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IMPROVEMENTS IN PUMP INTAKE BASIN DESIGN

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PREFACE

Typically, pump intake basins (or wet wells or pump sumps) designed in accordance with accepted criteria are relatively large structures with flat or nearly flat floors. The water surface fluctuates cyclically throughout a range in depth of about a meter. The influent fluid plunges to the surface in a free-falling cascade, and the cascade often degrades pump performance by entraining air and driving the bubbles deep into the pool, where they are often ingested by the pumps. Air entrainment in pumps causes unequal vane loadings and flow abnormalities that create excessive wear and significantly reduce head, capacity, and efficiency. Solids-bearing waters, such as storm water, wastewater, or even fresh raw water, deposit sand and sludge on the large floor. Scum, if present, spreads over the entire water surface area. These depositions are commonly removed only with great difficulty and expense. In wastewater systems, decomposing solids produce hydrogen sulfide gas, an odiferous toxic compound that promotes formation of sulfuric acid and consequent deterioration of concrete and metal surfaces. The cost of remedying the damage caused by acid attack amounts to many millions of dollars annually nationwide.

The high cost of these large basins and the difficulty and expense of cleaning them led to the development of the "self-cleaning" wet well--a narrow basin with one sloping side that culminates in a bottom trench containing the pump intakes. By using variable speed pumps, the pumped flow rate is adjusted to the inflow rate to eliminate the need for storage and thus minimize the size and cost of the basin. A cascade is avoided by keeping the normal water surface in the basin level with that in the upstream sewer. Hydraulic performance is improved because there is no air entrainment and no stray floor currents. Deposition is decreased because settling solids slide down the sloping side to the confining trench, where pump intake currents can capture adjacent material. Furthermore, if the water level is lowered (called "pump-down") into the trench, all floating material is confined in the trench, moved to the last pump by the current, and sucked into the pump by a vortex. At the same time, the currents along the trench sweep most of the deposits accumulated between pump intakes to the last intake, where they are also sucked out by the last pump. No labor is needed beyond switching automatic level controls off and on, hosing grease off walls (in raw sewage pumping stations), and repriming pumps--hence the term "self-cleaning." Pumping station designs such as these are not well known but have operated successfully for nearly three decades.

Variable speed drives are expensive and more difficult to maintain than the almost universally-used constant speed drives that consist of only an electric motor with an across-the-line starter. Furthermore, variable speed drives are inappropriate for pumping stations with flat pump and system head-capacity curves, because small changes in speed cause large changes in flow rate. The challenge was to extend the concept of self-cleaning to constant speed pumps while avoiding the disadvantages of the typical intake basin.

The uncertainties created by the need to store water in constant speed pumping stations and the consequent cyclical rise and fall of the water surface made model studies imperative. As most pumping stations are small and pumps are usually--though not always--submersible types, this project began as "Self-cleaning wet wells for constant speed submersible wastewater pumps." Discoveries during the research are, however, applicable to other types of pumps and to pumping storm water and clear water as well as wastewater. Hence, the title for this final report was changed to reflect the broader range of applications.

This project was more successful than was expected. The effectiveness of cleaning variable speed pumping stations has been greatly enhanced by means of an ogee ramp which conserves the energy in the incoming water at "pump-down." This high energy produces strong currents along the trench. Solids are quickly swept into the last pump intake.

The same effectiveness applies to constant speed pumping. Cascades in constant speed pumping station basins are eliminated by discharging water into the basin at or slightly below normal low water level (LWL). The required size of the basin for constant speed pumping is reduced, because, by sloping the influent pipe at a gradient of 2 percent between LWL and high water level (HWL), the volume of the pipe becomes part of the required storage volume. Entrained air bubbles are eliminated, because the few bubbles that are formed by the weak hydraulic jump in the pipe rise to the free water surface and escape up the pipe. Air is thereby eliminated from the water entering the pump basin.

The investigations revealed that arranging pumps in series along the axis of the incoming water does not affect the efficiency of intakes nor cause interference between intakes in terms of their loss coefficients, the pre-rotation of flow entering them, or the formation of vortices for the ranges of spacing and submergence tested. The long-held belief that this arrangement is not advisable is not justified for pumps with impellers at a considerable distance from the intake (such as dry pit pumps and self-priming pumps) nor for relatively small (600 L/s 2000 m^3/h, or 10,000 gal/min) and robust submersible or vertical column pumps wherein impellers are near the intake entrance.

With a horizontal inflow never above the water surface and the pump intakes well below the inflow and confined within the narrow trench that is characteristic of this type of intake basin, currents near the floor of the trench are very low even when surface currents are high. Column pumps of large size have been used in this type of basin successfully. However, universal application of this design without further model tests to ensure symmetrical approach velocities at the impeller is not advised due to the sensitivity of performance of large pumps--particularly large pumps at high specific speeds--to approach flow conditions.

The results of this research may be applied to dry pit pumps, and, in sizes below 600 L/s (10,000 gal/min), to submersible and vertical pumps, and they apply as well to pumping clear water as to wastewater and storm water. They can also apply to refurbishing existing basins where pump performance is sub-standard.

This report has been organized to present the design guide lines before the supporting research so as to make it convenient for designers to use the results without delving deeply into research details.

ABSTRACT

The pump intake basins described are improvements over existing conventional types because they:

- Reduce size by eliminating storage requirements through the use of variable speed pumps or utilize a steeply sloping inlet pipe to supplement the intake basin volume.
- Eliminate cascading flow into the intake basin and entrainment of air into the pumps by locating the inlet pipe elevation coincident with the low water level in the basin.
- Use the intake basin geometry to concentrate settled and floating solids in a limited zone near the pump inlets where they can be regularly removed by pump operation.

Recommendations for the design of both rectangular and round basins are given. The research findings that support those recommendations are included along with limited field observations.

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FIGURES

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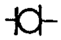
ABBREVIATIONS AND SYMBOLS

~	Approximately
≈	Approximately equal
	Ball valve
C/S	Constant speed (pump)
D	Outside diameter of the rim of the suction bell
D_h	Hydraulic depth, area/surface width
D_p	Pipe diameter (ID)
	Eccentric plug valve
ENSR	ENSR Consulting and Engineering
FM	Force main
ft	Feet
gal/min	U.S. gallons per minute. Common U.S. usage for pump capacity
hp	Horsepower
HWL	High water level (in a basin)
ID	Inside diameter
in	Inch
kW	Kilowatts
LWL	Low water level (in a basin)
M'f'r	Manufacturer
Mgal/d	Million U.S. gallons per day. Common usage for pumping station capacity
min	Minimum
MSU	Montana State University
±	More or less
m^3/h	Cubic meters per hour. Common usage in the U.S.
OD	Outside diameter
Ø	Round
pump-down	Reducing the water level to its lowest possible depth with the main pumps
rev/min	Revolutions per minute
R^2	An indicator of the goodness of fit of data in a regression analysis
SI	System International or metric units
ss	Stainless steel
typ	Typical(ly)
v_{pi}	Average superficial fluid velocity at the pump intake based on the area bounded by the OD of the suction bell rim
V/S	Variable speed (pump)
yd	Yard

ACKNOWLEDGEMENTS

Partial funding for this project was supplied by Fairbanks Morse Pump Corporation, The Gorman-Rupp Company, ITT Flygt AB, Montana State University Foundation, the Department of Civil Engineering, Montana State University, and R. L. Sanks. Additionally, the Department contributed materials and help from the staff, notably W. Keightley, and Professors W. E. Larsen, Otto Stein, and T. T. Lang. Calgon Carbon Corporation contributed granular activated carbon. In addition to funds donated, Fairbanks Morse Pump Corporation built and tested a full sized steel pump basin for two 63 L/s (227 m^3/h) pumps. As part of its contribution, ITT Flygt AB had three acrylic submersible pump models constructed and sent to the ENSR laboratory.

The city of Steilacoom, Municipality of Metropolitan Seattle (now King County, Washington Department of Metropolitan Services), the Town of Black Diamond, WA, G.S. Dodson & Associates, and ITT Flygt AB arranged visits to carry out investigations at pumping stations in Washington, California, and Sweden respectively.

William Wheeler, who calculated the data in Table 1, and B. E. Bosserman made valuable contributions to the manuscript. They and the three authors contributed all their time and effort *pro bono.*

SECTION I

INTRODUCTION

A. TYPICAL PUMP INTAKE BASINS

Typically, conventional pump intake basins, pump sumps, or wet wells, designed in accordance with generally accepted criteria [1, 2] have relatively large flat or nearly flat floors. If the fluid is any but polished or filtered water, deposits of sludge and/or silt and sand accumulate on the floor and they are removed only with difficulty and expense. The problem is troublesome for storm water, and it is particularly severe for wastewater because of the large amount of sludge deposited. The sludge putrefies, becomes odiferous, and the hydrogen sulfide gas generated develops an insulating deposit on electrical contacts and other electric and electronic surfaces. Sulfuric acid generated biologically from hydrogen sulfide attacks concrete and metal. Wastewater scum and grease balls spread over the entire liquid surface.

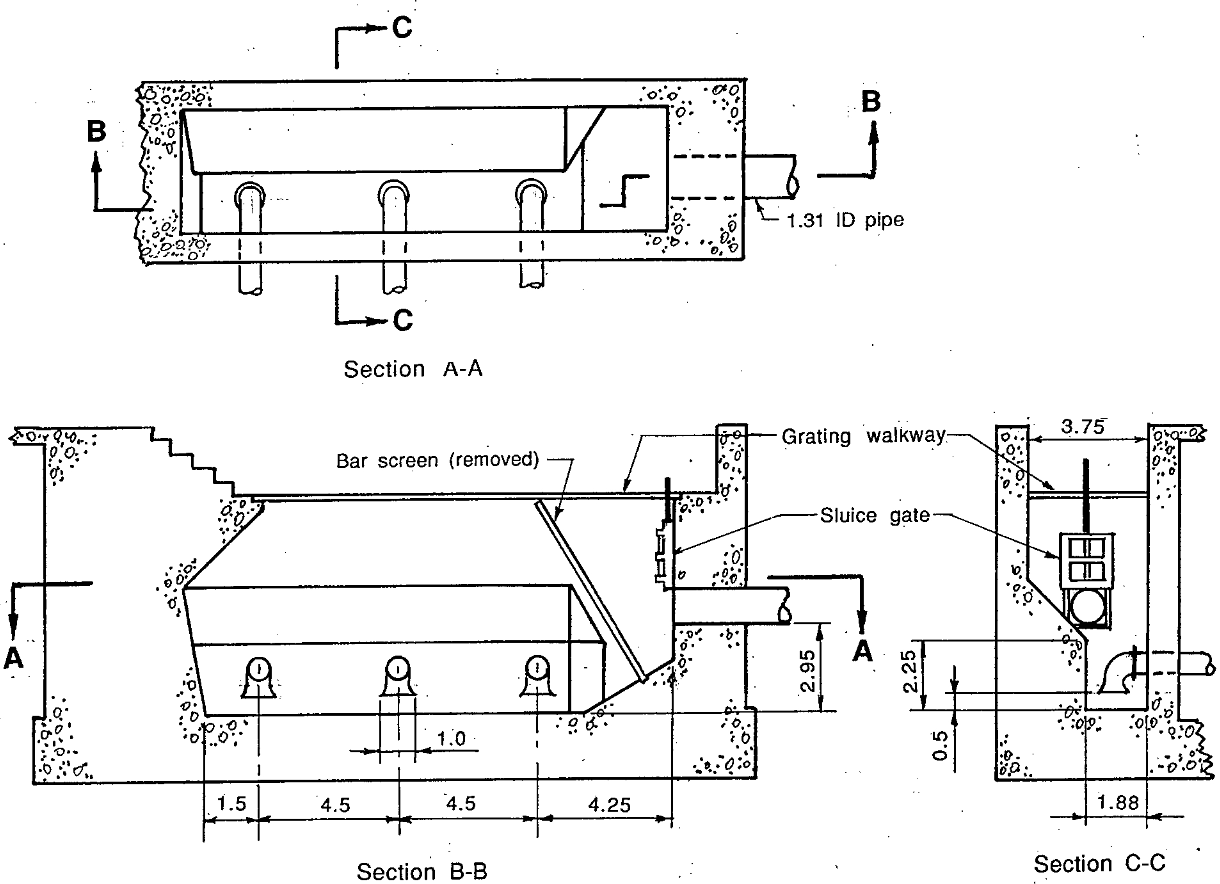
One pump manufacturer (ITT Flygt) does make machinery that mixes the contents of the basin so that the main pumps can remove the mixture if the basin is small enough or if there are enough mixing units. The disadvantages are the addition of more mechanical equipment, the power for operation, and the extra maintenance. Consequently, there is an advantage in coping with the problem by means of geometry or, perhaps, with piping layout rather than by means of added machinery.

In basins for constant speed pumps, the influent falls from the inlet pipe to the water surface below in a cascade that varies from a few centimeters to a meter or more in height. Masses of air bubbles are formed and driven deeply into the pool where, in many designs, there is insufficient distance between the waterfall and the first pump intake to allow all the bubbles to escape to the surface. The remaining bubbles, sometimes a large proportion, are drawn into the pumps with devastating effects on head, capacity, and efficiency even when a small percentage of air is present [1]. If air entrainment is possible, the station capacity should be increased by 10 to 20 percent. Wear on bearings and couplings is greatly increased, and excessive noise is present. If the liquid is raw wastewater, the turbulence caused by the cascade sweeps out hydrogen sulfide and other noxious gases that add to problems of odor control and protection of electric and electronic equipment.

B. TRENCH-TYPE INTAKE BASINS

The idea for the trench-type intake basin was conceived four decades ago by Caldwell [G.M. Jones, Brown and Caldwell Consultants, personal communication, 1984] who reasoned that variable speed (V/S) pumps could be programmed to eliminate the need for storage by matching the pumping rate with the inflow rate. With no need for storage the wet well could be small and by sloping the floor to a narrow trench, deposited solids could be so confined that they could be washed out by pumping the water level down (pump-down) to within a few centimeters of the intakes. The inflow would wash deposits in the trench to the last pump for discharge to the force main. Arranging the pumps in a series coaxial with the influent pipe is contrary to generally accepted criteria [1, 2]. Nevertheless, 27 trench-type pumping stations with pumps ranging in size from 150 mm (6 in) and 66 L/s (240 m^3/h or 1.5 Mgal/d) to 1.4 m (54 in) and 4.8 m^3/s (108 Mgal/d) installed a quarter century ago for the Municipality of Metropolitan Seattle (Seattle Metro) have proven to be eminently successful. Many other similar pumping stations have also been constructed and operate with equal success.

Seattle Metro's Kirkland Pumping Station in central downtown Kirkland is typical and its wet well was the one selected for modeling in this research. The plans for the intake basin are shown in Figure 1. The three V/S pumps are driven by electric motors through eddy-current couplings. Pump 3 (furthest from the inlet) is rated at 132 L/s (475 m^3/h) at 57.6 m (2100 gal/min at 189 ft) as is Pump 2. At full speed, the v_{pi} (superficial pump intake velocity based on the area of the outside diameter, D, of the suction bell) is 1.02 m/s (3.33 ft/s). Pump 1 is rated at 110 L/s (396 m^3/h or 1750 gal/min), and the v_{pi} is 0.85 m/s (2.78 ft/s). Either of the larger pumps is a standby, so the total firm capacity is 241 L/s (868 m^3/h or 5.5 Mgal/d). The peak flow was estimated by Seattle Metro to be 215 L/s (774 m^3/h or 4.9 Mgal/d). The station has been in operation for 26 years, but is as clean and attractive as though built last year. The manually-cleaned bar screens had been removed many years ago.



All dimensions are multiples of D (OD of bell mouth) 406 mm (16 in).

Figure 1. Kirkland pumping station.

During normal operation, the water level in the wet well is kept the same within a few millimeters as the water level in the upstream sewer by the variable speed pumps. Thus, there is no cascade and no currents faster than the velocity in the upstream sewer. There is minimum disturbance as wastewater enters the basin, and currents are slow by the time the fluid reaches the midpoint.

The basin is cleaned twice weekly to suppress odors. The operators first note the speed required by the last pump to hold the water level constant and thus match the inflow, which is typically about 66 L/s (227 m^3/h or 1.5 Mgal/d). The pump is then operated at full speed until the submergence of the pump intake is reduced to about 0.7 D, and the speed is then reduced to the first value noted (so pumping rate matches inflow rate) for as long as possible--between two and three minutes. This reduction in depth is called pump-down, and during pump-down, the pump is close to losing prime. At Kirkland, the pump air-binds at any submergence less than 0.56 D. The reduction in sludge volume is shown in Figure 2 as the difference between curves a and b. A second pump-down removed very little sludge and left an average residual depth of about 50 mm (2 in) of relatively hard material that was about 63 percent organic material and 37 percent sand by volume. This hard sludge appeared to be stabilized because cleaning virtually eliminated odors. The existing basin could, however, be cleaned more thoroughly with the better procedure described in Section II.C.d.

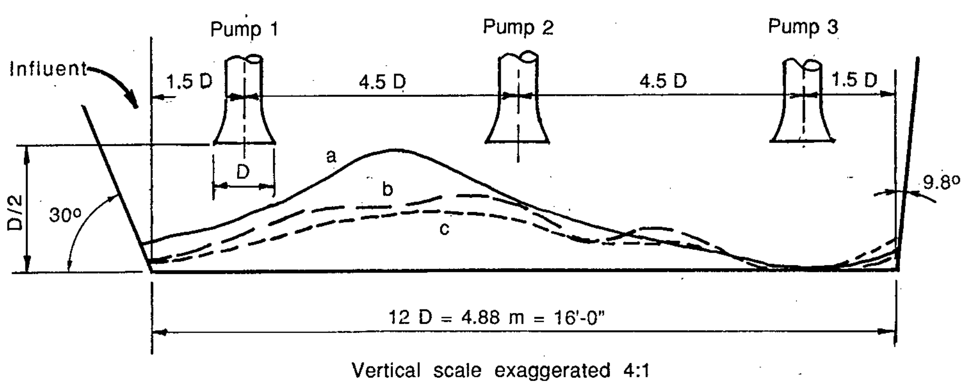


Figure 2. Sludge deposits in Kirkland Pumping Station sump on Sept. 14, 1992 (a) before pump-down, (b) after first pump-down, and (c) after second pump-down.

C. OBJECTIVES

The general objectives of this research program were (a) to modify the proven trench-type pump intake basin for constant speed (C/S) pumps of both dry pit and submersible wet pit types and (b) to provide guidelines for designing both rectangular and round wet wells. As the work progressed, it became apparent that the findings were as applicable to V/S pumps as to C/S units and to clean water and storm water as well as to wastewater.

SECTION II

CONCLUSIONS: DESIGN GUIDELINES

A. CAVEATS

The research was carried out on models of 132 L/s (475 m^3/h or 2100 gal/min) dry pit pumps, and it would be unwise to apply the results to pumps larger than about 600 L/s (2000 m^3/h or 10,000 gal/min) even though larger pumps have been used with trench-type basins. It is good insurance to require model tests of intake basins for larger pumps. Such pumping stations are expensive and the cost of model studies is an insignificant percentage of the total cost.

When pump intake piping exceeds 500 mm (20 in) in diameter and the pump capacity exceeds a maximum of 580 L/s (2090 m^3/h or 9300 gal/min), piping to dry pit pumps becomes costly and cumbersome. Draft tubes afford better access to the pumps and may be more economical. See Chapter 17 of *Pumping Station Design* [3].

High specific speed (axial flow) pumps are particularly sensitive to small variations of approach velocity across the plane of the impeller. As no approach velocity measurements were made during these investigations, the effect on such pumps of the current past their intakes is unknown. Consequently, axial flow pumps should not be used in these pump intake basins unless tests are made to establish satisfactory performance.

B. APPLICATION

Trench-type pump intake basins are suitable for clean water and, with modifications for cleaning, for storm water and wastewater. These basins are suitable for both V/S and, with modified influent piping, for C/S pumps.

C. RECOMMENDED DESIGN GUIDELINES

Guidelines are numbered for easy reference. They are arranged so that Nos. 1 to 6 apply to all pumping stations. Additionally, Nos. 7 and 8 apply to rectangular basins for clear water and V/S pumps, Nos. 9 and 10 for C/S pumps, No, 11 for solids-laden water and V/S pumps, No. 12 for C/S pumps. Nos.13 and 14 apply to submersible pumps, and No. 15 applies to round basins with self-priming pumps.

a. All Pumping Stations

These guidelines apply to all pumping stations with trench-type pump intake basins regardless of service or type.

1. Select the kind, size, and number of pumps. Guidelines are given in pp 265-288 of *Pumping Station Design* [3]. For flows less than 220 L/s (792 m^3/h or 5 Mgal/d), consider duplex or triplex submersible pumps or self-priming pump intakes in a round

basin. For flows above 110 L/s (400 m^3/h or 2.5 Mgal/d) consider a rectangular basin. For flows between 110 and 220 L/s consider both configurations.

2. Select suction bells for a maximum v_{pi} (entrance velocity based on the area of a circle of diameter D, the OD of the bell rim) of 1.1 to 1.5 m/s (3.5 to 5 ft/s). For pumps without suction bells such as most submersible pumps, however, follow the manufacturer's recommendations.

3. Suction bells for dry pit or self-priming pumps may be spaced as close as 1.0 D clear or even closer if adjacent pumps do not operate simultaneously. However, allow 1.1 m (42 in) clear between pumping machinery for working space. Submersible pumps should never have a clear spacing between volutes of less than 100 mm (4 in). Depending on type and size of pumps, more conservative (greater) spreading may well be required.

4 Make the trench close to 2 D wide, and allow for a submergence of the suction intake of at least 2 D for a v_{pi} of 1.5 m/s (5 ft/s) or less. Follow manufacturer's recommendations for the submergence of submersible pump intakes or any pump intake with an umbrella over it.

Dicmas [4] states that the above intake velocity can generate Type 4 vortices at a submergence of 1.8 D. (A Type 4 vortex has a pronounced surface depression and a core containing bubbles that are sucked into the pump intake.) But note that the Hydraulic Institute Standards [1] can be transformed to yield a required submergence of 1.6 D. Dicmas also states that the surface width of the basin affects vortex formation, and Type 4 vortices can form at a submergence of 1.8 D when the width is 2.7 D and at a submergence of 2.4 D when the width is reduced to 2 D. In tests of intakes in a trench 1.87 D wide at Montana State University, however, a submergence of more than 1.75 D was sufficient to suppress Type 4 vortices in a 1/3.63-scale model although Types 2 and 3 (less severe) occurred. Nevertheless, it is wise to be conservative here. Dicmas also claims that the most efficient floor clearance is 0.4 D. Mild submerged vortices form at walls beside suction intakes when trenches are 2.5 D wide or less, and the intensity increases somewhat as the width decreases. At trench widths of more than 2 D, performance during cleaning is likely to be adversely affected, however, so a width of 2 D appears to be the best compromise. Vortex suppressors can be used to ameliorate side wall vortices. See Section V.E.g.

5. Limit entrance velocity into the basin to 1.5 m/s (5 ft/s). Design the cross-sectional area above the trench (WH in Figure 3) to limit the average (plug flow) velocity to 0.3 m/s (1 ft/s).

The current entering the basin continues with some abatement to the end wall and is reversed, thereby setting up a recirculation pattern. When the provisions above are met, currents at the bottom of the trench are very low, and good pump intake entrance conditions are produced.

6. Decide whether to use rectangular or round intake basins and V/S or C/S pumps and go to the appropriate subsection for further guidelines.

