Water & Wastewater Engineering

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Worked-out Examples:

Population Forecast by Different Methods

Problem: Predict the population for the years 1981, 1991, 1994, and 2001 from the following census figures of a town by different methods.

Year	1901	1911	1921	1931	1941	1951	1961	1971
Population: (thousands)	60	65	63	72	79	89	97	120

Solution:

Year	Population: (thousands)	Increment per Decade	Incremental Increase	Percentage Increment per Decade
1901	60	-	-	-
1911	65	+5	-	(5+60) x100=+8.33
1921	63	-2	-3	(2+65) x100=-3.07
1931	72	+9	+7	(9+63) x100=+14.28
1941	79	+7	-2	(7+72) x100=+9.72
1951	89	+10	+3	(10+79) x100=+12.66
1961	97	+8	-2	(8+89) x100=8.98
1971	120	+23	+15	(23+97) x100=+23.71
Net values	1	+60	+18	+74.61
Averages	-	8.57	3.0	10.66

+=increase; - = decrease

Arithmetical Progression Method:

$$P_n = P + ni$$

Average increases per decade = i = 8.57

Population for the years,

1981= population 1971 + ni, here n=1 decade

= 120 + 8.57 = 128.57

1991= population 1971 + ni, here n=2 decade

= 120 + 2 x 8.57 = 137.14

2001= population 1971 + ni, here n=3 decade

= 120 + 3 x 8.57 = 145.71

1994= population 1991 + (population 2001 - 1991) x 3/10

= 137.14 + (8.57) x 3/10 = 139.71

Incremental Increase Method:

Population for the years,

1981= population 1971 + average increase per decade + average incremental increase

= 120 + 8.57 + 3.0 = 131.57

1991= population 1981 + 11.57

= 131.57 + 11.57 = 143.14

2001= population 1991 + 11.57

= 143.14 + 11.57 = 154.71

1994= population 1991 + 11.57 x 3/10

= 143.14 + 3.47 = 146.61

Geometric Progression Method:

Average percentage increase per decade = 10.66

$$P_n = P\,(1+i/100)^n$$

Population for 1981 = Population 1971 x $(1+i/100)^n$

= 120 x (1+10.66/100), *i = 10.66, n = 1*

= 120 x 110.66/100 = 132.8

Population for 1991 = Population 1971 x $(1+i/100)^n$

= 120 x $(1+10.66/100)^2$, *i = 10.66, n = 2*

= 120 x 1.2245 = 146.95

Population for 2001 = Population 1971 x $(1+i/100)^n$

= 120 x $(1+10.66/100)^3$, *i = 10.66, n = 3*

= 120 x 1.355 = 162.60

Population for 1994 = 146.95 + (15.84 x 3/10) = 151.70

Sedimentation Tank Design

Problem: Design a rectangular sedimentation tank to treat 2.4 million litres of raw water per day. The detention period may be assumed to be 3 hours.

Solution: Raw water flow per day is 2.4×10^6 l. Detention period is 3h.

Volume of tank = Flow x Detention period = $2.4 \times 10^3 \times 3/24 = 300\ m^3$

Assume depth of tank = 3.0 m.

Surface area = 300/3 = 100 m^2

L/B = 3 (assumed). L = 3B.

$3B^2 = 100\ m^2$ i.e. B = 5.8 m

L = 3B = 5.8 X 3 = 17.4 m

Hence surface loading (Overflow rate) = $\frac{2.4 \times 10^6}{100}$ = 24,000 l/d/m^2 < 40,000 l/d/m^2 (OK)

Rapid Sand Filter Design

Problem: Design a rapid sand filter to treat 10 million litres of raw water per day allowing 0.5% of filtered water for backwashing. Half hour per day is used for bakwashing. Assume necessary data.

Solution: Total filtered water = $\frac{10.05 \times 24 \times 10^6}{24 \times 23.5}$ = 0.42766 Ml / h

Let the rate of filtration be 5000 l / h / m^2 of bed.

Area of filter = $\frac{10.05 \times 10^6}{23.5} \times \frac{1}{5000}$ = 85.5 m^2

Provide two units. Each bed area 85.5/2 = 42.77. L/B = 1.3; $1.3B^2 = 42.77$

B = 5.75 m ; L = 5.75 x 1.3 = 7.5 m

Assume depth of sand = 50 to 75 cm.

Underdrainage system:

Total area of holes = 0.2 to 0.5% of bed area.

