Design of Sewage Pumping Stations

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This article provides guidelines for designing municipal pumping systems.

There are three types of sewage handling systems:

1. Municipal - These systems are designed to serve a given natural drainage area and are part of the public sanitary sewer system.

2. Industrial - These are designed to serve a given industry and, generally pump to the public sanitary sewers. They are usually owned and operated by the industry.

3. Residential - These serve either individuals or multi-family complexes.

They are usually owned and operated by the individual or complexes. Throughout the country, there are several different standards used to design sewage pumping stations. All of these, generally speaking, use the same principles, but there are differences that need to be addressed. The designer should become familiar with the standard that the local municipality is using. The following guidelines are provided to describe a total system that is both reliable and offers low maintenance. The location of the pumping station will be a function of its size. But even medium to small pump stations need access by maintenance crews and equipment, and ease of access should always be considered. In all cases, the pumping stations should be protected from physical damage by a local waterway, using one hundred (100) year flood.

FLOW DETERMINATION

To determine the daily average flow (DAF), the service area has to be set. This could include an initial service area and an ultimate service area. Land utilization should be available from local or regional planning and zoning agencies for the initial area. Future utilization should be based on plans for the service area. With land utilization determined, an equivalent residential population can be determined by multiplying the acres for each zoning class by the estimated flow rate for that class for the entire service area. *Table 1* illustrates the estimated sewage flow of undeveloped land.

Table 1 - Undeveloped Area-Wastewater Flows[1]

Zoning district	Dwelling units/acre	Population per unit - acre		Average gal/ cap/day - acre/day	
R-2*	2	4	8	105	840
R-3*	3.5	4	14	105	1,470
R-4**	3.75	4	15	105	1,575
R-5	6	4	24	105	2,520
R-6	15	3	45	105	4,725
R-7	30	3	90	105	9,450
R-8 & R-8A	50	3	150	105	15,750

Zoning districtt	Equivalent pop/acre	Average gal/ cap/day	Average gal/ acre day
R-9	150	105	15,750
R-10	150	105	15,750
Commercial	20	105	2,100
Industrial***	36	105	3,780
Non-Developable Land	1	105	105

* Saturation standards applicable to the design of collector or interceptor systems.
** In general, for undeveloped areas, the R-4 density is to be considered as a minimum for collection system design unless present development in the vicinity indicates that design for the actual zoning, with MSD approval, would be more prudent.
*** This figure may be adjusted by MSD if a major industrial user is anticipated.
(1)Louisville and Jefferson County Metropolitan Sewer District (MSD) Design Manual, Table 5-2.

After the DAF for both the initial and ultimate service area has been determined, it is multiplied by a peaking factor to determine a peak flow rate. This peaking factor will vary between two and four, depending on the service area and local requirements, as described in Section 32.38 of the Recommended Standards For Sewage Works, 1978 Edition, (i.e., Ten State Standards). The peaking factor is required so that the pump can handle variations of the inflow to the wet well during the day.

FORCE MAIN SIZING/MATERIAL

With the peak flow rate determined, the force main can be sized. The velocity in the force main should be a minimum of 2 feet per second (fps) and a maximum of 5 fps. This is to keep the solids in suspension, but not to generate a large head loss through the force main. If an initial and ultimate flow rate are being used to design the force main, the velocity may not be maintained within these ranges. If the flow rate varies too much, one option is to install dual force mains to allow the velocity to be better controlled. A second option is to provide a variable drive unit so that the pump could match the incoming flow closer. This would require the system to be designed for the ultimate flow.

Minimum pipe sizes should be 4 inches when wastewater pumps are used that have at least a 2%-in. solids passing capacity so that clogging of the force main is minimized. If smaller force mains are needed, then a grinder pump should be utilized.

Force mains can be constructed from several different materials. PVC and polyethylene are the most common materials used today because of cost and roughness coefficient. The construction of force mains should be similar to water lines in that thrust restraints and blocks should be provided at bends and tees. Also, expansion and contraction of the force main through the slip joints should be planned for. Air release valves should be provided at high points to prevent air locking and siphoning. Vacuum valves shall be provided as needed to admit air after a pumping cycle. Consideration should also be given to cleanouts so that places where clogs may develop can be cleaned; typically, at low spots or at changes in direction.

SYSTEM HEAD CURVE ANALYSIS

Now that a force main has been sized, the system head curve can be determined. All elbows, fittings, entrances, exits, and pipe lengths should be used to determine an equivalent pipe length. Force main friction losses can be based on the Hazen-Williams equation. With the force main size, material, and equivalent length, the system head curve can be determined.