<u>b. Rectangular Basins for Clean Water and V/S Pumps</u>

As cleaning is unnecessary, there is no need for the ogee ramp mentioned in the Preface and described later. The narrow trench is, however, beneficial in preventing rotation and stray floor currents at suction intakes, but a wider basin above the trench is usually needed to reduce the strong currents caused by the influent velocity.

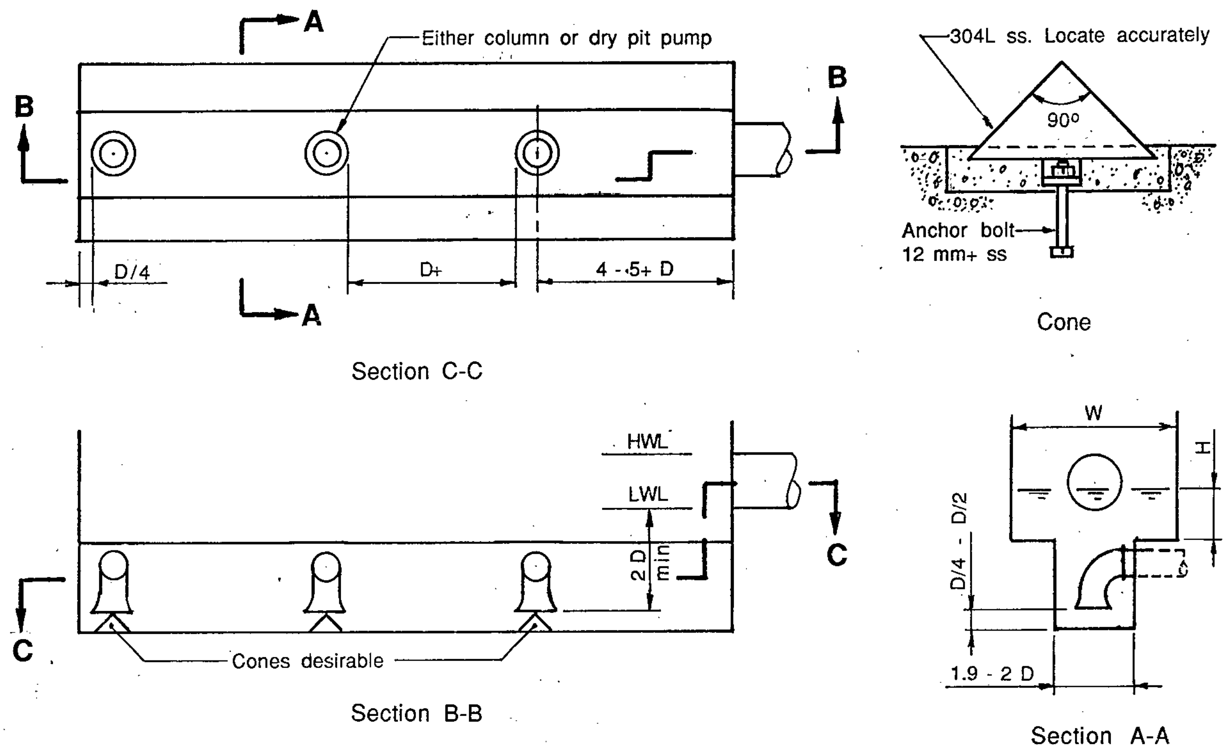


Figure 3. Rectangular sump for V/S pumps and clean water.

7. See Figure 3 for construction features. Cones with apexes in the plane of the intake are desirable to prevent floor vortices and inhibit pre-rotation. Vortex suppressors in walls beside the intakes can reduce side wall vortices. See Section V.E.g.

8. If the influent pipe discharges by gravity (i.e. is not under pressure), set the controls to give maximum discharge when the water level coincides with the soffit of the pipe and to give zero discharge when the water surface is at the invert. Details of control are given on p. 311 of *Pumping Station Design* [3]. Limit entrance velocity into the basin to no more than 1.5 m/s (5 ft/s) by enlarging the inlet pipe if necessary.

c. Rectangular Basins for Clean Water and C/S pumps

The difference between C/S and V/S pumping stems from the need in the former to introduce water without a cascade into a basin in which depth fluctuates over a wide range. One way to avoid a cascade is to slope the approach or inlet pipe from HWL at some upstream point downward to LWL at the basin at a gradient of about 2 percent. Water flowing freely down this pipe quickly reaches super-critical velocity. As it is desirable to keep the super-critical velocities as low as possible, a rough pipe is better than a smooth one, a large pipe is better than a small one, and gradients of more than 2 percent should be avoided. It is also desirable to inhibit turbulence in the basin by preventing the hydraulic jump from leaving the pipe. Hence, the LWL must be somewhat above the pipe invert. Fine tuning of LWL pump settings can be made at start-up. A horizontal section about 10 pipe diameters long at the basin entrance is also helpful in keeping the jump within the pipe.

A hydraulic jump occurs when water, flowing down the approach pipe at super-critical velocity, reaches the impoundment in the lower part of the pipe. Both the allowable flow rate and the Froude number must be small enough to allow all of the air that is entrained by the jump to escape up the pipe, because, as Wisner, Mohsen, and Kouwen [5] show, the velocity in the full pipe is too low to ensure that the water can drag bubbles and air pockets to the basin. It is conceivable that entrained air could accumulate into air pockets large enough to partially block the flow and cause surging. Reducing the super-critical velocity by means of a rough interior pipe surface, by using less slope, or by limiting the allowable flow rate are ways to avoid problems with air. Until prototype tests of large pipes have been made, it is well to be cautious.

The flow rates in Table 1 are thought to be very conservative. The Froude numbers, less than 2.0, indicate a weak hydraulic jump and a minimum of entrained air. The downstream depth does not exceed 60 percent of the diameter, so the free water surface downstream from the jump is 20 pipe diameters long. In such a long section, there is ample opportunity for air bubbles to rise (even from the invert) to reach the free water surface, burst, and move upstream along the soffit of the pipe. Note, too, that the sum of depth plus velocity head upstream of the jump is limited to about 75 percent of the diameter.

TABLE 1. MAXIMUM RECOMMENDED FLOW RATES IN APPROACH PIPES

Manning's n= 0.010[a] Slope = 2 percent

Diameter of pipe		Initial Froude number	Flow rate			
mm	inches		L/s	m^3/hr	Mgal/d	ft^3/s
250	10	1.6	20	71	0.5	1.0
300	12	1.6	31	110	0.7	1.1
375	15	1.7	53	190	1.2	1.9
450	18	1.7	81	290	1.9	2.9
525	21	1.7	120	420	2.7	4.1
600	24	1.8	160	580	3.7	5.7
675	27	1.8	210	770	4.9	7.5
750	30	1.8	270	990	6.3	9.7
825	33	1.9	530	1200	7.8	12
900	36	1.9	650	1500	9.7	15

[a] For n = 0.012 (rougher pipe), flow rate increases (not decreases) by about 15 percent.

The data in Table 1 were calculated by Wheeler [6] using his *Partfull©* program, equations, and templates to solve Equations 3-10, 3-13, and 3-18 in Chow [7] together with the well-known Manning equation. The equations are, respectively:

$$v^2/2g = D_h/2 \tag{1}$$

where v is velocity, g is acceleration due to gravity and D_h is hydraulic depth (wetted cross-sectional area divided by surface width). Equation 1 is the criterion for critical flow wherein specific energy is a minimum.

$$F = v/(gD_h\cos\theta/\alpha)^{0.5} \approx v/(gD_h)^{0.5} \tag{2}$$

where F is Froude number, θ is slope of channel, and α is an energy coefficient usually considered to be 1.0.

$$Q^2/gA_1 + Z_1 A_1 = Q^2/gA_2 + Z_2 A_2 \tag{3}$$

where Q is flow rate, Z is the distance from the water surface to the centroid of wetted cross-sectional area, A, and the subscripts indicate sections on either side of the jump.

$$v = (1/n)R^{2/3} s^{1/2} \quad \text{Manning's equation in SI units} \tag{4a}$$

$$v = (1.486/n)R^{2/3} s^{1/2} \quad \text{in U.S. customary (English) units} \tag{4b}$$

where n is a surface roughness coefficient, R is hydraulic radius or wetted area divided by wetted perimeter, and s is slope or h_f/L, head loss divided by length.

Without *Partfull*©, the combined solution of Equations 1 to 4 is difficult for round pipes but much easier for rectangular channels. Calculations based on a rectangular channel equal in area and Froude number to a pipe are only about 4 percent or less in error for the depth after the jump and only 10 percent or less for flow rate for the pipe sizes in Table 1.

9. Follow Guidelines 1 to 7, but see Figure 4 and Table1 for the design of the approach pipe.

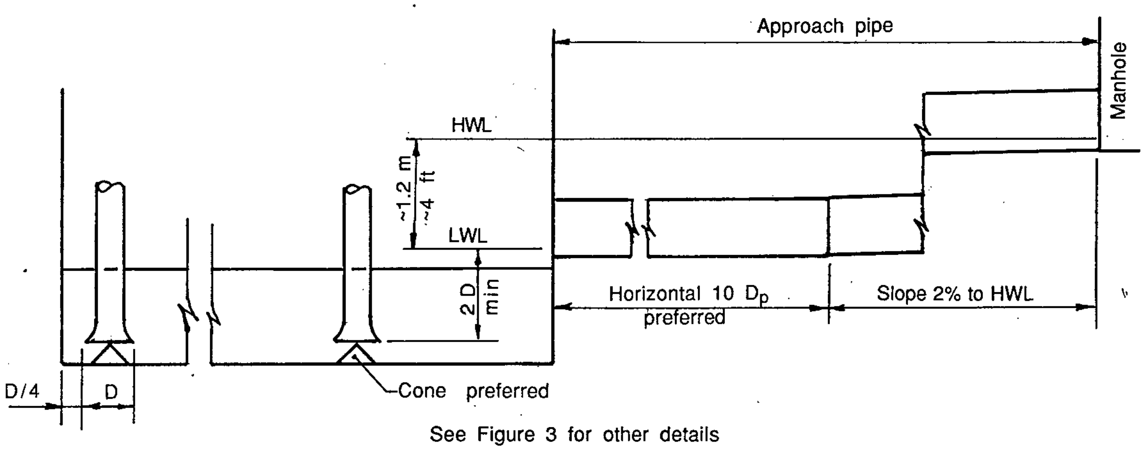


Figure 4. Rectangular sump for C/S pumps and clean water.

The pipe becomes part of the required storage volume. The volume that can be added to the wet well volume is the volume of the pipe when full less the volume required for the design flow at LWL. Calculate the active volume of required storage from

$$V = T\,Q/4 \tag{5}$$

where V is active volume (between LWL and HWL) in liters, Q is the flow rate of a single pump in L/s, and T is the minimum allowable time between motor starts in seconds. For English customary units, substitute gallons for V, gal/min for Q, and minutes for T. The oddly-shaped volume of water partly filling a sloping pipe can be calculated by the inaccurate average end area method or by the accurate prismoidal formula,

$$V = (A_1 + A_2 + 4\, A_m)L/6 \tag{6}$$

where A_1 and A_2 are cross-sectional areas of each end of the prism, A_m is the cross-sectional area at the middle, and L is the length.

Manufacturers of pumps, starters, and motors indicate that, depending on motor size and speed, cycle times can be increased from the traditional 6 starts and stops per hour to as much as 15 (long force mains excepted). As a rule, use 10 starts and stops per hour for motors of 37 kW (50 hp) and less operating at 880 rev/min or more, but always confirm these assumptions with candidate manufacturers and be certain that project specifications reflect the duty requirements for starters, motors, and pumps.

10. Allocate as much of the required active volume to the pipe as feasible even to increasing the diameter, because storage in pipes is less expensive than storage in basins. Adjust the dimensions of the basin to furnish the remainder of the required storage.

d. Rectangular Basins for Solids-bearing Water and C/S or V/S Pumps

These are basins that must be cleaned periodically. Once per week is usually sufficient for raw wastewater pumping. If a basin is to be cleaned at all, it should be designed for maximum effectiveness and ease of operation. The following features are needed to attain these objectives:

- Enough stored fluid available to complete the cleaning cycle.
- Easy switching from automatic to manual pump control and vice versa (a common feature) or even to manual control with a timer to switch "manual" to "automatic".
- Water guides (see Figure 5) to confine the influent within a rectangular trench only wide enough to allow installation of the sluice gate and proper trench width for suction bells.
- An ogee curve for the trench bottom that matches the longest trajectory of the fluid passing under the sluice gate.
- A return curve at the bottom of the ogee of sufficiently long radius.
- All but the last intake raised well above the theoretical depth of the super-critical flow along the floor at pump-down, because shock waves or "rooster tails" may impact on the suction bells and interfere with flow.
- Low floor clearance for the last pump to ensure the hydraulic jump migrates rapidly to the end of the basin. Less than D/4 can be achieved as shown by the construction left of Section B-B in Figure 5.
- A baffle between the last intake and the end wall to prevent circulation behind the inlet at pump-down.
- Soffits of approach and sewer pipes in the upstream manhole at the same elevation.

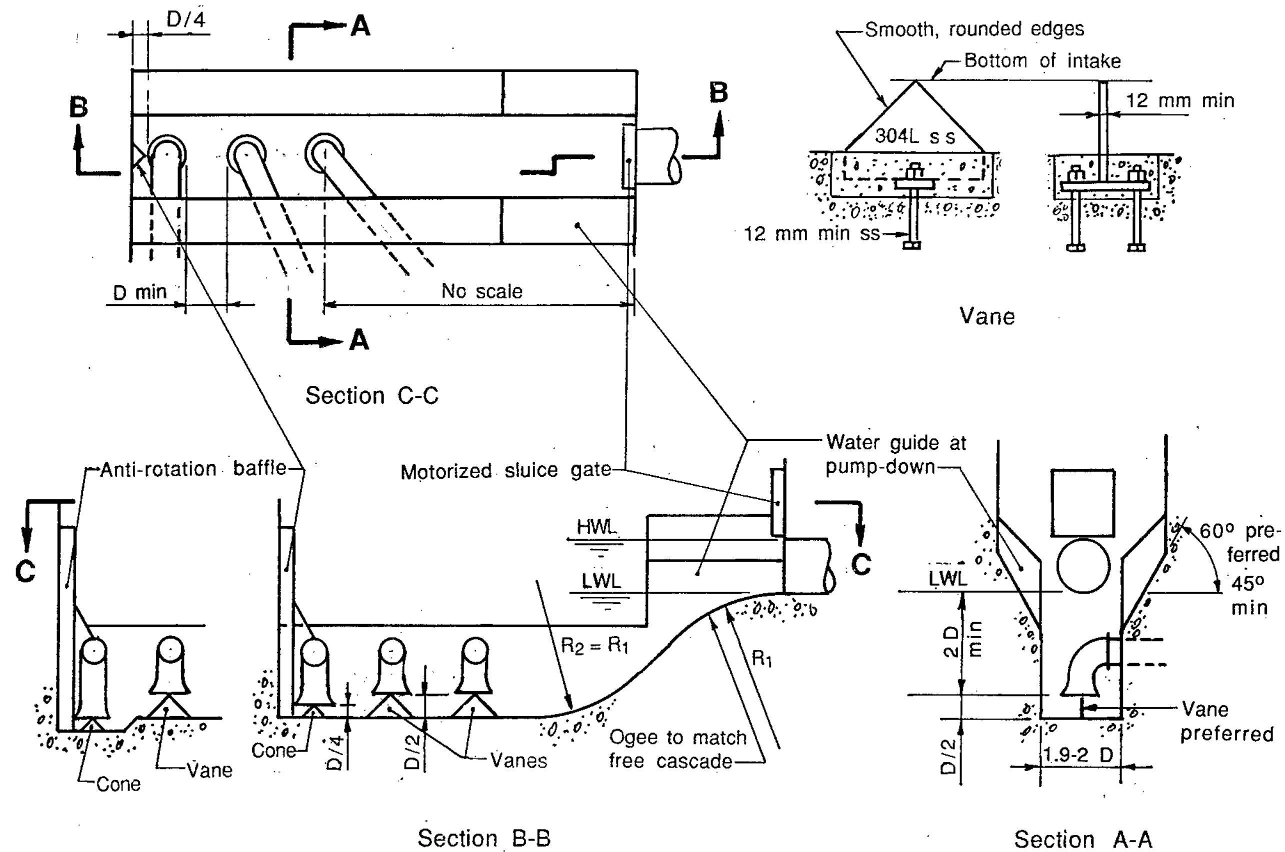


Figure 5. Rectangular sump for V/S pumps and solids-bearing water.

To achieve a satisfactory hydraulic jump in the trench during cleaning, the Froude number (Equation 2) near the last pump inlet should be at least 3.5. However, Froude numbers greater than 8 are very turbulent and may introduce excessive air entrainment that could cause premature air binding of the pump.

One easy cleaning procedure is:

- Turn off all pumps to store enough water to complete the cleaning cycle.
- Meanwhile, set the sluice gate to deliver about 85 percent (refined by trial) of the last pump's capacity.
- Turn on every pump to lower the water level as rapidly as possible. A hydraulic jump forms at the bottom of the ogee and progresses downstream while its turbulence suspends and fluidizes the sand and sludge. By the time the jump reaches the end pump, all scum, sand, and sludge have been sucked out, and the trench is clean.
- Optional: turn the pumps off one by one before the jump reaches their intakes.
- Reprime the pumps to put the station back into service.

By manipulating the sluice gate, the jump can be made to travel downstream either rapidly or slowly or even be made to go upstream or remain stationary.

Sometimes, sticky solids do not readily slide down a 45° slope, so a slope of 60° is preferable if it can be used. Sloping walls should be covered with a smooth, durable coating.

In wastewater pumping stations, it is necessary to wash grease off the walls periodically, so design unobstructed access for housekeeping. An 8-mm (5/16-in) nozzle on a 25-mm (1-in) hose delivering 1.6 L/s (26 gal/min) at a pressure of 414 kPa (60 lb/in^2) is adequate for a small basin, but a 13-mm (1/2-in) nozzle on a 38-mm (1.5-in) hose delivering 3.6 L/s (57 gal/min) at equal or greater pressure is better for large basins. If the supply is potable water, adequate backflow prevention is obviously required.

The cleaning process given in this subsection was proven in the research testing to be many times more effective than that described in Section I.B for the Kirkland Pumping Station.

e. Rectangular Basins for V/S Pumps and Solids-bearing Water

Because no storage volume is required, pump inlets can be set as close together as in Figure 5. Close intake spacing is advantageous for reducing both solids accumulation and the impact on the treatment plant when sludge arrives after a pump-down. But to provide adequate access for maintenance, pumps should be separated by at least 1.1 m (42 in) clear. Spacing machinery well apart and intakes close together requires intake pipes to be spread at angles as shown in Figure 5. Of course, the intake spacing can equal the pump spacing with intake pipes at right angles to the trench for somewhat simpler construction, but the basin would be longer, more expensive, and limited in the advantages above.

A cone under the last intake is highly desirable, and the anti-rotation baffle is mandatory at pump-down to ensure uniform flow and minimum water depth upstream. But cones cannot be used under upstream pump inlets, because they would interfere with super-critical flow at pump-down, so use vanes instead. Note that the vanes are self-cleaning because the edges are nearly parallel with the flow of water into the intakes. Intermittent Type 1 vortices tend to form at the walls beside the suction bells even in trenches 2.5 D wide. See Figure 6 for vortex classification. In trenches about 2 D wide, continuous vortices of Type 2 or 3 occur. The vortex severity can be reduced by attaching vortex suppressors to the walls as described in Section V.E.g.

11. Follow guidelines 1 to 6 and 8. Additionally, follow the recommendations shown in Figure 5.

f. Rectangular Basins for C/S Pumps and Solids-bearing Water

These basins are similar to those for V/S pumps except for the need for storage and the design of the approach pipe. Equation 5 can be satisfied by the volume of the basin and the pipe together. The LWL for normal operation should be somewhat above the invert of the approach pipe at the basin to keep the hydraulic jump from migrating into the basin.

12. Follow guidelines 1 to 6, 9, and 10 plus the details in Figure 7. Note that vanes reduce pre-rotation and floor vortices.

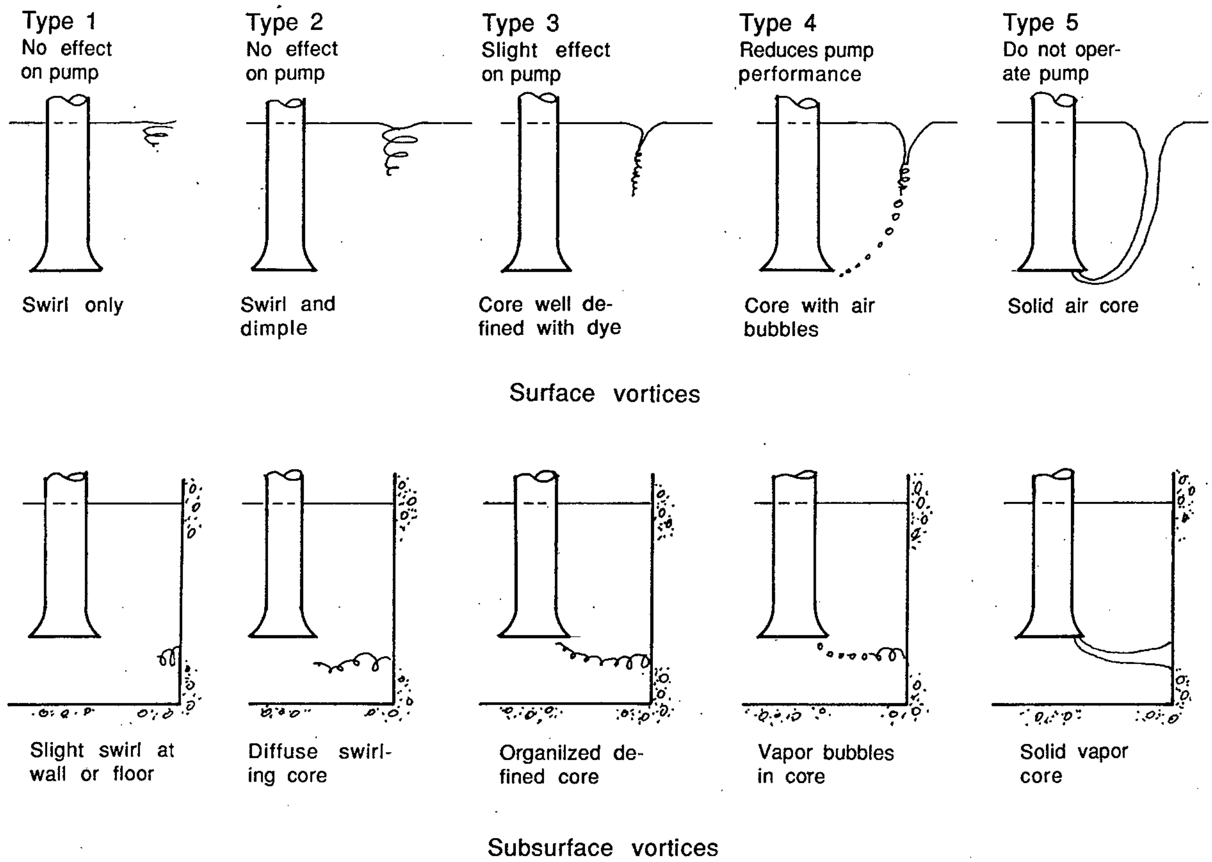


Figure 6. Vortex classification system

g. Submersible Pumps in Rectangular Basins

Because the trench is so important to the proper functioning of the intake basin, it must be included as shown in Figure 8, and hence the pumps must be fitted with suction bells. The upstream intakes should be well (say D/2) above the floor for the reason given in Section II.C.d, whereas the last intake should be no more than D/4 above the floor to ensure that the hydraulic jump moves quickly to the last pump at pump-down. (The floor under the last intake can be depressed if necessary to provide the generally accepted criterion of a minimum of 75 mm clearance for passing large solids.) These restrictions can be met by lengthening the suction bell of the last pump.

