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

---

**Design Periods and Capacities in Water-Supply Systems**

(Table 3.11 Chin 2000)

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

**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**

---

**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

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

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

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Hourly Demand (gpm)</th>
<th>Hourly Demand (gal)</th>
<th>Cumulative Hourly Demand (gal)</th>
<th>Surplus/Deficit (gal)</th>
<th>Deficits (gal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0600 - 0700</td>
<td>3.630</td>
<td>217,800</td>
<td>957,000</td>
<td>68,408</td>
<td>surplus</td>
</tr>
<tr>
<td>0700 - 0800</td>
<td>5.190</td>
<td>311,400</td>
<td>1,268,400</td>
<td>-25,193</td>
<td>-25,193</td>
</tr>
<tr>
<td>0800 - 0900</td>
<td>5.620</td>
<td>337,200</td>
<td>1,605,600</td>
<td>-50,993</td>
<td>-50,993</td>
</tr>
<tr>
<td>0900 - 1000</td>
<td>5.900</td>
<td>354,000</td>
<td>1,959,600</td>
<td>-67,793</td>
<td>-67,793</td>
</tr>
<tr>
<td>1000 - 1100</td>
<td>6.040</td>
<td>362,400</td>
<td>2,322,000</td>
<td>-76,193</td>
<td>-76,193</td>
</tr>
<tr>
<td>1100 - 1200</td>
<td>6.320</td>
<td>379,200</td>
<td>2,701,200</td>
<td>-92,993</td>
<td>-92,993</td>
</tr>
<tr>
<td>1200 - 1300</td>
<td>6.440</td>
<td>386,400</td>
<td>3,087,600</td>
<td>-100,193</td>
<td>-100,193</td>
</tr>
</tbody>
</table>

#### Surplus/Deficit

- **Average**
  - Sum of deficits = 1,465,275 gallons

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

---

### Determining Required Storage Amount: Example

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Hourly Demand (gpm)</th>
<th>Hourly Demand (gal)</th>
<th>Cumulative Hourly Demand (gal)</th>
<th>Surplus/Deficit (gal)</th>
<th>Deficits (gal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1300 - 1400</td>
<td>6.370</td>
<td>382,200</td>
<td>3,469,800</td>
<td>-95,993</td>
<td>-95,993</td>
</tr>
<tr>
<td>1400 - 1500</td>
<td>6.320</td>
<td>379,200</td>
<td>3,849,000</td>
<td>-92,993</td>
<td>-92,993</td>
</tr>
<tr>
<td>1600 - 1700</td>
<td>6.640</td>
<td>398,400</td>
<td>4,627,800</td>
<td>-112,193</td>
<td>-112,193</td>
</tr>
<tr>
<td>1700 - 1800</td>
<td>7.320</td>
<td>439,200</td>
<td>5,067,000</td>
<td>-152,993</td>
<td>-152,993</td>
</tr>
<tr>
<td>1800 - 1900</td>
<td>9.333</td>
<td>559,980</td>
<td>5,626,980</td>
<td>-273,773</td>
<td>-273,773</td>
</tr>
<tr>
<td>1900 - 2000</td>
<td>8.320</td>
<td>499,200</td>
<td>6,126,180</td>
<td>-212,993</td>
<td>-212,993</td>
</tr>
</tbody>
</table>

#### Surplus/Deficit

- **Average**
  - Sum of deficits = 1,465,275 gallons

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

---

**Chin 2000, Figure 3.24**
Example 3.18 (Chin 2000)

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:

\[ Q_m = 1.8(600 \text{ L/day/capita})(250,000 \text{ people}) = 2.7 \times 10^8 \text{ L/day} = 2.7 \times 10^4 \text{ m}^3/\text{day} \]

The corresponding volume is therefore:

\[ V_{peak} = 0.25(2.7 \times 10^4 \text{ m}^3/\text{day}) = 67,500 \text{ m}^3 \]

(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:

\[ V_{fire} = \left(0.62 \text{ m}^3/\text{sec}\right)(9 \text{ hours})(3,600 \text{ sec/hr}) = 20,100 \text{ m}^3 \]

