resistance is given in Plate D-2. It is assumed that flow is perpendicular to the trash rack (flow angle is zero).

For recommended velocities of one foot per second the loss is negligible. With coefficient of resistance of 2.7 the loss is

$$H_L = \delta \frac{V^2}{2g} = \frac{2.7\ (1.0)^2}{2\ (32.2)} = .042 \text{ ft.}$$

These losses ahead of the pump do not add to the TDH. However, for special cases where a trash rack (or protection barrier) is required on the discharge line this head loss must be overcome by the pump.

D-3 Sump Design

D-3.1 General

After the pumping elements have been selected it remains only to design a sump which will give optimum pump performance.

The requirements of a good sump design are as follows: it allows water to the pump with a minimum head loss, it requires the least submergence, and it is cheapest to build. It is unreasonable to assume that a given sump design will satisfy all three requirements; therefore the optimum sump design is a compromise of these parameters.

The sump design criteria in this manual is a compilation of practices recommended by pump manufacturers, the Hydraulic Institute, and independent researchers. Sump designs which have special configurations and do not follow recommended practices should be model tested to correct any possible trouble spots.

D-3.2 Hydraulic Considerations

Water should approach the pumps in a straight path with velocities approximately one foot per second. Pumps should not be arranged in line so that water must flow past one pump to another.

Structural columns in the pump pit should be streamlined to reduce vortex formations. Any abrupt changes in the flow path should be five (Pump) bell diameters in front of the pump.

D-3.3 Sump Features

D-3.3.1 Clean-Out Pump (Sump Pump)

A clean-out pump is provided to empty the station sump at the end of storm and to pump low flows (nuisance water) during the year. The clean-out pump is provided with a sump so that the station sump can be completely dewatered.

The clean-out pump should be located where it will not interfere with the flow to the main pumps. It can often be located along a side wall or, if space permits, near the back wall, between main pumps or in a corner. The station sump floor should slope (1/4-inch per foot) to the clean-out sump to facilitate drainage.

D-3.3.2 Hose Bibb

A two-inch water service line shall be provided in the sump terminating in a 1-1/2 inch hose bibb approximately three feet above the sump floor. The hose bibb is described in Section D-2.3.3.

D-3 Sump Design continued.
D-3.3 Sump Features

D-3.3.3 Sump Access

D-3.3.3.1 Personnel

A stairway shall be provided to allow access to the sump floor from the motor room floor. If space permits, a walkway over the trashrack above the maximum water surface elevation should be provided for raking of trash during a storm. This walkway will allow inspection of the pumps during operation. The opening in the motor room floor for the stairway must be covered by a steel door for pump station with underground inlets. See Section R-4.

D-3.3.3.2 Equipment

Access to the sump is required to allow equipment to be lowered and trash removed from the sump. The opening should be 4 feet by 4 feet minimum and should be covered with steel diamond plate. Pipe sleeves shall be provided for installing a three-sided railing when the cover is removed.

D-3.3.3.3 Trash Cleanout

Pump stations with an underground inlet will require an opening upstream of the trashrack for removal of trash retained by the trash rack. The opening may be covered with steel plate, standard manhole cover or grating.

Pump stations draining an open retention basin will be provided with an exterior monorail hoist for removal of trash from the floating platform. Access shall be provided to the floating trash rack.

D-4 Submergence

D-4.1 General

Submergence is the depth of water above the pump suction bell inlet. The minimum submergence of a pump will be governed by non-vortex criteria or NPSH requirements (whichever is greater) and will define the low water operating level or "off point" of the pump.

D-4 Submergence continued.

D-4.2 Vortexing

An ASME publication, "Development of an Optimum Sump Design for Propeller and Mixed Flow Pumps," by J. L. Dicmas defines three types of vortexes shown in Plate D-3.

Type 1 vortexing occurs at the very start of vortex action and is not detrimental to pump performance as tiny bubbles of air are pulled into the pump.

Type 2 vortexes form for less than 30 seconds and pull in air and floating debris into the pump affecting capacity and horsepower.

Type 3 vortexes are continuous and allow large amounts of air into the pump accompanied by sucking noise.

In a flood control pumping system, the pumps should operate free of Type 2 and Type 3 vortexes.

