7.5

Capacity of Facility

1. Basic Data and Condition

1.1. Design Period

Phase I = 2001 ~ 2005 (Modified Aeration Process)
Phase II = 2006 ~ 2010 (Modified Aeration Process)
Final Phase = 2011 ~ 2020 (Conventional Activated Sludge Process)

1.2. Sewer System

Combined Sewer System (Storm Water Discharged Overflow Chamber QmaxH → Diversion Chamber)

1.3. Design Location (Sewer Development Area)

Tau Hu – Ben Nghe, Doi – Te Area (THBNDT Area) in Ho Chi Minh City.

1.4. Design Area (Service Area)

Phase I = 828.4 ha
Phase II = 828.4 + 1,963.2 = 2,791.6 ha
Final Phase = 2,791.6 ha

1.5. Design Population

Phase I = 425,830 persons in 2010 year, 2005 year = 432,076 persons
Phase II = 1,390,282 persons in 2020 year, 2010 year = 1,421,778 persons
Final Phase = 1,390,282 persons in 2020 year

1.6. Domestic Wastewater Flow

1997 year = 170 l/person-day
2000 year = 200 l/person-day
2005 year = 260 l/person-day. Design Wastewater Flow per capita
2010 year = 300 l/person-day (Phase I & Phase II)
2020 year = 335 l/person-day (Final Phase)

1.7. Diurnal Wastewater Flow

QaveD = QmaxD
QmaxH = QmaxRH = 1.4 * QmaxD (Domestic Wastewater) + Groundwater

1.8. Design Wastewater Flow

Phase I: QmaxD (= QaveD) = 141,000 m^3/day
QmaxH (= QmaxRH) = 1.4 * QmaxD + Groundwater
= 192,000 m^3/day

Phase II : QmaxD (= QaveD) = 469,000 m^3/day
QmaxH (= QmaxRH) = 1.4 * QmaxD + Groundwater
= 640,000 m^3/day

Final Phase: QmaxD (= QaveD) = 512,000 m^3/day
QmaxH (= QmaxRH) = 1.4 * QmaxD + Groundwater
= 699,000 m^3/day

□ Ground Water Flow = 10% of Domestic Wastewater (QmaxD)

1.9. Formula Wastewater Flow Calculation

Manning's Formula
n = 0.013 (HP) – 0.010 (SP, DCIP, VP, etc)

1.10. Design Wastewater Characteristics (Pollutant Load)

BOD = 60 g/person-day*1,390,282 persons ÷ 512,000 m^3/day = 162.9 mg/l → 163 mg/l
SS = 60 g/person-day*1,390,282 persons ÷ 512,000 m^3/day = 162.9 mg/l → 163 mg/l

1.11. Design Wastewater Characteristics For WWTP

Refer to "Mass Balance" Calculation. Result from the Mass Balance Calculation take side streams into consideration and decide:
BOD = 163 * 1.20 = 196 → 200 mg/l
SS = 163 * 1.30 = 212 → 210 mg/l

1.12. Design Effluent Quality (VIETNAMESE STANDARD TCXD 188 – 1996)

Phase I:	BOD = 50 mg/l below	□ TCXD 188 – 1996:
	SS = 100 mg/l below	Urban Wastewater Standard For Discharge
Phase II:	BOD = 50 mg/l below	□ Phase I = Limitation values "B"
	SS = 100 mg/l below	Phase II = Limitation values "B"
Final Phase:	BOD = 20 mg/l below	Final Phase = Limitation values "A"
	SS = 50 mg/l below	

1.13. Design WWTP Removal Efficiency (Considering Recycle Water Loading)

Item	Designed Water Quality (mg/l)		Removal Efficiency (%)			Remarks
	Influent	Effluent	Primary Sed. Tank	AT + Final Sed. Tank	Total	
BOD	200	20	40	83.3	90.0	Total Removal Efficiency = 90~95% possibly
	(200)	(50)	(30)	(64.3)	(75.0)	Total Removal Efficiency = About 70%
SS	210	30	50	71.4	85.7	Total Removal Efficiency = 90~95% possibly
	(210)	(60)	(35)	(56)	(71.4)	-

Aeration Tank Water Quality: BOD = 120 mg/l (140 mg/l)
SS = 105 mg/l (137 mg/l)

( ) shows Phase I & II

1.14. Wastewater Treatment System and Necessary Train

Phase I:	Modified Aeration Process	(1/8 train)
Phase II:	Modified Aeration Process	(4/8 train)
Final Phase:	Conventional Activated Sludge Process (for AO System, take Step Aeration System together)	(8/8 train)

1.15. Sludge Treatment System

"Separate Thickening ~ Sludge Storage ~ Dewatering ~ Composting"

* Separate Thickening : Raw Sludge = Gravity Thickener
Excess Sludge = Centrifugal Thickener

* Sludge Storage Tank : Mechanical Mixing

* Dewatering : Centrifugal Dewatering

1.16. Excess Sludge Final Disposal System

Disposal of WWTP by sanitary landfill

* Composting, screenings, scum, grit etc…

1.17. Inlet Pipes

(Phase I): ⊡ 1,300^{W} * 1,200^{H} * 0.5‰ * 2Box, BL = - 3.314 m (L.P.S. Point)

(Phase II): ⊡ 2,000^{W} * 1,700^{H} * 0.4‰ * 2Box , BL = - 3.890 m (L.P.S. Point)

□ Lift Pumping Station BL = - 3.890 - 0.300 + α = - 4.200 m

1.18. WWTP Ground Level

Present GL □ + 0.600 m

Design GL = + 2.200 m (About 1.6 m banking)

1.19. Condition at Outlet Point

Outlet Point: "Tac Ben Ro Canal" (Tidal River)

HWL = + 1.650 m (100 Year Probability, Nha Be Point)

LWL = - 2.690 m (")

1.20. WWTP Inlet Fluctuation

Actual Result of Water Supply Following

1.21. WWTP Inlet Water Increase

Refer to "Inlet Wastewater Increase"

1.22 Initial Consideration

(1) Installation of Bypass Waterway

(Primary Sedimentation Tank, Aeration Tank, Final Sedimentation Tank, Disinfection Tank, Design to Bypass Waterway all Water Establishment)

(2) Division of Aeration Tank (devices into 4 = 1 : 1.5 : 1.5 : 2.25)

(3) Set up Step Waterway for Step Aeration System

(4) Extension of each Tank (including Disinfection Tank)

(5) Subdivision of Gravity Thickener. Total 2 tanks → change to 4 tanks

1.23. Characteristic of Inlet Pipes / Flow, Velocity, Water Depth, Water Level

Phase	Flow Category	Flow (m^3/sec)	Velocity (m/sec)	Water Depth (m)	Water Level (m)
Phase I	QmaxD	1.632	0.848	0.740	-3.460
	QmaxH	2.222	0.909	0.940	-3.260
Phase II	QmaxD	4.294	0.994	1.080	-3.120
	QmaxH	5.868	1.068	1.373	-2.827

□ Lift Pumping Station Inflow Bottom Level = - 4.200m

□ Inlet Pipes :

(Phase I): ⊡ $1{,}300^W * 1{,}200^H * 0.5‰ * 2$Box, BL = - 3.314 m (L.P.S. Point)

(Phase II): ⊡ $2{,}000^W * 1{,}700^H * 0.4‰ * 2$Box, BL = - 3.890 m (L.P.S. Point)

1.24. Sludge Density (Conventional Activated Sludge Process)

(1) Raw Sludge (Primary Sedimentation Tank Sludge) = 2.0 ~ 4.0 → 2.0%
(2) Excess Sludge (Final Sedimentation Tank Sludge) = 0.5 ~ 1.0% → 0.6%
(3) Density of Return Sludge = 3,000 ~ 6,000mg/l
(4) Combined Sludge = 1.0% (Raw Sludge + Excess Sludge)
(5) Gravity Thickened Sludge (Combined Sludge) = 2.5%
(6) " (Raw Sludge) = 3.0%
(7) Centrifugal Thickening Sludge = 4.0%
(8) Dewatering Sludge = 20 ~ 22% → 20% (Centrifugal Dewatering)
(9) Compost Sludge = 30 ~ 40% (No Additive)
= 40 ~ 50% (Additive)

1.25. Solids Capture Rate (SS)

(1) Gravity Thickening (Combined Sludge) = 80 ~ 90%
(2) " (Raw Sludge) = 80 ~ 90%
(3) Centrifugal Thickening = 85 ~ 95%
(4) Floatation Thickening = 85 ~ 95%
(5) Dewatering Sludge (Centrifugal, Beltpress, Filter Press) = 90 ~ 95%

1.26. Sludge Generated Rate (Removed SS)

(1) Conventional Process = 100%
(2) Batch Process (High Loading) = 100%
(3) Batch Process (Low Loading) = 75%
(4) OD Process = 75%
(5) Long Time Aeration Process = 75%
(6) Rotating Contractor Process = 92%

1.27. Design Amount of Sludge Production (Solid Amount = t/day): Dry Base

QmaxD (m^3/day) * Inlet SS Density (mg/l) * 10^{-6} *
Total WWTP SS Removal Rate (%) * 10^{-2} *
Sludge Generation Rate SS per Removed Sludge (%) * 10^{-2}.

1.28. Design Amount of Sludge Production (m^3/day): Wet Base

Design Amount Sludge Production (t/day) *
100 / Density of Sludge (%) ÷ Specific Gravity (t/m^3)

(Note): In general, Gravity Rate of wet base sludge is 1.0 t/m^3 (Dewatering Cake, Composting Sludge is shown considering a part).

2. Recycle Flow Loading (WWTP Design Inflow Water Quality – Design Influent Quality)

There is a case that SS load of wastewater quality (influent) reaches over 100% depending on the operation (condition). But the operation works is fine, SS load of recycle flow will be kept around 20 ~ 40%. Also BOD load of recycle flow is generally below 20% (Maintenance management guideline in Japanese Standard).

2.1. Mass Balance Calculation

(1) SS Solids Recovery (Solids Capture Rate)

① Gravity Thickening Sludge (Raw Sludge) = 80 ~ 90%
② Centrifugal Thickening Sludge (Excess Sludge) = 85 ~ 95%
③ Sludge Dewatering (Centrifugal Dewatering) = 90 ~ 95%
④ Additive Rate of Polymer = 1.0%

(2) Mass Balance Calculation

$D = 85 - (X_1 * r_1) + r_2$

D: Design Sludge Production
R: Solids in Recycle Flow
X_1: Inlet Solids at Thickener (Gravity and Centrifugal Thickener)
X_3: Inlet Solids at Dewatering Facility
X_4: Dewatering Cake
r_1: SS Recovery Rate of Thickener = 80 ~ 95% (Gravity and Centrifugal Thickener)
r_3: SS Recovery Rate of Dewatering Facility = 90 ~ 95%
r_2: Additive Rate of Polymer = 1.0%

(Note)

Calculate the Sludge Production of each facilities at the case of Inlet Solid = 100, Design Sludge Production (D) = 85, and take the mass balance of all WWTP then confirm the loading rate of recycle flows.