Assume 0.2% of bed area = $\frac{0.2}{100} \times 42.77 = 0.086\ m^2$

Area of lateral = 2 (Area of holes of lateral)

Area of manifold = 2 (Area of laterals)

So, area of manifold = 4 x area of holes = 4 x 0.086 = 0.344 = 0.35 m^2 .

$\therefore$ Diameter of manifold = $(4 \times 0.35/\pi)^{1/2}$ = 66 cm

Assume c/c of lateral = 30 cm. Total numbers = 7.5/ 0.3 = 25 on either side.

Length of lateral = 5.75/2 - 0.66/2 = 2.545 m.

C.S. area of lateral = 2 x area of perforations per lateral. Take dia of holes = 13 mm

Number of holes: $\frac{n\pi}{4}(1.3)^2 = 0.086 \times 10^4 = 860\ cm^2$

$\therefore n = \frac{4 \times 860}{\pi (1.3)^2} = 648$, say 650

Number of holes per lateral = 650/50 = 13

Area of perforations per lateral = $13 \times \pi (1.3)^2/4 = 17.24\ cm^2$

Spacing of holes = 2.545/13 = 19.5 cm.

C.S. area of lateral = 2 x area of perforations per lateral = 2 x 17.24 = 34.5 cm^2.

$\therefore$ Diameter of lateral = $(4 \times 34.5/\pi)^{1/2}$ = 6.63 cm

Check: Length of lateral < 60 d = 60 x 6.63 = 3.98 m. l = 2.545 m (Hence acceptable).

Rising washwater velocity in bed = 50 cm/min.

Washwater discharge per bed = (0.5/60) x 5.75 x 7.5 = 0.36 m^3/s.

Velocity of flow through lateral = $\frac{0.36}{\text{Total lateral area}} = \frac{0.36 \times 10^4}{50 \times 34.5}$ = 2.08 m/s (ok)

Manifold velocity = $\frac{0.36}{0.345}$ = 1.04 m/s < 2.25 m/s (ok)

Washwater gutter

Discharge of washwater per bed = 0.36 m^3/s. Size of bed = 7.5 x 5.75 m.

Assume 3 troughs running lengthwise at 5.75/3 = 1.9 m c/c.

Discharge of each trough = Q/3 = 0.36/3 = 0.12 m^3/s.

$$Q = 1.71 \times b \times h^{3/2}$$

Assume b =0.3 m

$$h^{3/2} = \frac{0.12}{1.71 \times 0.3} = 0.234$$

$\therefore$ h = 0.378 m = 37.8 cm = 40 cm

= 40 + (free board) 5 cm = 45 cm; slope 1 in 40

Clear water reservoir for backwashing

For 4 h filter capacity, Capacity of tank = $\frac{4 \times 5000 \times 7.5 \times 5.75 \times 2}{1000}$ = 1725 m^3

Assume depth d = 5 m. Surface area = 1725/5 = 345 m^2

L/B = 2; $2B^2$ = 345; B = 13 m & L = 26 m.

Dia of inlet pipe coming from two filter = 50 cm.

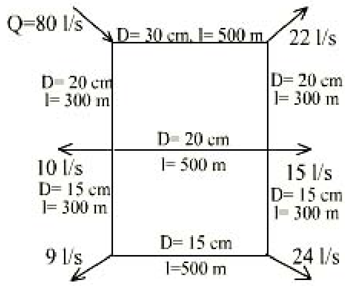
Velocity <0.6 m/s. Diameter of washwater pipe to overhead tank = 67.5 cm.

Air compressor unit = 1000 l of air/ min/ m^2 bed area.

For 5 min, air required = 1000 x 5 x 7.5 x 5.77 x 2 = 4.32 m^3 of air.

Flow in Pipes of a Distribution Network by Hardy Cross Method

Problem: Calculate the head losses and the corrected flows in the various pipes of a distribution network as shown in figure. The diameters and the lengths of the pipes used are given against each pipe. Compute corrected flows after one corrections.



Solution: First of all, the magnitudes as well as the directions of the possible flows in each pipe are assumed keeping in consideration the law of continuity at each junction. The two closed loops, ABCD and CDEF are then analyzed by Hardy Cross method as per tables 1 & 2 respectively, and the corrected flows are computed.

