M5: Distribution and Storage Reservoirs, and Pumps

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Design Periods and Capacities in Water-Supply Systems (Table 2.11 Chin 2006)

Component	Design period (years)	Design capacity
Sources of supply:		
River	indefinite	maximum daily demand
Wellfield	10–25	maximum daily demand
Reservoir	25–50	average annual demand
Pumps:		
Low-lift	10	maximum daily demand, one reserve unit
High-lift	10	maximum hourly demand, one reserve unit
Water-treatment plant	10–15	maximum daily demand
Service reservoir	20–25	working storage plus fire demand plus emergency storage
Distribution system:		
Supply pipe or conduit	25–50	greater of (1) maximum daily demand plus fire demand, or (2) maximum hourly demand
Distribution grid	full development	same as for supply pipes

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Distribution Reservoirs

- Provide service storage to meet widely fluctuating demands imposed on system.
- Accommodate fire-fighting and emergency requirements.
- Equalize operating pressures.

Types of Distribution and Service Reservoirs

- **Surface Reservoirs** – at ground level – large volumes
- **Standpipes** – cylindrical tank whose storage volume includes an upper portion (useful storage) – usually less than 50 feet high
- **Elevated Tanks** – used where there is not sufficient head from a surface reservoir – must be pumped to, but used to allow gravity distribution in main system

Location of Distribution Reservoirs

- **Provide maximum benefits of head and pressure (elevation high enough to develop adequate pressures in system)**
- **Near center of use (decreases friction losses and therefore loss of head by reducing distances to use).**
- **Great enough elevation to develop adequate pressures in system**
- **May require more than one in large metropolitan area**

Storage Tank Types

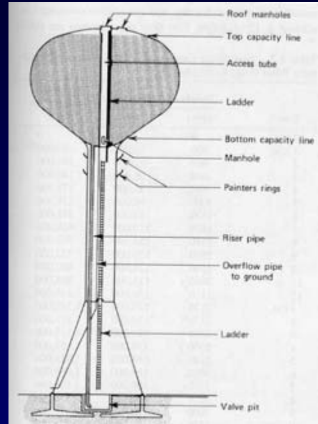


Figure 6-16 Elevated spheroid-shaped water storage tank available in capacities from 0.50 to 3.0 mil gal and specified pedestal height. (Courtesy of Chicago Bridge and Iron Co.)

Figure 6-17 Ornamented steel standpipe for ground-level water storage, designed and built in any capacity. (Courtesy of Chicago Bridge and Iron Co.)

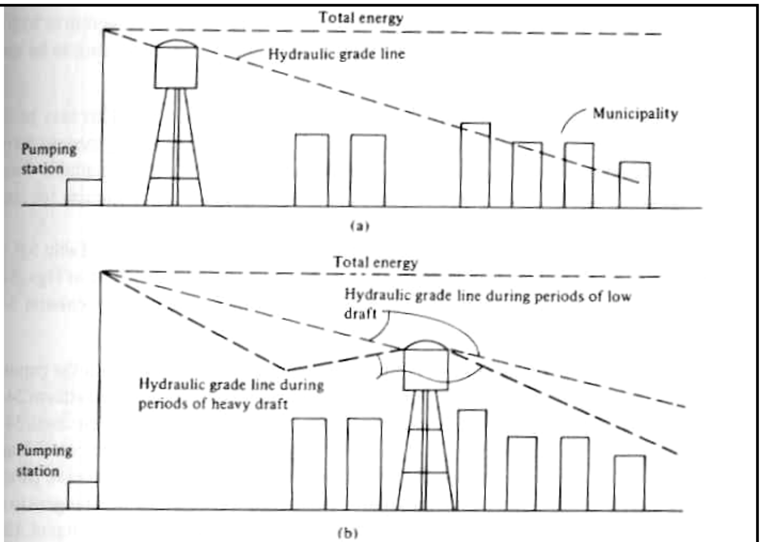


Figure 5.26 Pressure distribution as influenced by the location of a distribution reservoir.

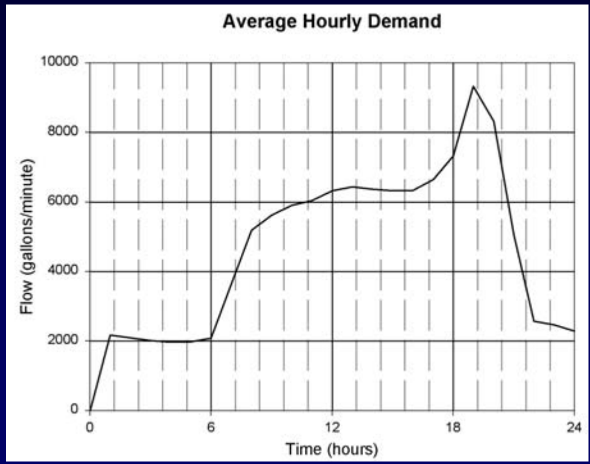
Determining Required Storage Amount

- Function of capacity of distribution network, location of service storage, and use.
- To compute required equalizing or operating storage, construct mass diagram of hourly rate of consumption.
  - Obtain hydrograph of hourly demands for maximum day (in Alabama, this would likely be either late spring or summer, and would include demands for lawn care, filling and/or maintaining swimming pools, outdoor recreation, etc.)
  - Tabulate the hourly demand data for maximum day.
  - Find required operating storage using mass diagrams, hydrographs, or calculated tables.

Determining Required Storage Amount: Example

Time (hr)	Hourly Demand (gpm)	Time (hr)	Hourly Demand (gpm)	Time (hr)	Hourly Demand (gpm)	Time (hr)	Hourly Demand (gpm)
0	0	7	3,630	13	6,440	19	9,333
1	2,170	8	5,190	14	6,370	20	8,320
2	2,100	9	5,620	15	6,320	21	5,050
3	2,020	10	5,900	16	6,340	22	2,570
4	1,970	11	6,040	17	6,640	23	2,470
5	1,980	12	6,320	18	7,320	24	2,290
6	2,080						

Determining Required Storage Amount: Example



To calculate total volume required for storage

- Calculate hourly demand in gallons.
- Calculate cumulative hourly demand in gallons.
- Divide cumulative demand by 24 hours to get the average hourly supply needed.
- Calculate the surplus/deficit between the hourly supply and hourly demand for each hour.
- Sum either the surplus or the deficit to determine the required storage volume.

REMEMBER: This result means that the tank is full when the surplus is greatest and empty when the deficit is greatest! It is filling when the slope of the pump curve is greater than the slope of the demand curve and is emptying when the slope of the demand curve is greater than the slope of the pump curve. There is no excess for emergencies!