Loading rate to design a WWTP facilities is calculated by the following equation:

Design WWTP Inlet Water Quality = WWTP Inlet Water Quality + Recycle Water Load (* WWTP Inlet Water Quality = Pollutant Loading / Design Wastewater)

(3) **Case Study (Mass Balance)**

[1] Case 1: (Lower Collection Rate)

$r_1 = 80\%$, $r_3 = 90\%$, $r_2 = 1.0\%$

$X_1 = D * 1/[r_1 * (r_3 + r_2)] = 85 * 1/[0.80 * (0.90 + 0.01)] = 116.8 \rightarrow 117$

$X_3 = X_1 * r_1 = 117 * 0.80 = 93.6 \rightarrow 94$

$X_4 = X_3 * r_3 = 94 * 0.90 = 84.6 \rightarrow 85$

[2] Case 2: (Higher Collection Rate)

$r_1 = 95\%$, $r_3 = 95\%$, $r_2 = 1.0\%$

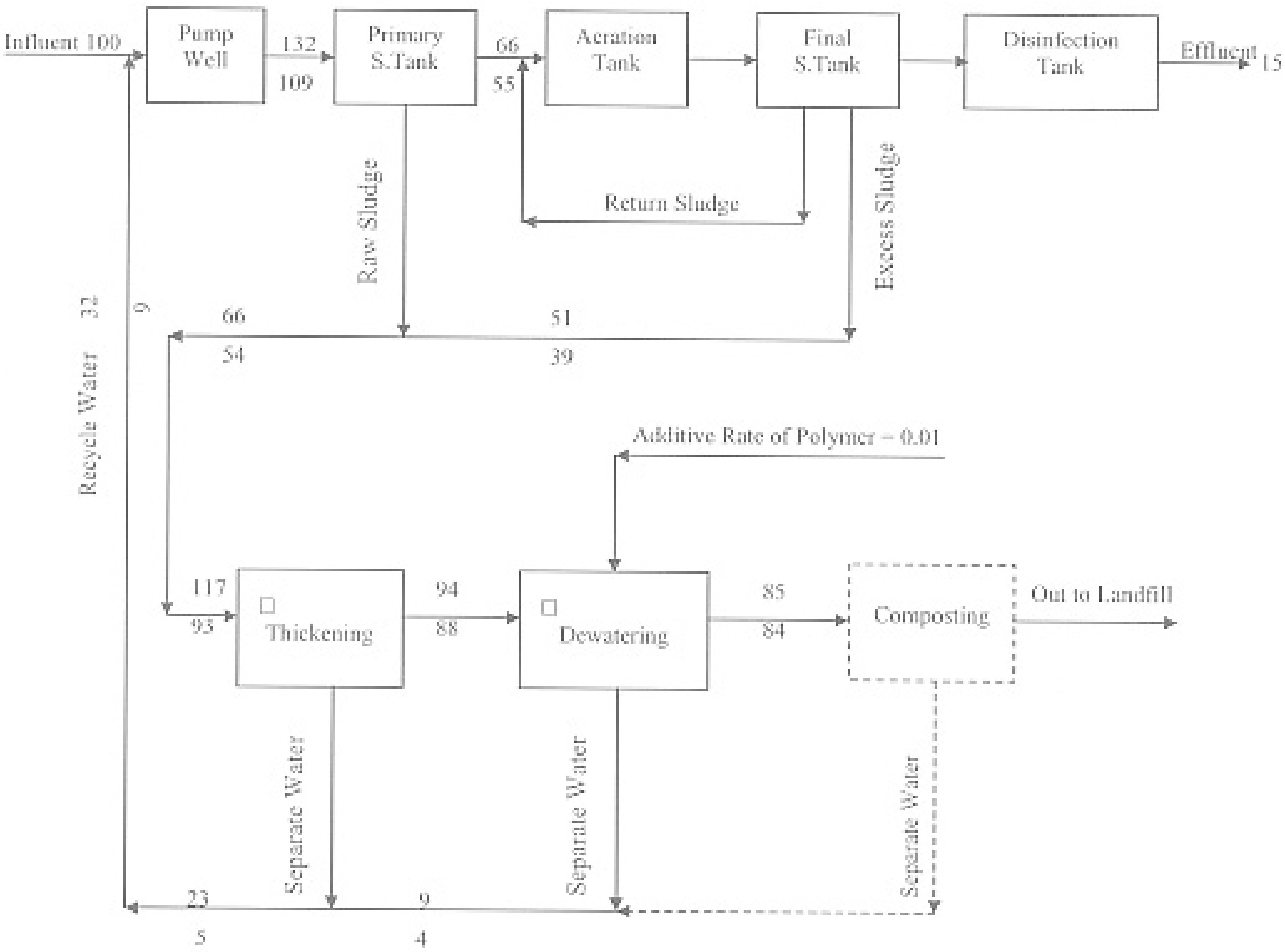
$X_1 = D * 1/[r_1 * (r_3 + r_2)] = 85 * 1/[0.95 * (0.95 + 0.01)] = 93.2 \rightarrow 93$

$X_3 = X_1 * r_1 = 93 * 0.95 = 88.4 \rightarrow 88$

$X_4 = X_3 * r_3 = 88 * 0.95 = 83.6 \rightarrow 84$

(4) **Mass Balance Calculation Result of WWTP (Final Phase)**

(Primary Sedimentation SS Removal Rate = 50%)



(Legend)

Upperstreams = Case 1. (Lower)

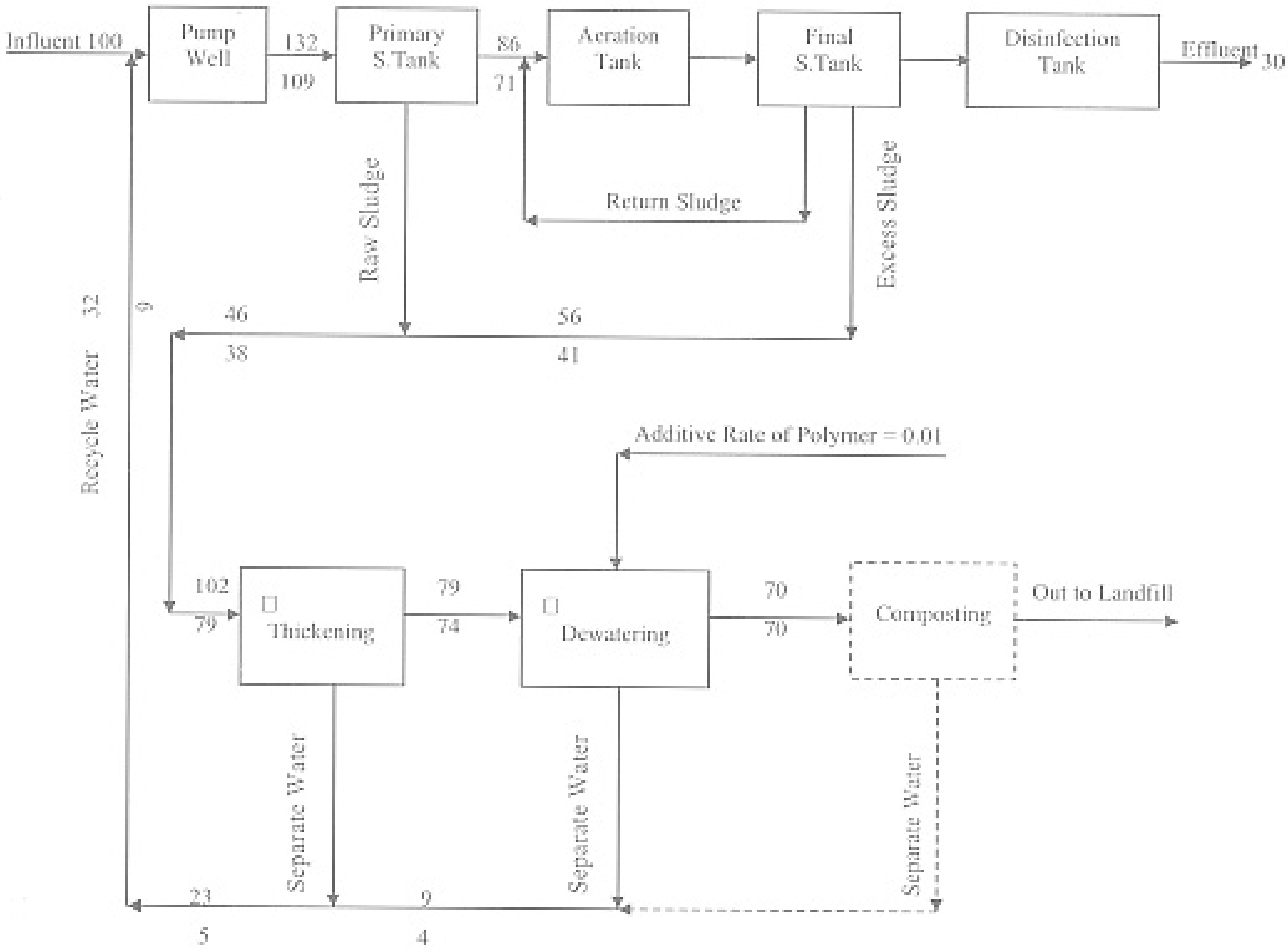
Downstreams = Case 2. (Higher)

□Thickening = Gravity + Centrifugal Thickener

□Dewatering = Centrifugal Dewatering

(5) Mass Balance Calculation Result of WWTP (Phase I, II)

(Primary Sedimentation SS Removal Rate = 35%)



(Legend)

Upperstreams = Case 1. (Lower)

Downstreams = Case 2. (Higher)

□Thickening = Gravity + Centrifugal Thickener

□Dewatering = Centrifugal Dewatering

2.2. WWTP Design Influent Water Quality Determination

(1) Consideration

It is difficult decided SS capture rate, sludge density, density of Separate Water, because there are many factors such as wastewater treatment flow, operating condition, influent character, character area etc …

[1] The density of total solids of Separate Water is desirable less than 1,000 mg/l. It is necessary to keep less than 5,000 mg/l in the worst case.

[2] Return Pollutant Load is generally described below:
In case of good operation, SS loading recycle flow generally 20 ~ 40%, sometimes it reaches 100%. BOD Loading Recycle Flow is generally below 20% (Design Criteria).

(2) WWTP Design Influent Quality

[1] Calculation Result of Mass Balance (SS Return Load)

9 ~ 32% → 30%

[2] Return Load

Recycle Flow Rate, result of mass balance calculation is decided as below:
SS Recycle Rate = 30%
BOD Recycle Rate = 20%

[3] WWTP Design Influent Quality

BOD = 163 * 1.20 = 196 → 200 mg/l
SS = 163 * 1.30 = 212 → 210 mg/l

3. Design Criteria for Main Facilities

3.1 Primary Sedimentation Tank

Item	Conventional Activated Sludge System	Modified Aeration System
Surface Loading	50 m^3/m^2-day	100 m^3/m^2-day About
Effective Depth	3.0 m	Same Left
Overflow Weir Loading	250 m^3/m-day	500 m^3/m^2-day About
Length : Width Ratio	3:1~5:1	Same Left
Width of Tank	2.5 ~ 5.0m (@0.5 m)	Same Left
Bottom Slope of Tank	1/100 ~ 2/100	Same Left
BOD Removal Efficiency	40%	30%
SS Removal Efficiency	50%	35%
COD Removal Efficiency	30 ~ 50% (Other reference)	-
Bypass Waterway	Establish	Same Left
Length of Sludge Collector	Up to about 40 m	Same Left
Slope of Hopper	60° over	Same Left
Baffle Wall Opening Efficiency	15% Above	Same Left
Baffle Plate	Install	Same Left
Freeboard	50 cm About (Beam to clear Tank)	Same Left
Diameter of Sludge Pipe	150 mm Above	Same Left

□ Load for Modified Aeration System = Result of "Pilot Plant Experiment Work".

