Total solids removed in 6 weeks (*i.e.* 6 × 7 days)

$$= 855 \times (6 \times 7)\text{ kg} = \mathbf{35{,}910\ kg.}$$

Since the produced sludge has a moisture content of 85%, we have

15 kg of solids (dry sludge) will make = 100 kg of wet sludge

∴ 35910 kg of solids (dry sludge) will make

$$= \frac{100}{15} \times 35{,}910 \text{ kg of wet sludge}$$

$$= 2{,}39{,}400 \text{ kg of wet sludge}$$

$$= \mathbf{239.4} \text{ tonne of wet sludge.}$$

Volume of wet sludge *produced per 6 week interval* $= \dfrac{\text{weight of wet sludge}}{\text{unit weight of wet sludge}}$

where, unit weight of wet sludge

= sp. gr. of wet sludge × unit weight of water

$$= 1.24 \times 1 \text{ t/m}^3 = 1.24 \text{ t/m}^3$$

$$\therefore \text{ Volume of wet sludge} = \frac{239.4\ t}{1.24 \text{ t/m}^3} = \mathbf{193.06\ m^3}.$$

Dimensions of the Tank

L = length of the tank = 2 B, where B is the width of the tank (given)

D_w = Water depth

= 3 m (assuming that the given "water depth just before cleaning" is only the water depth, and does not include the sludge depth)

Q = Flow in the tank = 3000 m^3/day

t = Detention time = 6 hrs.

Required capacity of the tank

Quantity of water to be treated per 6 hr

$$= 3000 \times \frac{6}{24} \text{ m}^3 = 750 \text{ m}^3$$

But capacity = L × B × D_w = (2B) (B) (3) = 750

$$B = \sqrt{\frac{750}{6}} = \mathbf{11.2\ m} \text{ (say)}$$

Hence L = 22.4 m

D = D_w + Sludge depth (average) + Free board;

where, Sludge depth (av.)

$$= \frac{\text{Sludge volume}}{\text{Plane area of the tank}}$$

$$= \frac{193.06 \text{ m}^3}{22.4 \text{ m} \times 11.2 \text{ m}}$$

$$= \mathbf{0.77m;\ say\ 0.8m.}$$

Assuming that a free-board of 0.5 m is provided, we have

total depth of the tank,

$$D = 3 + 0.8 + 0.5 = \mathbf{4.3\ m}.$$

Hence, the *tank size lobe used shall be* **22.4 m × 11.2 m × 4.3 m.**

11. Two primary settling basins are 26 m in diameter with a 2.1 m side water depth. Single effluent weirs are located on the peripheries of the tank.

For a water flow of 26,000 m^3/d, calculate:

(*i*) Surface area and volume,

(*ii*) Overflow rate in m^3/m^2.d

(*iii*) detention time in hours and

(*iv*) weir loading in m3/m.d.

Solution. Detention period

$$= \frac{d^2(0.011d + 0.785\,\text{H})}{\text{Q}}$$

where d = diameter of tank = 26 m,

H = side water depth = 2.1 m

Q = half the total discharge as two tanks are in operation

$$= 13000 \text{ m}^3\text{/day} = \frac{13000}{24} \text{ m}^3\text{/hr.}$$

$$\therefore \text{ Detention period} = \frac{(26)^2[0.011 \times 26 + 0.785 \times 2.1]}{13000/24}$$

$$= 2.41 \text{ hr.}$$

Quantity of water to be treated during the detention period of 2.41 hr.

$$= \frac{13000}{24} \times 2.41 \text{ m}^3$$

$$= 1308 \text{ cu. m.}$$

(*i*) Required capacity (*volume*) of each tank

= **1308 cu.m.**

Required surface area of tank

$$= \frac{\text{Volume}}{\text{Depth of water}}$$

$$= \frac{1308}{2.1} m^2 = \mathbf{6.23\ m^2}$$

(*ii*) *Overflow rate, i.e.* discharge per unit of surface area

$$= \frac{\text{Q}}{\text{Surface area}}$$

$$= \frac{13000}{623} \text{ m}^3\text{/m}^2\text{.day}$$

$$= \mathbf{20.87\ m^3/m^2.\ day.}$$

(*iii*) *Detention time* 2.41 hr. (Calculated) (*iv*) Length of the weir along peripheri of the tank

$= \pi . d = 3.14 \times 26 = 81.64$ m.

Weir loading per day

$$= \frac{\text{Discharge per day}}{\text{length of weir}}$$

$$= \frac{13000}{81.64} = \mathbf{159.23\ m^3/m.\ day.}$$

12. A rectangular settling tank without mechanical equipment is to treat 1.8 million litres per day of raw water. The sedimentation period is to be 4 hours, the velocity of flow 8 cm/minute, and the depth of the water and sediment 4.2 m. If an allowance of 1.2 m for sediment is made, what should be

(*i*) length of the basin; and

(*ii*) width of the basin?

Solution. Supply of water to be treated per day $= 1.8 \times 10^6$ litres

Supply of water to be treated during the detention period of 4 hours, *i.e.*

$$\text{capacity of the tank} = \frac{1.8 \times 10^6}{24} \times 4$$

$$= 0.3 \times 10^6 \text{ litres}$$

$$= 300 \text{ cu. m.}$$

Flow velocity = 8 cm/minute

Length of the tank = Flow velocity × Detention time

$= 8 \times (4 \times 60)$ cm

$= 1920$ cm $=$ **19.2 m.**

Cross-sectional area of the tank

$$= \frac{\text{Capacity of the tank}}{\text{Length of the tank}}$$

$$= \frac{300}{19.2} \text{ m}^2 = 15.63 \text{ m}^2$$

Depth of sediment and water = 4.2 m.

Depth of sediment = 1.2 m.

Water depth = 3 m.

$$\therefore \text{ Width of the tank} = \frac{\text{Cross-sectional area}}{\text{water depth}}$$

$$= \frac{15.63}{3} = \mathbf{5.21\ m.}$$

13. A circular sedimentation tank fitted with standard mechanical sludge removal equipment is to handle 3.5 million litres per day of raw water. If the detention period of the tank is 5 hours, and the depth of the tank is 3 m, what should be the diameter of the tank?

Solution. Quantity of the raw water to be treated per day $= 3.5 \times 10^6$ litres

$\therefore$ Quantity of the raw water to be treated during the detention period of 5 hours, *i.e.*

$$\text{Capacity of the tank} = \frac{3.5 \times 10^6 \times 5}{24} \text{ litres}$$

$$= 728 \times 10^3 \text{ litres}$$

$$= 728 \text{ cu.m.}$$

Capacity of a circular tank of depth H and diameter d is given by

$$\text{Volume} = d^2 (0.011\, d + 0.785 \text{ H}),$$

where H = 3m (given)

$$\therefore \quad 728 = d^2 (0.011\, d + 0.785 \times 3)$$

$$= d^2 (0.011\, d + 2355)$$

Solving this equation by Trial, we get $d =$ **16.95 m.**

14. A surface water treatment plant coagulates a raw water having a turbidity of 9 Jackson condle units by applying an alum dosage of 30 mg/l. Estimate the total sludge solids produced in grams per cubic metre of water processed. Compute the volume of sludge from the settling basin and filter backwash water using 1.0% solid concentration in the sludge and 500 mg/l of solids in the wash-water. Assume that 30% of the total solids are removed in the filter. Use the following equation:

Sludge solids (mg/l)

$$= \frac{\text{Alumdosage (mg/l)}}{4} + \text{raw water turbidity in JTU.}$$

Solution. Total sludge solids produced

$$= \left(\frac{30}{4} + 9\right) \text{mg/l} = 16.5 \text{ mg/l}$$

$$= 16.5 \times \frac{1000}{1000} \text{ gm/cu of water processed}$$

$=$ **16.5 gm/cu m of raw water.**

Consider 1 cu m of raw water processed

Sludge solids removed in the filter

$= 30\%$ of total solids.

$$= \frac{30}{100} \times 16.5 \text{ gm} = 4.95$$

Filter back wash contains 500 mg/l of solids, *i.e.* 500 gm/cu of solids, *i.e.* 500 gm of solids/cu m of wash water

$$\therefore \text{ Volume of filter wash} = \frac{1}{500} \times 4.95 \text{ cu. m.}$$

$=$ **0.0099 cu.m.**

$=$ **9.9 litres**

Hence, volume of sludge from the filter back wash = **9.9 litres**.

Total solids removed in the settling basin = $70\% \times 16.5$ gm = 11.55 gm.

Sludge contains 1% solids mean that 100 gm of sludge will contain 1 gm of solids and 99 gm of water.

$\therefore$ Weight of wet sludge in the settling basin

$$= 1155 \text{ gm} = 1.155 \text{ kg}$$

($\because$ 1 gm of solids in the wet sludge mean 100 gm of wet sludge)

Since this sludge largely contains water (99%) its unit weight can be assumed to be that of water *i.e.* 1000 kg/m^3.

Since volume of this wet sludge

$$= \frac{1.155 \text{ kg}}{1000 \text{ kg/m}^3} = \mathbf{0.001155 \ m^3}$$

$$= 1.155 \text{ litres} \approx \mathbf{1.16 \ litres}$$

Hence, *volume of sludge produced per cu m of raw water:*

In the settling tank = 9.9 litres/m^3 of raw water

In the filter wash = 1.16 litres/m^3 of raw water

15. Design six slow sand filter beds from the following data:

Population to be served = 50,000 persons.

Per capita demand = 150 litres/head/day

Rate of filtration = 180 litres/hour/sq. m

Length of each bed = Twice the breadth.

Assume max. demand as 1.8 times the average daily demand. Also assume that one unit, out of six, will be kept as stand-by.

Solution. Average daily demand

$$= \text{Population} \times \text{Per capita demand}$$

$$= 50{,}000 \times 150 \text{ litres/day}$$

$$= 7.5 \times 10^6 \text{ litres/day}$$

Maximum daily demand

$$= 1.8 \times 7.5 \times 10^6 = \text{litres/day}$$

Rate of filtration per day

$$= (180 \times 24) \text{ litres/sq m/day}$$

Total surface area of filters required

$$= \frac{\text{Maximum daily demand}}{\text{Rate of filtration per day}}$$

$$= \frac{13.5 \times 10^6}{180 \times 24} \text{m}^2 = 31.25 \text{ m}^2$$

Now, six units are to be used; out of them, one is to be kept-as stand-by, and hence only 5 units should provide the necessary area of filter required.

$\therefore$ Area of each filter unit

$$= \frac{1}{5} \times \text{ Total area required}$$

$$= \frac{1}{5} \times 31.25 = 6.25 \text{ m}^2$$

Now, if L is the length and B is the breadth of each unit, then L = 2B (given)

$\therefore$ $\quad 2B.B = 6.25$ sq m.

or $\quad B^2 = 312.5$ sq m

or $\quad B = 17.7$ m; say **18 m**

$\therefore$ $\quad L = 2(17.75) =$ **36m.**

Hence, use 6 filter units with one unit as stand-by, each unit of size 36 × 18 m, arranged in series with 3 units on either side.

16. A city is to treat 7000 m^3/day of water. Laboratory column analysis indicates that an overflow rate of 20 m/day will produce satisfactory removal at a depth of 3.5 m. Determine the size of the required sedimentation tank and the detention time in hours. (Assume length to width ratio of 3).

Solution. Overflow rate $= \dfrac{Q}{BL} = 20$ m/day,

$\Rightarrow$ $\quad Q = 20 \text{ BL m}^3\text{/day}$

Depth = 3.5 m

$$\text{Detention time} = \frac{BLH}{Q} = \frac{B.L. \times 3.5}{20 \text{ B.L.}}$$

$$= \frac{3.5}{20} \text{day} = \frac{3.5}{20} \times 24 \text{ hours}$$

$$= 4.2 \text{ hours}$$

Quantity of water to be treated in 4.2 hours, *i.e.*

$$\text{Capacity of tank} = 7000 \times \frac{4.2}{24} \text{m}^3 = 1225 \text{ m}^3$$

Now $\quad$ B.L.H. = V

or $\quad$ B.L. 3.5 = 1225

or $\quad$ B.L. = 350 m^2

Assuming, $\quad$ L = 3B, We have

$3 B^2 = 350$

$\therefore$ $\quad$ B = 10.8 m

and $\quad L = 3 \times B = 3 \times 10.8 =$ **32.4 m.**

17. A city has to treat 24 MLD of turbid water using rapid sand filters with a filtration rate of 5 m^3/h/m^2. Determine the size of filter bed if L:B = 2:1. Only one unit of filter is to be provided.

Solution. Total quantity of water to be treated

$$= 24\text{MLD}$$

$$= 24 \times 10^6 \times \frac{10^{-3}}{24} \text{ m}^3\text{/hour}$$

$$= 1000 \text{ m}^3\text{/h}$$

Total area of filter bed required

$$= \frac{\text{Quantity of water treated}}{\text{Rate of filtration}}$$

$$= \frac{1000}{5} = 200 \text{ m}^2$$

Now $L = 2B$

$\therefore$ $2B^2 = 200$

or $B = 10$ m

$\therefore$ $L = 2 \times B = 20$ m

Provide one number of rapid sand filter, with the size of unit as 10×20 m.

18. In the treatment of 25×10^3 m^3/day of water, the amount of chlorine used is 15 kg/day. The residual chlorine after 10 min contact is 0.20 mg/l. Determine the chlorine demand in mg/l.

Solution. Amount of chlorine used = 15kg/day

Quantity of water to be treated = 25×10^3 m^3/day

Residual chlorine after 10 min contact = 0.2 mg/l

$$= \frac{0.2 \times 10^{-6}\,\text{kg}}{10^{-3}\ \text{m}^3} \times 25 \times 10^3 \,\frac{\text{m}^3}{\text{day}}$$

$$= 0.2 \times 25 \text{ kg/day}$$

$$= 5 \text{ kg/day}$$

Chlorine demand is defined as the difference between the amount of chlorine used to treat water and the amount of chlorine remaining after the specified contact period.

$\therefore$ Chlorine demand = **15 – 5 = 10 kg/day**

19. A filter unit is 4.5 m by 9.0 m. After filtering 10,000 cubic metre per day in 24 hours period, the filter is back washed at a rate of 10 *l*/sq m/sec. for 15 min. Compute the average filtration rate, quantity, and percentage of treated water used in washing, and the rate of wash water flow in each trough. Assume 4 troughs.

Solution.

Area of' filter = 4.5×9.0 m^2 = 40.5 m^2

Filtered quantity in 24 hrs = 10,000 cum/day.

$$\text{Area of filter} = \frac{\text{Water filtered}}{\text{Rate of filtration}}$$

Rate of filtration in *l*/hr/m^2

$$= \frac{10{,}000 \times 1000}{24 \times 40.5}\, l/\text{hr/m}^2$$

$$= 10{,}288\ l/\text{hr/m}^2$$

Average rate of filtration (by counting $\frac{1}{2}$ hr. lost in cleaning)

$$= \frac{10{,}288}{24.5} \times 24$$

$$= \mathbf{10{,}078\ \mathit{l}/hr/m^2}.$$

Amount of water used in cleaning @ 10 *l*/m^2/sec for 15 min

$$= 10 \times (4.5 \times 9.0) \times (15 \times 60)\ l$$

$$= 364500 \text{ litres} = 364.5 \text{ m}^3$$

Quantity of wash water expressed as percentage of total filtered water

$$= \frac{364.5 \times 100}{10{,}000} = 3.645\%$$

Wash water discharge through each trough

$$= \frac{\text{Total wash water discharge through the filter}}{\text{No. of troughs}}$$

$$= \frac{10\ l/\text{m}^2/\text{sec} \times (4.5 \times 9)\ \text{m}^2}{4} = 101.25\ \mathbf{\mathit{l}/sec.}$$

20. Chlorine usage in the treatment of 20,000 cubic metre per day is 18 kg/day. The residual after 10 min. contact is 0.20 mg/l. Calculate the dosage in milligrams per litre and chlorine demand of the water.

Solution. Water treated per day = 20,000 cum.

$= 20{,}000 \times 10^3$

$= 20 \times 10^6$ litres

$= 20$ M.l.

Chlorine consumed per day = 8 kg = 8 M.mg

$\therefore$ Chlorine used per litre of water

$$= \frac{8 \text{ M.mg}}{20 \text{ M.l}} = 0.4 \text{ mg/l}$$

Hence, the given chlorine dosage = 0.4 mg/l.

Given, residual chlorine left = 0.2 mg/l

Hence, actual chlorine dosage, which has reacted in water, *i.e.*

chlorine demand of water = 0.4 - 0.2 = **0.2 mg/l**

DISTRIBUTION SYSTEM

1. For a town with a population of 2 lakhs, a water supply scheme is to be designed. The maximum daily demand may be assumed as 200 litres/capita/day. The storage reservoir is situated 5 km away from the town. Assuming loss of head from source to the town as 10 m and coefficient of friction for the pipe material as 0.012, recommend the size of the supply main. 50% of daily demand has to be pumped in 8 hours for the proposed scheme.

Solution. Maximum water demand per day

$= 2{,}00{,}000 \times 200$ l/Day = 40 MLD

Maximum water demand for which supply main is to be designed

$= 50\% \times 40$ ML per 8 hr

$$= \frac{20 \times 10^6}{10^3} \text{m}^3/8 \text{ hr}$$

$$= \frac{20 \times 10^3}{8 \times 60 \times 60} \text{ m}^3\text{/s}$$

$$= 0.694 \text{ m}^3\text{/s}$$

For supply main,

$$Q = 0.70 \text{ m}^3\text{/s}, f' = 0.012$$

$$L = 5 \text{ km} = 5000 \text{ m}, H_L = 10 \text{ m}$$

We have, $$H_L = \frac{f' L V^2}{2gd}$$

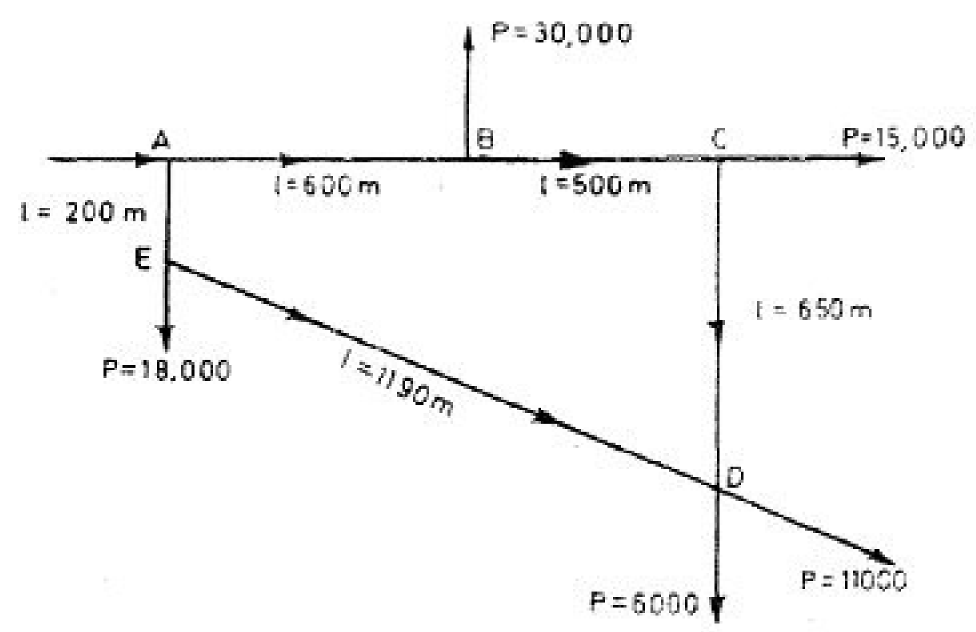
where, $$V = \frac{Q}{A} = \frac{0.694}{\frac{\pi}{4} d^2}$$

$$\therefore \quad 10 = \frac{0.012 \times 5000 \times \left(\frac{0.694}{\frac{\pi}{4} d^2} \right)}{2 \times 9.81 \times d}$$

Solving we get, diameter of supply main,

d = **0.75 m**.

2. Figure given below shows one of the circuits of a distribution system, along with the populations (P) to be served at different points.



The pressure at the start point A is 35 m head of water and the min. pressure desired at the point D is 20 m head of water. Design the various pipes of this circuit by assuming the per capita demand to be 190 litres/person/day, and the peak rate of demand for the design of distribution system as 2.7 times the average.

Solution. Average demand =190 litres/person/day

Maximum demand for which distribution pipes are to be designed

$= 2.7 \times 190 = 513$ litres/person/day.

The off-take discharges required at various points to serve the given populations are:

$$\text{At point B} = \frac{513 \times 20{,}000}{24 \times 60 \times 60} = 119 \ l/s$$

$$\text{At point C} = \frac{513 \times 15{,}000}{24 \times 60 \times 60} = 89 \ l/s$$

$$\text{At point D along CD} = \frac{513 \times 6{,}000}{24 \times 60 \times 60} = 36 \ l/s$$

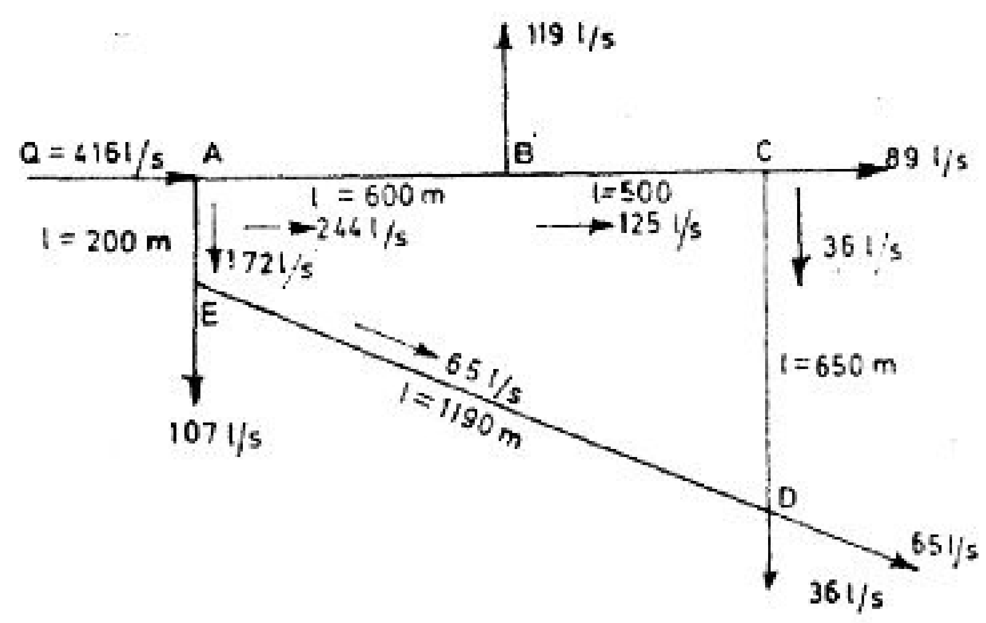
$$\text{At point D along CD} = \frac{513 \times 11{,}000}{24 \times 60 \times 60} = 65 \ l/s$$

$$\text{At point E} = \frac{513 \times 18{,}000}{24 \times 60 \times 60} = 107 \ l/s$$

$$\text{Total} = 416 \ l/s$$

$\therefore$ Total discharge input (inflow) required at A = 416 l/s.

The flow discharges (their magnitudes as well as directions) are now assumed in all the pipes, keeping in consideration the law of continuity (i.e. the input must equal the offtake (outflow) at each junction). The flows have been assumed as shown in the figure below. On the assumption that the balancing point lies near the point D in the system (i.e. all the demand along the ABCD will be satisfied by the water flowing via AB, and all the demand along the route AED will be satisfied by the water flowing along AE.



The sizes of the pipes should now be such as the velocities produced in them are somewhere between optimum values (i.e. 0.9 to 1.5 m/sec) and the total head loss from A to D, via either of the two routes (i.e. ABCD or AED) remains approximately equal, and less than the available head of 35 – 20 = 15 m.

Size of the pipes are assumed as shown in col. (2) of Table. The William-Hazen's nomogram is used to determine the head losses instead of using the formula, so as to facilitate calculations.

Route	Pipe	Discharge flowing in l/s	Proposed diameter of pipe in mm	Length in metres L	Loss of head per 1000 m in metres	H_L = Loss of head in pipe of length $L = \frac{col.(5) \times col.(6)}{1000}$	$\left\lvert\frac{H_L}{Q_a}\right\rvert$ in $m/cumec$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	AB	244	450	600	8	4.8	$\frac{4.80}{0.244} = 19.7$
ABCD	BC	125	350	500	8	4.0	$\frac{4.0}{0.125} = 32.0$
	CD	36	220	650	8	5.2	$\frac{5.2}{0.036} = 144.2$
Sum						(+) 14.0	196.2
	AE	172	380	200	10	2.0	$\frac{2.0}{0.172} = 11.6$
AED							
	ED	65	280	1190	10	11.9	$\frac{11.9}{0.065} = 183$
Sum						(–) 13.9	194.6
	Grand Total						390.8

Using Hardy-Cross equation, and Table, we have

Correction to discharges,

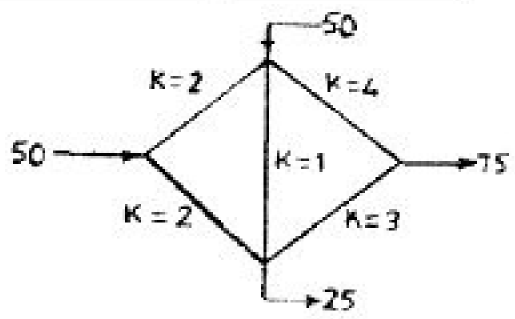
$$\Delta_1 = \frac{14.0 - 13.9}{1.85 \times (390.8)} \text{ cumecs}$$

$$= \frac{0.1 \times 1000}{1.85 \times 390.8} \, l/s$$

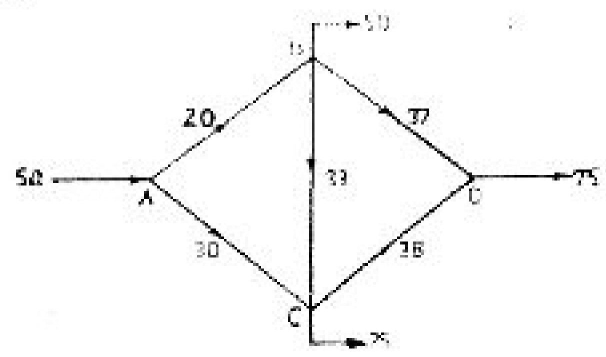
$$= 0.14 \, l/s.$$

The correction to be applied is too small and hence the adopted diameters are O.K.

3. Determine the distribution of flow in the pipe network shown. The head loss, h_l, may be assumed as kQ^n. The flow is turbulent and pipes are rough. The value of k for each pipe is indicated in the figure. Use Hardy-Cross method.



Solution. Assume a suitable distribution of flow as shown

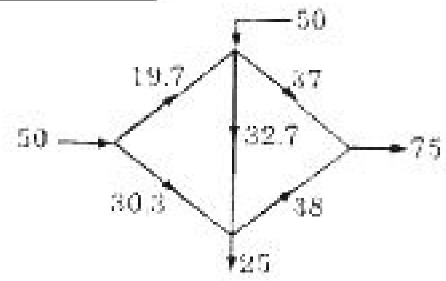


Computing h_L for circuits ABC and BCD, assuming clockwise flows as positive

TABLE Circuit ABC		TABLE Circuit BCD	
kQ^2	$2kQ$	kQ^2	$2kQ$
$2 \times 10^2 = 800$	$2 \times 2 \times 10 = 80$	$4 \times 37^2 = 5476$	$2 \times 4 \times 37 = 296$
$1 \times 33^2 = 1089$	$2 \times 1 \times 33 = 66$	$-3 \times 38^2 = -4332$	$2 \times 3 \times 38 = 228$
$-2 \times 30^2 = -1800$	$2 \times 2 \times 30 = 120$	$-1 \times 33^2 = -1089$	$2 \times 1 \times 33 = 66$
$\Sigma kQ^2 = 89$	$\Sigma 2\,kQ = 266$	$\Sigma kQ^2 = 55$	$\Sigma 2\,kQ = 590$

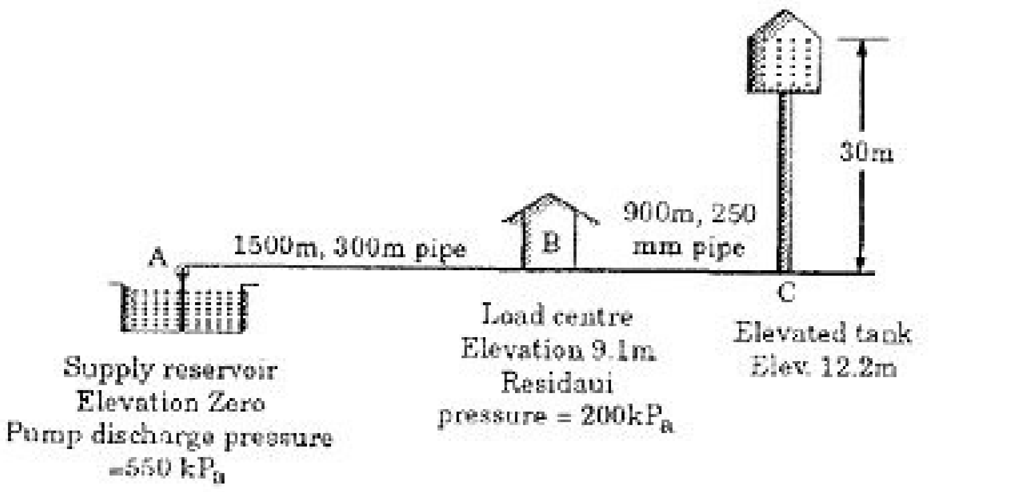
For circuit ABC, $\quad \Delta Q = \dfrac{-\Sigma kQ^2}{\Sigma 2kQ} = \dfrac{-89}{266} = -0.33$

For circuit BCD, $\quad \Delta Q = \dfrac{-\Sigma kQ^2}{\Sigma 2kQ} = \dfrac{-55}{590} = -0.09$



Here the correction in the circuits BCD is negligible and that in circuit ABC is of order of 0.3. So necessary correction can be made in circuit ABC. Hence revised circuit is

4. A water supply system consisting of a reservoir with lift pump, elevated storage tank, piping, and withdrawal points is shown in the diagram



(i) Based on the following data, sketch the hydraulic gradient for the system.

$Z_A = 0$ m, $P_A = 550$ kPa,

$Z_B = 9.1$ m, $P_B = 200$ kPa,

$Z_C = 12.2$ m,

$P_C = 30$ m (water level in the tank).

(ii) For these conditions, compute the flow available at point B from both the supply pumps and elevated storage. Use C = 100 and pipe sizes as shown in the diagram. Hydraulic values are given below.

TABLE : Hydraulic Values for C = 100

d (mm)	h_L *(m/1000)*	*Q (l/s)*	*V (m/s)*
300	17.8	133	1.7
250	14.1	117	1.5
200	20.0	48	1.6
150	30.0	30	1.7
100	40.0	13	1.5

Solution. The pressure given at A and B is in S.I. units of pressure.

