การออกแบบระบบผลิตภัณฑ์
สำหรับน้ำประปาขนาด 5,000 ลูกบาศก์เมตรต่อวัน

โดย นายพรศักดิ์ สมทรัพย์วิจิตร
### 2. Design Criteria

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Design Criteria</th>
<th>Design Flow (5000 m³/day)</th>
<th>Max. Flow (7500 m³/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. Rapid Mixing</strong></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>- Time</td>
<td>sec</td>
<td>1 to 3</td>
<td>2.17</td>
<td>1.45</td>
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<tr>
<td>- G - Value</td>
<td>sec⁻¹</td>
<td>500 to 700</td>
<td>612</td>
<td>1100</td>
</tr>
<tr>
<td>- GT</td>
<td></td>
<td></td>
<td></td>
<td>1328</td>
</tr>
<tr>
<td><strong>2. Flocculation Basin</strong></td>
<td></td>
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</tr>
<tr>
<td>- Type</td>
<td></td>
<td>Baffle Channel</td>
<td></td>
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<tr>
<td>- No. of Stage</td>
<td>No.</td>
<td>2 - 7</td>
<td>4</td>
<td>4</td>
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<tr>
<td>- Energy Input</td>
<td>sec⁻¹</td>
<td>20 - 60</td>
<td>stage 1 = 60</td>
<td>110</td>
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<tr>
<td></td>
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<td></td>
<td>stage 2 = 35</td>
<td>64</td>
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<td></td>
<td>stage 3 = 20</td>
<td>37</td>
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<tr>
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<td></td>
<td>stage 4 = 15</td>
<td>27</td>
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<tr>
<td>- Detention Time</td>
<td>minute</td>
<td>20 - 40</td>
<td>31</td>
<td>21</td>
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<tr>
<td><strong>3. Sedimentation Basin</strong></td>
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<tr>
<td>- Type</td>
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<td>Rectangular</td>
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<td>Rectangular</td>
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<tr>
<td>- Detention Time</td>
<td>hour</td>
<td>1.5 - 4</td>
<td>2.5</td>
<td>1.67</td>
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<tr>
<td>- Surface Loading</td>
<td>m/min</td>
<td>0.02 - 0.06</td>
<td>0.019</td>
<td>0.029</td>
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<tr>
<td>- Water Depth</td>
<td>m.</td>
<td>3.0 - 4.5</td>
<td>3</td>
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<tr>
<td>- Mean Velocity</td>
<td>m/min</td>
<td>≤ 1.5</td>
<td>0.129</td>
<td>0.193</td>
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<tr>
<td>- Inlet Diffuser Wall</td>
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<tr>
<td>Port Velocity</td>
<td>m/sec</td>
<td>0.15 - 0.60</td>
<td>0.165</td>
<td>0.247</td>
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<td>Port Spacing</td>
<td>m.</td>
<td>0.40 - 0.70</td>
<td>0.5</td>
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<td>Port Diameter</td>
<td>mm.</td>
<td>100 max</td>
<td>75</td>
<td>75</td>
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<tr>
<td>Description</td>
<td>Unit</td>
<td>Design Criteria</td>
<td>Design Flow (5000 m³/day)</td>
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<td>-----------------------------------</td>
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<tr>
<td>Effluent Weir Loading</td>
<td>m³/m./hr.</td>
<td>12 max</td>
<td>8.68</td>
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<td><strong>4. Filtration Basin</strong></td>
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<tr>
<td>Number of Filter Basin</td>
<td>No.</td>
<td>min. 2</td>
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<tr>
<td>Filtration rate</td>
<td>m/hr</td>
<td>5 to 7</td>
<td>5.5</td>
<td>8.33</td>
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<td>Filter Flow Control System</td>
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<tr>
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<td>Filter Media</td>
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<tr>
<td>Type of Media</td>
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<td>Sand</td>
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<tr>
<td>Effective Size</td>
<td>mm.</td>
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<td>0.55 - 0.65</td>
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<td>Uniformity Coefficient</td>
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<td>1.40 - 1.70</td>
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<tr>
<td>Depth</td>
<td>mm.</td>
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<td>700</td>
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<tr>
<td>Back Wash Rate</td>
<td>m./min.</td>
<td>0.60 - 0.70</td>
<td>0.7</td>
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<tr>
<td>Surface Wash System</td>
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<tr>
<td>Type</td>
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<td>Fixed Nozzle</td>
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<tr>
<td>Rate</td>
<td>m./min.</td>
<td>0.12 - 0.16</td>
<td>0.15</td>
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<tr>
<td>Surface Jet Pressure</td>
<td>m.</td>
<td>15 - 20</td>
<td>15</td>
<td></td>
</tr>
</tbody>
</table>
1. ข้อมูลพื้นฐาน

1.1 คุณภาพน้ำ
- Turbidity = NTU
- Alkalinity = mg/l as CaCO₃
- pH =
- Temperature = °C
- Fe = mg/l
- Mn = mg/l
- Hardness = mg/l as CaCO₃

1.2 แหล่งน้ำ = Surface Water

1.3 Jar Test
- Alum Dose = mg/l
- Alkalinity = mg/l as CaCO₃
- pH =
- Temperature = °C

1.4 Design Flow
Design Plant Capacity = 5000 m³/d
Design Operation Flow = 24 hr

1.5 Type of Water Treatment Plant
- Hydraulic Design System
3. Coagulation Basin Design

3.1 Raw Water Pipe

Design Flow (Q Design) = 5,000 m$^3$/day

Velocity in Pipe = 1.8 - 2.0 m/s (Kawamura)

Use = 1.8 m/s

∴ Pipe Diameter (D) = $\sqrt{\frac{4Q}{\pi v}}$

= $\sqrt{\frac{4 \times 5,000}{3.14 \times 1.8 \times 3600 \times 24}}$

= 0.2023751 m.

Use Pipe Diameter (D) = 0.2 m.

∴ Acture Velocity = $\frac{Q}{A}$

= 1.8420711 m/s

3.2 Rapid Mixing

= Hydraulic Type

Type = Static Mixer

Design Criteria

- Detention Time = 1 - 3 sec
- G - Value = 500 - 700 sec$^{-1}$
- GT = 350 - 1500

3.3 Calculation

Theory: Rule of Thumb - estimating the Length of one element is designated the length as 1.5 - 2.5 times the pipe diameter the base on this criteria, the length of one element is in the range of 3 - 5 ft (Kawamura, page no. 88)

- 2 stage = 2 element

Static Mixer Length = 1.5xDiameter(m) x2(element) - 0.5xDiameter(m)
- 3 stage = 3 element
  \[ \text{Static Mixer Length} = 1.5 \times \text{Diameter}(m.) \times 3(\text{element}) - \text{Diameter}(m.) \]

- 4 stage = 4 element
  \[ \text{Static Mixer Length} = 1.5 \times \text{Diameter}(m.) \times 4(\text{element}) - 1.5 \times \text{Diameter}(m.) \]

- 5 stage = 5 element
  \[ \text{Static Mixer Length} = 1.5 \times \text{Diameter}(m.) \times 5(\text{element}) - 2.0 \times \text{Diameter}(m.) \]

- 6 stage = 6 element
  \[ \text{Static Mixer Length} = 1.5 \times \text{Diameter}(m.) \times 6(\text{element}) - 2.5 \times \text{Diameter}(m.) \]

- 7 stage = 7 element
  \[ \text{Static Mixer Length} = 1.5 \times \text{Diameter}(m.) \times 7(\text{element}) - 3.0 \times \text{Diameter}(m.) \]

- 8 stage = 8 element
  \[ \text{Static Mixer Length} = 1.5 \times \text{Diameter}(m.) \times 8(\text{element}) - 3.5 \times \text{Diameter}(m.) \]

- 9 stage = 9 element
  \[ \text{Static Mixer Length} = 1.5 \times \text{Diameter}(m.) \times 9(\text{element}) - 4.0 \times \text{Diameter}(m.) \]
Try Static Mixer Diameter = 200 mm.
Use 3 stage = 3 element
∴ Static Mixer Length (L) = 0.7 m.
∴ Acture Velocity = \( \frac{Q}{A} \)
= 1.842 m/s
∴ Detention Time (t) = 0.380007 sec. too less

Try Static Mixer Diameter = 250 mm.
Use 3 stage = 3 element
∴ Static Mixer Length (L) = 0.875 m.
∴ Acture Velocity = \( \frac{Q}{A} \)
= 1.179 m/s
∴ Detention Time (t) = 0.7422013 sec.
∴ Bring Acture velocity to Find Head loss from PWA Graph
∴ Head Loss Across Static Mixer = 1 m.
\[ \rho_L = 997.1 \text{ kg/m}^3 \text{ at 25 } ^\circ\text{C} \]
\[ \mu = 0.000895 \text{ kg/m.s (N/m.s) at 25 } ^\circ\text{C} \]

Theory
\[ G = \sqrt{\frac{h_f \times g \times \rho}{\mu \times t}} \]
= 2848.0866 sec \(^{-1} \) too hight
\[ Gzt = 2113.8535 \text{ too hight} \]

Try Static Mixer Diameter = 300 mm.
Use 2 stage = 2 element
∴ Static Mixer Length (L) = 0.75 m.
∴ Acture Velocity = \( \frac{Q}{A} \)
\[ A = 0.819 \text{ m/s} \]

∴ Detention Time \( t \) = 0.9160884 sec.