3.2. Aeration Tank (Conventional Activated Sludge Process)

HRT = 6.0 hr (Step Aeration Process = 4.0 ~ 6.0 hr)
MLSS = 1,500 ~ 2,000 mg/l (Step Aeration Process = 1,000 ~ 1,500 mg/l)
Return Sludge Ratio = 50 ~ 100% (Normal ~ Maximum)

□ Return Sludge Pump = No stand by Pump
Effective Depth = 5.5 m
Density of Return Sludge = 3,000 ~ 6,000 mg/l
BOD-SS Loading = 0.2 ~ 0.4 kg-BOD/kg-SS-day
ASRT = 3 6 days, SRT ≥ 4 day, Sa ≥ 3 day
Width Waterway = Water Depth 1 ~ 2 times with in
Freeboard = 80 cm about (Beam to clear Tank)
Bypass Waterway = Establish
Division of Tank = 4 Division (1 : 1.5 : 1.5 : 2.25)
Baffle Wall opening efficiency: Part enter bottom = $1.0^{(W)}$m *$1.0^{(H)}$m*1Place/Wall
Upper part overside bottom = $0.5^{(W)}$m * $0.3^{(H)}$m * 2 Place/Wall
* Opening way for water flow
1 Scale Tank = 2,500 ~ 7,500 m^3/day-tank

Operation process: 0 = Aerobic zone, A = Anaerobic zone or No Air zone (Only mixing)
(1) Operation with Regular Conventional Activated Sludge Process = 0 + 0 + 0 + 0
(2) Operation with Controlling Filamentous Bulking = A + 0 + 0 + 0 or A + 0 + A + 0
□Operation with Controlling Nitrification Reaction N-BOD increase in treated water.
(3) Operation along Energy = A + 0 + 0 + 0 or A + 0 + A + 0
(4) Beginning of operation, Loading operation

3.3. Aeration Tank (Modified Aeration Process)

BOD-SS Loading = 1.5 ~ 3.0 kg-BOD/kg-SS-day (4.0 kg-BOD/kg-SS-day below)
BOD Volume Loading = 0.6 ~ 2.4 kg/m^3-day
MLSS = 400 ~ 800 mg/l
Sludge Age = 0.3 ~ 0.5 day
Air Feeding = 2 ~ 4 Time of Wastewater Volume
HRT = 1.5 ~ 2.5 hr
Return Sludge Ratio = 5 ~ 10%
SVI = 50 About
BOD Removal Rate = 70% About
Excess Sludge Production Rate = 1 ~ 2 % About

□ (BOD-SS Loading) = Result of "Pilot Plant Experiment Work".

3.4. Final Sedimentation Tank

Item	Conventional Activated Sludge System	Modified Aeration System
Surface Loading	25 m^3/m^2-day [After revise of temperature]	58 m^3/m^2-day below
Effective Depth	3.5 m	Same Left
Overflow Weir Loading	120 m^3/m-day	240 m^3/m^2-day About
Sludge Hopper	QmaxD * 30 min.	Same Left
Length : Width Ratio	3:1 ~ 5:1	Same Left
Bottom Slope of Tank	1/100 ~ 2/100	Same Left
Slope Hopper	60° over	Same Left
Baffle Wall opening efficiency	15% Above	Same Left
Bypass Waterway	Establish	Same Left
Freeboard	50 cm About (Beam to clear Tank)	Same Left
Baffle Plate	Install	Same Left
Diameter of Sludge Pipe	150 mm Above	Same Left

☐ Load for Modified Aeration System = Result of "Pilot Plant Experiment Work".

3.5. Disinfection Tank

Item	Conventional Activated Sludge System	Modified Aeration System
Contact Time	More than 15 minute (QmaxRH to consider)	Same Left
In Tank Average Velocity	Velocity at with Sludge does not settlement	Same Left
Freeboard	50 cm About (Beam to clear Tank)	Same Left

3.6. Gravity Thickener (For Raw Sludge)

Solid Surface Loading = 90 kg/m^2-day
Effective Depth = 4.0 m About
Density of Thickened Sludge = 2.0 ~ 4.0% → 3.0%
Dimension Thickener = Cycle (Standard)
Bottom Slope = 5/100 Above
Diameter of Sludge Pipe = 150 mm Above
Diameter for Sludge Pump = 80 mm Above

3.7. Centrifugal Thickener (For Excess Sludge)

Density of Thickened Sludge = 4.0 %
Operation Time = 24 hours continuous operation, with stand by.

3.8. Sludge Dewatering (Centrifugal Dewatering Method)

Density of Cake = 18 ~ 25% → 20% (Centrifuge)
Operation Time = 24 hours continuous operation, with stand by.

3.9. Sludge Composting Facilities

For market product = During composting material has to be kept at a temperature over 65C for 2 days.

Composting Period = 10 ~ 14 days (Standard)

☐ If not stabilized material should be cured without any Mechanical Aeration for another = 30 ~ 60 days (Natural Aeration), and = 20 ~ 30 days (Mechanical Drying)

High of Storing Compost at Composting Tank = Less than 3.0 m (Standard)

= Less than 5.5 m (Additive)

Air = 10 ~ 20 Nc/min-m^3 (Standard)

☐ In case reducing of composting period

Stores Area, Capacity of Hopper = 1 ~ 2 day

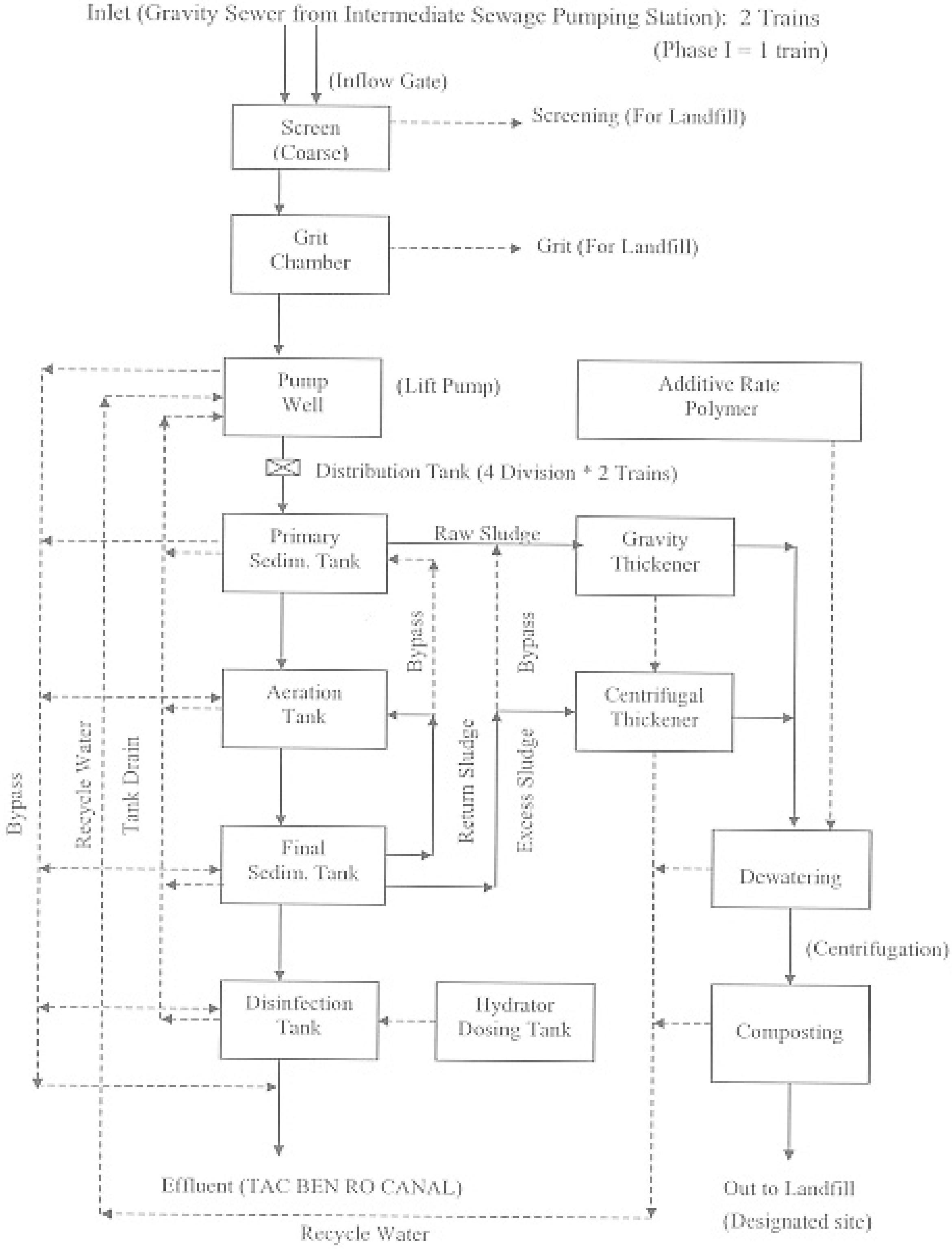
Purpose of composting in this Study = Safety Stabilization, handling (No balance for product market).

Required Quality of Product for Market:

PH = 6.0 ~ 8.5, Water Content = 30 ~ 40%

BOD = 30 mg/g-DS less, C/N Ratio = 20 less

4. Wastewater Treatment System Flow



5. Capacity Calculation of WWTP

5.1. Inflow Gate

Item	Final Phase	Phase I
QmaxH	699,000 m^3/day	192,000 m^3/day
Dimension	1,500mm * 4 unit	1,500mm * 2 (1) unit
Inflow Water Depth	H = 1.373 m	H = 0.940 m
Velocity	V = Q/A = 2.023/(1.5*1.373) = 0.982 m/sec	(2pond) V_1 = 1.111/(1.5*0.940) = 0.788 m/sec (1pond) V_2 = 2.222/(1.5*0.940) = 1.576 m/sec
Gate Loss	h = 1.5*V^2/2g = 1.5 * 0.982^2/2g = 0.074 m ☐ Use h = 200mm	h_1 = 1.5* V_1^2/2g (2 pond) = 1.5 *0.788^2/2g = 0.048m h_2 = 1.5* V_2^2/2g (1pond) = 1.5 *1.576^2/2g = 0.190m ☐ Same Left

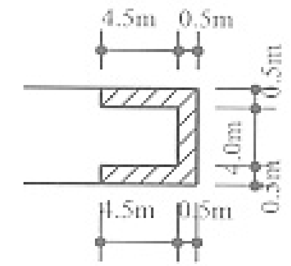
5.2. Coarse Screen

Item	Final Phase	Phase I
QmaxH	$699{,}000\ m^3/day$	$192{,}000\ m^3/day$
Dimension	$3.5m^W$*100mm*60°*FB–9* 4unit	$3.5m^W$*100mm*60°*FB–9* 2(1) unit
Inflow Water Depth	H = 1.373 m	H = 0.940 m
Velocity	V= Q/A = 2.023/(0.100*1.373*3.5/ 0.109) = 0.459 m/sec	V_1 = Q/A (1pond) = 2.222/(0.100* 0.940 * 3.5/0.109) = 0.736 m/sec V_2 = Q/A (2pond) = 1.111/(0.100* 0.940 *3.5/0.109) = 0.368 m/sec
Screen Loss	$h = 2.34 * \sin 60° * (9/100)^{4/3} * 0.459^2/2g$ = 0.001 m	$h_1 = 2.34 * \sin 60° * (9/100)^{4/3} * 0.736^2/2g$ = 0.002 m (1pond) $h_2 = 2.34 * \sin 60° * (9/100)^{4/3} * 0.368^2/2g$ = 0.001 m (2pond)
	☐ Use h = 200mm	☐ Same Left (Note) The Pumping Station will extend in the Phase II, But in the Phase I, just about 1/2 of capacity of Pumping Station will be constructed. Facilities depend on the requirements of the run off.