D-4.3 Suction Bell Velocity

Suction bell velocity is an important parameter in the determination of the minimum submergence requirement for a pump. For optimum sump design (employing clearances set forth in Section D-5.2) the suction bell velocity and vortex criteria determine required submergence.

The suction bell velocity is given by

$$V_B = \frac{\text{flow}}{\text{area}} = \frac{\text{cfs}}{.785\ D^2} = \frac{571\ \text{gpm}}{D^2}$$

where: V_B = suction bell velocity, fps
D = bell diameter, ft.

The pump flow at the minimum water surface elevation should be used in calculating bell velocity.

D-4 Submergence continued.

D-4.4 Submergence Requirements

D-4.4.1 Non-Vortex Criteria

The non-vortex criteria will usually be the determining factor in establishing submergence requirements.

The minimum submergence ratio (S/D) for Type 1 vortexes can be determined by referring to Plate D-3. The submergence thus calculated is based on clearances which will be defined in Section D-5.2. Deviation from these clearances may required more submergence.

D-4.4.2 NPSH Requirements (Net Positive Suction Head)

After the vortex submergence requirement is known, it is necessary to check to see if there is adequate NPSH. When pumping water at sea level, an NPSH of 33 feet is equivalent to zero submergence of the impeller eye. The NPSH requirement at the operating point is found from the pump manufacturer's curve. If the NPSH requirement is more than 33 feet the difference is the required submergence over the impeller eye. If this submergence is greater than that declared by non-vortex criteria then it is the governing value.

NPSH is not usually the limiting submergence on large-low speed flood control pumps but must always be checked.

D-4.4.3 Use of Suction Umbrellas

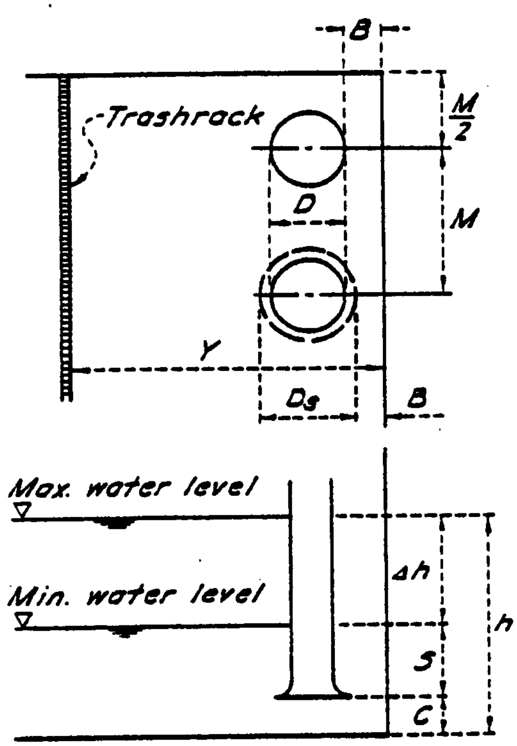
Suction umbrellas are used to lower the inlet velocity and reduce the submergence requirement as shown in Plate No. D-3. Suction umbrellas shall be employed whenever the reduction in submergence will allow a more economical sump. When suction umbrellas are used, a submergence to bell diameter (S/D) ratio of 0.8 should be used.

The suction umbrella diameter should be such that the inlet velocity is reduced to approximately two feet per second. See Section E-4. The use of suction umbrellas requires clearances described in Section D-5.2.

D-5 Sump Clearances

D-5.1 Definition of Terms

The sump design parameters used in this manual are described below. These clearances are specified to achieve optimum sump design and are dimensions recommended by the Hydraulic Institute and pump manufacturers.



D = Bell diameter

B = Backwall clearance

C = Bottom clearance

S = Minimum submergence

Δh = Pumping range

M = Pump spacing

h = Maximum water level

D_s = Suction umbrella diameter

D-5.2 Recommended Sump Dimensions

Table D-1 gives recommended values for the sump design parameters. In cases of various pump sizes (various D's) these clearances must be calculated for each pump size.