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

\[ V_{emer} = \left(250,000 \text{ people}\right)(600 \text{ L/day/capita}) = 150 \times 10^6 \text{ L} = 150,000 \text{ m}^3 \]

The required volume of the service reservoir is therefore:

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

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 3.19 (Chin 2000)

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 \( z_0 \) of the hydraulic grade line (HGL) at the reservoir location is:

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

where:

- \( p_{min} = 300 \text{ kPa} \)
- \( \gamma = 9.79 \text{ kN/m}^3 \)
- \( z_{min} = 5.4 \text{ m} \)
- \( h_L = 10 \text{ m} \)

Therefore, under average conditions:

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

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

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

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

<table>
<thead>
<tr>
<th>Type of pump</th>
<th>Range of specific speeds, $n_s^*$</th>
<th>Typical flowrates, (L/s)</th>
<th>Typical efficiencies, (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Centrifugal</td>
<td>0.15–1.5 (400–4,000)</td>
<td>&lt; 60</td>
<td>70–94</td>
</tr>
<tr>
<td>Mixed flow</td>
<td>1.5–3.7 (4,000–10,000)</td>
<td>60–300</td>
<td>90–94</td>
</tr>
<tr>
<td>Axial flow</td>
<td>3.7–5.5 (10,000–15,000)</td>
<td>&gt; 300</td>
<td>84–90</td>
</tr>
</tbody>
</table>

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

(Chin 2000 Table 3.3)

### Schematic of hydraulic grade line for a pumped system

(Walski, et al. 2001 figure 2.16)

### Pump effect on flow in pipeline

(Chin 2000 Figure 3.6)
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

- 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
- \( \gamma \) = 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.

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 1/2-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/ft³.

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 $V_2$.

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

Calculation of TDH from Pump Test Data

$$\text{TDH} = H_L + H_F + H_V$$

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

Example for Head Added by Pump

- Substituting into Bernoulli’s Equation:

$$h_x = \frac{P_x - P_y}{\gamma} + \frac{V_x^2 - V_y^2}{2g} + z_x - z_y + h_f$$

$$h_x = \frac{0}{60.6 \text{ lbf/ft}^3} + \frac{(57.4 \text{ ft/sec})^2 - 0}{2(32.2 \text{ ft/sec}^2)} + 10 \text{ ft} - 0 \text{ ft} + 26.7 \text{ ft}$$

$$h_x = 87.9 \text{ ft}$$

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).
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 m. The pipe is 50 mm in diameter and the performance curve is given by:

$$h_p = 24.4 - 7.65 Q^2$$

where $h_p$ (the pump head) is in meters and $Q$ is in L/s.

<table>
<thead>
<tr>
<th>Point</th>
<th>Demand</th>
<th>Pressure (psi)</th>
<th>Flow (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Peak demand</td>
<td>80</td>
<td>520</td>
</tr>
<tr>
<td>B</td>
<td>Normal operating point with tank</td>
<td>100</td>
<td>420</td>
</tr>
<tr>
<td>C</td>
<td>Average demand</td>
<td>110</td>
<td>350</td>
</tr>
<tr>
<td>D</td>
<td>Night-time demand</td>
<td>125</td>
<td>180</td>
</tr>
</tbody>
</table>

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

\[ h_p = \Delta z + Q \left[ \sum_{i} \frac{f L}{2 g A^2 D} \right] = 15.2 m + \frac{f L}{2 g A^2 D} Q^2 \]

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.

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

Substituting this friction factor into the energy equation:

\[ h_p = 15.2 m + \frac{f L}{2 g A^2 D} Q^2 = 15.2 m + \frac{(0.0192) (21.3 m)}{2 (0.81 m/s^2) (0.00196 m^2)(0.05 m)} Q^2 \]

\[ = 15.2 + 10850 Q^2 \text{ in units of } m^3/s; \text{ for } Q \text{ in } L/sec \]

\[ = 15.2 + 0.109 Q^2 \]

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

\[ Q = 1.09 L/sec \]

The next step is to verify if the flow in the pipeline was completely turbulent.
The friction factor can now be re-calculated using the Jain equation:

\[
\frac{1}{\sqrt{f}} = -2 \log \left[ \frac{\varepsilon}{3.7} + \frac{5.74}{\text{Re}^{0.9}} \right] = -2 \log \left[ \frac{0.000920}{3.7} + \frac{5.74}{(2.78 \times 10^4)^{0.9}} \right] = 6.17
\]

and \( f = 0.0263 \)

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

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.