5.3. Grit Chamber (Simple Type)

Item	Final Phase	Phase I
QmaxH	699,000 m^3/day	192,000 m^3/day
Dimension of Tank	$3.5m^W * 3.0m^L * 4$ tank	$3.5m^W * 3.0m^L * 2$ (1) tank
Inflow Water Depth	H = 1.373 m	H = 0.940 m
Surface Area	A = 3.5 * 3.0 * 4 = 42.0m^2	A_1 = 3.5 * 3.0 * 1 = 10.5m^2 A_2 = 3.5 * 3.0 * 2 = 21.0m^2
Surface Loading	699,000 / 42.0 = 16,643 m^3/m^2-day	192,000 / 10.5 = 18,286 m^3/m^2-day 192,000 / 21.0 = 9,143 m^3/m^2-day
Velocity	V = 2.023/(3.5*1.373) = 0.421m/sec	V_1 = 2.222/(3.5*0.940) = 0.675m/sec V_2 = 1.111/(3.5*0.940) = 0.338m/sec
(Note)	Establish simple Grit Chamber in WWTP, because Grit Chamber and Screen already treated at Intermediate Sewage Pumping Station	

5.4. Primary Sedimentation Tank

Item	Final Phase	Phase I
QmaxD	512,000 m^3/day	141,000 m^3/day
Treatment System	Conventional Activated Sludge Process	Modified Aeration Process
Number of Train	10tank 20waterway/train * 8 trains = 80 tank 160 waterway	10tank 20waterway/train * 1 train = 10 tank 20 waterway
Required Surface Area	SA = 512,000/(50*8*20) = 64.0 m^2/waterway	SA = 141,000/(50*1*20) = 141.0 m^2/waterway
Dimension of Tank	$5.0m^W * 13.0m^L * 3.0m^H$ * 10tank 20waterway/train*8trains (Hunch of bottom) = $300^W * 600^H$ 0.3 * 0.6 * 1/2 = □ 0.09 m^3/m	$5.0m^W * 13.0m^L * 3.0m^H$ * 10tank 20waterway/train*1train Same Left
Required Weir Length	L = 512,000/(250*8) = 256.0 m/train = 12.8 m/waterway -> 13.0 m/waterway  Both side 4.5m + Outlet side 4.0m	L = 141,000/(250*1) = 564.0 m/train = 28.2 m/waterway Same Left Same Left
Surface Area	A = 5.0 * 13.0 * 1 = 65.0 m^2/waterway	Same Left
Volume	V = 65.0*3.0–[0.09*(13.0*2+5.0)] = 192.21 m^3/waterway	Same Left
Surface Loading	50 m^3/m^2-day (Standard) 512,000/(65*160) = 49.2 m^3/m^2-day (OK)	- 141,000/(65.0*20) = 108.5 m^3/m^2-day
Retention Time	1.5 hr (Standard) 192.21/3,200 * 24 = 1.44 hr (OK)	- 192,21/7,050 * 24 = 0.65 hr

Item	Final Phase	Phase I
Overflow Weir Loading	250 m^3/m-day (Standard) 3,200/(4.5 * 2 + 4.0) = 246.2 m^3/m-day (OK)	- 141,000/[(4.5 * 2 + 4.0)*20*1] = 542.3 m^3/m-day
Length and Width Ratio	3:1 ~ 5:1 (Standard) L:W = 13.0: 5.0 = 2.6:1 □ 3:1 (OK)	Same Left
Average Velocity	0.20 m/min below (Standard) QmaxD = 3,200 m^3/day-waterway = 2.22 m^3/min-waterway V(ave) = 2.22/(5.0*3.0-0.09*2) = 0.150m/min (OK)	Same Left QmaxD = 7,050 m^3/day-waterway = 4.90 m^3/min-waterway V(ave) = 4.90/(5.0*3.0-0.09*2) = 0.331m/min
Volume of Raw Sludge	Density of Sludge = 2.0% 512,000 * 210 * 0.50 * 10^{-6} = 53.760 t/day 53.760/2.0*10^2 = 2,688 m^3/day	Same Left 141,000 * 210 * 0.35 * 10^{-6} = 10.364 t/day 10.364/2.0*10^2 = 518.2 m^3/day
Bypass Waterway	Establish	Same Left
Sludge Hopper	1tank 2 waterway Establish 1place	Same Left

5.5. Aeration Tank

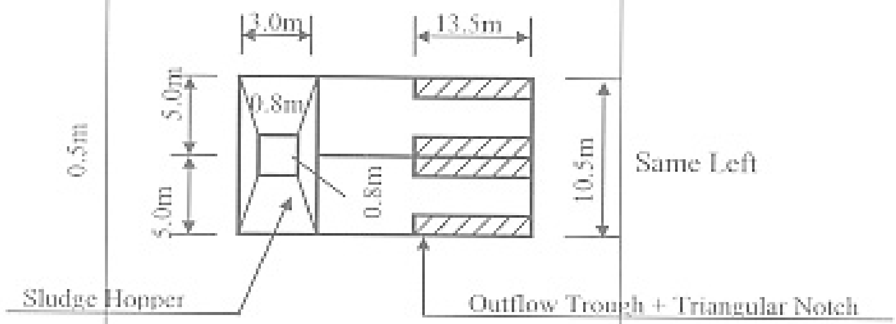
Item	Final Phase	Phase I
QmaxD	512,000 m^3/day = 64,000 m^3/day-train = 6,400 m^3/day-tank	141,000 m^3/day = 141,000 m^3/day-train = 14,100 m^3/day-tank
Treatment System	Conventional Activated Sludge Process	Modified Aeration Process
Number of Train	10 tank / train * 8trains = 80 tank	10 tank / train * 1train = 10 tank
BOD (in)	512,000 * 200 * (1 - 0.40) * 10^{-3} = 61,440 kg/day	141,000 * 200 * (1 - 0.30) *10^{-3} = 19,740 kg/day
SS (in)	512,000 * 210 * (1 - 0.50) * 10^{-3} = 53,760 kg/day	141,000 * 210 * (1 - 0.35) * 10^{-3} = 19,247 kg/day
BOD (Removal)	512,000 * (120 - 20) *10^{-3} = 51,200 kg/day	141,000 * (140 - 50) * 10^{-3} = 12,690 kg/day
Required Volume	V = 64,000 / 24 * 6.0 = 16,000 m^3/train = 1,600 m^3/tank	V = 141,000 / 24 * 2.0 = 11,750 m^3/train = 1,175 m^3/tank
Dimension of Tank	10.5 m^W * 28.0 m^L * 5.5m^H * 10 tank / train * 8 trains (Hunch of bottom and top side) 500^W * 500^H * 4 place	10.5 m^W * 28.0 m^L * 5.5m^H * 10 tank / train * 1 train Same Left
Volume	V = 10.5 * 28.0 * 5.5 * 1 - (0.5^2 * 1/2 *2 * 28) = 1,610.0 m^3/tank	Same Left
HTR	6.0 hr (Standard) 1,610.0/6,400 * 24 = 6.04 hr (OK)	1.5 ~ 2.5 hr (Standard) 1,610.0/14,100 * 24 = 2.74 hr (OK)
BOD-SS Loading	0.2~0.4kg-BOD/kg-SS-day (Standard) 61,400/(1,610.0*80*1,500 ~ 2,000/10^3) = 0.318 ~ 0.239 kg-BOD/kg-SS-day (OK)	1.5~3.0kg-BOD/kg-SS-day (Standard). 4.0 kg-BOD/kg-SS-day below (Result of Pilot Plant Experiment Work) 19,740/(16,100.0*80*400 ~ 800/10^3) = 3.07 ~ 1.53 kg-BOD/kg-SS-day (OK)
BOD Volume Loading	0.600 kg/m^3-day (Standard) 61,440/(1,610.0 * 80) = 0.477 kg/m^3-day (OK)	0.60 ~ 2.40 kg/m^3-day (Standard). 19,740/(16,100.0 * 80) = 1.226 kg/m^3-day (OK)

Item	Final Phase	Phase I
Volume of Return Sludge	Return Sludge Ratio = 50% (Ordinary) ~ 100 % (Max) Rr = (MLSS-Ci) / (Cr-MLSS) * 10^2 Ci: Density of SS Influent (= 105mg/l) Cr: Density of Return Sludge (= 6,000mg/l) MLSS = 1,500~2,000mg/l → 1,700mg/l Rr1 (In case MLSS = 1,700mg/l) = (1,700-105)/(6,000-1,700)*10^2 = 37.09% Rr2 (In case MLSS = 2,000mg/l, Normal Operation) = (2,000-1050)/(6,000-2,000) * 10^2 = 47.38% (□ 50%) Qr = Qi * Rr / 10^2 Qr: Volume of Return Sludge Qi: Influent Sewer Flow Rate Rr: Return Sludge Ratio Qr = 512,000 * 50 ~ 100/10^2 = 256,00 ~ 512,000 m^3/day = 177.78 ~ 355.56 m^3/min □ No Stand-by Pump	Return Sludge Ratio = 5 ~ 10% MLSS = 400 ~ 800 mg/l Same Left Qr = 141,000 * 5 ~ 10/10^2 = 7,050 ~ 14,100 m^3/day = 4.90 ~ 9.79 m^3/min □ Same Left
F/M Ratio	0.40kg-BOD/kg-MLVSS (Standard) F/M = (61,440+512,000*0.5~1.0*20* 10^{-3})/(1,610.0*80*1,500~2,000 * 0.8 * 10^{-3}) = 0.431 ~ 0.348 kg - BOD/kg -MLVSS (OK) □ MLVSS/MLSS = 0.80	- F/M = (19,740+141,000*0.05~0.10 *50*10^{-3})/(16,100*400~800 * 0.8 * 10^{-3}) = 3.900 ~ 1.984 kg-BOD/kg-MLVSS □ Same Left
Sludge Age ①	Sa 3.0days (Standard) Sa = 1,610.0*80*1,500~2,000*10^{-3}/ 53,760 = 3.6 ~ 4.8 days (OK)	0.3 ~ 0.5day (Standard) Sa = 16,100 * 400~800*10^{-3}/19,247 = 0.3 ~ 0.7 day (OK)

Item	Final Phase	Phase I
②	SRT4.0days	-
	SRT= (Va * MLSS + Vs * MLSS)/ (Qw * Cr + Q * SSo)	Same Left
	Vs: Volume of Final Sedimentation Tank = 71,979 m^3	Vs = 17,995 m^3
	Qw: Volume of Excess Sludge = 6,397.5 m^3/day	Qw = 1,796.3 m^3/day
	SSo: Density of SS Wastewater = 30mg/l	SSo = 60 mg/l
	SRT=(1,610*80*1,500~2,000+71,979* 1,500~2,000)/(6,397.5*6,000+512,000* 30) = 5.6 ~ 7.5 days (OK)	SRT = (16,100*400~800+17,995* 400~800)/(1,796.3*2,000+141,000* 60) = 1.1 ~ 2.3 days (OK)
③	ASRT (Part of Aerobic SRT)	Same Left
Operation Process	According to the Operating Condition. O + O + O + O (Regular Operation) A + O + O + O (Controlling Filamentous Bulking and Other Operation) A + O + A + O (") □ O = Aerobic Part □ A = Anaerobic Part	
Water Volume per 1 tank	2,500 ~ 7,500 m^3/day-tank (Standard) V = 512,000/80 = 6,400 m^3/day-tank (OK)	Same Left
Tank Division	4 Division (1 : 1.5 : 1.5 : 2.25)	Same Left
Opening of Baffle Wall Size	Up-size (Both side * 2 places) = $0.5m^W$ * $0.3m^H$ *2 places	Same Left
	Down-size (central Bottom * 1 place) = $1.0m^W$ * $1.0m^H$ * 1 place	Same Left
Tank Width	Water Depth * 1 ~ 2 times (Standard) T.W. = 10.5/ 5.5 = 1.91 times (OK)	Same Left
Bypass waterway	Establish	Same Left
Step Waterway	Establish	Same Left