Determining Required Storage Amount: Example

Time (hrs)	Hourly Demand (gpm)	Hourly Demand (gal)	Cumulative Hourly Demand (gal)	Surplus/ Deficit* (gal)	Deficits (gal)
0000 - 0100	2,170	130,200	130,200	156,008	surplus
0100 - 0200	2,100	126,000	256,200	160,208	surplus
0200 - 0300	2,020	121,200	377,400	165,008	surplus
0300 - 0400	1,970	118,200	495,600	168,008	surplus
0400 - 0500	1,980	118,800	614,400	167,408	surplus
0500 - 0600	2,080	124,800	739,200	161,408	surplus

* Surplus or deficit = average hourly demand (286,208 gal) – actual hourly demand

Determining Required Storage Amount: Example

Time (hrs)	Hourly Demand (gpm)	Hourly Demand (gal)	Cumulative Hourly Demand (gal)	Surplus/ Deficit (gal)	Deficits (gal)
0600 - 0700	3,630	217,800	957,000	68,408	surplus
0700 - 0800	5,190	311,400	1,268,400	-25,193	-25,193
0800 - 0900	5,620	337,200	1,605,600	-50,993	-50,993
0900 - 1000	5,900	354,000	1,959,600	-67,793	-67,793
1000 - 1100	6,040	362,400	2,322,000	-76,193	-76,193
1100 - 1200	6,320	379,200	2,701,200	-92,993	-92,993
1200 - 1300	6,440	386,400	3,087,600	-100,193	-100,193

Determining Required Storage Amount: Example

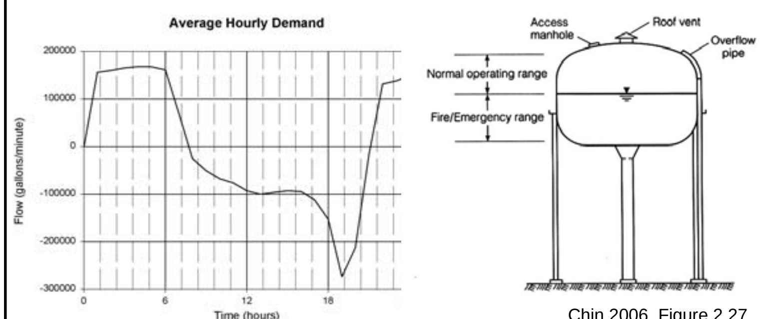
Time (hrs)	Hourly Demand (gpm)	Hourly Demand (gal)	Cumulative Hourly Demand (gal)	Surplus/ Deficit (gal)	Deficits (gal)
1300 - 1400	6,370	382,200	3,469,800	-95,993	-95,993
1400 - 1500	6,320	379,200	3,849,000	-92,993	-92,993
1500 - 1600	6,340	380,400	4,229,400	-94,193	-94,193
1600 - 1700	6,640	398,400	4,627,800	-112,193	-112,193
1700 - 1800	7,320	439,200	5,067,000	-152,993	-152,993
1800 - 1900	9,333	559,980	5,626,980	-273,773	-273,773
1900 - 2000	8,320	499,200	6,126,180	-212,993	-212,993

Determining Required Storage Amount: Example

Time (hrs)	Hourly Demand (gpm)	Hourly Demand (gal)	Cumulative Hourly Demand (gal)	Surplus/ Deficit (gal)	Deficits (gal)
2000 - 2100	5,050	303,000	6,429,180	-16,793	-16,793
2100 - 2200	2,570	154,200	6,583,380	132,008	surplus
2200 - 2300	2,470	148,200	6,731,580	138,008	surplus
2300 - 2400	2,290	137,400	6,868,980	148,808	surplus
					Sum of deficits = 1,465,275
Average*		286,208 gallons			
Storage Required		1,465,275 gallons (sum of all deficits) plus additional for emergencies			

* Average = cumulative hourly demand / 24 hours = 6,868,980 gal/24 hrs = 286,208 gal/hr

Surplus/Deficit



Chin 2006, Figure 2.27

Example 2.17 (Chin 2006)

A service reservoir is to be designed for a water supply serving 250,000 people with an average demand of 600 L/day/capita and a fire flow of 37,000 L/min.

The required storage is the sum of:

(1) volume to supply the demand in excess of the maximum daily demand,
(2) fire storage, and
(3) emergency storage

(1) The volume to supply the peak demand can be taken as 25% of the maximum daily demand. The maximum daily demand factor is 1.8 times the average demand. The maximum daily flowrate is therefore:

$$Q_m = 1.8(600\,L/d/capita)(250{,}000\ people) = 2.7x10^8\,L/day = 2.7x10^5\,m^3/day$$

The corresponding volume is therefore:

$$V_{peak} = (0.25)(2.7x10^5\,m^3/day) = 67{,}500m^3$$

(2) The fire flow of 37,000 L/min (0.62 m^3/sec) must be maintained for at least 9 hours. The volume to supply the fire demand is therefore:

$$V_{fire} = (0.62m^3/\sec)(9hours)(3{,}600\sec/hr) = 20{,}100m^3$$

(3) The emergency storage can be taken as the average daily demand:

$$V_{emer} = (250{,}000\,people)(600L/day/capita) = 150x10^6\,L = 150{,}000m^3$$

The required volume of the service reservoir is therefore:

$$V = V_{peak} + V_{fire} + V_{emer}$$
$$= 67{,}500m^3 + 20{,}100m^3 + 150{,}000m^3 = 237{,}600m^3$$

Most of the storage for this example is associated with the emergency volume. Of course, the specific factors must be chosen in accordance with local regulations and policy.

Example 2.18 (Chin 2006)

A water supply system design is for an area where the minimum allowable pressure in the distribution system is 300 kPa. The head loss between the low pressure service location (having a pipeline elevation of 5.40 m) and the location of the elevated storage tank was determined to be 10 m during average daily demand conditions. Under maximum hourly demand conditions, the head loss is increased to 12 m. Determine the normal operating range for the water stored in the elevated tank.

Under average demand conditions, the elevation (z_o) of the hydraulic grade line (HGL) at the reservoir location is:

$$z_0 = \frac{p_{\min}}{\gamma} + z_{\min} + h_L$$

$where:$

$p_{\min} = 300kPa$

$\gamma = 9.79kN/m^3$

$z_{\min} = 5.4m$

$h_L = 10m$

Therefore, under average conditions:

$$z_0 = \frac{300kPa}{9.79kN/m^3} + 5.4m + 10m = 46.0m$$

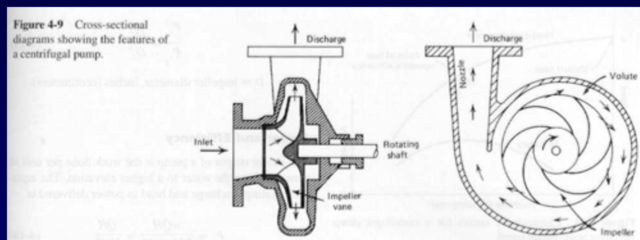
and under maximum demand conditions, the elevation of the HGL at the service reservoir, z_1, is:

$$z_1 = \frac{300kPa}{9.79kN/m^3} + 5.4m + 12m = 48.0m$$

Therefore, the operating range in the storage tank should be between 46.0 and 48.0 m.