Stringy material is apt to collect on obstructions, so it is desirable to keep unnecessary objects such as guide rods out of the water. Telescoping guide rods (one tube within another) that are raised above the water except when needed to install a pump are therefore preferred.

13. Follow Guidelines 1 to 6, 9, 10, and the details in Figure 8.

14. Ensure that influent currents do not overstress the connection between pump and discharge elbow. Consult the pump manufacturer.

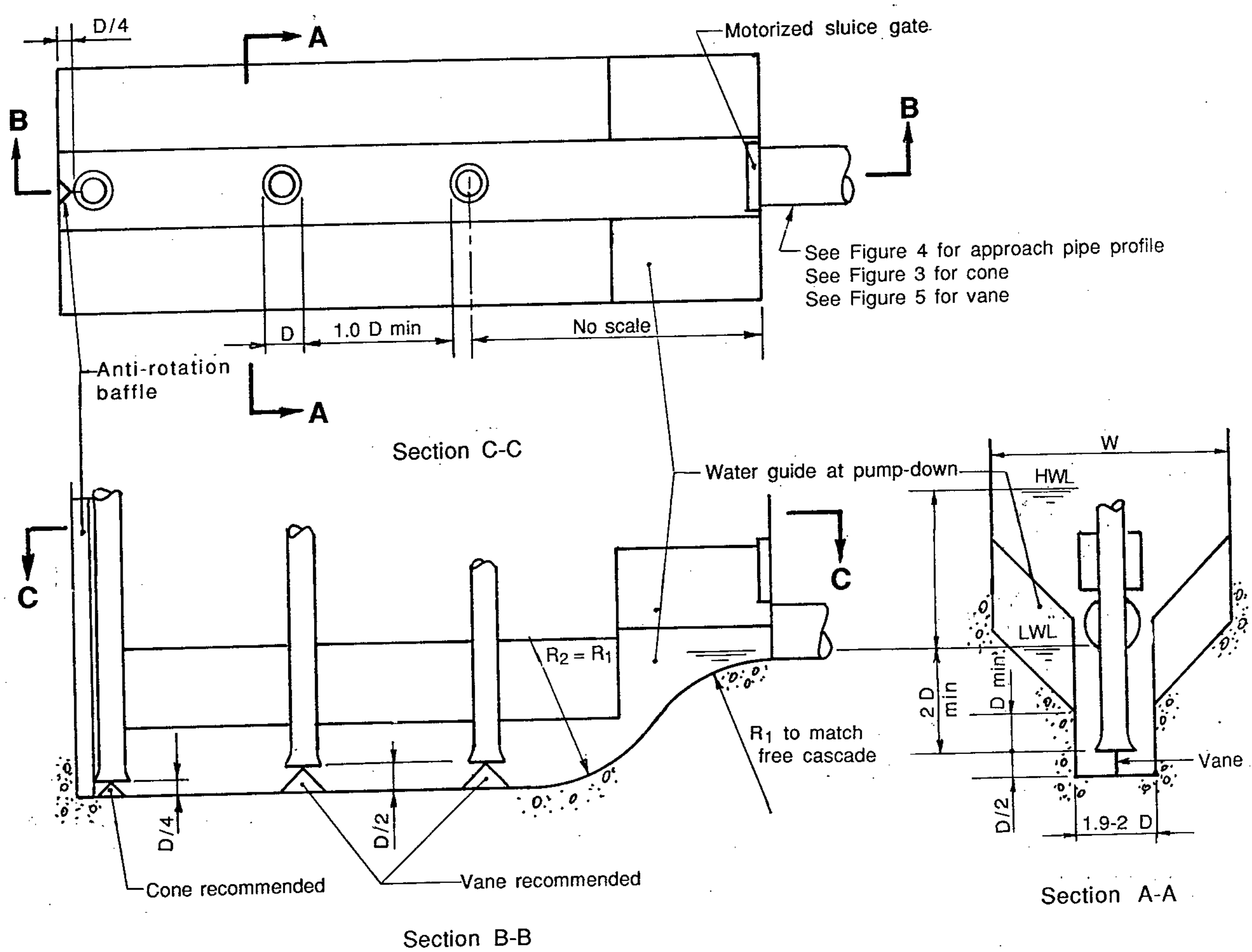


Figure 7. Rectangular sump for C/S pumps and solids-bearing water.

h. Round Basins for Solids-bearing Water and C/S Pumps

Round basins are typically used for dry pit, submersible, and self-priming pumps in small lift stations. Although V/S pumps could be used, the advantage of low cost would be compromised. The recommendations are similar to commonly accepted design criteria except for the sloping inlet pipe discharging slightly below LWL and a hopper bottom with walls sloping 45^{o} or steeper, as shown in Figures 9 and 10, to give the smallest possible flat floor size.

A very small free water surface at pump-down is required to discharge all scum. Note that pumps begin to lose prime when the submergence of the inlet becomes less than the inlet diameter, D. Flatter hopper bottoms or pumps that lose suction when the water level falls to the top of the volute of submersible pumps may not be fully effective in sucking out scum.

The smallest free water surface area at pump-down (and consequently, better and quicker cleaning) can be achieved by equipping submersible pumps with suction bells (a standard feature on some submersible pumps) set within a vertical-sided trench as shown in Alternate Section B-B in Figure 9.

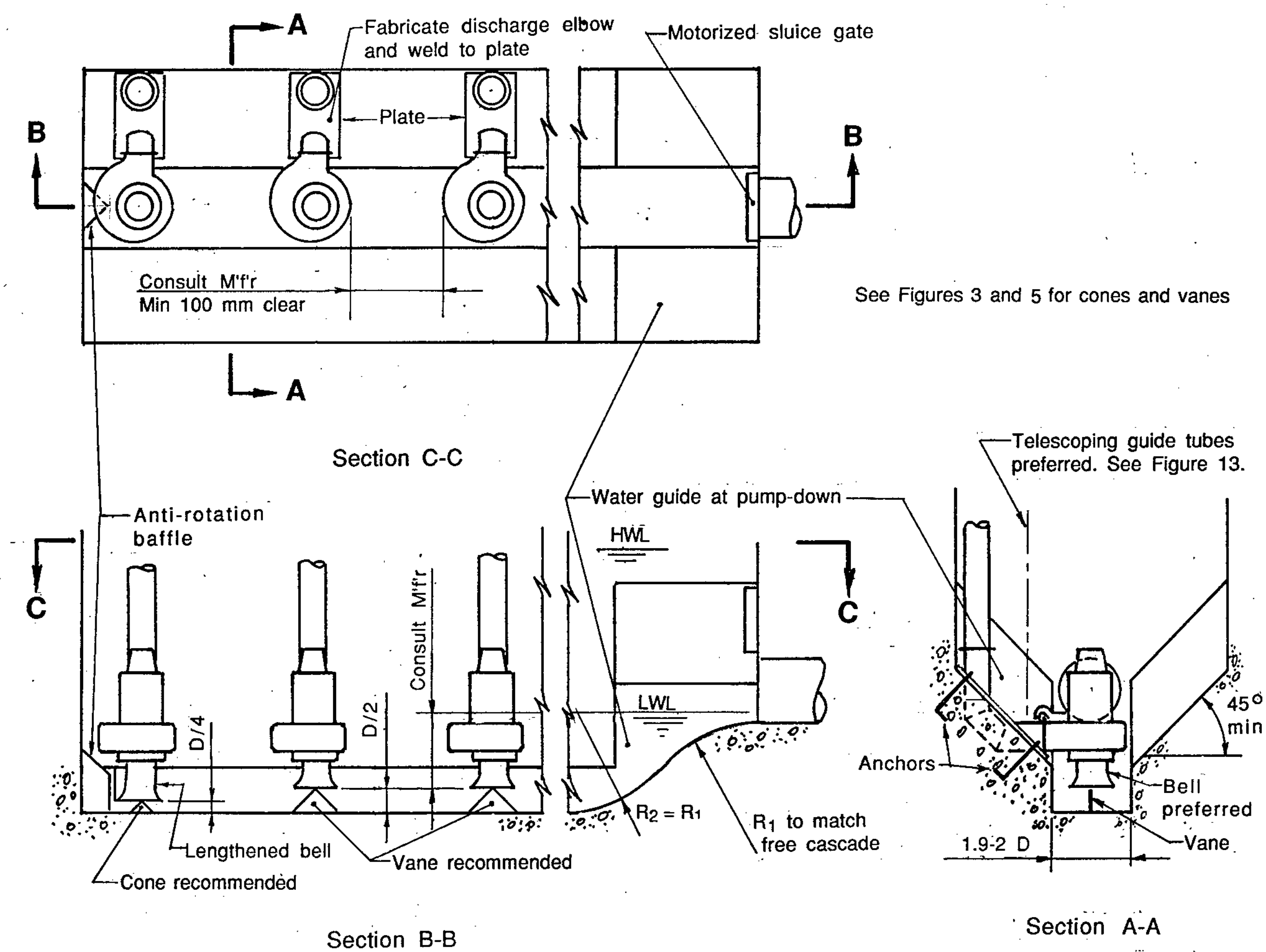


Figure 8. Rectangular sump for submersible pumps and solids-bearing water.

15. Follow Guidelines 1 to 6, 9, and 10. Additionally, follow the recommendations shown in Figure 9 for submersible pumps or Figure 10 for self-priming pumps.

For duplex and especially for triplex pump installations, good cleaning can be obtained without pump-down (or with only partial pump-down) by operating the pump(s) while mixing the contents. Either a mechanical mixer or the piping system described in Section III.D is effective. By avoiding pump-down, pumps do not have to be reprimed. Basins can also be kept continuously clean with a mechanical mixer by programming the mixer to operate for a few minutes in every pump start-stop cycle. However, if intakes are set as close together as shown in Figures 9 and 10, settleable solids cannot accumulate to a significant amount.

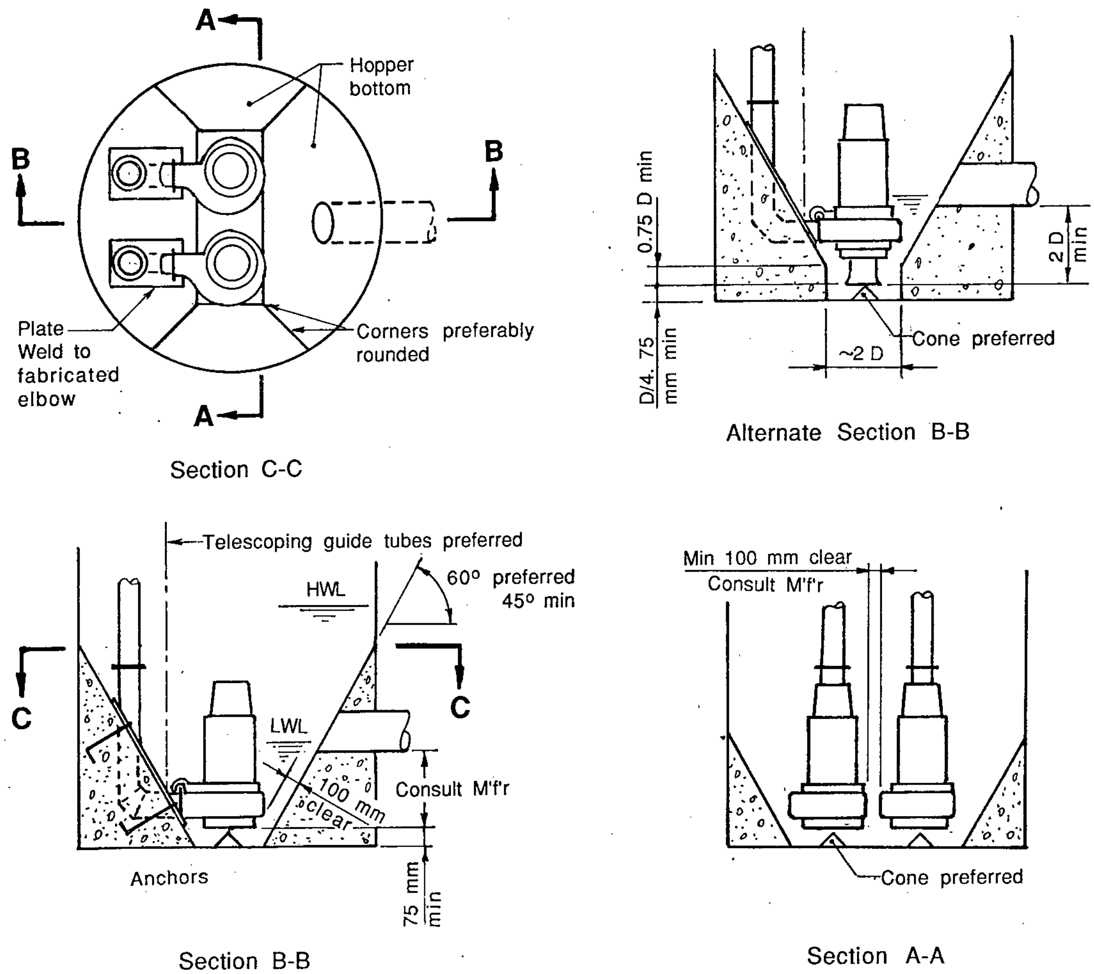


Figure 9. Duplex submersible pumps in round sump.

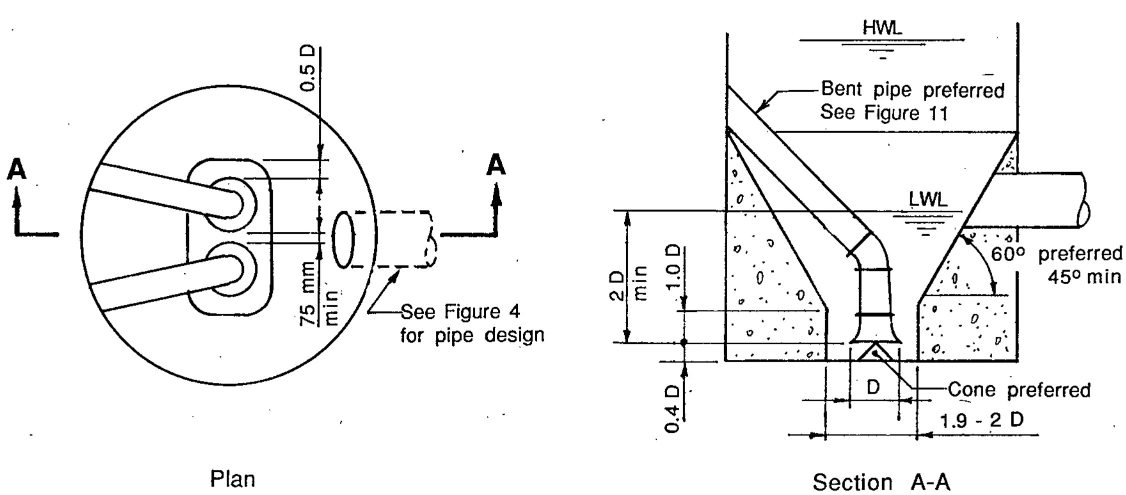


Figure 10. Sump for duplex, self-priming pumps

SECTION III

INVESTIGATIONS OF PROTOTYPES

Inspection visits were made April 6 and 7, 1992 to four rectangular, trench-type pump sumps and to two traditional pump sumps in the Seattle area. Kirkland pumping station was revisited May 24, 1993.

Small pumping stations with round pump intake basins were visited May 25, 1993, at Black Diamond, WA and at Clyde, CA on May 27, 1993. On August 28, 1993, a pumping station at Vallby, Sweden and two similar ones not far away were inspected.

Tests were made December 1 to 3, 1993 on the full-sized pump inlet basin constructed by Fairbanks Morse Pump Corporation at their plant in Kansas City, KA.

A. KIRKLAND PUMPING STATION

Twenty seven Seattle Metro pumping stations have pump inlet basins somewhat similar to Kirkland pumping station (Figure 1). Because the Kirkland station is typical, it was chosen for modeling.

The station and its operation are described in Section I.B. The phenomenon that limits the velocity along the trench during pump-down is the formation of a Type 5 vortex beside the suction bell at 7 o'clock (where 12 o'clock points upstream). A Type 5 vortex pulls a solid air core into the pump intake. See Figure 6. The air sucked into the vortex prevents reducing the submergence of the suction bell mouth to less than about 0.7 D. A strong counterclockwise (as viewed from above) circulation of water around the last pump intake causes an upstream movement of water along one wall, and it terminates in an unstable stagnation area where movement of deposits is extremely slow.

1. Critique of procedure--The cleaning procedure described in Section I.B results in a minimum pump-down water depth of 1.06 D. At the estimated flow rate of 66 L/s (227 m^3/h or 1.5 Mgal/d), the fluid velocity along the trench is only 0.20 m/s (0.66 ft/s) which is insufficient to move grit, sand, or even heavy sludge although turbulence from the cascade does move some sludge. After cleaning, however, an average depth of about 50 to 75 mm (2 to 3 in) of sludge remains, and this depth increases the average velocity by about 10 percent.

Without altering the design, more effective cleaning could be accomplished by following the procedure of Section II.C.d, namely, by setting the sluice gate to pass only about 85 percent of the last pump's capacity, shutting off the pumps for about 15 minutes (or for enough time to store sufficient fluid in the upstream pipe), turning on all pumps until the submergence of the pump intakes is reduced to about 1.0 D, then operating only the last pump at top speed until the stored fluid is pumped out. Both the velocity along the trench and the energy in the influent cascade would increase by about 70 percent, so cleaning would be improved although the average fluid velocity along the trench would still be too low to move sand.

2. Critique of design--As a result of the research described in the following sections, it would be possible to improve the design by a quantum leap.

Lowering the last suction bell to a floor clearance of D/4 would increase the velocity in the trench at pump-down by 25 percent. Adding a baffle between the last suction bell and the end wall to eliminate the circulation behind the bell would produce a uniform upstream velocity, reduce the water depth to about 0.5 D, and thus increase the velocity to about 1 m/s (3.2 ft/s)--nearly five times that attained by the existing procedure described in Section I.B. Such a velocity is enough to move sand at nearly 15 mm/s (3 ft/min).

These improvements would be dwarfed when compared to the improvement achievable by converting the potential energy of the influent stream into kinetic energy at pump-down by means of a smooth ogee entrance as shown in Figures 5, 7, and 8. Velocities of 3 to 4 m/s (10 to 13 ft/s) can be developed, and it is apparent from both model and prototype studies that all sand, sludge, and scum could be ejected from such pumping stations in less than half a minute after the water is pumped down to a depth of 1.0 D.

B. OTHER SEATTLE AREA PUMPING STATIONS

The Wilburton and North Mercer Island pump intake basins were similar in design to the Kirkland pump basin, and, as the stations were operated in the same manner, the responses and results were similar. This similarity demonstrated that the response was a function of design and not differences in sewage due to different origins.

At Steilacoom the trench was very wide (2.29 D), and, due to a construction mistake, the suction bells were set too far (0.9 D) from the floor. The sump could not be cleaned adequately, and odors during cleaning were very strong. This response demonstrates the value of narrow trenches and bells mounted close to the floor. However, a survey of the sludge deposits was of value to the project, because it furnished different conditions for testing model sludge. To guard against construction errors, designers should specify that floor clearance of bells is a controlling dimension.

The 53rd and 63rd Street pumping stations are entirely different sumps with large flat floors that collect great quantities of grit in piles that reach three feet in depth. Both produce suffocating odors, and both can be cleaned only with difficulty and at considerable expense. They amply demonstrate the need for designs that make frequent cleaning quick, easy, and economical.

C. BLACK DIAMOND PUMPING STATION

This station was designed to meet most of the concepts developed during this research project for C/S duplex pumps in a round, self-cleaning pump inlet basin. The plans are shown in Figure 11. The 400-mm (16-in) influent pipe slopes at a 2 percent gradient for 61 m (200 ft) but is horizontal for 10 pipe diameters before entering the wet well. There are two self-priming pumps of 63 L/s (230 m^3/h or 1000 gal/min) each housed in a nearby building with only the 250-mm (10-in) suction pipes in the wet well. The diameter of each bell is 400 mm (16 in), so the entrance velocity is only 0.49 m/s (1.6 ft/s).

a. Critique

The behavior of the steeply sloping (2 percent gradient) approach pipeline was satisfactory. In the sump, the smooth sides sloping at 60° kept solids from sticking during pump-down. The pumps broke suction when the submergence of the bells was about 1 D. At this water depth, the area of the water surface was too large for sufficient confinement of scum and a second pump-down was needed. It appeared that operating the two pumps simultaneously would remove the scum in a single pump-down. The LWL shown on the plans is too low to prevent the hydraulic jump in the approach pipe from reaching the sump.

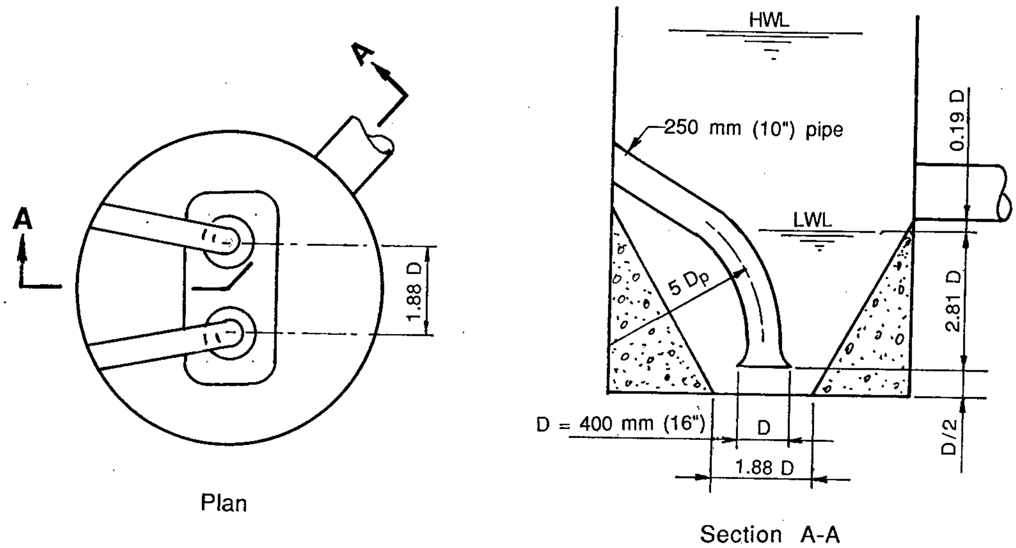


Figure 11. Black Diamond Pumping Station sump.

The pump entrance velocity is very low and might not suck out sand and gravel quickly. Nevertheless, cleaning is accomplished with reasonable dispatch. However, the designer stated he would follow the more restrictive dimensions of Figure 10 next time.

The success of the Black Diamond Pumping Station demonstrates the practicality and usefulness of the concepts developed for this research.