5.6. Final Sedimentation Tank

Item	Final Phase	Phase I
QmaxD	512,000 m^3/day = 64,000 m^3/day-train = 3,200 m^3/day-waterway	141,000 m^3/day = 141,000 m^3/day-train = 7,050 m^3/day-waterway
Treatment System	Conventional Activated Sludge Process	Modified Aeration Process
Number of Train	10tank 20waterway/train * 8trains = 80 tank 160 waterway	10tank 20waterway/train * 1train = 10 tank 20 waterway
Required Surface Area	A = 64,000/25 = 2,560 m^2/train = 2,560/20 = 128 m^2/waterway	A = 141,000/58 = 2,431 m^2/train = 2,431/20 = 122 m^2/waterway
Dimension of Tank	$5.0m^W$ * $26.0m^L$ * $3.5m^H$ * 10 tank 20waterway/train*8trains (Hunch of Bottom) $300mm^W$ * $600mm^H$ 0.3 * 0.6 *1/2 = 0.09 m^3/m	$5.0m^W$ * $26.0m^L$ * $3.5m^H$ * 10 tank 20waterway/train*1train Same Left
Required Weir Length	L = 64,000/120 = 534.0 m/train = 534/20 = 26.7 → 27.0 m/waterway □ 13.5m * Both side * 2place/waterway	L = 141,000/120 = 1,175 m/train = 1,175/20 = 58.8 m/waterway Same left
Surface Area	A = 5.0 * 26.0 * 1 = 130.0 m^2/waterway (OK)	Same Left
Volume	V = 130.0 * 3.5 - [0.09*(26.0*2 +5.0)] = 449.87 m^3/waterway	Same left
Surface Loading	25 m^3/m^2-day (Standard) 3,200/130.0 = 24.6 m^3/m^2-day (OK)	58 m^3/m^2-day (Result of Pilot Plant Experiment Work) 7,050/130.0 = 54.2 m^3/m^2-day (OK)
Retention Time	3.0 hr (Standard) 449.87/3,200 *24 = 3.37 hr (OK)	- 449.87/7,050 * 24 = 1.53 hr
Overflow Weir Loading	120 m^3/m-day (Standard) 3,200/(13.5*2) = 118.5 m^3/m-day (OK)	- 7,050/(13.5*2) = 261.1 m^3/m-day
Length and Width Ratio	3:1 ~ 5:1 (Standard) 26.0 : 5.0 = 5.2 : 1 □ 5 : 1 (OK)	Same Left Same Left

Item	Final Phase	Phase I
Average Velocity	0.200m/min (Standard) V(ave) = 2.22/(5.0*3.5 - 0.09*2) = 0.128 m/min (OK)	- V(ave) = 4.90/(5.0*3.5 - 0.09*2) = 0.283 m/min (OK)
Volume of Excess Sludge	Density of Sludge = 0.6% 512,000 * 210 * (1-0.50) * 0.714 * 10^{-6} = 38.385 t/day 38.385/0.6*10^2 = 6,397.5 m^3/day	Same Left 141,000 * 210*(1-0.35)*0.560*10^{-6} = 10.778 t/day 10.778/0.6*10^2 = 1,796.3 m^3/day
Volume of Sludge Hopper	QmaxD * 30 min V = 3,200/(24*60)*30 = 66.7 m^3/waterway = 133.4 m^3/tank Upper Tank Volume = 10.5 * 3.0 * 3.5 = 110.3 m^3	Same Left
	Necessary Hopper Volume = 133.4 – 110.3 = 23.1 m^3 above □ (10.5 * 3.0 + 0.8^2) / 2 * H ≥ 23.1 m^3(V) □ H = 1.5 m above - ① Excess Sludge Volume = 6,397.5 m^3/day-tank □ (10.5 * 3.0 + 0.8^2) / 2 * H ≥ 80 * 1/2 [12 hour Pull Out Sludge] □ H = 2.5 m above - ②	Same Left
		Same Left
	□ Hopper = 2waterway 1Place	Same Left
	□ 1Tank 2waterway	Same Left
	□ Sludge Hopper Depth = 2.5m above	Same Left
Bypass Waterway	Establish	Same Left

5.7. Disinfection Tank

Item	Final Phase	Phase I
QmaxD	512,000 m^3/day = 355.56 m^3/min = 5.926 m^3/sec	141,000 m^3/day = 97.92 m^3/min = 1.632 m^3/sec
Treatment System	Conventional Activated Sludge Process	Modified Aeration Process
Required Volume	V = 512,000/(24*60)*15 = 5,334 m^3	1/2 System will be constructed on the Phase I
Dimension of Tank	$5.0m^W$ * $54m^L$ * $5.0m^H$ * 3bend 4waterway 1tank (Hunch of bottom) $300mm^W$ * $300mm^H$ 0.3*0.3*1/2 = 0.045 m^3/m	$5.0m^W$ * $27.0m^L$ * $5.0m^H$ * 3bend 4waterway 1tank Same Left
Section Area	A = 5.0*5.0 - (0.3*0.3*1/2*2) = 24.910 m^2/waterway	Same Left
Volume	V = 24.910*54.0*4 = 5,380.56 m^3 > 5,334 m^3 (OK)	V = 24.910*27.0*4 = 2,690.28 m^3 > 2,667 m^3 (OK)
Average Velocity	V(ave) = 5.926/24.910 = 0.238 m/sec (OK)	V(ave) = 1.632/24.910 = 0.066m/s (OK)
Contact Time	T = 5,380.56/355.56 = 15.1 min (OK) □ Contact Time of Effluent Pipe is not considered	T = 2,690.28/97.92 = 27.5 min (OK) □ Same Left
Bypass Waterway	Establish	Same Left

5.8. Gravity Thickener (For Raw Sludge)

Item	Final Phase	Phase I
Volume of Raw Sludge	Density = 2.0% (1.5%) 53.760 t/day (D.S) 2,688 m^3/day (3,584 m^3/day)	Same Left 10.364 t/day (D.S) 518.200 m^3/day (690.933 m^3/day)
Required Surface Area	$A = 53.760*10^3/90 = 598\ m^2$	$A = 10.364*10^3/90 = 115.2\ m^2$
Dimension of Tank	14.0m * 3.5m^H * 4tanks	14.0m * 3.5m^H * 1tank
Surface Area	$A = \pi * 7.0^2 * 4 = 615.8\ m^2$ (OK)	$A = \pi * 7.0^2 * 1 = 153.9\ m^2$ (OK)
Volume	$V = 615.8 * 3.5 = 2,155.3\ m^3$	$V = 153.9 * 3.5 = 538.65\ m^3$
Solid Surface Loading	$53.760 * 10^3/615.8 = 87.3$ kg/m^2-day (OK)	$10.364*10^3/153.9 = 67.3$ kg/m^2-day (OK)
Retention Time	T = 2,155.3/2,688*24 =19.2 hr (OK) (In case Density of Raw Sludge = 1.5%): CHECK T' = 2,155.3/3,584*24=14.4 hr (OK)	T=538.65/518.200*24 =24.9 hr (OK) (Same Left) T'=538.65/690.933*24=18.7 hr (OK)
Volume of Thickened Sludge	Density of Sludge = 3.0% $53.760/3.0*10^2 = 1,792.0\ m^3$/day	Same Left $10.364/3.0*10^2 = 345.5\ m^3$/day
(Note)	① In case of Effective Depth = 4.0m, Retention Time will be more than 22.0 hr. Consider H= 3.5m to keep thickened and Sedimentation zones.	① Same Left
	② In case of sludge is small at the operation start Time, pipes are arranged to use a gravity thickener for the Combined Sludge. (Bypass Pipe Establish).	② Same Left

5.9. Centrifugal Thickener (For Excess Sludge)

Item	Final Phase	Phase I
Volume of Excess Sludge	Density of Sludge = 0.6% 38.385 t/day (D.S) 6,397.5 m^3/day	Same Left 10.778 t/day (D.S) 1,796.3 m^3/day
Volume of Thickened Sludge	Density of Sludge = 4.0% $38.385/4.0*10^2 = 959.6$ m^3/day	Same Left $10.778/4.0*10^2 = 269.5$ m^3/day
Centrifugal Thickener Capacity	70m^3/hour * 6 unit	70m^3/hour * 2 unit

5.10. Sludge Storage Tank

Item	Final Phase	Phase I
Volume of Inlet Sludge	① Gravity Thickened Sludge: Density of Sludge = 3.0% 53.760 t/day (D.S) 1,792.0 m^3/day ② Centrifugal Thickened Sludge: Density of Sludge = 4.0% 38.385 t/day (D.S) 959.6 m^3/day ③ Total: Density of Sludge (Ave) = 3.35% 92.145 t/day (D.S) 2,751.6 m^3/day	① Same Left: Same Left 10.364 t/day (D.S) 345.5 m^3/day ② Same Left: Same Left 10.778 t/day (D.S) 269.5 m^3/day ③ Same Left: Density of Sludge (Ave) = 3.44% 21.142 t/day (D.S) 615.0 m^3/day
Required Volume	Retention Time = 12 hr above □ Operation Time of Dewatering Machine = 24 hr/day. V = 2,751.6*12/24 = 1,376m^3	Same Left □ Same Left V = 615.0*12/24 = 308m^3
Dimension of Tank	Will be change with Dewatering and Centrifugal Thickener Building. V 1,376 m^3	Same Left V 308 m^3
Retention Time	T = 1,376/2,751.6 * 24 = 12.0 hr above	T = 308/615.0 * 24 = 12.0 hr above

5.11. Sludge Dewatering (Centrifugal Dewatering)

Item	Final Phase	Phase I
Volume of Inlet Sludge	Density of Sludge = 3.35% 92.145 t/day (D.S) 2,751.6 m^3/day	Density of Sludge = 3.44% 21.142 t/day (D.S) 615.0 m^3/day
Operation Time	24 hr/day Continued Operation □ Stand by Correspondence.	Same Left □ Same Left
Dewatering Cake Volume	Density of Sludge = 20.0% $92.145/20.0*10^2$ = 461 m^3/day	Same Left $21.142/20.0*10^2$ = 106 m^3/day
Centrifugal Dwatering Capacity	30m^3/hour * 6 unit	30m^3/hour * 2 unit

5.12. Composting Facility

Item	Final Phase	Phase I
Volume of Inlet Sludge	Dewatering Cake: Density of Sludge = 20.0% 92.145 t/day 461 m^3/day	Dewatering Cake: Density of Sludge = 20.0% 21.142 t/day (D.S) 106 m^3/day

7.6

Hydraulic Calculation

1. BASIC DESIGN CRITERIA.

1.1 Sewerage System

Combined Sewer System. (Over QmaxH wastewater volume won't influence into the WWTP, because it will discharged at the Storm Overflow Chamber. Refer to Sewer Pipe Design).

1.2 Formula

Manning's Formula

n = 0.013 (HP, DCIP, SP), 0.010 (VP, VU, FRPM)

1.3 Design Wastewater Quantity (Unit: m^3/day)

Items	Final Phase	Phase I	Phase II	Remarks
QmaxD	512,000	141,000	469,000	QaveD = QmaxD
QmaxH	699,000	192,000	640,000	QmaxH = QmaxRH

□ Return Sludge Quantity = Daily Maximum Wastewater Quantity * 100% (Max)

(Daily Fluctuation Rate)

QaveD = QmaxD

QmaxH = QmaxRH

= 1.4 * QmaxD (Domestic wastewater) + groundwater

1.4 Specification of Inlet Pipe

(Phase I) ⊡ $1{,}300^W * 1{,}200^H * 0.5‰ *$ 2Box ; BL = - 3.314 m (L.P.S. Point)
(Phase II) ⊡ $2{,}000^W * 1{,}700^H * 0.4‰ *$ 2Box; BL = - 3.890 m (L.P.S. Point)

1.5 Specification of Effluent Pipe

⊙ 2,500 mm * 1.2‰ (HP), L □ 100m

1.6 Outline of WWTP Site

1) Location: Binh Hung / Binh Chanh District

2) Administrative Situation: Green Area (Future)

3) Design Ground Elevation GL = + 2.200m (Present condition GL = + 0.600m)

1.7 Condition of Outlet

(1) Outlet: Tac Ben Ro Canal

(2) Water Level of Outlet (Tidal River): HWL = + 1.650m

LWL = - 2.690 m

Nha Be Point (100 Year Probability)

1.8 Water Velocity, Water Depth, Water Level

(Bottom Level of Inlet Pipe) Phase I = - 3.314 m, Phase II = - 3.890 m

□ L.P.S. Inflow Bottom Level (BL) = - 3.890 – 0.300 + α = - 4.200 m

Item		QmaxD	QmaxH	Remarks
Discharge	m^3/s	1.632	2.222	QaveD = QmaxD
		4.294	5.868	
Velocity	m/s	0.848	0.909	QmaxH= QmaxRH
		0.994	1.068	
Water Depth	m	0.740	0.940	
		1.080	1.373	
Water Level	m	-3.460	-3.260	
		-3.120	-2.827	

