BCS.

Environmental Engineering - 30.

Water Supply and Sewage

Prepared by

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BCS + Non-cadre

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Sanitary Engg বইটি must লাগবে.

Syllables:

Water supply:

→ Planning and design consideration of water treatment plant.
→ Various methods of water treatment. (sedimentation,
→ coagulation, filtration, dis-infection, chemical precipitation)
→ Design of water supply system.
→ Environmental Impact Assessment.

Sources: 1. Water supply and Sanitation by. M.A. Aziz. C-18, 15.5. 14
2. Water supply ~~and~~ Engineering S.K. garg.
3. ~~Water~~ Environmental Engg. Howard. S. Peavy.

Sewage:

→ Physical, chemical and Biological treatment of sewage.
→ Planning and design of sewage treatment
→ Industrial wastes and their treatment.
→ Microbiology of waste water.
→ solid waste management — ITN. chap 14.
→ Introduction to aerobic and anaerobic treatment of wast water.

→ self purification of stream BOD removal kinetics.

→ Design of domestic and storm sewers.

(Bangladesh standards P-338, ITN)

Book References:

i) ITN. water supply and sanitation.

C-14,

ii) Environmental Engineering Howard S. Peavy.

C-6, 2,

36th BCS.

15+3

1. (a) Enumerate the factors to be considered in planning and for designing a municipal water supply system. What do you meam by "per capita demand of water"? Describe briefly the factors affecting per capita water consumption. ITN-300, SK garg W.S. P-16. / ITN-326.

(b). What are the main purpose of EIA? 6+6

2. Write short notes on. i) aerobic and anaerobic decomposi of sewage. ii) Sanitary and storm sewer. iii) Solid waste management system. (P-260, ITN) P-349. Peavy.

(b). Design a septic tank for a small residential area of 300 persons with average daily sewer flow of 85 litre/per. Assume reasonable values for required design data. P-150. ITN.

35th BCS.

5+3+7

1. (a) Explain different methods of water distribution with relative advantages and disadvantages. P-391 ITN.

(b) Explain the factors to be kept in view in the design of water distribution system.

(P-390. ITN, P-661 Article-10.2 SK. garg

(c) Discuss the steps involved in calculating the flow in a looped network using Hardy cross method - P-400 ITN.

2. Write short notes on.

i) a) aerobic and anaerobic decomposition of sewage [36th]
b) Gray sewage and dark sewage.
c) Solid wast management — P-260. ITN.

ii) List the properties that are required to be analyzed if solid wastes are to be used as resources. P-265. IT

iii) Calculate BOD_{1day} at 37°C of a sewage whose standard BOD is 100 mg/l. Assume $K_{20} = 0.1$. P-43. Peavy.

7+8

3. a) Briefly describe the transmisson rate of water and waste related disease. P-330, ITN.

b) A sewer with an isoceles (two side equals). triangular section having side of 10 feet with n=0.015 is laid on a grade 0.018. P-461. W.S/SK gary.

i) What is the capacity of sewer when the depth of flow is 5 feet.

ii) What will be the velocity when the depth of flow is 3 feet? and the section is circular of diameter 3/4 of the triangular same side?

34.th BCS.

6+4+5.

1. a) What do you understand by disinfection? Describe various methods of disinfection. P-359 ITN.

b) Distinguish between:-

i) Primary clarifier and secondary clarifier.

ii) Tappered aeration and tepped aeration.

iii) BOD loading and hydraulic loading.

c. Classify the main types of water distribution network with their advantages and disadvantages. P-394 ITN

5+10.

2. a) Explain with example a fixed growth and suspended growth treatment system.

b) What are the environmental significance of BOD and COD? Mension with reasons the type of treatment system you would prefer for the following industries.

W/W	Industry-1	Industry-2
pH	8	12
BOD, mg/L	2500	250
COD, mg/L	3000	5000

5+5+5

3. Define EIA. Why baseline environment is to be evaluated for EIA.

b) Define per capita consumption of water. Explain three factors affecting per capita consumption. P-16. S.K

c) Define the functional element of waste management. State the effect of solid waste management.

P-260 ITN.

33th. BCS.

8+7

1. a) Explain the process of coagulation sedimentation. How does it differ from plain sedimentation?

ITN b) state systematically the process of design of branched as well as looped network of distribution system.

8+7

2 a) Explain the biological treatment process of sewage.

b) Design a standard filterer to treat 8 ML/d sewage of BOD_5 of 210 mg/l. The final effluent should be 30 mg/l. and organic loading rate is 320 gm^3/day. Assume reasonable data, if necessary. (23th BCS)

7+8

3. ⓐ ~~state~~ Name some of the factors influencing water consump: state the ways in which ~~thete~~ these influence " .

*** ⓑ Design a rectangular sedimentation tank to treat 2.4 million litre of raw water per day. the overflow rate is 0.5 m/hr. and the detension time is 3 hours. ITN. 18.6

$\boxed{32^{th} \text{ BCS.}}$

1. ⓐ Explain the process of coagulation sedimentation. How does it differs from plain sedimentation.? ITN.

ⓑ Draw a typical ~~eth~~ chlorination curve and explain reaction zones. explain break-point chlorination. -ITN.

** ⓒ calculate the dimension of a rectangular settling tank to treat 120 m³ of water per hours. when the overflow rate is 0.75 m/hr and detension time is 2 hours. ITN. 18.6.

2. ⓐ Explain the principle of biological waste treatment process. Outline the differences between aerobic and anaerobic process of waste treatment.

6+9

3. Describe the functional element of solid waste management. P-261. ITN.

b) What are the methods of sanitary land filling of final disposal of solid wastes.? Describe.

P-285. ITN.

সূত্রাবলি :

i) $BOD_5 = (DO_i - DO_f) * D.F.$

ii) $L_5 = L_{ult}(1 - e^{-kt})$

iii) $L_t = L_0(1 - 10^{-k_1 t})$

iv) Volume of latrine, $V = 1.33\,CPN$.

v) $K_T = K_{20}\,\theta^{T-20}$

vi) $Q = \dfrac{2\pi k m (D-d)}{\ln R_2/R_1}$

vii) $Q = \dfrac{\pi k (D^2 - d^2)}{\ln R/r}$.

viii) $\mu g/m^3 = \dfrac{PPM \times g\ mole\ mass \times 10^3}{L/mole}$.

D.F. = dilution facto

L_t = BOD at t day.

L_0 = Ultimate BOD

$C = 0.04 \sim 0.06$

= sludge accum

rate

$= 0.06\ m^3/p/yr$. $N < 5$ yrs

$= 0.04$ > 5 yr

$\theta = 1.047$

$K_{20} = 0.1\ d^{-1}$

—o—

⇨ কিছু Theory পড়তে হবে BUET নোট

P-175, 77 : BOD. NBOD, CBOD.
P-183 : Objective of water supply system, layout
185, 87 : Sanitary Significance of EV parameters.
201 : P^H, BOD, Mg... Bangladesh and WHO standards.

Example-1.1: Design a simple pit Latrin for a family of 6 persons for the design of 5 years. Water table is below G.L.

Solution: Volume of pit, $V = 1.33 \times CPN$
$= 1.33 \times 0.06 \times 6 \times 5$
$= 2.4$ m3

P-196. ITN.
$c = 0.06$ $m^3/p/yr$ for dry season.
$= 0.04$ $m^3/p/yr$ for wet
D = Normally 1 to 1.5 m.

Assume dia of pit $d = 1.25$ m ∴ depth, $h = \frac{2.4}{\pi/4 \times 1.25^2}$
$= 1.95$ m ≈ 2m.

Design of pit rectangular $= \frac{2.4}{1.25 \times 1.25} = 1.54$ m.

Example-1.2: Design a VIP latrine for a family of eight. The family uses water for anal cleansing. The groundwater table is m below the ground surface.

Assume. N = 3 yrs.

Sol^n: $V = CPN = 0.05 \times 8 \times 3$
$= 1.2$ m^3.

Provide 1m×1.2m ~~×~~ simple pit of 1.5 m depth. with 0.5 m free space.

(b) Alternating twin-pit option:

$V = CPN = 0.05 \times 8 \times 2 = 0.8\ m^3$

∴ The dimension of each pit would be 0.8m×1m with depth of 1.5m including 0.5m free space.

Example-1.3: Design a leach pit for both single and ~~twin~~ alternating twin offset pit pour flush latrines for a family of 8 members in a peri urban area. Wastewater flow is 12 lpcd and soil is porous silty loam.