The two elements of the system head curve are (1) the static head and (2) the friction head.

1) Static head is defined as the vertical lift of the fluid that the pump has to overcome. It is assumed to be a constant head after the station is put into operation for a baseline of the system head curve. It is defined as follows:

Static Head = Highest elevation opened to the atmosphere* minus the system’s low point**

*This will typically be the pipe outlet.

**All pumps off elevation (Suggestion: Use the average elevation between the “lead pump on” and “all pumps off”. This will give the mid point of the pump operation range.)

2) Friction head will vary during pumpdown of the wet well, as noted above. See Figure 1 (below), which notes the pumping range caused by the change in the static head during the pumping cycle.

In a given system, the friction head will vary with the flow rate, as defined by the following equation:

$$H^{L} = 10.4397\,(L)\frac{(Q)^{1.85}}{(C)^{1.85}\,(d_i)^{4.8655}}$$

where, **H_L**= Total friction head loss, feet of water

L=Length of equivalent

pipe length of diameter d_i ft

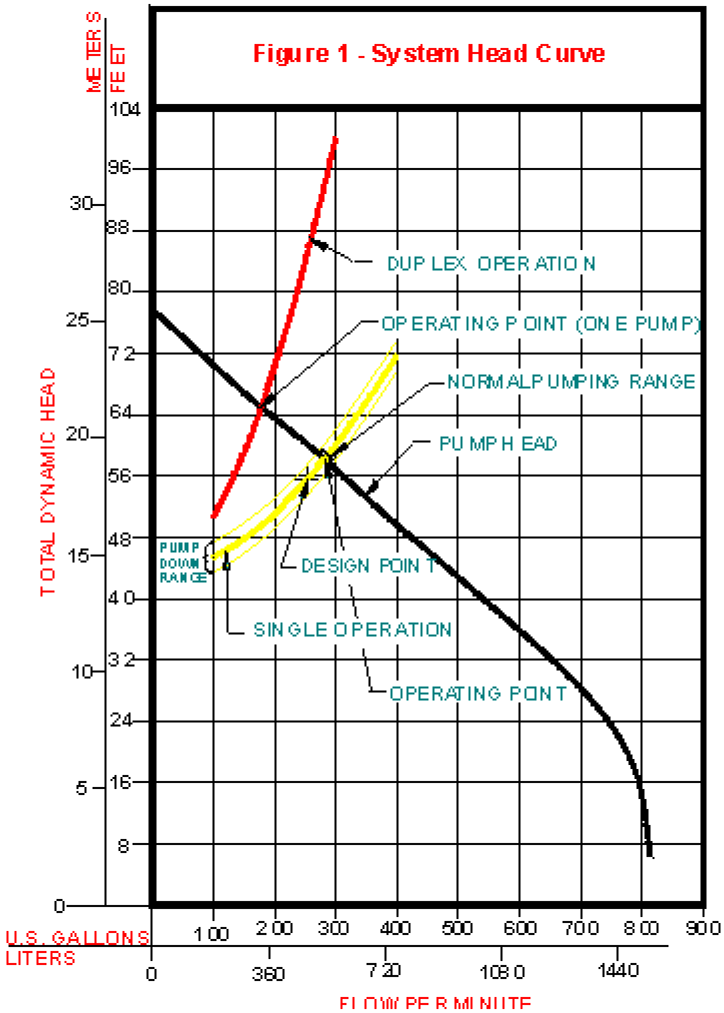
C = Hazen-Williams flow coefficient (see Table 2)

Q= Flow rate, gallons per minute (gpm)

d = Internal pipe diameter, inches (in)

The head loss through the system should be determined for each section of the system separately based on pipe material, pipe diameter, and amount of flow. If multiple pumps of the same size are to be operating at the same time, then the flow rate from the pump to the common force main is assumed to be equal to one divided by the number of pumps running.

A general rule of thumb is to generate the system head curve with approximately 10 points from 50 percent to 150 percent of the design flow. A separate system head curve is generally required to determine the total capacity of a multiple operating pump station. The systems can then be plotted. A system head curve calculation follows in the next section.



TOTAL DYNAMIC AND STATIC HEAD CALCULATIONS

I. Pump Station Design Flow Data

A. Average Daily Flow 122,400 gpd
B. Average flow/1,440 85 gpm
C. Pump Rate 300% (Peaking Factor) 255 gpm required

II. Roughness Coefficient

C = 120 for Ductile Iron Pipe (DIP); ID DIP = 6 in.
C = 150 for PVC; ID PVC = 6 in.