Upper Row = Phase I

Lower Row = Phase II

1.9 Outline of Wastewater Treatment Plant (Final Phase)

(1). Primary Sedimentation Tank

$5.0m^{(W)} * 13.0m^{(L)} * 3.0m^{(H)}$ * 10 tanks 20 waterway/train*8 trains,

Overflow Weir Length = 13.0m/waterway, Including Bypass Waterway

(2). Aeration Tank

$10.5m^{(W)} * 28.0m^{(L)} * 5.5m^{(H)}$ * 10 tanks / train * 8 trains

Including Bypass Waterway

(3). Final Sedimentation Tank

$5.0m^{(W)} * 26.0m^{(L)} * 3.5m^{(H)}$ * 10 tanks 20 waterway/train*8 trains,

Overflow Weir Length = 27.0m/waterway, Including Bypass Waterway

(4). Disinfection Tank

$5.0m^{(W)} * 54.0m^{(L)} * 5.0m^{(H)}$ * 3 bends 4 waterway * 1 tank * 1 train,

Including Bypass Waterway

□ $5.0m^{(W)} * 27.0m^{(L)} * 5.0m^{(H)}$ * 3 bends 4 waterway * 1 tank * 2 trains,

(Future: In Case of Two Outflow System) , Including Bypass Waterway

1.10 Design Condition and Future Consideration

(1) Treatment process and capacity is designed in each phase as follows:

	Process	Train	QmaxD (m^3/day)	QmaxH (m^3/day)
Phase I	Modified Aeration	1/8	141,000	192,000
Phase II	Modified Aeration	4/8	469,000	640,000
Final Phase	Conventional Activated Sludge	8/8	512,000	699,000

(2) Design Capacity

Facilities to be constructed in Phase I and Phase II are mainly designed at a capacity of QmaxD or QmaxH of Phase I and Phase II, respectively. However, some of facilities in Phase I and Phase II are designed at a capacity of Final Phase because of more economical and only 8% increase in design Flow of wastewater from Phase II to Final Phase. The following items are considered to be designed at a capacity of Final Phase.

1) Phase I - Facilities to be constructed at a capacity of Final Phase
- Effluent Pipe of WWTP (Φ2,500 * 1.2‰ * 1 unit)
- Connection Pipe between Outflow Pit of FST and Secondary Treated Effluent Tank (Φ2,500 * 1 unit)
- Inlet Pipe (pressurized) (1,800mm * 2 unit)

2) Phase II - Facilities to be constructed at a capacity of Final Phase
- Disinfection Tank

(3) Review of Outlet Location of WWTP in Phase II

It is very important to review another outlet location in Phase II. If effluent can be flowed to Xom Cui Canal in addition to Tac Ben Ro Canal, wastewater processing facilities can be lowered one meter. It can reduce the construction cost. The decision, whether another outlet of WWTP can be installed or not, is determined by the result of EIA and other relating considerations. Therefore, the review will be executed very carefully in Phase II. It will be important review because hydraulic characteristic of WWTP is determined.

2. FORMULA

2.1 Effluent Pipe / Connection Pipe

(1) Circle Pipe

$$\boxed{h = (fe + \partial * L/D + fo) * V^2 / (2 \times g)}$$

h: Head Loss (m)

fe: Inlet Loss (= 0.5)

fo: Outlet Loss (= 1.0)

g: Gravitation Acceleration (= 9.8m/s)

V: Velocity (= Q/A, m/s)

∂: Friction Head Loss Coefficient ($8 * g * n^2/R^{1/3}$, R = D/4)

L: Length (m)

D: Diameter (m)

(2) General Section

$$\boxed{hf = (n^2 * V^2 * L) / R^{4/3}}$$

hf: Friction Head Loss (m)

n: Roughness Coefficient (= 0.013 ~ 0.010)

R: Hydraulic Radius (m)

R = D/4 (Circular Pipe Flowing full)

R = Area / Wetted Perimeter = B * H/(B+2H) (General Section)

B: Width (m), H: Water Depth (m)

$$\boxed{hf = (f' * L \times V^2) / (2 * g * R)}$$

f': Friction Head Loss Coefficient (General Section) = $2*g * n^2/R^{1/3}$

2.2 Disinfection Tank.

(1). Suppressed Rectangular Weir

$$Q = C * B * h^{3/2}$$
$$h = [Q/(1.84 * B)]^{2/3}$$

Q: Overflow Rate (m^3/s)

B: Weir Width (m)

h: Overflow Water Depth (m)

C: Discharge Coefficient (=1.84)

(2) Friction Head Loss

$$h_L = I * L$$

I: Hydraulic Slope

$$I = (Q/K)^2 = [(n * V) / R^{2/3}]^2$$
$$R = B * H / (B + 2 * H)$$

B: Width (m)

H: Water Depth (m)

(3) Bend Loss: 180° Bend

$$hf = f * V^2/2g * n = 1.4 * V^2 / 2g * n$$

f: Bend Loss Coefficient (=1.4)

n: Number of Bends

(4) Inflow Gate (Submerged Orifice)

$$\boxed{Q = C * B * H * \sqrt{(2 * g * h)}}$$

C: Discharged Coefficient (= 0.6)

B: Gate Width (m)

H: Gate Hight (m)

h: Head Loss (m)

$$\boxed{h = [Q / (0.6 * B * H)]^2 * 1 / (2 * g)}$$

(5) Inflow Gate (Gate Discharge Formula)

$$\boxed{h = [Q/ (0.73 * B * H)]^2 * 1 / (2 * g)}$$

2.3 Sedimentation Tank

(1) Outflow Waterway (Inlet Waterway): Friction Head Loss

$$\boxed{h_L = I * L}$$

h_L: Friction Head Loss (m)

I: Slope

L: Length (m)

$$\boxed{I = (n * V / R^{2/3})^2}$$

(2) Outflow Trough:

$$\boxed{h_c = [r * Q^2 / (g * B^2)]^{1/3}}$$

h_c: Critical Depth (m)

r: Non Uniform Velocity Distribution Coefficient (= 1.10)

Q: Outlet Discharge Rate (m^3/s)

B: Trough Width (m)

$$\boxed{h_o = \sqrt{3} * h_c \quad \text{(Free Overflow)}}$$

h_o: Water Depth at Upstream of Contracted Trough

□ Bottom Slope of Trough i = 0

$$\boxed{h_o = \sqrt{(2 * h_c^3 / h_c + h_c^2)} \quad \text{(Submerged Overflow)}}$$

(3) Triangular Notch (Triangle Weir)

$$\boxed{Q = C * h^{5/2}}$$

Q: Discharge (m^3/s)

C: Discharge Coefficient (= 1.42)

h: Overflow Depth (m)

$$\boxed{h = (q / 1.42)^{2/5}}$$

q: Overflow Rate Per Notch (m^3/s)

(4) Inflow Gate (Case of Entrance Loss / Outlet Loss)

$$\boxed{h = 1.5 * V^2 / (2 * g)}$$

□ In other cases, refer to Gate Discharge Formula

2.4 Aeration Tank / Distribution Tank

(1) Suppressed Rectangular Weir

$$\boxed{h = [Q / (1.84 * B)]^{2/3}}$$

(2) Movable Weir

$$\boxed{Q = C * B * H^{3/2}}$$

Q: Overflow Rate for 1 unit (m^3/s)

C: Discharge Coefficient (= 1.84)

B: Weir Width (m)

H: Overflow Water Depth (m)

$$\boxed{h = [Q/(1.84 * B)]^{2/3}}$$

2.5 Measurement Range of Discharge at Outflow Weir (Reference)

Refer to Hydraulic Formula Handbook (Japan)

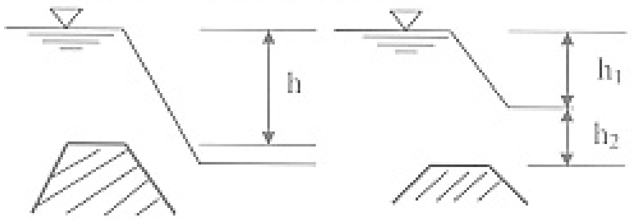
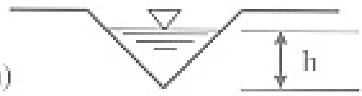
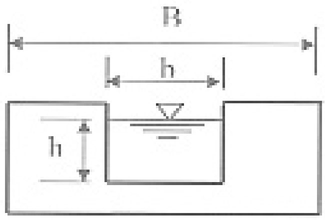
Suppressed Rectangular Weir of Disinfection Tank

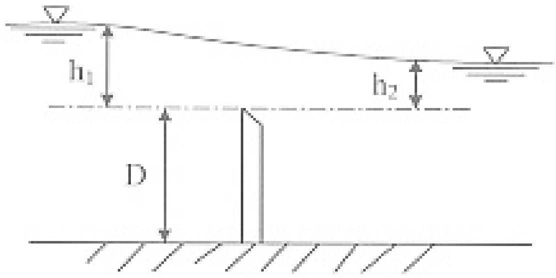
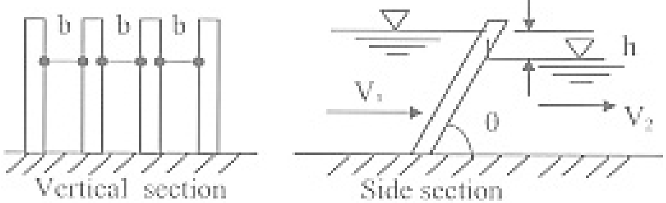
B = 5.0m (Maximum Width at Hydraulic Formula Handbook (Japan))

Range of Water Head	= 0.030 ~ 0.800 m
Measurement Range of Discharge	= 4,320 ~ 611,712 m^3/day
	= 50 ~ 7,080 l/sec

* Here in B = 5.0 m

List of Formula

Name of formula	Formulas	Application
(1) Manning	$V = (1/n) * I^{1/2} R^{2/3}$ V: Average Velocity (m/s) n: Roughness Coefficient I: Hydraulic Gradient R: Hydraulic Radius (m)	Iron Concrete Pipe, Waterway
(2) Darcy-Weisbach	$hf = f * (L/D) * (V^2/2g)$ f: Coefficient of Friction Head Loss L: Straight Length Pipe (m) D: Diameter (m) $f = 0.02 + 1/(2.000D)$	Cast iron or Pressure Pipe with mortar lining or Steel Pipes
(3) Francis	$L = Q/(1.84 * h^{3/2})$: Full Overflow $L = Q/[1.84 * (h_1 + 1.4h_2) * \sqrt{h_1}$: Not Full Overflow L: Length Weir (m) Q: Over Quantity (m^3/s) h, h_1, h_2: Overflow Depth (m) 	Suppressed Rectangle Weir
(4) Thomson	$Q = 1.42h^{5/2}$ h: Overflow Depth (m) 	Triangle Weir Sedimentation Tank Weir Plate
(5) Oki	$Q = 1.84\ b\ h^{3/2}$  b: Length of Weir (m) h: Water Depth (m)	Rectangle Weir
(6) Bill Mount	$Q = Q_1\{1 - (h_2/h_1)^n\}^{0.385}$ Q_1: Over Quantity with Water Depth when Gravity Free Flow (m^3/s) h_1: Weir Angle Upstream Water Depth (m) h_2: Weir Angle Downstream Water Level (m) n: (Suppressed Weir 1.50, Rectangular Weir 1.45, Rectangular Weir 2.50)	Submerged Sharp-crested Weir