PUMPING

- Two types of pumps commonly used in water and sewage works:
- Centrifugal Pumps – typical use is to transport water and sewage



- Displacement Pumps – typical use is to handle sludge in a treatment facility

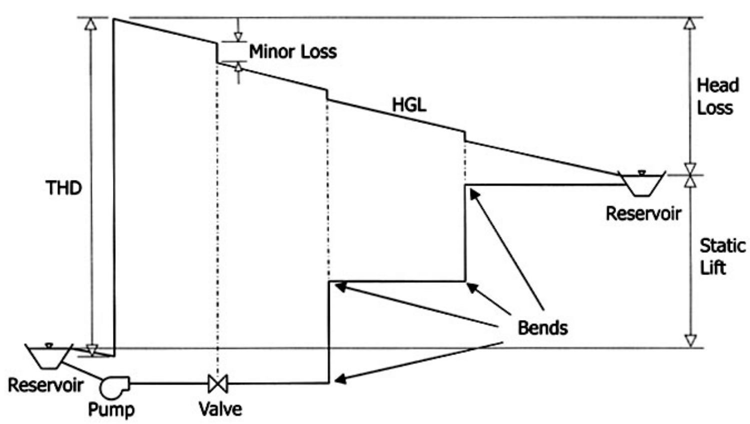
Pump selection guidelines

Type of pump	Range of specific speeds, n_s*	Typical flowrates (L/s)	Typical efficiencies (%)
Centrifugal	0.15–1.5 (400–4,000)	< 60	70–94
Mixed flow	1.5–3.7 (4,000–10,000)	60–300	90–94
Axial flow	3.7–5.5 (10,000–15,000)	> 300	84–90

*The specific speeds in parentheses correspond to N_s given by Equation 3.112, with ω in rpm, Q in gpm, and h_p in ft.

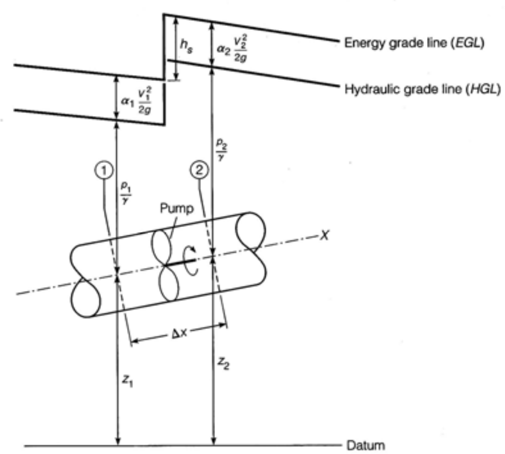
(Chin 2006 Table 2.3)

Schematic of hydraulic grade line for a pumped system



(Walski, *et al.* 2001 figure 2.16)

Pump effect on flow in pipeline

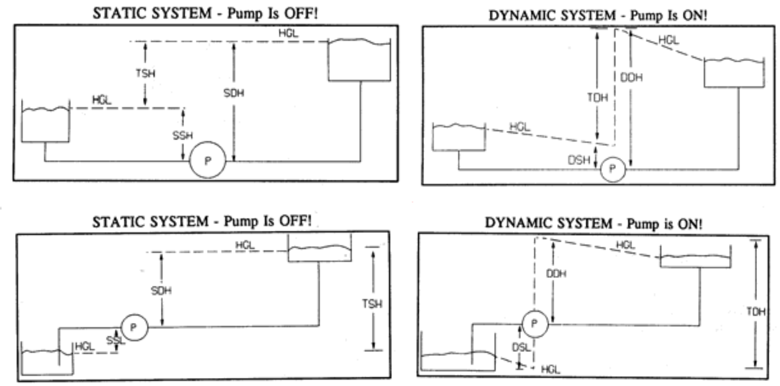


(Chin 2006 Figure 2.6)

Sizing Pumps

- To determine the size of the pump, must know the total dynamic head that the pump is expected to provide. Total Dynamic Head (TDH) consists of:
  - the difference between the center line of the pump and the height to which water must be raised
  - the difference between the suction pool elevation and center line of the pump
  - friction losses in the pump and fittings
  - velocity head

Static vs. Dynamic Heads



HGL = hydraulic grade line
TSH = total static head
SSH = static suction head
SSL = static suction lift
SDH = static discharge head

TDH = total dynamic head
DDH = dynamic discharge head
DSH = dynamic suction head
DSL = dynamic suction lift

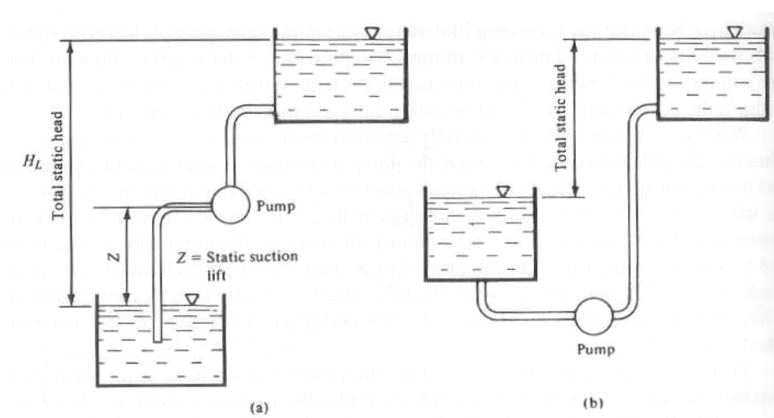


Figure 5.31 Total static head: (a) Intake below the pump centerline. (b) Intake above the pump centerline.

Head Added by Pump (Total Dynamic Head)

- If the pump has been selected, Bernoulli's Equation can be rearranged to solve for the head added by a pump:

$$h_A = \frac{P_2 - P_1}{\gamma} + \frac{V_2^{\,2} - V_1^{\,2}}{2g} + Z_2 - Z_1 + h_f$$

where
h_a = head added by pump (total dynamic head)
P = atmospheric pressure
γ = specific weight of fluid
V = velocity
Z = elevation
h_f = head loss in attached pipe and fittings

Installation of Pumps into Water Supply System

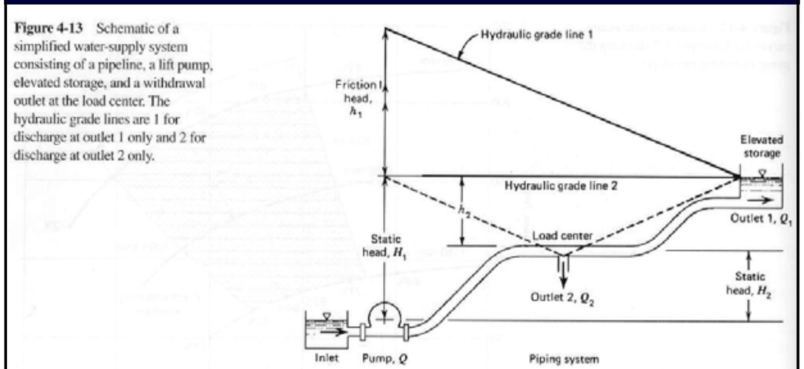


Figure 4-13 Schematic of a simplified water-supply system consisting of a pipeline, a lift pump, elevated storage, and a withdrawal outlet at the load center. The hydraulic grade lines are 1 for discharge at outlet 1 only and 2 for discharge at outlet 2 only.