D-5 Sump Clearances continued.
D-5.2 Recommended Sump Dimensions

TABLE D-1

Recommended Sump Design Dimensions

Dimension	Recommended Value	Explanation
B	.25 D .10 D	Without suction umbrellas With suction umbrellas
C	.5 D	Minimum
Y	See Plate No. D-4	Minimum
M	See Plate No. D-4	With or without suction umbrellas(minimum)

D-6 Depth of Sump

After the maximum water surface elevation, required operating range, submergence, and floor clearance are known for each pump, the depth of sump can be calculated.

Each pump should be analyzed separately, calculating the depth of sump required for each pump in reverse order of operation. The sump pump is not considered in these calculations. The lowest depth of sump calculated will be the correct elevation of the sump floor.

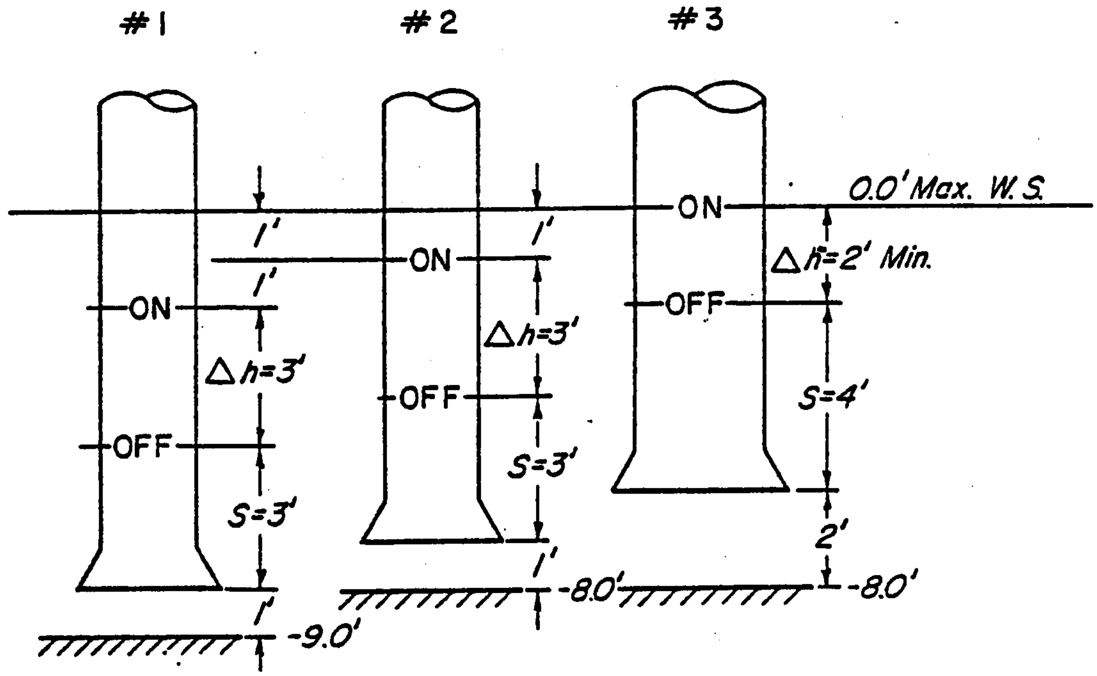
In the following example the operating ranges have been determined for an installation with two pumps having a Δh governed by cycling limitations and the third pump having a two-foot minimum operating range. The pumps are operated in the order 1, 2, and 3. With the pump start points one foot apart, the sump depth for each pump is calculated.

For pump No. 3, the required sump depth is two feet (operating range) plus 4 feet (minimum submergence) plus two feet (bottom clearance) or 8.0 feet.

For pump No. 2, the required sump depth is one foot (difference in start elevations) plus three feet (operating range required for non-excessive cycling) plus three feet (minimum submergence) plus one foot (bottom clearance) or 8.0 feet.

For pump No. 1, the required sump depth is two feet (difference in start elevations) plus three feet (operation range required for non-excessive cycling) plus three feet (minimum submergence) plus one foot (bottom clearance) or 9.0 feet.

D-6 Depth of Sump continued.



The correct sump elevation is 9.0 feet below the maximum water surface elevation. Since the sump floor will be at one elevation, pumps 2 and 3 must be lowered one foot to maintain the required floor clearance. Since the on and off points remain the same, the submergence for pumps 2 and 3 is one foot greater than the minimum required.