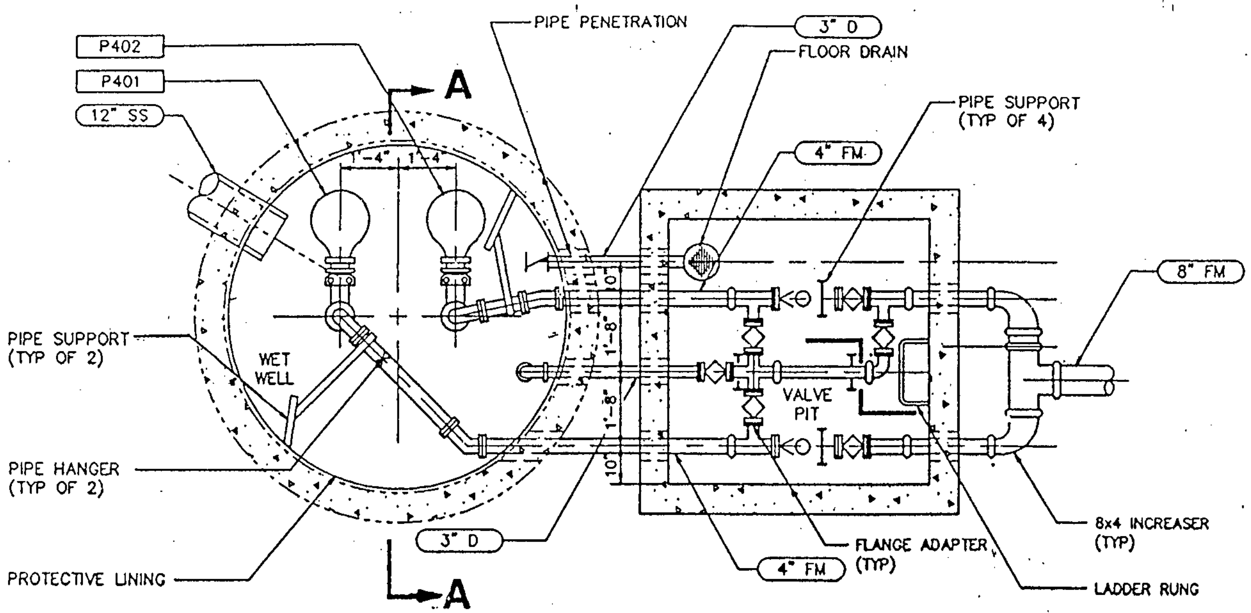
D. CLYDE PUMPING STATION

The pumping station at Clyde in Contra Costa County, California is shown in Figure 12. The 3" piping system is used to mix the contents of the sump for discharging scum and sludge to the force main. Smaller pipes might plug. The four eccentric plug valves allow either pump to be the wash-water pump while the other one pumps the "homogenized" mixture into the force main. Alternatively, either or both pumps can discharge to the force main, and the force main itself can be tapped for about 15 percent of its flow to be recirculated as wash water. The 3" pipe discharges just above LWL. At the time of inspection, the sump was remarkably clean with no material floating on the surface.

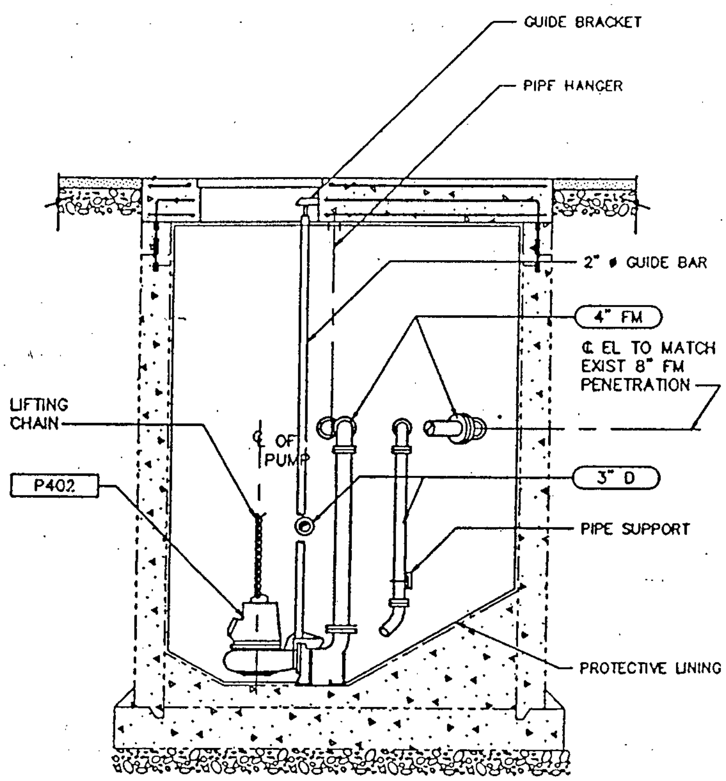
a. Critique

The 30° bottom slopes allow more active storage volume than 45° or steeper slopes would, but the penalty is that mixing is required to eject scum and sludge. The 3" piping system allows maximum versatility in mixing. If a less expensive system is wanted, the system could be reduced to one valve at the 4" F M. Flexibility would suffer, but there would be little loss of effectiveness. Discharging the 3" piping at or above the LWL drives bubbles into the pool, whereas discharge at a lower elevation would be just as effective and would avoid the bubbles.

A disadvantage of the system is the necessity for the operator to enter the valve vault to operate the valves. Steeper hopper bottoms (as in Figure 9) avoid the need for mixing but may require a wider range between HWL and LWL to achieve an adequate active storage volume.



Plan



Section A-A

Figure 12. Clyde Pumping Staton. Courtesy of G.S. Dodson & Associates.

The only modifications required to make the Clyde wet well comply with the concepts in this research are a sloping approach pipe discharging at low water level and steeper slopes designed to hug the pumps more closely on all sides.

E. PUMPING STATIONS IN SWEDEN

Three small pumping stations in Västerås were inspected. The Vallby pumping station was the first one built, and because of its success, the other two are similar. Other pumping facilities continue to be constructed along the same lines.

a. Vallby Pumping Station

The Vallby pumping station, shown in Figure 13, was designed by an experienced operator (not an engineer) who wanted a facility that would require minimum attention and maintenance. The hopper bottom is made of 18-8 stainless steel plate 12 mm (1/2 in) thick bent so that its top fits the round, vertical concrete pipe and its bottom conforms to a rectangle of minimum size with rounded corners. The sides are inclined at about 60^{o}. Special discharge elbows were made and welded to a heavy plate for bolting to the hopper side. The space between the hopper bottom and the side of the round pipe is filled with concrete. The sump was perfectly clean at the time of inspection, and a demonstration of pump-down to the lowest achievable water level demonstrated that clean-out would be very effective indeed. The station has several unique features worthy of description.

- Guide rails are stainless steel telescoping tubes that are raised out of the water except when a pump is to be reinstalled.

- Instead of using floats to monitor water level and control the pumps, a piezo-electric pressure cell is placed within an open 100-mm (4-in) PVC pipe near the bottom of the sump. These cells are reliable, long-lived, and are excellent for sophisticated systems because they can, unlike floats, provide input throughout the liquid level range to a programmable logic controller for activating the pumps for both normal operation and pump-down. In the long run, they are probably less expensive than floats. (Some floats must be replaced yearly, although there are some that contain micro-switches instead of mercury and are supplied with more resistant electrical cables that can last for many years).

- A fresh water supply for washing is equipped with a quick-connect and valve and is protected by a large pipe with a padlocked cover.

- Inside the wet well, there is a wash-down water lance that slides in a collar supported by a universal joint that permits freedom of direction. The water lance is equipped with a quick-connect at the top and a nozzle at the bottom. A short hose with mating quick connects on each end is carried in the operator's truck. The system makes wash-down not only quick and easy, but the jet strikes the pump or hopper bottom so far away that the operator is not splashed.

- Two grates consisting of heavy rods spaced at about 150-mm each way covers the opening and give workers a feeling of safety. They are hinged under the cover of the curb. Only one must be lifted to remove a pump. The curb is a support for the 150-mm pipe that carries the power cable from the control box to the sump. It takes only seconds to disconnect the power cable.

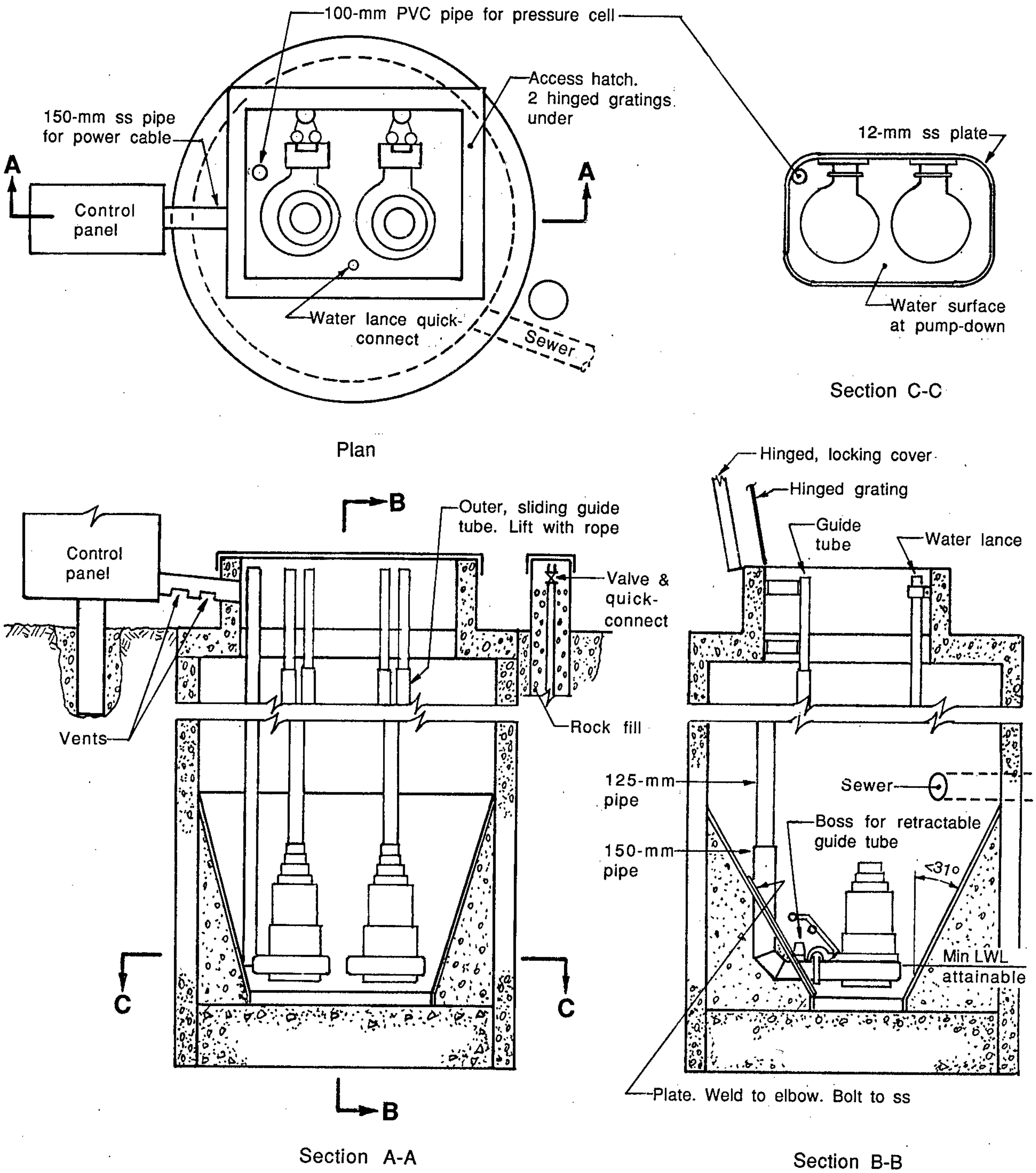


Figure 13. Schematic diagram of Vallby Pumping Station.

- Operating water levels can be easily changed at the control panel. A push button is provided tor making a motor run backwards to clear a clogged pump.

1. Critique--If the waterfall into the wet well were avoided by means of an approach pipe laid at a 2 percent grade and discharging at low water level, these wet wells would conform to all the concepts for self-cleaning wet wells developed in this research project.

The stainless steel hopper-bottom insert is excessively thick and expensive. It need only be stiff enough to allow concrete to be placed between it and the concrete pipe wall. A molded plastic shell would be much cheaper (particularly if standardized and made in quantity) and just as satisfactory.

The hopper bottom is so small that significant amounts of sludge cannot accumulate. But if scum is a problem, the controls could be programmed for automatic pump-down at some suitable interval, say, once per day.

b. Other Swedish Pumping Stations

The other Swedish pumping stations are similar to the Vallby not only in design but also operationally. One, designed for a future development, was connected to only a few houses, so the detention time was very long and produced overwhelming odors. The steep-walled sump is advantageous, because detention time and odors can be reduced by setting the HWL close to the LWL to increase the frequency of pump starts and keep the wastewater at least a little fresher. Adding a small flow of fresh water would also help as would feeding iron chloride to sequester the sulfide ion. A small, obviously inexpensive hydro-pneumatic tank was installed in this station for controlling water hammer. Many such tanks have been in use for many years and are said to be quite satisfactory and devoid of excessive maintenance problems.

F. FAIRBANKS MORSE EXPERIMENTAL PUMP INTAKE BASIN

A full-sized, steel, rectangular pump intake basin was constructed at Fairbanks Morse Pump Corporation in Kansas City, Kansas. Pump 3 was an end suction pump always throttled to 63 L/s (230 m^3/h or 1000 gal/min), 67 percent of its capacity. It was mounted beside the sump as shown in Figure 14. Pump 2 was a self-priming pump mounted 3.3 m above the trench floor. To have a basin of proper proportions, space was left for Pump 1 (never installed).

a. General Procedure

Water was pumped from a test pit under the floor to the pump intake basin by a supply pump (not shown) until the desired depth was reached. Then Pump 3 (sometimes augmented by Pump 2) was operated to recirculate water to the standpipe. A bypass pipeline allowed water to be returned directly to the test pit. Before pump-down, water was recirculated to the standpipe at 63 L/s (230 m^3/h or 1000 gal/min), and it entered the sump at a velocity of 0.94 m/s (3.1 ft/s)--roughly the same as the drowned pipe velocity for two pumps operating. During pump-down, water was bypassed (except as noted) at 12.6 L/s (45 m^3/h or 200 gal/min) so that influent to the sump was only 50.4 L/s (181 m^3/h or 800 gal/min).

b. Short Radius Toe

In the first pump-down tests, influent to the basin spread laterally over the ogee apron and some flowed over the 45^o sloping wall only to drop into the side of the trench, interfere with the hydraulic jump and create a shock wave or "rooster tail" nearly as high as the vertical trench wall. The velocity along the centerline of the trench moved at about 1.8 m/s (5.8 ft/s), a speed that, if uniform, would move sand at about 2 m/min and thus clean the trench in about a minute and a half. But the velocity was decidedly non-uniform and when a test with sand was made the hydraulic jump stayed at the toe of the ogee. The pump lost prime in 1.5 minutes, so much of the sand was not removed.

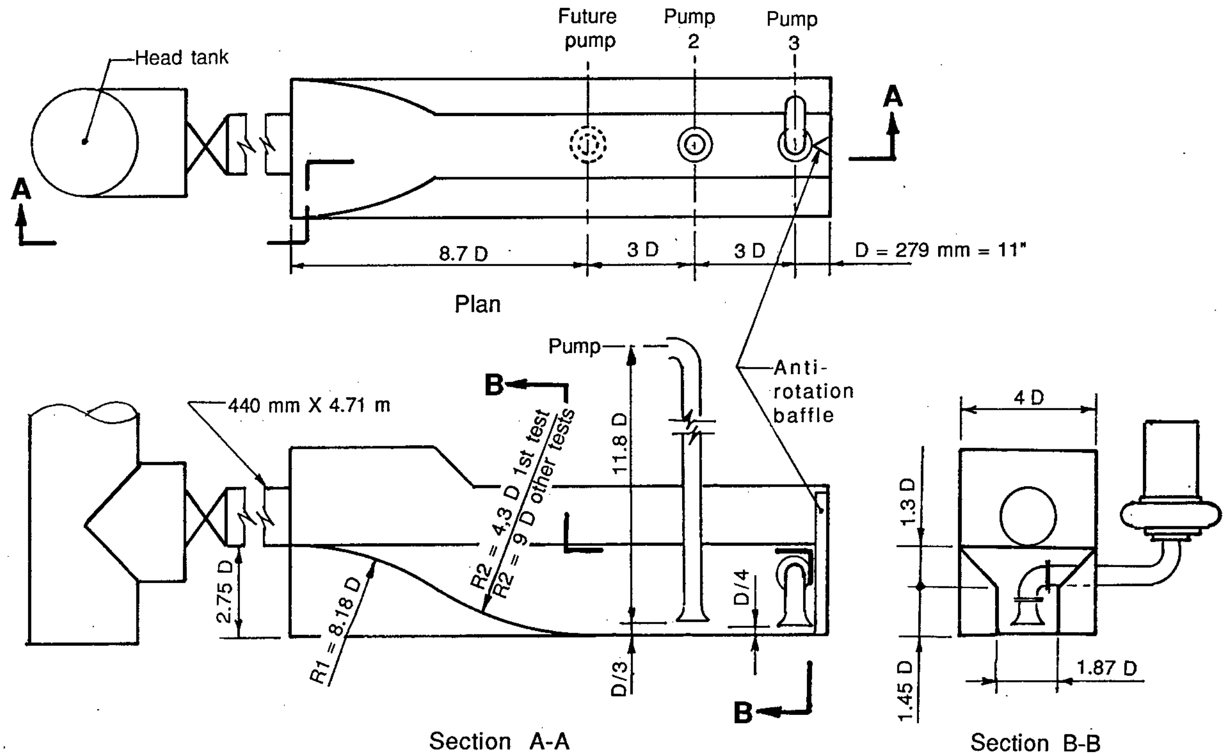


Figure 14. Experimental pump intake basin at Fairbanks Morse Pump Corporation plant.

c. Long Radius Toe

The short-radius (4.3 D) toe was replaced with a long-radius (9 D) toe and 50 mm (2 in) of sand was placed over the entire bottom of the trench. The hydraulic jump was better, it moved downstream about 3 D, and the rooster tail was much smaller. Although the pump lost prime one minute after the jump formed, nearly all (probably 90 percent) of the sand was removed. A second pump-down removed the remaining 10 percent during a 45 second run. The Froude number at the toe was estimated to be 7.6 based on rather crude measurements of the depth. On the assumption that Manning's n-value was 0.010, calculations gave a Froude number of 6.5 at the toe and a value of 1.99 (a very weak jump) at the end of the basin. Ignoring friction produces very large errors in calculated Froude numbers. (See Section VI.E.)

d. Straight Wing Walls

After adding the wing walls shown in Figure 15, the hydraulic jump improved markedly and moved almost to Pump 3. The rooster tail was subdued to a height of about D/2. The Froude number at the toe of the ogee was calculated, ignoring friction, to be 10.1--a value friction would reduce. Crude water depth measurements resulted in an experimental value of 7.6. A Froude number above 9 indicates a very strong jump, perhaps too strong. Operators do have a limited control of the jump by regulating the amount of water released by a sluice gate.

By operating Pump 2 at 50 L/s (180 m^3/h or 800 gal/min) and Pump 3 at 63 L/s (230 m^3/h or 1000 gal/min) and bypassing 31 L/s (110 m^3/h or 500 gal/min), the hydraulic jump behavior was improved still more. The jump went under Pump 2 (which had stopped pumping a moment before) and migrated all the way to Pump 3 in 23 seconds. This performance was excellent, so the system was tested with about 0.037 m^3 (1.3 ft^3) of sand placed at the upstream end of the floor. All of the sand was ejected in 21 seconds following the formation of the jump at the toe of the ogee. Losing prime was expected, because the volume of water was being continually reduced at 12 L/s (44 m^3/h or 200 gal/min). The sand washout rate was excellent. In subsequent model tests at Montana State University, sand was removed in the same length of time. See Section V.B.c.

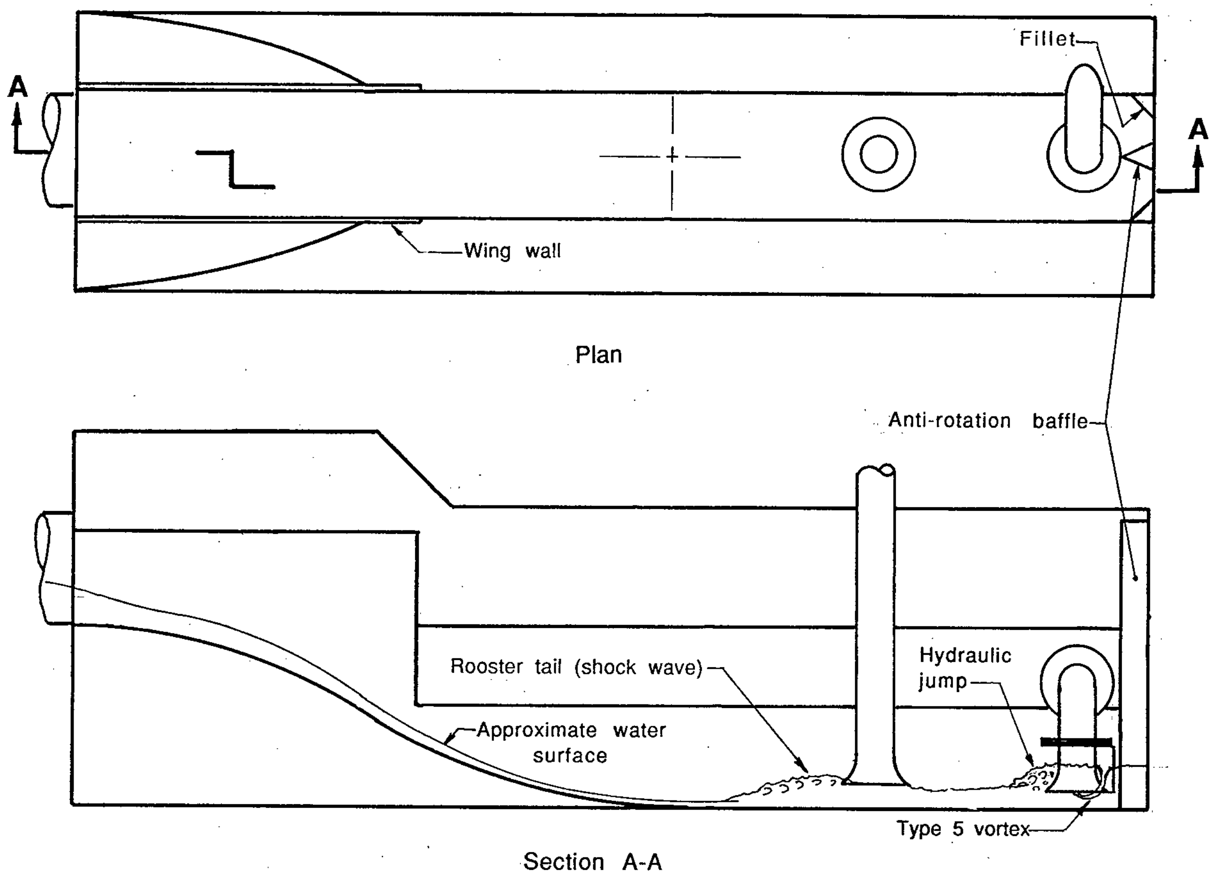


Figure15. Straight wing walls in Fairbanks Morse pump intake basin.

e. Fillets

Fillets placed in the corners of the trench at Pump 3 as shown in Figure 15, considerably reduced the size and vigor of the vortices that enter the intake, reduced the amount of air ingested, apparently lowered the upstream water surface slightly, and helped to move the hydraulic jump downstream somewhat quicker.

f. Tapered Wing Walls

Tapered wing walls, shown in Figure 16, were substituted for the straight ones. Tests with water levels ranging from the center of the influent pipe to halfway down the ogee gave excellent results with stable, uniform currents. At pump-down, however, water flowed over the sloping walls, spilled into the trench from the sides, and interfered with the hydraulic jump. The rooster tail was nearly as high as the trench and impinged on Pump 2 intake where it substantially interfered with the water passing under the intake.