∴ Bring Acture velocity to Find Head loss from PWA Graph

∴ Head Loss Across Static Mixer = 0.25 m.

\[ \rho_L = 997.1 \text{ kg/m}^3 \text{ at } 25 \text{ °C} \]
\[ \mu = 0.000895 \text{ kg/m.s (N/m.s) at } 25 \text{ °C} \]

Theory

\[ G = \sqrt{\frac{h_j x g x \rho}{\mu x t}} \]

\[ = 1727.0052 \text{ sec}^{-1} \text{ too high} \]

\[ G_{xt} = 1582.0895 \text{ too high} \]

Try Static Mixer Diameter = 400 mm.

Use 2 stage = 2 element

∴ Static Mixer Length \( L \) = 1 m.

∴ Acture Velocity = \[ \frac{Q}{A} \]

\[ = 0.461 \text{ m/s} \]

∴ Detention Time \( t \) = 2.1714688 sec.

∴ Bring Acture velocity to Find Head loss from PWA Graph

∴ Head Loss Across Static Mixer = 0.08 m.

\[ \rho_L = 997.1 \text{ kg/m}^3 \text{ at } 25 \text{ °C} \]
\[ \mu = 0.000895 \text{ kg/m.s (N/m.s) at } 25 \text{ °C} \]

Theory

\[ G = \sqrt{\frac{h_j x g x \rho}{\mu x t}} \]

\[ = 634.54224 \text{ sec}^{-1} \text{ OK} \]

\[ G_{xt} = 1377.8887 \text{ OK} \]
3. Coagulation Basin Design

3.1 Raw Water Pipe

Design Flow (Q Design) = 5,000 m$^3$/day
Velocity in Pipe = 1.8 - 2.0 m/s (Kawamura)
Use = 1.8 m/s

∴ Pipe Diameter (D) = \( \sqrt{\frac{4Q}{\pi v}} \)
= \( \sqrt{\frac{4 \times 5,000}{3.14 \times 1.8 \times 3600 \times 24}} \)
= 0.2023751 m.
Use Pipe Diameter (D) = 0.2 m.

∴ Actual Velocity = \( \frac{Q}{A} \)
= 1.8420711 m/s

3.2 Rapid Mixing

Type = Hydraulic Type
Design Criteria
- Detention Time = 1 - 3 sec
- G - Value = 500 - 700 sec$^{-1}$
- GT = 350 - 1500

3.3 Calculation

Theory: Rule of Thumb - estimating the Length of one element is to designate the length as 1.5 - 2.5 times the pipe diameter. The base on this criteria, the length of one element is in the range of 3 - 5 ft (Kawamura, page no. 88)

- 2 stage = 2 element

\[
Static \ Mixer \ Length = 1.5 \times Diameter(m) \times 2 \times (element) - 0.5 \times Diameter(m)
\]
- 3 stage = 3 element

\[ \text{Static Mixer Length} = 1.5 \times \text{Diameter(m.) \times 3 (element)} - \text{Diameter(m.)} \]

- 4 stage = 4 element

\[ \text{Static Mixer Length} = 1.5 \times \text{Diameter(m.) \times 4 (element)} - 1.5 \times \text{Diameter(m.)} \]

- 5 stage = 5 element

\[ \text{Static Mixer Length} = 1.5 \times \text{Diameter(m.) \times 5 (element)} - 2.0 \times \text{Diameter(m.)} \]

- 6 stage = 6 element

\[ \text{Static Mixer Length} = 1.5 \times \text{Diameter(m.) \times 6 (element)} - 2.5 \times \text{Diameter(m.)} \]

- 7 stage = 7 element

\[ \text{Static Mixer Length} = 1.5 \times \text{Diameter(m.) \times 7 (element)} - 3.0 \times \text{Diameter(m.)} \]

- 8 stage = 8 element

\[ \text{Static Mixer Length} = 1.5 \times \text{Diameter(m.) \times 8 (element)} - 3.5 \times \text{Diameter(m.)} \]

- 9 stage = 9 element

\[ \text{Static Mixer Length} = 1.5 \times \text{Diameter(m.) \times 9 (element)} - 4.0 \times \text{Diameter(m.)} \]
Try Static Mixer Diameter = 200 mm.

Use 3 stage = 3 element

∴ Static Mixer Length (L) = 0.7 m.

∴ Acture Velocity = \( \frac{Q}{A} \) = 1.842 m/s

∴ Detention Time (t) = 0.380007 sec.

∴ Check Renolds Number(R_e) = \( \frac{D_p \rho_L V_s}{\mu} \)

\[ \rho_L = 997.1 \text{ kg/m}^3 \text{ at } 25 \degree \text{C} \]

\[ \mu = 0.000895 \text{ kg/m.s (N/m.s) at } 25 \degree \text{C} \]

\[ R_e = 410442.26 \]

Theory

\[ f = 0.048xR_e^{-0.2} \text{ if } 10^4 < R_e < 10^6 \]

\[ f = 0.193xR_e^{-0.35} \text{ if } 3 \times 10^3 < R_e < 10^4 \]

∴ \( f = 0.003619 \)

Theory

Darcy Formular \[ h_f = f \frac{L v^2}{D 2g} \]

∴ \( h_f = 0.0021906 \)

Theory

\[ G = \sqrt{\frac{h_f x g \rho}{\mu t}} \]

\[ = 251.00527 \text{ sec}^{-1} \text{ too less} \]

\[ Gxt = 95.383772 \]
Try Static Mixer Diameter  =  250 mm.

Use 3 stage  =  3 element

∴ Static Mixer Length (L)  =  0.875 m.

∴ Acture Velocity  =  \( \frac{Q}{A} \)

  =  1.179 m/s

∴ Detention Time (t)  =  0.7422013 sec.

∴ Check Renolds Number\( (R_e) \)  =  \( \frac{D_p \rho_L v_s}{\mu} \)

\( \rho_L = 997.1 \text{ kg/m}^3 \text{ at } 25 \text{ °C} \)

\( \mu = 0.000895 \text{ kg/m.s (N/m.s) at } 25 \text{ °C} \)

\( R_e = 328353.8 \)

Theory

\[
f = 0.048xR_e^{-0.2} \quad \text{if} \quad 10^4 < R_e < 10^6
\]

\[
f = 0.193xR_e^{-0.35} \quad \text{if} \quad 3\times10^3 < R_e < 10^4
\]

∴  \( f = 0.0037842 \)

Theory

Darcy Formular  \( h_f = f \cdot \frac{L \cdot v^2}{D \cdot 2g} \)

∴  \( h_f = 0.0009382 \)

Theory

\[
G = \sqrt{\frac{h_f \cdot xg \cdot p}{\mu \cdot t}}
\]

\[
= 117.54084 \text{ sec}^{-1} \quad \text{too less}
\]

\[
Gxt = 87.238961
\]
Try Static Mixer Diameter = 300 mm.
Use 2 stage = 2 element
∴ Static Mixer Length (L) = 0.75 m.
∴ Acture Velocity = \( \frac{Q}{A} \)
= 0.819 m/s
∴ Detention Time (t) = 0.9160884 sec.

∴ Check Renolds Number\( (R_e) \) = \( \frac{D_p \rho_L \nu_s}{\mu} \)
\( \rho_L = 997.1 \text{ kg/m}^3 \) at 25 °C
\( \mu = 0.000895 \text{ kg/m.s (N/m.s)} \) at 25 °C
\[ R_e = 273628.17 \]

Theory
\[ f = 0.048xR_e^{-0.2} \text{ if } 10^4 < R_e < 10^6 \]
\[ f = 0.193xR_e^{-0.35} \text{ if } 3 \times 10^3 < R_e < 10^4 \]
∴ \[ f = 0.0039247 \]

Theory
Darcy Formular \[ h_f = f \frac{L \nu^2}{D \cdot 2g} \]
∴ \[ h_f = 0.0003352 \]

Theory
\[ G = \sqrt{\frac{h_f \cdot x \cdot g \cdot \rho}{\mu \cdot t}} \]
= 63.237189 \( \text{ sec}^{-1} \) too less
\[ Gxt = 57.930857 \]
Try Static Mixer Diameter $= 400 \text{ mm.}$
Use 3 stage $= 3 \text{ element}$
\[ \therefore \text{Static Mixer Length (L)} = 1.4 \text{ m.} \]
\[ \therefore \text{Acture Velocity} = \frac{Q}{A} = 0.461 \text{ m/s} \]
\[ \therefore \text{Detention Time (t)} = 3.0400564 \text{ sec.} \]
\[ \therefore \text{Check Renolds Number}(R_e) = \frac{D_p \rho_L v_s}{\mu} \quad \rho_L = 997.1 \text{ kg/m}^3 \text{ at } 25 \text{ oC} \]
\[ \mu = 0.000895 \text{ kg/m.s (N/m.s) at } 25 \text{ oC} \]
\[ R_e = 205221.13 \]