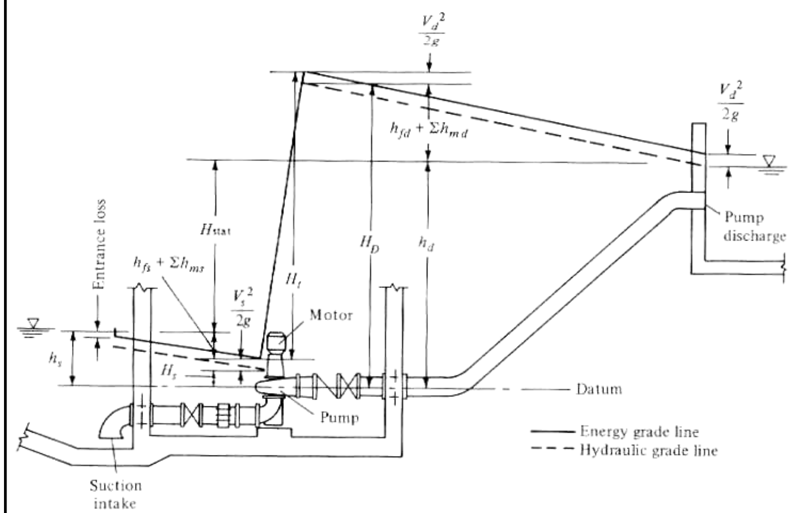


Figure 8-1 Definition sketch for the head on a pump.

Example of Head Added by Pump

Example:

- A pump is being used to deliver 35 gpm of hot water from a tank through 50 feet of 1-inch diameter smooth pipe, exiting through a ½-inch nozzle 10 feet above the level of the tank. The head loss from friction in the pipe is 26.7 feet. The specific weight of the hot water is 60.6 lb_f/ft^3.

Example for Head Added by Pump

- Solve:

$$h_A = \frac{P_2 - P_1}{\gamma} + \frac{V_2^{\,2} - V_1^{\,2}}{2g} + Z_2 - Z_1 + h_f$$

- Set reference points. Let point 1 be the water level in the tank. Let point 2 be the outlet of the nozzle.

Example for Head Added by Pump

- Since both the end of the nozzle and the top of the tank are open to the atmosphere, let $P_1 = P_2 = 0$ (gage pressure).
- In a tank, the velocity is so small as to be negligible, so $V_1 = 0$.
- Calculate V_2.

$$V_2 = \frac{Q}{A_2} = \frac{35gpm \frac{1\,ft^3/\sec}{449gpm}}{\left(\frac{\pi(0.5in)^2}{4}\right)\left(\frac{1\,ft^2}{144in^2}\right)} = 57.4\,ft/\sec$$

Example for Head Added by Pump

- Substituting into Bernoulli's Equation:

$$h_A = \frac{P_2 - P_1}{\gamma} + \frac{V_2^2 - V_1^2}{2g} + Z_2 - Z_1 + h_f$$

$$h_A = \frac{0}{60.6lbf/ft_3} + \frac{(57.4\,ft/\sec)^2 - 0}{2(32.2\,ft/\sec^2)} + 10\,ft - 0\,ft + 26.7\,ft$$

$$h_A = 87.9\,ft$$

Calculation of TDH from Pump Test Data

$$TDH = H_L + H_F + H_V$$

Where H_L = total static head (difference between elevation of pumping source and point of delivery)

H_F = friction losses in pump, pipes and fittings

H_V = velocity head due to pumping

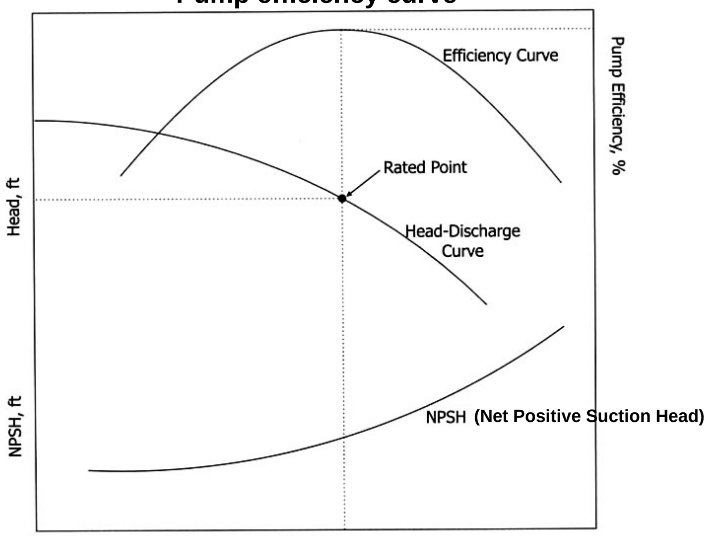
Calculation of TDH from Pump Test Data

- Substituting terms from Bernoulli's Equation for Velocity Head:

$$TDH = H_L + H_F + \frac{V^2}{2g}$$

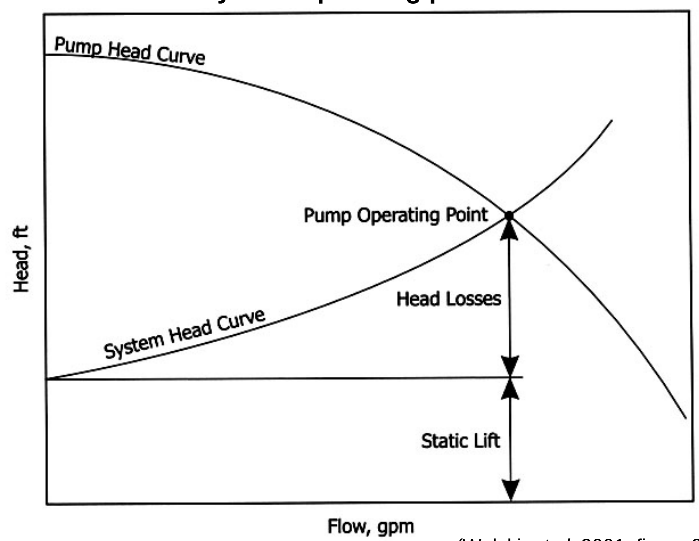
- Can plot system head (total dynamic head) versus discharge. TDH may not be constant because of differences between elevations during pumping (depleting supply and adding to storage). TDH also may not be constant because discharge rate will affect friction losses (as well as the velocity head term).

Pump efficiency curve

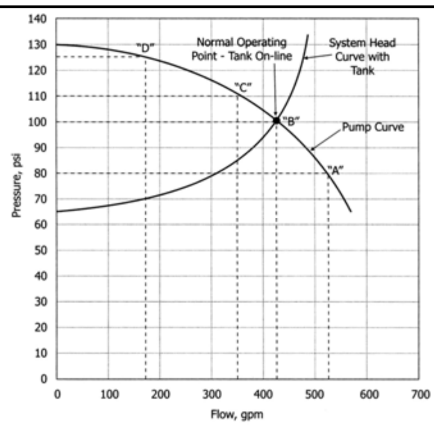


(Walski, *et al.* 2001 figure 2.18)

System operating point



(Walski, *et al.* 2001 figure 2.17)



Point	Demand	Pressure (psi)	Flow (gpm)
A	Peak demand	80	520
B	Normal operating point with tank	100	420
C	Average demand	110	350
D	Night-time demand	125	180

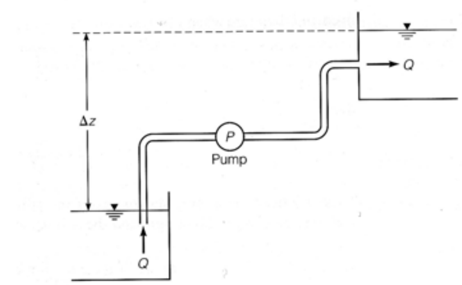
Effect of a change in demand on a constant-speed pump in a closed system