III. Equivalent Lengths and Minor Losses

DIP (ft)		
Components	**Eq Length**	**Feet**
1, Gate Valve	3.5	3.50
1, Check Valve	40.0	40.00
1, Tee	30.0	30.00
3, 90° Elbows	14.0	42.00

Pipe Length	--	26.00
Total Equivalent Length		141.50

PVC (ft)		
Components	**Eq Length**	**Feet**
7, 90° Elbows	14	98.00
4, 90° Elbows	7.5	30.00
Pipe Length	--	2,348.00
Total Equivalent Length		2,476.00

IV. Static Head

A. High Point in System 475.60
B. Low Point in System 432.40
Total Static Head 43.20

V. Design Curves

System Curve Calculations (See Tables A and B.)

In considering a pump to meet system needs, the operating point for the selected pump should coincide as closely as possible with the design flow and best efficiency point of the pump. Pump efficiency is also an important factor to consider in the selection process. Pump efficiencies will vary because of impeller design (vortex, semi-open, closed) and pump housing design (concentric or convolute). While all these features have unique characteristics, they must be considered in the pump selection process to give long term service and reliability. If pump efficiency is not published, they can be obtained directly from the manufacturer.

After the pump is selected, the useful water horsepower (whp) can be determined which is defined as:

whp = (1) (TDH) /3,960

where, **q =** pumping rate (gpm)
TDH = total dynamic head (ft) at q
Brake horsepower (bhp) = whp/ pump efficiency

The system head curve can then be plotted on the pump performance curve for both single and dual pump operations to determine the operating points of the system. The change in the static head during drawing down will change the pumping rate. As shown in Figure 2, the normal pumping range will vary by the change in the static head.

Table 2 - Values of Hazen-Williams Coefficient C[1]	
Pipe Material	**C**
Asbestos-Cement	140
Brass	130
Brick Sewers	100
Cast Iron:	
New	130
5 Yrs. Old	120
10 Yrs. Old	100
Concrete (regardless of age)	130

Copper	130
Galvanized iron	120
Polyethylene	140
PVC	150
Riveted Steel, New	110
Vitrified Clay	110
Welded Steel, New	120
Wood Stave (regardless of age)	120

(1) Louisville and Jefferson County Metropolitan Sewer District (MSD) Design Manual, Table 5-2.

WET WELL SIZING

"Design of Wastewater and Stormwater Pumping Stations" Water Pollution Control Federation, Manual of Practice No. FD-4, 1981, p. 18, indicates that the wet well shall be sized so that the cycle time for each pump will not be less than five minutes or that the average cycle time will not be more than 30 minutes. The shortest operating cycle occurs when the inflow equals to one-half the pump discharge rate. Therefore, if

V = drawndown volume, gal

q = Pump discharge rate, gpm

Q = Inflow rate into the wet well, gpm

t = Minimum time of one pumping cycle in minutes, start to start

t = (time to fill) + (run time)

then $$t = \frac{V}{Q} + \frac{V}{q-Q}$$

When Q = q/2,

then $$t = \frac{V}{q/2} + \frac{V}{q-(q/2)}$$

which is reduced to the operating volume where

$$V = \frac{tq}{4}$$

With the operating volume, the vertical distance between the lead pump on and all pumps off floats can be determined for various wet well sizes. Between the operating volume and emergency storage requirement, the wet well size can be determined. Emergency storage volume will be dependent on the required response time and the average inflow. The emergency storage volume requirement will vary between governing agencies, but storage should be provided within the sewer system below the lowest sewer tap or the lowest overflow of the sewer system. Storage should be contained within the wet well, surge tank, incoming sewer lines, or upstream manholes.

After the size of the wet well has been determined, then the distance between the floats for lead pump on and all pumps off floats can be determined. This would be a function of wet well size and the operating volume requirement. The vertical distance between the common stop elevation and the bottom of the wet well is a function of the pump selected. The common stop elevation shall not be less than the top of the pump housing or as the manufacturer specifies, whichever is greater.

The distance between the lead, lag, and high water levels are generally a function of the local requirement. If mercury floats are utilized, then these should not be spaced less than six inches apart, with the high water alarm level being at or lower than the lowest incoming sewer line.

These settings will determine the depth of the wet well which will allow the buoyancy calculations to be completed. The buoyancy analysis on the wet well will determine whether additional methods of restraint will be necessary. Mechanical equipment, water weight, and other temporary loads should not be included in the analysis. The soil angle of repose should be assumed to be zero degrees, unless soil analysis determines that another value is warranted.

The buoyancy force equals the displaced volume of the wet well and bottom slab multiplied by the unit weight of water.

