Nile Higher Institute

For Engineering and Technology

Civil Engineering Department

(Sanitary Project)

(مشروع صحية)

Water Treatment Plant

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(Sanitary Engineering Project)

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Table OF Contents

Abstract

We worked in this project (sanitary engineering project) on how to design a water network for a city, where we knew how to calculate the expected population, whether in the first or second stage, designing tanks for Rapid mixing tank, sedimentation zone, clarri_flocculation, and the ground tank. We worked on knowing the type of pipes used in city planning and designing a water station. For distribution with engineering standards according to the Egyptian code

We have worked on designing a sewage treatment plant and the used pipes, which in the station or planning the city sewage pipes.

Introduction

It is a very important concept for the livelihood of towns and its communities. The entire discipline of sanitary engineering typically deals with the application of proven engineering methods to ensure an efficient sanitation system for human communities and also improve the accessibility of drinking water for them.

In the older engineering sciences, it was seen as a subset of civil engineering. But now with the growing emphasis on the environment, it comes under the environmental engineering branch.

The different skills which one can gain by learning the content in this field are to ensure clean and potable drinking water for humans, proper waste disposal within the economic boundaries of the community and also the treatment of wastewater. One unique feature of this field of engineering is that it is more of an open system, unlike the mechanical or electrical sciences. That is, the content in this field contains a lot from the other disciplines and some examples include, hydraulics, microbiology, project design, information technology and even environment technology.

Water scarcity is one of the main challenges faced by the sanitation engineers today. There is a constant need for new forms of technology that will enable more efficient use of the water available. Some of the technologies such as sonar mapping is used to determine the amount of the water contained in the well. Some cities which are near the sea also use desalination plants, which clean the saltwater from the sea and make it drinkable. The downside to this technology now is that it is highly expensive, and cities which do not have good revenue are still unable to adopt this process.

The second challenge in sanitation engineering is climate change. For instance, in the treatment of wastewater, a lot of greenhouse gases such as Methane and Carbon Dioxide are present. These gases are produced when the anaerobic bacteria decompose the organic matter. But this is not the direct contributor to climate change. This is because the sewer system is mostly blocked off from the overground. So, when there is a sewer blockage, methane and other greenhouse gases will be trapped inside, usually reprocessed and treated. The emissions from the machines that undertake this process produce far more harmful gases which

contribute to climate change. So, a lot of facilities around the world is being renovated to make them eco – friendly as much as possible.

(Water Treatment Plant)

Population and Water Consumption

The previous census records for the required city are as follows :

P1996 = 95610 capita

P2006 = 116048 capita

P2016 = 142721 capita

Forecasting Population:

A. Arithmetic Method:

Year	Population(p)	ΔP	Δt	Ka=ΔP/Δt
1996	95610	--	--	---
		20438	10	2043.8
2006	116048			
		26673	10	2667.3
2016	142721			
				Sum of Ka=5440

Ka (Average)= sum of ka / N.O.Ka = 4711/2= 2356 capita/year.

$P_n = P_o + ka * \Delta t$

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

= 142721+ 2356 (2040 – 2016) = 199265 capita

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

= 142721 + 2356 (2060 – 2016) = 246385 capita

B. Geometric Method:

Year	Population (P)	Ln(P)	ΔLn(P)	Δt	Kg
1996	95610	11.468	--	--	--
			0.193	10	0.0193
2006	116048	11.661			
			0.207	10	0.0207
2016	142721	11.868			
					Sum of kg=0.04

Ka (Average) = 0.04/2 = 0.02 capita/year.

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

$Ln(P_{2040}) = ln P_{2016} + kg * \Delta t$

= 11.868 + 0.02 (2040 – 2016) = 12.348 * P_{2040} = 230499 capita.

$Ln(P_{2060}) = ln P_{2060} + kg * \Delta t$

= 11.868 + 0.02 (2060 – 2016) = 12.748* P_{2060} = 343864 capita

C. Annual Growth Rate Method:

Year	Population (P)	P_n/P_o	Δt	$(P_n/P_o)^{\frac{1}{\Delta t}}$	m/100
1996	95610	--	--	---	---
		1.214	10	1.0196	0.0196
2006	116048				
		1.23	10	1.021	0.021
2016	142721				
					Sum of m= 0.0406

M (average) = sum of m/N.o.m = 0.0406/2 =0.0203 capita/year

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

- $P_{2040} = P_{2016} * (1 + mav)^{\Delta t}$
  = 142721 * $(1 + 0.0203)^{2040-2016}$ = 231184 capita
- P2060 = P2016 * $(1 + mav)^{\Delta t}$
  = 154800 * $(1 + 0.0225)^{2060-2016}$ = 345554 capita