(Walski, *et al*. 2001 figure 9.11 and table 9.1)

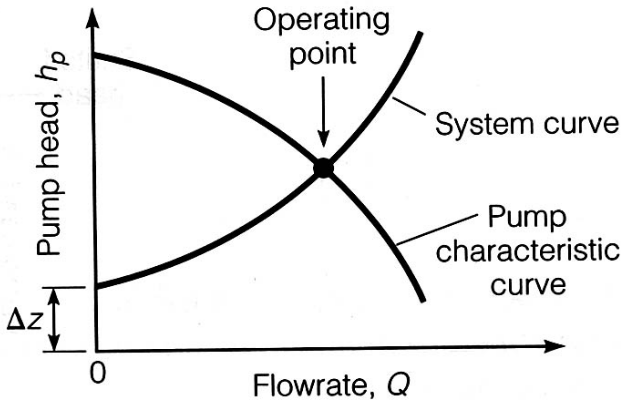
Example 3.12 (Chin 2000)

Water is pumped from a lower reservoir to an upper reservoir through a pipeline system as shown below. The reservoirs differ in elevation by 15.2 m and the length of the steel pipe (k_s = 0.046m) connecting the reservoirs is 21.3. The pipe is 50 mm in diameter and the performance curve is given by: $h_p = 24.4 - 7.65Q^2$

where hp (the pump head) is in meters and Q is in L/s



Operating point in pipeline system (Example 3.12)



(Chin 2006 Figure 2.18)

Using this pump, what flow do you expect in the pipeline? If the motor on the pump rotates at 2,400 rpm, calculate the specific speed of the pump in US Customary units and state the type of pump that should be used.

Neglecting minor losses, the energy equation for the pipeline system is:

$$h_p = \Delta z + Q\left[\sum \frac{fL}{2gA^2 D}\right] = 15.2m + \frac{fL}{2gA^2 D}Q^2$$

Where h_p is the head added by the pump, f is the friction factor, L is the pipe length, A is the cross-sectional areas of the pipe, and D is the diameter of the pipe.

$$A = \frac{\pi}{4}D^2 = \frac{\pi}{4}(0.05m)^2 = 0.00196m^2$$

In general, f is a function of both the Reynolds number and the relative roughness. However, if fully turbulent, then the friction factor depends on the relative roughness according to:

$$\frac{1}{\sqrt{f}} = -2\,\log\left(\frac{\varepsilon}{3.7}\right)$$

and the relative roughness is given by:

$$relative\ roughness = \varepsilon = \frac{k_3}{D}$$

$$k_s = equivalent\ sand\ roughness = 0.046mm\ \ for\ this\ case$$

$$\varepsilon = \frac{0.046mm}{50mm} = 0.000920$$

$$\frac{1}{f} = -2\log\left(\frac{0.000920}{3.7}\right) = 7.21$$

$$f = 0.0192$$

Can also use the Moody diagram and read the friction factor by reading "straight across" from the roughness value if assuming turbulent flow.

Substituting this friction factor into the energy equation:

$$h_p = 15.2m + \frac{fL}{2gA^2 D}Q^2 = 15.2m + \frac{(0.0192)(21.3m)}{2(9.81m/s^2)(0.00196m^2)^2(0.05m)}Q^2$$

$$= 15.2 + 108500Q^2\ \ in\ units\ of\ \ m^3/s;\ \ for\ Q\ in\ L/\sec$$

$$= 15.2 + 0.109Q^2$$

Combining the system curve and the pump characteristic curve leads to:

$$15.2 + 0.109Q^2 = 24.4 - 7.65Q^2$$

$$Q = 1.09L/\sec$$

The next step is to verify if the flow in the pipeline was completely turbulent

$$V = \frac{Q}{A} = \frac{1.09x10^{-3} m^3/s}{0.00196 m^2} = 0.556 m/s$$

$$\mathrm{Re} = \frac{VD}{\nu} = \frac{(0.556 m/s)(0.05 m)}{(1.00x10^{-6} m^2/s)} = 2.78x10^4$$

The friction factor can now be re-calculated using the Jain equation:

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{\varepsilon}{3.7} + \frac{5.74}{\mathrm{Re}^{0.9}}\right] = -2\log\left[\frac{0.000920}{3.7} + \frac{5.74}{(2.78x10^4)^{0.9}}\right] = 6.17$$

$$and \quad f = 0.0263$$

Therefore the original approximation of the friction factor (0.0192) was in error and the flow calculations need to be repeated, leading to: $Q = 1.09 L/s$

$$h_p = 24.4 - 7.65Q^2 = 24.4 - 7.65(1.09 L/s)^2 = 15.3m$$

In U.S. Customary units:

$$Q = 1.09 L/s = 17.3 gpm$$
$$h_p = 15.3m = 50.2 ft$$
$$\omega = 2{,}400 rpm$$

The specific speed is given by:

$$N_s = \frac{\omega Q^{0.5}}{h_p^{0.75}} = \frac{(2400)(17.3)^{0.5}}{(50.2)^{0.75}} = 529$$

Type of pump	Range of specific speeds, n_s*	Typical flowrates (L/s)	Typical efficiencies (%)
Centrifugal	0.15–1.5 (400–4,000)	< 60	70–94
Mixed flow	1.5–3.7 (4,000–10,000)	60–300	90–94
Axial flow	3.7–5.5 (10,000–15,000)	> 300	84–90

*The specific speeds in parentheses correspond to N_s given by Equation 3.112, with ω in rpm, Q in gpm, and h_p in ft.

Table 2.3 Chin 2006

Therefore, with a specific speed of 529 and the low flowrate, the best type of pump for these conditions is a centrifugal pump.

Cavitation

If the absolute pressure on the suction side of a pump falls below the saturation vapor pressure of the fluid, the water will begin to vaporize, this is called cavitation. This causes localized high velocity jets than can cause damage to the pump through pitting of the metal casing and impeller, reducing pump efficiency and causing excessive vibration. This sounds like gravel going through a centrifugal pump.

Example 3.13 (Chin 2000)

A pump with a performance curve of $h_p = 12-0.01Q^2$ (h_p in m, Q in L/s) is 3 m above a water reservoir and pumps water at 24.5 L/s through a 102 mm diameter ductile iron pipe (k_s = 0.26 mm). If the length of the pipeline is 3.5 m, calculate the cavitation parameter of the pump. If the specific speed of the pump is 0.94, estimate the critical value of the cavitation parameter and the maximum height above the water surface of the reservoir that the pump can be located.

The cavitation parameter is defined by: $\sigma = \frac{Net\ Postive\ Suction\ Head}{head\ added\ by\ the\ pump} = \frac{NPSH}{h_p}$

Where NPSH (net positive suction head) is: $NPSH = \frac{p_o}{\gamma} - \Delta z_s - h_L - \frac{p_v}{\gamma}$

where:

p_o = atmospheric pressure, 101 kPa

γ = specific weight of water, 9.79 kN/m^3

Δz_s = suction lift, 3 m

p_v = saturated vapor pressure of water at 20°C, 2.34 kPa

The head loss can be estimated using the Darcy-Weisbach and minor loss coefficients. The flow rate and Reynolds numbers are:

$$V = \frac{Q}{A} = \frac{0.0245 m^3/s}{\frac{\pi}{4}(0.102m)^2} = 3.0 m/s$$

$$\mathrm{Re} = \frac{VD}{\nu} = \frac{(3m/s)(0.102m)}{1.00x10^{-6} m^2/s} = 3.06x10^5$$

The Colebrook equation gives a friction factor, *f*, of 0.0257 using these values.

