- We can classify water in supply network, wastewater network and storm network depend on using water to:

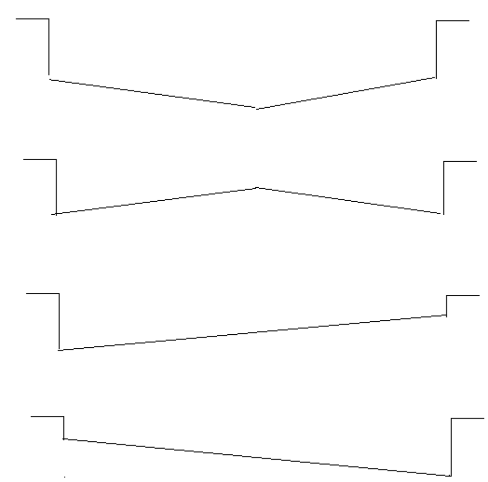
1. Supply network - - - unused
2. wastewater network - - - used
3. Storm network - - - unused

- We can classify water in supply network, wastewater network and storm network depend on pollution to:

1. Supply network - - - un pollution
2. wastewater network - - - high pollution (organic pollution and in organic pollution)
3. Storm network - - - pollution (inorganic)

- We can classify water in supply network, wastewater network and storm network depend on pressure to:

1. Supply network - - - under pressure
2. wastewater network - - - at atmosphere pressure
3. Storm network - - - at atmosphere pressure

<table>
<tr>
<td>❖ Cross-section Road type:
1. Concave
2. Roof
3. Planar (right or left)</td>
<td></td>
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Population

$$\frac{dP}{dt} = K \tag{2-1}$$

in which dP/dt is the rate of change of population with time and K is a constant. K is determined graphically or from consideration of actual populations in successive censuses as

$$K = \frac{\Delta P}{\Delta t} \tag{2-2}$$

The population in the future is then estimated from

$$P_t = P_0 + Kt \tag{2-3}$$

where P_t is the population at some time in the future, P_0 is the present population, and t is the period of the projection.

Ex: Designed at 2001, project lifetime is 30-years, Project operate at 2025

1940	1.50×10^6
1965	1.90×10^6
1981	2.31×10^6
2001	2.60×10^6

Solution:

Lecture 2 Sanitary content:

- Part One: Water Supply System
- Part Two: Waste Water System + storm

Introduction to Water Supply System: A branch of civil engineering concerned with the development of sources of supply, transmission, distribution, and treatment of water. The term is used most frequently in regard to municipal water works, but applies also to water systems for industry, irrigation, and other purposes. A complex of engineering works and measures aimed at catch, transportation, purification, storage and deliver of water to the consumers with necessary quantity, quality and pressure in a compliance with the best technical and economic practice.

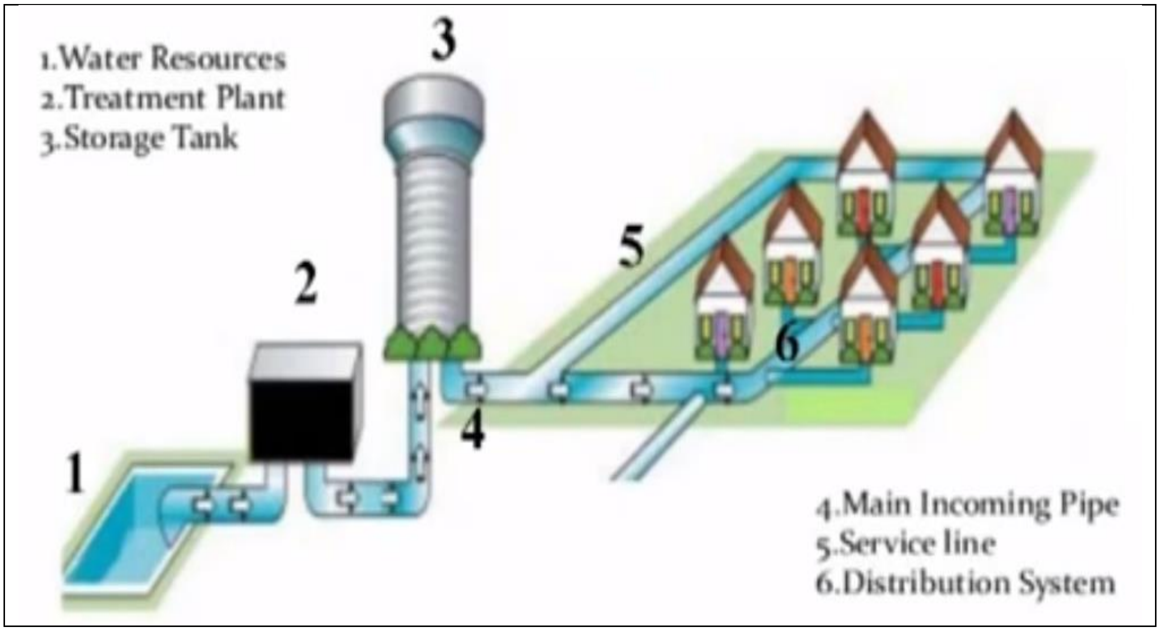
Water Supply System Divided to:

- Studies Required for Water Supply System (Required Capacity)
- Water Distribution System Design and Analysis
- Water Distribution System: Modeling and Computer Applications and
- Water Quality Characteristics and Water Treatment Systems

The objectives of protected water supply system are:

1- To supply safe water to consumers
2- To supply water in sufficient quantities
3- To supply water at convenient points and timings
4- To supply water at reasonable cost to the user.

Water Supply System Components /Works
Municipal water systems generally comprise
(a) Collection works (pump and intakes)
(b) Transmission works
(c) Purification works
(d) Distribution works



<u>Required Capacity Water supply systems:</u> are designed to meet population needs for a reasonable number of years in the future depend on the design period.

<u>Sources of Water Supply System</u> The various sources of water can be classified into two categories:

- Surface sources, such as
1- Ponds and lakes
2- Streams and rivers
3- Storage reservoirs
- Ground sources, such as
1- Springs
2- Wells and Tube-wells
- Reclamation Sources, Such as:
1- Desalination
2- Re-use of treated waste water

Design Criteria of water source:

- Sufficient Quantity
- Good Quality

Water Distribution systems:

- Distribution system
- Requirements of Good Distribution System
- Methods of Supplying Water
- Layout of Distribution System
- Types of Networks Systems
- Types of Supplies
- Distribution Reservoirs

Distribution system

To deliver water to individual consumers with appropriate quality, quantity, and pressure in a community setting requires an extensive system of.

- Pipes
- Storage reservoirs
- Pumps.
- Other related accessories

Requirements of Good Distribution System:

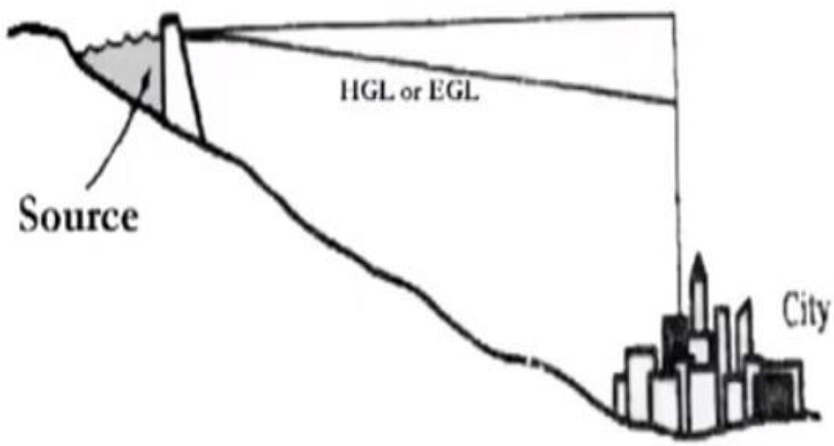
- Water quality should not get deteriorated in the distribution pipes
- It should be capable of supplying water at all the intended places with sufficient pressure
- It should be capable of supplying the requisite amount of water during fire fighting

Methods of Supplying Water:

- Depending on the topography relationship between the source of supply and the consumer, water can be transported by:
    1. Canals.
    2. Pipelines
- The most common methods are
    1. Gravity supply
    2. Pumped supply
    3. Combined supply

<u>Gravity Supply</u>

The source of supply is at a sufficient elevation above the distribution area (consumers). So that the desired pressure can be maintained.

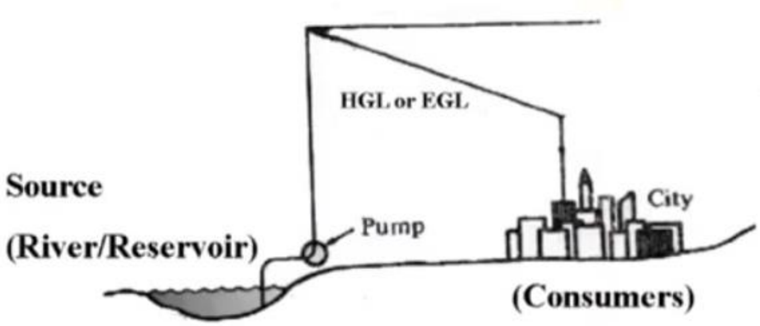


Advantages of Gravity Supply:

- No energy cost.
- Simple operation (fewer mechanical parts, independence of power supply, ….)
- Low maintenance costs
- No sudden pressure changes

<u>Pumped Supply:</u>

- Used whenever:
    1. The source of water is lower than the area to which we need to distribute water to (consumers).
    2. The source cannot maintain minimum pressure required.
- pumps are used to develop the necessary head (pressure) to distribute water to the consumer and storage reservoirs



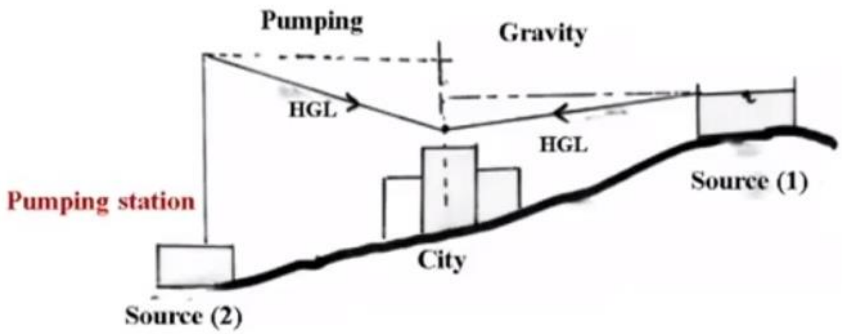
Disadvantages of pumped supply:

- Complicated operation and maintenance.
- Dependent on reliable power supply.
- Precautions have to be taken in order to enable permanent supply:
    1. Stock with spare parts
    2. Alternative source of power supply

Combined Supply: (pumped-storage supply)

- Both pumps and storage reservoirs are used.
- This system is usually used in the following cases:

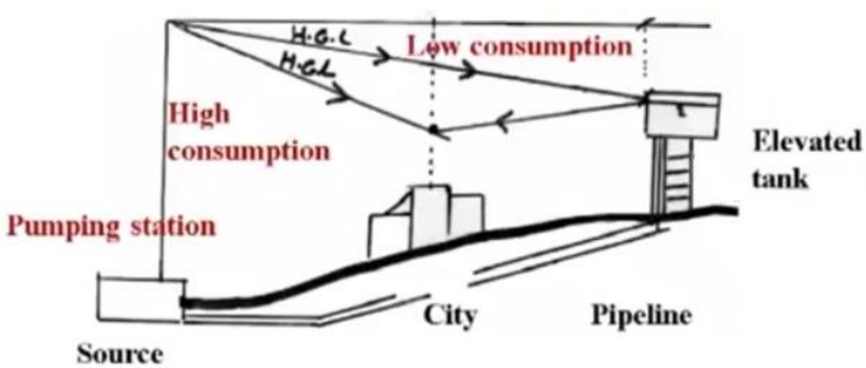
1) When two sources of water are used to supply water



Combined Supply

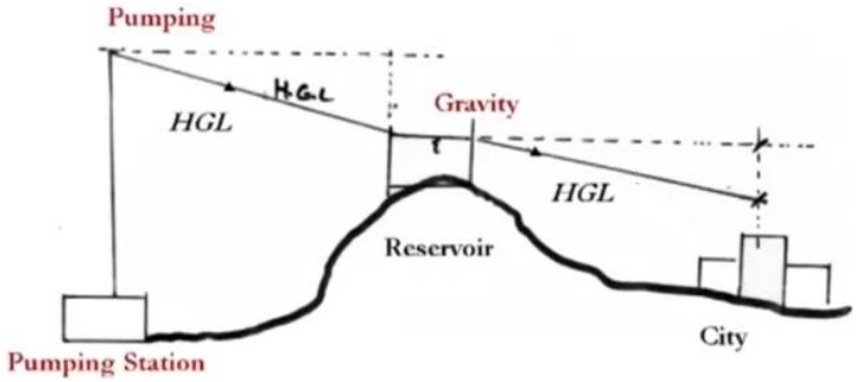
2) In the pumped system sometimes a storage (elevated) tank is connected to the System.

- When the water consumption is low, the residual water is pumped to the tank.



3) When the source is lower than the consumer area

- A tank is constructed above the highest point in the area.
- Then the water is pumped from the source to the storage tank (reservoir)
- And the water is distributed from the reservoir by gravity



Layout of Distribution System:

- The distribution pipes are generally laid below the road pavements, and as such their layouts generally follow the layouts of roads.
- Street plan, topography, and location of supply works, together with service storage, determine the type of distribution system.
- There are general, three different types of pipe networks; any one of which either single or in combinations, can be used for a particular place.

Distribution Systems (Network Layout): In laying the pipes through the distribution area, the_following configuration can be distinguished:

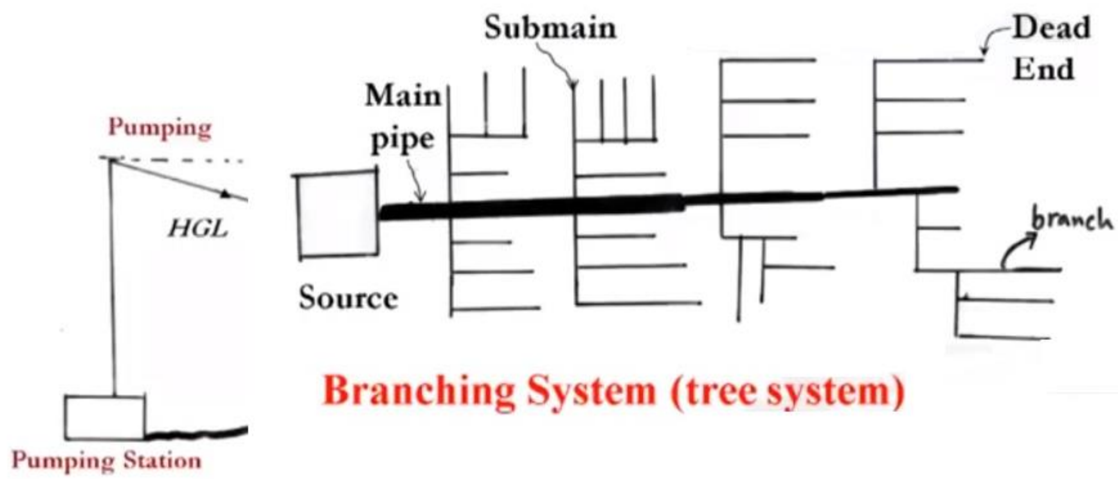
1. Branching system (Tree/dead end)
2. Grid system (Looped)
3. Combined system.

Branching System (tree system):
Advantages:

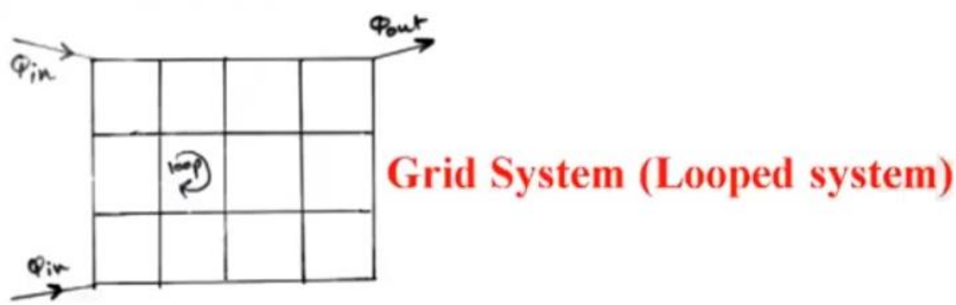
- Simple to design and build.
- Less expensive than other systems.

Disadvantages:

- The large number of dead ends which results in sedimentation and bacterial growths.
- When repairs must be made to an individual line, service connections beyond the point of repair will be without water until the repairs are made.
- The pressure at the end of the line may become undesirably low.



Grid System (Looped system):



Advantages:

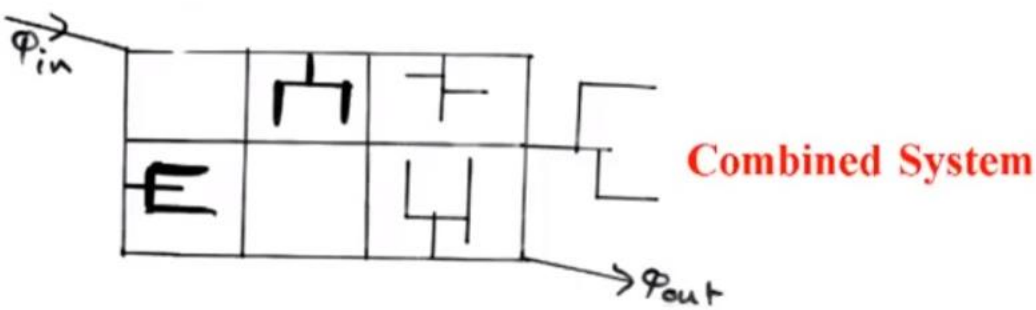
- The grid system overcomes all of the difficulties of the branching system discussed before.
- No dead ends. (All of the pipes are interconnected).
- Water can reach a given point of withdrawal from several directions.

Disadvantages:

- Hydraulically far more complicated than branching system (Determination of the pipe sizes is somewhat more complicated)
- Expensive (consists of a large number of loops).

- But, it is the most reliable and used system

Combined System:
It is a combination of both Grid and Branching systems
This type is widely used all over the world.



Types of Supplies (by time): Water may be supplied to the consumers by the flowing supplies types:

- Continues Supply: water is available 24 hours of the day
- Intermittent Supply: water is supplied only during some fixed hours of the day.

Distribution Reservoirs: Distribution reservoirs, also called service reservoirs, are the storage reservoirs, which store the treated water for supplying water during emergencies (such as during fires, repairs, etc.) and also to help in absorbing the hourly fluctuations in the normal water demand.

Function of distribution reservoirs:

- To absorb the hourly variations in demand.
- To maintain constant pressure in the distribution main.
- Water stored can be supplied during emergencies.

Location & Elevation Distribution Reservoirs:

- Volume and location of service storage depend on topography and water needs.
- Should be located as close as possible to the centre of demand.
- Water level in the reservoir must be at sufficient elevation to permit gravity flow at an adequate pressure.

Types Of Reservoirs: Depending upon the elevation with respect to ground,

It may be classified into:

- Surface reservoirs
- Elevated reservoirs

Surface reservoirs: These also called ground reservoir.

- Mostly circular or rectangular tanks.
- Underground reservoir is preferred especially when the size is large.
- These reservoirs are constructed high natural grounds and are usually made of stones, brick, plain or reinforced concrete cement.
- The side walls are designed to take up the pressure of the water, when the reservoir is full and the earth pressure, when it is empty.

Elevated reservoir:

- It is also referred to as overhead tanks are required at distribution areas which are not governed and controlled by the gravity system of distribution.
- These are rectangular, circular or elliptical in shape.
- They are constructed where combine gravity and pumping system of water distribution is adopted.
- These tanks may be steel or RCC.

Storage Capacity of Distribution Reservoirs: The total storage capacity of a distribution reservoir is the summation of:

Balancing Storage + Breakdown Storage + Fire Storage

Lecture** **4 ,5 & 6

Outlines for this Lecture:

- Topographical Studies
- Water Source Quality & Quantity
- Design Period
- Design Population
- Water Demand
- Water Treatment Plant Sizing & Location.
- Distribution System Design Requirements

Topographical Studies:

1. Contour lines Maps.
2. Digital maps showing present (and future) houses, streets, lots, and so on... to help identifying the suitable, type of networks distribution
3. Location of water sources to help locating distribution reservoirs & treatment plant.

Water Source Quality & Quantity:

- The quality of the raw (untreated) water plays a large role in determining the unit operations and processes required to treat the water
- Water quantity must be sufficient for the required water demand & project design period.

For Groundwater source (borehole/well) we should check the following:

1. Well productivity
2. Expected well service life
3. The suggested location for the alternative future wells
4. Well water quality

Design Period for Water Supply Components:

- Water supply projects includes permanent constructions at Intakes, treatment plants and overhead tanks and pipe lines
- They cannot be replaced or resized or reconstructed easily in every year

- The design period is varying from component to another
- It's between 10-50 years.

The design periods for various project components mas be taken as follows:

Component	Design period
Storage reservoir	50 years
Pipe connections	30 years
Distribution systems	30 years
Water treatment units	15 years
Electric motors and pumps	15 years

Design Population (present and future): Water Supply project must design to serve current and future population based on the design period

Factors affecting population increase:

1. Industrial and commercial activity
2. Transportation facilities
3. War and diseases
4. Immigration

Population Forecasting Methods:

- Arithmetic method
- Population density method

Water Demand: Unite of water consumption Liter/ capita. day (LCD)

Consumption for various purposes:

1. Domestic consumption
2. Industrial consumption
3. Commercial consumption
4. Public use
5. Losses and waste (about 15 percent of the total)

Variations in Rate of Demand: The unit demand estimates are averages. Water consumption changes with the seasons, the days of the week, and the hours of the day.

Per Capita Demand: The demand of water per person per day.

per capita demand (q_{ave})= $\frac{vearly\ Water\ Demand}{population*365}$

An average (people in town) per capita consumption varies from 190 to 340 liters.

<u>Variations in Rate of Demand</u>: The variations in rate of demand is normally reported as a factor of the average day average factors.

$$q_{ave} = \sum \text{yearly consumption}/365$$

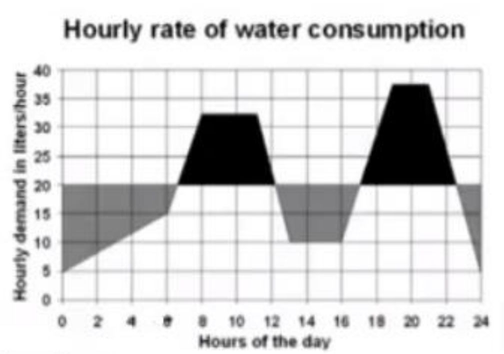
$$q_{max\ monthly} = 1.5\ q_{ave}$$

$$q_{max\ daily} = 2\ q_{ave}$$

$$q_{max\ hourly} = 4.5\ q_{ave}$$

$$Q_{ave} = \frac{population \times q_{ave}}{1000 \times 24 \times 60 \times 60} = m^3/s$$

Where: q_{ave} = Average daily consumption (l.c.d)
Q_{ave}= Average city discharge (m^3/s)



<u>Effect of Variation in Consumption on Design:</u>

- The design basıs for the water source and treatment facilities shall be for the **<u>maximum day demand</u>** at the design year.
- Pumping facilities and distribution system piping are designed to carry **<u>the peak hour flow rate or the maxımum day demand</u>** **<u>plus fire demand</u>** whichever is greater.

<u>Factors affecting the rate of water Demand</u>:

1. Climate
2. Culture
3. Standard of living
4. Water pressure
5. Quality of water
6. Cost of water
7. Water availability.

<u>Fire Demand</u>: Fire demand can be estimated according to the community size by using the following empirical equation.

$Q_{fire} = 3860.7(P_k)^{0.5}[1 - 0.01(P_k)^{0.5}]$

Where: Q_{fire} is the fire demand [in L/min], and P_k is the population [in thousands].

<u>Water Treatment Plant Sizing & Location</u>:

- The selection of the water treatment units types are based on (Water source type & quality)
- The selection of the water treatment units size & numbers are based on:
    1. Water demand
    2. Land availability and cost
- The selection of the water treatment location is based on:
    1. Water source location
    2. Area topography
- In addition, the site should allow for expansion

<u>Distribution System Design Requirements:</u> A properly designed water distribution system should fulfill the following requirements:

- Main requirements: Satisfied quality and quantity standards
- Additional requirements:
    1. To enable reliable operation during irregular situations (power failure, fires…)
    2. To be economically and financially viable, ensuring income for operation, maintenance and extension.
    3. To be flexible with respect to the future extensions.

<u>Design Criteria:</u> Are the design limitations required to get the most efficient and economical water-distribution network.

1. Velocity
2. Pressure
3. Pipe Sizes
4. Head Losses

Velocity:

- Not be lower than 0.6 m/s to prevent sedimentation.
- Not be more than 3 m/s to prevent erosion and high head losses.
- Commonly used values are 1 - 1.5 m/sec.

Pressure:

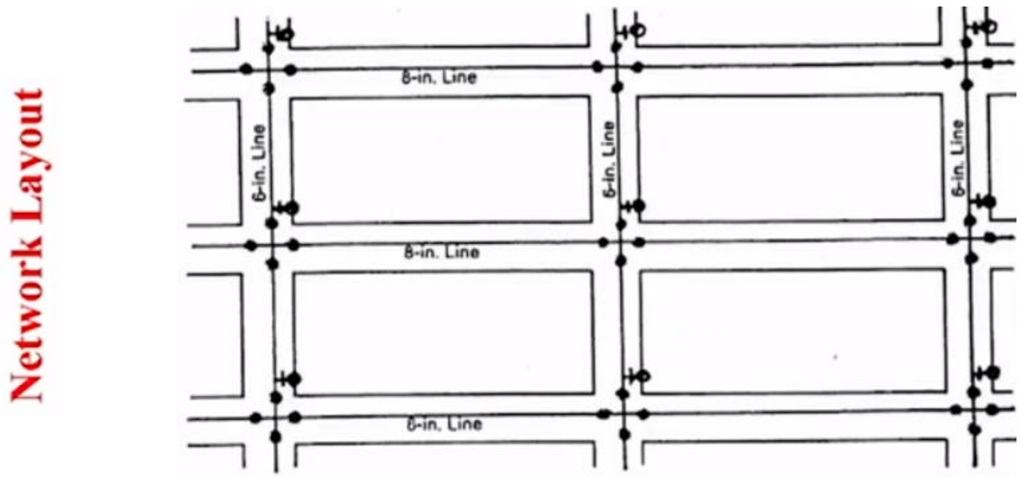
- Pressure in municipal distribution systems ranges from 150-300 kPa in residential districts with structures of four stories or less and 400-500 kPa in commercial districts.
- Also, for fire hydrants the pressure should not be less than 150 kPa (15 m of water).
- In general, for any node in the network the pressure should not be less than 25 m of water.
- Moreover, the maximum pressure should be limited to 70 m of water

Pipe sizes:

- Lines which provide only domestic flow may be as small as 100 mm (4in) but should not exceed 400 m in length (if dead-ended) or 600 m if connected to the system at both ends.
- Lines as small as 50-75 mm (2-3 in) are sometimes used in small communities with length not to exceed 100 m (if dead-ended) or 200 m if connected at both ends.
- The size of the small distribution mains is seldom less than 150 mm (6 in) with cross mains located at intervals not more than180 m,
- In high-value districts the minimum size is 200 mm (8 in) with cross-mains at the same maximum. spacing Major streets are provided with lines not less than 305 mm (12 in) in diameter.

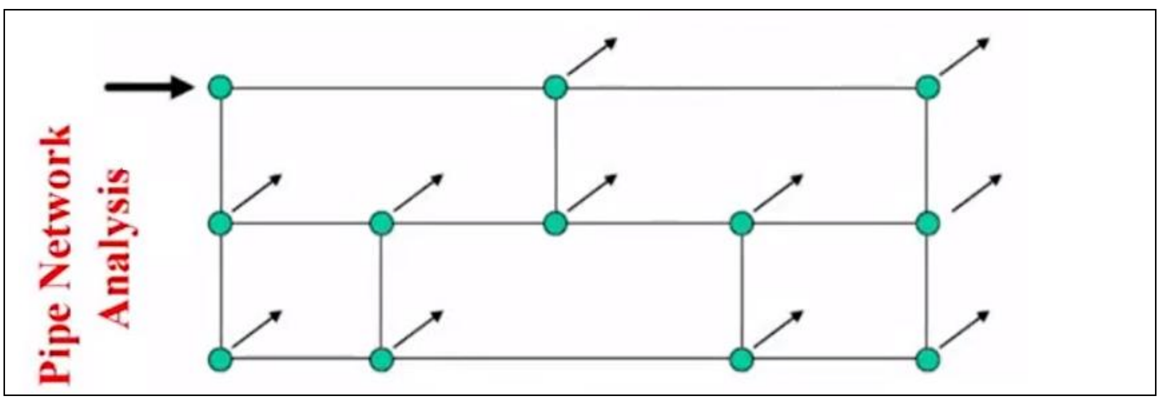
Head Losses: Optimum range is 1-4 m/km.

Maximum head loss should not exceed 10 m/km.



Network Layout:

- Next step is to estimate pipe sizes on the basis of water demand and local code requirements.
- The pipes are then drawn on a digital map (using AutoCAD, GIS, waterCAD) starting from the water source.
- All the components (pipes, valves, fire hydrants) of the water network should be shown on the lines.



Pipe Networks Analysis:

- A hydraulic model is useful for examining the impact of design and operation decisions.
- Simple systems, such as those will be discussed later can be solved using a hand calculator.
- However, more complex systems require more effort even for steady state conditions, but, as in simple systems, the flow and pressure-head distribution through a water distribution system must satisfy the laws of conservation of mass and energy.

Pipe Networks Analysis: The equations to solve Pipe network must satisfy the following condition:

- The net flow into any junction must be zero

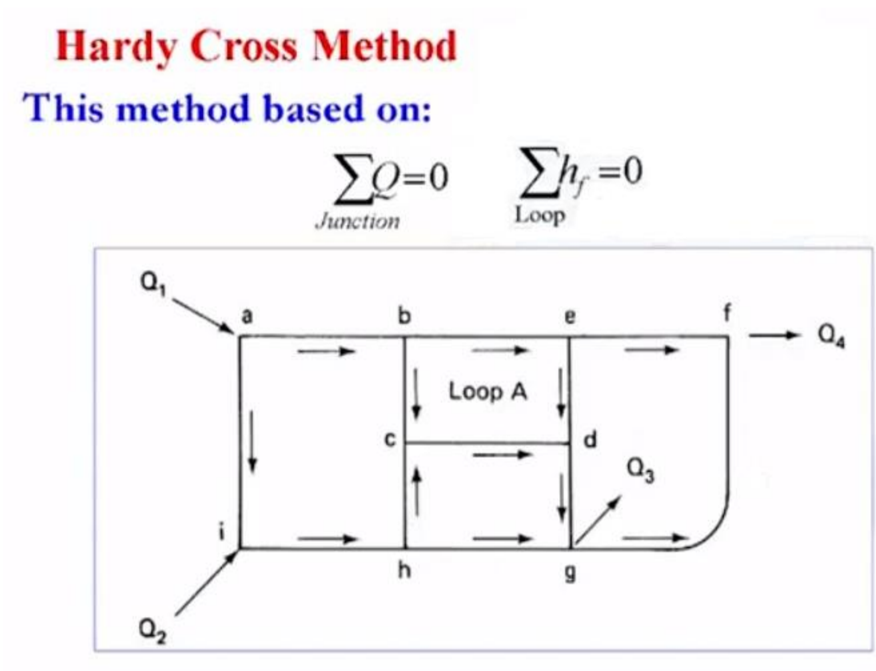
$$\sum_{Junction} Q = 0$$

- The net head loss a round any closed loop must be zero h_f

$$\sum_{Loop} hf = 0$$

Hydraulic Analysis: After completing all required studies and layout drawing of the network, one of the methods of hydraulic analysis is used to

- Size the pipes
- Assign the pressures and velocities required



Assumptions / Steps of this method:

1. Assume that the water is withdrawn from nodes only not directly from pipes.
2. The discharge (Q) entering the system will have (+) value, and the discharge (Q) leaving the system will have (-) value.
3. Usually neglect minor losses since these will be small with respect to those in long pipes.
4. Assume flows for each individual pipe in the network.

5. At any junction (node), as done for pipes in parallel,

$$\sum Q\ in = \sum Q\ out \qquad or \qquad \sum Q = 0$$

6. Around any loop in the grid, the sum of head losses must equal to zero:
   $\sum_{Loop} h_f = 0$
   - Conventionally, clockwise flows in a loop are considered (+) and produce positive head losses; counterclockwise flows are then (-) and produce negative head losses.
   - This fact is called the head balance of each loop, and this can be valid only if the assumed Q for each pipe, within the loop, is correct.
   - The probability of initially guessing all flow rates correctly is virtually null.
   - Therefore, to balance the head around each loop. a flow rate correction (Δ) for each loop in the network should be computed, and hence some iteration scheme is needed.
7. After finding the discharge correction, Δ (one for each loop), the assumed discharges Q_0 are adjusted and another iteration is carried out until all corrections (values of Δ) become zero or negligible, At this point the condition of:
   $\sum_{Loop} h_f \cong 0.0$ is satisfied

Notes:

- The flows in pipes common to two loops are positive in one loop and negative in the other.
- When calculated corrections are applied, with careful attention to sign, pipes common to two loops receive both corrections.

$$\Delta = \frac{-\sum kQ_0^{\,n}}{\sum nkQ_0^{\,(n-1)}} = \frac{-\sum hf}{n\sum\frac{hf}{Qo}}$$

Note: that if Hazen Williams (which is generally used in this method) is used to find the head losses, then:

$$h_f = kQ^{1.85} \qquad n = 1.85 \quad then: \qquad \Delta = \frac{-\sum h_f}{1.85\sum\frac{h_f}{Q}}$$

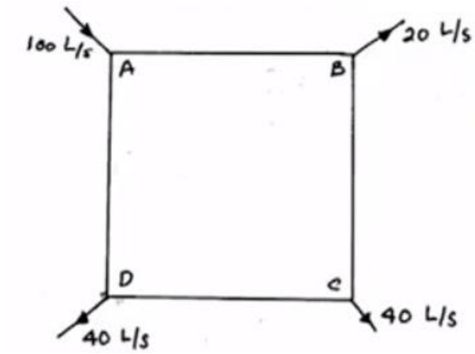
If Darcy-Wiesbach is used to find the head losses, then:

$$h_f = kQ^2 \qquad n = 2 \quad then: \qquad \Delta = \frac{-\sum hf}{2\sum\frac{hf}{Q}}$$

Example: For the square loop shown:

1. find the discharge in all the pipes. All pipes are 1 km long and 300 mm in diameter, with a friction factor of 0.0163. Assume that minor losses can be neglected.
2. If the required pressure at point d=25 m, then find the required pressure in point A

Solution:



- Assume values of Q to satisfy continuity equations all at nodes.
- The head loss is calculated using, $H = KQ^2$
- $H_L = h_f + h_{Lm}$
- But minor losses can be neglected: → $h_{Lm} = 0$
- Thus $H_L = h_f$
- Head loss can be calculated using the Darcy-Weisbach equation: $h_f = \lambda \frac{L}{D}\frac{v^2}{2g}$

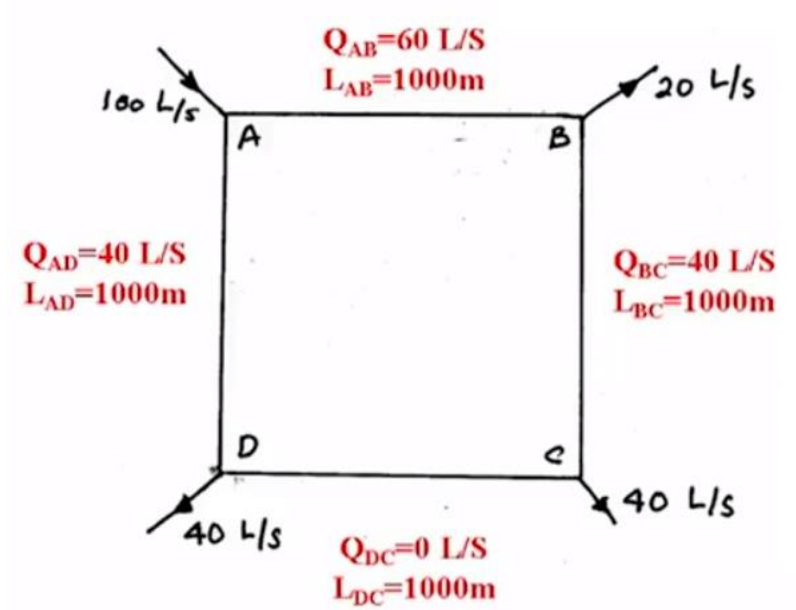
$$H_L = h_f = \xi \frac{L}{D}\frac{v^2}{2g}$$

$$H_L = 0.0163 \frac{1000}{0.3}\frac{v^2}{2 * 9.81}$$

$$H_L = 2.77 * \frac{Q^2}{A^2}$$

$$H_L = 2.77 * \frac{Q^2}{\left(\frac{\pi}{4} * 0.3^2\right)^2}$$

$$H_L = 554 * Q^2$$
$$H_L = k * Q^2$$
$$k = 554$$



Trial 1

Pipe	Q (L/s)	H_L (m)	H_L/Q
AB	60=	2	0.033
BC	40	0.886	0.0222
CD	0	0	0
DA	- 40	- 0.886	0.0222
$\sum$		2	0.0774

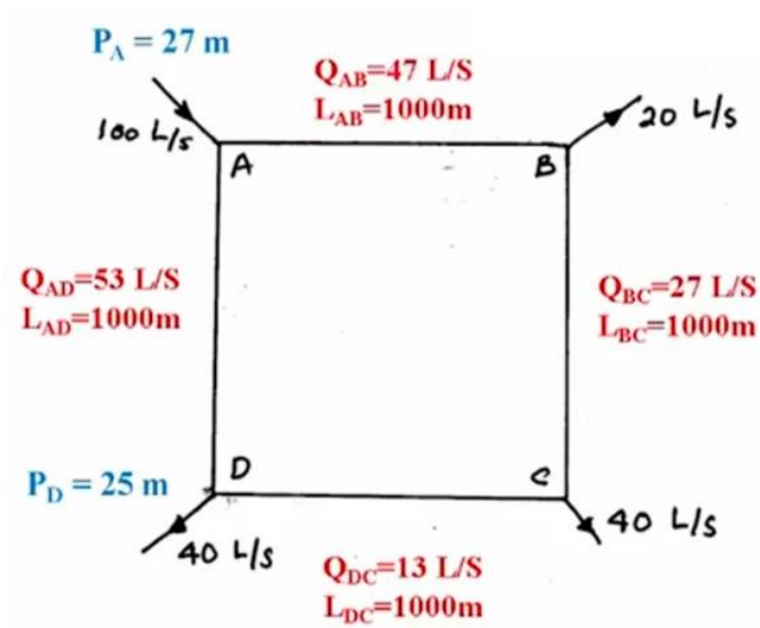
Since $\sum H_L > 0.01m,$ then correction has to be applied.

Trial 2 $\Delta Q = \frac{-\sum H_L}{2\sum {H_L}/{Q}} = \frac{-2}{2*0.0775} = -12.92\ L/s$

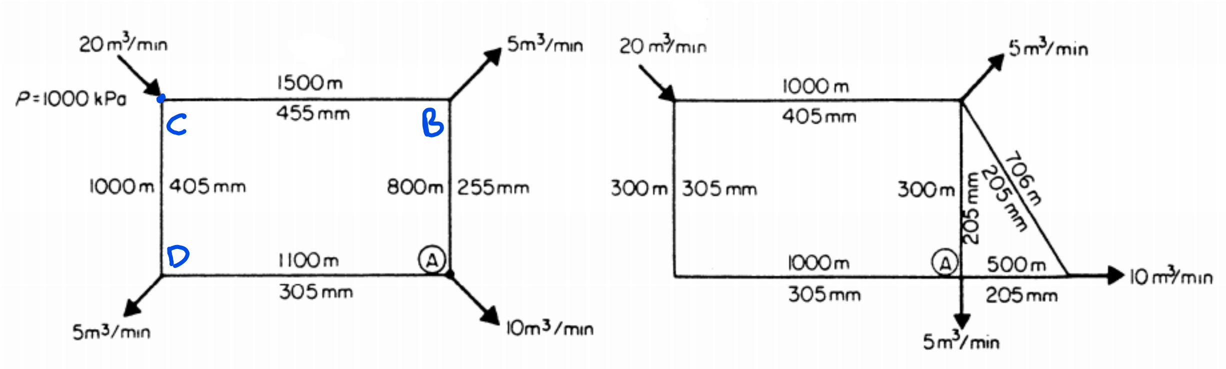
Pipe	Q (L/s)	H_L (m)	H_L/Q
AB	47.08 $\approx$ **47**	1.23	0.0261
BC	27.08 $\approx$ **27**	0.407	0.015
CD	-12.92 $\approx$ **-13**	-0.092	0.007
DA	- 52.92 $\approx$ **-53**	- 1.555	0.0294
$\sum$		-0.0107	0.0775

Since $\sum H_L \approx 0.01m,$ then it is OK.

Thus, the discharge in each pipe (to the nearest integer).



H.W: In the pipe systems show, find the distribution of flow, with a friction factor of 0.0163. Assume that minor losses can be neglected. If the pressure at first point (Q in) =1000 kPa, then find the pressure at point A



Solution:

m^3/min

Q	NAME	DIA.	L	H	H/Q	ΣH	ΣH/Q	Δ	A^2	K
T 1										
15	CB	0.455	1500	6.481439	0.432096				0.026411003	103.703
10	BA	0.255	800	27.78713	2.778713				0.002605554	1000.337
0	AD	0.305	1100	0	0				0.005332596	561.8892
-5	DC	0.405	1000	0.85925	-0.17185	33.40932	3.382659	-4.93832	0.016579041	123.732
T 2										
10.06168				2.916285	0.289841					
5.06168				7.119232	1.406496					
-4.93832				3.806332	-0.77077					
-9.93832				3.394732	-0.34158	10.44712	1.267142	-4.12231		
T 3										
5.93937				1.016178	0.171092					
0.93937				0.245198	0.261024					
-9.06063				12.81342	-1.41419					
-14.0606				6.794995	-0.48326	-18.347	2.329566	3.937866		
T 4										
9.877236				2.810346	0.284528					
4.877236				6.609845	1.355244					
-5.12276				4.095971	-0.79956					
-10.1228				3.521906	-0.34792	1.802314	2.787254	-0.32331		
T 5										
9.553926				2.629376	0.275214					
4.553926				5.762563	1.265406					
-5.44607				4.629299	-0.85003					
-10.4461				3.75047	-0.35903	0.01217	2.749676	-0.00221		
T 6										
9.551716				2.62816	0.275151					
4.551716				5.756971	1.264791					
-5.44828				4.633057	-0.85037					
-10.4483				3.752058	-0.35911	1.66E-05	< 0.01	m		

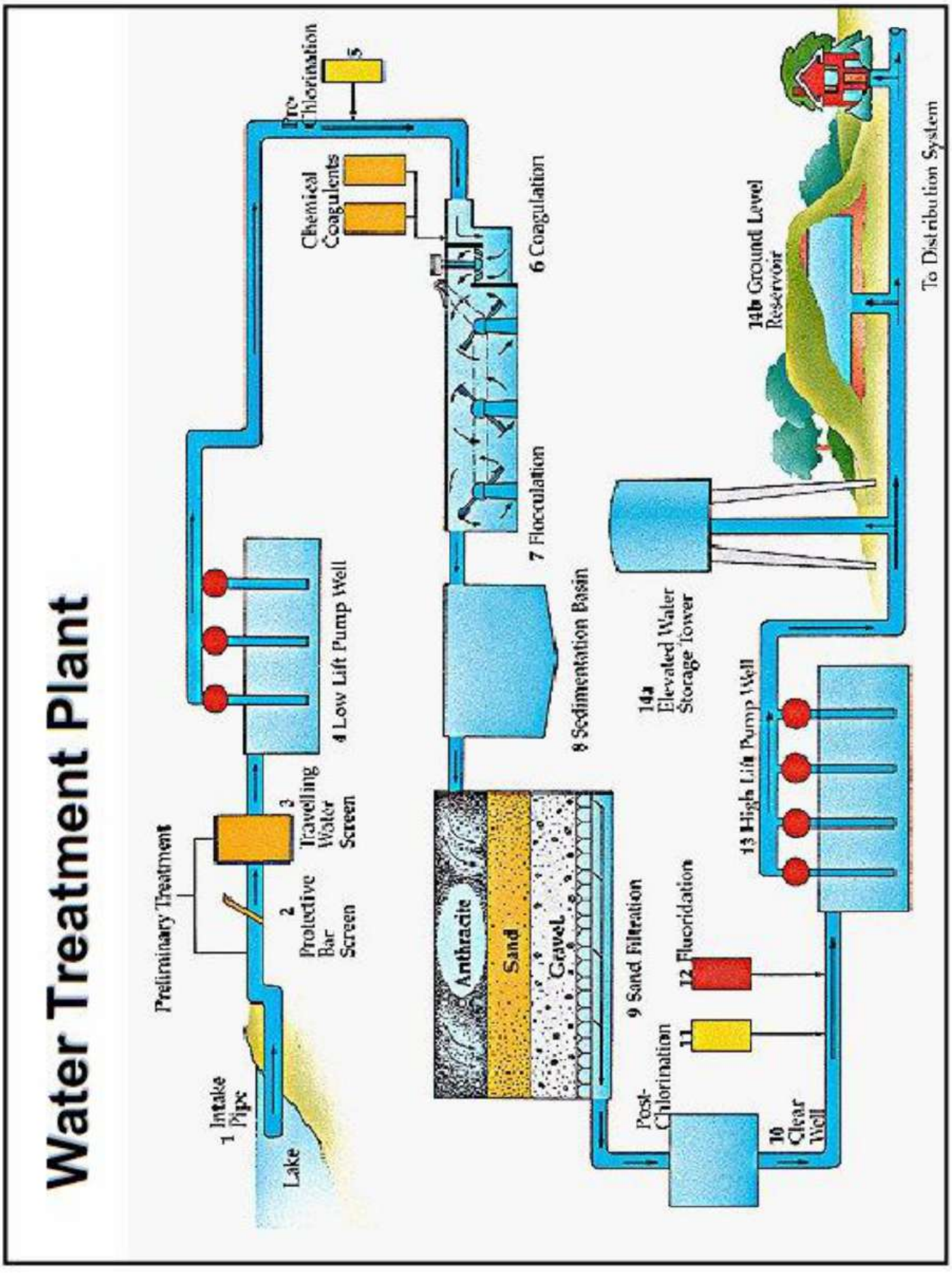


Fig (3.1) processes train of the water treatment plant

Table (3.1) The capability of the various treatment techniques for removing contaminants

	Bacteria	Cysts	Viruses	Algae	Coarse Particles	Turbidity	Colour	Aluminium	Ammonia	Arsenic	Iron & Manganese	Nitrate	Pesticides	Solvents	Taste & Odour
Coagulation/flocculation[1]	+	+	+	+	‡	‡	‡	‡		+	‡				
Sedimentation					‡	+		+			+				
Gravel filter/screen				+	‡	+		+			+				
Rapid sand filtration	+	+	+	+	‡	+		+			+				
Slow sand filtration	‡	‡	‡	‡	‡	‡		+			+				
Chlorination	‡		‡	+			+		‡						
Ozonation	‡	+	‡	‡			+						‡		‡
UV	‡	+	‡	+											
Activated carbon							+						+	+	‡
Activated alumina										‡					
Ceramic filter	‡	‡		‡	‡	‡									
Ion exchange									+	+		‡			
Membranes	‡	‡	‡	‡	‡	‡	‡	‡		+	‡	‡	‡		‡

3.2 Intake Structure

The basic function of the intake structure, is to help in safely withdrawing water from the source over predetermined pool levels and then to discharge this water into the withdrawal conduit (normally called intake conduit), through which it flows up to water treatment plant. Some types of intakes structure are shown in Figs. (3.2-3.7).

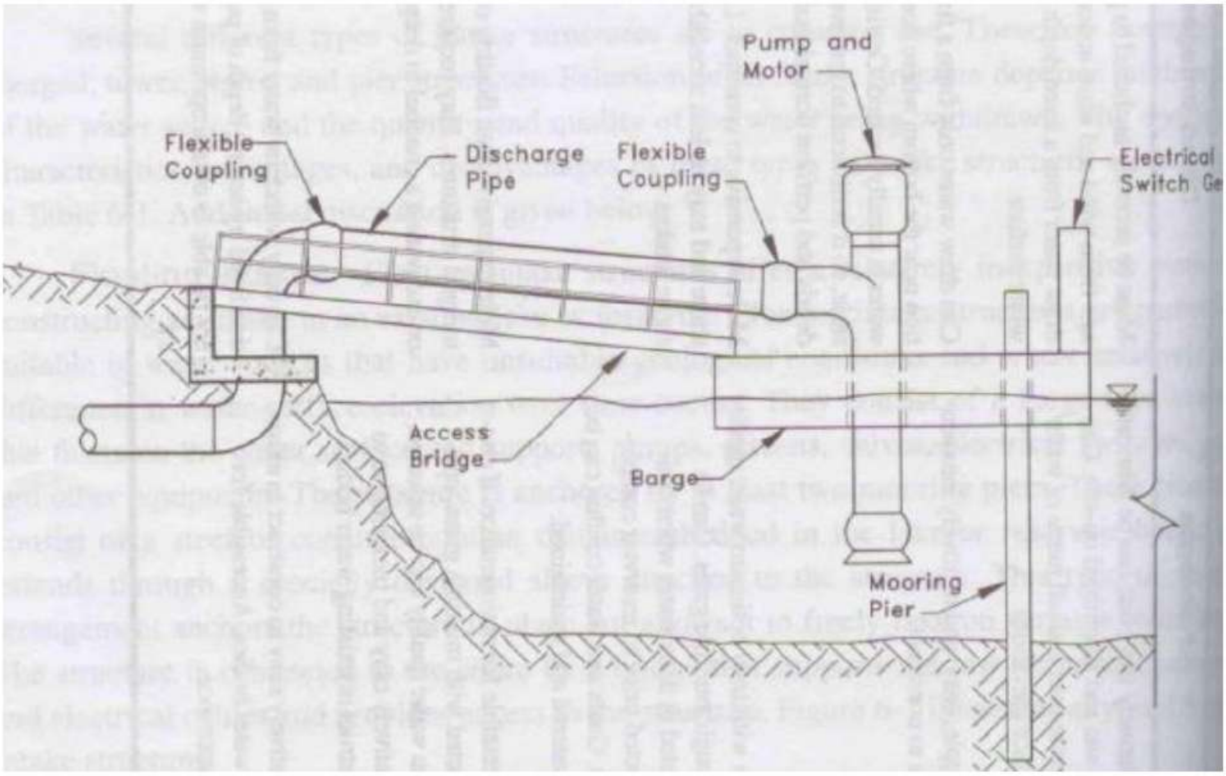


Fig. 3.2 Floating intakes

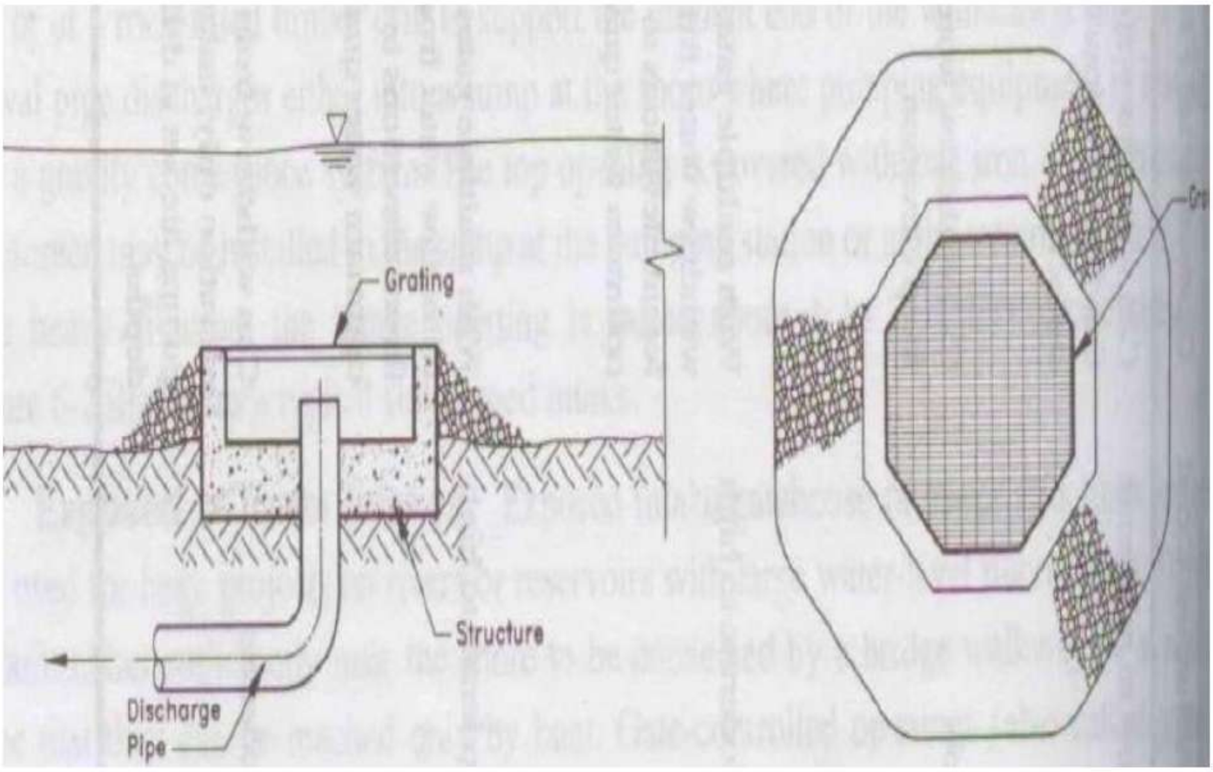


Fig. 3.3 Submerged intakes

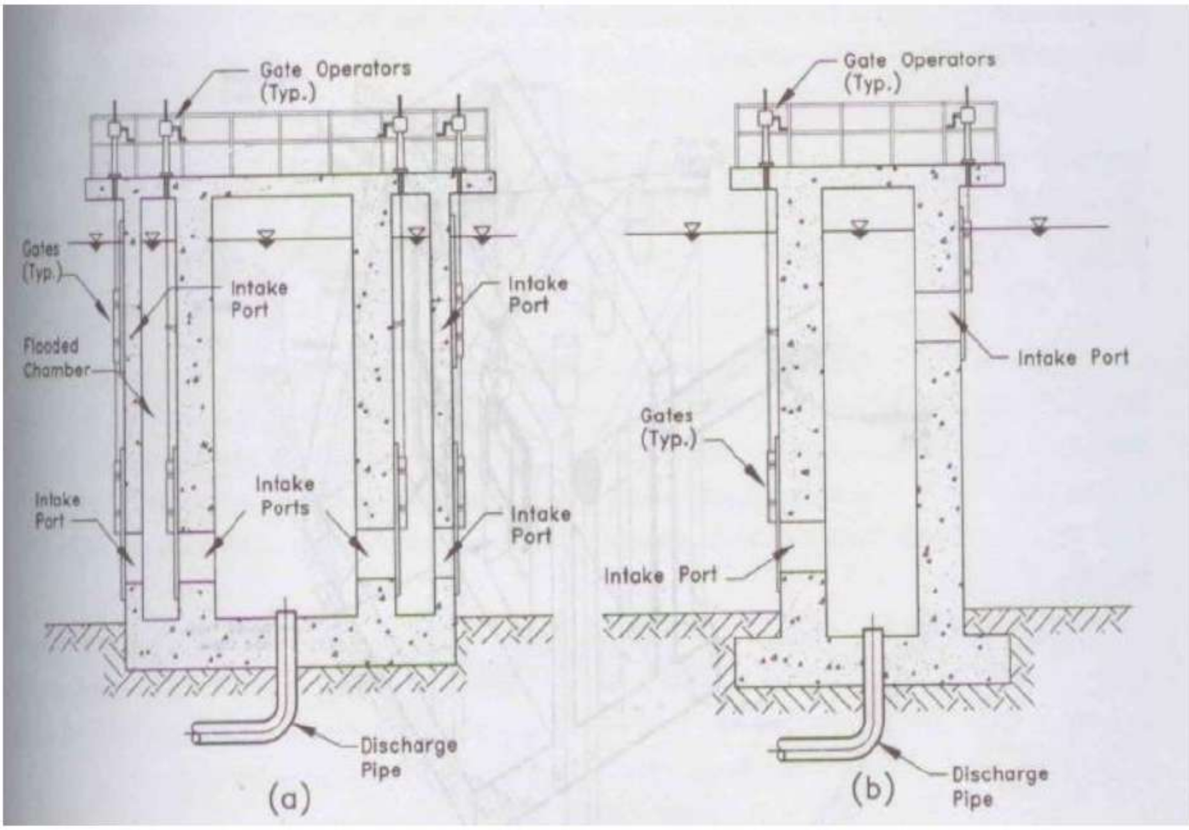


Fig. 3.4 Exposed or Tower intakes

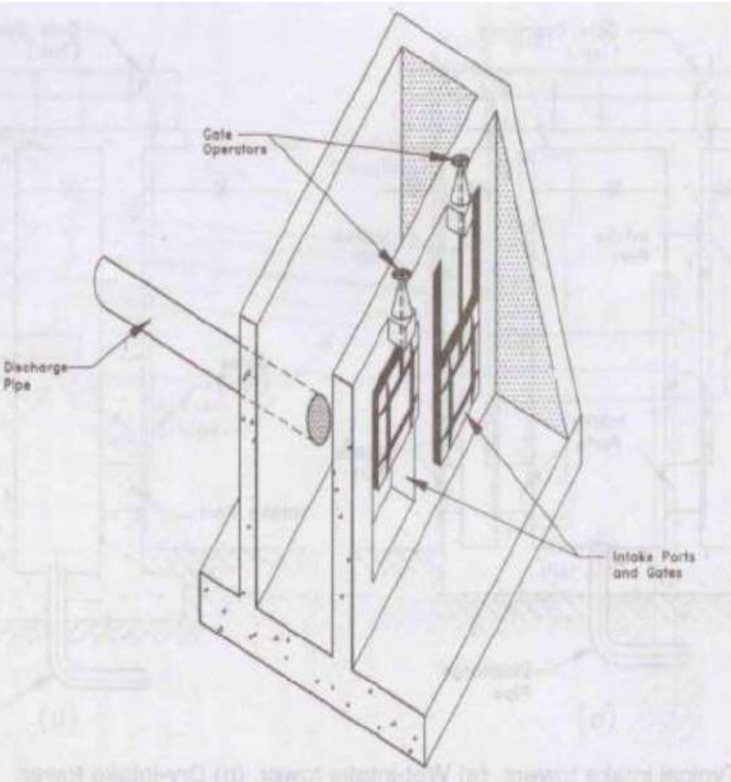


Fig. 3.5 Shore-intake structure

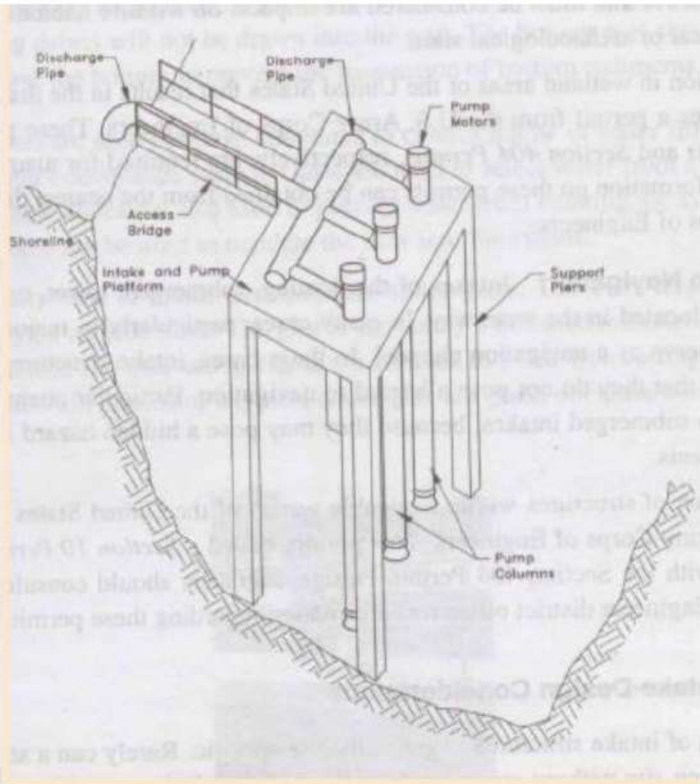


Fig. 3.6 Pier structure

3.2.1 Factors Governing Location of Intake

1) The site should be near the treatment plant so that the cost of conveying water to the city is less.