The S.I. unit of pressure is kilo pascal (*k*Pa).

$$1 \text{ pascal} = 1 \text{ N/m}^2$$

$$1k\text{Pa} = 1000 \text{ N/m}^2$$

$$= \frac{1000}{9.81} \text{ kg/m}^2 = \frac{1000}{9.81} \times \frac{1}{10^4} \text{ kg/cm}^2$$

$$= \frac{1}{9.81} \text{ kg/cm}^2 = \frac{1}{98.1} \times 10 \text{ m of water}$$

$$= \frac{1}{9.81} \text{ m of water}$$

$$\therefore \quad 100\,k\text{Pa} = \frac{100}{9.81} \text{ m of water}$$

$$= 10.1937 \text{ m of water.}$$

Here, Pressure at

$$A = P_A = 500\,k\text{Pa}$$

$$= \frac{550}{100} \times 10.1937 \text{ m (of water)}$$

$$= 56.065 \text{ m} = \mathbf{56.7\ m} \text{ (say)}$$

Similarly, the residual pressure at

$$B = P_B = 200\,k\text{Pa}$$

$$= \frac{200}{100} \times 10.1937 \text{ m (of water)}$$

$$= 20.37 \text{ m} = \mathbf{20.39\ m} \text{ (say)}$$

Also $Z_A = 0$; $Z_B = 9.1$

$\therefore$ Total head (*i/c* datum head) at

$$A = Z_A + P_a = 0 + 56.07 = \mathbf{56.07\ m} \quad ...(i)$$

Similarly total head (*i/c* datum head) at

$$B = Z_B + P_b = 9.1 + 20.39 = \mathbf{29.49\ m} \quad ...(ii)$$

Similarly total head (*i/c* datum head) at

$$C = Z_C + P_c = 12.2 + 30 = \mathbf{42.2\ m} \quad ...(iii)$$

Now we can easily draw the H.G. line, as shown in the figure below.

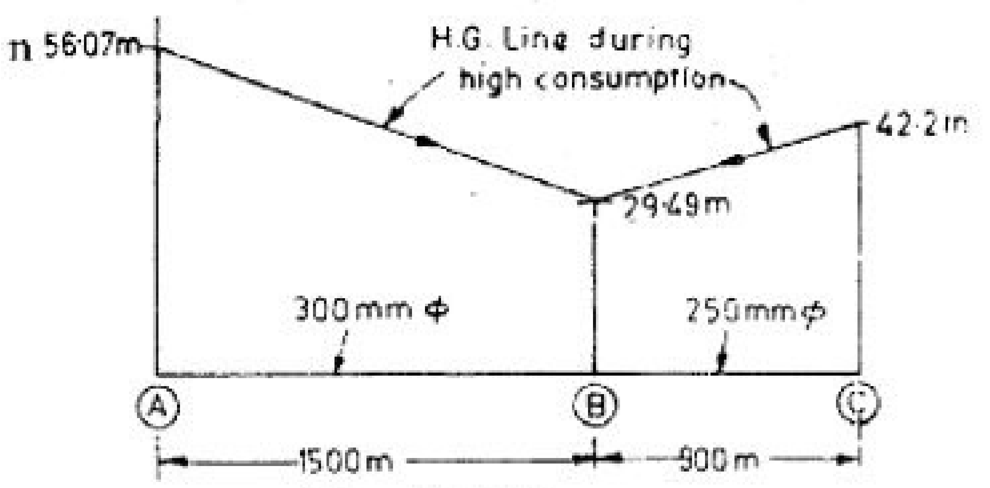


Fig. H.G. line

Now, for pipe AB of 300 mm ϕ,

S = gradient of energy line

$$= \frac{56.07 - 29.49}{1500} \times 1000 \text{ m/1000 m length}$$

$$= 17.72 \text{ m/1000 m}$$

$$\approx 17.8 \text{ m/1000 m. (say)}$$

The given table of hydraulic values of the pipe shows that for h_l of 17.8 m/1000 m, and 300 mm ϕ pipe, the discharge is 133 l/s. Hence, discharge at B due to AB *i.e.* due to pumps = **133 l/s.** Similarly, the gradient of energy line for pipe CB of 250 mm ϕ

$$= \frac{42.2 - 29.47}{900} \times 1000 \text{ m/1000 m length}$$

$$= 14.1 \text{ m/1000 m}$$

From the given table, for a pipe of 250 mm and h_L of 14.1 m/1000 m, discharge = 117 *l*/s.

Hence, discharge at B caused by the storage reservoir = **117 l/see.**

Hence, the flow available at point B from pumps is 133 l/sec, and from the elevated storage is 117 l/sec, respectively, giving a total flow of 250 l/s. **Ans.**

CONDUITS AND PUMPS

1. For water supply of a town, water is pumped from a river 3 km away into a reservoir. The maximum difference of levels of water in river and the reservoir is 20 m. The population of the town is 50,000 and per capita water demand is 120 litre per day. If the pumps are to operate for a total of 8 hours and the efficiency of pump is 80%, determine the horsepower of the pumps. Assume friction factor as 0.03, the velocity of flow as 2 m/s and maximum daily demand as 1.5 times the average daily demand.

Solution. Given, l = 3000 m, h = 20 m, η = 80%, f = 0.03, v = 2 m/sec.

Pumps operate for 8 hours, Population = 50,000

Per capita water demand = 120 litres/day

Maximum daily demand = 1.5 times Average daily

Rate of flow, $Q = \dfrac{(50{,}000 \times 120)1.5}{1000 \times 8 \times 60 \times 60} = 0.3125$ cumec

Diameter of pipe, $d = \sqrt{\dfrac{4Q}{\pi v}} = \sqrt{\dfrac{4 \times 0.3125}{\pi \times 2}} = 0.446$ m

Head loss along pipeline,

$$h_L = \frac{fLv^2}{2gd} = \frac{0.03 \times 3000 \times 2^2}{2 \times 9.81 \times 0.446}$$

$$= 41.14 \text{ m}$$

Total head, h_m = 41.14 + 20 = 61.14 m

$$\text{Horse power of pumps} = \frac{\rho Q h_m}{75\eta}$$

$$= \frac{1000 \times 0.315 \times 61.14}{75 \times 0.8}$$

$$= \mathbf{318.44\ hp}$$

2. In a pumping station, 18,000 cum water is to be raised per day from an intake well to a Sedimentation tank under the static head of 21 m. Length of suction pipe and rising main are 40 m and 150 m respectively. Diameter of pipes is 50 cm. There are two shifts of working pumps each of 8 hours. Take coefficient of friction as 0.01, and combined efficiency of motor and pump as 80%. Recommend the number of units of pumps each having B.H.P. of 30.