- ## Final Results:

	Method 1	Method 2	Method 3
P_{2040}	199265	230499	231184

P_{2060}	246385	343864	345554

P_{2040} Av = 220316 capita   *P_{2060} Av = 311935 capita*

- **Design Flow:**

❖ Stage (1): (2040)

q_{2040} = 250 L/c/d

q = 0.25 m/c/d

P (2040) av = 220316 capita

$Q_{2040\ av} = P_{2040\ av} * q_{2040}$

= 220316 * 0.25 = 55079 m^3 / day

$Q_{max\ monthly}$ = 1.4 Qav

= 1.4 * 55079 = 77111 m^3 / day

$Q_{max\ daily}$ = 1.8 Qav

= 1.8 * 55079= 99143 m^3 / day

$Q_{max\ hourly}$ = 2.5 * Qav

= 2.5 * 55079 = 137698 m^3 / day

Q_{design} = 1.1 * 1.4 * Qav

= 1.1 * 1.4 * 55079 = 84822 m^3 / day

❖ Stage (2): (2060)

q_{2060} = 270 L/c/d

q = 0.27 m/c/d

P (2060) = 311935 capita

$Q_{2060\,av} = P_{2060} * q_{2060}$

$= 311935 * 0.27 = 84223\ m^3$ / day

$Q_{max\ monthly} = 1.4 * q_{av}$

$= 1.4 * 84223 = 117913\ m^3$ / day

$Q_{max\ daily} = 1.8 * q_{av}$

$= 1.8 * 84223 = 151602\ m^3$ / day

$Q_{max\ hourly} = 2.5 * q_{av}$

$= 2.5 * 84223 = 210558\ m^3$ / day

$Q_{design} = 1.1 * 1.4 * q_{av}$

$= 1.1 * 1.4 * 84223 = 129704\ m^3$ / day

Flow	Stage (1)		Stage (2)	
……..	$\frac{m^3}{day}$	m^3/Sec	$\frac{m^3}{day}$	m^3/Sec
Qav	55074	0.64	84223	0.97
$Q_{max\ monthly}$	77111	0.90	117913	1.36
$Q_{max\ daily}$	99143	1.15	151602	1.75
$Q_{max\ hourly}$	137698	1.6	210558	2.44
Q_{design}	84822	0.98	129704	1.5

(Water Treatment Plant Units)

❖Design Discharge Qd:

- **Stage 1:**

$Q_d = \frac{84822}{24*60*60} = 0.98\ m^3$ / sec

- **Stage 2:**

  $Q_d = \frac{129704}{24*60*60}$ = 1.5 m^3 / sec

❖Design Of Conduit Pipes:

Q_d = 1.5 m^3 / sec

Assume V = 1.0 m^3 / sec

Assume N = 4

A = N $\frac{\pi\,\varphi^2}{4}$

1.5 = 4 * $\frac{\pi\varphi^2}{4}$ φ = 691mm = 700 mm

A_T= 4*$\frac{\pi*(0.7)^2}{4}$ = $1.54m^2$

$V_{act\ stage\ (2)}$ = $\frac{Qd}{At}$ = $\frac{1.5}{1.54}$ = 0.97 m / sec → (0.8 → 1.5) Ok.

Check Of V_{act} → N = 3

A = $\frac{N\,\pi\,\varphi^2}{4}$ = 3*$\frac{\pi*(0.7)^2}{4}$ = 1.15 m^2

$V_{act\ stage\ (1)}$ = $\frac{0.98}{1.15}$ = 0.85 m/sec → (0.8 → 1.5) Ok.

☒ Stage 1 = 3 Ø 700

☒ Stage 2 = 4 Ø 700

❖ Losses:

L = 100 m

★ Stage 1:

h.L = $\frac{F.L.V}{2g\emptyset}$ = $\frac{0.04*100*(0.85)^2}{2*9.81*(0.7)}$ = 0.24 m

★ Stage 2:

h.L = $\frac{F.L.V}{2g\emptyset}$= $\frac{0.04*100*(0.97)^2}{2*9.81*(0.7)}$ = 0.29 m