The head loss, h_L in the pipeline between the reservoir and the section side of the pump is:

$$h_L = \left(1 + \frac{fL}{D}\right)\frac{V^2}{2g} = \left[1 + \frac{(0.0257)(3.5m)}{(0.102m)}\right]\frac{(3.0m)^2}{2(9.81m/s^2)} = 0.863m$$

The NPSH can now be calculated:

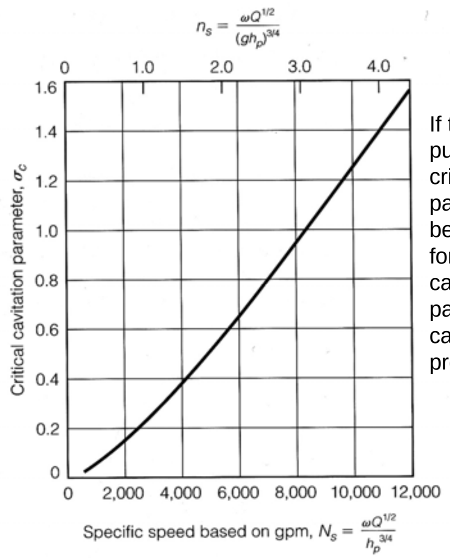
$$NPSH = \frac{p_o}{\gamma} - \Delta z_s - h_L - \frac{p_v}{\gamma} = \frac{101kPa}{9.79kN/m^3} - 3m - 0.863m - \frac{2.34kPa}{9.79kN/m^3} = 6.21m$$

The head added by the pump, h_p, can be calculated from the pump performance curve and the given flow rate:

$$h_p = 12 - 0.01Q^2 = 12 - 0.01(24.5L/s)^2 = 6m$$

The cavitation parameter of the pump is therefore:

$$\sigma = \frac{NPSH}{h_p} = \frac{6.21m}{6m} = 1.04$$



Chin 2000, Figure 3.18

If the specific speed of the pump is 0.94, then the critical cavitation parameter is estimated to be 0.2 (using the top scale for SI units). Since the calculated cavitation parameter was 1.04, cavitation should not be a problem.

When cavitation is imminent, the cavitation parameter is equal to 0.20 and the resultant NPSH is:

$$NPSH = \sigma_c h_p = 0.20(6m) = 1.2m$$

$$1.2m = \frac{p_o}{\gamma} - \Delta z_s - h_L - \frac{p_v}{\gamma}$$

The head loses between the pump and reservoir is estimated as:

$$h_L = \left[1 + \frac{f(z_s + 0.5)}{D}\right]\frac{V^2}{2g}$$

$$h_L = \left[1 + \frac{(0.0257)(z_s + 0.5)}{0.102m}\right]\frac{(3m)^2}{2(9.81m/s^2)} = 0.517 + 0.116z_s$$

Combining the equations results in:

$$1.2m = \frac{p_o}{\gamma} - z_s - h_L - \frac{p_v}{\gamma}$$

$$= \frac{101kPa}{9.79kN/m^3} - z_s - (0.517 + 0.116z_s) - \frac{2.34kPa}{9.79kN/m^3}$$

$$= 9.56m - 1.116z_s$$

and solving for z_s:

$$z_s = 7.49m$$

Therefore, the pump should be located no higher than 7.49 m (about 25 ft) above the water surface in the reservoir to prevent cavitation (this must always be less than 1 atm, or about 33 ft).

Calculation of the Theoretical Required Power of a Pump

Power (hp) = Qγ(TDH)/550

Where Q = discharge (ft^3/sec)
γ = specific weight of water (at sea level, 62.4 lb_f/ft^3)
TDH = total dynamic head (ft)
550 = conversion factor from ft-lb_f/sec to horsepower

- Each pump has its own characteristics relative to power requirements, efficient and head developed as a function of flow rate. These are usually given on pump curves for each pump. In general, the efficient for centrifugal pumps is between 50 and 85%, with pump efficiency generally increasing with the size and capacity of the pump.

Calculation for Pump Efficiency

$$\eta_P = \frac{P}{BHP} * 100$$

where η_P = pump efficiency
P = fluid power
BHP = brake horsepower actually delivered to the pump

Pump Power and Efficiency

Example:

- A pump is being used to deliver 35 gpm of hot water from a tank through 50 feet of 1-inch diameter smooth pipe, exiting through a ½-inch nozzle 10 feet above the level of the tank. The head loss from friction in the pipe is 26.7 feet. The specific weight of the hot water is 60.6 lb_f/ft^3. What is the power delivered to the water by the pump? If the efficiency is 60%, calculate the power delivered to the pump.

Pump Power and Efficiency

$$Power(hp) = \frac{Q\gamma(TDH)}{550}$$

- Convert the discharge to cfs.

$$Q = 35gpm\left(\frac{1cfs}{449gpm}\right) = 0.078\ ft^3/\sec$$

Pump Power and Efficiency

- Substituting back into Power equation:
  - TDH was calculated to be 87.9 ft.

$$Power(hp) = \frac{Q\gamma(TDH)}{550}$$

$$Power(hp) = \frac{(0.078\ ft^3/\sec)(62.4 lb_f / ft^3)(87.9\ ft)}{550\ ft - lb_f / hp}$$

$$Power(hp) = 0.755hp$$

Pump Power and Efficiency

- Efficiency calculation:

$$BHP = {P*100}/{\eta_P}$$

$$BHP = \frac{(0.755hp)*100}{60}$$

$$BHP = 1.26hp$$

Generating Pump Curves

Example:

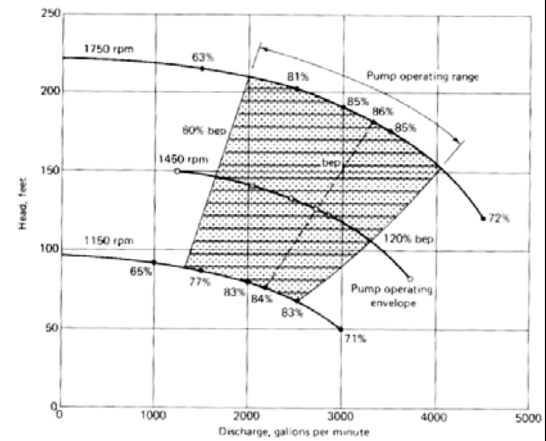
- The characteristics of a centrifugal pump operating at two different speeds are listed below. Graph these curves and connect the best efficiency points. Calculate the head-discharge values for an operating speed of 1450 rpm and plot the curve. Sketch the pump operating envelope between 60 and 120 percent of the best efficiency points.

Generating Pump Curves

Speed = 1750 rpm			Speed = 1150 rpm		
Discharge (gpm)	Head (ft)	Efficiency (%)	Discharge	Head (ft)	Efficiency (%)
0	220	NA	0	96	NA
1500	216	63	1000	93	65
2500	203	81	1500	89	77
3000	192	85	2000	82	83
3300	182	86	2200	77	84
3500	176	85	2500	70	83
4500	120	72	3000	49	71

Generating Pump Curves

Figure 4-12 Characteristic pump curves for Example 4-7 showing the pump operating envelope.