Sol^n: Pit volume with respect to infiltration:

for porous silty loam (table 9.2) $I = 20\ l/m^2 day$ and $Q = 12 \times 8 = 96\ l/day$

∴ Area required for infiltration is $A = \frac{Q}{I} = \frac{96}{20} = 4.8\ m^2$

Assume pit dia of 1.2 m. ∴ Volume, $V_i = \frac{AD}{4} = \frac{4.8 \times 1.2}{4} = 1.44\ m^3$.

Pit volume with respect to solids storage:

$V_s = CPN = 0.04 \times 8 \times 2 = 0.64\ m^3$

Assume
$C = 0.04\ m^3/P/yr.$
$P = 8$ persons.
$N = 2$ yrs.

a) Single pit pour flush system.

effective volume, $V = V_s + V_i = 1.44 + 0.64 = 2.08\ m^3 \approx 2.25\ m^3$

Assume pit dia of 1.2m ∴ effective depth, $h = \frac{2.25}{\pi/4 \times 1.2^2} = 2m$.

Assume clear spacing 0.5m. ∴ Total depth $= 2 + 0.2 = 2.5$ m.

a) Alternative twin pour flous latrin:

$V_i > V_s$ ∴ $V = 1.44\ m^3$ assume dia of 1.2 m.

$h = \frac{1.44}{...} = ...28 \approx 1.8\ m$ (Assuming clear

Example-1.4: If the soil is sandy loam with a long-term infiltration rate of about 30 l/m²day, design a soakage pit for the disposal of effluent from the septic tank. 10 person, wastwater flow is 90 lpcd.

Solⁿ: Effluent flow = 90x10 l/day = 900 litre/day.
long term infiltration rate = 30 l/m²/day.

$\therefore$ Infiltration ~~rate~~ Area = $Q/I = \frac{900}{30} = 30\ m^2$

Assume 1.25 dia, the effective depth of soak pit $h = \frac{30}{\pi d} = \frac{30}{3.14 \times 1.25}$
$= 7.6\ m$.

Assume clear spacing = 0.5m $\therefore h = 7.6 + 0.5 = 8.1\ m$

If G.T is high 2 soak pit of 4 m depth may be provided.

Example-1.5: Design a circular leach pits for both single and alternating twin offset pit pour flush latrine for service of a family of 10 members living in a upazila town. Design period 2 years. Average wastewater flow rate 10 lpcd. The soil is porous silty loam with long term infiltration rate is 20 l/m²/day.

Solⁿ. Storage volume, $V_s = CPN = 0.04 \times 10 \times 2 = 0.8\ m^3$

infiltration volume $V_i = \frac{Q A_i D}{4} = \frac{Q/I \times D}{4} = \frac{(10\times10)/20 \times 1.2}{4}$ Assume.
$= 1.5\ m^3$

i) Single pit pour flush latrine, $V = V_s + V_i = 0.8 + 1.5 = 2.3\ m^3$
$\approx 2.5\ m^3$

$\therefore h = \frac{2.5}{\pi/4 \times 1.2^2} = 2.21 \approx 2.25\ m.$

ii) Alternating twin offset pour flush latrine: V_s or V_i which is greater. $\therefore V = 1.5\ m^3$.

$\therefore h = \frac{1.5}{\pi/4 \times 1.2^2} = 1.33 \approx 1.5\ m.$ Ans

Example-1.6: A rectangular sediment is to treat 400000 gpd of raw water. The sedimentation of time is 4 hours. velocity of flow 3 in/min. The depth of water and sediment is 14 ft. Sediment allowance is 4 ft. What should be the Length and width of the sediment tank?

Solⁿ: $V = 3\ in/ft = \frac{13}{12}\ ft/min.$

length, $s = L = vt = \frac{3}{12} \times 4 \times 60 = 60\ ft.$

$\therefore$ Volume of tank $= Q \times t = 400000 \times 4/24$ gallon.

$= \frac{66666.67}{7.48}\ cft = 891265\ cft.$

X-sctional Area $= \frac{8912.65}{60} = 148.5\ sft.$

effective depth $= 14 - 4 = 10\ ft.$ $\therefore$ widh $= \frac{148.5}{10} \approx 15\ ft.$

$\therefore$ Tank size. $L \times B \times H = 60 \times 15 \times 10\ m^3.$

Example-1.7: Calculate the total stream flow in gpm for a town having a population of 10000. Assume each will spray 250 gpm. on the fire simultaneously.

Solⁿ: $F = 2.8\sqrt{P}$

$= 2.8 \times \sqrt{10}$

$= 9$

$\therefore$ Total stream flow $= 250F$

$= 250 \times 9$

$= 2250$ gpm.

P = population in thousand

F = No. of structural simultaneous stream.

Example-1.8: 1 million gallon of water passes through a sediment tank (per day). which is 20′ wide 50′ long and 10′ deep.

a) Find detension time for basin.

b) What is the average velocity of flow through the basin.

c) If the suspended solids average 40 ppm. What weight of dry solids will be deposited every 24 hours assuming 75% removal of basin

d) What is the overflow rate?

1 cft = 7.48

Solⁿ: a) Detention time. $t = \frac{\text{Volume}}{\text{Discharge}} = \frac{20 \times 50 \times 10}{\left(\frac{1 \times 10^6}{24 \times 7.48}\right)}$ hr ≈ 1.8 hr. Ans

b) Velocity $= \frac{Q}{A} = \frac{1 \times 10^6 / 7.48}{20 \times 10 \times 24 \times 3600}$ fps $= 7.74 \times 10^{-3}$ fps.

c) Total suspended solids $= \frac{40}{10^6} \times 10^6 \times 8.34 \times 0.75$ lb/day

$= 250.2$ lb/day.

d) overflow rate $= \frac{Q}{BL} = \frac{10^6}{\cancel{7.48} \times 20 \times 50} = 1000$ gpd/sft.

Example-1.9: Design the transmission main and pumping unit from the following data, water supply rate 40 gpcd population 85000 R.L of reservoir at pumping hour 102.5 ft. R.L of treatment plant 193 ft. velocity through pipe is 8 fps. pumping time 10 h/day. length of pipe 3500 ft. frictional factor = 0.012 pump efficiency = 65%. Neglect minor losses.

Solⁿ: Total water required $= 40 \times 85000$ g/day $= 3400000$ g/day

$\therefore$ Pumping rate $Q = \frac{3400000}{10}$ g/hr $= \frac{340000}{7.48 \times 3600} = 12.63$ ft³/sec

$Q = AV \Rightarrow \pi/4 \times d^2 \times 8$

$\Rightarrow d = \sqrt{\frac{4Q}{8\pi}} = \sqrt{\frac{4 \times 12.63}{8\pi}}$

$= 1.42$ ~~m~~ ft.

Let $d = 1.5$ ft

Actual velocity $= \frac{12.63}{\pi/4 \times 1.5^2}$

$= 7.15$ fps,

frictional head $= \frac{4fLV^2}{2gd}$

$= \frac{4 \times 0.012 \times 3500 \times 7.15^2}{2 \times 32.2 \times 1.5}$

$= 88.9$ ft

Velocity head $= \frac{v^2}{2g} = \frac{7.15^2}{2 \times 32.2} = 0.79$ ft

static head $= 193 - 102.5 = 90.5$ ft

$\therefore$ Total head $= H_s + H_v + H_f = 90.5 + 0.79 + 88.9 = 180.19$ ft.

$Q = \frac{3400000}{10 \times 60}$ gpm $= 5666.67$ gpm.

$\therefore$ BHP $= \frac{QH}{3960\eta} = \frac{5666.67 \times 180.19}{3960 \times 0.65} = 396.68 \approx 397$

Example-1.10: It is required to disinfect 50000 gpd of water with 0.3 mg/l of chlorine. If bleaching powder is used (33.3% available chlorine). How many pounds of bleaching powder are needed to treat the daily flow of water.

Sol^n: We know 1mg/l of chlorine = 8.34 lbs of chlorine per million gallon of water.

$\therefore$ Amount of chlorine $= \frac{500000}{10^6} \times 8.34 \times 0.3$ lbs. $= 1.251$ lbs.

Now. 33.33 lb chlorine is available in 100 lb bleaching powder

$\therefore$ 1.251 " " $= \frac{100 \times 1.251}{33.33}$ lb

$= 3.75$ lb

$= 1.70$ kg.

1kg = 2.2 lb.

Example-1.11: How many kg of bleaching powder with 30% of avilable chlorine will be required to treat 4 million litres of water with a dosage of 0.5 mg/l.

Ans: Amount of chlorine $= 0.5 \times 4 \times 10^6$ mg $= 2 \times 10^3$ g.
$= 2$ kg

Now. 30 kg chlorine is available in 100 kg bleaching powder
$\therefore$ 2 kg ... $\frac{100 \times 2}{30}$ kg $= 6.67$ kg.