2) The intake must be located in the purer zone of the source to draw best quality water from the source, thereby reducing load on the treatment plant.

3) The intake must never be located at the downstream or in the vicinity of the point of disposal of wastewater.

4) The site should be such as to permit greater withdrawal of water, if required at a future date.

5) The intake must be located at a place from where it can draw water even during the driest period of the year.

6) The intake site should remain easily accessible during floods and should not get flooded. Moreover, the flood waters should not be concentrated in the vicinity of the intake.

3.2.2 Design Considerations

1) Sufficient factor of safety against external forces such as heavy currents, floating materials, submerged bodies, ice pressure, etc.

2) Should have sufficient self-weight so that it does not float by up thrust of water.

3) Intake velocity plays an important role if the intake is a gate. High intake velocity increase head losses, and low intake velocity require the intake port to be larger and so add to the cost of the structure.

3.2.3 Design of intakes

The hydraulic consideration in intake structure is energy losses due to the acceleration. The losses through the intake port can be calculated by using the orifice equation:

$$h_L = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2 \quad \text{..........3.1}$$

where,

h_L = head loss, m
Q = discharge, m^3/s
C = coefficient of discharge (0.6-0.9)
A = effective submerged open area, m^2

3.3 Screening

A screen is a device with openings for removing bigger suspended or floating matter in water which would otherwise damage equipment or interfere with satisfactory operation of treatment units.

3.3.1 Types of Screens

Coarse Screens**:** Coarse screens also called racks, are usually bar screens, composed of vertical or inclined bars spaced at equal intervals across a channel through which raw water flows. Clear space between bars ranges from 20 to 50 mm. Bar screens are usually hand cleaned and sometimes provided with mechanical devices. These cleaning devices are rakes which periodically sweep the entire screen removing the solids for further processing or disposal. Hand cleaned racks are set usually at an angle of 45° to the horizontal to increase the effective cleaning surface and also facilitate the raking operations. Mechanical cleaned racks are generally erected almost vertically. The angle of inclination of rack with horizontal is between 30° and 60°.

Fine screen: Fine screen (< 2cm) is used to remove smaller objects such as leaves, twigs and fish. that may damage pumps or other equipment. They may be located either at the intake structure or at the raw water pump station.

Figure 3.2 shows types of screen.

a) Coarse screen

b) Fine screen

c) Manual cleaning screen

d) Automatically cleaning screen

<u>Fig (3.2)</u> Types of screen

3.3.2 Design of screen

The design velocity should be such as to permit 100% removal of material of certain size without undue depositions. Velocities of 0.6 to 1.2 m/s through the open area for the peak flows have been used satisfactorily. Further, the velocity at low flows in the approach channel should not be less than 0.3 m/s to avoid deposition of solids. Head loss varies with the nature of screenings (open area, blocked area, shape of the screen) and with hydraulic parameters at the upstream of the screen. The head loss through a **vertical bar screens** is calculated from the following formula:

$$h_L = \frac{V_b^2 - V_a^2}{2g} \times \frac{1}{0.7} \quad \ldots\ldots\ldots 3.2$$

where, h_L = head loss v_b = velocity through bar opening in m/s, v_a =
in m, approach velocity in m/s
velocity in channel.

Another formula often used to determine the head loss through an **inclined bar** rack is Kirschmer's equation:

$h_L = \beta\ (W/b)^{4/3}\ (v^2/2g)\ \sin\theta$ ……………...3.3

where h_L = head loss, m

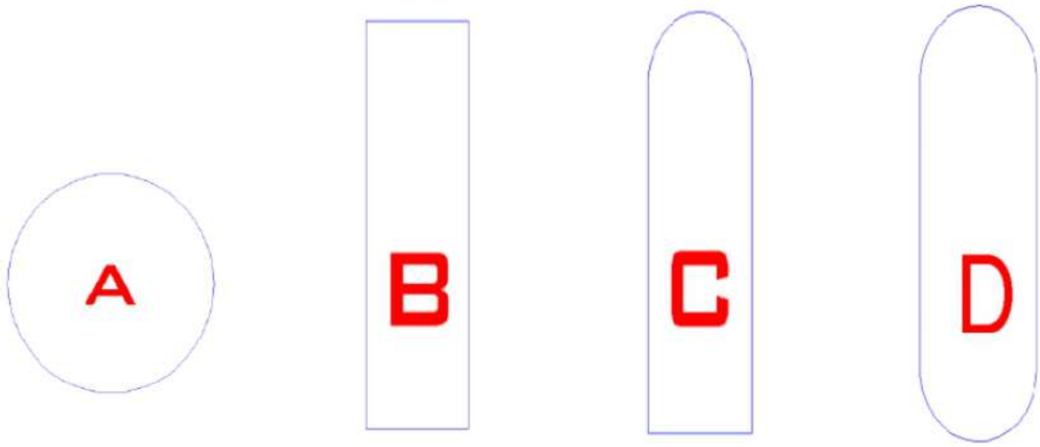
β = bar shape factor (1.79 for circular bar, 2.42 for sharp edge rectangular bar, 1.83 for rectangular bar with semicircle upstream, and 1.67 for rectangular bar with both u/s and d/s face as semicircular).

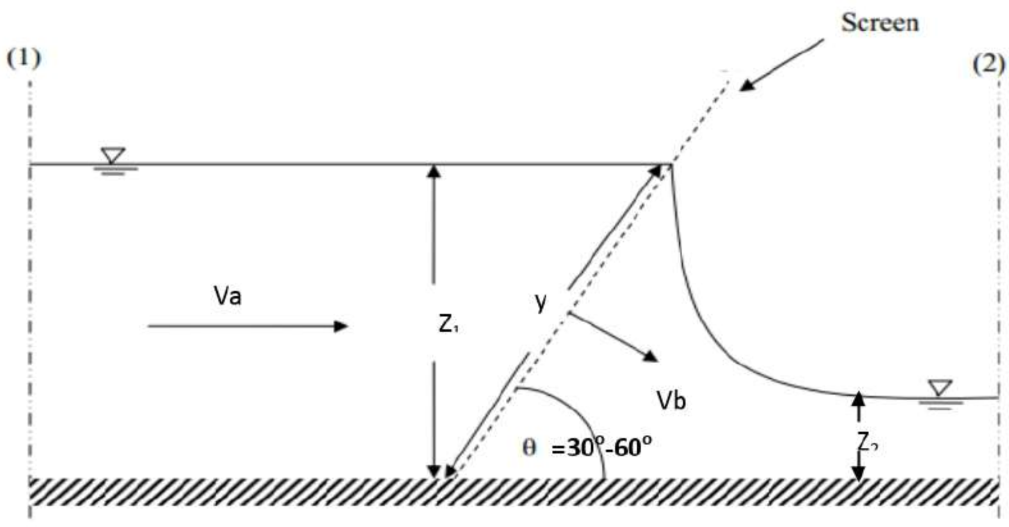
W = maximum width of bar u/s of flow, m

b = minimum clear spacing between bars, m

v = approach velocity, m/s

θ = angle of inclination of rack with horizontal (30-60°)





r by

orifice equation (3.1). Orifice formula can be used when the screen is located at the pumping station.

<u>Intake design example</u>

Design an intake tower with gates meet the following requirement:

- Minimum reservoir elevation = 70 m msl
- Maximum reservoir elevation = 90 m msl
- Normal water surface elevation = 85 m msl
- Bottom elevation= 60 m msl
- Flow rate = 113500 m^3/day
- Velocity = 0.08 m/s

<u>Solution</u>

Q = 113500 m^3/day = 1.31 m^3/s

A = 1.32 / 0.08 = 16.38 m^2

This is too large for a single gate, so select two equal size square gates

Width = $(16.38 / 2)^{0.5}$ = 2.86 m, then use width and height = 3 m

Set the highest gate with its top two meters below the normal water surface elevation of 85 m, then a centerline elevation = 81.5 m (85-2-1.5). Now, set the lowest gate at a centerline elevation = 65 m. Provide additional gates at two levels equally spaced over 16.5 m range (81.5-65).

Spacing = 16.5 / 3 space = 5.5 m/space

Gates will be provided at centerline elevations of 81.5, 76, 70.5 and 65 m, as shown in the figure (1).