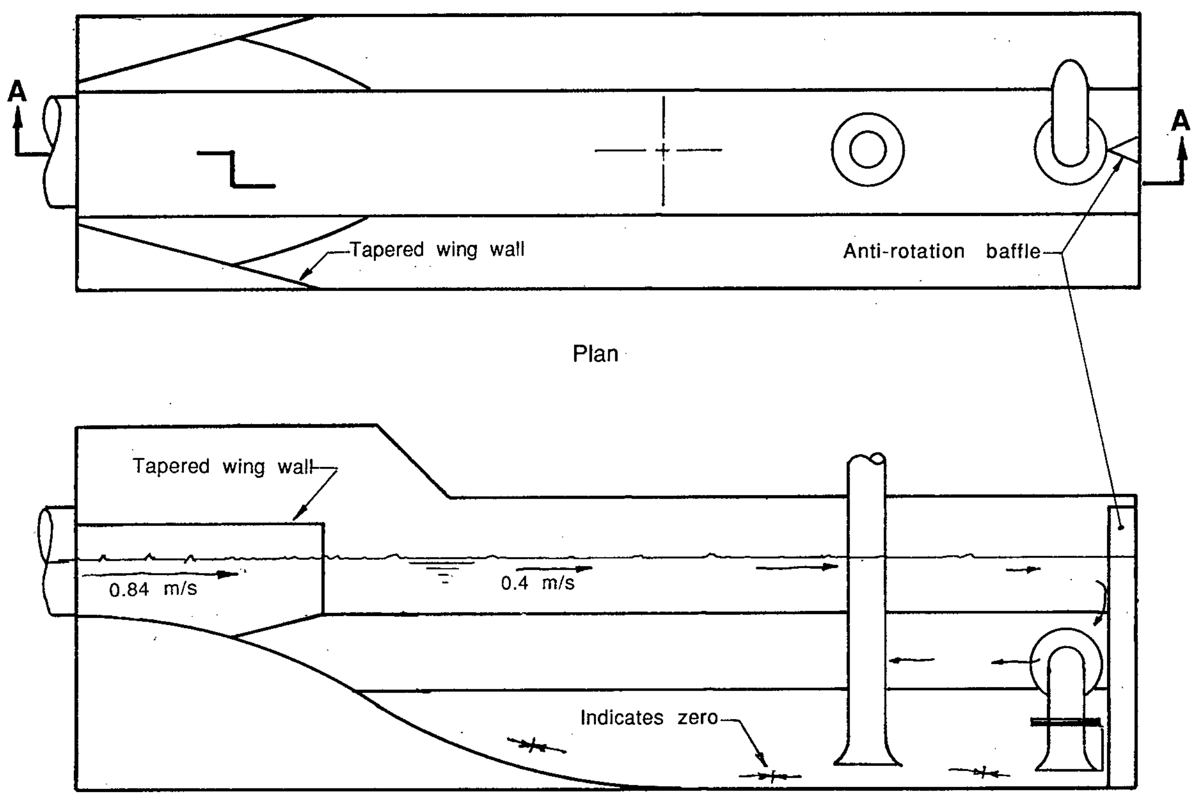


Figure 16. Tapered wing walls and relative velocity vectors in Fairbanks Morse pump sump.

g. Currents

At a circulation rate of 63 L/s (230 m^3/h or 1000 gal/min) and a water level at the midpoint of the influent pipe, the entrance velocity was 0.84 m/s (2.75 ft/s). Currents traced with dye are shown in Figure 16. Equal and opposite vectors indicate stagnation. As dye quickly disappeared due to the turbulence, velocities were difficult to obtain, and their values are only approximate. In all tests, however, it was noticeable that at more than I D from an operating pump intake, currents were very low, and at more than 2 or 3 D from an operating pump intake, water--if it moved at all--generally migrated upstream. The velocity past the pump intakes was insignificant, and it certainly could not be defined by assuming plug flow.

h. Conclusions

- The effectiveness of the trench-type pump intake basin was confirmed for both normal operation and for expelling sand and scum.
- Relatively large quantities of sand were expelled within about a half minute at pump-down.

- The last pump intake must be D/4 or less above the trench floor. A lower placement (with a shallow pit to ensure the passage of a large (75-mm diameter) solid would hasten sludge removal and keep the pump primed longer.

- Upstream pump intakes must be at least D/3 above the trench floor. Because of shock waves at pump-down, a clearance of 0.4 or 0.5 D is safer.

- Fillets at the corners by the last pump are helpful and the anti-rotation baffle is necessary to move the jump to the end of the basin.

- The movement of the jump can be regulated by throttling the inflow.

- Currents at intakes are insignificant even when surface currents are substantial.

SECTION IV

MODEL STUDIES AT ENSR

Models of the entire pump intake basins and the inclined approach pipeline were studied by ENSR Consulting and Engineering (ENSR) in their hydraulic laboratory in Redmond, Washington. Laboratory personnel were supervised by Sweeney, who has tested more than 70 models of pump sumps. The research that could be adequately accomplished in a small flume was done by Sanks in the hydraulic laboratory of the Department of Civil Engineering, Montana State University, Bozeman, Montana.

A. MODEL SIMILITUDE

True similitude requires that both Froude number (Equation 2), a dimensionless function of gravitational and inertial forces, and Reynolds number, a dimensionless function of viscous and inertial forces, be the same for both model and prototype. Reynolds number is

$$R = vD/\nu \qquad (7)$$

where v is velocity, D is depth, and ν is kinematic viscosity. Both numbers cannot be satisfied simultaneously unless the model/prototype scale ratio is unity, so a choice must be made. When a free surface exists such as in a pump intake basin, flow patterns are influenced primarily by gravitational forces, and therefore, similitude must be based on Froude number, F. Despite the incompatibility of Froude and Reynolds scaling, the model linear scale must still allow flow to be turbulent. Reynolds numbers greater than 10^4 to 10^5 during model operation assures adequate turbulence. To allow the use of commercial pipe sizes, the model/prototype linear scale ratio, L, was chosen to be 1/3.63. For equal model and prototype values of F, other model/prototypes scale ratios are:

Velocity, $v = (1/L)^{0.5} = (1/3.63)^{0.5} = 1/1.91$

Time, $t = (1/L)^{0.5} = 1/1.91$

Volume, $V = (1/L)^3 = 1/3.63)^3 = 1/47.8$

Flow rate $Q = V/t = (3.63)^{0.5}/(3.63)^3 = 1/25.1$

Flow in pipes is a function of Reynolds number.

B. MODEL TESTS OF PUMP SUMPS

Model tests of wet wells or pump sumps are always made with clear water. Gathering data consists of visual observations of current patterns by tracing them with dye and measuring quantifiable sump performance parameters. Swirling in pump intakes (promoting cavitation) is indicated by a neutrally-pitched rotor in the suction pipe or within the casing representing the pump. Flow into each pump intake is induced by a central suction system and controlled by valves and measured by individual elbow flow meters. Flow patterns are assessed visually.

a. Critical Measurements with Clear Water

The pump intake basin performance parameters that were measured are as follows:

1. Individual pumps
   - Pre-rotation in pump suction intakes was measured by a neutrally pitched rotor (that revolves only if the water is swirling).
   - Vortex formation was visually observed with the aid of dye.
   - Bubbles entering pump were visually observed.
2. Pump intake basin
   - Stable hydraulic conditions were confirmed by watching dye patterns.
   - Velocity past pump intakes was measured by using the Nixon meter or estimated by using dye tracers.
3. Approach pipe
   - Dissipation of high entrance velocities in the basin pool was recorded using a Nixon meter.
   - Eddy generation was visually observed by using dye.
   - Bubble formation within the pipe was visually observed.
   - Persistence of bubbles in the basin was visually observed.

Criteria for allowable errors and the precision and accuracy of the instruments used are given in Table 2. There is no single measurement that can be said to be the most critical one. All are important for assessing the adequacy of the basin. Flow rates were determined by measuring pressure differences across the flow meters with an air/water manometer.

TABLE 2. QUANTITATIVE CRITICAL MEASUREMENTS

Observation	Instrument	Allowable error	Precision	Accuracy	Detection limit
Total model flow	Orifice meter	5%	1-2%	1-2%	2.5 mm differential or 0.3 L/s
Individual pump flow	Elbow meter	6%	3%	4%	2.5 mm differential or 0.3 L/s
Swirling in pump intakes	Rotor meters (Time average basis)	$\pm 1^{o}$	$\pm 0.1^{o}$	$\pm 0.5^{o}$	0.1^{o} (visual)
Currents in pipe and near inlets	Nixon meter (Time average basis)	5%	1%	3%	0.03 m/s
Vortices	Visual using dye and classifying into Type. See Figure 6.				

Visual observations are particularly revealing. The use of dye (usually potassium permanganate) ejected from a wand (long, small tube) at any desirable point in the wet well allows easy detection of dead spots, unusual currents, eddying, or vortex formation all of which indicate potential design problems.

The standard orifice meter is an aluminum plate machined to ASME specifications and calibrated volumetrically. The meter discs are so chosen that the Reynolds number at minimum model flow is greater than 10^4.

The elbow meters are calibrated as follows:

- Use the orifice meter as the calibration standard.
- For at least five flow rates within the expected range for each elbow, measure the head differential, H, across the elbow and note the flow rate indicated by the orifice flow meter.
- Perform a log-log regression on the set of data pairs to get a calibration equation of the form:

$$Q = a\,H^b \tag{8}$$

- Check R^2, and if less than 0.98, repeat the calibration procedure.

Swirling in the pump intakes is determined by rotors made of brass with neutrally-pitched vanes. They are mounted on a freely turning axis coincident with the pump axis. The angle of deviation of the approach flow is computed as

$$\beta = \arctan\,(U_l/U_a) \tag{9}$$

where β = angle of approach, U_l = average tangential speed of the blade tips, and U_a = average axial velocity.

Flow velocities are measured with a velocity probe manufactured by Nixon Instruments of Great Britain. The probe has a 1 cm diameter plastic rotor, the blades of which pass between an electrode and the probe housing or ground, thus changing the underwater electrical resistance. This provides a current impulse which can be counted over a specific time interval. The resulting frequency is compared to a calibration curve of frequency versus velocity. The original calibration curve was developed in the ENSR calibration tank. The probe can be rotated and positioned to take readings at any point. A digital counter and timer is used to determine average values of the current impulses from the velocity probe. Any time-averaged velocity, U, is computed as

$$U = \sum_{i=1}^{i=n} U_i/n \tag{10}$$

Temporal fluctuations of velocities (turbulence) can be measured using the Nixon velocity probe connected to a strip chart recorder. The current impulse from the miniature rotor velocity probe is recorded on the strip chart recorder for a thirty second period. The maximum temporal fluctuation ($\Delta\Sigma U_i$) for each thirty second period is measured and compared to the time-averaged velocity.

Vortices are classified in accordance with Figure 6. When severe enough, they can cause cavitation and vibration which can quickly destroy a pump, so they are indeed a critical measure of wet well performance.

b. Allowable Levels of Performance

Allowable levels usually adopted for sump performance are:

- No organized vortices (greater than or equal to Type 2) should enter the pump intakes from either a free water surface or a submerged boundary.

- Pre-rotation of the flow entering a pump intake should be less than 3° to 5° from axial.
- There should be no excessive turbulence or instability of flow entering the pump intakes.
- Velocities of flow at the impeller entrance should be symmetrical with respect to the impeller axis.

Velocity measurements were not made at the model impeller entrance locations for these studies because of the small size of the model and prototype pumps.

c. Non-Critical Measurements and Independent Test Variables

The geometry and dimensions of the basin, placement of pumps or pump intakes, orientation of pumps, and water level (a variable measured with a staff gauge) are examples of independent variables. The accuracy needed in a staff gauge is about 8 mm (about 25 mm in the prototype) whereas the accuracy obtainable with ease is 3 mm.

C. KIRKLAND MODEL

The purpose of the tests on Kirkland pump intake basin was to establish a relation between model and prototype performance and to have a basis for comparing other basins and procedures.

a. Construction

The model pump sump was constructed in a basin with one acrylic wall to permit visual inspection of currents and sediment deposits. Details of the sump corresponded exactly with the plans in Figure 1.

b. Normal Operation. V/S mode

The slope of the influent sewer in the model was set at 2.0 percent to simulate the actual Kirkland condition for V/S pumps.

Normal operation with a water level between the invert and soffit of the influent pipe occurs about 99.7 percent of the time. Performance is excellent at Kirkland, where the original pumps are still in service after nearly 30 years of operation.

Typical current patterns for full pumping capacity are shown in Figure 17 for two different duty pumps and for a simulated prototype influent velocity of about 1.5 m/s (5 ft/s) The arrows show only directions and not velocity. This velocity produced a small amount of air entrainment, but nearly all of it escaped to the surface by the time flow reached Pump 3.

1. Test of Pump 3--Pre-rotation was 14.5° clockwise, a value that considerably exceeded the desirable maximum of 5°. There were no organized surface vortices entering the

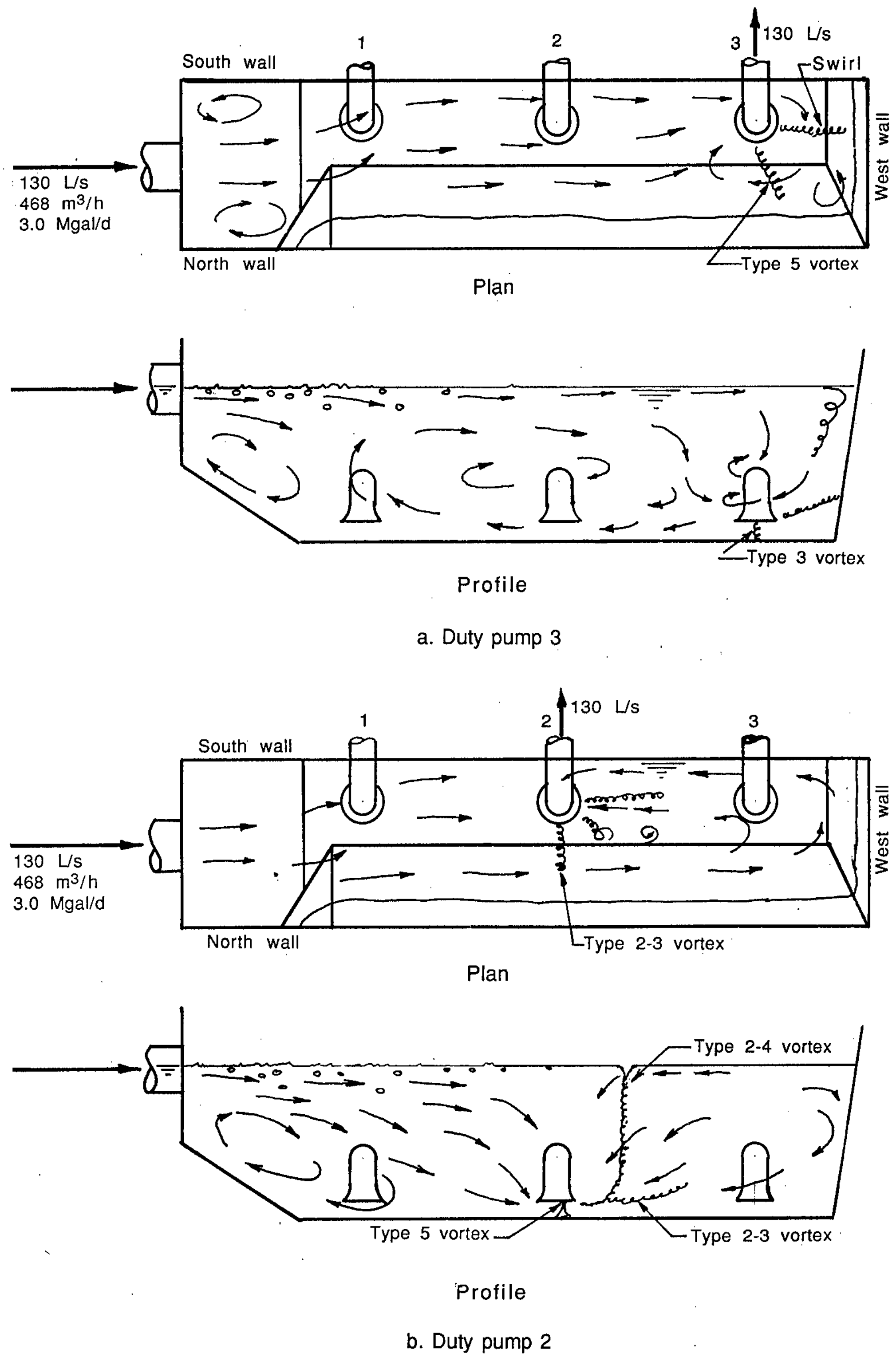


Figure 17. Typical pump sump flow patterns during tests of Kirkland Pumping Station model in V/S (steady state) pumping mode.

pump, although swirling was observed at the water surface in the Northwest corner of the basin. There was some submerged swirling of flow from the West wall (opposite from the inlet) of the basin. Well organized Type 3 submerged vortices intermittently formed and entered Pump 3 intake from the floor directly beneath the intake and from the North wall (which slopes at 45°) directly opposite the intake.

2. Test of Pump 2--Flow followed a pattern similar to Test 1, but when it encountered the West wall, it curved toward the North wall as well as down toward the floor and approached Pump 2 from both directions. Air entrained at the sump entrance was mostly eliminated by midway between Pumps 1 and 2. Eddies in the flow past the intake occasionally accelerated to vortex strength and pulled air into the intake. A constant vapor core Type 5 vortex was pulled from the sump floor and entered the intake. Intermittent submerged vortices of Type 2 to 3 formed from the North wall opposite the intake and entered it, and a similar vortex occasionally formed from mid-flow to the West and entered the intake.

3. Test of Pumps 1 and 2--The flow pattern was erratic and undefined. Air entrained at the entrance entered Pump 1 but exhausted from the water surface prior to entering Pump 2. No organized vortices were observed at Pump 1 with the exception of some intermittent air-entraining surface vortices. A constant vapor core vortex formed from the floor under Pump 2 intake and entered it. Intermittent submerged Type 2 to 3 vortices also formed from the North and South walls and in mid-flow from the West and entered the pump.

4. Test of Pumps 2 and 3--The flow pattern was similar to that for the test of Pump 2. No surface vortices were observed to form. Surface turbulence seemed to dissipate them. No air was entrained into the intakes from the water surface. Vortices formed from the sump floor and entered both of the operating intakes. The strength of these varied from Type 1 to 2 at Pump 2 and 2 to 5 at Pump 3. An intermittent mid-flow vortex formed between Pumps 2 and 3.

5. Conclusions--The currents in the model in variable speed mode were quite similar to those that could be seen and recorded by a video camera in the Kirkland pumping station both during normal operation and during pump-down. In the model tests, air-entraining and vapor-core vortices would lead to the conclusion that the design is only fair. However, the frequency and strength of air entraining vortices may be too low to degrade pump performance significantly. At the water levels and pumping rates seen in the prototype during normal operation, no vortex occurred, and only a small and slow circular motion of surface trash was seen. Factors that would significantly mitigate adverse effects are length of suction piping and the size of pumps. Long suction piping reduces pre-rotation and dissipates vortices. Pumps with impellers closer to the intakes might not fare as well. One could speculate that the performance would be improved if the approach (inlet) pipe were in the same vertical plane as the pump intakes and if there were a cone under Pump 3 intake and a vane under Pump 2 intake. A symmetrical cross-section with both side walls sloping would also seem beneficial. Some means of dissipating the high (~1 m/s = 3 ft/s) entrance velocity might also reduce deleterious currents. Air entrainment can be prevented by raising the water level slightly.

c. Normal Operation. C/S mode

Performance was evaluated with clear water simulating the fill and draw mode of operation that characterizes C/S pump operation. Acquisition of data on currents and pre-rotation was impractical during these "dynamic" experiments.

1. Test of approach pipe--The slope of the influent pipe was set at 6 percent to give the same scaled model velocity that would correspond to the velocity in the prototype pipe partly full at 2 percent slope. The lower end of the pipe was transparent to allow the hydraulic jump and any air entrainment to be observed. The pipe was operated at maximum prototype flow, 232 L/s (868 m3/h or 5.5 Mgal/d) and at other, lesser flow rates in a vain effort to discover whether problems would develop. None did.

Water at super-critical velocity impacting the impounded water in the partly-drowned pipe formed a rather weak jump at about 2 m (6 ft) from the basin. The Froude number was estimated to be 2.8--an oscillating jump according to Chow [7]. The jump entrained a modest amount of air bubbles that rose to the soffit of the pipe to form a series of air pockets. The largest air pocket was about $D_p/4$ deep by 2 D_p long (where D_p is the inside diameter of the pipe), but it persisted only a few seconds. Most air pockets were less than $D_p/8$ deep and D_p long and they migrated downstream. At the entrance of the basin, the air quickly rose to the surface.

Model tests with air are not directly applicable to prototypes unless Froude, Reynolds, and Weber numbers are equal for model and prototype. To achieve the proper model fluid velocities, the approach pipe was inclined at 6 percent, whereas the prototype gradient was 2 percent. Consequently, bubbles that would escape to a free water surface in the prototype were trapped due to the short length of free water surface in the model. The low model fluid velocities were able to drag small air pockets downstream, but in the prototype, the velocities required to do so are very high. Nevertheless, the model results were encouraging, because it was apparent that size of air pockets was self-limiting. Large air pockets tended to move upstream, but they quickly broke up into small ones that were dragged into the basin.

Approach pipes will collect solids while partially drowned, so it will be necessary to flush the pipes frequently. As it is possible that impounded water may persist in the lower part of the pipe for long intervals, it might be necessary to program the controller to pump the basin to LWL often to allow the super-critical velocities to wash solids to the basin. An alternative or supplementary scheme is to install a sluice gate in the upstream manhole for flushing the pipe.

2. Test of Pump 2--Air bubbles created by the hydraulic jump in the approach pipe entered the basin in a steady stream. The jump moved downstream and entered the basin when the water depth in the pipe reached 0.06 D_p. When the water level dropped below the invert by only 0.23 D_p, air entrained by the free fall began entering Pump 3. When the level dropped below the invert by 0.51 D_p, air entrainment into Pump 2 became too great. These results prove that even small free falls (less than 0.3 m or 1 ft) cause air entrainment in pumps and should not be allowed. No organized vortex activity was observed throughout the range of water levels.

3. Test of Pump 1--Little air was entrained by the hydraulic jump when the jump was in the sewer. There was no "burping" of air slugs into the wet well. A smooth transition occurred as the hydraulic jump came down the sewer and entered the wet well as the water level was lowered to about 0.2 D_p below the invert. Air entrained by the free fall from the sewer to the water surface was carried to Pump 1 and began to enter it when the water level reached about 0.5 D_p below the invert. No organized vortex activity was observed throughout the range of water levels.

d. Simulated Clean-Out

Horizontal flow experiments at sub-critical velocities at Montana State University (see Section V.B.b) had established close similarity between the transport rates of a bed of sand at prototype velocity and a bed of carbon at model velocity. The carbon was Calgon GRC-20 6x16 granular activated carbon wet screened to pass the No. 6 sieve and be retained on the No. 16 sieve. So the model was loaded with carbon to the profile of the sludge measured at Kirkland. See Figure 2. Pump 3 was adjusted to represent full speed, and the inflow was adjusted to model a flow rate of 66 L/s (240 m^3/h or 1.5 Mgal/d), the estimated observed flow rate at Kirkland.

In the upstream half of the model, carbon did not behave like prototype sludge. Turbulence due to the waterfall from the inlet suspended the carbon in a roiling motion that swept the upstream half of the wet well clean before the water depth fell to 1.27 D. Where no turbulence existed, as in the downstream half of the wet well, carbon did behave like sludge.