Pumps in Series and Parallel

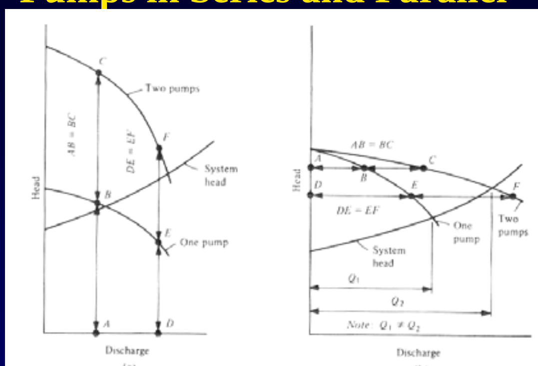
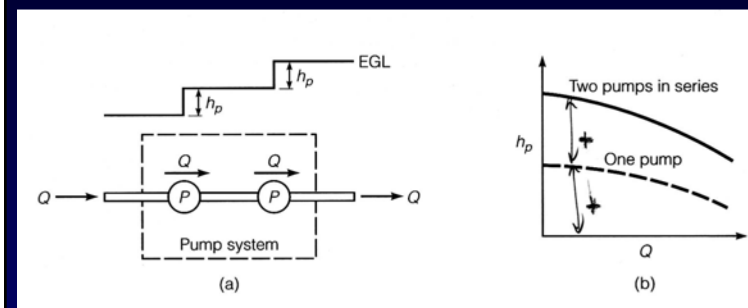


Figure 5.34 Characteristic curves for (a) Series. (b) Parallel pump operations of equal pumps.

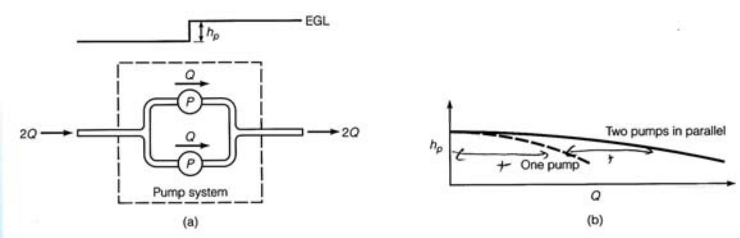
For series operation at a given capacity/discharge, the total head equals the sum of the heads added by each pump.
For parallel operation, the total discharge is multiplied by the number of pumps for a given head.

Pumps in series



(Chin 2006 Figure 2.20)

Pumps in parallel



(Chin 20060 Figure 2.21)

Multiple Pump Example 2.13 (Chin 2006)

A pump performance curve is: $h_p = 12 - 0.1Q^2$

What is the pump performance curve for a system having three of these pumps in series and a system having three of these pumps in parallel"

Pumps in Series:
For the pump in series, the same flow (Q) goes through each pump and each pump adds one-third of the total head, H_p:

$$\frac{H_p}{3} = 12 - 0.1Q^2$$

The characteristic curve of the pump system is therefore:

$$H_p = 36 - 0.3Q^2$$

Pumps in Parallel:

For a system consisting of three pumps in parallel, one-third of the total flow, Q, goes through each pump, and the head added by each pump is the same as the total head, H_p added by the pump system:

$$H_p = 12 - 0.1\left(\frac{Q}{3}\right)^2$$

and the characteristic curve of the pump system is therefore

$$H_p = 12 - 0.011Q^2$$

Variable Speed Pumps

- Equations for Variable Speed Pumps:

$$\frac{Q_1}{Q_2} = \frac{N_1}{N_2}$$

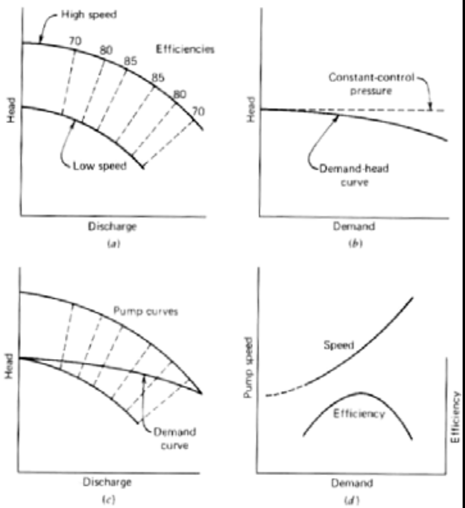
$$\frac{H_1}{H_2} = \frac{N_1{}^2}{N_2 2}$$

$$\frac{P_{i1}}{P_{i2}} = \frac{N_1{}^3}{N_2{}^3}$$

where Q = Pump discharge
H = head (total discharge/dynamic head)
P = power input
N = pump speed (revolutions/time – typically in minutes)

Variable Speed Pumps

Figure 4-17 Characteristic curves for a variable-speed pump. (*a*) Head-discharge curves with efficiency values for a pump operating at two impeller speeds. (*b*) System demand-head curve for a constant pressure discharge corrected for transducer operation. (*c*) Superimposed pump head-discharge and demand curves. (*d*) Curves of speed and efficiency versus demand.



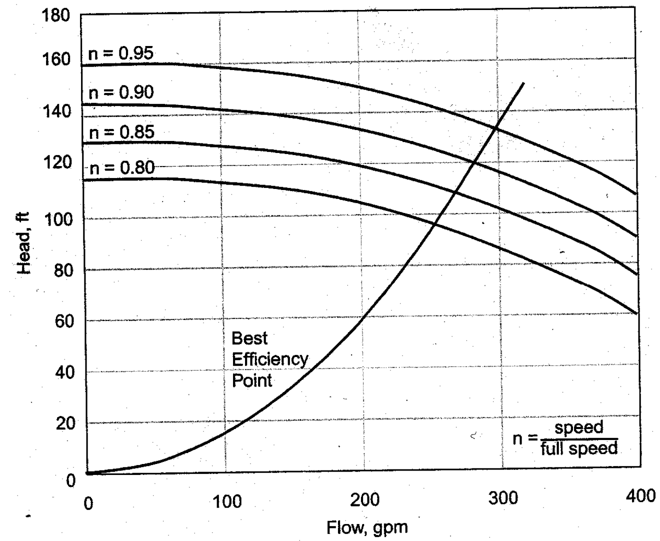


Figure 6-4: Relative speed factors for variable-speed pumps

Pump Selection Example

- A pump station is to be designed for an ultimate capacity of 1200 gpm at a total head of 80 ft. The present requirements are that the station deliver 750 gpm at a total head of 60 ft. One pump will be required as a standby.