The head loss through the intake can be calculated from the orifice formula (eq 3.1). Two gates is used and others are standby. Therefore the flow rate = 1.31 / 2 = 0.66 m^3/s

flow for 1 gate

$$h_l = \frac{1}{2\times 9.81}\left(\frac{0.66}{0.6\times \underbrace{3\times 3}_{\text{Area of the gate}}}\right)^2 = 7.6138\times 10^{-4}\ m$$

C

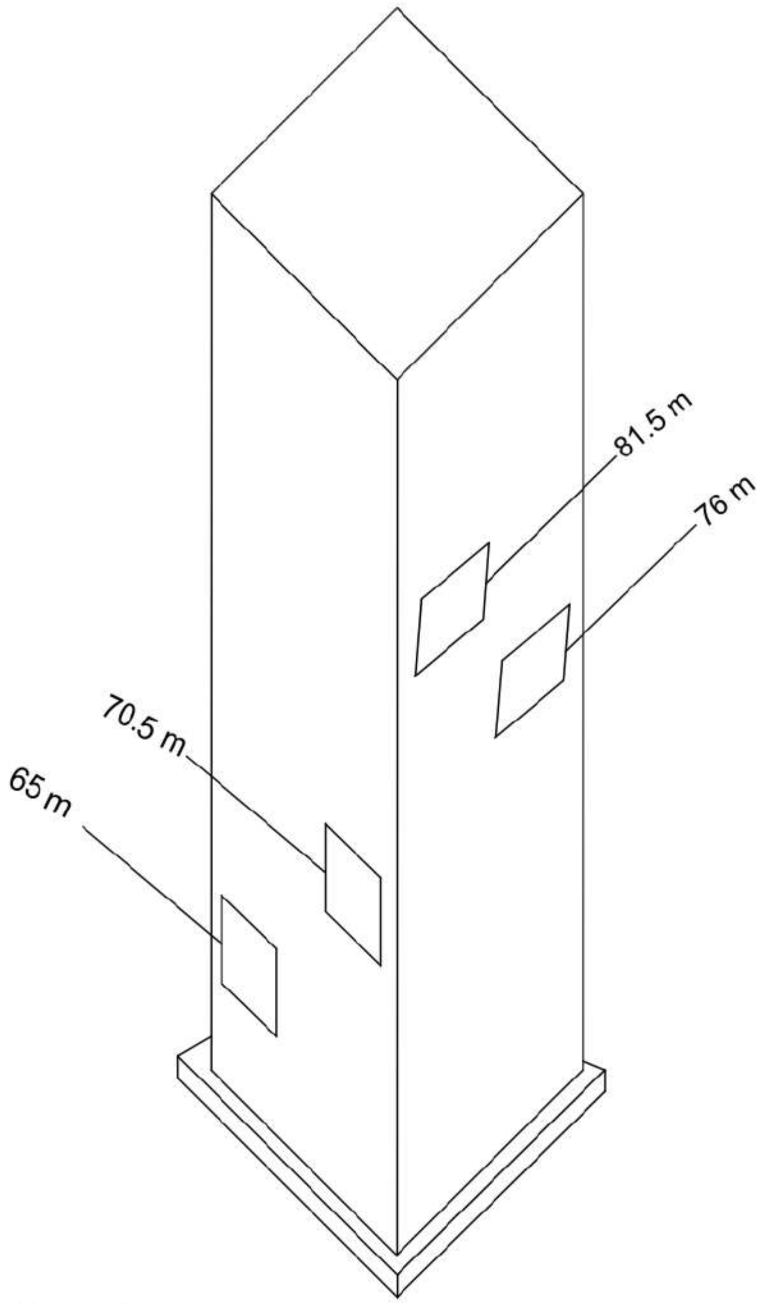
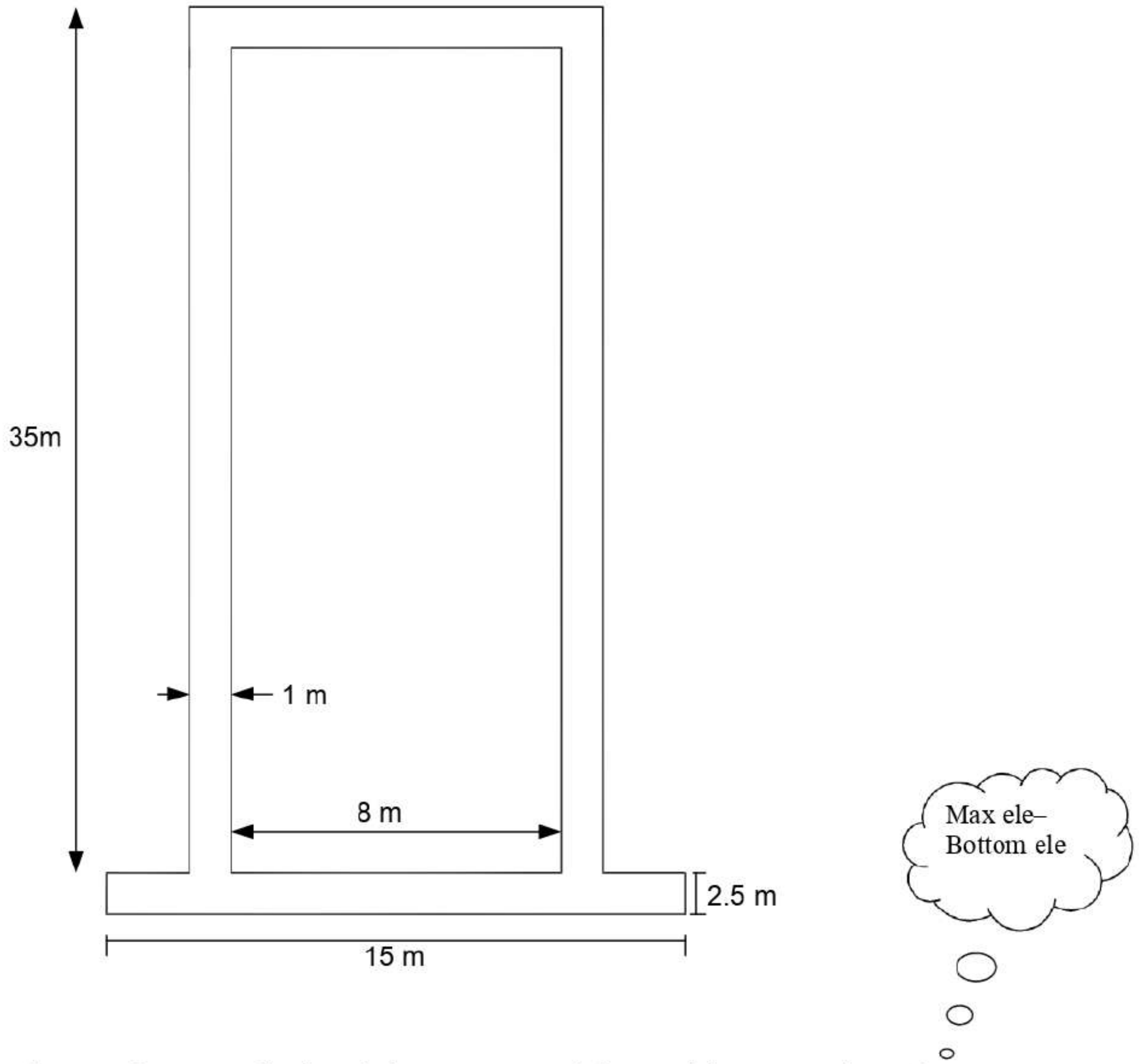


Fig (1) Intake structure

Now, to calculate the stability of the structure we must compare the weight of water displaced at the maximum elevation with the weight of tower when it is empty (worst condition).



Volume of water displaced by tower and base slab = 10m*10m*30m + 15m*15m*2.5m = 3563 m^3

The weight of water displaced = 3563 $m^3 \star$ 1000 kg/m^3 = 3.56*10^6 kg

The weight of structure equal the weight of side walls plus the weight of the foundation slab.

Weight of side walls = (10m*10m*35m - 8m*8m*35m) * 2308 kg/m^3 = 2.91*10^6 kg

Weight of the foundation slab = 15m*15m*2.5m* 2308 kg/m^3 =
1.3*10^6 kg

Total weight of structure = 2.91*10^6 kg + 1.3*10^6 kg = 4.21*10^6 kg> 3.56*10^6 kg OK

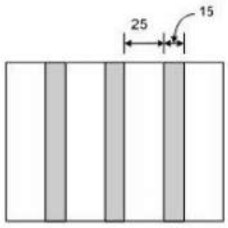
Safty factor = 4.21*10^6 / 3.56*10^6 = 1.2 Very GOOD

Design example

A mechanical bar screen is to be used in an approach channel with a maximum velocity of 1 m/s. The bars are 15mm thick, and the opening are 25mm wide. Determine the velocity between bars and the head losses.

Solution

- Assume the channel has a width (W) and depth (D)
- Net area of screen = WD [25 / (25+15) = (5/8)WD
- Area of channel = WD
- Use continuity equation



- $V_a A_a = V_b A_b$ OR $V_b = \dfrac{V_a A_a}{A_b}$

$$V_b = \frac{1 \times WD}{(5/8)WD} = 1.6 \text{ m/s}$$

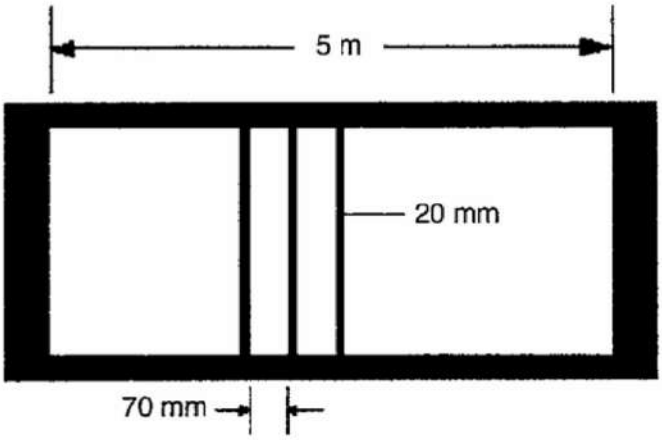
$$h_L = \frac{(V_b^2 - V_a^2)}{2g} \times \frac{1}{0.7}$$

$$h_L = \frac{(1.6)^2 - 1^2}{2g} \times \frac{1}{0.7} = 0.114m$$

Example

A bar screen measuring 2 m by 5 m of surface flow area is used to protect the pump in a shoreline intake of a water treatment plant. The plant is drawing raw water from the river at a rate of 8 m^3/sec . The bar width is 20 mm and the bar spacing is 70 mm. If the screen is 30% clogged, calculate the head loss through the screen. Assume Cd = 0.60

Solution



For screens used in shoreline intakes, the velocity of approach is practically zero. Thus, from the previous figure, the number of spacings is equal to one more than the number of bars. Let x number of bars,

20x + 70(x+1) = 5000
Then x = 54.7 = 55
Area of clear opening = 70(55+1)*2000 = 7.48 m^2

$$h_l = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2$$

$$h_l = \frac{(8)^2}{2 \times 9.8(0.6 \times 7.48 \times 0.7)^2} = 0.33m$$

In this example we choose the orifice equation as a result to the exit of pump in a shoreline intake.

Design examples

Use Kirschmer's equation_to find the head loss through a rectangular bar screen used in treatment plant, if the bar width is 15 mm and spacing is 25 mm. Assume that the width of channel (w) equal two times of water depth. Use Q = 0.6 m^3/s, approach velocity = 0.6 m/s and theta = 60°.