The cavitation parameter is defined by:

\[
\sigma = \frac{\text{Net Positive Suction Head}}{h_p} = \frac{\text{Head added by the pump}}{h_p}
\]

Where NPSH (net positive suction head) is:

\[
NPSH = \frac{P_r - \Delta z - h_L - P_s}{g}
\]

In U.S. Customary units:

\[
Q = 1.09L/s = 17.3\text{gpm}
\]

\( h_p = 15.3m = 50.2\text{ft} \)

\( \omega = 2,400\text{rpm} \)

The specific speed is given by:

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

In Imperial units:

\[
Q = 1.09L/s = 17.3\text{gpm}
\]

\( h_p = 15.3m = 50.2\text{ft} \)

\( \omega = 2,400\text{rpm} \)

The specific speed is given by:

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

Table 3.3 Chin 2000

<table>
<thead>
<tr>
<th>Type of pump</th>
<th>Range of specific speeds, ( n_s )*</th>
<th>Typical flowrates (L/s)</th>
<th>Typical efficiencies (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Centrifugal</td>
<td>0.15–1.5 (400–4000)</td>
<td>&lt; 60</td>
<td>70–94</td>
</tr>
<tr>
<td>Mixed flow</td>
<td>1.5–3.7 (4000–10,000)</td>
<td>60–300</td>
<td>90–94</td>
</tr>
<tr>
<td>Axial flow</td>
<td>3.7–5.5 (10,000–15,000)</td>
<td>&gt; 300</td>
<td>84–90</td>
</tr>
</tbody>
</table>

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

Therefore, with a specific speed of 529 and the low flowrate, the best type of pump for these conditions is a centrifugal pump.
where:
\( p_0 = \text{atmospheric pressure, } 101kPa \)
\( \gamma = \text{specific weight of water, } 9.79kN/m^3 \)
\( \Delta z_s = \text{suction lift, } 3m \)
\( p_v = \text{saturated vapor pressure of water at } 20^\circ \text{C, } 2.34kPa \)

The head loss can be estimated using the Darcy-Weisbach and minor loss coefficients. The flow rate and Reynolds numbers are:
\[
V = \frac{Q}{A} = \frac{0.0245m^3/s}{\pi (0.102m)^2} = 3.0m/s
\]
\[
Re = \frac{VD}{\nu} = \frac{(3m/s)(0.102m)}{1.00 \times 10^{-6} m^2/s} = 3.06 \times 10^5
\]

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:
\[
h_L = \left(1 + \frac{fL}{D}\right) \frac{V^2}{2g} = \left[1 + \frac{(0.0257)(3.5m)}{(0.102m)}\right] \frac{(3m)^2}{2(9.8m/s^2)} = 0.863m
\]

The NPSH can now be calculated:
\[
\text{NPSH} = \frac{P_x - \Delta z} - \frac{P_x}{\gamma} - \frac{h_L}{\gamma} = \frac{101kPa - 9.79kN/m^3 - 3m - 0.863m - 2.34kPa}{9.79kN/m^3} = 6.21m
\]

The head added by the pump, \( h_p \), can be calculated from the pump performance curve and the given flow rate:
\[
h_p = 12 - 0.01Q^2 = 12 - 0.01(24.5 L/s)^2 = 6m
\]

The cavitation parameter of the pump is therefore:
\[
\sigma = \frac{NPSH}{h_p} = \frac{6.21m}{6m} = 1.04
\]