Theory
\[ f = 0.048xR_e^{-0.2} \text{ if } 10^4 < R_e < 10^6 \]
\[ f = 0.193xR_e^{-0.35} \text{ if } 3x10^3 < R_e < 10^4 \]
\[ \therefore \quad f = 0.0041572 \]

Theory
\[ Darcy Formular \quad h_f = f \frac{L v^2}{D 2g} \]
\[ \therefore \quad h_f = 0.0001573 \]

Theory
\[ G = \sqrt{\frac{h_f x g x p}{\mu t}} \]
\[ = 23.778303 \text{ sec}^{-1} \quad \text{too less} \]
\[ Gxt = 72.287381 \]
Try Static Mixer Diameter = 200 mm.
Use 6 stage = 6 element
∴ Static Mixer Length (L) = 1.3 m.
∴ Acture Velocity = \( \frac{Q}{A} \)
= 1.842 m/s
∴ Detention Time (t) = 0.7057274 sec.

∴ Check Renolds Number\( (R_e) \) = \( \frac{D_p \rho_L v_s}{\mu} \)
\( \rho_L = 997.1 \text{ kg/m}^3 \) at 25 °C
\( \mu = 0.000895 \text{ kg/m.s (N/m.s)} \) at 25 °C
\( R_e = 410442.26 \)

Theory
\[
f = 0.048xR_e^{-0.2} \quad \text{if} \quad 10^4 < R_e < 10^6
\]
\[
f = 0.193xR_e^{-0.35} \quad \text{if} \quad 3\times10^3 < R_e < 10^4
\]
∴ \( f = 0.003619 \)

Theory
Darcy Formular
\[
h_f = f \frac{L v^2}{D 2g}
\]
∴ \( h_f = 0.0040683 \)

Theory
\[
G = \sqrt{\frac{h_f x g x \rho}{\mu x t}}
\]
= 251.00527 sec\(^{-1}\) too less
∴ \( G x t = 177.14129 \)
Try Static Mixer Diameter = 150 mm.
Use 6 stage element
∴ Static Mixer Length (L) = 0.975 m.
∴ Acture Velocity = \( \frac{Q}{A} \) = 3.275 m/s
∴ Detention Time (t) = 0.2977287 sec.

∴ Check Renolds Number\( (R_e) \) = \( \frac{D_p \rho_L v_s}{\mu} \) \( \rho_L = 997.1 \) kg/m\(^3\) at 25 °C
\( \mu = 0.000895 \) kg/m.s (N/m.s) at 25 °C
\( R_e = 547256.34 \)

Theory
\[ f = 0.048 x R_e^{-0.2} \quad if \quad 10^4 < R_e < 10^6 \]
\[ f = 0.193 x R_e^{-0.35} \quad if \quad 3\times10^3 < R_e < 10^4 \]
∴ \( f = 0.0034167 \)

Theory
Darcy Formular
\[ h_f = f \frac{L v^2}{D 2g} \]
∴ \( h_f = 0.0121391 \)

Theory
\[ G = \sqrt{\frac{h_f x g \sqrt{\rho}}{\mu t}} \]
\[ = 667.53577 \text{ sec}^{-1} \]
\[ Gxt = 198.74458 \]
Graph Static Mixer (PWA)
4. Flocculation Basin Design

4.1 Hydraulic Mixing
Type: Round and End Baffle Wall
Design Plant Capacity: 5000 m$^3$/day
Design Operation Flow: 24 hr.

4.2 Design Criteria
- G - Value = 20 - 60 sec$^{-1}$
- No. of Stage = 2 - 7
- Detention Time = 20 - 40 minute

4.3 Calculation
No. of Flocculation basin = 2 Tank
Flow Rate per Tank = 2500 m$^3$/day
= 104.2 m$^3$/hr
Give Detention Time = 30 min
Theory
\[
(G_{\text{opt}})^{2.8} = \frac{44 \times 10^5}{Cxt_d}
\]
Where:
C is Optimum Dose Alum = 30 mg/l
t$_d$ is Detention Time = 30 min
\[
G_{\text{opt}} = 20.78 \text{ sec}^{-1}
\]
Use \[G_{\text{opt}} \approx 25 \text{ sec}^{-1}\]
Give:
No. of Stage = 4
G1 - Value = 60 sec$^{-1}$
G2 - Value = 35 sec$^{-1}$
G3 - Value \[= 20 \text{ sec}^{-1} \quad (G \text{ is reduce } 50 \%) \]

G4 - value \[= 15 \text{ sec}^{-1} \]

Water Depth of Flocculation Basin = 2 m.

\[\therefore \quad \text{Flocculation basin Volume} \quad = \quad Q \times t \quad = \quad 52.08 \quad \text{m}^3\]

\[\therefore \quad \text{Flocculation Area} \quad = \quad 26.04 \quad \text{m}^2\]

Baffle Area 15% of Flocculation Basin Area

\[\therefore \quad \text{Total Area} \quad = \quad 29.95 \quad \text{m}^2\]

Give Width of Flocculation Basin = 4.5 m.

\[\therefore \quad \text{Then Length of Flocculation Basin} \quad = \quad 6.7 \quad \text{m.}\]

Use \[= \quad 7.0 \quad \text{m.}\]

Give No. of Baffle at Width \[= \quad 7.0\]

Give No. of Baffle at Length \[= \quad 4.0\]

Total No. of baffle

\[\therefore \quad - \text{No. of baffle at Width} \quad = \quad \text{Width of Flocculation Basin} \times \text{No. of Baffle} \quad = \quad 31.50\]

\[\therefore \quad - \text{No. of baffle at Length} \quad = \quad \text{Length of Flocculation Basin} \times \text{No. of Baffle} \quad = \quad 28.00\]

Total No. of baffle \[= \quad 59.50\]

Give Width of Concrete \[= \quad 0.08 \quad \text{m.}\]

\[\therefore \quad \text{Total Area of Baffle} \quad = \quad 4.76 \quad \text{m}^2\]

\[\therefore \quad \text{Acture Flocculation Area} \quad = \quad 26.74 \quad \text{m}^2\]

\[\therefore \quad \text{Acture Flocculation Volume} \quad = \quad 53.48 \quad \text{m}^3\]

\[\therefore \quad \text{Acture Detention Time} \quad = \quad 31 \quad \text{min}\]
Give No. of Stage = 4
Give No. of Baffle per Stage = 10

Stage 1  \[ Acture\ Volume\ in\ stage 1 = \frac{Acture\ Flocculation\ Volume}{G1 - Value} = 60\ sec^{-1} \]

at Q Design = 104.2 \( m^3/hr \)
= \( 0.028935185\ m^3/s \)

Theory

\[ \Delta H = \frac{(G(s^{-1}))^2 x \gamma (m^2/s) x V(m^3)}{g(m/s^2) x Q(m^3/s)} \]

Where: \( \gamma = 0.898 \times 10^{-6}\ m^2/s\ at\ 25^\circ C \)

\[ \therefore \Delta H = 0.15\ m. \]

\[ \therefore \text{Head loss in each bend(slit)} = 0.015\ m. \]

Theory

\[ \Delta H = K \frac{v^2}{2g} \]

Give \( K = 1.6\ ) (Dr. Kawamura)

\[ \therefore v = 0.43\ m/s \]

\[ \therefore \text{The required width for each slit in the stage 1 channel is calculate to be} \]

\[ Q = Av \]

when \( A = width\ for\ each\ slit(m) x\ water\ depth(m) \)

\[ \therefore \text{width of each slit in stage 1} = 0.033\ m. \]

= 33.4811 mm.