The opposing force is equal to the weight of the wet well, bottom slab, top slab, and the soil over the bottom slab extension, if applicable. The safety factor is equal to the opposing force divided by the buoyancy force. The safety factor should be $\geq$1.5.

Table A- Single Pump Operation

Q	DIP		PVC		Col5 (Col1 + Col3)		Col7 (Col5 + Col6)
	Col 1	Col 2	Col 3	Col 4		Col 6	
(GPM)	Friction head (ft.)	V (fps)	Friction head (ft.)	V (fps)	Total friction head (ft)	Static head (ft.)	Total dynamic (ft)
100	0.2	1.16	2.0	1.16	2.2	43.2	45.4
125	0.3	1.45	3.0	1.45	3.3	43.2	46.5
150	0.4	1.74	4.3	1.74	4.7	43.2	47.9
175	0.5	2.03	5.7	2.03	6.2	43.2	49.4
200	0.6	2.32	7.3	2.32	7.9	43.2	51.1
225	0.8	2.61	9.1	2.61	9.9	43.2	53.1
250	1.0	2.90	11.0	2.90	12.0	43.2	55.2
275	1.2	3.19	13.1	3.19	14.3	43.2	57.5
300	1.4	3.48	15.4	3.48	16.8	43.2	60.0
375	2.1	4.35	23.3	4.35	25.4	43.2	68.6
400	2.3	4.64	26.3	4.64	28.6	43.2	71.8
Design Point 255	1.0	2.96	11.4	2.96	12.4	43.2	55.6

Table B - Duplex Pump Operation; Single Pump Curve

Q	DIP		PVC*		Total friction head (ft.)	Static head (ft)	Total dynamic (ft)
(GPM) Ea. Pump	Friction (ft.)	V (fps)	Friction (ft.)	V (fps)			
100	0.2	1.16	7.3	2.32	7.5	43.2	50.7
125	0.3	1.45	11.0	2.90	11.3	43.2	54.5
150	0.4	1.74	15.4	3.48	15.8	43.2	59.0
175	0.5	2.03	20.5	4.06	21.0	43.2	64.2
200	0.6	2.32	26.3	4.64	26.9	43.2	70.1
225	0.8	2.61	32.7	5.23	33.5	43.2	76.7
250	1.0	2.90	39.7	5.81	40.7	43.2	83.9
275	1.2	3.19	47.4	6.39	48.6	43.2	91.8
300	1.4	3.48	55.7	6.97	57.0	43.2	100.2
375	2.1	4.35	84.2	8.71	86.2	43.2	129.4
400	2.3	4.64	94.9	9.29	97.2	43.2	140.4

* - Common Force main. The total flow in the common force main will be twice the flow rate of one pump operating when two pumps are operating in parallel.

FORCE MAIN PRESSURE AND WATER HAMMER CALCULATIONS

Water hammer is an increase in pressure in the pipe caused by a sudden change in the velocity. The velocity change usually results from the closing of a valve. From Uni-Bell Handbook of Pipe, Design and Construction, 1986, Chapter V, the maximum surge pressure encountered is a function of the wave velocity as follows:

a = 4,660/ (1 + (kd/ET)) $^{1/2}$

where, **a =** Wave velocity (fps)
k= Fluid bulk modulus (300,000 psi for water)
d =- Pipe I.D. (in.)
E = Modulus of elasticity of the pipe;
400,000 psi for PVC pipe
24,000,000 psi for ductile iron
110,000 psi for polyethylene
t = Pipe wall thickness (in.)

a = 4,660/((1 + (k/E) (DR-2))$^{1/2}$

where, **DR =** Dimension Ratio
= O.D. (in.)/Wall thickness (in.)

The maximum surge pressure, P, then equals
P =- aV/2.31g

where, **a =** Wave velocity (fps), as defined above
V = Maximum change in velocity (fps)
g = Acceleration due to gravity (32.2 ft/sec^2)

To determine the maximum change in velocity, a worst-case scenario should be used. This would occur when all the pumps are running at minimum static head condition (this is the maximum velocity the pumps will produce), and are suddenly shut down. The minimum static head would occur when the system has the maximum water level present. Care should be taken to consider a future ultimate flow condition when larger pumps might be installed in the wet well which would, in turn, generate larger flows and velocities. The current method to approach this problem would be to select a pump for the ultimate flow condition and determine the operating point at the minimum static head.

The total pressure (surge pressure plus static pressure) can then be checked against the pressure rating of the pipe.

Cyclic surge failure is another consideration in the selection of pipe material for the force main. Research has demonstrated that in piping systems where the total variation in pressure cycle surges equals or exceeds 50 percent of the working pressure, the force main may fail due to fatigue. As research on the subject continues, understanding of fatigue failure phenomenon is expanding; consequently, design treatment to accommodate cyclic surging and “cycle fatigue” is being refined.