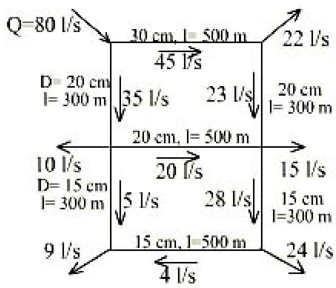


Table 1

Consider loop ABCD

Pipe	Assumed flow		Dia of pipe		Length of pipe (m)	K = $\frac{L}{470\, d^{4.87}}$	$Q_a^{1.85}$	$H_L = K.Q_a^{1.85}$	$\lvert H_L/Q_a \rvert$
	in l/sec	in cumecs	d in m	$d^{4.87}$					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
AB	(+) 43	+0.043	0.30	2.85 X10^{-3}	500	373	3 X10^{-3}	+1.12	26
BC	(+) 23	+0.023	0.20	3.95 X10^{-4}	300	1615	9.4 X10^{-4}	+1.52	66
CD	(-) 20	-0.020	0.20	3.95 X10^{-4}	500	2690	7.2 X10^{-4}	-1.94	97
DA	(-) 35	-0.035	0.20	3.95 X10^{-4}	300	1615	2 X10^{-3}	-3.23	92
Σ								-2.53	281

* $H_L = (Q_a^{1.85}L)/(0.094 \times 100^{1.85} \times d^{4.87})$
or $K.Q_a^{1.85} = (Q_a^{1.85}L)/(470 \times d^{4.87})$
or $K = (L)/(470 \times d^{4.87})$

For loop ABCD, we have $\delta = -\Sigma H_L / x.\Sigma |H_L/Q_a|$

$= (-)\ -2.53/(1.85 \times 281)$ cumecs

$= (-)\ (-2.53 \times 1000)/(1.85 \times 281)$ l/s

$= 4.86$ l/s $= 5$ l/s (say)

Hence, corrected flows after first correction are:

Pipe	AB	BC	CD	DA
Corrected flows after first correction in l/s	+ 48	+ 28	- 15	- 30

Table 2

Consider loop DCFE

Pipe	Assumed flow		Dia of pipe		Length of pipe (m)	$K = \frac{L}{470\ d^{4.87}}$	$Q_a^{1.85}$	$H_L = K.Q_a^{1.85}$	$\lvert H_L/Q_a \rvert$
	in l/sec	in cumecs	d in m	$d^{4.87}$					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DC	(+) 20	+0.020	0.20	3.95×10^{-4}	500	2690	7.2×10^{-4}	+1.94	97
CF	(+) 28	+0.028	0.15	9.7×10^{-5}	300	6580	1.34×10^{-3}	+8.80	314
FE	(-) 8	-0.008	0.15	9.7×10^{-5}	500	10940	1.34×10^{-4}	-1.47	184
ED	(-) 5	-0.005	0.15	9.7×10^{-5}	300	6580	5.6×10^{-5}	-0.37	74
Σ								+8.9	669

For loop ABCD, we have $\delta = -\Sigma H_L / x.\Sigma |H_L/Q_a|$

$= (-)\ +8.9/(1.85 \times 669)$ cumecs

$= (-)\ (+8.9 \times 1000)/(1.85 \times 669))$ l/s

$= -7.2$ l/s

Hence, corrected flows after first correction are:

Pipe	DC	CF	FE	ED
Corrected flows after first correction in l/s	+ 12.8	+ 20.8	- 15.2	- 12.2

Trickling Filter Design

Problem: Design a low rate filter to treat 6.0 Mld of sewage of BOD of 210 mg/l. The final effluent should be 30 mg/l and organic loading rate is 320 g/m^3/d.

Solution: Assume 30% of BOD load removed in primary sedimentation i.e., = 210 x 0.30 = 63 mg/l. Remaining BOD = 210 - 63 = 147 mg/l.
Percent of BOD removal required = (147-30) x 100/147 = 80%

BOD load applied to the filter = flow x conc. of sewage (kg/d) = 6 x 10^6 x 147/10^6 = 882 kg/d

To find out filter volume, using NRC equation

$$E_2 = \frac{100}{1+0.44(F_{1.BOD}/V_1.Rf_1)^{1/2}}$$

$$80 = \frac{100}{1+0.44(882/V_1)^{1/2}}$$ $Rf_1 = 1$, because no circulation.

$V_1 = 2704\ m^3$

Depth of filter = 1.5 m, Fiter area = 2704/1.5 = 1802.66 m^2, and Diameter = 48 m < 60 m

Hydraulic loading rate = 6 x $10^6/10^3$ x 1/1802.66 = 3.33$m^3/d/m^2$ < 4 hence o.k.

Organic loading rate = 882 x 1000 / 2704 = 326.18 g/d/m^3 which is approx. equal to 320.