When cavitation is imminent, the cavitation parameter is equal to 0.20 and the resultant NPSH is:
\[
\text{NPSH} = \sigma_h_p = 0.20(6m) = 1.2m
\]

\[
1.2m = \frac{P_x - \Delta z} - \frac{P_x}{\gamma} - \frac{h_L}{\gamma}
\]

The head loses between the pump and reservoir is estimated as:
\[
h_L = \left[1 + \frac{f(z_s + 0.5)}{D}\right] \frac{V^2}{2g}
\]
\[
h_L = \left[1 + \frac{(0.0257)(z_s + 0.5)}{0.102m}\right] \frac{(3m)^2}{2(9.8m/s^2)} = 0.517 + 0.116z_s
\]
Combining the equations results in:

\[ 1.2m = \frac{P_s}{\gamma} - z_s - h_l - \frac{P_s}{\gamma} \]

\[ = \frac{101kPa}{9.79kN/m^2} - z_s - (0.517 + 0.116z_s) - \frac{2.34kPa}{9.79kN/m^2} \]

\[ = 9.56m - 1.116z_s \]

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) = \( Q\gamma(TDH)/550 \)

Where \( Q \) = discharge (ft\(^3\)/sec)

\( \gamma = \) specific weight of water (at sea level, 62.4 lb/ft\(^3\))

TDH = total dynamic head (ft)

550 = conversion factor from ft-lb/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**

\[ \eta_P = \frac{P}{BHP} \times 100 \]

where \( \eta_P = \) 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/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.

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

Pump Power and Efficiency

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

\[ Power(hp) = \frac{Q\gamma(TDH)}{550} \]
\[ Power(hp) = \left(0.078 \text{ft}^3/\text{sec}\right)\left(62.4 \text{lb}_f/\text{ft}^3\right)(87.9 \text{ft}) \]
\[ Power(hp) = 0.755hp \]

Pump Power and Efficiency

• Efficiency calculation:

\[ BHP = \frac{P \times 100}{\eta_p} \]
\[ BHP = \frac{(0.755hp) \times 100}{60} \]
\[ BHP = 1.26hp \]

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

<table>
<thead>
<tr>
<th>Speed = 1750 rpm</th>
<th>Speed = 1150 rpm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discharge (gpm)</td>
<td>Head (ft)</td>
</tr>
<tr>
<td>0</td>
<td>220</td>
</tr>
<tr>
<td>1500</td>
<td>216</td>
</tr>
<tr>
<td>2500</td>
<td>203</td>
</tr>
<tr>
<td>3000</td>
<td>192</td>
</tr>
<tr>
<td>3300</td>
<td>182</td>
</tr>
<tr>
<td>3500</td>
<td>176</td>
</tr>
<tr>
<td>4500</td>
<td>120</td>
</tr>
</tbody>
</table>

### Pumps in Series and Parallel

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 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

\[
H_p = 12 - 0.011Q^2
\]

Multiple Pump Example 3.14 (Chin 2000)

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:

\[
H_p = 36 - 0.3Q^2
\]

Variable Speed Pumps

• Equations for Variable Speed Pumps:

\[
\begin{align*}
Q_1 &= N_1 \\
Q_2 &= N_3 \\
H_1 &= N_1^2 \\
H_2 &= N_4^2 \\
P_1 &= N_1^3 \\
P_2 &= N_2^3
\end{align*}
\]

where \( Q = \) Pump discharge

\( H = \) head (total discharge/dynamic head)

\( P = \) power input

\( N = \) pump speed (revolutions/time – typically in minutes)
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

- Possibilities are Pump A and/or Pump B.

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.

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:

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

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

\[ 5 \text{ min} = \frac{V}{Q_{out} - 0.5 Q_{out}} + \frac{V}{0.5 Q_{out}} \]

\[ V = \frac{5 \text{ min} \times 0.5 (420,000 \text{ L/day})}{1440 \text{ min/day}} = 365 \text{ L} \]

The filling time with the pump off is:

\[ t_f = \frac{V}{Q_{in}} \]

The total cycle time is therefore:

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

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:

\[ t_r = \frac{V}{Q_{out} - Q_{in}} \]

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: