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Sanitary Engineering Project

Group 1

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Abstract

This research project focuses on the examination of aquatic vegetation, the quality of water, its treatment, as well as the supply and distribution of water. Additionally, we analyze population estimates and water quantities. Furthermore, our investigation extends to the field of sewage treatment facilities, encompassing aspects such as sewage volume, wastewater treatment, collection, sanitation, the potential for wastewater reuse, and the volume of wastewater following the treatment process.

1- PROJECT DEFINITION

This project involves the construction of a water treatment plant and the design of both the water distribution network and the sewage network.

1.1 THE PROBLEM

The challenge at hand is to create a water treatment plant capable of supplying water to a city in both 2040 and 2060. This entails determining the appropriate number and sizes of pipes for the water distribution network to ensure efficient delivery to households, as well as identifying the optimal number and diameters of pipes for the sewage network.

1.2 STUDY OBJECTIVES

The primary objectives of this study are as follows:

- **Designing a water treatment plant**
- **Planning the water distribution network**
- **Establishing the wastewater network design**

1.3 EXISTING SOLUTIONS

For the water treatment plant, the existing solutions involve the use of pumping mechanisms, filters, and tanks to treat the water and produce clean, potable water suitable for human consumption. Regarding the distribution network, pipes, pumps, and elevated tanks are utilized to transport water to residential areas. In the case of the wastewater network, the design revolves around determining pipe slopes, utilizing gravity, and selecting appropriate diameters to convey the wastewater to the sewage plant.

1.4 DESIGN CONSTRAINTS

There are no specific design constraints or limitations identified for this project

2- CUSTOMER NEEDS and BACKGROUND

The customer's needs revolve around water treatment, extraction of drinking water, ensuring water delivery to the highest level in the distribution network, maintaining suitable water pressure during peak consumption hours, and efficient collection of wastewater from households for transport to sewage stations. These requirements should be met while ensuring economic viability and maintaining water quality.

3 -GENERATED IDEAS Water treatment plant:

The water treatment process involves treating water obtained from the source and disinfecting it by adding chlorine (CL2). Water distribution network: The design of the distribution network includes determining the appropriate pipe diameters to withstand pressure and ensure water reaches all areas of the city, even the farthest points. Wastewater network: The wastewater network is responsible for transporting wastewater from the city to the wastewater treatment plant.

4 -FINAL DESIGN

Water treatment plant: The final concept includes the utilization of pumps, filters, and tanks to treat water and produce purified water suitable for human consumption. Water distribution network: The distribution network involves the use of pipes, pumps, and overhead tanks to deliver water to residential areas. Sewage pipe network: The design of the sewage pipe network considers factors such as pipe slope and gravity, along with the selection of suitable pipe diameters for transporting wastewater to the treatment plant.

5-Population and Water Consumption

$P_{1996} = 60200$ capita

$P_{2006} = 72000$ capita

$P_{2016} = 87400$ capita

5.1- Forecasting Population

Arithmetic Method:

Year	Population (P)	ΔP	ΔT	Ka=ΔP/ΔT
1996	60200			
2006	72000	11800	10	1180
2016	87400	15400	10	1540
				∑Ka =2720

Ka (average) = ∑Ka /N = 2720/2=1360

$P_n = P_o + K_a * \Delta t$

$P_{2040} = P_{2016} + 1360 * \Delta t$

=87400+1360*(2040-2016) =120040 capita

$P_{2060} = P_{2016} + 1360 * \Delta t$

=87400+1360*(2060-2016) = 147240 capita

Geometric method:

Year	opulation (P)	lnp	ΔlnP	ΔT	Kg=ΔlnP/ΔT
1996	60200	11.005			
2006	72000	11.184	0.17899377	10	0.0179
2016	87400	11.378	0.193829164	10	0.0194
					∑Kg =04

Kg (average) = ∑Kg /N = .04/2=.02

$\ln(P_n) = \ln(P_o) + Kg * \Delta t$

$\ln P_{2040} = \ln P_{2016} + Kg * \Delta t$

$\ln P_{2040} = 11.379 + 0.02*24 = 11.859$

$P_{2040} = 141350$ capita

$\ln P_{2060} = \ln P_{2016} + Kg*\Delta t$

$\ln P_{2060} = 11.37 + .02*44 = 12.25$

$P_{2060} = 208981.288$ capita

Annual growth rate method

Year	Population (P)	P_n/P_o	ΔT	$m=(P_n/P_o)1/\Delta T$	m-1
1996	60200				-1
2006	72000	1.196	10	1.0180	0.018
2016	87400	1.214	10	1.0195	0.0195
					∑Km=.04

Km (average) = ∑Km /N = .04/2=.02

$P_n = P_o * (1+m-1)^{\Delta t}$

$P_{2040} = P_{2016} * (1+m-1)^{\Delta t}$

$P_{2040} = 87400(1+0.02)^{24} = 140577.4$ capita

$P_{2060} = P_{2016} * (1+m-1)^{\Delta t}$

$P_{2060} = 87400(1+0.02)^{44} = 208890.6$ capita

Year	Arithmetic Method	Geometric method	Annual growth rate method	P average
2040	120040	141350	140577.4	133989.133
2060	147240	208981.2	208890.6	188370.6

5.2- Design Flow:-

For stage (1)(at 2040)

$q_{2040} = 250$ L/c/d

$Q_{2040\ avg} = P_{avg2040} * q_{2040}$

$Q_{2040\ avg} = 133989.133 * \frac{250}{1000} = 33497.28\ m^3/d \& 0.387 m^3/s$

For stage (2) (at 2060):-

$q_{2060} = 270 L/c/d$

$Q_{2060\ avg} = P_{avg2060} * q_{2060}$

$Q_{2060\ avg} = 188370.6 * \frac{270}{1000} = 50860.062\ m^3/d\ \&\ 0.588 m^3/s$

Flow	2040	2060
Q av	$0.387 m^3/s$	$0.588 m^3/s$
Q_{Max} Monthly $= 1.4 * Q_{av}$	$0.5418 m^3/s$	$0.8232\ m^3/s$
Q_{max} daily $= 1.8 * Q_{av}$	$0.6966\ m^3/s$	$1.0584\ m^3/s$
Q_{max} hourly= $= 2.5 * Q_{av}$	$0.9675\ m^3/s$	$1.47\ m^3/s$
Q Design $= 1.1 * 1.4 * Q_{av}$	$0.59598\ m^3/s$	$0.90552\ m^3/s$

5.3- Shore Intake

- **For stage (2) at 2060:-**

  $Q_d = .905\ m^3/sec$

Design of Conduit Pipes :-

$Q_d = A * v$

Assume $v = 1.0\ m/sec$ & $A = .905\ m^2$

$A = N \frac{\pi \varphi^2}{4}$ & Assume $N = 3$

$.905 = 3 * \frac{\pi \varphi^2}{4}$ $\varphi = .61m \rightarrow \varphi = 700\ mm$

$v\_act = Q/A = 0.905 / 1.15$ $= .78\ m/s$

- **Check for first stage 2040:-**

$Q_d = .59\ m^3/sec$ N=2

$$v = \frac{Q_d}{N * \frac{\pi \varphi^2}{4}} = \frac{.595}{2 * \frac{\pi * (0.7)^2}{4}} = < (0.76 \sim 1.5\ ) m/sec$$

Stage (1): 2φ 700

Stage (2):3 φ 700

- ***Head Losses Through The Pipe:-***

1-For stage (2) at 2060

Assume :-

L = 100m & f = .04

Friction losses:-

$$hf = \frac{flv^2}{2g\emptyset} = \frac{.04 \times 100 \times .78^2}{2 \times 9.81 * .7} = 17.7\ cm$$

$$h\,min = .2 * hf = .2 * 17.7 = 3.5\ cm$$

$$Total\ losses = h_f + h_{min}$$

$$Total\ losses = 17.7 + 3.5 = 21.2cm$$

Water level in sump = water level in canal – total losses

$$Water\ level\ in\ sump\ = \ 15 - .212 = 14.7\ m$$

2-For stage (1) at 2040

Friction losses:-

$$hf = \frac{flv^2}{2g\emptyset} = \frac{.04 \times 100 \times .766^2}{2 \times 9.81 * .7} = 16.3\ cm$$

$$h\,min = .2 * hf = .2 * 16.7 = 3.2\ cm$$

$$Total\ losses = h_f + h_{min}$$

$$Total\ losses = 16.3 + 3.2 = 19.5cm$$

- ***Design of screen:-***

Assume:

B = 1.5 φ S = 3cm = 0.03m a=1.5 cm

*B=N*S +a(N-1)*

1.5.7= 3N+1.5N-1.5 N= 23.6 ≃ 24 no.of bars 24-1=23*

Losses of screen

$$h_{screen} = \frac{1.4({v_{th}}^2 - v_{app^2})}{2g}$$

$$v_{th} = \frac{Q_d}{n(d \times s \times N)/sin(\theta)}$$

- *d = H.W.L- Bed Level* *23.5-20=3.5*
- *s = 3cm*
- $\theta = 60$
- *n =3*
- *N=24* $v_{th} = \frac{.905}{3(3.5 \times .03 \times 24)/sin(60)}$ *=.103m/s*

$v_{app} = \frac{Q_d}{n \times l \times d} = \frac{.905}{3 \times 10.5 \times 3.5}$ *=.082m/s*

$$h_{screen} = \frac{1.4(.103^2 - .082^2)}{2 * 9.81} = 2.7 \times 10^{-4} = .002 < 30 \ Ok$$

- ***Design of Force main:-***

Assume v=1.5 m/s

1-For stage (2) at 2060

Q_d *= A*V &.905 =1.5* A*

$A = .603m^2 = \frac{\pi \emptyset^2}{4}$ & $\emptyset = .87 \simeq 900mm$ $A_{acc} = .63m^2$

v_{ac} *=.905/.63 =1.4 m/s OK*

Head loss (hl)

$s = (\frac{Q_d}{.278 \times c \times D^{2.63}})^{\frac{1}{.54}} = (\frac{.905}{.278 \times 120 \times .9^{2.63}})^{\frac{1}{.54}} = 2.09*10^{-3}$

$h_f = S * L = 2.09*10^{-3}*100 = .209\ m$

$h_m = .2\ h_f = .041\ m$ & $hL = .209 + .041 = .25\ m$

2-For stage (1) at 2040

$A_{acc} = .63m^2$ v_{ac} *=.595/.63 =.944 m/s OK*

$s = (\frac{Q_d}{.278 \times c \times D^{2.63}})^{\frac{1}{.54}} = (\frac{.595}{.278 \times 120 \times .9^{2.63}})^{\frac{1}{.54}} = 9.6*10^{-4}$

$h_f = S * L = 9.6*10^{-4}*100 = .096\ m$

$h_m = .2\ h_f = .0912\ m$ & $hL = .0912 + .096 = .1152\ m$

- Design of low lift Pumps

1-For stage (2) at 2060

- $Q_d = .905\ m^3/s$ & 905lit/s
- Assume $Q_{pump} = 300$ lit/s

$$no\ of\ pump = \frac{Q_d}{Q_{pump}} = \frac{905}{300} = 3.016 \quad using\ N = 5$$

$$n_{working} = 5\ working + 3\ stand\ by$$

$$Q_{pump} = \frac{905}{5} = 181\ L/s$$

- Head of Pump $\quad HL = h_{static} + h_{dynamic}$

$$H_{static} = berm\ level - water\ level\ in\ sump + (6-8)m$$

$24.5 - 22.5 + 6 = 8\ m \qquad h_{dynamic} = 0.25\ m$

$HL = 8 + .25 = 8.25\ m$

$Ht = 8.25 + 0.25 + 0.02 + .212 = 8.48\ m$

$$H.P = \frac{\gamma * Q_d * H_t}{75\eta_1\eta_2} = H.P = \frac{1*181*8.48}{75*.63} = 32.48\ HP$$

2-For stage (1) at 2040

- $Q_d = .595\ m^3/s$ & 595/s
- Assume $Q_{pump} = 300$ lit/s

$$no\ of\ pump = \frac{Q_d}{Q_{pump}} = \frac{595}{300} = 1.98 \quad using\ N = 2$$

$$n_{working} = 2\ working + 2\ stand\ by$$

- Design of Sump:-

1.For stage (2) at 2060

$T = 5\ min$

$V = Q * T$

$V = 0.905 * (5 * 60) = 271.5\ m^3$

$$depth = water\ level\ in\ sump - bed\ level + \frac{L_{conduit\ pipe}}{100} + 1$$

$$depth = 22.5 - 20 + \frac{100}{100} + 1 = 4.5\ m$$

$A = \frac{V}{d}$ $A = \frac{271.5}{4.5} = 60.33\ m^2$

$A = W * L$

*L = N * S assume S= 2.5 L= 8*2.5 = 20 w= 60.33/20 3.02m*

2.For stage (1) at 2040

$$V = 0.595 * (5 * 60) = 187.5\ m^3$$

$A = \frac{V}{d}$ $A = \frac{187.5}{4.5} = 39.66\ m^2$

L= 13.13 w=3.02m d=4.5

5.4- Rapid Mixing Tank

Assume :-

Circular section & Detention time T = 45sec & Depth d = 2m

G value = 700 sec^{-1}

$V = Q_d * T$ = 0.905 * 45 = 40.725 m^3

Cross section area $= \frac{V}{d}$ $A = 40.725/2 = 20.36m^2$

$A = \frac{\pi\varphi^2}{4}$ $20.36 = \frac{\pi\varphi^2}{4}$ $\varphi = 5m$

∴ Dimension 5 * 2

5.5-Power

$P = G^2 * V * \mu = (700)^2 * 40.725 * 1.002 * 10^{-3} = 19955.25\ watt = 19.9\ K.watt$

5.6- Coagulation:

- *Alum Solution Tank*

S= (20-40) mg/L

Q_d =1.1 Q $_{Max\ Month}$ =0.90552 m^3/s =78236.9 m^3/d

*Q_y*S = 78236.9*40*365*10^{-6}=1142.259 t/year*

$Vol = \frac{Q_d * S}{C * Y * 10^6} = \frac{78236.9 * 40}{1.05 * 0.1 * 10^6} = 29.804\ m$

Assume No. of Tanks = 3 Vol [for / tank] = $\frac{29.8}{3} = 9.93 m^3$

$A = \frac{Vol}{d} = \frac{9.93}{1.5} = 6.623\ m^3$ $A = L^2$ $L = 2.57m$

Use 3 Tanks with (2.57*2.57*1.5)m

5.7-Sedimentation Tank:

Assume S.l.R = 33m3/m^2/d circular section

$\emptyset = 35m$ T= 3h

- **Design:**

$A = \frac{Q_d}{S{\cdot}L{\cdot}R} = \frac{78236.9}{33} = 2370.81\ m^2$

$A = N * \frac{\pi \emptyset^2}{4}$ $N = \frac{4 * 2370.81}{\pi * (35)^2} = 2.4$ Taking N = 3 Tanks

$2370.81 = 3 * \frac{\pi \emptyset^2}{4}$ $\emptyset = 31.7m \simeq 32\ m$

$Vol = Q_d * T = 78236.9 * \frac{3}{24} = 9779.5\ m^3$

$Depth = \frac{Vol}{Surface\ Area} = \frac{9779.5}{2370.81} = 4m$

Use 3 Tanks (32m * 4 m)

- **b-Checks:-**

$T = \frac{Vol}{Q} = \left[3 * \frac{\pi * (32)^2}{4} * 4\right] / 78236.9$

T = 0.123 day = 2.95 hr [2-3]hr Safe

$Vr = \frac{Q}{\pi \emptyset\ dN} = \frac{78236.9}{\pi * 32 * 4 * 3} = 64.88\ m/d = 0.045 m/min$ $Vr < 0.3\ m/min$

Safe

$Over\ flow\ weir = \frac{Q}{\pi * \emptyset * N} = \frac{78236.9}{\pi * 32 * 3} = 259.54\ m^3/m^2/d < 300\ m^3/m^2/d$

Safe

Use 3 Tanks (32 * 4) m

Volume of studge hopper:

Assume:-

- Suspend solid concentration (s) = 80 mg/Lit
- Removal ovation ® = 90 %
- water content (W_c) = 96%

- *Specific weight of sludge (γ_s) = 1.03 t/m^3*
- *Rate of sludge empting (m) = 3*

$$V_{ol} = \frac{Q_d * S * R}{m * n * ((1 - w_c) * \gamma_s * 10^6}$$

- ***Clari Flocculation Tank:***

For sedimentation Zone

Assume

- ***Detention time (T_1) = 2.5 hr.***
- ***Depth (d_1) = 4m***
- ***$\varnothing_1$ = 30 [$\varnothing_1 \leq 40$]***
- ***For flocculation Zone***

Assume

- ***Detention time (T_2) = 0.5 hr.***
- ***Depth (d_2) =d1-0.5=3.5 m***

a- Design of sedimentation Zone:

$T_t = T_1 + T_2 = 2.5+0.5=3hr$

$Vol = Q_d * T_t = 78236.9 * \frac{3}{24} = 9779.61\ m^{3\sim\sim}$

$At = \frac{vol}{d_1} = \frac{78236.9}{4} = 2444.90 m^2 = \frac{N * \emptyset^2}{4}$

$N = \frac{4 * 2444.90}{\pi * (30)^2} = 3.46 \sim 4$ *Take N = 4* $2444.90 = \frac{4 * \pi * (\varnothing)^2}{4}$

$\varnothing_1 = 27.9\ m \approx 28\ m$

- ***Design of flocculation Zone:***

$V_{ol} = Q_d * T_2 = 78236.9 * \frac{0.5}{24} = 1629.93\ m^3$

$A_{flc} = \frac{volume}{d_2} = \frac{1629.93}{3.5} = 465.69\ m^2$

$A_{flc} = N \frac{\pi(\emptyset_2)^2}{4}$, $N=4$ $\varnothing_2 = 13m$

- ***C- Cheek***

$S.L.R = \frac{Q/N}{\frac{\pi}{4}(\emptyset_1^2 - \emptyset_2^2)} = \frac{78236.9/4}{\frac{\pi}{4}28^2 - (13)^2} = 40.5\ m^3/m^2/d < 45\ m^3/m^2/d$ *safe*

- ***Design of V-notch weir:-***

$$weir\ over\ flow\ rate\ = \frac{Q}{\pi\varphi}$$

$$= \frac{78236.9}{4*\pi*28} = 222.466m^3/m^2/d\ <\ 300m^3/m^2/d \qquad ok\ \ safe$$

the discharge through V-notch weir is given by :- $q = 1.46h^{2.5}$

Assume

- *h = 0.05m*
- $q = 1.46*(0.05)^{2.5} = 0.816*10^{-3}m^3/sec = 0.816lit/sec$

$$Number\ of\ required\ V - notch\ weir\ (N)\ = \frac{Q}{q}$$

$$Q = \frac{78236.9}{4} = 19559.2\ m^3/d = 0.226m^3/sec\ = 226lit/sec$$

$$N = \frac{226}{0.816} = 276.96 = 277\ weirs$$

$$distance\ between\ weirs\ center\ lines\ = \pi\frac{\varphi}{N} = \ \pi\frac{28}{277} = 0.31m$$

- ***design of pipes :-***

Inlet and outlet pipes Q_d of one tank = $\frac{74679}{4} = 78236.m^3/d = 0.226\ m^3/sec$

Inlet pipe velocity = 1.0 m/sec

$Q = A*v$

0.226= $A*1$ *A =0.226 m²*

$$A = \frac{\pi\varphi^2}{4} \quad 0.226 = \frac{\pi\varphi^2}{4} \qquad \varphi = 0.536m \qquad Use\varphi = 550mm$$

$$v_{act} = \frac{Q}{A} = \frac{0.226}{\pi\frac{(0.55)^2}{4}} = 0.95m/sec < 1.5m/sec \qquad ok$$

Outlet pipe velocity = 0.6 m/sec

$Q = A*v$ *0.226=* $A*0.6$ *A =0.376 m²*

$$A = \frac{\pi\varphi^2}{4} \quad 0.376 = \frac{\pi\varphi^2}{4} \qquad \varphi = 0.692m \qquad Use\ \ \varphi = 700mm$$

$v_{act} = \frac{Q}{A}$ $\quad v_{act} = \frac{0.226}{\pi\frac{(0.7)^2}{4}} = 0.587m/sec < 0.7m/sec \quad ok$

- ***Volume of sludge hopper(∀):-***

$$\forall = \frac{Q_d * S * R}{m * n * (1 - W_c)\gamma_s * 10^6}$$

$$\forall == \frac{78236.9 \ * 80 * 0.9}{3 * 4(1 - 0.96) * 1.03 * 10^6} = \ 11.393\ m3 = 10.87m^3$$

Assume the time of sludge emptying = 10min

$$Q_{sludge} = \frac{\forall_{sludge\ hopper}}{T} = \frac{11.393}{10*60} = 0.0189\ m^3/sec$$

Assume the velocity (v) = 1.5m/sec (1-2) m/sec

$A = \frac{Q}{v}$ $\quad A = \frac{0.018}{1.5} = 0.0126m^2 \qquad A = \frac{\pi\varphi^2}{4} \qquad 0.0126 = \frac{\pi\varphi^2}{4}$

$\varphi = 0.126m \qquad use\varphi = 150mm$

5.8 -Design of rapid sand filter:-

Assume :-

- **Rate of filtration = R.O.F = 150 $m^3/m^2/d$**
- **Surface area of filter = A_{filter} = L*W ≤ 100 m^2**
- **Thickness of sand layer = 0.65 m**
- **Thickness of gravel layer = 0.45 m**
- **Time of back wash = 15 min**

Design:-

1-For stage (1) at 2040

$$Q_d = 1.07 * 1.4 * 133989.133 * \frac{250}{1000} = 50178.93\ m^3/d$$

$A_{filters}\frac{Q_d}{R.O.F} = \frac{50178.93}{150} = 334.52\ m^2$ **Use filter 6*6 m**

$$\text{Number of filters (N)} = \frac{A_{filters}}{A_{filter}}$$

$$\text{N} = \frac{334.52}{8 * 6.5} = 8\ filters$$

∴use 8 filters + 2 for back wash

Pipe	Velocity (m/s)
Inlet	0.3-0.8
Outlet	1-2
Wash water supply	1.5-3
Wash water drain	0.9-2
Preparing filter to waste	1.6-3.2
Air supply	15-20

- ***For one filter:***

$Q = \frac{Q_d}{N} = \frac{50178.93}{10} = 5017.893\ m^3/d = 0.058 m^3/s$

$Q_{backwasher} = A * rate\ of\ backwash_1 = (6*6*) * 450 = 16200\ m^3/s = 0.19 m^3/s$

$Q\ air = A * rate\ of\ air = (6*6) * (1*6) = 216 m^3/d = 0.003\ m^3/s$

- ***Design of pipes:***

1. ***Inlet Pipe***

$V = 0.8 m/s \quad Q = A * v \quad A = \frac{0.058}{0.8} = 0.0725\ m^2$

$A = \frac{\pi Ø^2}{4} \quad Ø = \sqrt{\frac{4*0.0725}{\pi}} \qquad Ø = 0.303\ m \qquad Use\ Ø = 350\ mm$

2. ***Outlet pipe:***

$V = 2\ m/s \quad A = \frac{0.058}{2} = 0.029\ m^2$

$Ø = \sqrt{\frac{4*0.029}{\pi}} = 0.192\ m \qquad Use\ Ø = 200 m\ m$

3. ***wash supply pipe:***

$V = 2\ m/s \qquad A = \frac{0.19}{2} = 0.95\ m^2$

$Ø = \sqrt{\frac{4*0.95}{\pi}} \qquad Ø = 0.35 \qquad Use\ Ø\ 400mm$

2-For stage (2) at 2060

$$Q_d = 1.07 * 1.4 * 188370.6 * \frac{270}{1000} = 76188.372\, m^3/d$$

$A_{filters} \frac{Q_d}{R.O.F} = \frac{76188.372}{150} = 507.922\, m^2$ Use filter 6*6 m

$$\text{Number of filters (N)} = \frac{A_{filters}}{A_{filter}}$$

$$N = \frac{507.922}{8 * 6.5} = 10 filters$$

∴use 10 filters + 2 for back wash

5.9-Ground Tank

$C_1 = Q\_{(max.monthly)*}0.5\ hr$

Q Max = 0.8232*24*60*60 =71884.8 m^3/day

C_1= 71884.8 *0.5/24=1497.6 m^3

C_2=0.4*Q_avg*day

$_{C2}$=0.4*50860.062 *1=20344.02 m^3

C_3=Q_(m ax.monthly)*8 hr

C_3= 71884.8 *8/24=23961.6 m^3

C_{fire} =(120*P)/10000

C_{fire} =(120*188370.6)/10000=2260.4472 m^3

∀=max.of c_1 or c_2 or c_3+4/5 c_fire

∀=23961.6 +4/5*2260.4472 =25769.95 m^3

∀=N*L*W*d

Assume:-

L = 50 m & W = 50 m & d = 6 m

25769.95 =N*50*50*6

N = 1.7 Use N = 2 tanks

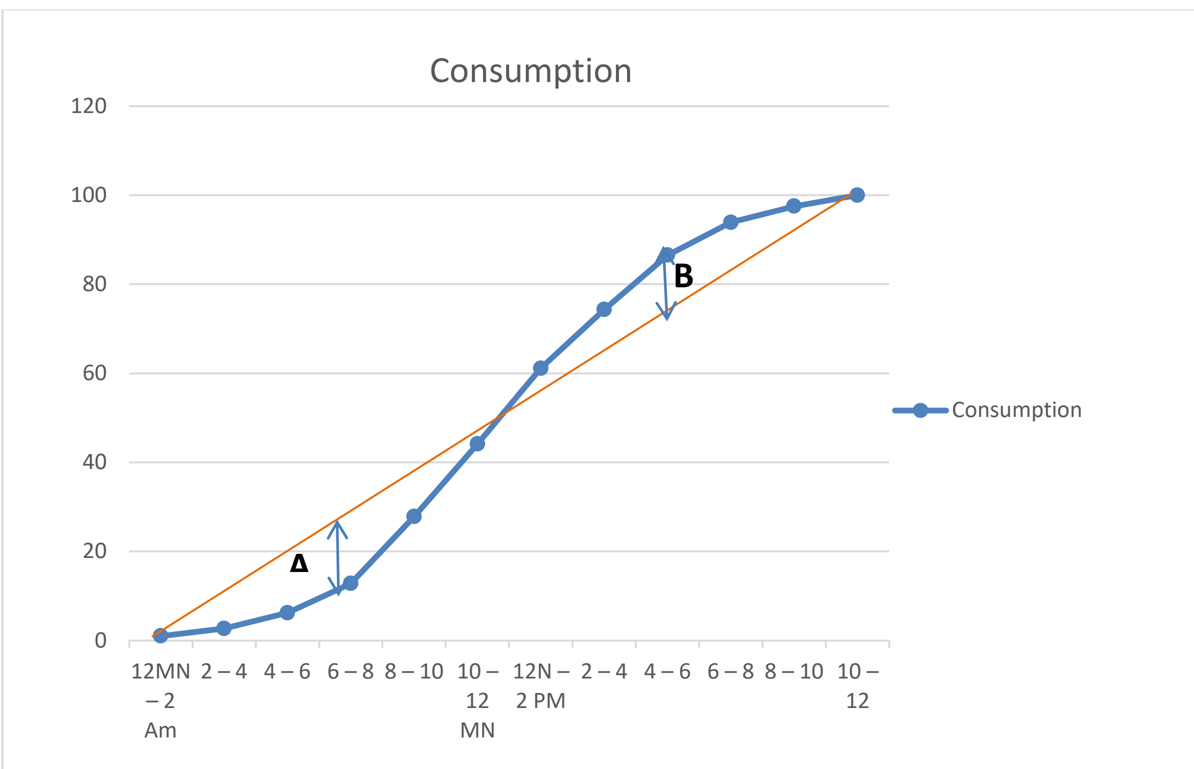
25769.95 =2*50*W*6 W = 42.9≃43 m

Use 2 tanks with L = 50 m & W = 43m & d = 6 m

	Stage (1)	Stage (2)
conduit pipe Losses	2 ∅ 700 .195 m	3 ∅ 700 0.212 m
Screen Lossess	N screens =24 Ø=0.9m N bars =23 B=1.5Ø=1.05m 0..02m	N screens =24 Ø=0.9m N bars =23 B=1.5Ø=1.05m 0.02m
Sump	(12*4*4.5)m	(15*4*4.5) m
low Lift pump	2 pump + 1 stand by	5 pumps + 3 stand by
Force Main Lossess	Q= 0.595 d= 1m h_f = 0.096m h_{min} = 0.091m	Q = 0.905 d= 1 m h_f = 0.209m h_{min} = 0.041m
Rapid mixing tank	∅ = 5m d= 2 m N = 1 tanks	∅ = 5m d= 2 m N = 1 tanks
clari = flocculation	$\varnothing_{sed}$ = 32 m $\varnothing_{Floc}$ = 13 m d_{sed} = 4 m d_{floc} = 3.5 m	$\varnothing_{sed}$ = 32 m $\varnothing_{Floc}$ = 13 m d_{sed} = 4 m d_{floc} = 3.5 m
Filtration	8 Filters + 2 back wash = 10 filters	10 Filters + 2 back wash = 12 filters
Ground Tank	2 tank (L=50, W = 43 , d=5)m	3 tank (L=50, W = 43 , d=5)m

5.10-Elevated tank

	Time	Consumption lit/capita/2hrs	Accumulative
1	12 MN - 2Am	1	1
2	2-4	1.7	2.7
3	4-6	3.5	6.2
4	6-8	6.6	12.8
5	8-10	15.1	27.9
6	10-12N	16.2	44.1
7	10N-2PM	17	61.1
8	2-4	13.2	74.3
9	4-6	12.2	86.5
10	6-8	7.4	93.9
11	8-10	3.6	97.5
12	10-12Mn	2.5	100



A =30-12 = 18

B= 90-75 =15

$$Capacity = \frac{(A+B)P}{1000} + \frac{1}{5}\,Cfire$$

$$C = \frac{(18+15)P}{1000} + \frac{120 * P}{5 * 10000}$$

1-For stage (2) at 2060

$$C = \frac{(18+15)188370.6}{1000} + \frac{120*188370.6}{5*10000} = 6668.319\ \mathrm{m}^3$$

Assume NO. of tanks = 6 tanks.

$V_{one} = \frac{6668.319}{6} = 1111.3865\ m^3$ Assume $d = 8\ m$ $A = 138.9\ m^2$

$$A = \frac{\pi\varphi^2}{4} \quad \rightarrow \quad \varphi = 1330\ mm$$

2-For stage (1) at 2040

$$C = \frac{(18+15)133989.133}{1000} + \frac{120*133989.133}{5*10000} = 4743.21\mathrm{m}^3$$

Assume NO. of tanks = 4743.21/1111.3865= 4.26   using 5

5.11-Design of network

- **transmission mains**

$Q_d = Q_{\text{max.daily}} + Q_{\text{fire}}$

$Q_{\text{av}} = 588$ L/s   & Qmax.daily = 1.8 *588 = 1059.58 L/s

$= 1059.58 + 50 = 1109.58\ \mathrm{l}/S$

- **Main and secondary pipes**

$Q_d =$ the biggest of $(Q_{\text{max.hourly}}\ \&Q_{\text{max.daily}} + Q_{\text{fire}})$

$Q_{\text{max.hourly}} = 2.5*588 = 1470\ \mathrm{l}/S$

$Q_{\text{max.daily}} + Q_{\text{fire}} = 1109.58\ \mathrm{l}/S$

$Q_d = 1470\ \mathrm{l}/S$

- **Minor distributions**

Q_d = Fire flow = 50 l/S

- **Service connection.**

peak hourly flow = 1470 l/S

- **Design consideration for distribution system**

1- **Minimum size in pipe network 150mm for the secondary**
2- **pipes 200mm for the main pipes**

- **Hydraulic gradient (S) = 1-3 % for main pipes.**
- **The Velocity (v) = 0.8 – 1.5 M\S**
- **The Pressure (p) in my point P = (25m) for residential areas (30-40) high value of industrial and commercial area**

- **Hazen William Equation**

  Q =(0.278 C D2.63 $S^{0.54}$

  C = 120 *From* table

Section (1) :

Qd = 1109.58 l/s Use pipe 1Ø1000

Section (2) :

Qd = 1470 d/sec Use pipe 4Ø 700

At S = 0. 002 → 4 * 455 = 1820 Popullation = 188370.6 c

Section (3) :

Qd = 0.7 * 1470 l/S = 1029 l/S

Use pipes 6 Ø 500 = 1128 l/S Population =131859 C

Section (4) :

Qd = 0.3*1470 = 441 l/S

Use pipes 4 Ø 400 + 1 Ø 300 = 495 l/S Population = 56511.18 C

Section (5) :

Qd = 0.05 *1470 = 73.5 l/S

Use pipes 4 Ø 200 + 1 Ø 300 =117 l/S Popullation = 9418.53 C

sec	population	Precent of total area	Q fire l/sec	Qmax daily + Q fire l/sec	Qmax hourly l/sec	Qdes l/sec	Q pipe S (0.02)	check

1	188370.6	100%	50	1109.58	-	1109.58	1163.5	safe
2	188370.6	100%	50	1109.58	1470	1470	1820	safe
3	131859	70%	50	776.706	1029	1029	1128	safe
4	56511.18	30%	40	332.874	441	441	495	safe
5	9418.53	5%	25	55.47	73.5	73.5	117	safe

sec	Demand discharge	Cut pipes numbers	N0 of pipes	Diameter of pipes	length
2	1470	4	1	700	393
			1	700	218
			1	700	381
			1	700	212
3	1029	5	1	500	466
			1	500	462
			1	500	265
			1	500	158
			1	500	225
4	441	5	1	400	457
			1	400	436
			1	400	359
			1	400	286
			1	300	142
5	73.5	5	1	200	645
			1	200	645
			1	200	284
			1	200	326
			1	300	366

5.12- Wast Water Treatment

Stage(2)

- **In summer :**

Qav=0.8*588=470.4 lit/sec ، p=188370.6capita

$Q = (1+\frac{14}{4+\sqrt{188.370}}) * 470.4 = 948.10\ m^3/s$

Qinf = 0.2*Qav waste = 0.2 * 470 = 94 lit/sec

Qmax = 1.8* 470.4+94 = 940.72 lit/sec = 0.940 m3/sec

- **In winter :**

$M.F = 0.2p^{0.167} = 0.2 * (188.3706)^{0.167} = 0.48$

Qmin = 0.8*0.48*470.4+94 = 274.74 lit/sec

Stage (1) :

- **In summer :**

Qav = 0.8 * 387= 309lit/sec ، P = 133989.133 capita

$P.F = \frac{18+\sqrt{133.989}}{4+\sqrt{133.989.}} = 1.89$

Qinf = 0.2 *309= 61.8 lit/sec

Qmax = 1.89 * 309 + 61.8 = 645.8 lit/sec

- **In winter :**

$M.F = 0.2 * (133.989)^{0.167} = 0.45$

Qmin = 0.8 * 0.45 * 309 + 61.8 = 173.82 lit/sec

Lt = 15468 m $\frac{Qmax}{Lt} = \frac{940}{15468} = 0.07$

∅ Mm	Slope %	Vfill m/s	Q N L/S	$\frac{d_{max}}{\emptyset}$	$\frac{Q_{max}}{Q_{fill}}$	Qmax*(1/sec) For each pipt	L service (m)
200	5	0.71	22			20	285
250	4	0.74	36			32	455
300	3.33	0.67	54			49	700
350	2.8	0.68	75			68	970
400	2.5	0.80	100	0.75	0.90	90	1285
450	2	0.77	122			110	1570
500	1.8	0.79	155			140	2000

600	1.4	0.97	223			200	2855
700	1.3	0.84	323			291	4150
800	1.0	0.81	407			431	6155
900	0.8	0.78	496	0.90	.06	526	7510
1000	0.8	0.84	660			700	10000

8- Conclusion

Recognizing the importance of enhancing sanitation practices is more effective than simply introducing technological advancements. This approach is considered advantageous for the community as it emphasizes collaboration between suppliers and beneficiaries through dialogue and information sharing. Ultimately, individual users have the ultimate authority in deciding whether to adopt or reject new technologies. The success of the project lies in the hands of these users, as the value of the investment relies not only on community support but also on the acceptance of families and individuals. It is crucial to convince individuals about the benefits of improved hygiene and the advantages that come with adopting new technologies, outweighing any associated risks. Providers must also take into account the social context and limitations that influence personal decisions. By understanding the community's perspectives, providers can identify positive attributes that may elicit negative responses and utilize the community's values, beliefs, and practices to drive positive change.

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