Near the end of the test, the water began to turn opaque due to the abrasion of the carbon in the recirculation pump located at the downstream end of the manifold into which the suction intakes discharged. Only about half of the carbon passing through the pump was reusable. To counteract this problem would require separation of the carbon before pumping the recirculating water. Any separator would have to be both large and capable of sustaining a vacuum--an expensive and somewhat impractical vessel.

Difficulty was experienced in this first carbon test in properly controlling the wet well water level. In the prototype, the water level falls from the pipe invert elevation to the lowest attainable level in about 1.5 minutes. In the model test, however, it took 5 times as long to fall a comparable height because the pump and siphons were partially blocked with carbon. Ordinarily, a second test would have been made with modifications of the apparatus to prevent the blockages. But as carbon did not model sludge in turbulent regions and enough had been learned about the behavior of both sludge and carbon in these and other studies, water velocity alone was sufficient for evaluating clean-out potential.

The carbon, however, had served its purpose. It focused attention on the movement of sludge to a downstream pump intake, it emphasized the requirement for producing unobstructed flow at high (greater than 1.2 m/s or 4 ft/s) velocity along the bottom of the trench during cleaning (a conclusion corroborated by surveys of prototype pumping stations and hydraulic tests of sand in the laboratory), and it produced the information needed for designing the self-cleaning aspects of pump intake basins. To achieve velocities higher than 1.2 m/s would require an ogee entrance to convert potential head to energy head. All subsequent models incorporated this feature.

D. TRAPEZOIDAL SUMPS FOR SUBMERSIBLE PUMPS

Tests on the Kirkland model made it evident that: (a) to reach high velocities along the floor during clean-out, the potential energy of the influent must be preserved as kinetic energy by flowing down an ogee entrance, and (b) symmetry would contribute stability to the currents. Furthermore, submersible pumps are usually C/S units (although V/S can be used), so the wet well must have sufficient volume. To prevent a cascade during normal operation, the approach pipe must discharge at LWL and it may slope upward at a severe gradient to HWL. Storage in the pipe augments storage in the basin.

The addition of these features to the design resulted in greatly improved performance.

a. First Model

The first model tested is shown in Figure 18 wherein dimensions are converted to prototype units in both SI (metric) and U.S. customary (feet and inches). The pumping capacity chosen for the prototype was 75 L/s (270 m^3/h or 1190 gal/min). As the model was built on a linear scale ratio of 1/4.0, the corresponding model flow rate was 2.34 L/s (0.083 ft^3/s).

The large triangular flow splitters (horizontal fillets) between pumps were added to induce uniform flow down both sides of the trench. The top of the ogee was a wide gently-curved surface. At pump-down, the influent spread laterally over the entire surface, thereby causing concern that solids would be deposited and remain there because of low currents in that region at all water levels.

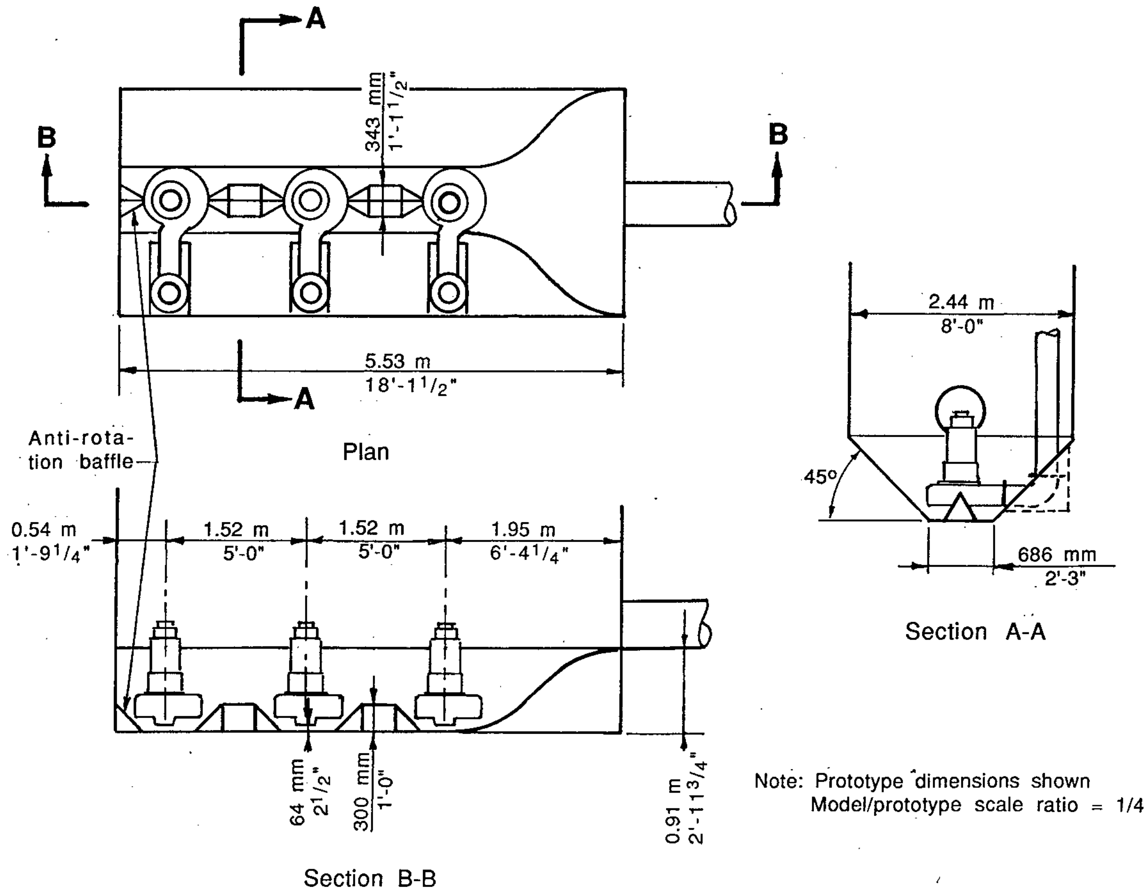


Figure 18. Self-cleaning pump sump with triangular flow splitters for C/S submersible pumps.

1. Single pump tests--In general, operation of single pumps at the calculated station low water level (LWL) demonstrated good approach flow conditions to the pumps with little adverse phenomena other than anticipated floor vortices. All tests were performed at a prototype intake to floor clearance of 100 mm (4 in) or D/2. Note that all following dimensions are converted to prototype values. Floor vortices would intermittently coalesce into well defined dye cores but free from vapor or debris entrainment.

During operation of Pump 1, the hydraulic jump characteristics of the influent sewer jet were observed. At a water level of 125 mm (5 in) above the invert, the velocities measured were: 2.5 m/s (8.2 ft/s) at the exit of the influent pipe, 1.6 m/s (5.2 ft/s) at the surface immediately upstream from Pump 1, 0.7 m/s (2.2 ft/s) near mid depth upstream from Pump 1, and 0.1 m/s (0.4 ft/s) near the upstream base of Pump 1. Very little surface current was evident between Pumps 2 and 3. At a water level of 29 mm (1.15 in) above the invert, however, the current descended the ogee and the velocity immediately upstream from Pump 1 was 0.15 m/s (0.5 ft/s) at the surface, 0.5 m/s (1.5 ft/s) at mid depth, and 1.2 m/s (4 ft/s) near the floor. Clearly, the hydraulic jump should be confined within the influent pipe by drowning the exit. At water levels less than the minimum recommended level of 125 mm (5 in) above the influent sewer invert, a hydraulic jump was present immediately upstream of the Pump 1 casing which resulted in upwelling against the motor and impact velocities of 1.5 m/s (5.2 ft/s). Such velocities might be too great and if so, baffles may be required. A simple beam spanning the width of the wet well would probably suffice. At water levels higher than 125 mm (5 in) above the invert, the jump moved upstream into the sewer and the upwelling was eliminated. Consequently, LWL for normal operation was established at 150 mm (6 in) above the sewer invert.

Testing of Pump No. 2 revealed a potential weakness in the existing horizontal flow splitter design, which was probably responsible for the occasional high (7.2^{o}) pre-rotation measured during this test. The breaks in the flow splitters between pumps permitted flow expansion to exist immediately below the pump intake--an unstable condition. This condition occasionally produced an unwanted circulation, encouraged by the guide rails and discharge piping situated along one side of the wet well. As Pump 2 was positioned at the center of the wet well, the approach flows consisted of a combination of floor level currents from the Pump 3 side and surface currents from the sewer. These currents sheer at the body of Pump 2 and cause some significant eddies, although they were not observed to enter the pump. Performance otherwise was acceptable for this condition.

Operation of Pump No. 3 produced good surface and subsurface currents approaching the pump. Eddies shed from the bodies of Pumps 1 and 2 and particularly from the pump guide rails were subdued and did not produce vortices entering the pump. Pre-rotation at the pump impeller was low and attributable to the symmetrical approach currents entering the pump. The end wall fillet was particularly beneficial for good approach flow, since it redirected flow into the intake and thus prevented the development of a broad circulation current. At normal LWL, a surface recirculation occurred between Pump 1 and the influent sewer exit, but all other surface flow approached Pump 3 uniformly.

2. Multiple pump tests--During tests of two duty pumps, the water level was held at 500 mm (20 in) above the sewer invert. All three combination tests demonstrated similar flow patterns and were typically characterized by good performance. The floor vortices previously mentioned were still present and the eddies from guide rails and discharge piping persisted. Movement past the pump casings was uniform and did not demonstrate any potential problems. Pre-rotation was low and surging of flow entering the pumps was minimal.

The most prevalent concern identified during combination tests was due to low velocity zones in the upstream corners of the wet wells where grit and solids would probably accumulate. Velocities in these areas were typically lower than 60 mm/s (0.2 ft/s) which also presents an opportunity for vortices to form due to shear from the adjacent influent jet.

3. Clean-out operation--Clean-out operation was performed with Pumps 2 and 3 operating because Pump 1 was not anticipated to contribute to the removal of grit due to air-binding. When water levels approached scour depths, the flow splitter between Pumps 1 and 2 and the pump discharge nozzle and piping presented significant disruptions in the flow. These obstacles caused hydraulic jumps and energy dissipation which slowed the flow significantly. The channel velocities downstream of Pump 1 along the discharge piping side of the flow splitter were typically lower than 0.6 m/s (2 ft/s) and reverse currents were observed.

4. Critique--Tests of this configuration revealed a few weak points. The flow splitters positioned between pumps should be removed or modified. The presence of discharge piping

elements in the channel continued to block the flow partially and cause upwelling. It was thought that tapered wing walls would eliminate the upstream corners where grit could accumulate and vortices could form, and that these wing walls could improve clean-out by removing areas of the sump which cannot be scoured by the influent jet.

Some device, such as a horizontal beam across the width of the sump, should be added to break up the influent jet at lower water levels. Otherwise, excessive loads may occur on the motor casing of Pump 1. The beam should be positioned such that the lower edge is coincident with the sewer invert with the upper edge reaching almost to the midpoint of the sewer.

b. Second Model

Tapered wing walls were installed to confine the flow and prevent deposition and eddies. A horizontal beam (velocity breaker) was added to break up the jet from the influent pipe. The beam was reasonably effective, but it would undoubtedly collect stringy material and be difficult to clean. The massive triangular flow splitters of Figure 18 were replaced by a full-length steel plate that, as shown in Figure 19, fit closely around the pump volutes. Unfortunately, this flow splitter would, no doubt, also collect stringy material.

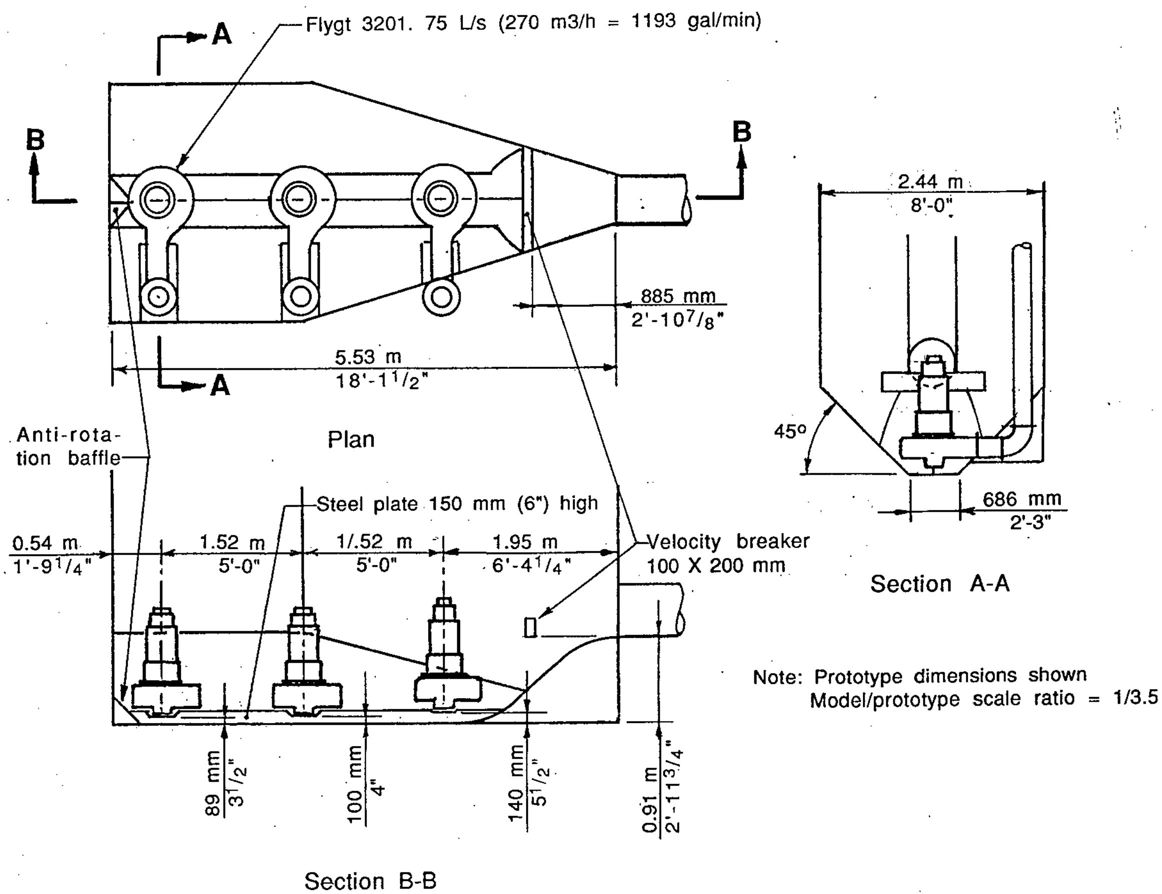


Figure 19. Plate-type flow splitters in submersible pump intake basin.

This design distinctly improved normal operation, but clean-out was still poor. Clean-out tests were performed for this configuration using fine sand in the model instead of the granular carbon previously used. The sand provided a more realistic representation of the movement of grit under the turbulent scouring action of the hydraulic jump. The sand also provided a more conservative estimate of the potential for clean-out in the prototype. After a stable water level was reached during pump-down and the hydraulic jump formed at the base of the ogee, it took from a half-minute to 1-1/2 minutes to remove sand between the ogee and Pump 2. But to scour sand from the middle of the floor between Pumps 2 and 3 required another 3-1/2 minutes, and to remove all but a thin strip of sand along the edges of the floor required still another 2 minutes. During the next 2-1/2 minutes, Pump 3 continued to operate without air binding, although considerable air entered the "pump." The flow down both sides was uniform, so the thin flow splitter was effective and did not cause unstable flow as the triangular flow splitter did.

c. Third Model

The third model was similar to the second except that the floor was "excavated" to form a narrow trench as shown in Figure 20. Suction bells projecting into the trench were added to the pumps. (For some prototype pumps, the suction bell could be a standard flanged flare.) The velocity breaker was fabricated from pipe.

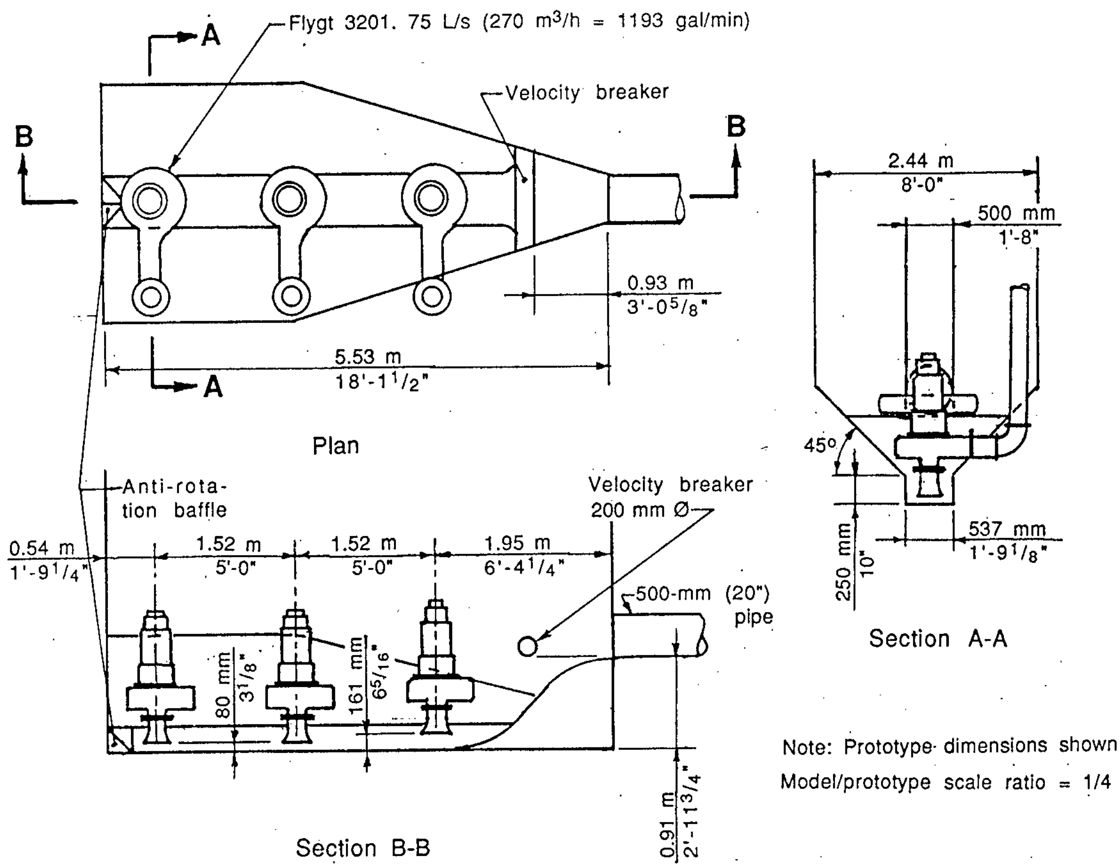


Figure 20. Trench-type sump for submersible pumps.

1. Normal operation and clean-out--At normal operation, this model performed as well as the second model and was greatly superior to either of the other models for clean-out operations. After pump-down was reached, the sand between the ogee and Pump 2 was scoured

out in 3/4 minutes. Only another 1/4 minute was required to clean the floor to Pump 3. From start to finish, all sand was removed in 68 seconds, a great improvement over the performance of the second model. Some of the improvement may have been due to better operational procedure, but the trench was obviously the main source.

2. Mixers--One way to clean a sump is to mix the contents thoroughly for a minute before starting a pump and to continue mixing for two to five minutes depending on the size and configuration of the basin. The basin can thus be kept continuously cleaned. The concept of a submersible motor driving a propeller was invented by ITT Flygt Corporation and mixers are now made by several manufacturers.

A typical mixer for a pump sump containing three pumps with a firm pumping capacity of 150 L/s (540 m^3/h or 3.4 Mgal/d) would consist of a shroud about 0.7 m long by 0.35 m (28 in x 14 in) in diameter enclosing a 1 to 2.2 kW (1.3 to 3 hp) submersible motor driving a 225 mm (9-in) propeller. The entire device would weigh about 50 kg (110 lb) and can easily be placed and oriented in any position.

A demonstration mixer 300 prototype mm (12 in) long with a 3-bladed propeller 200 mm (8 in) in diameter driven by an encapsulated d.c. motor was supplied by ITT Flygt and installed at mid depth between Pumps 1 and 2, oriented at 45^{o} downward toward Pump 2 intake. The current generated upwelled between Pump 3 and the end wall. When the pumps were started, the sump was cleaned while the water level was kept at 500 mm (20 in) above the invert. The cleaning was complete.

Mixers have much to recommend them. They are effective, small, easily placed and oriented as desired, and use little power. On the other hand, they add machinery and maintenance. It seems preferable to provide for cleaning by geometry where possible (as in new pump intake basins) and to use mixers for retrofitting existing sumps.

3. Critique--Although it may be possible to have all intakes at the same critical elevation at some particular installation, there is no margin for error and no assurance that "rooster tails" or standing waves cannot interfere with the super-critical flow. Therefore, it is better to have all upstream pump intakes well above the critical depth or even above the jump and to have the last intake close to the floor. A good way to meet this recommendation is to set all pumps at the same elevation but lengthen the suction bell for the last pump.

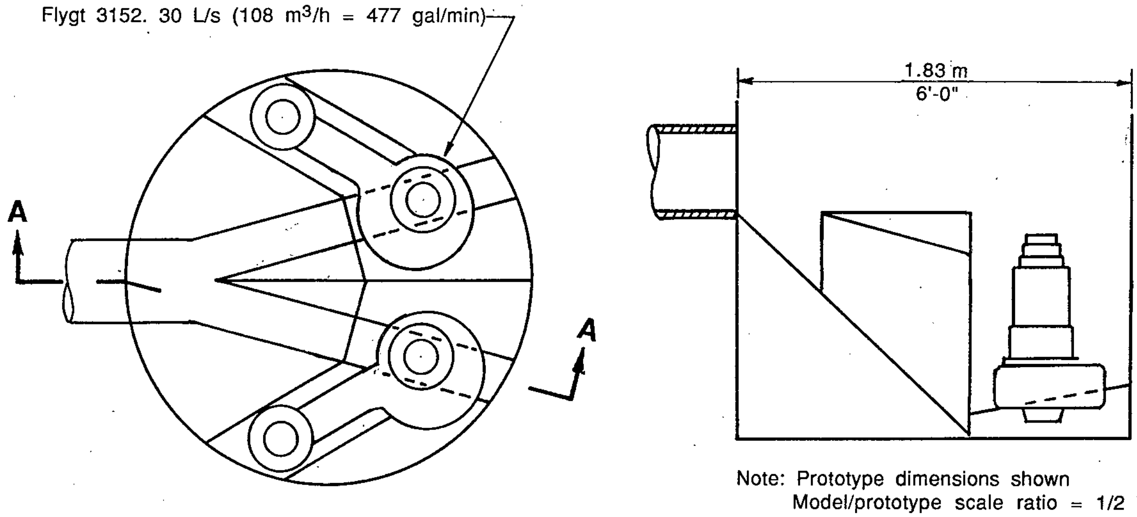
The anti-rotation baffle at the end pump might better be attached to the suction nozzle and designed for minimum clearance (say, 25 mm or less) at floor and end wall. Stringy material caught in this space would not interfere with the pump.

Floor vanes under upstream pump intakes and a 90^{o} cone under the last pump intake would reduce the pre-rotation. Floor vanes and cones must be installed with accuracy.