Stage 2  \[ Acture\ Volume\ in\ stage 1 = \frac{Acture\ Flocculation\ Volume}{G2 - Value} = 35\ sec^{-1} \]

at Q Design = 104.2 \( m^3/hr \)
= \( 0.028935185\ m^3/s \)
Theory

\[ \Delta H = \frac{(G(s^{-1}))^2 \gamma (m^2/s) x V(m^3)}{g(m/s^2) x Q(m^3/s)} \]

Where: \( \gamma = 0.898 \times 10^{-6} \text{ m}^2/\text{s} \text{ at 25° C} \)

\[ \therefore \Delta H = 0.05 \text{ m.} \]

\[ \therefore \text{Head loss in each bend(slit)} = 0.005 \text{ m.} \]

Theory

\[ \Delta H = K \frac{v^2}{2g} \]

Give \( K = 1.6 \) (Dr. Kawamura)

\[ \therefore v = 0.25 \text{ m/s} \]

\[ \therefore \text{The required width for each slit in the stage 2 channel is calculate to be} \]

\[ Q = Av \]

when \( A = \text{width for each slit (m)} \times \text{water depth (m)} \)

\[ \therefore \text{width of each slit in stage 2} = 0.057 \text{ m.} \]

\[ = 57.3961 \text{ mm.} \]

Stage 3

\[ \text{Acture Volume in stage} = \frac{\text{Acture Flocculation Volume}}{4} \]

G3 - Value = 20 sec\(^{-1}\)

at Q Design = 104.2 m\(^3\)/hr

\[ = 0.028935185 \text{ m}^3/\text{s} \]

Theory

\[ \Delta H = \frac{(G(s^{-1}))^2 \gamma (m^2/s) x V(m^3)}{g(m/s^2) x Q(m^3/s)} \]

Where: \( \gamma = 0.898 \times 10^{-6} \text{ m}^2/\text{s} \text{ at 25° C} \)

\[ \therefore \Delta H = 0.02 \text{ m.} \]

\[ \therefore \text{Head loss in each bend(slit)} = 0.002 \text{ m.} \]

Theory

\[ \Delta H = K \frac{v^2}{2g} \]
\[ \Delta H = K \frac{v^2}{2g} \]

Give \( K = 1.6 \) (Dr. Kawamura)

\[ \therefore v = 0.14 \text{ m/s} \]

\[ \therefore \text{The required width for each slit in the stage 3 channel is calculate to be} \]

\[ Q = Av \]

when \( A = \text{width for each slit(m)} \times \text{water depth(m)} \)

\[ \therefore \text{width of each slit in stage 3} = 0.100 \text{ m.} \]
\[ = 100.4432 \text{ mm.} \]

\underline{Stage 4}

\[ \text{Acture Volume in stage} = \frac{\text{Acture Flocculation Volume}}{4} \]

G4 - value = 15 sec\(^{-1}\)

at Q Design = 104.2 m\(^3\)/hr
\[ = 0.028935185 \text{ m}^3/\text{s} \]

\[ \text{Theory} \]

\[ \Delta H = \frac{(G(\text{s}^{-1}))^2 \gamma (m^2/\text{s})xV(m^3)}{g(m^3/\text{s}^2)xQ(m^3/\text{s})} \]

Where:
\[ \gamma = 0.000000898 \text{ m}^2/\text{s} \text{ at } 25^\circ \text{C} \]

\[ \therefore \Delta H = 0.01 \text{ m.} \]
\[ \therefore \text{Head loss in each bend(slit)} = 0.001 \text{ m.} \]

\[ \text{Theory} \]

\[ \Delta H = K \frac{v^2}{2g} \]

Give \( K = 1.6 \) (Dr. Kawamura)

\[ \therefore v = 0.11 \text{ m/s} \]

\[ \therefore \text{The required width for each slit in the stage 4 channel is calculate to be} \]

\[ Q = Av \]

when \( A = \text{width for each slit(m)} \times \text{water depth(m)} \)

\[ \therefore \text{width of each slit in stage 4} = 0.134 \text{ m.} \]
\[ = 133.9243 \text{ mm.} \]
CHECK

\[ G_{\text{average}} = \frac{G_1 + G_2 + G_3 + G_4}{4} \]

\[ = 32.5 \text{ sec}^{-1} \]

\[ G_{\text{average}_{xt}} = 60,069 \text{ OK.} \]

Design Criteria

\[ 1 \times 10^4 < G_{xt} < 1 \times 10^5 \] (Kawamura)

\[ 1 \times 10^4 < G_{xt} < 15 \times 10^4 \] (Qasim)
5. Sedimentation Basin Design

5.1 Design Criteria (Dr. Kawamura)

5.1.1 Rectangular Basin (Horizontal Flow)

- Surface Loading = 0.83 - 2.5 m/hr
- Water Depth = 3 - 5 m
- Detention Time = 1.5 - 3.0 hr
- Width/Length = > 1/5
- Weir loading = < 11 m³/m.hr.

5.1.2 Upflow type (Radial-Upflow type)

- Circular or square in shape
- Surface Loading = 1.3 - 1.9 m/hr
- Water Depth = 3 - 5 m
- Settling Time = 1 - 3 hr
- Weir loading = 7 m³/m.hr.

5.1.3 Reactor Clarifiers

- Flocculation Time = approx 20 min
- Settling Time = 1 - 2 hr.
- Surface Loading = 2 - 3 m/hr
- Weir loading = 7.3 - 15 m³/m.hr.
- Upflow Velocity = < 50 mm./min.

5.1.4 Sludge Blanket Clarifier

- Flocculation Time = approx 20 min
- Settling Time = 1 - 2 hr.
- Surface Loading = 2 - 3 m/hr
- Weir loading = 7.3 - 15 m³/m.hr.
- Upflow Velocity = < 10 mm./min.
Slurry circulation rate = up to 3 - 5 times the raw water inflow rate

5.2 Calculation

Type: Horizontal rectangular Tank

Design Plant Capacity: 5000 m³/day
Design Operation Flow: 24 hr.

Design Criteria for Horizontal Rectangular Tank

5.2.1 Inlet and Outlet of the Basin

Headloss through the ports = 0.3 - 0.9 mm.
The Size of Ports in Diameter = 0.075 - 0.20 m.
The Ports spacing approx = 0.25 - 0.5 m.
Velocity through Diffuser wall = 0.15 - 0.60 m/s (Prof. Munsin)
Weir Loading rate = 6 - 11 m³/hr.m (Prof. Munsin)

5.2.2 Horizontal rectangular Tank Design (Dr. Kawamura)

Minimum number of tank = 2
Water Depth = 3 - 4.5 m.
Mean Flow Velocity = 0.3 - 1.7 m/min
= 0.15 - 0.91 m/min (Prof. Munsin)
Surface Loading = 0.02 - 0.06 m/min (Prof. Munsin)
= 1.4 - 3.4 m/hr
Detention Time = 1.5 - 4 hr.
= 2 - 4 hr.
Length/Width Ratio (L/W) = Minimum of 4:1
Water Depth/Length Ratio = Minimum of 1:15
Sludge Collector Speed
For the Collection path = 0.3 - 0.9 m/min
For the Return = 1.5 - 3.0 m/min
Design Plant Capacity = 5000 m$^3$/day
Number of Sedimentation Tanks = 2
∴ Flow rate per Basin = 2500 m$^3$/day

5.3 Inlet Zone

Inlet Diffusion Wall

Darcy - Weisbach Formular

$$h_f = f \frac{L}{D} \frac{v^2}{2g}$$

Relationship with G

$$G^2 = \frac{f v^3 \rho}{2 g D \mu}$$

Where:

$G =$ Mean velocity gradient (s$^{-1}$)

$f =$ Friction Factor

$v =$ velocity passed through orifice (m / s)

$D =$ Diameter of Orifice (m)

$\rho =$ Mass Density of Water (kg / m$^3$) = 997.1 kg/m$^2$ at 25 oC

$\mu =$ absolute viscosity (kg / m.s) = 0.000895 kg/m.s (N/m.s) at 25 oC

Give $G$ = 10 1/sec (Design Criteria $G = 10 - 75$ s$^{-1}$)

Orifice made from concrete: $\varepsilon = 1.22$ mm.

Give Diameter of Orifice = 100 mm. ∴ Area = 0.007854 m$^2$

$\varepsilon/D = 0.0122$

find $f$ Give from Reynolds number

Give Reynolds number (R) = 17,000

From Moody Diagram

then $f = 0.042$
∴ \[ v = 0.161 \text{ m/sec} \]
(PWA Criteria 0.15 - 0.20 m/sec)

Check Reynolds Number (R)

\[ R = \frac{vD}{\nu} \quad \text{or} \quad \frac{D_f \delta L v_x}{\mu} \quad \gamma = \frac{\mu}{\delta} \]

17,957 O.K.

∴ Total Area of Pores = \[ \frac{Q}{v} \] m²

0.35888 m²

46 pores

Theory Headloss = \[ \frac{1}{2g} \left( \frac{v}{C} \right)^2 \]

Give C for Orifice = 0.65

∴ Headloss = 0.0031 m.
5.4 Horizontal Rectangular Basin

Design Plant Capacity \(= 5000 \, \text{m}^3/\text{day}\)

Number of Sedimentation \(= 2\) Tank

Flow rate per Basin \(= 2500 \, \text{m}^3/\text{day}\)

Give Water Depth \(= 3\) m.