Solution. Given, Q = 18000 m^3/day,

Static head = 21 m

Length of suction pipe + Rising main = 40 + 150 = 190 m

Diameter of pipes = 50 cm, f = 0.01

Efficiency of motor + Pump = 80%, B.H.P. of each unit = 30

With two 8 hours shifts of pumps,

$$Q = 0.3125 \text{ cumec}$$

Head loss due to friction,

$$h_L = \frac{fLQ^2}{12.10d^5} = \frac{.01 \times 190 \times .3125^2}{12.10(.5)^5}$$

$$= 0.49 \text{ m}$$

Power of pumps required,

$$P = \frac{wQh_m}{75} = \frac{1000 \times .3125 \times 21.49}{75 \times .80}$$

$$= 111.98 \text{ H.P.}$$

$\therefore$ Number of pumps required,

$$N = \frac{111.93}{30} = \mathbf{4}$$

3. A pump is to deliver water from an underground tank against a static head of 40 m. The suction pipe is 50 m long and is of 25 cm diameter with Darcy-weisbach friction factor f = 0.02. The delivery pipe is of 20 cm diameter, 1600 m long and has f = 0.022. The pump characteristics can be expressed as :

$$H_p = 00 - 6000\,Q^2$$

where Hp = pump head in metres; and

Q = discharge in m^3/s

Calculate the head and discharge of the pump.

Solution. Static head,

$$H_1 = 40 \text{ m.}$$

Head loss in suction pipe,

$$H_2 = \frac{fLQ^2}{2g.\frac{\pi}{16}.d^5} = \frac{0.02 \times 50 \times Q^2 \times 16}{2 \times 9.81 \times \pi^2 \times (0.25)^2}$$

$$= 84.68\,Q^2$$

Head loss in delivery pipe,

$$H_3 = \frac{0.022 \times 1600\,Q^2 \times 16}{2 \times 9.81 \times \pi^2 \times (0.2)^5} = 90.98\,Q^2$$

Total head loss,

$$H_p = H_1 + H_2 + H_3$$

$$= 40 + 84.68\,Q^2 + 90.98\,Q^2$$

$$= 40 + 175.66\,Q^2$$

Using given equation of pump, we have

or $\quad H_p = 1000 - 6000\,Q^2$

or $\quad 40 + 175.66\,Q^2 = 100 - 6000\,Q^2$

$\quad 6175.66\,Q^2 = 60$

or $\quad Q = \sqrt{\dfrac{60}{6175.66}} = \mathbf{0.0986\ cumecs.}$

$$H_P = 1000 - 6000\,.\,(0.0986) = \mathbf{41.7}$$

4. Water is pumped from a low level reservoir to a high level reservoir through a main pipeline of 0.45 m diameter and 1400 m length. The pump is located at low level reservoir. At a point along the main line at a distance of 450 m from the high level reservoir, a branch line of 0.3 m diameter and 360 m length takes off the discharge 180 l/sec in the atmosphere

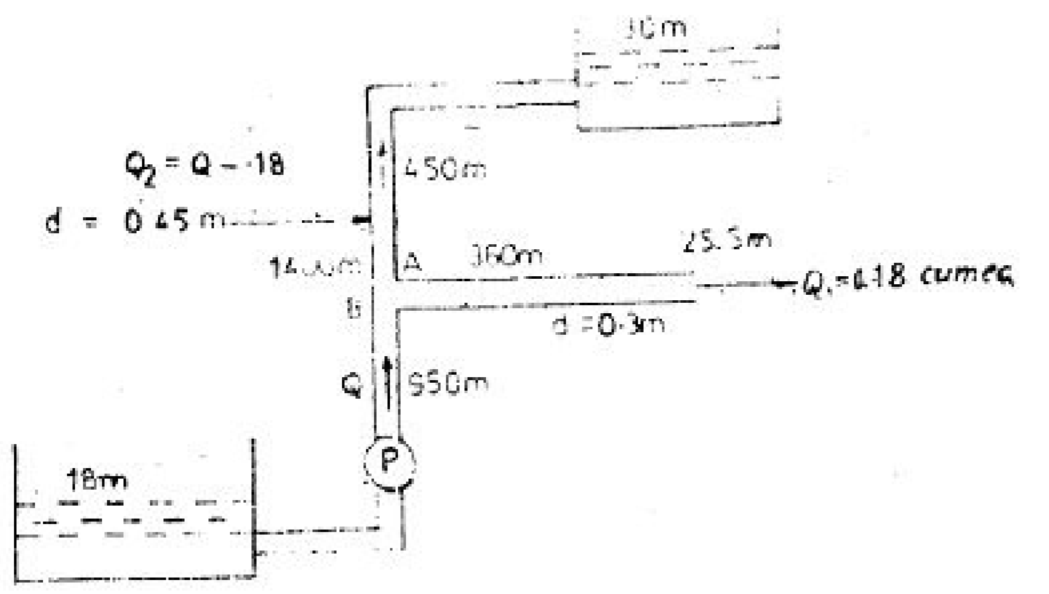
Level of water surface in high level reservoir = 30.0 m

Level of water in the open end of the 0.3 m diameter branch line = 25.5 m

Level of water in the low level reservoir = 18.0 m

Darcy's frictional coefficient for both pipe = 0.032

Determine the flow rate into the high level reservoir and the theoretical H.P. for the pump assuming the delivery value of the pump to be at + 20.00 m.



Solution. $f = 0.032$

Head loss in branch pipe,

$$h_L = \frac{fLQ^2}{12.10\,d^5}$$

$$= \frac{0.032 \times 360 \times 0.18^2}{12.10(.3)^5}$$

$$= 12.69 \text{ m}$$

$$\text{Velocity at exit of branch} = \frac{0.18 \times 4}{\pi \times .3^2}$$

$$= 2.546 \text{ m/sec}$$

$$\text{Energy head at exit} = z + \frac{p}{w} + \frac{v^2}{2g}$$

$$= 0 + 0 + \frac{2.546^2}{19.62} = 0.33 \text{ m}$$

Energy head at A in Branch pipe

$$= 12.69 + 0.33 = 13.02 \text{ m}$$

Total head at A = 13.02 + 25.5 = 38.52 m

i.e. Energy head at B in main pipe = 38.52 m

Loss of head in upper portion of main pipe,

$$h_L = 38.52 - 30.0 = 8.52 \text{ m}$$

Now $$h_L = \frac{0.032 \times 450 \times Q_2^2}{12.10(.45)^5} = 8.52$$

or $$Q_2 = 0.3634 \text{ cumec.}$$

Again $$Q = Q_1 + Q_2 = .3634 + .18$$

$$= 0.5434 \text{ cumec.}$$

Head loss in bottom 950 m,

$$h_L = \frac{0.032 \times 950 \times 0.5434^2}{12.10(.45)^5} = 40.20 \text{ m}$$