E. ROUND SUMPS FOR SUBMERSIBLE PUMPS

Limited tests were made on the round sump shown in Figure 21. Observations with dye showed excellent flow patterns both for clean-out and for normal operation. The sharp nose of the flow splitter downstream of the end of the influent pipe was considered unacceptable because it would quickly be fouled with stringy material.

The design was abandoned because the investigators thought that consulting engineers would not accept a design so difficult to form. The success of the Vallby and Clyde stations rendered model tests of small, round, duplex pump sumps unnecessary.



Plan

Section A-A

Figure 21. A round self-cleaning pump sump at ENSR.

SECTION V

MODEL STUDIES AT MSU

Models of rectangular pump sump trenches and pump intakes were studied in the Department of Civil Engineering hydraulic laboratory at Montana State University by Sanks, usually working alone but sometimes assisted by other university personnel. Of the research completed, enough is presented herein to support the conclusions and recommendations of Section II.

A. FACILITIES

Although the small size of the facilities would not permit the use of a complete pump sump model, they were adequate for studies of the trench, pump-down, and cleaning. Flexibility was a distinct advantage denied in the large ENSR or Fairbanks Morse models except at great expense.

a. Flume

The basic container was the flume depicted in Figure 22. The addition of plywood sides at the headworks allowed a model of the Kirkland sump (minus the sloping side and widened upper section) to be built at a linear scale ratio of 1/3.63. The supply pipe valve was fitted with a large quadrant to enable quick adjustment of inflow. Baffles for regulating fluid depth or a plate containing a vee-notch weir could be inserted into slots at the downstream end. The slope of the flume could be set anywhere between zero and three percent. Fluid depth was measured with a movable point gauge.

Inserts to represent end walls, influent pipes, ogee ramps, and the like could easily be clamped in place. The sand trap underneath could entrap all the solids removed at pump-down for reuse. The water supplied in these experiments ranged from 40 to 780 L/s (144 to 2800 m^3/h or 0.9 to 17.8 Mgal/d) converted to prototype values.

b. "Pumps" or Siphons

Prototype pumps were represented by siphons. The principal siphon is shown in Figure 22. The suction bell, made by ENSR, is transparent acrylic and fitted with a rotor for measuring pre-rotation or swirl. A long pointer and a large quadrant allowed the ball valve to be set to deliver any prototype flow rate between 35 and 200 L/s (130 to 720 m^3/h or 560 to 3200 gal/min) ±5 L/s. A somewhat more accurate setting could be obtained with patience and the use of the vee-notch weir and venturi meter. Near the end of the project, a pitot tube was set into the siphon and connected to an air-water manometer that was inclined at 45° for greater sensitivity and convenience.

The siphons were set in saddles that could be positioned anywhere along the flume, and they were held in place by bungee cords. Exact location was obtainable by rotating the siphon about its vertical axis so that the horizontal leg (made extra long) swept over an arc that permitted the bell to be centered or even to touch either side of the flume. Floor clearances were accurately set by means of wooden "feeler" blocks.

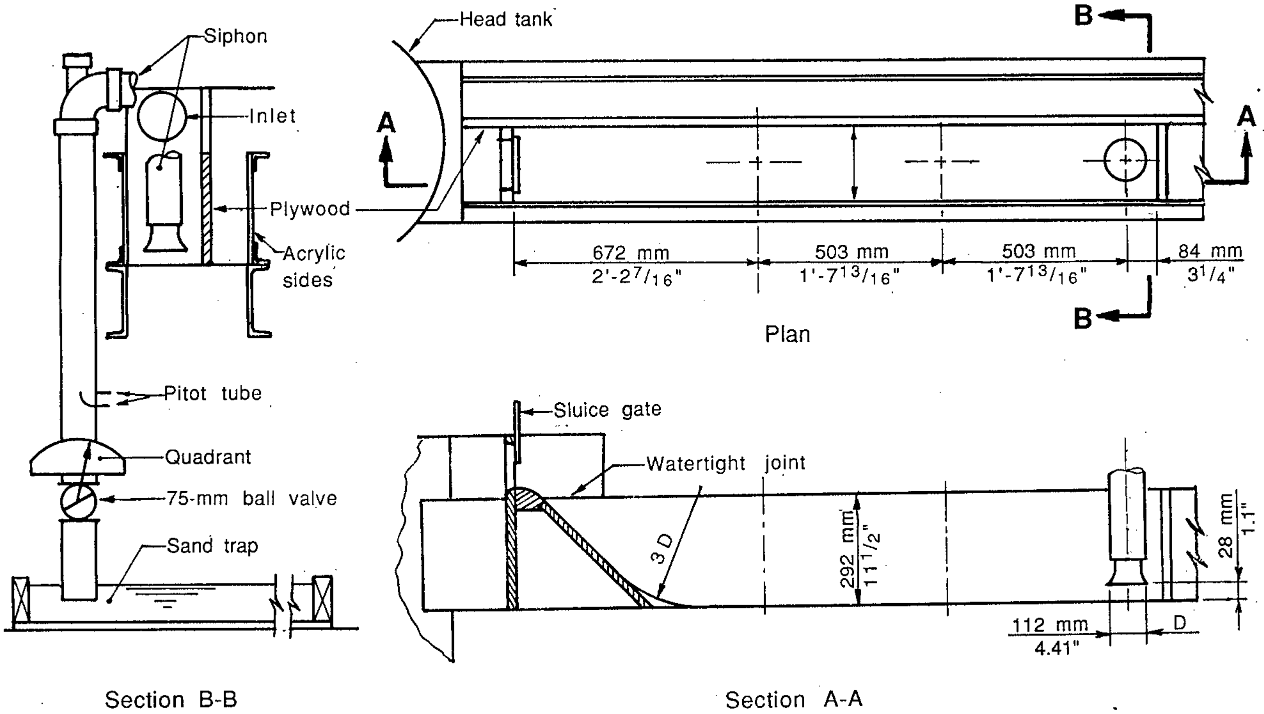


Figure 22. Model of trench of improved Kirkland pump intake basin.

c. Critical Measurements

Critical measurements are similar in most respects to those for research at ENSR described in Section III.B.a, namely:

1. Pump performance.
   - Pre-rotation. Measured by a neutrally-pitched rotor in the suction bell over a time interval of 5 minutes or more.
   - Output flow rate. Measured by the difference between the venturi meter and the vee-notch weir flows for a single pump or by the pitot tube for multiple pumps and sometimes for a single pump.
   - Vortex formation. Visual observation of dye.
   - Bubbles entering pump. Visual.
2. Movement of solid deposits.
   - Elapsed time for front face to move a measured distance or for all but a few particles to be ejected.
3. Pump sump performance.
   - Vortices.
   - Stable hydraulic conditions as delineated by dye.
   - Velocity past pump intakes. Dye and calculations.
   - Scouring time to eject deposits.
   - Hydraulic jump. Visual observation and speed of transit.

Criteria for accuracy and precision are given in Table 3.

TABLE 3. CRITICAL MEASUREMENTS AT MSU

Observation	Device	Allowable error	Precision error	Accuracy error
Total flow	Venturi	5%	0.2 - 1%	1 - 2%
Overflow	Vee-notch & point gauge	3%[a,b]	0.1 - 1%[a]	0.5 - 2%
Pump	Venturi & vee-notch	6%	0.5 - 2%	1- 4%
Pump	Pitot	6%	1/2 - 1%	5 - 6%
Pre-rotation	Rotor	1°	0.1°	0.5°
Currents	Dye & stopwatch	20%	15%	20%
Vortices	Visual	Classification: see Figure 6		
Currents	v = Q/A	10%	2 - 3%	5 - 10%
Scour	Stopwatch	20%	1%	5 - 20%

[a]Error is expressed as percentage of normal pump flow, not weir flow.
[b]Except for intake proximity, floor clearance, and current past intakes. (See text.)

A regression analysis of the gravimetric calibration of the venturi meter yielded an R^2 of 0.99991, and R^2 was 0.99998 for a later volumetric calibration. For the vee-notch weir, R^2 was 0.9988. Flow rate for a single pump in steady-state flow was obtained by setting the inflow from 2 to 10 percent higher than the pump rate and measuring the excess with the vee-notch weir. So the error in weir discharge is best expressed as a percentage of pump flow.

High accuracy is not needed for observing currents, cleaning capability, and other facets of performance. But to determine the effects of proximity of intakes, floor clearance, cones versus flat floors, only tiny differences in flow rates are expected, and very precise measurements are desirable. As the most appropriate instrumentation was unavailable, great pains were taken to get the best from the facilities on hand and to plot curves of trends as the variables were manipulated over a wide range.

B. SCOUR OF DEPOSITS

At the beginning of the project, it was thought necessary to use a fluid mix comparable (in physical characteristics) at model velocities to raw wastewater (which contains scum, stringy material, organics, and grit) at prototype velocities. Each individual component was indeed successfully modeled with a particulate substance that could be screened out to avoid contaminating the laboratory sump. Short strings were adequate for stringy material, floating plastic beads represented scum, saturated sawdust was scoured and transported at model velocities just as organics at prototype velocities, and granular activated carbon was a good substitute for grit. But when the components were mixed, the behavior was abnormal. Mixed deposits were washed away too quickly, so wastewater could not be modeled with a mix of different kinds of particles.

During the visits to the pumping stations described in Section III, scum was found to be easily removed when the area occupied by the scum was sufficiently confined near a pump intake. The narrow trench confined the width, and the currents pushed floating material toward the back wall, so during pump-down the area occupied by scum was small indeed. When the submergence of the intake fell to about 0.8 D, the Type 5 vortices formed quickly sucked the scum into the pump. So scum was no problem and neither was stringy material (rags and paper). The real problem was moving bottom deposits of sludge (organic material and grit) to the pump inlets for discharge into the force main. Sludge could not be entirely removed from the

Kirkland or Steilacoom pumping stations even with three successive "cleanings". So what was really needed was a material that would, in successive pump-downs, result in deposits like those in Figure 2. Thus, while there was no need for a model sewage, there was a need for a model sludge.

From model tests both at ENSR and MSU, granular activated carbon was suspended too easily by turbulent currents to represent sludge. Sand, however, was excellent and in model tests with sand, the profiles of deposits after a cleaning cycle were satisfyingly similar to those shown for the Kirkland station in Figure 2. Consequently, a single-component substance, sand, was successfully used to represent sludge. Absolute similarity between model and prototype cleansing was not important. It was only necessary to compare the original Kirkland model with improved successors to choose the best design. The final design was more than an order of magnitude better than the Kirkland model.

a. Grit Movement at Prototype Fluid Velocities

The plywood wall in the flume (Figure 22) was moved to make a channel 100 mm (4 in) wide. Tests with sand at two depths, 50 and 100 mm (1 and 2 in), were made for deposits 1 to 2.4 m (3 to 8 ft) long at prototype fluid velocities ranging from 0.6 to 1.4 m/s (2 to 4.6 ft/s). When a flow of water was deflected into the channel, the sand front became thinner, then began to move slowly at first, then faster. The average rates of movement are shown in Figure 23. The size of particles had little effect on results. Large (4.75 mm) particles moved as readily as small (0.59 mm) ones.

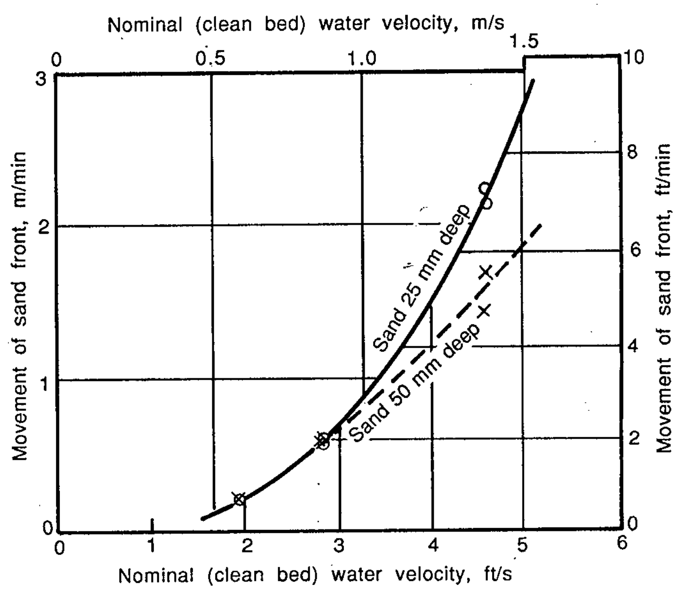


Figure 23. Average rate of sand movement as a function of fluid velocity.

b. Scour in the Kirkland model

To furnish a standard for comparison, the Kirkland pump intake basin was modeled as faithfully as possible. The unique shape at the entrance (see Figure 1) could not be modeled, but the influent pipe was set to discharge water in a free cascade to splash off center on the flat floor just as it does at Kirkland.

A uniform layer of sand 50 mm (2 in) deep in prototype units completely covered the flat floor. The basin was filled with water carefully so as not to disturb the sand. The influent flow was set at 66 L/s (237 m^3/h or 1.5 Mgal/d), and Pump 3 was set at twice that amount, so of course, the water level was rapidly lowered to its minimum depth--0.7 D. Because of the formation of a Type 5 vortex beside the pump intake, the pump became self-regulating to match the influent and the water level could not be lowered further.

Although the trench was cleaned in the vicinity of the cascade and directly under the suction bell, only 8 percent of the sand was ejected. The nominal velocity of the water over the sand bed was approximately 0.37 m/s (1.2 ft/s). Such a low velocity is, according to Figure 23, below the threshold for transporting sand.

A second test was made with the influent flow rate increased to 132 L/s (474 m^3/h or 3 Mgal/d) to see how thoroughly the basin could be cleaned under the best operating conditions. (See Section II.C.d). Under these conditions, most of the sand was ejected in 2 minutes, but some was left, notably a bank of sand in a stagnant area upstream from Pump 3. As shown in Figure 24, counterclockwise currents between Pump 3 and the end wall caused the stagnation. After another 22 minutes, all but a few grains of sand were ejected.

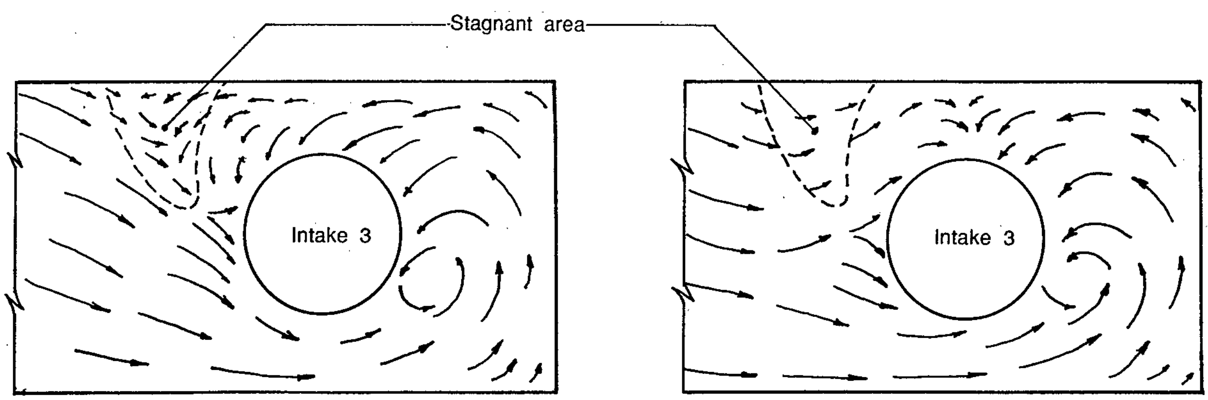


Figure 24. Flow patterns around Intake 3 in replica of the original Kirkland Pumping Station at pump-down.

During normal operation, at a pump intake submergence of 2 D, vortices of Types 3 to 4 formed near the end wall. But if the wall was tilted to the vertical, vortexing was less severe and the severity continued to decrease as the wall was moved closer to the suction bell.

c. Scour in the improved Kirkland model

It was clear that, by allowing the influent to flow down an ogee ramp to convert the potential energy of the water to kinetic energy and thereby obtain high velocity, the effectiveness of cleaning could be greatly improved. The ramp is shown in Figure 22. Note that the vertical end wall is only D/4 from the edge of the suction bell, and the pump intake is lowered to D/4.

At pump-down, a hydraulic jump formed at the toe of the ogee, but it progressed only halfway to the end of the channel before becoming asymmetrical due to circulation behind the pump intake as shown in Figure 24. Large, triangular flow splitters like those in Figure 18 were installed between pump intakes. They were not effective, nor was the effectiveness improved by installing the triangular flow splitters up the ramp. Substituting high (0.7 D) thin plate flow splitters was no improvement either. The anti-rotation baffle shown in Figure 25 between the pump intake and the rear wall was completely effective, however, so all subsequent tests were made with the baffle and with the cone and vane under the bell in place. Some swirling occurred without the vane but virtually none with it. Neither cone nor vane was installed in the Fairbanks Morse pump sump where the anti-rotation baffle was sufficient by itself to produce symmetrical approach flow. Swirling in the Fairbanks Morse pump sump could not, however, be observed because no rotor meter was installed.

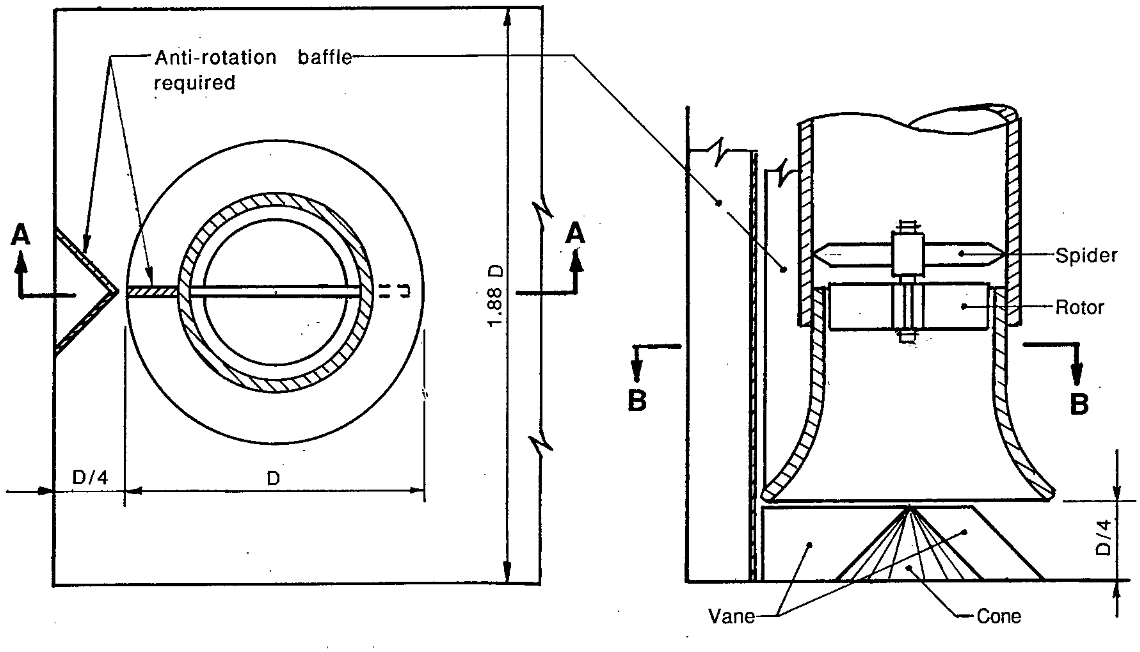


Figure 25. Details of Intake 3 of improved Kirkland Pumping Station.

Pump-down with the same sand bed described in Section V.B.b (50 prototype mm deep) and with an influent flow rate of 85 percent (112 L/s, 403 m^3/h, or 1800 gal/min) of Pump 3's capacity was superb. Within 25 seconds from the formation of the hydraulic jump at the toe of the ogee, the jump had progressed downstream to just in front of Pump 3 and all sand was ejected. Note that in tests of the Fairbanks Morse prototype, sand was ejected in 23 seconds. The surface of the water at super-critical velocity was well below the upstream intakes, so, of course, upstream siphons lost prime. The downstream siphon did not. If removal of the last bit of sand is the criterion, the improved model was nearly 60 times as effective as the Kirkland model!

If the siphon valve setting was unchanged, the maximum capacity at pump-down was no more than 85 percent of capacity at normal depth because of the reduced capacity due to ingested air. The effect of air on prototype pumps might be less or greater. But the reserve of scouring capability with this design is so great that the reduction in flow rate is not significant.

Note that there is no difference in either cleaning procedure or effectiveness between basins for V/S and C/S pumps.

C. OTHER OBJECTIVES

With so much effectiveness having been developed for cleaning, the rest of the research was devoted to various factors affecting hydraulic performance during normal operation, such as: allowable currents past pump intakes, proximity of other pump intakes, optimum floor and end wall clearances, fillets at end wall, effectiveness of straightening vanes in floor, bell, and walls as well as the general performance of currents.

a. Floor Currents

The effect of currents past a pump intake is an important factor in design and, indeed, in choosing whether to use pumps in tandem at all. A number of experiments were made at prototype pump intake velocities of 1 and 1.5 m/s (3 and 5 ft/s) with currents of 0 to 2.7 m/s (0 to 9 ft/s) past the intakes. Performance was evaluated on the basis of pump (siphon) discharge aided by observation of dye patterns. But neither discharge nor dye is adequate to measure performance for column pumps with impellers adjacent to the mouth of the suction bells. Velocity distribution and fluctuations in the throat of the bell are required for an adequate assessment of the effect of approach velocities on column pumps. Pump discharge is, however, adequate for assessing the performance of a dry pit or self-priming pump because impellers are far from intakes and are therefore more affected by piping configuration than by moderate irregularities in the intake.

Currents past the pump were based on inflow to the flume minus the pump discharge divided by the wetted cross-section of the flume. Plug flow (true velocity everywhere equals average velocity) was obtained by installing a fine screen upstream from the pump so as to create a slight head loss and produce uniform velocity. Visual observation of dye proved currents were indeed uniform. Pump discharge was measured with the pitot tube. Water level was controlled with baffles at the downstream end of the flume. Three successive measurements per determination with a maximum reading deviation of 1 mm were required.

A straight line fit of all results showed discharge decreased only 3.1 ± 1.5 percent per m/s of current (0.95 ± 0.45 percent per ft/s of current). As currents at pump intakes in trenches are very low (see Sections III.F.g and IV.C.b), their effect on pump discharge is negligible.

b. Floor Clearance for Pump Intakes

Appropriate floor clearance is controversial. The cylindrical area of the waterway under the bell rim is πDZ, where Z is the floor clearance. The area enclosed by the rim is $\pi D^2/4$, so the two areas are equal when the floor clearance is D/4. Of course, the nominal velocity under the rim equals that across the bell mouth, so D/4 is the minimum that can prevent turbulence due to expanding flow. Dicmas [4], acknowledging that D/2 is a generally accepted standard value, shows that relative to standard pump performance, the head loss at D/4 is 0.5 percent, zero at D/3, a gain of 0.4 percent at 0.4 D or 0.5 D, and zero again at 1.0 D.

Pump-down tests with Pump 3 intake D/4, D/3, and D/2 from the floor were made to find the effect of floor clearance on the hydraulic jump. From the results in Table 4, the intake must be no more than D/4 above the floor. If D/4 is less than 75 mm (3 in) a pit under the intake is required to pass large solids.