Example-1.12: Determine the size of the tile drain if the tile grade is 0.25%. Manning's coefficient 0.018 and drainage area is 10 ha.

Ans: Assume 1% to drained in 24 hours and Annual rainfall $= 100$ cm.

$\therefore$ Run-off $= Q = CIA = \dfrac{\frac{100}{100} \times \frac{1}{100} \times 10 \times 10^4}{24 \times 3600}$ m3/sec.
$= 0.0116$ m3/sec.

Now. $Q = \frac{1}{n} A R^{2/3} S^{1/2} \Rightarrow 0.0116 = \frac{1}{0.018} \times \frac{\pi}{4} d^2 \times \left(\frac{d}{4}\right)^{2/3} \left(\frac{0.25}{100}\right)^{1/2}$

$\Rightarrow d = 0.198$ m ≈ 20 cm.

Example-1.13: BOD_5 at 20° of sewage is 276 mg/l. and ultimate BOD is 380 mg/l. Find BOD reaction rate (k).

Ans: $BOD_5 = BOD_{ult} (1 - e^{-kt}) \Rightarrow 276 = 380 (1 - e^{-k \times 5})$
$\Rightarrow e^{-5k} = 0.273 \therefore k = 0.26\ d^{-1}$ (10 base নিয়ে 3 হবে)

Example-1.14: A sample of sewage was incubeted for 2 days and BOD of the sample is 165 ppm at 20°C. determine its 5 days BOD. and 10 days BOD at 20°C. $K_1 = 0.1 d^{-1}$.

Ans: We know. $L_t = L_o(1 - 10^{-K_1 t})$

$\therefore 165 = L_o(1 - 10^{-0.1 \times 2}) \Rightarrow L_o = 447$ ppm.

Round value
নিতে হবে

$\therefore BOD_5 = 447(1 - 10^{-0.1 \times 5}) = 305.\cancel{65}$ ppm.

$\therefore BOD_{10} = 447(1 - 10^{-0.1 \times 10}) = 402$ ppm.

Example-1.15: The BOD_5 of a sewage at 20°C is 200 mg/l if $k_1 = 0.17 d^{-1}$ find the Ultimate BOD.

Ans: $L_t = L_o(1 - 10^{-k_1 t})$

$$\Rightarrow L_o = \frac{L_t}{1 - 10^{-k_1 t}} = \frac{200}{1 - 10^{-0.17 \times 5}} = 233 \text{ mg/l.}$$

Example-1.16: The population of a town was 180000 in 1980 and 220000 in 1990 what will be the population in 2005?

Solⁿ: Population rate $= \sqrt[n]{\frac{P_2}{P_1}} - 1 = \sqrt[10]{\frac{220000}{180000}} - 1 = 0.02026$ | n = year

$\therefore$ Population in 2005 is $= 220000(1 + 0.02026)^{15}$
$= 297225$.

Example-1.17: A test bottle containing just seded diluted water has its DO level drop 0.8 mg/l in a 5 day test. A 300 ml BOD bottle filled with 30 ml of water and rest with seeded dilution water expansive a drop of 7.3 mg/l in the same period. Calculate BOD_5 of water.

Ans: We know $BOD_m V_m = BOD_w V_w + DOD_d V_d$

$\Rightarrow \quad 300 \times 7.3 = BOD_w \times 30 + 270 \times 0.80$

$\Rightarrow \therefore BOD_w = 65.8$ mg/l.

≈ 66 mg/l.

Example-1.18: In a BOD test on a diluted waste water supply (1:20 dilution but not seeded). The initial DO is 8.4 mg/l and final DO is 4.2 mg/l after 5 days. if the reaction rate is $0.22 d^{-1}$

a) find BOD_5 of wastewater.
b) ultimate carboneous BOD.
c) Remaining oxygen demand after 5 days.

$20 = D.F$

Ans: a) $BOD_5 = (DO_i - DO_f) * D.F$

$= (8.4 - 4.2) * 20$

$= 4.2 \times 20 = 84$ mg/l.

b) $L_5 = L_0 (1 - e^{-kt}) \Rightarrow L_0 = \frac{L_5}{(1 - e^{-kt})} = \frac{84}{(1 - e^{-5 \times 0.22})}$

$\therefore L_0 = 126$ mg/l.

c) Remaining $BOD = L_0 e^{-kt} = \cancel{8}\ 126 \times e^{-5 \times 0.22}$

$= 42$ mg/l.

Example-1.19: A sample of sewage is mixed with water (No seeded) in the ratio of 1:20 [1 ml of sewage diluted to 20 mg by water] for BOD test. The initial BOD is 8.5 mg/l. and final BOD after 5days is 3.1 mg/l. Calculate BOD_5?

Ans: $BOD_5 = (DO_i - DO_f) \times D.F$ | $D.F = 20/1 = 20$

$= (8.5 - 3.1) \times 20$

$= 5.4 \times 20$

$= 108$ mg/l.

Example-1.20: Estimate the mix hourly, average hourly and minimum hourly residual sewage flows from an area occupied by 750 people having per capita ~~consumption~~ sludge flow 59 gpcd. Consider the length of sewage and house connection to be 1.3 miles and infiltration to be 30000 gpd.

Ans: Total average daily flow $= 750 \times 59 = 44250$ gpd.

infiltration $1.3 \times 30000 = 39000$ gpd

$\therefore$ Average daily flow $= 44250 + 39000 = 83250$ gpd. (Ans)

Maximum hourly flow $= 44250 \times 3 + 39000 = 171750$ gpd. (Ans)

Minimum hourly flow $= 44250 \times 0.33 + 39000 = 53602.5$ gpd.

$= 53602$ gpd. (A)

Example-1.21: Design a sediment tank for 3 hours concentrati period with discharge 125 litre/hour. Where the ratio of length Breadth and Height is H&B:L = 1:1:3? BWDB-13

Ans: Total volume = 125 × 3 litre = 375 litre = 0.375 m^3

Assume L = 4B ∵ H:B = 1:1 So, H = B.

Now. B × B × 4B = 0.375 ⇒ B = 0.954 m.

∴ L = 4B = 1.817 m

Provide a size of B × H × L = 0.46 m × 0.46 m × 1.8 m tank

Example-1.22: Determine the sweage BOD_5 in the following data. A sample having 25 ml sewage water to ~~treat~~ dilute in ... ml D_i = 8.7 and D_f = 3.7 and the water seed sample D_i = 3.7 and D_f = 2.6.

Ans: $V_m BOD_m = V_w BOD_w + V_d BOD_d$

$300 \times 5 = 25 \times BOD_w + 275 \times 1.1$

$\Rightarrow BOD_w = 47.9 \approx 48$ mg/l.

$V_m = 300$ ml

$BOD_m = 8.7 - 3.7 = 5$ mg/l

$BOD_d = 3.7 - 2.6 = 1.1$ mg/l

confused!!!

Example-1.23 2% solution is used from sewage of 5 day at 20°C and dissolved oxygen depletion was 5 ppm. Determine BOD_5.

PGCB-17

Ans: $BOD_5 = (DO_i - DO_f) \times D.F$

$= 5 \times 50 = 250$ PPM.

2% solution,

$\therefore D.F = \frac{100}{2} = 50$

Example-1.24: convert 9 ppm CO into mg/m^3, $\mu g/m^3$.

$\therefore$ $9 ppm \cdot CO = \frac{9 \times \cancel{10^{-6}} \times 28 \times 10^3}{22.4} \mu g/m^3 = 11250 \mu g/m^3$

$= 11.25 \ mg/m^3$.

Example-1.25: How much chlorine will be required (lb/hr) To treat 10 mgd water with 0.4 mg/l. of chlorine (use 1 mg/l Cl_2 = 8.34 lb/mgd).

Ans: Amount of chlorine = $10 \times 0.4 \times 8.34$ lb/day

$= 1.39$ lb/hr.

Example-1.26 1° french hardness = 10 mg/l of hardness as $CaCO_3$.

Practice Example-9.30, 9.31 upto 9.35.

P-508. water supply S.K Garg

Planning and design consideration of water treatment plant.

Enumerate the factors ~~of~~ for planning and des[ign] of a water supply system / treatment plant.

1. The quality of water should not deteriorate below certain acceptable level (BD or WHO standards)
2. Adequate quantity of water should be provided at all times and at a convenient location.
3. Traditional sources should be given consideration for development of water supply system.
4. Construction, operation, maintenance and repai[r] should be within the competence of local tech[nical] staff.
5. The equipment used shall be reliable, availa[ble] locally and cheap.
6. Construction and operation cost should be minimum and imported material should be avoided.
7. The use of pumping and chemical should be minimum.

8. The system should be planned together with the community to enable adaptation to local condition, needs and to take advantage of local skill and knowledge.