Total head required along main pipe

$$= 12 + 40.2 + 8.52 + \frac{2.29^2}{19.62} = 61 \text{ m}$$

Along Branch = 7.5 + 40.2 + 12.69 + 0.33

$$= 60.72 \text{ m}$$

H.P. of pump required

$$= wQh_m = \frac{9810 \times .5434 \times 61}{1000}$$

$$= \mathbf{325.17\ kW}$$

5. For water supply of a town, water is pumped from a river 3 km away into a reservoir. The maximum difference of levels of water in river and the reservoir is 20 m. The population of the town is 50,000 and per capita water demand is 120 litre per day. If the pumps are to operate for a total of 8 hours and the efficiency of pumps is 80%, determine the horsepower of the pumps. Assume friction factor as 0.03, the velocity of flow as 2 m/s, and maximum daily demand as 1.5 times the average daily demand:

Solution. Population of town = 50,000; Per capita demand = 120 1/day

$$\therefore \text{ Average water required} = \frac{50{,}000 \times 120}{1000} \text{ m}^3\text{/day}$$

$$= 6000 \text{ m}^3\text{/day}$$

Maximum daily water required = 1.5 × 6000 m³/day = 9000 m³/day

The above quantity has to be pumped from the river in 8 hours to the reservoir.

$$\therefore \text{ Water lifted per second} = \frac{9000}{8 \times 60 \times 60} \text{ m}^3\text{/day}$$

$$= 0.3125 \text{ m}^3\text{/s}$$

Given, velocity through the pipe lifting the above discharge = 2 m/s

Now, velocity × area = discharge

or $$2\left(\frac{\pi}{4}d^2\right) = 03125$$

$$\therefore \text{ Diameter of pipe, } d = \sqrt{\frac{0.3125 \times 4}{2\pi}}$$

$$= 0.446 \text{ m; say } 0.45 \text{ m.}$$

or $H_L = 0.0026L$

$\therefore$ Head loss from source to service reservoir in a length of 1.8 km (or 1800 m)

$$= 0.0026 \times 1800 = 4.68 \text{ m.}$$

Head difference between the elevation of sump well and service reservoir = 36 m (given).

Head loss in supply main =4.68 m

Total head lift required = 36 + 4.68 =40.68 m.

Brake horse power of pumps required (metric),

Assuming, η = efficiency of set = 0.65, we get

$$\text{H.P.} = \frac{w.\text{Q.H}}{75\eta} = \frac{1000 \times 0.624 \times 40.68}{75 \times 0.65}$$

$$= \mathbf{522 \text{ H.P.}}$$

Note. The horse power of the pumps can be reduced by reducing the head loss (at present 4.68 m) in conveyance by choosing smaller velocity and, thus, bigger sized pipe. But in that case, the cost of pipe shall go up. That is why an optimum velocity of about 1.2 to 2.6 m/sec. is generally adopted. In fact, it is a question of economics and the economy can be worked out by designing pipes at various velocities and the cheapest velocity chosen, keeping in consideration its non-silting and non-scouring value.

9. A pipe line 0.6 m diameter is 1.5 km long. To augment the discharge, another pipe line of the same diameter is introduced parallel to the first in the second half of its length. Find the increase in discharge, if f = 0.04 and head at the inlet is 30 m.

Solution.

From Darcy Weisbach equation,

$$H_L = \frac{f'\text{L.V}^2}{2gd} = \frac{f'.\text{L.}\left(\dfrac{\text{Q}}{\dfrac{\pi}{4}d^2}\right)^2}{2gd}$$

$$= \frac{8}{g\pi^2}.\frac{f'\text{ L.Q}^2}{d^5}$$

In the first case, when a single pipe line is laid, if the discharge is Q_1, then

H_L = 30m, d = 0.6 m, f' = 0.04, L = 1.5 km = 1500 m

$$\therefore \quad 30 = \frac{8}{9.81 \times (3.14)^2} \frac{0.04 \times 1500 \times Q_1^2}{(0.6)^5}$$

or Q_1 = 0.686 cumecs.

In the second case, when additional pipe is laid in the second half length of the pipe, assume that the discharge Q_2, then this discharge Q_2 flows through original pipe for a length

$$= \frac{1}{2}.1500 = 750 \text{ m}$$

and then half of the discharge $\left(i.e. \dfrac{Q_2}{2}\right)$ flows through each, as both pipes are of equal diameter.

Total head loss = 30 = Head loss in a length of 750 m with Q_2 discharge + Head loss in a length of 750 m with $\dfrac{Q_2}{2}$ discharge

$$= \left[\frac{8}{g\pi^2}.\frac{0.04 \times 750 (Q_2)^2}{(0.6)^5}\right] + \left[\frac{8}{g\pi^2} \frac{0.04 \times 750 \times \left(\dfrac{Q_2}{2}\right)^2}{(0.6)^5}\right]$$

$$= \frac{8}{g\pi^2}.\frac{0.04 \times 750}{(0.6)^5}\left[Q_2^2 + \frac{Q_2^2}{4}\right]$$

$$\text{or} \quad \frac{5}{4}Q_2^2 = \frac{30 \times 9.81 \times (3.14)^2 \times (0.6)^5}{8 \times 0.04 \times 750} = 0.940$$

or Q_2 = 0.857 cumecs.

Hence, increase in discharge

$$= Q_2 - Q_1$$

$$= 0.867 - 0.686 = \mathbf{0.181 \text{ cumecs}}.$$

13. A water supply main trifurcates at junction point J into three branches each feeding a separate reservoir. The details of the pipes and reservoirs are as follows:

Pipe	*Diameter*	*Length*	*Darcy Weisbach friction factor (f)*	*Terminal Reservoir*	*Reservoir elevation (m)*
JA	20 cm	2.0 km	0.02	A	80.00
JB	20 cm	2.5 km	0.02	B	70.00
JC	20 cm	3.0 km	0.02	C	60.00

If the inflow from the main at the junction is 0.250 m^3/s, determine the delivery into each reservoir.