Assume = B = 1.5 ∅= 1.5 * 70 = 105 m

a = 1.5 cm

∅ = 60

S = 3.0 cm

B = N * S + (N – 1) a → 1.05 = 0.03 N + (N – 1) * 0.015 →

N = 23

No. Of Opening = 23

No. Of Bars = N – 1 = 22

❖ Screen:

h.LScreen = $\frac{1.4\,[(v_2{}^2 - v_1{}^2)]}{2g}$

v1 = $\frac{\varphi_t/N}{B*d}$ = $\frac{1.5/4}{1.05*2.5}$=0.142

d = w.L – b.L = 6.68 – 4.5 = 2.18 m

d1 = d – (0.5 – 1) = 2.18 – 0.8 = 1.38 m

v1 = $\frac{\emptyset}{A_{net}}$ = $\frac{1.8\,/3}{N*s*d/sin\emptyset}$ = $\frac{0.98/3}{23*0.03*2/sin(60)}$ = 0.205 m/sec

v2 = $\frac{1.5/4}{23*0.03*2/sin(60)}$ = 0.235 m/sec

h.L = 1.4 * $\frac{(0.205)^2-(0.124)^2}{2*9.81}$= 1.902*$10^{-3}$ m → stage 1

h.L = 1.4 * $\frac{(0.235)^2-(0.142)^2}{2*9.81}$ = 2.502*10^{-3} m → stage 2

Design of sump:

- **Stage 2:**

  Assume T = 5 min

  V = Q * T = 5*60*1.5 = 450 m^3

  D_{sump} = [w.L –b.L_{source} +1+1] =

  = 6.68-4.5+1+1 = 4.2m

  $Area_{sump}$= $\frac{\forall}{d}$ = $\frac{450}{4.2}$ = 107.14 m^2

  Assume width = 3 m

  ∀ = L * w * d

  450 = L * 3 * 4.5 → L = 34 m

  Sump = [3 * 4.5 * 34]

- **Stage 1:**

  ∀ = Q * T = 5 * 60 *0.98 = 294 m^3

  width = 3

294 = L * 3 * 4.5→ L = 22

Sump [3 * 4.5* 22]

❖Design of low left pump:

- suction pipe & rising main:

Q_d = 1.5 $\frac{m^3}{sec}$ N = 9 → 1.5n → n = 6

N = 9 →[6working + 3stand by]

$Q_p = \frac{Q_d}{n} = \frac{1.5}{6} = 0.250\ \frac{m^3}{sec} \rightarrow$ 250 Lit/sec

Assume ∀ = 1.5 m/sec

$A = \frac{1.5}{1.5} = 1\ m^2 \rightarrow 1 = \frac{N\,\pi\,\emptyset^2}{4} \rightarrow N = 1$

$\emptyset = 1236$ mm $\rightarrow V_{act} = \frac{1.236}{1.2} = 1.03$ m/sec

$H_{static} = G.L - w.L_{sump} + 6$

$= 24 - 21 + 6 = 9$ m

- ## Stage 1:

  N = 6 →[4 working + 2 stand by]

  → Neglect the friction losses in suction pipe

  $Q_2 = 0.98\ \frac{m^3}{sec} \rightarrow v = 1.5$ m/sec

  $A = \frac{0.98}{1.5} = 0.65\ m^2$

❖Design of Suction Pipe:

A. Stage 2:

$Q_d = 1.5\ \frac{m^3}{sec}$

$Q_d = A * V$, assume V = 1 m/sec

$A = n * \frac{\pi}{4}\emptyset^2$, assume n = 4

$Q_d = A * V$

$1.5 = 4 * \frac{\pi}{4}\emptyset^2 * 1,\ \emptyset = 0.7$

$A_{act} = n * \frac{\pi}{4}\emptyset^2 = 4 * \frac{\pi}{4}(0.7)^2 = 1.53\ m^2$

$V_{act} = \frac{Q_d}{A_{act}} = \frac{1.5}{1.53} = 0.98$ m/sec

B. Stage 1: (Check)

$Q_d = 0.98\ \frac{m^3}{sec}$

$Q_d = A * V$, assume V = 1 m/sec

$A = n * \frac{\pi}{4}\emptyset^2$, assume n = 3

$Q_d = A * V$

$0.98 = 4 * \frac{\pi}{4}\emptyset^2 * 1, \emptyset = 0.7$

$A_{act} = n * \frac{\pi}{4}\emptyset^2 = 3 * \frac{\pi}{4}(0.7)^2 = 1.5\ m^2$

$V_{act} = \frac{Q_d}{A_{act}} = \frac{0.98}{1.5} = 0.85$ m/sec , Ok .