9. steps should be taken to consult the women to understand their needs and involve them in local management in keeping the system functional.

10. An appropriate in-built system should be made to monitor the performance of the treatment system.

11. Sustainability of the system should be given preference in planning, design and pricing of water supply.

* Write down the various methods of water treatment: Methods involve.

i) Sedimentation.
ii) Coa-gulation.
iii) Filtration.
iv) chemical precipitation
v) Dis-infection.

R.S. khurmi - p-308

Q. Explain the process of sedimentation with coagulation. How does it differs from plain sedimentation?

Sedimentation:/Plain sedimentation

It is the process of retaining water in a basin so that the suspended particles settles down as a result of the action of gravity forces.

The settling velocity of particles depends upon

$$v = \frac{g}{18}(s-1)\frac{d^2}{\gamma}$$

i) horizontal flow velocity of water.
ii) Specific gravity of the particles
iii) particle size and shape.
iv) viscosity of water
v) Density of water.
vi) Temperature of water

[D] Sedimentation with coagulation.

The process of adding certain chemicals known as coagulants ($Al_2(SO_4)_3 \cdot nH_2O$, $Fe_2(SO_4)_3 \cdot 9H_2O$ } $FeCl_3 \cdot 6H_2O$ to water in order to form flocculent precipitate for absorbing and entraining colloidal matter is called sedimentation with coagulation. The sedimentation with coagulation becomes necessary when the turbidity of water exceeds about 45 PPM

How differs from plain sedimentation.

plain sedimentation removes settleable particles but it does not remove the colloidal particles. and very fine particles. having very low or no settling velocity.

In sedimentation with coagulation removes all types of particles and colloids.

Filtration.

Filtration is a process of water purification in which water is allowed to pass through a bed of filtering media usually sand and gravel. The purified water is collected at the bottom through an undrained system. The filter media are very efficient in retaining finer and colloidal particles including bacteria and viruses.

Two types:

1. Slow sand filteration (SSF).
2. Rapid sand filtration.

SSF: In slow sand filtration water is allowed to pass through a bed of fine sand which retains most of the impurities present in water. It is suitable for the development of surface water supply system in developing countries.

Objectives:

1. Reduce number micro-organisms present in water.
2. Retain fine organic and inorganic solid matters
3. Oxidized organic compounds dissolved in water.

characteristics:

→ Rate of filtration is low. (0.1~0.3 m^3/hr)

→ very high removal of turbidity and colour.

→ No pre-treatment is generally required.

→ Operation and maintenance is low.

Dis.adv.

1. Not suitable for water having turbidity > 30 NTU

2. Not very effective in removal of colloidal matters.

3. R.F.S:

In rapid sand fitration, water is allowed to pass througth a relatively larger and uniform size sand filter media.

The filter bed consists of about 1m thick coarse sand layer laid on top of a 0.5 m thick gravel layer. The gravel is underlaid by an under drainage system.

Advantages:

1. High filtration rate (5-15 m^3/hr)
2. high removal of turbidity and colour
3. ~~Re~~ Suitable for all types of turbid and color water

Disadvantages:

1. Relatively high cost of and operation and maintenance
2. Pre-treatments are required (coagu: sedi: floc.)
3. cleaning of filter bed by backwashing.

Dis-infection.

The process of killing pathogenic bacterias from water and making it safe to the public use, is called dis-infection. The most commonly used disinfectant for drinking water throughout the world is chlorine.

The process of applying chlorine or chlorine compounds in small quantity to water to disinfect it is called chlorination. (at lest 20 min)

Two types:

(i) physical dis-infection → Boiling
→ Ultra-violet rays.
→ Sun light

(ii) chemical disinfection. (Cl_2)

Factors influencing disinfection of water.

1. The Nature and number of organisms to be destroyed
2. Type and concentration of disinfectant.
3. Temperature of water.
4. The nature of water to be disinfected
5. The p^H of water.
6. Mixing of water; good mixing ensures proper dispersal of disinfectants.

Free available chlorine:

When chlorine is added to water, it reacts according to the following ~~reactions~~, equation

$$Cl_2 + H_2O \longleftrightarrow HOCl + HCl$$

$$HOCl \longleftrightarrow H^+ + OCl^-$$

The chlorine available in water in any of the above forms is defined as free available chlorine which accomplishes the task of disinfection.

combined available chlorine

If ammonia is present in water, monochloramine, dichloramine and nitrogen trichloride are formed according to the following reactions.

$$NH_3 + HOCl \longrightarrow NH_2Cl + H_2O$$

$$NH_2Cl + HOCl \longrightarrow NHCl_2 + H_2O$$

$$NHCl_2 + HOCl \longrightarrow NCl_3 + H_2O$$

The chlorine present in water in chemical combination with ammonia or other nitrogenous compounds is known as combined available chlorine.

Q. Draw a typical chlorination curve and explain the reaction zones. Also explain the Break point chlorination.

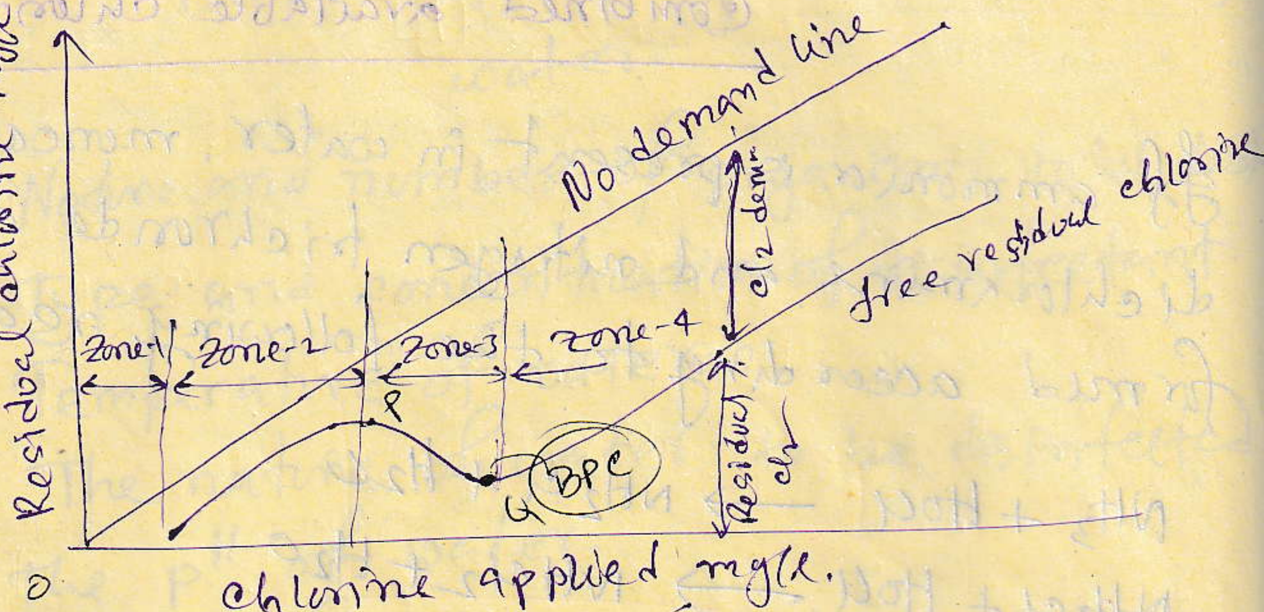


fig: Typical chlorine curve.

1 <u>Zone-1</u>: consumption of chlorine by reducing compounds

<u>Zone-2</u>: formation of chloro-organic compounds and chloramines.

<u>Zone-3</u>: Destruction of chloro-organic compounds and chloramines.

<u>Zone-4</u>: Formation of free available chlorines.

Break point chlorination.

When the chlorine dose increases, the combined available chlorine also increases. On further increases of chlorine, the compound gets oxidised and the substances which are newly formed do not react to show any residual. this phenomenon is called break point.

The addition of chlorine at the break is termed break point chlorination. This indicat the point at which free residuals begin to appear.

⟹ From figure. residual chlorine increases with appl chlorine dose at the begining but after point P, i suddenly drops upto point Q. and then increases. The portion OP shows the formation of chlorom PQ shows their oxidation. The point Q at whic the residue chlorine again starts increasing is known as BPC.

→ It destroys completly all pathogenic bacteria.
→ It oxidize the impurities of water such as ammon
→ It also prevents the growth of weeds and removes taste and odor from the water

Design of water supply system.

Objectives of water supply.

→ To supply water in adequate quantity
→ To supply safe and wholesome water to the consumers
→ To make water easily available to the consumers.

Elements of water supply system.

Essential elements of water supply systems are.
1. Source of supply.
2. collection system.
3. treatment and
4. distribution system.