Pump Selection Example

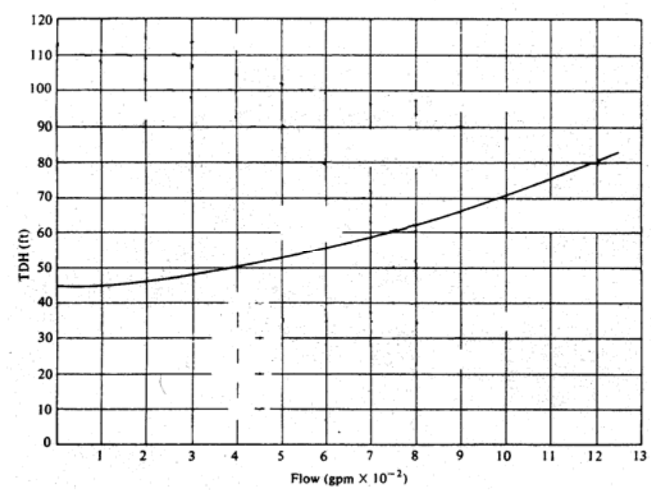


Figure 5.35 Solution to Example 5.8.

Pump Selection Example

- Possibilities are Pump A and/or Pump B.

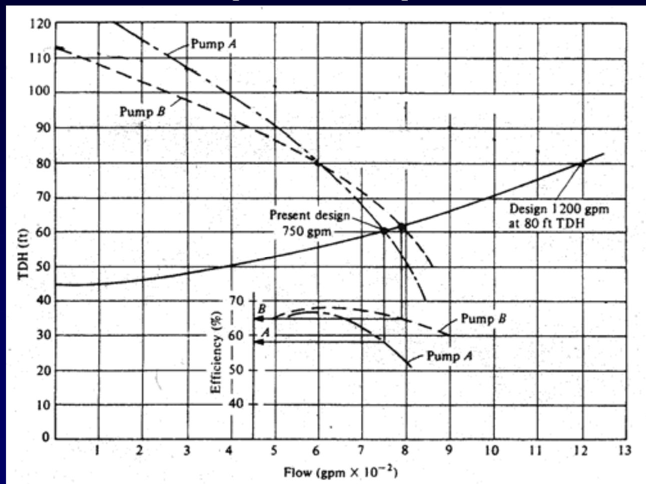


Figure 5.35 Solution to Example 5.8.

Pump Selection Example

- One pump will not supply the needed head at future conditions. Pump B is better choice.
- Therefore, need to look at the three conditions imposed on this design and see if B can supply.
  - Two pumps must produce 1200 gpm at 80 ft TDH.
  - One pump must produce 600 gpm at 80 ft TDH.
  - One pump must meet present requirement of 750 gpm at 60 ft TDH.

Pump Selection Example

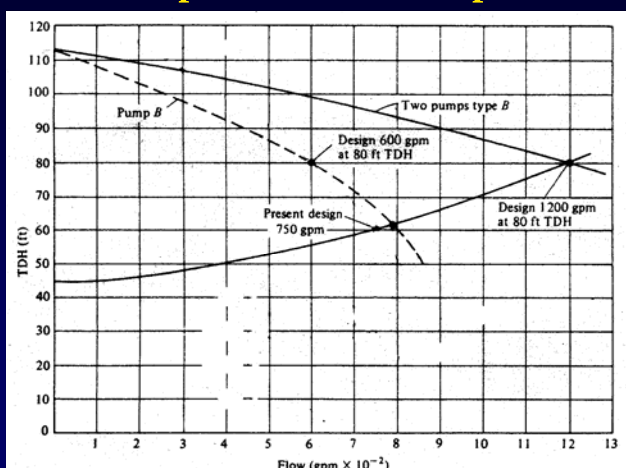


Figure 5.35 Solution to Example 5.8.

Pumping Stations for Sewage

Constant-speed pumps should not be turned on and off too frequently since this can cause them to overheat. In small pumping stations there may be only two pumps, each of which must be able to deliver the maximum anticipated flow. Lower flows accumulated in the wet well until a sufficient volume has accumulated to run the pump for short period (the run time). The wet well may also be sized to ensure that the pump will not start more often than a specified time period (the cycle time).

Example 15-3 (McGhee 1991)

A small subdivision produces an average wastewater flow of 120,000 L/day. The minimum flow is estimated to be 15,000 L/day and the maximum 420,000 L/day. Using a 2-minute running time and a 5-minute cycle time, determine the design capacity of each of two pumps and the required wet well volume.

For a 2-minute running time:

$$V = \frac{2\min}{1440\min/day}(420{,}000L/day - 15{,}000L/day) = 562.5L$$

The cycle time will be shortest when Q_{in} is 0.5 Q_{out}. Therefore, for a minimum 5 minute cycle time:

$$5\min = \frac{V}{Q_{out} - 0.5Q_{out}} + \frac{V}{0.5Q_{out}}$$

$$V = \frac{5\min}{1440\min/day} x \frac{0.5(420{,}000L/day)}{2} = 365L$$

The required volume is determined by the 2-minute running time requirement in this example and will therefore be larger than 563L.

Each pump must be able to deliver the peak flow of 420,000 L/day.

The pump running time is the working volume of the wet well divided by the net discharge, which is the pumping rate minus the inflow:

$$t_r = \frac{V}{Q_{out} - Q_{in}}$$

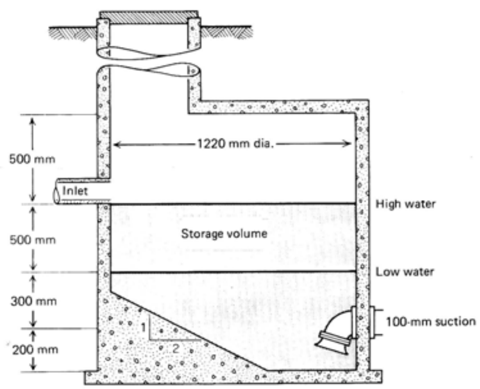
The filling time with the pump off is:

$$t_f = \frac{V}{Q_{in}}$$

The total cycle time is therefore:

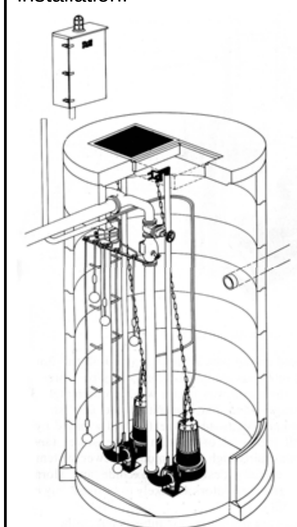
$$t_c = t_r + t_f = \frac{V}{Q_{out} - Q_{in}} + \frac{V}{Q_{in}}$$

A minimum depth needs to be maintained over the pump suction. With an intake velocity of about 0.6 m/sec (2 ft/sec), a submergence of about 300 mm (1 ft) is needed. It is also common to provide additional freeboard (about 600 mm, or 2 ft) above the maximum water level. This is a suitable wet well for this example:

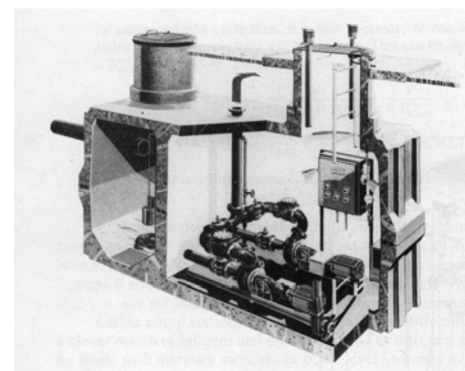


McGhee 1991, Figure 15-13.

Submersible sewage pump installation:



Wet pit-dry pit sewage pumping station:



McGhee 1991, Figures 15-14 and 15-15