❖Design of Header:

- **Stage 2:**

$Q_d = A * V$, assume V = 1m/sec

$A = \frac{\pi}{4}\emptyset^2$

$Q_d = A * V$

$1.5 = \frac{\pi}{4}\emptyset^2 * 1 \rightarrow \emptyset = 1.3$

$A_{act} = \frac{\pi}{4}\emptyset^2,\ \frac{\pi}{4}(1.3)^2 = 1.32\ m^2$

$V_{act} = \frac{Q_d}{A_{act}} = \frac{1.5}{1.32} = 1.13$ m/sec

- **Stage 1: (Check)**

$Q_d = A * V$, assume V = 1 m/sec

$A = \frac{\pi}{4}\emptyset^2$

$Q_d = A * V$

$0.98 = \frac{\pi}{4}\emptyset^2 * 1,\ \emptyset = 1$

$A_{act} = \frac{\pi}{4}\emptyset^2 = \frac{\pi}{4}(1)^2 = 0.78\ m^2$

$V_{act} = \frac{Q_d}{A_{act}} = \frac{0.98}{0.78} = 1.25$ m/sec, Ok .

❖Design of Force Main:

- ## Stage 2:

$Q_d = A * V$, assume V = 1.5 m/sec

$A = \frac{\pi}{4}\emptyset^2$

$Q_d = A * V$

$1 = \frac{\pi}{4}\emptyset^2 *, \emptyset = 1.1$

$A_{act} = \frac{\pi}{4}\emptyset^2 = \frac{\pi}{4}(1.1)^2 = 0.95 m^2$

$V_{act} = \frac{Q_d}{A_{act}} = \frac{1.5}{0.95} = 1.58$ m/sec

☒ **Losses:**

$h.L = \frac{f\,L\,V^2}{2\,g\,\emptyset}$, assume L = 50 m

$h.L = \frac{0.04*50*(1.58)^2}{2*9.81*1.1} = 0.23$ m

- ## Stage 1: (Check)

$Q_d = A * V$, assume V = 1.5 m/sec

$A = \frac{\pi}{4}\emptyset^2$

$Q_d = A * V$

Assume, $\emptyset = 0.8$

$A_{act} = \frac{\pi}{4}\emptyset^2 = \frac{\pi}{4}(0.8)^2 = 0.5\, m^2$

$V_{act} = \frac{Q_d}{A_{act}} = \frac{0.98}{0.5} = 1.96$ m/sec

☒ **Losses:**

$h.L = \frac{f\,L\,V^2}{2\,g\,\emptyset}$, assume L = 50 m

$h.L = \frac{0.04*50*(1.96)^2}{2*9.81*0.8} = 0.49$ m

❖Design of Horse Power:

- ## Stage 2:

Assume, Q_p = 1500 L/sec = 1.5 m^3 / sec

$Q_{pump} = \frac{Q_d}{n} =$

n = 10

$Q_{pump} = \frac{1500}{6}$ =250 Liter/Sec

$Hp = \frac{\gamma\, Q_p\, H_t}{75\, \lambda_1 \lambda_2}$

$= \frac{1*250*7}{75*0.63}$ =37 HP

H_t = hstatic + hf + hminor

= (R.M.T – W.L) + hf + hminor

= (12 – 6.68) + 0.14+0.2 (0.14) = 5.5

9 (22)Hp

- **Stage 1:**

Assume 7∅ 22Hp

$Qp = \frac{Q_d}{n}$

$= \frac{980}{7}$ =140 Liter/Sec

(Coagulation)

- Alum solution tanks:

S = (20 → 40) mg/L

Q_d * s * (365 * 10^{-6})

= 129704 * 40 * (365 * 10^{-6}) = 1894.2 t/year

$\forall = \frac{Q_d * s}{c * \gamma * 10^6} = \frac{129704*40}{1.05*0.1*10^6}$ = 49.41 m^3 / sec

Assume No. of tanks = 3 tanks

∀ for one tank = $\frac{\forall}{3} = \frac{49.41}{3}$ = 16.47 m^3 / day

A = $\frac{\forall}{d}$ = $\frac{16.47}{1.5}$ = 10.98 → A = L^2

L = $\sqrt{10.98}$ = 3.31 ≈ 3.3 m

3 tanks with dimension [3.3 * 3.3 * 1.5] m

- **Outlet Pipe:**
- Assume V = 0.8 m/sec

Q = A * V = $\frac{49.41}{24*60*60}$ = A * 0.8

A = 1.875 * $10^{-4} m^2$

A = $\frac{\pi}{4}\emptyset^2$ → 1.875 * 10^{-4} = $\frac{\pi}{4}\emptyset^2$, ∅ = 1.5

A_{act} = $\frac{\pi}{4} 1.5^2$ = 1.767 m^2

(Rapid Mixing Tank)

T = 60 sec d = 3 m

★ **Stage 2:** Q_d = 1.5 m^3 / sec

∀ = Q_d * T = 1.5 * 60= 90 m^3

A = $\frac{\forall}{d}$ = $\frac{90}{3}$ = 30 m^2

Assume n = 3.76 n=4

A = 2 * $\frac{\pi}{4}\emptyset^2$, ∅ = 4.4 m

$\forall_{act}$ = d * A = 2 * (2 * $\frac{\pi}{4}$ * 4.4^2) =91.3 m^3

T_{act} = $\frac{\forall_{act}}{Q_d}$ = $\frac{91.3}{1.5}$ = 61 sec, Ok

★ **Stage 1:** Q_d =0 .98 $\frac{m^3}{sec}$

A_{act} = $\frac{\pi}{4}(4.4)^2$ = 15.2 m^2

$\forall_{act}$ = d * A = 3 * 15.2 = 45.6 m^3

$T_{act} = \frac{\forall_{act}}{Q_d} = \frac{45.6}{0.98}$ = 46 sec, Ok

(Power Tank)

- P = $G^2 * \mu * \forall$ Assume G = 700

  = 800^2 * (1.002 * 10^{-3}) * 45.6

  = 36 KW

  Power required for stage 1, 2 = 36 KW

(Clari-Flocculation Tank)

❖Stage 2:

-Sedimentation Zone:

Q_d = 1.5 m^3 / sec

t → 3.5 hr

ds → 4 m

$\forall = Q_d * t_{total}$ =1.5*3.5*60*60 = 18900m^3

A = $\frac{\forall}{d} = \frac{18900}{4}$ = 4725 m^2

A = n * $\frac{\pi}{4}\emptyset^2$,

4725 = n * $\frac{\pi}{4}$ * 40^2 , n = 3.76 ≈ 4

A = 4 * $\frac{\pi}{4}{\emptyset_2}^2$, $\emptyset_2$ = 38.7 = 39 m

- Flocculation Zone:

$T_f \rightarrow 0.5$

$D_f \rightarrow 3.5$

$\forall = Q_d * t_f = 1.5 * 0.5*60*60 = 2700\ m^3$

$A = \frac{\forall}{d_f} = \frac{2700}{3.5} = 771.4\ m^2$

$A = n * \frac{\pi}{4}\emptyset^2$,

$771.4 = 4 * \frac{\pi}{4}{\emptyset_1}^2, \quad \emptyset_1 = 15.6$ m

❖ Stage 1:

-Sedimentation Zone:

D = 4 m

$Q_d = 4174.5\ m^3$ / hr

$A = n * \frac{\pi}{4}{\emptyset_T}^2$

$= 3 * \frac{\pi}{4} * (40\ )^2 = 3769.9m^2$

$\forall = A * d$

$= 3769.9 * 4 = 15079.64\ m^3$

$T_f = \frac{\forall}{Q_d} = \frac{15079.64}{0.98*60*60} = 4.2$ hr

-Flocculation Zone:

D = 3.5 m

$Q_d = 4174.5\ m^3$ / hr

A = n * $\frac{\pi}{4} {\emptyset_f}^2$

= 3 * $\frac{\pi}{4}$ * $(15.6)^2$ = 573.4 m^2

∀ = A * d

= 573.4 * 4 = 2293.6 m^3

$T_f = \frac{\forall}{Q_d} = \frac{2293.6}{0.98*60*60}$ = 0.65 hr

- S.L.R:

○ Stage 2:

Q_d= 129704 m^3 / day

= $\frac{Q/_n}{\frac{\pi}{4}({\emptyset_2}^2 - {\emptyset_1}^2)} = \frac{129704/_4}{\frac{\pi}{4}(38.8^2 - 15.6^2)}$ = 32.7, Ok

- S.L.R:

○ Stage 1:

Q_d = 84822 m^3 / day

= $\frac{84822/_3}{\frac{\pi}{4}(38.8^2 - 15.6^2)}$ = 28.52, Ok

⋆Volume of Sludge:

∀ = $\frac{Q/_n * S.s * R}{(1-wc)\,\gamma * 10^6}$

$$= \frac{{}^{129704}/_{4} *50*0.95}{(1-0.97)*1.05*10^6} \approx 48.8 \ m^3$$

- **Sludge must be exited three times daily.**

∀ of sludge zone = $\frac{\forall}{3} = \frac{48.8}{3} = 16.2 \ m^3$

Desludging duration = 20 min

= 1200 sec

Q = $\frac{\forall}{t} = \frac{16.29}{20*60} = 0.0135 \ m^3$ **/ sec**

Q = A * V →

0.0135 = A * 1.5, A = 9 * 10^{-3}

A = $\frac{\pi}{4}\emptyset^2$

$9 * 10^{-3} = \frac{\pi}{4}\emptyset^2$ **,** $\emptyset$ **= 0.12 m**

= 120 mm

★Design of inlet pipes from (R.m.T) :

☒ Stage 2:

Q for one tank = $\frac{Q}{n} = \frac{1.5}{2} = 0.75 \ m^3$ **/ sec**

Q = A * V →

0.75 = A * 1, A = 0.75 m^2

A = $\frac{\pi}{4}\emptyset^2$ **→**

0.75 = $\frac{\pi}{4}\emptyset^2$ **,** $\emptyset$ **= 0.97 m**

★Design of outlet pipes to filter:

☒ Stage 2:

Q = $\frac{1.5}{4}$ = 0.375 m^3 / sec

Q = A * V →

0.375 = A * 0.6, A = 0.625 m^2

A = $\frac{\pi}{4}\emptyset^2$ →

0.625 = $\frac{\pi}{4}\emptyset^2$, $\emptyset$ = 0.89m

-Losses $= \frac{F*L*V^2}{2*g*\emptyset} = \frac{0.4*200*0.6^2}{2*9.81*0.89}$=1.65

(Filteration Tank)

☒ Stage 2:

Q$_d$ = 129700 m^3 / d

R.o.f = 140 m^3 / m^2 / d

A = $\frac{Q_d}{R.o.f} = \frac{129700}{140}$ = 926.42 m^2

A one filter = 8 * 8 = 64 m^2

No. of filter = $\frac{926.4}{64}$ = 14.4 ≈16 filter

[16 filter + 3 filter for backwash]

☒Stage 1:

Q_d = 84822 m^3 / d

A = n * 64 = 10 * 64 = 640m^2

R.o.f = $\frac{Q_d}{A}$ = $\frac{84822}{640}$= 132.5 m^3/m^2/d

(Safe)

[10 filter + 2 filter for backwash]

☒Amount of wash water:

R.o.R * A * time =

(6 * 140) * 64 * ($\frac{15}{24*60}$) = 560 m^3

☒ Amount of Air:

R.O.A * A one filter * time =

1 * 64 * 5 = 320 m^3

☒ Head losses:

Re = $\frac{p_w * v_s * d_p}{\mu}\emptyset$

= $\frac{1000 * \left(\frac{140}{24*60*60}\right) * 0.6 * 10^{-3}}{1.002*10^{-3}}$ * 1 = 0.97

F' = $\frac{150\,(1-e)}{Re}$ + 1. 75

$$= \frac{150\ (1-0.4)}{0.97} + 1.75 = 94.5$$

$$Hf = \frac{F' \gamma\ L\ (1-e) * {v_s}^2}{e^3 * d_p * g}$$

$$= \frac{94.5 * 0.7 * (1-0.4) * (0.0016)^2}{0.4^3 * 0.6 * 10^{-3} * 9.81} = 2m$$

☒Wash water Gutter:

$$Q_{Gutter} = \frac{R.O.B * A\ of\ filter}{no.of\ Gutter} = \frac{5 * 140 * 64}{3} = 14933.33\ m^3/d$$

$$\frac{14933.3 * 1000}{24 * 60} = 10370.4\ L/min$$

$$Q = 0.76 * y * h^{\frac{3}{2}}$$

$$10370.4 = 0.76 * 50 * h^{\frac{3}{2}} ,$$

Take h = 42 cm

(Ground Tank)

$Q_{max\ monthly} = 117913\ m^3$ / day = 81.88 m^3 / min

P = 311935 capita

☒Stage 2:

$C1 = Q_{max\ monthly} * T\ (20 \rightarrow 40)$

$$= 117913 * \left(\frac{30}{24 * 60}\right) = 2456.5\ m^3$$

$C2 = 0.4\ Q_{av} * T\ (day)$

$$= 0.4 * 84223 * 1 = 33689.2\ m^3$$

C3 = $Q_{max\ monthly}$ * T (6 → 10 hr)

= $\frac{8}{24}$ * 117913 = 39304.3 m^3

C_{fire} = $\frac{120*p}{10000}$ = $\frac{120*311935}{10000}$ = 3743.22 m^3

Capacity = C_{max} + $\frac{4}{5}$ C_{fire}

= 39304.3 + $\frac{4}{5}$ * 3743.22 = 42298.87

Capacity = N * L * w * d

42298.87 = N * 50 * 50 *5=3.38

N = 4

42298.87 = 5 * 50 * w * 5, w_{act} = 33.83 m

$$\{4\ tanks\ with\ \text{Dimensions}\ (50 * 33.83 * 5)\}$$

☒Stage 1:

$Q_{max\ monthly}$ = 77111 m^3 / day

Q_{av} = 55074

P = 220316 capita

C1 = 53.55 * 30 = 1606.5

C2 = 0.4 * 55074 * 1 = 22029.6

C3 = $\frac{77111}{24}$ * 8 = 25703.67

C_{fire} = $\frac{120 * 220316}{10000}$ = 2643.8

Capacity = 22029.6+ $\frac{4}{5}$ * 2643.8= 24144.64

24144.64= N * 50 * 50 * 5, N = 2.6

Take N = 3 m

$$24144.64 = 3 * 50 * w * 5, \quad w_{act} = 32.19 \text{ m}$$

$$\{3 \; \textit{tanks with} \text{ Dimensions } (50 * 33.8 * 5)\}$$

Final results		
Stage	I	II
Conduit pipe	$3\phi700$	$4\phi700$
Sump	(3*4.5*22)	(3*4.5*34)
Screen losses	S=3 cm, B=105 cm n.of opening =23 no.of pars =22	S=3 cm, B=105 cm n.of opening =23 no.of pars =22
Low Lift Pump	4*1236+2*1236 working + stand by	6*1236+3*1236 Working + stand by
Force Main	ϕ=0.9m L=50m h.l=0.23m	ϕ=1.236 m L=50m h.l=1.8m
Rapid mixing Tank	N=1 ϕ=4.4m T=46 sec	N=2 ϕ=4.4m T=61 sec
Power	36 kw	36kw
Clari-Flocculation	3 tanks ϕ_T=39m ϕ_f=15.6m	4tanks Df= 39 d_s= 15.6
Filteration	10 filter+ 2 back wash	16 filter + 3 back wash

R.O. F	132.5 $m^3/m^2/day$	140 $m^3/m^2/day$
Ground Tank	3 Tanks (50*33.8*5)	4 Tanks (50*33.8*5)

▪ Sludge Tank :

☒ Stage 1 & 2 :

$Q_{sludge} = Q_{sludge\ R.M.T} + 2\ Q_{Backwash}$

$= \frac{782}{24*60*60} + (2 * \frac{560}{24*60*60}) = 0.02\ m^3$ / sec

$\forall = Q * T = 0.02 * (15 * 60) = 18\ m^3$

$A = \frac{\forall}{d} = \frac{18}{4} = 4.5\ m^2$

$A = L^2 \rightarrow 4.5 = L^2$, L = 2.12 m

(2.12*2.12*4)

Inlet &Outlet Pipe

Qd =1.5 m^3/Sec & v = (0.6 :1.5)/Sec,

Take 1 m/Sec

$A=\frac{Qd}{v} = 1.5 = \frac{\pi}{4}\emptyset^2$

$\emptyset = 1.38\ m$

(Elevated Tank)

☒ Stage 2:

P = 391827 capita, qn = 100 L/c/d

Time	consumption	Accumulative
12 N-2AM	**1**	**1**
2 – 4	**1.7**	**2.7**
4 – 6	**3.5**	**6.2**
6 – 8	**6.6**	**12.8**
8 – 10	**15.1**	**27.9**
10 – 12 N	**16.2**	**44.1**
12N-12PM	**17**	**61.1**
2 – 4	**13.2**	**74.3**
4 – 6	**12.2**	**86.5**

6 – 8	**7.4**	**93.9**
8 – 10	**3.6**	**97.5**
10 – 12 N	**2.5**	**100**

Capacity = (A + B) * P + $\frac{1}{5}$ C_{fire}, where

$$C_{fire} = \frac{120 * P}{10000} = \frac{120 * 311935}{10000} = 3743.2$$

$$= (15 + 17) * \frac{311935}{1000} + \frac{1}{5} * 3743.2 = 10730.56\ m^3$$

$$N = \frac{Capacity}{Cone\ Tank \rightarrow (1000 \rightarrow 2000)}$$

$$= \frac{10730.56}{1788} = 6$$

$$Cone\ Tank_{act} = \frac{Capacity}{V} = \frac{10730.56}{6} = 1788\ m^3$$

$$Cone\ Tank = \frac{\pi}{4} \emptyset^2\ d, \quad d = \frac{3}{4} \emptyset$$

$$1588 = \frac{\pi}{4} * \frac{3}{4} \emptyset^3, \quad \emptyset = 14.4\ m, \quad d = 10\ m$$

☒ Stage 1:

P = 260231 capita

Capacity = (A + B) * P + $\frac{1}{5}$ C_{fire} ,

$$C_{fire} = \frac{120 * p}{10000} = \frac{120 * 220316}{10000} = 2643.8$$

$$= (15 + 17) * \frac{220316}{1000} + \frac{1}{5} * 2643.8 = 7578.9\ m^3$$

$N = \frac{Capacity}{cone\ Tank} = \frac{7578.9}{2000} = 3.8 \approx 4$

Cone $Tank_{act} = \frac{Capacity}{N} = \frac{7578.9}{4} = 1894.7\ m^3$

Cone Tank = $\frac{\pi}{4} \emptyset^2$ d, d = $\frac{3}{4} \emptyset$

$1894.7 = \frac{\pi}{4} * \frac{3}{4} \emptyset^3$, $\emptyset$ = 14.4 m, d = 10 m

Chapter 2

(Waste Water Treatment Plant)

(Waste Water)

- Design Flow:

❖Stage 1:

$Q_{av} = 55074\ m^3$ / day , p = 220316

☒ **In Summer:**

$P.F = \frac{5}{p^{0.167}} = \frac{5}{220^{0.167}} = 2$

$Q_{av\ waste} = 0.8 * 55074 = 44059.2\ m^3$ / day

$Q_{max} = P.F * Q_{av\ waste} + Q_{in\ filteration}$

$= 2 * 44059.2 + (0.1 * 44059.2) = 92524.32$

☒ **In Winter:**

M.F = 0.2 $P^{0.167}$ = 0.2 * $220^{0.167}$= 0.49

Q_{min} = 0.7 * M.F * $Q_{av\ waste}$ + $Q_{in\ filteration}$

= 0.7 * 0.49 * 44059.2 + (0.1 * 44059.2)

=19518.23

❖ Stage 2 :

Q_{av} = 84223 m^3 / day, P = 311935 capita

- ☒ In Summer:
- ☒ P.F = $\frac{5}{P^{0.167}}$ = $\frac{5}{311^{0.167}}$ =1.9

$Q_{av\ waste}$ = 0.8* 84223 = 67378.4 m^3 / day

Q_{max} = 1.9 * 67378.4 + (0.2 * 67378.4)

=141494.6

- ☒ In winter:

M.F = 0.2 * $(311)^{0.167}$ = 0.52

Q_{min} = 0.8 *0.52 *67378.4+(0.2 * 67378.4)

= 41505

Stage 1		
Units	m^3/ Day	m^3/ Sec
Q_{max}	92524.32	1.07

Q_{av}	44059.2	0.5
Q_{min}	19518.23	0.226
Stage 2		
Units	m^3/ Day	m^3/ Sec
Q_{max}	141494.6	1.6
Q_{av}	67378.4	0.78
Q_{min}	41505	0.48

Factor = $\frac{Q_{max}}{L_{total}}$

L_{total} = 105230.5 m

Factor = $\frac{7525}{1052300.5}$ = 0.07

L_{served}= $\frac{Q_{design}}{factor}$

$\emptyset mm$	Slope %	V full (L/sec)	d max ∅	Q max Q full	Q max L/s For each pipe	L served
200	5	0.71	0.75	1.3	20	285.7
250	4	0.74			32	457.1
300	3.33	0.76			49	700
350	2.8	0.78			68	971.4
400	2.5	0.80			90	1285.7
450	2	0.77			110	1571.4
500	1.8	0.79			140	2000

600	**1.4**	**0.79**			**200**	**2857.1**
700	**1.3**	**0.84**			**291**	**4157.1**
800	**1.0**	**0.81**			**431**	**6157.1**
900	**0.8**	**0.78**	**0.90**	**1.95**	**526**	**7514**
1000	**0.8**	**0.84**			**700**	**10000**

❖Pump Station: -

$$v = \frac{Q * \theta}{4}$$

$$v = \frac{1.55 * 20 * 60}{4} = 465 \text{ m}^3$$

D=2m

A TOTAL = 465/2=232.5 m^2

$A_{net}=0.4*A_{total}$

$A_{net}= 0.4*232.5 = 93 \ m^2$

$*Ø^2 A = \frac{\pi}{4}$

$*Ø^2 \ 93 = \frac{\pi}{4}$

Ø= 11 m

Conclusion

It is a very important concept for the livelihood of towns and its communities. The entire discipline of sanitary engineering typically deals with the application of proven engineering methods to ensure an efficient sanitation system for human .communities and also improve the accessibility of drinking water for them

In the older engineering sciences, it was seen as a subset of civil engineering. But now with the growing emphasis on the environment, it comes under the .environmental engineering branch

The different skills which one can gain by learning the content in this field are to ensure clean and potable drinking water for humans, proper waste disposal within the economic boundaries of the community and also the treatment of wastewater. One unique feature of this field of engineering is that it is more of an open system, unlike the mechanical or electrical sciences. That is, the content in this field contains a lot from the other disciplines and some examples include, hydraulics, microbiology, project design, information technology and even environment technology

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Standard

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