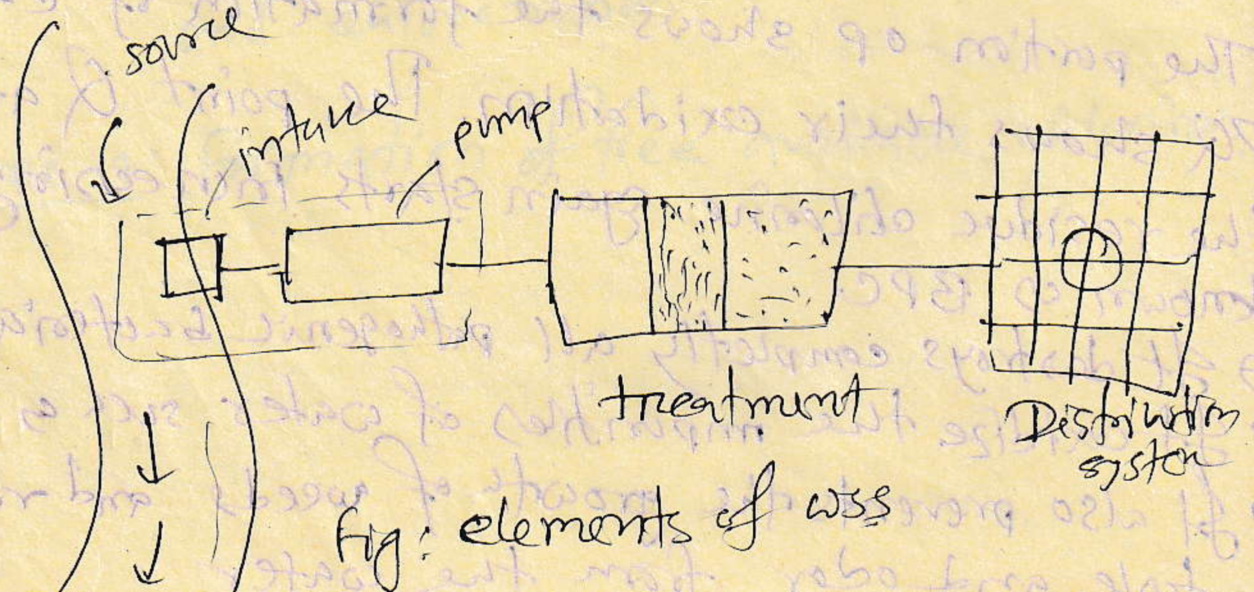


Fig: elements of wss

Define per capita consumption of water.

per capita consumption of water:

Per capita consumption of water is total consumption divided by the population and the number of ~~year~~ days in the year.

$$\therefore \text{per capita consumption} = \frac{\text{total consumption in gallon/lit}}{\text{population} \times 365}$$

Various factors affect the per capita consumption are as follow:

$\boxed{\text{P- 28. M. A Aziz}}$

1. Size of the city
2. characteristics of people.
3. climate conditions.
4. commerce and Industries.
5. Pressure of water
6. Quality of water.
7. sewerage facilities.
8. Water rates and metering.
9. Nature of supply.
10. Number of inhabitants.

A Factors to be considered in planning a Municipal Water Supply System.

(P-24 MA AZIZ)

(i) Estimate the future population of the community and determine the quantity of water which must be provided.

(ii) Location of a reliable source of water of adequate quantity.

(iii) Design of suitable collection system.

(iv) Determine the physical, chemical and biological characteristics of water

(v) Design of various units of the treatments plants.

(vi) Design of the distribution systems including distribution reservoirs, pumping stations, elevated storage and location of fire hydrants.

vii) Provision for the establishment of an organization which will maintain and

Q. What are the types of distibution systems. Explain with advantages and disadvantages.

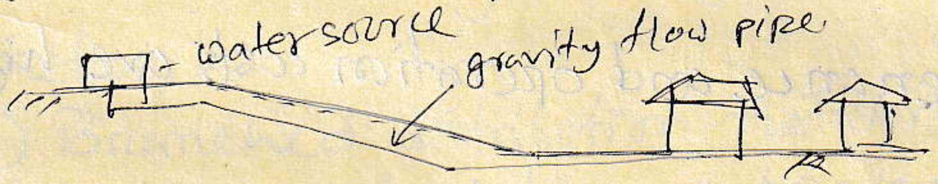
Distribution systems are

i) Gravity system.

ii) System with direct pumping.

iii) system with pumping and storage.

Gravity System: A gravity system is adopted when the source of supply is at a sufficient elevation with respect to the consumption points



Advantage:

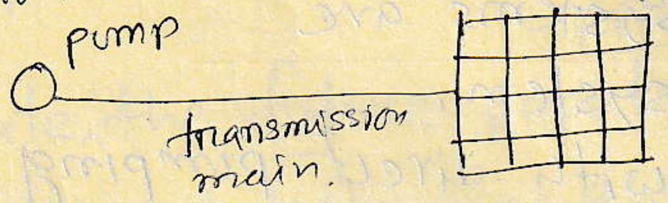
1. Water flows due to gravity force
2. No pump requires.
3. construction, operation and maintenance are simple.

Disadvantage:

1. Not applicable in flat countries.
2. Water loss by leakage and wastage comperatively high.

Direct pumping system: (P-391 TTN)

In this system water is pumped into the transmission main or distribution system.



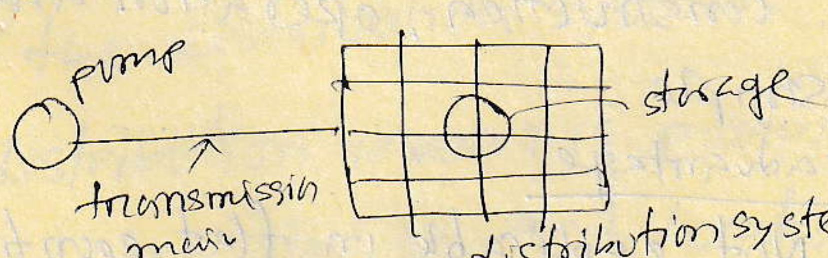
Advantage:

1. Water can be pumped when required
2. low water required loss due to system leakage

Dis:advantage:

1. A power failure means breakdown of the system.
2. Maintenance and operation costs are high.
3.

pumping with storage system: A system pumping with storage is called a direct or indirect or dual system.



Advantages:

1. The system is more reliable and can cope with fluctuation of water demand

2. The pumps can be operated at rated capacity
3. Reasonable pressure can be maintained with varying water demand.

Dis-advantages:

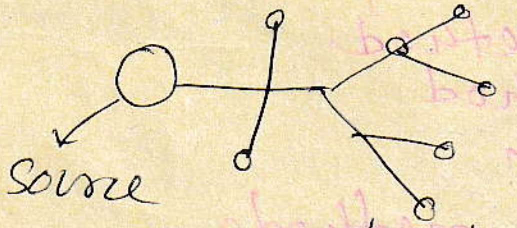
1. Relatively higher initial cost.
2. Comperatively higher loss due to leakage and westage.

⊞ Distribution networks.

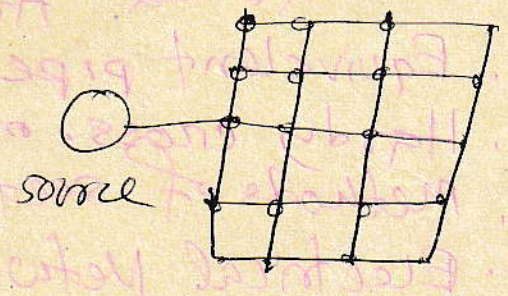
There are mainly two types of distribution networks.

i) Branched distribution networks.
ii) looped distribution networks.

looped distribution network may be further divided into.
- ↳ Grid iron system.
- → Ring system
- → Radial system.



Branced distribution system.



looped network system.

Advantage of Branched network:

1. Relatively cheap.
2. Easy to hydraulic design.
3. Can be easily expanded to coverage newly developed areas.

Disadvantages:

1. Stagnant water at dead ends promotes sedimentation and water contamination.
2. frequent floushing is needed to keep the system clean.

Advantages of looped Network. (P - 399. TTM

1. No stagnation of water.
2. Continuity of water supply anywhere in the system.
3. provides very good control over flow of water.