TABLE 4. BELL CLEARANCE VS. FLOW RATE FOR AN ADEQUATE HYDRAULIC JUMP

Floor clearance of intake	Required flow rate, % of pump capacity	Notes
D/4	50	Safe
D/3	73	Low safety factor
D/2	98	Unsafe

As a check on the work reported by Dicmas, the effect of floor clearance on the discharge capacity of pumps was tested. Again, differences were expected to be small, so the work was done as carefully as described for the previous subsection. The pump was first tested for a floor clearance of D/2, then (without stopping, touching any valve, or otherwise affecting the pump discharge rate), the pump was lowered to D/3, tested, then lowered to D/4. The results are given in Table 5

TABLE 5. PUMP CAPACITY VS. INTAKE FLOOR CLEARANCE

Floor clearance	Pump intake velocity		
	1 m/s 3.3 ft/s	1.5 m/s 5 ft/s	2.1 m/s 7 ft/s
D/2	100%	100%	100%
D/3	98.4%	100.4%	100%
D/4	98.4%	100.7%	99.4%
D/4 with cone	100%	--	--

The apparently anomalous results were rerun with the same findings. Note that the precision of manometer readings could result in an error of 0.7 percent.

At the same time, average and maximum angles of swirl or rotation were obtained without vanes either in the bell or on the floor. The average angle of swirl (using number of revolutions per 3 minute interval) varied from 0.4^{o} to 2.0^{o}. The maximum angle (using number of revolutions during the period while rotation occurred) varied from 2.5^{o} to 3.9^{o} except at 2.1 m/s intake velocity where it varied from 3.6^{o} to 5.3^{o}.

In conclusion, there is no significant disadvantage in setting bells at D/4 from the floor, especially if a cone is installed. On the other hand, a floor clearance near to D/4 for the last pump is prerequisite for cleaning.

c. Proximity of Intakes

A series of experiments was made to resolve the uncertainty about the required or optimum spacing of pump intakes. Again, the only measured parameter of performance was flow rate, an insensitive indication of performance for column pumps as described in Section V.C.a. Flow from a middle pump in a group of three was monitored as the outer pumps were moved closer. With no other siphons yet placed in the basin, the main siphon was set to discharge 131 L/s (472 m^3/h or 2100 gal/min) in prototype units as a base of 100 percent. Thereafter, nothing about that siphon was changed. The other two outside pumps were then added, each operated with its valve wide open and discharging 153 L/s (551 m^3/h or 2430 gal/min). Once more, variations in flow rate were expected to be small and the work was done as carefully as that for Section V.C.a.

The downstream siphon created a current past the main pump that reduced the capacity of the middle siphon to 98.6 percent. From Table 6, there is no penalty due to proximity until the clearance between suction bells is less than D/2.

TABLE 6. PUMP CAPACITY AS A FUNCTION OF PROXIMITY

Description	Pump spacing, c. - c.				
	4.5 D	3 D	2 D	1.5 D	1.0 D Bells touch
Capacity, % of an isolated pump.	98.6	98.9	98.4	98.4	97.4
Capacity, % due only to velocity past intake.	98.6	98.6	98.6	98.6	98.6
Net effect of proximity, %.	0	0	0	0	1.2

When the two outside pumps were turned off, they had no effect whatever on the middle pump even when the bells were touching.

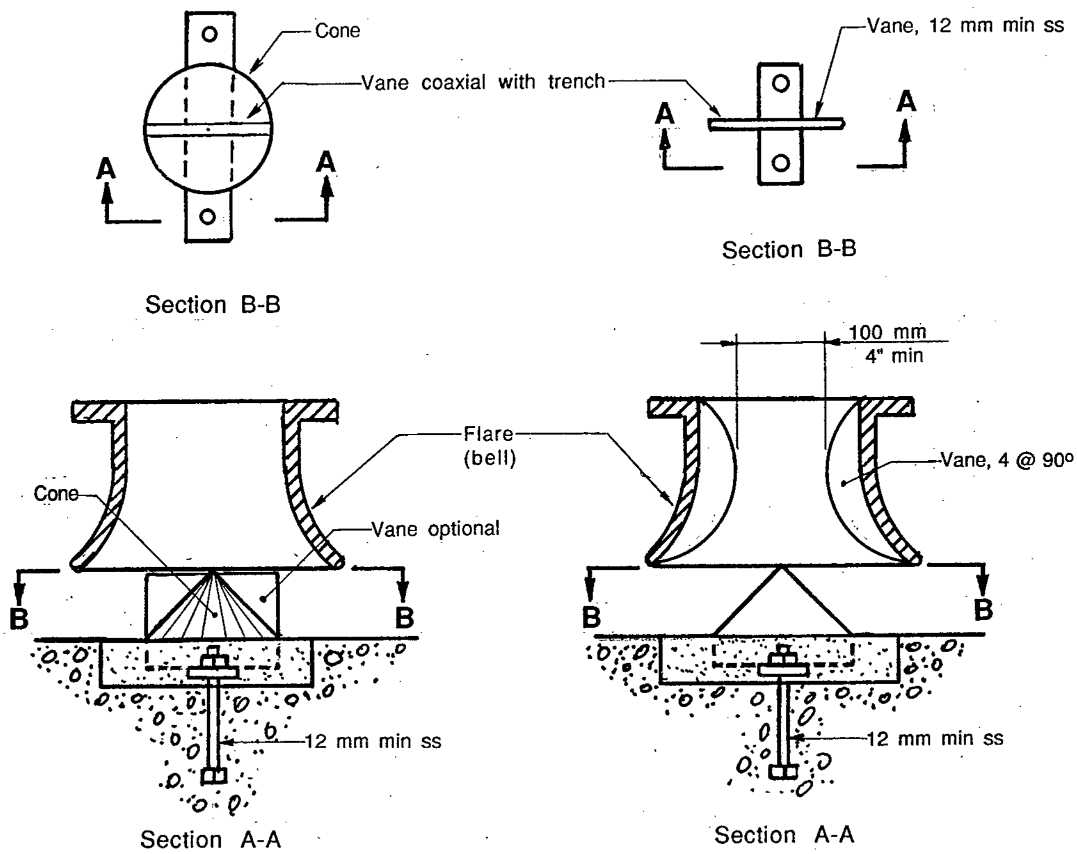
d. Pre-rotation

Pre-rotation or swirling changes the angle of attack on the leading edge of the impeller and thereby reduces efficiency. Swirling is measured in terms of the angular deviation from axial flow at the boundary of the pipe by Equation 9. The maximum permissible angle is considered to be 5°. Swirling is not affected by vortices, because they can pass between the blades of a stationary rotor. Swirling is diminished by well-defined, uniform currents past an intake. It can be increased by moving the suction bell off the center line of the trench, and the direction of rotation can be reversed by moving the bell from side to side. Swirling increases with intake velocity and becomes severe at velocities exceeding 1.5 m/s (5 ft/s). When swirling does occur, the rotor (Section IV.B.a) stops, starts, and spins slowly or rapidly, sometimes too rapidly for counting revolutions except by observing a video at slow speed. Because water levels change, pumps go on and off, suction bells might be slightly off-center, and floor currents can be low and unstable, the variety of conditions at intakes is endless and predictions of swirling and its intensity are chancy. Consequently, several means to reduce or eliminate pre-rotation were investigated.

e. Cones and Vanes

Cones under suction bells are unexcelled for eliminating floor vortices. They create smooth, stable streamlines, and decrease head loss. Two types were tested: one a 90° cone with a sharp apex in the plane of the suction bell rim whereas the other included the attached vane shown in Figure 26 a. Although cones are highly recommended, particularly for floor clearances of D/2 or less, they can not be used under upstream intakes in sumps designed for cleaning because of interference with water flow at super-critical velocities.

In one test, a simple cone reduced swirl from 10° (with no other anti-rotation device) to about 1°--a 90 percent reduction. Adding a small vane (Figure 26 a) coaxially with the trench was nearly as effective as adding a much larger vane (not shown) and reduced the occurrence of swirling in another test from 41 percent of the time for the cone only to 14 percent of the time for the cone with the small vane. The large vane shown in Figure 25 essentially eliminated all swirling at the last intake. The vane of Figure 26 a would be unsuitable for wastewater because stringy material would collect on the leading edge. Stringy material would not collect on the leading edge of the vane in Figure 25, because the edge is almost parallel with the streamlines.



a. Cone and floor vane for clean water   b. Floor and bell vanes for raw sewage

Figure 26. Anti-swirl devices.

Thin (e.g., 12 mm or 1/2 in) floor vanes (Figure 26 b) oriented coaxially with the trench do not interfere with flow at super-critical velocity, are effective, and are therefore recommended at upstream suction intakes. For example, the swirling in a suction bell D/4 above the floor and with an intake prototype velocity of 1 m/s (3.3 ft/s) is shown in Table 7. From these results, vanes reduce swirling by about 60 percent with or without floor currents.

TABLE 7. EFFECT OF VANES AND FLOOR CURRENTS ON SWIRLING

	No floor current		Floor current = 0.48 m/s (1.6 ft/s)	
Conditions	No vane	Vane present	No vane	Vane present
Angle of swirl	9°	3.3°	1.7°	0.7°
Swirl reduction due vane		63 %		59 %
Swirl reduction due current			81%	79%
Swirl reduction due vane & current				92%

Vanes also reduce floor vortices although they do not eliminate them as do cones. Vanes must be designed to pass rags and other stringy materials. Edges sloping at 45^{o} are almost parallel with flow when the pump is operating, and if the edges are smooth and rounded, the intake current will wash stringy material away.

Because floor vanes do not entirely eliminate swirling, the bell vanes of Figure 26 b were tried. The combination of floor vanes and bell vanes virtually eliminated pre-rotation, both in frequency and angle of swirl during the rare times when the rotor was turning. Of course, the bell vanes must allow rags and other solids to pass without hindrance, so edges must be smooth and rounded and the passageway must allow a solid 75 mm (3 in) in diameter to pass. Bells with vanes might be costly because of the limited numbers needed and special set-ups required for welding or special patterns for casting, but they are very effective and may well be worth the added expense.

Both vanes and cones must be anchored in place by some means that leaves no other protrusions above the floor to interfere with super-critical currents. For example, two stainless steel bolts 12 mm (1/2 in) or larger should be more than adequate to anchor a vane or cone in place. The tops of the bolts should be below floor level.

f. End Wall Clearance and Fillets

Vortices tend to form downstream from obstructions such as column pumps or vertical suction pipes. Currents past the obstructions wash the vortices away while they are still just swirls and before they can become organized. If there is no current past the pump as occurs at the last pump, the vortices increase in intensity. Reducing the area downstream from the pump inhibits vortices. For example, severe vortices occurred in the Kirkland model between the end wall and the last pump. The vortices were reduced by making the end wall vertical and moving it closer to the pump. The best distance is the least distance, but D/4 is, perhaps, the best practical clearance. Vortices can also be reduced or eliminated by increasing the submergence of the intake.

Another method for reducing the area downstream from an intake is to add fillets in the corners. There seems to be no way to quantify the benefit, but the fillets are indeed somewhat beneficial.

g. Optimum Trench Width and Side Wall Vortex Suppression

Tests for optimum trench width were made by using (as a worst case) an upstream intake set D/2 above the floor. There was no downstream pumping, so water downstream of the intake was practically stagnant. Intermittent Type 2 vortices tended to form at walls beside the pump intakes when the trench width was 2.5 D. The severity of the vortices increased with a reduction of trench width. At a width of 2 D, steady vortices of Type 2 or 3 were seen, and the severity was, perhaps, somewhat less than half that of the strong Type 3 vortex at the flat floor. But even at a trench width of 1.13 D, the wall vortex did not become as severe as the floor vortex. The test was not definitive for establishing limits on trench width, but from practical considerations, little is gained by using a width much less than 2 D, whereas more than 2 D would tend to interfere with cleaning. Widths of 1.875 to 2 D have been used with satisfaction in Seattle Metro pumping stations, and a width of 2 D is often illustrated in the literature [1, 2, 8].

Trials of several types of vortex suppressors were made in a trench 2 D wide with the suction bell 0.5 D above a flat floor. The types included cones, long vortex suppressors with triangular cross-sections, and triangular vanes. All were of some benefit, but the best was the long, horizontal vortex suppressor shown in Figure 27. Judging on the basis of dye injection, this device reduced the severity of the vortices by at least 50 percent.

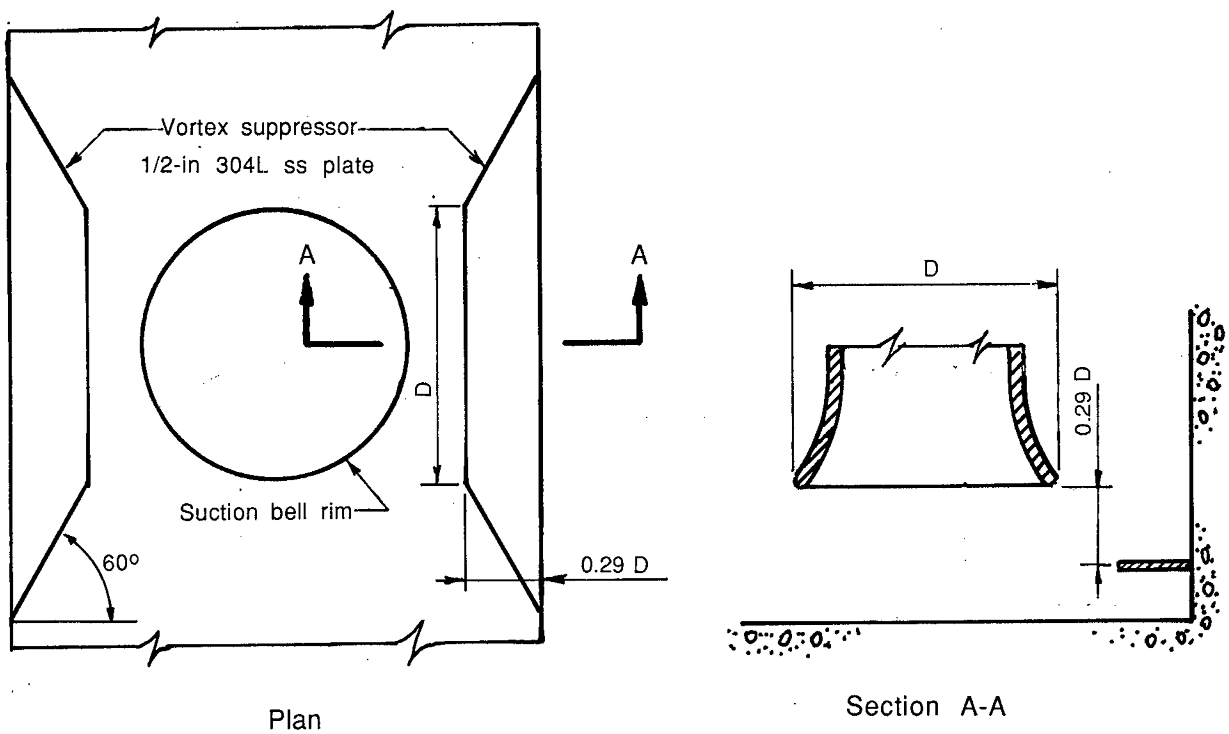


Figure 27. Vortex suppressor for walls.

A Type 3 vortex has only a little effect on a pump and can be safely ignored. But even a vortex of Type 2 in a small model can become a vapor-entraining Type 4 or 5 vortex in a large model or prototype, so it would be wise to install vortex suppressors in walls adjacent to suction bells of 400-mm (16-in) diameter or more.

h. Model Tests of the Fairbanks Morse Pump Intake Basin

The improved Kirkland model was rebuilt to represent (at a linear scale ratio of 1/4.33) the experimental pump basin at the Fairbanks Morse Corporation plant in Kansas City (Section III.F.d) so that the shock wave or "rooster tail" could be investigated. A shock wave did occur but it was insignificant and would hardly have been noticed before tests of the prototype were made. It seems evident that the shock wave was caused by the lower velocity along the side of the trench as compared to the middle. If the sides of the trench were very smooth compared to the bottom, the shock wave might not form at all. Until there is a reliable means for designing such a unique construction and predicting its performance, it seems prudent to set upstream intakes at a substantial floor clearance (no less than D/2) so as to guard against interference from shock waves.

Flow down the ogee ramp was smooth and the water surface downstream was flat. The flow rate converted to prototype units was 49.7 L/s (179 m^3/h or 1.13 Mgal/d). The average depth of flow at the foot of the ogee was, in prototype units, 30 mm (0.10 ft), so the Froude number was 5.7--less than that found in the Fairbanks Morse sump because of the greater friction of a model surface as compared to a prototype surface.

i. General Current Patterns

Many studies of current patterns were recorded, but because only the trench and not the portion above the trench was modeled, the current patterns in Figure 16 is more representative of prototypes. In general, however, the strong surface currents emerging from the influent pipe diminished all along the basin to the end wall, dived, and the resulting weaker currents moved upstream just above the trench. So if surface currents are less than 1 m/s (3 ft/s) at the first pump, currents at pump intakes are sure to be very much less and probably insignificant.

j. Conclusions

The research results prove that a wet well with pumps in tandem and with intakes in a narrow trench has significant advantages and no significant disadvantages. The wet wells are small and therefore less expensive than many common types. Cascades and air entrainment are eliminated and in sewage pumping, odors are less apt to be swept into the atmosphere. The hydraulic environment for the intakes is excellent. Suppositions of interference caused by adjacent intakes for dry pit pumps are found to be without substance.

SECTION VI

RECOMMENDATIONS

A. APPROACH PIPE

The 125-mm (5-in) sloping approach (influent) pipe was tested at only two gradients, 2 and 6 percent and at limited flow rates. The hydraulic jump was never strong enough to entrain large volumes of air. Instead, the jump was relatively weak and some air could escape up the pipe. It is imperative to ensure that air pockets can never block the pipe under any conditions. Until full-scale test are made, designers should not exceed the conservative flow rates given in Table 1.

If the approach pipe is larger than the upstream pipe, the construction of Figure 28 is recommended.

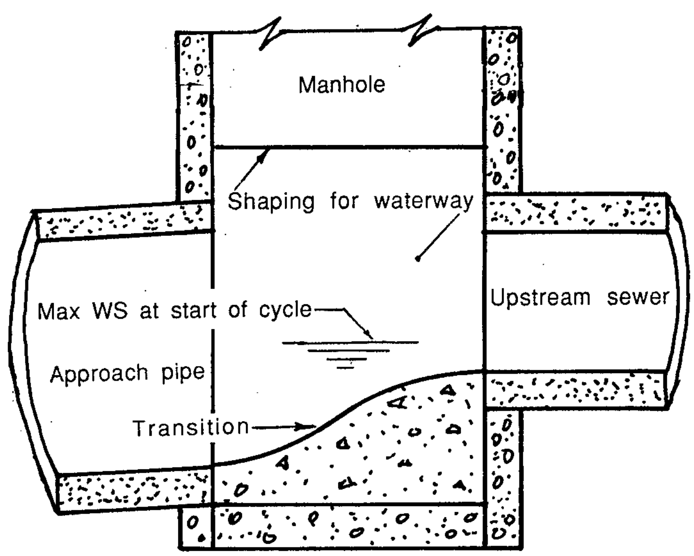


Figure 28. Recommended manhole detail at junction of sewer and approach pipes.

B. SIPHONS VS. PUMPS

The effect of velocity and stray currents has been studied only with siphons--not with real pumps, and real pumps might be affected more than model studies reveal. Pumps are characterized by type number or specific speed by the equation

$$n_s = nQ^{1/2}/H^{3/4} \tag{11}$$

where customarily in Europe, n_s is type number, n is rotation in rev/min, Q is discharge in m^3/s, and H is head in m. Customarily in the U. S., n_s is specific speed, n is rev/min, Q is gal/min, and H is ft. Type numbers greater than 135 (specific speeds greater than 7000) indicate impellers that are very sensitive to irregularities in approach velocities. Also, as pumps of any type get larger, their sensitivity to abnormal fluid approach velocities increases. Furthermore, large pumps are generally less robust than smaller ones, and a condition that might not harm a small pump might destroy a large one in a few years.

Research aimed at analyzing intake throat velocities for pumps in different locations in the trench, at different spacings, at a wide range of water levels and influent flow rates is needed to establish suitable design parameters for column pumps, particularly those of large size.

C. CURRENTS IN PUMP INTAKE BASINS

Although currents near pump intakes were found to be very low, only a few measurements were made and only a few conditions were studied. A fruitful subject for further research is to determine the magnitude of such currents with respect to inflow, water level, floor clearance, pumps operating, distribution of velocities in the throats of the intakes, and dimensions of the basin--especially length and cross-sectional area of the basin above the trench and the effect of width and depth of the trench. For example, small changes in water level resulted in quite different current patterns. Unless currents are very low under a wide range of dimensions and water levels or unless the currents and their effects can be forecast, pumps for trench-type sumps may have to be limited to those relatively insensitive to currents past intakes. Otherwise, model tests for specific basins and pumps will be needed to ensure good performance.

D. FROUDE NUMBERS DURING CLEANING

The limits of acceptable Froude numbers at the base of the ogee and at the last pump should be delineated. The Froude number at the last pump should probably be between about 3.5 and 8. Froude numbers much less than 3.5 signify a weak jump insufficiently effective for moving sand, whereas 8 indicates a strong jump that may entrain too much air. As hydraulic jumps get stronger, more air is entrained, and the Froude number that results in enough air to cause a pump to air bind is unknown as yet.

E. CALCULATING FROUDE NUMBERS

Froude numbers obtained by ignoring head loss due to friction have unacceptable errors because friction is very high when velocities exceed 3 m/s (10 ft/s). To include friction head loss involves dividing the length of pipe or channel into segments and analyzing each in turn by solving Bernoulli's equation.

$$z_1 + h_1 + v_1^2/2g = z_2 + h_2 + h_f + v_2^2/2g \qquad (12)$$

in conjunction with, for example, Manning's equation (Equation 4) and the equation for form resistance (such as the transition from a round pipe to a rectangular channel)

$$h_f = c\, v^2/2g \qquad (13)$$

wherein z is elevation from a datum plane, h is depth of water, v is velocity, g is acceleration due to gravity, Subscripts 1 and 2 refer to the beginning and end respectively of a short length, L, of pipe or channel, h_f is friction head loss, and c is a coefficient of head loss. Accuracy increases with the number of segments, but five or six should be sufficient.

The equations are implicit which means values of "v" or "h" must be estimated for the beginning and end of successive segments and the equations checked for equality, then re-calculated if the error is significant. It is a simple--but an excruciatingly tedious--procedure if done by hand. The computer, however, can eliminate the tedium. For example, MathCAD is one of several computer programs that allow the use of a template to solve a particular kind of problem. Templates can be constructed in about half a day or less by an expert, but even this amount of time may repel designers, so templates for the various popular computer programs should be made universally available.

F. MISCELLANEOUS

A number of questions remain to be fully resolved, such as:

- What parameters of entrance velocity, length of basin, wetted cross-sectional dimensions, and depth below the invert of the influent conduit are required to suppress undesirable floor currents?
- What are the velocity patterns in the throats of the suction bells for all intakes? What are the largest pumps of each type that can be used with confidence and without model tests?
- Can any reasonable construction feature reduce or control shock waves (rooster tails) downstream from the ogee ramp at pump-down?
- How will a trench-type pump intake basin perform if the entrance is normal to the trench? And what are the critical parameters such as trench width, depth below influent conduit invert, approach velocity, and (perhaps) baffles?
- Can conventional pump intake basins be improved by setting the pump intakes in trenches (or depressions of other shapes)?
- Will full scale tests alter the hydraulic limitations imposed by Table 1 on the approach pipeline for preventing surging or any other undesirable flow conditions?
- Can the vane (attached to the bell) at the last pump intake (Figure 26 a) be safely omitted for all types of pumps? For what kinds and sizes of pumps can vortex suppressors be safely omitted?
- Are there combinations of geometry, size, and flow rate that will guarantee successful performance in generic trench-type pumping stations for dry pit, submersible, and column pumps of any reasonable size?

SECTION VII

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