(6) Bill Mount		
(7) Tomas-Camp	$h_o = \sqrt{3}\ hcl$: Gravity Downstream Flow $h_o = \sqrt{(2\ hcl^3/\ hl) + hl^2}$: Non-Gravity of Downstream Flow h_o : Shape Upstream Side Water Depth (m) hcl: Maximum Water Depth (m) hl: Shape Downstream Trough Water Level (m) $hcl = \sqrt[3]{(\alpha Q^2/gB^1)}$ α: Kinetic Correction Coefficient by Flow Velocity (distribution connection = 1) Q: Downstream Total Flow (m^3/s) B: Trough Width (m) □ Bottom Slope Trough Water Level	Overflow Trough at Sedimentation
(8) Kirusmel	$h = \beta \sin\theta\, (t/b)^{4/3}\ V^2/2g$ h: Head Loss of Screen Bar (m) β: Coefficient of Screen Section Bars θ : Angle of Screen (°) t: Slope of Screen (mm) b: Space between of Screen Bar (mm)   β =1.60 (a) β =1.77 (b) β = 2.34 (c) β = 1.73 (d)	An amendment of 10 cm Head Loss is usually considered rubbish accumulated

[Loss of Shape]

$h = f * V^2/2g$

f: Shape Loss Coefficient

		[f]
(1) Outlet Flow		1.0
(2) Bend	(90°)	1.0
〃	(180°)	1.4 ~ 3.0
(3) Turn	(45°)	0.13 (CIP Φ900)
〃	(90°)	0.2 (CIP Φ900)
(4) Orifice		3.0
(5) Inlet Flow		0.5

Reference

Shape of Weir	Flow Range Q(l/s)	Size of Reference Eeir and Overflow Range								
		Width Bxb(m)	Range of Overflow water h(m)	L_1(m)	L_3(m)	L_2(m)	L(m)	W(m)	P_1(m)	P_2(m)
Suppressed	6~67	0.6	0.030~0.150	≥ 1.35	0.30	≥ 1.05	0.15	0.30	0.50	0.60
	9~190	0.9	0.030~0.225	≥ 2.05	0.45	≥ 1.60	0.23	0.30	0.60	0.75
	12~400	1.2	0.030~0.300	≥ 2. 70	0.60	≥ 2.10	0.30	0.30	0.70	0.90
	15~690	1,5	0.030~0.375	≥ 3.40	0.75	≥ 2.65	0.38	0.4	0.90	1.05
	20~1430	2.0	0.030~0.500	≥ 4.50	1.00	≥ 3.50	0.50	0.50	1.20	1.50
	30~3950	3.0	0.030~0.750	≥ 6.75	1.50	≥ 5.25	0.75	0.75	1.70	2.00
*(This time)	**50~7080**	**5.0**	**0.030~0.800**	**≥ 9.0**	**1.60**	**≥ 7.40**	**0.80**	**1.00**	**2.00**	**2.50**
Rectangle	4~92	0.9x0.36	0.030~0.270	≥ 1.71	0.54	≥ 1.44	-	0.20	0.50	0.60
	5~150	1.2x0.48	0.030~0.312	≥ 2.14	0.63	≥ 1.83	-	0.25	0.60	0.75
Triangle	2~25	0.60	0.070~0.200	≥ 1.00	0.40	≥ 0.80	-	0.12	0.35	0.50
	2~48	0.80	0.070~0.260	≥ 1.32	0.52	≥ 1.06	-	0.30	0.60	0.75

Quantity Formula

Suppressed Rectangle Weir

$$Q = C*B*h^{3/2}$$

$$C = 1.785 + (0.00295/h + 0.237 * h/W) * (1+\varepsilon)$$

Here in:

Q: Overflow (m^3/s), B: Width of Weir (m), h: Depth of Water (m), C: Discharge Coefficient ($m^{1/2}/s$), W: Height from Bottom of Trough to the Top of the Weir (m).

Explanation:

This Formula is used = JIS B 8302. The Adapted Range is B □ 0.5m, W = 0.3 ~ 2.5m,

h = 0.03 ~ 0.8m (Use in the Range of $h < W$ and $h < B/4$)

In this Range, 95% of Flow Quantity is certain.

Uncertain Quantity is ± 1.7 %.

3. CALCULATION OF HYDRAULIC PROFILE

3.1 Effluent Pipe Head Loss (Disinfection Tank Outflow Pit Water Level)

Item		Unit	QmaxD	QmaxH
Quantity		m^3/d	512,000	699,000
		m^3/s	5.926	8.090
Establishment Number		Number	1	1
Unit Flow		m^3/s	5.926	8.090
Effluent Pipe Form		-	⊙2,500 mm (HP) * 1.2‰	
Section Area		m^2	A = 4.909	
Length		m	About 100	
Velocity		m/s	1.207	1.648
Effluent Water Level		m	HWL = + 1.650, LWL = - 2.690	
Head Loss	Inlet Loss	m	0.037	0.069
	Outlet Loss	m	0.074	0.139
	Straight Pipe Loss	m	0.045	0.083
	Total	m	0.156	0.291
Water Level		m	**+ 1.810**	**+ 1.950**

NOTE:

$h = (fe + \partial * L/D + fo) * V^2/2g$, $\partial = 8gn^2/R^{1/3}$, $R = D/4$, $n = 0.013$

(Section Area)
$A = \pi * 2.5^2 * 1/4 = 4.909\ m^2$

(Velocity) V = Q/A
V(QmaxD) = 5.926/4.909 = 1.207 m/s
V(QmaxH) = 8.090/4.909 = 1.648 m/s

$R=2.500/4 = 0.625$, $\partial = 8 * 9.8 * 0.013^2/0.625^{1/3} = 0.015$

(Inlet Loss)
$h(QmaxD) = 0.5 * 1.207^2/2g = 0.037$ m
$h(QmaxH) = 0.5 * 1.648^2/2g = 0.069$ m

(Outlet Loss)
$h(QmaxD) = 1.0 * 1.207^2/2g = 0.074$ m
$h(QmaxH) = 1.0 * 1.648^2/2g = 0.139$ m

(Straight Pipe Loss)
$h(QmaxD) = 0.015 * 100/2.5 * 1.207^2/2g = 0.045$ m
$h(QmaxH) = 0.015 * 100/2.5 * 1.648^2/2g = 0.083$ m

(Water Level)
$WL(QmaxD) = + 1.650 + 0.156 + \alpha = + 1.810$ m
$WL(QmaxH) = + 1.650 + 0.291 + \alpha = + 1.950$ m

3.2 Disinfection Tank, Outflow Weir Head Loss (End Point Water Level of Disinfection Tank)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	1	1
Unit Flow	m^3/s	5.926	8.090
Dimension of Overflow Weir	Suppressed Rectangular Weir B = 5.0m (Fixed Weir)		
Downstream Water Level	m	**+ 1.810**	**+ 1.950**
Weir Level	m	+ 2.000	
Overflow Water Depth	m	0.746	0.918
Water Level	m	**+ 2.750**	**+ 2.920**

NOTE:	$h = [Q/(1.84 * B)]^{2/3}$
	(Overflow Water Depth) $h(QmaxD) = [5.296/(1.84 * 5.0)]^{2/3} = 0.746$ m $h(QmaxH) = [8.090/(1.84 * 5.0)]^{2/3} = 0.918$ m **(Water Level)** $WL(QmaxD) = +2.000 + 0.746 + \alpha = +2.750$ m $WL(QmaxH) = +2.000 + 0.918 + \alpha = +2.920$ m Comment: In case of H.W.L of discharged river, it is possible to discharge by gravity flow. Because the top level of discharged weir is set higher than the High water level in the discharged pit, therefore, hydraulically, it is separated from discharged river. Therefore, gate at discharged point or closing equipment is not hydraulically necessary.

3.3 Disinfection Tank Inside Head Loss (Inlet Point Water Level of Disinfection Tank)

Item		Unit	QmaxD	QmaxH
Quantity		m^3/d	512,000	699,000
		m^3/s	5.926	8.090
Establishment Number		Number	1	1
Unit Flow		m^3/s	5.926	8.090
Dimension of Disinfection Tank		-	$5.0m^{(W)} * 54.0m^{(L)} * 5m^{(H)} * 3$ bend 4 waterway *1 tank	
Downstream Water Level		m	+ 2.750	+ 2.920
Velocity		m/s	0.237	0.313
Head Loss	Straight Pipe Loss	m	0.001	0.002
	Bend Loss	m	0.012	0.021
	Total	m	0.013	0.023
Water Level		m	+ 2.770	+ 2.950

NOTE:

$h_L = I * L$, $I = (Q/K)^2 = [(n * V)/R^{2/3}]^2$,
$R = B * H/(B + 2H)$, $hf = 1.4 * V^2/2g * n$

(Velocity) $V = Q/A$
V(QmaxD) = 5.926/(5.0 * 5.00) = 0.237 m/s
V(QmaxH) = 8.090/(5.0 * 5.17) = 0.313 m/s

(Hydraulic Slope)
$I(QmaxD) = [(0.013 * 0.237)/1.667^{2/3}]^2 = 4.803 * 10^{-6}$
$I(QmaxH) = [(0.013 * 0.313)/1.685^{2/3}]^2 = 9.685 * 10^{-6}$

(Hydraulic Radius)
R(QmaxD) = 5.0*5.00/(5.0 + 2 * 5.00) = 1.667 m
R(QmaxH) = 5.0*5.17/(5.0 + 2 * 5.17) = 1.685 m

(Friction Head Loss)
$hL(QmaxD) = 4.803 * 10^{-6} * 54.0 * 4 = 0.001$ m
$hl(QmaxH) = 9.685 * 10^{-6} * 54.0 * 4 = 0.002$ m

(Bend Loss)
$hf(QmaxD) = 1.4 * 0.237^2/2g * 3 = 0.012$ m
$hf(QmaxH) = 1.4 * 0.313^2/2g * 3 = 0.021$ m

(Water Level)
$WL(QmaxD) = + 2.750 + 0.013 + \alpha = + 2.770$ m
$WL(QmaxH) = + 2.920 + 0.023 + \alpha = + 2.950$ m

(Disinfection Tank Bottom Level) H = 5.0 m
BL = + 2.770 – 5.000 = - 2.230 m

3.4 Disinfection Tank Inflow Gate Head Loss (Inlet Pit Water Level of Disinfection Tank)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	1	1
Unit Flow	m^3/s	5.926	8.090
Downstream Water Level	m	**+ 2.770**	**+ 2.950**
Dimension of Inflow Gate	-	$2{,}000mm^{(W)} * 2{,}000mm^{(H)} * 1$ Gate	
Velocity	m/s	1.482	2.023
Gate Head Loss	m	0.168	0.313
Water Level	m	**+ 2.940**	**+ 3.270**

NOTE:	**Bypass Waterway Line (Bypass Gate = 2,000mm * 1 Gate + Stop Log)** Inlet gate ($2{,}000mm^{(W)} * 2{,}000mm^{(H)} * 1$ Gate) $h = 1.5 * V^2/2g$, or $h = [Q/(0.73 * B * H)]^2 * 1/2g$.
	(Velocity) $V = Q/A$ $V(QmaxD) = 5.926/(2.0 * 2.0) = 1.482$ m/s $V(QmaxH) = 8.090/(2.0 * 2.0) = 2.023$ m/s **(Gate Head Loss)** $h(QmaxD) = 1.5 * 1.482^2/2g = 0.168$ m $h(QmaxH) = 1.5 * 2.023^2/2g = 0.313$ m **(Water Level)** $WL(QmaxD) = + 2.770 + 0.168 + \alpha = + 2.940$ m $WL(QmaxH) = + 2.950 + 0.313 + \alpha = + 3.270$ m

3.5 Disinfection Tank ~ Secondary Treatment Water Tank Connection Part Head Loss (Water Level of Secondary Treatment Water Tank)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	1	1
Unit Flow	m^3/s	5.926	8.090
Downstream Water Level	m	**+ 2.940**	**+ 3.270**
Dimension of Connection	-	5.0m□	
Section Area	m^2	25.0	
Connection Method	-	Direct Connection	
Velocity	m/s	0.237	0.324
Connection Part Head Loss	m	0.004	0.008
Water Level	m	**+ 2.950**	**+ 3.280**