From a regression analysis of research data related to cyclic surge pressure effects, H.W. Vinson developed the following formula:

C = (5.05 S. 10^{21}) S $^{-4.906}$

where, **S =** Peak hoop stress (psi)
C = Average number of cycles to failure

This formula implies that at the defined number of cycles, C, 50 percent of the PVC pipes tested would not fail. It is recommended that the design be approached as follows:

1) Determine the peak pressure, P, from the system's hydraulics, including both working and surge pressure. This is to be compared against the assumed pipe strength.

2) Using the assumed pipe strength, determine the maximum allowable hoop stress [i.e., International Standards Organization (ISO) for PVC pipe: S = P(DR 1)/2].

3) Calculate the average number of cycles to failure.

4) Estimate the years of service before failure for the proposed system and check it against its design lifetime:

$$C = \frac{\text{cycles}}{\text{hrs}} + 24\ \frac{\text{hrs}}{\text{day}}$$

$$\times\ 365\ \frac{\text{days}}{\text{year}}\ \times\ \text{years}$$

where, **t =** minimum time of one pumping cycle in minutes, start to start

The following examples illustrate the surge pressure and cyclic surge calculations:

Assume **SDR =** 32.5 for 6-in. PVC force main

Pressure rating = 125 psi

$$a = \frac{4{,}660}{\left[1 + \frac{(30{,}000)(6)}{400{,}000\,(0.204)}\right]^{1/2}}$$

$$= \frac{4{,}660}{\sqrt{23.06}} = 970.4\ \text{fps}$$

$$P_S = \frac{970.4\,(4.06)}{2.31\,(32.2)} = 52.97\ \text{psi}$$

$$P_T = 52.97 + 43.2(62.4)/144$$

$$P_T = 52.97 + 18.72 = 71.69\ \text{psi}$$

<125 psi rated (SDR 32.5)

Using International Standards Organization (ISO)

$$S = \frac{P(D.R. - 1)}{2} =$$

$$\frac{71.69(32.5 - 1)}{2} = 1{,}129.1\ \text{psi}$$

$$C = (5.05 \times 10^{21})(1{,}129.1)^{-4.906}$$

$$= 5.3 \times 10^{6}\ \text{cycles}$$

Solve for years of service; assume 6 cycles per hour

$$\text{Year} = 5.3 \times 10^{6}/(6 \times 24 \times 365)$$

$$= 101\ \text{years}$$

Although the Vinson cyclic surge equations are useful tools in determining the fatigue due to cyclic surges, engineers must appreciate the limitations of these equations, which are:

1) The formulas were developed using large surges (25 or 50 percent above and below a base pressure) in PVC pipe specimen.

2) The cycle frequencies were 6 to 10 cycles per minute. The equations provide no allowance for the stress relaxation phenomenon.

The following are several design considerations that have not been covered, but are critical in some applications.

1) Odor Control - Generally speaking, if the detention in either the wet well or force main based on the average flow is less than 30 minutes, then there should be very few problems. The wet well should be properly vented to the atmosphere.

2) Net Positive Suction Head (NPSH) - In small to medium submersible pump stations, if the pump housing is submerged and the wet well is vented to the atmosphere, there should be few problems. When there are high flows, cavitation could be a major consideration.

3) Air/Vacuum Valves - Depending upon the profile and size of the force main, air or vacuum pressure could be a major factor in the life cycle of the system. Air entrapment can cause an excessive head requirement that the pump cannot overcome and large down grade profiles that open to the atmosphere can cause excessive negative head that could collapse the force main or exceed the pump's capacity, causing it to overheat and burn up because of the negative head.

4) Safety - The design of a pumping station requires a review of the components of the system to assure that the system is safe to operate. Access ladders for the wet well and valve vault, a hoist for lifting out the pump, lighting, ventilation to remove dangerous gases and security for the electrical system are the major safety items that need to be considered.

5) Wet Well Dead Zones - In all wet wells, there are areas that will allow solids to drop out of suspension. These areas need to be eliminated or a method provided to resuspend the solids so that they are moved along.

Bibliography

1) Recommended Standards for Sewage Works, 1978 Edition, Great Lakes Upper Mississippi River Basin Board of State Sanitary Engineers.

2) Water Pollution Control Federation (WPCF), Manual of Practice No. FD-4, 1981, "Design of Wastewater and Storm- water Pumping Stations".

3) UNI-BELL Handbook of Pipe, Design, and Construction, 1986.