Give Surface Loading \(= 1.9\) m/hr.

\[= 0.031667 \, \text{m/min} \quad \text{(Design Criteria : 0.02 - 0.06 m/min)}\]

\[\therefore \text{The Required total surface Area } = \frac{Q(m^3/\text{min})}{Surface \, Loading(m/\text{min})} \, \text{m}^2\]

\[= 54.82 \, \text{m}^2\]

Give Tank Width \(= 4.5\) m. \quad \text{(Because Flocculation basin Width : 4.5 m)}

\[\therefore \text{Tank Length } = 12.18 \, \text{m.}\]

Give Detention Time \(= 3\) hr.

\[\therefore \text{Tank Volume } = Q \times t\]

\[= 312.50 \, \text{m}^3\]

\[\therefore \text{Acture Tank Length } = 23.15 \, \text{m.}\]

Use Acture Tank length \(\approx 24\) m.

\[\therefore \text{Width : Length } = 1 : 5.14 \quad \text{(Design Criteria > Minimum 1:5)} \quad \text{OK.}\]

\[\therefore \text{Acture Surface Loading } = 1.00 \, \text{m/hr.}\]

\[= 0.0167 \, \text{m/min.} \quad \text{(PWA. Design Criteria : 0.02 - 0.06 m/min)}\]
5.5 Outlet Zone

Give Weir Loading = 12 m³/hr.m.
Weir Length = 8.68 m.
Give Outlet Zone Width = 2.5 m.

Theory
Launder Size (d) = \( Q^{0.4} \) m.
Use 2 Launder per Basin
= \( \left( \frac{2500}{24 \times 3600 \times 2} \right)^{0.4} \)
∴ d = 0.18 m.
Use d = 0.25 m.

Check Weir Length (L)
Theory
\[ L = \frac{0.2Q}{Hv_s} \]
where:

L = Combined weir length (m)
Q = Flow rate (m³/day) = 2500 m³/day
H = Depth of Tank (m) = 3 m
\( v_s \) = Settling velocity (m/day)

Give \( v_s \) = 0.04812 m/min = 69.2928 m/day

\[ L = \frac{0.2 \times 2500(m^3/day)}{3(m) \times 69.3(m/day)} \]
∴ Weir Length (L) = 2.41 m.
Use weir length = 2.5 m.
∴ Weir length/Basin = 10 m.
Check Launder Depth

**Theory**

\[ W = \left( \frac{Q(m^3/s)}{1.4B(m)} \right)^{2/3} \]

where:

\( W \) = Launder Depth (m.)
\( Q \) = Total flow rate of discharge (m³/sec)
\( B \) = inside width of the Launder (m.)

\( W = 0.19 \) m.
Use \( W = 0.3 \) m.

Use V-notch weir 90 degree

**Theory Discharge of water over V-notch weir**

\[ Q = \frac{8}{15} C_d \sqrt{2g} \tan \left( \frac{\theta}{2} \right) H^{5/2} \]

where:

\( Q \) = Overflow Discharge (m³/s)
\( C_d \) = Discharge Coefficient = 0.584
\( \theta \) = V-notch angle 90 degree
\( H \) = Heigh of flow (m.)

Give 1 V-notch weir have length = 0.15 m.
∴ Total V-notch weir = 67

Flow rate per V-notch weir = 1.56 m³/hr

\[ H^{5/2} = 0.000194 \]
∴ \( H = 0.033 \) m.
6. Filter Tank Design

Design Criteria (Dr. Kawamura)

6.1 Number of Filter

Theory

\[ N = 1.2Q^{0.5} \]

Where:

\( N \) = Total number of filters

\( Q \) = Maximum plant flow rate in (mgd)

6.2 Size of Filter

6.2.1 Ordinary gravity filters

- Width of Filter cell = 3 - 6 m.
- Length to width ratio = 2 : 1 to 4 : 1
- Area of Filter cell = 25 - 100 m²
- Depth of the filter = 3.2 - 6 m.

6.2.2 Self-backwash filters

- Depth of the filter = 3 - 6 m.
- Length to width ratio = 2 : 1 to 4 : 1
- Area of Filter cell = 25 - 80 m²
- Depth of the filter = 5.5 - 7.5 m.

6.3 Filter Bed

Type of Medium and Depth

\[ \frac{L}{d_e} > 1000 \]

for ordinary monosand and media bed

6.4 Filtration Rate

Filter rate = 15 - 20 m³/hr/m²
6.5 Headloss across the filter
- Total Headloss across each filter (for ordinary gravity filter) = 2.7 - 4.5 m.
- Net Headloss available for filtration (for ordinary gravity filter) = 1.8 - 3.6 m.

6.6 Filter washing
- Ordinary rapid sand bed = 0.6 - 0.74 m/min
- Ordinary dual media bed = 0.74 - 0.9 m/min

6.7 Surface Wash Rate: Fix nozzle type
- Flow rate = 0.12 - 0.16 m/min
- Pressure = 55 - 83 KPa

6.8 Filter Media
6.8.1 Medium Sand for rapid sand filter
- Filter rate = 7.0 - 7.5 m/hr.
- Effective Size = 0.45 - 0.65 mm.
- U.C. = 1.4 - 1.7
- Depth = 0.6 - 0.75 m.
- S.G. = 2.63

6.8.2 Multimedia filter
- High rate filtration = 10 - 30 m/hr.
  Sand
  - Effective Size = 0.45 - 0.65 mm.
  - U.C. = 1.4 - 1.7
  - Depth = 0.3 m.

  Anthracite Coal
  - Effective Size = 0.90 - 1.4 mm.
  - S.G. = 1.5 - 1.6
  - Depth = 0.45 m.
6.9 Underdrain

6.9.1 Normal backwash filters

- Pipe lateral
  - Headloss at ordinary backwash rate = 0.9 - 1.5 m.
  - Ordinary size (diameter) = 6 - 10 mm.
  - Lateral spacing = 12 inch
  - Orifices are spaced 3 - 4 in. apart and 45° down-angle from the horizontal on both sides of the lateral
  - Maximum lateral length of 20 ft

- Precast concrete laterals
  - Headloss at ordinary backwash rate
  - Orifice size (diameter) = 8 - 10 mm.
  - 12 in lateral spacing
  - 3 in. orifice spacing on either side of the lateral
  - Maximum lateral length of 16 ft.

6.9.2 Self-backwash type of filter

- Headloss at design backwash rate: 0.15 - 0.3 m.

Gravel Support Bed

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>Size</th>
<th>Depth of Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20 - 40 mm.</td>
<td>100 - 150</td>
</tr>
<tr>
<td>2</td>
<td>12 - 20 mm.</td>
<td>75 mm.</td>
</tr>
<tr>
<td>3</td>
<td>6 - 12 mm.</td>
<td>75 mm.</td>
</tr>
<tr>
<td>4</td>
<td>3 - 6 mm.</td>
<td>75 mm.</td>
</tr>
<tr>
<td>5</td>
<td>1.7 - 3 mm.</td>
<td>75 mm.</td>
</tr>
</tbody>
</table>
### 6.9.3 Basic Hydraulic

<table>
<thead>
<tr>
<th></th>
<th>Ordinary Filter (m/s)</th>
<th>Self-backwash filter (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Influent channel</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>2. Influent valve</td>
<td>0.91</td>
<td>1.52</td>
</tr>
<tr>
<td>3. Effluent Channel</td>
<td>1.52</td>
<td>0.6</td>
</tr>
<tr>
<td>4. Effluent Valve</td>
<td>1.52</td>
<td>0.6</td>
</tr>
<tr>
<td>5. Backwash main</td>
<td>3.05</td>
<td>0.91</td>
</tr>
<tr>
<td>6. Backwash valve</td>
<td>2.4</td>
<td>1.52</td>
</tr>
<tr>
<td>7. Surface wash line</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td>8. Wash-waste main</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td>9. Wash-waste valve</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td>10. Inlet to filter underdrain lateral</td>
<td>1.37</td>
<td>1.37</td>
</tr>
</tbody>
</table>
6.10 Filtration Design

Filtration type : Single Filter Media
Backwash by : Elevation Tank and Surface wash

- Q design = 5000 m³/day

Theory

\[ N = 1.2Q^{0.5} \]  
(Dr. Kawamura, 210 page)

Where :

- \( N \) = Total Number of filters
- \( Q \) = Maximum plant flow rate in mgd

\[ N = 1.37922 \quad \text{Use 3 Tanks} \]

Give Hydraulic Loading = 7 m³/hr/m²

Surface Area of Filter Tank = 29.76 m²

Area per Tank = 9.92 m²

Use Tank area = 4.45 × 2.23 (Length to width ratio 2 : 1 to 4 : 1)

Use Acture Tank Area ≈ 5 × 2.5 m²

Acture tank Area = 12.5 m²

Flow per Tank = 1667 m³/day

1. Inlet Pipe Design

- Give velocity = 0.6 m/s (JWWA)

\[ Q = Av \]

\[ D = \sqrt{\frac{4Q(m^3/s)}{\pi v}} \]

\[ D = 0.20238 \quad \text{m.} \]

\[ \therefore \quad \text{Use inlet pipe diameter (D) = 0.2 m.} \]
Acture Velocity \( = \frac{Q(m^3/s)}{A(m^2)} \)

\therefore \text{ Acture Velocity} \quad = \quad 0.61434 \quad m/s

Headloss
- Give Pipe length (L) \( = \) 2.5 m.

- Friction Loss

Theory
Hazen - William Equation

\[ Q(m^3/s) = 0.278CD(m)^{2.63}S^{0.54} \]

\therefore \quad S = \left( \frac{Q(m^3/s)}{0.278CD(m)^{2.63}} \right)^{0.54} = \left( \frac{3.597Q(m^3/s)}{CD(m)^{2.63}} \right)^{1.85}

From Slope of Energy grade Line (S) = \( \frac{h_L}{L} \)

\therefore \quad h_L = S \times L

\therefore \quad h_L = \left( \frac{3.597Q(m^3/s)}{CD(m)^{2.63}} \right)^{1.85} \times L

New Pipe use \( C = 120 \)

\therefore \quad h_L = 0.00644 \quad m

- Miner Loss

\[ K - \text{Value} \]

1 - Inlet = 0.5  (velocity head)
1 - Outlet = 1  (velocity head)
1 - Gate Valve = 0.2
Total = 1.7
Miner Loss \( = \frac{Kv^2}{2g} \)

Miner Loss \( = 0.0327 \) m

Total Headloss \( = \) Friction Loss + Miner Loss
\( = 0.03914 \) m

2. Filter Media

Sand
- Effective Size \( = 0.45 - 0.65 \) mm. \( \approx 0.55 \) mm.
- Uniformity Coefficient \( = 1.40 - 1.70 \)
- Sand Filter Depth (L) \( = 0.65 \) m
- \( L/d_e \) \( = 1182 \) more than 1000 OK.

Gravel Support Bed

<table>
<thead>
<tr>
<th>Layer</th>
<th>Size (mm.)</th>
<th>Depth of Layer (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper</td>
<td>1</td>
<td>1.7 - 3.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>3 - 6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>6 - 1 2</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>12 - 20</td>
</tr>
<tr>
<td>Lower</td>
<td>5</td>
<td>20 - 40</td>
</tr>
</tbody>
</table>

Underdrain design

- Type : Pipe lateral
- Velocity in lateral pipe \( = 1.37 \) m/s
- Lateral spacing \( = 0.08 - 0.20 \) m (Mahidol University)
  use \( = 0.2 \) m
- Orifice diameter \( = 6.38 - 12.7 \) mm. (Mahidol University)
  use \( = 7 \) mm
- Orifice area/crosssection area of filter tank
  \[ = 0.0015 - 0.005 \]
- Orifice area/pipe area
  \[ = 0.25 - 0.5 \]
- Number of lateral
  \[ = 25 \text{ (give lateral spacing 0.2 m. Length Tank)} \]
- Flow rate
  \[ = \text{Velocity rate in lateral pipe (m/s)} \times \text{Surface Area (m}^2) \]

\[ \therefore \text{Surface area of Pipe lateral} = 0.00056 \text{ m}^2 \]

- Flow per Lateral
  \[ = 3 \text{ m}^3/\text{hr} \]

- **Pipe Lateral Diameter**
  \[ = \sqrt{\frac{4Q(m^3/s)}{\pi v(m/s)}} \text{ m} \]
  \[ = 0.02679 \text{ m} \]

\[ \therefore \text{Use Pipe Lateral Diameter} (D \approx 27 \text{ mm}) \]

- Total Orifice area/ filter area = 0.35 % (Design Criteria: 0.2 - 1.5 %)

\[ \therefore \text{Total Orifice Area} = 0.04375 \text{ m}^2 \]

- Give Number of Orifice
  \[ = N \]

\[ \therefore \frac{\pi D(m)^2}{4} N = \text{Total Orifice Area (m}^2) \]

\[ \therefore N = 1137 \]

- Number of Orifice / Lateral = 45.5 pores

\[ \therefore \text{Orifice Spacing} = 0.05498 \text{ m} \]

Use Orifice Spacing = 0.05 m
3. Clear Water Pipe in Filter tank

3.1 Clear water pipe in Filter tank (Lateral pipe)
- Velocity = 1 m/s (JWWA)
- Flow per tank = 1667 m$^3$/day
  = 69.4 m$^3$/hr
- Pipe Diameter (D)
  \[ Q = Av \]
  \[ D = \sqrt{\frac{4Q(m^3/s)}{\pi v}} \]
  = 0.157 m
- Use pipe diameter = 200 mm.
∴ Lateral Pipe length = 2.5 m.

3.2 Clear water pipe in Filter tank (Maniflow)
- Pipe Diameter (D)
  \[ Q = Av \]
  \[ D = \sqrt{\frac{4Q(m^3/s)}{\pi v}} \]
  = 0.2715 m
∴ Use \approx 300 mm.

4. Backwash Pipe
- Backwash rate = 0.7 m/min (Design Criteria 0.6 - 0.7 m/min)
  \[ Q = Av \]
- Flow rate = 525 m$^3$/hr
- Velocity = 2 m/s (JWWA)
- Pipe Diameter
  \[ D = \sqrt{\frac{4Q(m^3/s)}{\pi v}} \]
5. Lateral Backwash Pipe

- Flow rate = 525 m³/hr
- Velocity = 2 m/s (JWWA)
- Pipe Diameter

\[ D = \sqrt{\frac{4Q}{\pi v}} \]

\[ = 0.3048 \text{ m} \]

\[ \therefore \text{ Use pipe diameter} = 300 \text{ mm.} \]

6. Collector Backwash Pipe

- Flow per tank = 69.4 m³/hr
- Velocity = 3 m/s (JWWA criteria 2.5 - 6.0 m/s)
- Pipe Diameter

\[ D = \sqrt{\frac{4Q}{\pi v}} \]

\[ = 0.091 \text{ m} \]

\[ \therefore \text{ Use pipe diameter} = 100 \text{ mm.} \]

7. Surface wash Pipe

- Surface wash rate = 0.15 m/min (Design Criteria 0.12 - 0.16 m/min)
- Flow rate \[ Q = Av \]

\[ = 112.5 \text{ m}^3/\text{hr} \]

Use Flow rate \[ = 115 \text{ m}^3/\text{hr} \]

- Surface jet pressure = 15 - 20 m. (Design criteria)
  (headloss)

Use \[ = 15 \text{ m} \]
Theor

dy

\[ h = \frac{1}{2g} \left( \frac{v}{c} \right)^2 \]

\[ \therefore \quad \text{Velocity} \quad = \quad 11.151 \quad \text{m/s} \]

- Give Orifice Diameter = 5 mm.
- Orifice Area = \( \frac{\pi D^2}{4} \)

\[ = \quad 1.9635 \times 10^{-5} \quad \text{m}^2 \]

- Flow per orifice = 0.0002 m³/s
- Number of Orifice = \( \frac{Q_{\text{total}}}{Q_{\text{orifice}}} \)

\[ = \quad 146 \quad \text{holes} \]

- Use 2 Pipe lateral

\[ \therefore \quad \text{Number of Orifice per lateral} = \quad 73.0 \quad \text{holes} \]

\[ \therefore \quad \text{Orifice spacing} \quad = \quad 0.0658 \quad \text{m} \quad \text{(Tank Length} \quad = \quad 5 \quad \text{m}) \]

\[ \text{(minus length from Wall tank} \quad = \quad 0.1 \quad \text{m.} \]
\[ \text{2 side} = 0.2 \quad \text{m}) \]

- Try orifice Diameter = 6 mm.

\[ \therefore \quad \text{Orifice area} \quad = \quad 2.82743 \times 10^{-5} \quad \text{m}^2 \]

\[ \text{- Flow per orifice} \quad = \quad 0.0003 \quad \text{m}^3/\text{s} \]

\[ \therefore \quad \text{Number of Orifice per lateral} = \quad 101.3198 \quad \text{holes} \]
- Use 2 Pipe lateral

\[ \therefore \quad \text{Number of Orifice per lateral} = \quad 50.65992 \quad \text{holes} \]

\[ \therefore \quad \text{Orifice spacing} \quad = \quad 0.1 \quad \text{m} \]

-Surface wash pipe Diameter

Main Pipe
Velocity = 2.4 m/s (Dr. Kawamura)

Pipe Diameter

\[ D = \sqrt{\frac{4Q(m^3/s)}{\pi v}} \]

Use Pipe Diameter = 0.1302 m
∴ Use Pipe Diameter = 150 mm.

Lateral = 2 pipe
Velocity = 2.4 m/s (Dr. Kawamura)

Pipe Diameter

\[ D = \sqrt{\frac{4Q(m^3/s)}{\pi v}} \]

Use Pipe Diameter = 0.092 m.
∴ Use Pipe Diameter = 100 mm.
froth and suspended matter is often trapped under the trough bottom and may never be washed out. In either case, the troughs should be large enough to carry the maximum expected wash rate with 5–10 cm free-fall into the trough at the upper end. They should also provide a free-fall to the main collection outlet gullet at the lower end. The bottom of the trough may be either horizontal or sloping.

The required cross-sectional area of the wash trough for a given design flow can be quickly estimated from Figure 21-36. For troughs that have level inverts and rectangular cross section, required trough height can be computed by the following formula:

\[
W + \frac{H}{2} + \frac{B}{2} = \left( \frac{Q}{4B} \right)^{2/3} + \text{free board}
\]

where \(Q\) is the total flow rate of discharge (m³/sec), \(B\) is the inside width of the trough (m), and, freeboard should be a minimum of 50 mm (2 in.).

**Filter Underdrains**

Filter underdrainage systems differ primarily due to the different filter-washing systems and filter types.
8. Water Through Design

Theory

Minimum trough height = \[
\left(\frac{Q(m^3/s)}{1.4B(m)}\right)^{2/3} + \text{free board}^{1/7}
\]

Where:
- \(W\) = water depth inside of the trough from base line (m)
- \(B\) = inside width of the trough (m)
- \(Q\) = total flow rate of discharge per trough (m\(^3\)/s)

Design Criteria backwash water and air = 0.25 - 0.7 m\(^3\)/m\(^2\)/min. Use 0.7 m\(^3\)/m\(^2\)/min

\[
\text{backwash rate} = \frac{Q(m^3/s)}{\text{Surface area}(m^2)}
\]

\[
Q = 0.14583 \text{ m}^3/\text{s}
\]

Use 2 trough per Filter tank

\[
\text{total flow rate of discharge per trough} = 0.072917 \text{ m}^3/\text{s}
\]

Give Free board = 0.051 m. (free board should be a minimum of 50 mm.)

Give inside width of the trough = 0.4 m.

\[
\text{Minimum trough height(P)} = 0.307897 \text{ m.}
\]

\[
= 30.78971 \text{ cm.}
\]

\[
\approx 31 \text{ cm.}
\]

From Sand Layer Depth (L) = 650 mm. = 0.65 m.
Theor

\[ 0.75L + P < H_o < L + P \]

\[ 0.7954 < H_o < 0.957897 \]

Use \( H_o = 0.88 \) m

Theor

\[ 1.5H_o < S < 2H_o \]

\[ 1.31497 < S < 1.753294 \]

Use \( S = 1.53 \) m
Hydraulic Design

1. Head loss (Run)
   1.1 Head loss Sand
   1.2 Head loss Gravel
   1.3 Head loss Underdrain
   1.4 Head loss at outlet piping

2. Head loss (Backwash)
   2.1 Head loss Sand
   2.2 Head loss Gravel
   2.3 Head loss Underdrain
   2.4 Head loss piping from Elevation Tank

3. Head loss from Surface wash

   - Calculation later from Layout and find out Hydraulic grade line and Surface & Backwash pipe
9. Chlorination Design

Design Criteria (Dr. Kawamura)

- **Dosage**: 1 - 5 mg/l (2.5 mg/l average)
- **Number of chlorine feeder**: Minimum of two: one standby is required
- **Residual Chlorine**: Over 0.5 mg/l (Higher Level)
- **Contact time**: Over 30 min (longer)
- **pH**: 6 - 7
- **Chlorine solution tank**: Enough to produce a 1 day supply
- **Chlorine stock**: Minimum of 15 days storage
- **Safety features**: Eye wash, shower, gas masks

**Design**

Use liquid chlorine concentration 1 % prepare from stock liquid chlorine 50 % feed to main pipe before Elevation tank. Keep Contact time = 30 min (minimum)

1. Chlorine Feeder

\[
Q \text{ - Design} = \frac{5000 \text{ m}^3}{d} = \frac{208.3 \text{ m}^3}{hr}
\]

Assume Chlorine demand of water = 1 mg/l

For residual chlorine about 0.5 - 1 mg/l

Use chlorine dosage 1.5 - 2 mg/l

Required chlorine = 208.3 x (1.5 to 2 mg/l)

\[
= 312.5 \text{ to } 416.7 \text{ g/hr}
\]

Meaning of liquid chlorine 1 % is chlorine = 10 g/l (1 L of water = 1000 g)

Chlorine feeder rate = 31.25 to 41.67 L/hr

∴ Use Chlorine feeder rate = 35 to 40 L/hr

2. Dilution stock liquid chlorine solution 50 % to 1 % liquid chlorine solution
Assume stock liquid chlorine solution 50% one plastic equals = 20 liters

Theory

\[ N_1 \times V_1 = N_2 \times V_2 \]

Where:

- \( N \) = Chlorine concentration (%)
- \( V \) = Volume of Liquid (liters)

Give

\[ \begin{align*}
N_1 &= 50 \% \\
V_1 &= 20 \text{ liters} \\
N_2 &= 1 \% \\
V_2 &= ? \text{ liters}
\end{align*} \]

50 \% \times 20 = 1\% \times V_2

\[ \therefore V_2 = 1000 \text{ liters} \]

Use mixing tank volume = 1000 liters made from plastic
Fill stock Liquid Chlorine = 20 liters in mixing tank and fill water until limited 1000 liters

3. Period of Mixing

Maximum chlorine feeder rate = 40 Liters/hr
Required Liquid Chlorine = 960
= 960 Liters/day
So period of mixing = Every Day

4. Liquid Chlorine 50% Stock

Use storage time = 30 days (1 month)
Required Liquid Chlorine = 1 %
= 28800 Liters per month
Required Liquid Chlorine = 50 %
= 576 Liters per month

\[ \therefore \text{Stock Liquid Chlorine} = 28.8 \text{ Plastic Tank per Month} \]
\[ \therefore \text{For Order per Month say} = 29 \text{ Plastic Tank} \]
10. Surface Wash and Backwash System

- Use water from Elevation tank
- Water Level in Elevation tank = 22 - 25 m.
- From Site Pant : Pipe length = 35 m.

**Backwash System**

Use Pressure for Backwash

- Head loss due to water flowing through a sand bed fluidized

Theory

\[
\frac{h_L}{L} = (1 - \varepsilon) \left( S_g - 1 \right)
\]

Where:

- \( h_L \) = head loss through the media bed during backwash. (m)
- \( \varepsilon \) = porosity of the clean stratified bed at rest. (not fluidized) = 0.4
- \( L \) = depth of the stratified bed at rest. (m) = 0.65 m
- \( S_g \) = specific gravity of the media. = 2.65

**Calculation**

\[
\frac{h_L}{L} = 0.6 \quad \text{m}
\]

- Head loss through the supporting gravel bed fluidized

\[
H/L = (150 \nu V/g) \left[ (1-\varepsilon)^2 / \varepsilon^2 \right] (1/\omega)^2 \sum (xi/di)^2 + (1.75 V^2/g)(1-\varepsilon / \varepsilon^2)(1/\omega) \sum (xi/di)
\]

\[
\nu = 0.9629 \quad \text{mm}^2/\text{s}
\]
\[
V(Backwash rate) = 11.67 \quad \text{mm/s}
\]
\[
\varepsilon = 0.4
\]
\[
\omega = 0.8
\]
<table>
<thead>
<tr>
<th>Layer</th>
<th>Size (mm)</th>
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<th>xi</th>
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<tr>
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<td>28.28</td>
<td>75</td>
<td>0.1667</td>
<td>0.0059</td>
<td>0.0002</td>
</tr>
</tbody>
</table>

\[
\begin{array}{lcccc}
 & 450 & 1.0000 & 0.2232 & 0.0778 \\
\end{array}
\]

From \( H/L = 0.0995 \)

\[ H = 0.0448 \text{ m.} \]

- Backwash trough height

Give Fluidized Bed Expand \( = \) 25 \% 

\[
\begin{align*}
\therefore \quad & \text{Media Depth(Fluidized)} = 0.65 + 0.1625 \\
& = 0.8125 \text{ m} \\
\therefore \quad & \text{Expanded sand depth} = 0.8125 - \text{Sand Filter Depth} \\
& = 0.1625 \text{ m} \\
\therefore \quad & \text{Backwash trough height} = \text{Expanded Sand Depth} + \text{trough height} + 6 \text{ in} \\
& \quad \quad \text{(Munsin Tuntuvate)} \\
& = 0.6227971 \\
\therefore \quad & \text{Head loss for Backwash system} = \\
& \text{Headloss in sand bed fluidized} + \text{gravel bed fluidized} + \text{sand depth} + \text{backwashtrough} \\
& = 1.9611 \text{ m}
\end{align*}
\]
11. Headloss in Piping System

Give backwash rate \(= 0.7 \text{ m/min} \) (Design criteria = 0.6 - 0.7 m/min)

Acture tank Area \(= 12.5 \text{ m}^2\)

\[\therefore \text{ Backwash flow rate} = A(m^2) \times v(m/hr)\]
\[= 525 \text{ m}^3/\text{hr}\]

Give Main Pipe Diameter \(= 300 \text{ mm}\)
\[= 0.3 \text{ m}\]

\[\therefore \text{ Acture velocity} = 2.0642 \text{ m/s}\]