Disadvantages:

1. Relatively high initial cost.
2. A large number of valves is needed.

Analysis of pipe networks

1. Equivalent pipe method.
2. Hardy cross method
3. Methods of section
4. Electrical Network methods
5. Computer method.

Irrigation and Water Supply

সূত্রাবলি:

1. Manning's Formula. $V = \frac{1}{n} R^{2/3} S^{1/2}$ [S.I unit]
$= \frac{1}{n} * (3.28)^{1/3} \times R^{2/3} S^{1/2}$ [fps unit]

2. Chezy's formula $V = C\sqrt{RS}$.
3. Darcy's formula $h_f = \frac{fLV^2}{2gd} \Rightarrow V = \sqrt{\frac{8g}{f}} \cdot \sqrt{RS}$.
4. Hazen Williams formula
$V = 0.85\, C_H\, R^{0.63}\, S^{0.54}$

$C_H = 100 \sim 140$
$= 100$ old pipe
$f = 0.02$ new pipe
$= 0.075$ old "

Laecy's formulae.

i) $V = \left(\frac{Qf^2}{140}\right)^{1/6}$ ii) $R = \frac{5}{2} * \frac{V^2}{f}$

iii) $f = 1.76\sqrt{d}$ iv) $P = 4.75\sqrt{Q}$

v) $S = \frac{f^{5/3}}{3340\, Q^{1/6}}$ vi) $A = y(b + y/2)$

vii) $P = b + 2\sqrt{1.25}\, y$. viii) $h = 0.47\left(Q/f\right)^{1/3}$

$Q =$ in m^3/sec.

$h =$ Scour depth.

5. For confined layer discharge $Q = \frac{2\pi km(D-d)}{\ln R/r}$
6. For unconfined layer . $Q = \frac{\pi k(D^2 - d^2)}{\ln R/r}$.

Example – 2.1 | An 18 in sewer with n=0.013 is laid on a grade at 0.015

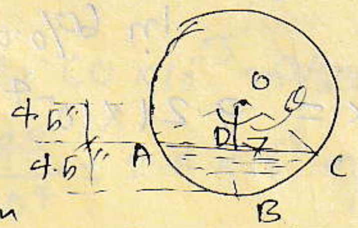
a) capacity when flowing full.

b) velocity when depth of flow = 4.5"

Ans: a) $Q = \frac{1.486}{n} A R^{2/3} S^{1/2} = \frac{1.486 \times \pi \times 1.5^2}{0.013 \times 4} \times \left(\frac{1.5}{4}\right)^{2/3} \times (0.015)^{1/2}$ | fps units...

$= 12.86\ ft^3/sec.$ (Ans)

b) when depth of flow is 4.5"



$AD = \sqrt{OA^2 - OD^2} = \sqrt{9^2 - 4.5^2} = 7.79$ in

$\therefore \tan\theta/2 = \frac{AD}{OD} = \frac{7.79}{4.5} \Rightarrow \theta = 2\tan^{-1}\frac{7.79}{4.5} = 120°$

$\therefore$ wetted perimeter = $ABC = R\theta = \frac{9}{12} \times \frac{120}{180} \times \pi = 1.57$ ft.

$\therefore$ Area of $ABCD = \frac{120}{360} \times \pi \times \left(\frac{9}{12}\right)^2 - \frac{1}{2} \times 2 \times \frac{7.79}{12} \times \frac{4.5}{12} = 0.3456\ ft^2$

$\therefore R = \frac{A}{A_P} = \frac{0.3456}{1.57} = 0.22$ ft.

$\therefore V_P = \frac{1.486}{n} R^{2/3} S^{1/2} = \frac{1.486 \times 0.22^{2/3} \times 0.015^{1/2}}{0.013}$

$= 5.10$ fps.

Example-00: specific gravity of water $\gamma_w = 1\ gm/cc = 1000\ kg/m^3$
$= 9.81\ kN/m^3$.

$WHP = \frac{Qh}{76}$ [Q = litre/sec, h = m.] $= \frac{Qh}{3960}$ [Q = gpm, h = ft.]

$= \frac{QP}{1715}$ [P = psi

Example-2.2: A 150 mm dia tubewell produce 100 lps with a drawdown of 3m and a circle of influence of 120 m diameter. The static depth of water in the well is 40 m. Calculate the co-efficient of pearmeability of the aquifer.

Ans: We know. $Q = \frac{\pi k (D^2 - d^2)}{\ln R/r}$

$\Rightarrow 0.1 = \frac{\pi * k (40^2 - 37^2)}{\ln 60/0.075}$

$\therefore k = 9.21 \times 10^{-4}$ m/sec.

$Q = 100$ l/sec.
$= 0.1$ m3/sec

$R = D/2 = \frac{120}{2} = 60$m

$r = \frac{0.150}{2} = 0.075$ m

drawdown $= D - d$

$\Rightarrow 3 = 40 - d$

$\therefore d = 37$ m

Example-2.3 A rectangular channel has a bottom width at 6 m $\alpha = 1.12$, $n = 0.020$, $h_n = 1$ m. Determine critical slope.

BWDB-13

Ans: Now, $Q = \frac{1}{n} R^{2/3} S^{1/2} . A$

$17.757 = \frac{1}{0.02} \times 0.75^{2/3} \times S^{1/2} \times 6$

$\therefore S \approx \frac{1}{195}$ Ans

$h_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}}$

$1 = \sqrt{\frac{1.12 \times Q^2}{9.81 \times 6^2}}$

$\therefore Q = 17.757$ m3/s

$R = \frac{6 \times 1}{2 \times 1 + 6} = 0.75$ m

Example-2.4 The value of total direct run-off volume of catchment 6×10^3 m3. If rainfall depth is 8cm. and run-off co-efficient is 0.5. Determine the catchment Area in km2?

Ans: $Q = CIA$

$\Rightarrow \frac{V}{t} = C . \frac{h}{t} . A$

$\Rightarrow \frac{6 \times 10^3}{t} = \frac{0.5 \times 0.08 \times A}{t}$

$\therefore A = 1.5 \times 10^5$ m2 $= 0.15$ km2.

$V = 6 \times 10^3$ m3

$I = \frac{h}{t} = \frac{0.08}{t}$ m/sec.

$A =$ m2?

$C = 0.5$

Example-2.5: The field capacity and moisture content at the time of irrigation are 27% and 19% respectively. The apparent specific gravity is 1.3 and the root zone depth is 100cm. Determine the time required to irrigate 2ha with a flow 60 l/s. if the water application losses are takes to be 20%?

BWDB-16

Ans: Storage capacity at root zone depth

$$= \frac{\gamma_d}{\gamma_w} \times H_z \times (FC - OMC) = 1.3 \times \frac{100}{\cancel{100}} \times (0.27 - 0.19) = 10.4 \text{ cm.}$$

Net discharge $= Q - 20\%$ of $Q = 0.8 \times 60 \times 10^{-3}$ m³/sec $= 0.048$ m³/s

$\therefore$ Consumptive use $= \frac{Q}{A} = \frac{0.048}{2 \times 10^4} = 2.4 \times 10^{-6}$ m/s $= 2.4 \times 10^{-6}$ m/s.

$= 0.20736$ m/day $= 20.736$ cm/day.

$\therefore$ 20.736 cm water is utilized in 1 day.

$\therefore$ 10.4 cm " $= \frac{10.4 \times 1}{20.736}$ day $= 12$ hours.

Example-2.6: Determine the scour depth for discharge 2500 m³/s and sediment size 0.15 mm. i) Hydraulic geometry ii) Scour depth.

Ans: $f = 1.76\sqrt{d} = 1.76 \times \sqrt{0.15} = 0.68$

i) $\therefore$ Scour depth $h = 0.47 \sqrt[3]{Q/f} = 0.47 \times \left(\frac{2500}{0.68}\right)^{1/3} = 7.25$ m.

ii) Velocity $= \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{2500 \times 0.68^2}{140}\right)^{1/6} = 1.52$ m/sec

Area $= \frac{Q}{V} = 1645$ m².

Perimeter $= 4.75\sqrt{Q} = 4.75 \times \sqrt{2500} = 237.5$ m.

slope, $S = \frac{f^{5/3}}{3340\, Q^{1/6}} = \frac{0.68^{5/3}}{3340 \times (2500)^{1/6}} = \frac{1}{23400}$.

Example-2.7: A canal commands an irrigation area of 400 ha. The duty of water on the field during peak period is 220 ha/cumec. Determine the design discharge of the canal at the off-take if the water loss in canal is 30%.

BWDB-16

Ans:

$$Q = \frac{A}{D} \times \frac{1}{70\%}$$

$$= \frac{400}{220} \times \frac{100}{70} = 2.597 \ m^3/s$$

$$= 2.60 \ m^3/s$$

effective discharge
$= (100-30)\%$
$= 70\%$

Example-2.8 A canal commands an irrigated area of 350 ha. The peak flood required is 9 mm/day. Determine the design discharge of canal at the off-take. water loss is 25%.

Ans:

$$Q = A \times V \times \frac{1}{75\%}$$

$$= 350 \times 10^4 \times 9 \times 10^{-3} \times \frac{1}{75} \ m^3/day$$

$$= 420 \ m^3/day$$

$$= 4.86 \times 10^{-3} \ m^3/sec$$

effective discharge
$= (100-25)\%$
$= 75\%$

Example-2.8 for rainfall intensity $I = 2.4$ mm/yr run-off co-efficient $c = 0.70$, 10 person of 15 lpcd water consumption Find maximum catchment Area.

Ans:

$$Q = CIA$$

$$A = \frac{Q}{CI} = \frac{150 \times \times 365}{0.70 \times 2.4 \times 10^{-3}}$$

$$= 32590 \ m^2$$

$Q = 15 \times 10$ l/day $= 0.15 \ m^3/day$

$c = 0.70$

$I = \frac{2.4 \times 10^{-3}}{365}$ m/day

Example-2.9: Destermine the peak discharge for highway crossing canal. Mainstream length $L = 3$ km. $A = 7.5\ km^2$ $S = 5$ m/km catchment type = medium type $c = 0.48$ Recurrence interval 5yr. Also determine the dia of stream swer crossing the highway $n = 0.013$ $S_o = 0.001$ velocity of full flow not less than 3 m/s.

Ans: a) Time of concentration $t_c = \frac{FL}{A^{0.1} S^{0.2}} = \frac{58.5 \times 3}{7.5^{0.1} \times 5^{0.2}}$

$= 103.986$ min.

| F = 58.5 যখন
| A = in km^2
| F = 92.7 যখন
| A = ha.
| S = m/km.

For $t_c = 104$ min and recurrence interval of 5 yr.

$I = 70$ mm/hr. $\therefore$ Peak discharge $= FCIA$

$= 0.278 \times 70 \times 0.48 \times 7.5$

$= 70.056\ m^3/sec.$

| I = mm/hr
| F = 0.278 যখন
| A = km^2
| F = 0.00278
| A = ha
| F = 1 যখন
| A = m^2.

b) Again.

$\pi/4 \times d^2 \times V = Q$

$\Rightarrow d = \sqrt{\frac{4Q}{\pi V}} = \sqrt{\frac{4 \times 70.056}{\pi \times 3}} = 5.452\ m \simeq 5.45m.$

Use ~~55~~ 45000mm dia pipe. $\therefore$ Velocity $= \frac{70.056}{\frac{\pi}{4} \times 5.45^2} = 3.003$ m/s

(ok)

Example-2.10: Find out the dimension of a settling tank [to] treat 45 m^3 of raw per hour when the over flow rate [is] 0.5 m/hr. The detention time is 3 hr.

[An]s: Detention time $= \frac{vol^m}{Q} \Rightarrow 3 = \frac{BHL}{45}$

$\therefore BHL = 45 \times 3$

$\Rightarrow H = \frac{45 \times 3}{BL} = \frac{45 \times 3}{90} = 1.5$ m.

[A]ssume. $L = 4B$. $\therefore 4B^2 = 90$

$\therefore B = 4.743$ m

overflow rate $= \frac{Q}{BL}$

$\Rightarrow 0.5 = \frac{45}{BL}$

$\therefore BL = 90$ --(i)

Provide $B \times H \times L = 4.75 \times 19 \times 1.5$ m^3 tank

Example-2.11: Given discharge of a bridge is $5800\ m^3/s$. Hydraulic mean depth of the river is 58 m. mean regime velocity is 10.5 m/s. Calculate the scour depth using lacy's silt factor.

BWDB

Ans: We know.

$$R = 5/2\left(\frac{v^2}{f}\right)$$

$$\Rightarrow f = \frac{5}{2} \times \frac{v^2}{R}$$

$$= 2.5 \times \frac{10.5^2}{58}$$

$$= 4.752$$

$$\therefore \text{Scour depth} = 0.47\left(\frac{Q}{f}\right)^{1/3}$$

$$= 0.47 \times \left(\frac{5800}{4.752}\right)^{1/3}$$

$$= 5.02\ m.\ \text{A}$$

Example-2.12 The value of dimensionless effective stress is 0.045. Find the i) effective velocity

ii) Incipient depth of wide river channel.

Also given, sediment size 0.2mm longitudinal bed slope 0.001.

BWDB-16

Ans: from lacey's theory.

$$\therefore \text{i) } v = \left(\frac{Qf^2}{140}\right)^{1/6}$$

$$= 0.0813\ m/sec$$

$$\text{ii) } A = \frac{Q}{v} = \frac{6.56\times10^{-5}}{0.0813} =$$

$$\therefore h_n \times 1 = 8.06\times10^{-4}\ m$$

$$= 0.80\ mm$$

$$f = 1.76\sqrt{d} = 1.76\sqrt{0.2} = 0.787$$

$$S = \frac{f^{5/3}}{3340\,Q^{1/6}} \Rightarrow 0.001 = \frac{0.787^{5/3}}{3340\,Q^{1/6}}$$

$$\Rightarrow Q = 6.56\times10^{-5}\ m^3/sec.$$

wide channel = h = R

$$\text{or } R = 5/2 \times \frac{v^2}{f} = 0.021\ m$$

Example-2.13 The sequent depth ratio of a channel jump in a rectangular channel is 16.48 at the beginning of jump. find the type of jump.

Ans: We know. $\frac{h_2}{h_2} = \frac{1}{2}(-1+\sqrt{1+8Fr_1^2})$

$\Rightarrow 16.48 = \frac{1}{2}(-1+\sqrt{1+8Fr_1^2})$

$\Rightarrow Fr_1 = 12. > 9$

∴ so the jump is strong.

Example-2.14: An artesian aquifer 10m thick with pizometric surface 40m. above the bottom containing the layer. The aquifer is medium sand ($k = 1.5\times10^{-4}$ m/s) steady state drawdown of 5.0 and 1.0 m. above at two non-pumping well located 20m and 200m. respectively from pump level. Determine the disch?

SGFL-17

Ans:

We know,

discharge, $Q = \frac{2\pi k m (s_1 - s_2)}{\ln R_1/r_1} = \frac{2\pi\times1.5\times10^{-4}\times10\,(5-1)}{\ln 200/20} = 0.0164$ m³/s.

Example-2.15 Initial and final BOD bottle after 3 days is 7 mg/l and 3 mg/l. and $k = 0.2d^{-1}$. Determine BOD_5 and ultimate BOD.

BGFCL-17

Ans: $BOD_t = L_0(1-e^{-kt})$

$\Rightarrow 7 = L_0(1-e^{-0.2\times3})$

$\therefore L_0 = 15.51 \approx 16$ mg/l

$BOD_5 = L_0(1-e^{-kt})$

$= 16\times(1-e^{-0.2\times5})$

$= 10$ mg/l.

Hydrograph & Math

Unit hydrograph: A unit hydrograph is defined as the hydrograph of direct run-off resulting from one unit depth (1cm) of rainfall excess occuring uniformly over the basin and at a uniform rate for a specified duration (D hours).

Subramanya-P-262

Example-6.8 a) The peak flood hydrograph due to a 3h duration isolated storm in a catchment is 270 m^3/s. The total depth of rainfall is 5.9 cm. Assume infiltration loss 0.3 cm/hr. and constant base flow is 20 m^3/s. Estimate the peak of the 3-h unit hydrograph.

b) if the catchment area is 567 km^2. determine the base width of the 3-h unit hydrograph. by Assuming it to be triangular shape.

Ans: a

a) duration of rainfall excess = 3 hours.

$\therefore$ Total loss = 0.3 cm/h × 3h = 0.9 cm

Total depth of water = 5.9 cm

Rainfall excess = (5.9 − 0.9) = 5 cm

Peak flow

Peak flood = 270 m^3/s.

base flow = 20 m^3/s.

Peak drainage hydrograph = 250 m^3/s.

$\therefore$ Peak of 3-h unit hydrograph $= \frac{DRH}{R.\ exc}$

$= \frac{250}{5} = 50\ m^3/s$.

b) let B = base width of 3-h UH

$\therefore$ Area of UH = Volume of 1cm depth over the catchment

$\Rightarrow \frac{1}{2} \times B \times 50 \times 3600 = 567 \times 10^6 \times \frac{1}{100}$

$\therefore B = 63$ hours.

Example - 3.11 A storm with 10 cm of precipitation produced a direct run off 5.8 cm. The duration is 16 hrs. Estimate ϕ-index.

P - 110. Subramanya

Pulse		1	2	3	4	5	6	7	8
time	0	2	4	6	8	10	12	14	16
Rainfall	0	.4	1.3	2.8	5.1	6.9	8.5	9.5	10.
increment per hr	0.0	0.4	0.9	1.5	2.30	1.80	1.60	1.0	0.5

Sol^n: Assume $\Delta t = 2h$. $M = 6$ $\therefore t_e = M \times \delta t = 12$ hr.

Select 6 pulse in decreasing order of I_i. So pulse 1 and 8 are ommitted.

Run-off; $R_d = 5.8 = \sum_{1}^{6} (I_i - \phi)\Delta t$

$$5.8 = \sum I_i \times \Delta t - \sum_{1}^{6} \phi \Delta t = (0.9 + 1.5 + 2.30 + 1.80 + 1.60 + 1) \times 2 - \phi \times 6 \times 2$$

$$12\phi = 9.1 - 5.8 \Rightarrow \phi = 0.275 \text{ cm/hr} \quad \underline{\text{Ans}}$$

Example. 3.14: Precipitation value is 9 cm and runoff is 5 cm. Rainfall data is given below. with respect to time

BWDB-17

time	1	2	3	4	5	6
Rainfall	0.4	2.2	4	2.2	1.4	0.9
Rainfall intensity I cm/h	0.4	2.2	4	2.2	1.4	0.9

Assume pulse $M = 5$ $\delta t = 1$ hr. $\therefore t_e = \Delta t \times M = 5$ hr.

Neglect first pulse.

$\therefore$ Run-off, $R_d = 5 = \sum_{i=1}^{5} (I_i - \phi)\Delta t \Rightarrow 5 = (2.2 + 4 + 2.2 + 1.4 + 0.9) \times 1 - (5 \times 1)\phi$

$\Rightarrow 5\phi = 10.7 - 5$

$\therefore \phi = 1.14$ cm/h. $\because I_i \geq \phi$.

So $\phi = 1.14$ cm/hr. for strip. $M = 6$.

Example-3.16: Precipitation is 9 cm and Runoff is 5 cm. Rainfall data is given below.

time	1	2	3	4	5	6
Rainfall	0.4	2.2	4	2.2	1.4	0.9

Solⁿ: Assume pulse $m=5$, $\Delta t = 1$ hr. $\therefore T_e = \Delta t \times m = 5$ hr.

Rainfall, I_i = Rainfall intensity

$$\therefore \text{Run off, } R_d = 5 = \sum_{i=1}^{5}(I_i - \phi)\Delta t = \sum_{i=1}^{5} I_i \Delta t - \phi \times 5 \times \Delta t$$

$$5 = (2.2+4+2.2+1.4+0.9) - 5\phi$$

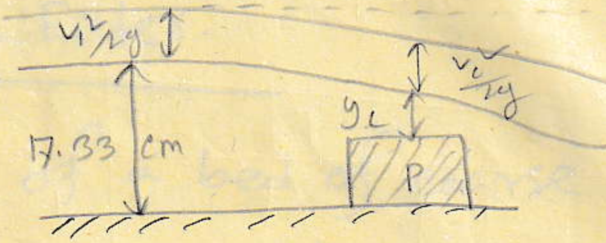
$$\Rightarrow \phi = \frac{10.7-5}{5} = 1.14 \text{ cm/hr} \leq I_i$$

$\therefore \phi = 1.14$ cm/hr. Ans:

[Total rainfall $P = \sum_{1}^{N} I_i \cdot \Delta t$, then $\boxed{P - \phi \cdot t_e = R_d}$ P-110 subramanya.

φ-index: φ-index is the average rainfall above which the rainfall volume is equal to the run-off volume.

Example-2.16: Determine the height of broad crest weir (P) from given data. width of flume = 25 cm Actual discharge = 7020.5 cm³/s Ignore head loss.



Ans:- $Q = 7020.5\ cm^3/s$. $b = 25\ cm$

$h_1 = 17.33\ cm$ $P = ?$

$$\therefore h_1 + \frac{V_1^2}{2g} + \cancel{Z_1}^{0} = P + y_c + \frac{V_c^2}{2g}$$

$$\Rightarrow P = h_1 - y_c + \frac{V_1^2 - V_c^2}{2g}$$

$$= 17.33 - 4.315 + \frac{16.20^2 - 65^2}{2 \times 981}$$

$$= 10.995\ cm.\ \underline{Ans.}$$

$$= 11\ cm.$$

$$y_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}} = \sqrt[3]{\frac{1 \times 7020.5^2}{9.81 \times 25^2}}$$

$$= 4.315\ cm$$

$$\therefore V_c = \frac{Q}{A_c} = \frac{7020.5}{25 \times 4.15} = 65\ cm/s$$

$$V_1 = \frac{7020.5}{25 \times 17.33} = 16.20\ cm/s.$$

Trickling Filter.

A trickling filter consists of a bed of coarse materials such as stones, slats or plastic materials overwhich wastewater is passed. It is a biological treatment process. The wastwater is typically distributed over the surface of the rocks by a rotating arm. As the wastwater trickles through the bed a microbial growth establishes itself on the surface of the stone.

The wastwater passes over the stationary microbial population providing contact between the micro-organisms and organics.

Recirculation ratio.

→ An important element in trickling filter design is the provision for return of a portion of the effluent to flow through the filter. This practice is called recirculation.

→ The ratio of the return flow to the incoming flow is called recirculation ratio.

Example-5: Design a septic tank to serve a household of ten persons who produce 90lpcd of wastwater. The tank is to be desludged every three years. [BCS - 36th]

Sedimentation:

Hydraulic mean time

$t_h = 1.5 - 0.3 \log (Pq)$

$= 1.5 - 0.3 \log (90 \times 10) = 0.61$ day.

$\therefore$ Sedimentation volume $= V_h = 10^{-3} (Pq)\ t_h$

$= 10^{-3} \times 10 \times 90 \times 0.61$

$= 0.55\ m^3$

Sludge digestion:

Assume design temperature 25^0.

$\therefore t_d = 30 \times 1.035^{35-t}$

$= 30 \times 1.035^{35-25} = 42.3$ days

$\therefore$ Sludge disgestion volume, $V_d = 0.5 \times 10^{-3} \times P t_d$

$= 0.5 \times 10^{-3} \times 10 \times 42.3$

$= .21\ m^3$

Sludge storage.

Assume sludge accumulation rate $c = 0.06\ m^3/P/y$

$\therefore V_{sl} = cPN = 0.06 \times 10 \times 3 = 1.8\ m^3$

∴ Over all effective volume

$V = V_h + V_d + \cancel{V}\, 1.4 V_{sl} = 0.55 + 0.21 + 1.4 \times 1.8$
$= 3.28\ m^3$

Tank effective depth:

Assume a cross-sectional area, $A = 3\ m^2$

∴ Depth of sludge, $d_{sl} = V_{SL}/A = 1.8/3 = 0.60\ m$

Max^n. submerged scum depth, $d_{ss} = 0.4\, V_{SL}/A$
$= 0.4 \times 1.8/3$
$= 0.24\ m$

Minimum scum clear depth $= 0.075\ m$

" sludge " $= 0.3\ m$

∴ Total clear space depth $= 0.075 + 0.3 = 0.375\ m$

Depth of sedimentation $= V_h/A = 0.55/3 = 0.183$
< 0.375

∴ The effective depth $= 0.60 + 0.375 + 0.24$
$= 1.215\ m$

∴ Suitable overall dimention of septic tank can be chosen as $1 \times 3 \times 1.5\ m^3$

Use two compartment septic tanks first comp vol^r $3\ m^3$ and $1.5\ m^3$ respectively

Methods for the analysis of pressure in the distribution system are

1. - Equivalent pipe method
2. - Hardy cross method
3. - Method of sections and circle method.
4. Graphical method
5. Electric network analysis method.
6. Pitometer distribution studies method.

Principles / law of Hardy cross method are

1. In each seperate pipe there will be a relation between the head loss in the element and the quantity of water flowing through it.
2. At each junction, the algebric sum of the quantities of water entering and leaving the junction is zero. $\Sigma Q = 0$.
3. In any closed path or circuit, the algebric sum of the head loss in the individual elements is zero. i.e $\Sigma h = 0$,

Procedure for Hardy cross method: P-400 ITN

1. Carefully examine the network and assume reasonable rates of flow in each pipe such that inflow equals outflow at each junction.

2. In each loop, Determine the head loss H and H/Q for all pipes.

3. With due attention to sign, compute the total head loss around each circuits

4. Compute $\Sigma H/Q$ for the same circuit without considering sign.

5. The correction, Δ is computed for each loop by the equation $\Delta = -\dfrac{\Sigma H}{x\, \Sigma H/Q_a}$

6. Apply the correction to each pipe in each loop. when the sign is negative (-) decrease the clockwise flow and increase the counter clockwise flow. and vice-versa.

7. With adjusted flows, Repeat the procedure for 2nd Approximation. The procedure is continued untill the desire accuracy is achived.