11.1 Friction Loss \(h_L\) at Main Pipe

Theory  
Hazen-William Equation

\[Q(m^3/s) = 0.278CD_{(m)}^{2.63}S^{0.54}\]

\[h_L = SxL\]

\[\therefore h_L = \left(\frac{3.597Q(m^3/s)}{CD_{(m)}^{2.63}}\right)^{1.85}xL(m)\]

Give New Pipe  
\(C = 120\)
Pipe Diameter \(= 0.3 \text{ m}\)
Pipe Length \(= 35 \text{ m}\)

\[\therefore h_L = 0.5288 \text{ m} \quad \text{too Low}\]

Try Main Pipe Diameter \(= 250 \text{ mm}\)
\[= 0.25 \text{ m}\]

\[\therefore \text{ Velocity} = 2.9724 \text{ m/s}\]

\[\therefore h_L = 1.2839 \text{ m} \quad \text{OK}\]
11.2 Velocity Miner HeadLoss

11.2.1. accessory

1 - Inlet = 1
3 - 90° Bend = 2.25
Total = 3.25

Theory

\[ \text{Headloss} = \frac{K \cdot v^2}{2g} \]

Headloss = 1.464 m

11.2.3. Velocity headloss in Main pipe

1. Pipe Diameter = 0.3 m
2. Velocity = 2.0642 m/s
3. K = 2.2

Headloss = 0.4778 m

11.3 Headloss at lateral

Flow per Lateral = 21 m³/hr

Theory Hazen-William Equation

\[ h_L = \left( \frac{3.597Q(m^3/s)}{CD_{(m)}^{2.63}} \right)^{1.85} xL(m) \]

Pipe Diameter = 0.0268 m

Headloss = 12.472 m

11.4 Headloss at Orifice

Flow per Orifice = 0.061 m³/hr
- Orifice diameter = 7 mm
Velocity = 0.4411 m/s
Theory Headloss \(= \frac{1}{2g} x \left(\frac{v}{C}\right)^2\)

Give C for Orifice \(= 0.65\)

\[\therefore \] Headloss \(= 0.0235\) m

\[\therefore \] Total Headloss \(= 16.409\) m

- Sand Depth \(= 0.65\) m
- Gravel Depth \(= 0.45\) m
- Sand Expansion \(= 0.16\) m
- Trough Height \(= 0.31\) m

\[\therefore \] Trough Height From Base Line \(= 1.57\) m

\[\therefore \] Total Dynamic Head \(= 17.98\) m

- Water Level in Elevation tank \(= 22 - 25\) m. Choose \(= 22\) m.

\[\therefore \] Different Head Loss \(= 4.02\) m.

11.5 Use Orifice Plate

Theory Headloss \(= \frac{1}{2g} x \left(\frac{v}{C}\right)^2\)

Give C for Orifice \(= 0.65\)

\[\therefore \] Velocity (v) \(= 5.58\) m/s

Theory Pipe Diameter

\[D = \sqrt{\frac{4Q(m^3/s)}{\pi v}}\]

\[\therefore \] Pipe Diameter \(= 0.182\) m.

\(= 182\) mm.
12. Headloss and Hydraulic Profile

\[ Q_{\text{design}} = 5000 \text{ m}^3/\text{d} \]
\[ Q_{\text{max}} = 1.5Q_{\text{design}} \]
\[ = 7500 \text{ m}^3/\text{d} \]

- Headloss at static Mixer
  at Q design = 0.08 m.

- Headloss at Flocculation
  for 1st Stage
  at Q design = 0.15 m.

  for 2nd Stage
  at Q design = 0.05 m.

  for 3rd Stage
  at Q design = 0.02 m.

  for 4th Stage
  at Q design = 0.01 m.

- Headloss at Diffuser wall (Inlet Zone Sedimentation)
  at Q design = 0.0031 m.

- Headloss Over V-notch weir (Outlet Zone Sedimentation)
  at Q design = 0.0328 m.
at \( Q \) design \( = 0.0391 \) m.

- Head loss pass through clean filter media (Filtration)

\[
H/L = \frac{(5\nu V/g)[(1-\epsilon)^2/\omega^3][(6/\omega)^2 \sum (xi/di^2)]}{\nu = 0.9629 \text{ mm}^2/\text{s}}
\]

\[
\nu = 0.9629 \text{ mm}^2/\text{s}
\]

\[
V = 1.94 \text{ mm/s}
\]

\[
\epsilon = 0.4
\]

\[
\omega = 0.8
\]

\[
\text{ชั้นทรายสูง} = 0.65 \text{ m.}
\]

\[
H/L = 1.00
\]

\[
H = 0.65 \text{ m.}
\]

- Headloss Through Gravel (Filtration)

\[
H/L = \frac{(150\nu V/g)[(1-\epsilon)^2/\omega^2] \sum (xi/di^2) + (1.75V^2/g)(1-\epsilon/\omega^2)(1/\omega) \sum (xi/di)}{\nu = 0.9629 \text{ mm}^2/\text{s}}
\]

\[
\nu = 0.9629 \text{ mm}^2/\text{s}
\]

\[
V = 1.94 \text{ mm/s}
\]

\[
\epsilon = 0.4
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\[
\text{From } H/L = 0.0076
\]

\[
H = 0.0034 \text{ m.}
\]
\[ Total \ Head \ loss \ across \ the \ filter \ Media \ at \ Q_{\text{design}} = \ Head \ loss \ pass \ through \ clean \ filter \ media \ (\text{Filtration}) + \ Headloss \ Through \ Gravel \ (\text{Filtration}) \]

\[ = \ 0.652 \ \text{m}. \]

- Head loss for underdrain

1. Head loss at orifice

\[ Total \ Number \ of \ filters = 3 \ \text{Tanks} \]
\[ - \ Q \ design = 5000 \ \text{m}^3/\text{day} \]
\[ \therefore \ \text{Flow rate per Tank} = 1667 \ \text{m}^3/\text{day} \]

1 Tank have Number of lateral

\[ = 25 \]
\[ \therefore \ \text{Flow rate per lateral} = 66.67 \ \text{m}^3/\text{day} \]

1 lateral have Number of orifice

\[ = 45.47 \ \text{pores} \]
\[ \therefore \ \text{Flow rate per orifice} = 1.466 \ \text{m}^3/\text{day} \]
\[ = 0.061 \ \text{m}^3/\text{hr} \]

Theory

\[ Q = Av \]

\[ \therefore \ \text{Velocity pass through orifice} = 0.441 \ \text{m/s} \]

\[ \therefore \ \text{Head loss at orifice} \]

Theory

\[ Q = C_d A \sqrt{2gh_L} \quad (1) \]
\[ Q = Av \quad (2) \]
\[ \therefore \ Av = C_d A \sqrt{2gh_L} \quad (3) \]
\[ \left( \frac{v}{C_d} \right)^2 = 2gh_L \]
\[ \therefore \ h_L = \frac{1}{2g} \left( \frac{v}{C_d} \right)^2 \quad (4) \]
\[
2g(C_d)
\]

\[C_d \text{ Orifice} = 0.61\]

\[\therefore \text{ Head loss at orifice} = 0.027 \text{ m.}\]

2. Head loss at Lateral

Flow rate per lateral \(= 66.67 \text{ m}^3/\text{day}\)

\[= 2.778 \text{ m}^3/\text{hr}\]

Theory

\[Q = Av\]

\[\therefore \text{ Velocity pass through lateral} = 1.37 \text{ m/s}\]

Minor loss \[= \frac{K v^2}{2g}\]

\[k \text{ outlet} = 1\]

\[\therefore \text{ Minor loss} = 0.096 \text{ m.}\]

Theory

\[h_L = \left(\frac{3.597Q(m^3/s)}{CD_{(m)}^{2.63}}\right)^{1.85}xL\]

New Pipe C \(= 120\)

\[h_L = 0.296 \text{ m.}\]

\[\therefore \text{ Total Head loss at } Q_{\text{design}} = h_L + \text{ Minor loss} = 0.391 \text{ m.}\]

- Head loss at delivery Pipe (Lateral Clear water Pipe in Filtration Tank)

Theory

\[Q = Av\]
- Flow per tank = 69.44 m³/hr
- Use pipe diameter = 0.2 m.

∴ Velocity pass through lateral clear water pipe

= 0.614 m/s

- Miner Loss
  1 - Inlet = 0.5 (velocity head)
  1 - Outlet = 1 (velocity head)
  1 - Gate Valve = 0.2
  90° bend = 0.9
  Tee = 1.8
  Total = 4.4

Theory Miner Loss = \( \frac{Kv^2}{2g} \)

= 0.085 m.

- Head loss at delivery Pipe (Maniflow Clear water Pipe in Filtration Tank)

Give Pipe Length = 20 m.

Theory

\[ h_L = \left( \frac{3.597Q(m^3/s)}{CD^{2.63}} \right)^{1.85} xL \]

New Pipe C = 120

\[ h_L = 0.089 \text{ m.} \]