NOTE:	$V = Q/A,\ h = 1.5 * V^2/2g$
	(Section Area) $A = 5.0 * 5.0 = 25.0\ m^2$ **(Velocity)** $V(QmaxD) = 5.926/25.0 = 0.237$ m/s $V(QmaxH) = 8.090/25.0 = 0.324$ m/s **(Connected Part Head Loss)** $h(QmaxD) = 1.5 * 0.237^2/2g = 0.004$ m $h(QmaxH) = 1.5 * 0.324^2/2g = 0.008$ m **(Water Level)** $WL(QmaxD) = +2.940 + 0.004 + \alpha = +2.950$ m $WL(QmaxH) = +3.270 + 0.008 + \alpha = +3.280$ m

3.6 Secondery Treatment Water Tank ~ Final Sedimentation Tank, Outflow Pit Connection Pipe Head Loss (Final Sedimentation Tank Outflow Pit Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	1	1
Unit Flow	m^3/s	5.926	8.090
Downstream Water Level	m	**+ 2.950**	**+ 3.280**
Dimension of Pipe Connection	mm	⊙2,500 (SP) * 1 pipe	
Section Area	m^2	4.909	
Length	m	About 65	
Velocity	m/s	1.207	1.648
Connection Pipe Head Loss	m	0.140	0.262
Water Level	m	**+ 3.090**	**+ 3.550**

NOTE:

$h_c = (1.5 + \delta * L/D) * V^2/2g$, $R = D/4 = 2.5/4 = 0.625$

$\delta = 8 * g * n^2/R^{1/3}$, $n = 0.013$

(Section Area)

$A = \pi * 2.5^2 * 1/4 = 4.909\ m^2$

(Velocity)

V(QmaxD) = 5.926/4.909 = 1.207 m

V(QmaxH) = 8.090/4.909 = 1.648 m

(Connection Pipe Head Loss) $\delta = 8 * 9.8 * 0.013^2/0.625^{1/3} = 0.015$

$h(QmaxD) = (1.5 + 0.015 * 65/2.5) * 1.207^2/2g = 0.140$ m

$h(QmaxH) = (1.5 + 0.015 * 65/2.5) * 1.648^2/2g = 0.262$ m

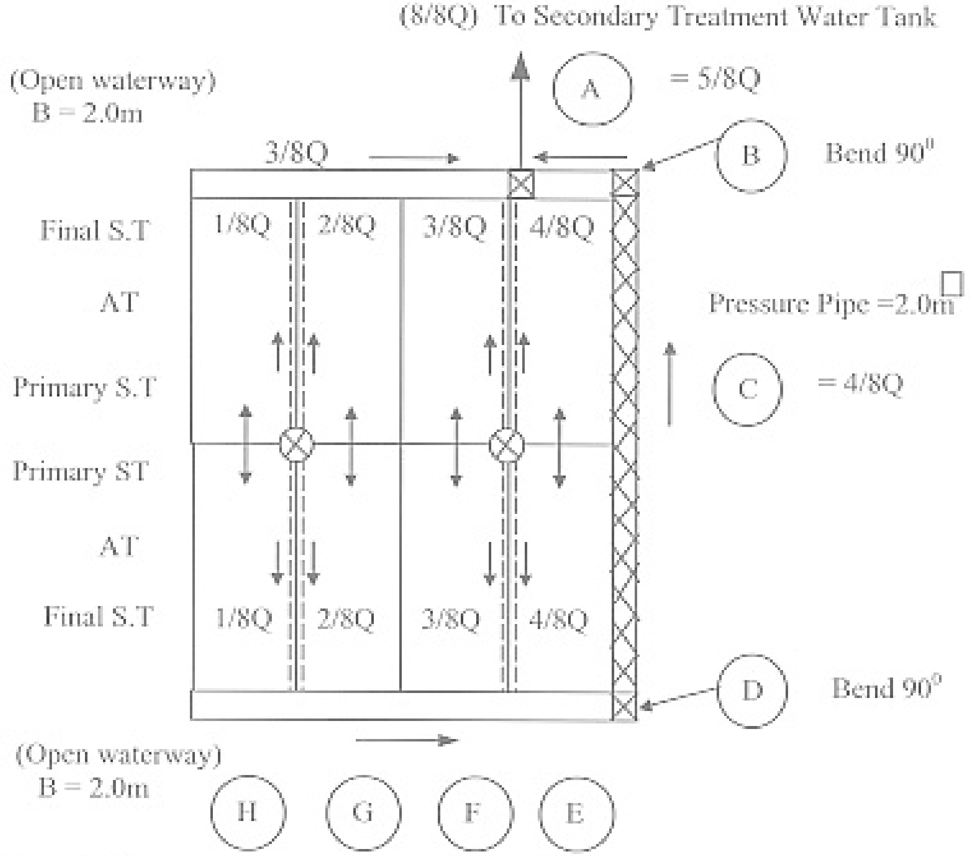
(Water Level)

WL(QmaxD) = + 2.950 + 0.140 + α = + 3.090 m

WL(QmaxH) = + 3.280 + 0.262 + α = + 3.550 m

3. 7 Final Sedimentation Tank, Outflow Waterway Head Loss (Final Sedimentation Tank, Outflow Waterway, Upstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Process of Calculation	Normally the Head Loss of waterway system is high, if all the waterway is designed by open waterway, therefore the combination system of pressure pipe and open waterway are used for designing. Head Loss is calculated by longest waterway for fail-safety.		



⊗ Distribution Tank (4 Division * 2 trains)

- - - - - - - - Bypass Waterway (Open), B = 1.5 m

(A) Division:	5/8 Q;	L □ 100 m (About)
(B) 90^0 Bend:	4/8 Q;	Pit
(C) Division:	4/8 Q;	L □ 185 m (About), Pressure Pipe
(D) 90^0 Bend:	4/8 Q;	Pit
(E) Division:	4/8 Q;	L □ 120 m (About)
(F) Division:	3/8 Q:	L □ 120 m (About)
(G) Division:	2/8 Q;	L □ 120 m (About)
(H) Division:	1/8 Q;	L □ 120 m (About)

(1) **Part (A) Critical Depth (Part (A) Downstream Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	-	5/8 Q	5/8 Q
Unit Flow	m^3/s	3.704	5.056
Downstream Water Level	m	**+3.090**	**+3.550**
Dimension of Waterway	-	$2.0\ m^W * 1$ (open)	
Critical Depth	m	0.727	0.895
Bottom Level	m	+ 2.700	
Water Level	m	**+3.430**	**+3.600**

NOTE:	$h = [1.10 * Q^2 /(g * B^2)]1/3$
	(Critical Depth) $h(QmaxD) = [1.10 * 3.704^2 /(9.8 * 2.0^2)]^{1/3} = 0.727\ m$ $h(QmaxH) = [1.10 * 5.056^2 /(9.8 * 2.0^2)]^{1/3} = 0.895\ m$ **(Water Level)** $WL(QmaxD) = + 2.700 + 0.727 + \alpha = + 3.430\ m$ $WL(QmaxH) = + 2.700 + 0.895 + \alpha = + 3.600\ m$

(2) **Part (A) Friction Head Loss (Part (A) Upstream Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	Number	5/8 Q	5/8 Q
Unit Flow	m^3/s	3.704	5.056
Downstream Water Level	m	**+ 3.430**	**+ 3.600**
Dimension of Waterway	-	$2.0\ m^{W} * 1$ (open)	
Length	m	About 100	
Water Depth	m	0.730	0.900
Velocity	m/s	2.537	2.809
Bottom Level	m	+ 2.700	
Friction Head Loss	m	0.344	0.361
Water Level	m	**+ 3.780**	**+ 3.970**

NOTE:	$V = Q/A$; $I = [(n * V)/R^{2/3}]^2$; $h = I * L$ $R = B * H /(B + 2H)$; $n = 0.013$
	(Velocity) V(QmaxD) = 3.704/(2.0 * 0.730) = 2.537 m/sec V(QmaxH) = 5.056/(2.0 * 0.900) = 2.809 m/sec **(Hydraulic Radius)** R(QmaxD) = 2.0 * 0.730/(2 + 2 * 0.730) = 0.422 m R(QmaxH) = 2.0 * 0.900/(2 + 2 * 0.900) = 0.474 m **(Hydraulic Slope)** $I(QmaxD) = [0.013 * 2.537)/0.422^{2/3}]^2 = 3.436 * 10^{-3}$ $I(QmaxH) = [0.013 * 2.809)/0.474^{2/3}]^2 = 3.608 * 10^{-3}$ **(Friction Head Loss)** $h\ (QmaxD) = 3.436 * 10^{-3} * 100 = 0.344$ m $h\ (QmaxH) = 3.608 * 10^{-3} * 100 = 0.361$ m **(Water Level)** $WL(QmaxD) = + 3.430 + 0.344 + \alpha = + 3.780$ m $WL(QmaxH) = + 3.600 + 0.361 + \alpha = + 3.970$ m

(3) **Part (B): 90^0 Bend Head Loss (Part (B) Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	-	4/8 Q	4/8 Q
Unit Flow	m^3/s	2.963	4.045
Downstream Water Level	m	+ 3.780	+ 3.970
Head Loss	m	0	0
Water Level	m	+ 3.780	+ 3.970

NOTE:	90^0 Bend Head Loss at pit "part (B)" is neglected, because a bottom elevation of the pit is designed to the same level as invert of Pressure Pipe. However Head Loss Inlet and Outlet, and Friction Loss of Pressure Pipe is considered.

(4) **Part (C): Pressure Conduit Pipe Head Loss (Part (C) Upstream Water Level)**

Item		Unit	QmaxD	QmaxH
Quantity		m^3/d	512,000	699,000
		m^3/s	5.926	8.090
Flow Rate		Number	4/8 Q	4/8 Q
Unit Flow		m^3/s	2.963	4.045
Downstream Water Level		m	**+3.780**	**+3.970**
Dimension of Pressure Pipe		-	$2.0\ m^{(W)}*2.0\ m^{(H)}*$ 1 (Pressure Conduit Pipe)	
Section Area		m^2	4.000	
Length		m	About 185	
Velocity		m/s	0.741	1.011
Head Loss	Inlet/Outlet Loss	m	0.042	0.078
	Straight Pipe Loss	m	0.029	0.055
	Total	m	0.071	0.133
Water Level		m	**+3.860**	**+4.110**

NOTE:

$h = 1.5 * V^2/2g$, $hf = (n^2 * V^2 * L)/R^{4/3}$
$R = B * H/(B+2H)$; $V = Q/A$; $n = 0.013$

(Section Area)
$A = 2.0 * 2.0 = 4.000\ m^2$

(Velocity)
$V(QmaxD) = 2.963/4.000 = 0.741$ m/s
$V(QmaxH) = 4.045/4.000 = 1.011$ m/s

(Hydraulic Radius)
$R = 2.0 * 2.0/(2.0+2*2.0) = 0.667$ m

(Inlet and Outlet Loss)
$h(QmaxD) = 1.5 * 0.741^2/2g = 0.042$ m
$h(QmaxH) = 1.5 * 1.011^2/2g = 0.078$ m

(Straight Loss)
$hf(QmaxD) = (0.013^2 * 0.741^2 * 185)/0.667^{4/3} = 0.029$ m
$hf(QmaxH) = (0.013^2 * 1.011^2 * 185)/0.667^{4/3} = 0.055$ m

(Water Level)
$WL(QmaxD) = +3.780 + 0.071 + \alpha = +3.860$ m
$WL(QmaxH) = +3.970 + 0.133 + \alpha = +4.110$ m

(5) **Part (D): 90^0 Bend Head Loss (Part (D) Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	-	4/8 Q	4/8 Q
Unit Flow	m^3/s	2.963	4.045
Downstream Water Level	m	**+3.860**	**+4.110**
Head Loss	m	0	0
Water Level	m	**+3.860**	**+4.110**

NOTE:	The Bend Loss will be neglected, because upstream of the bend is open Waterway. However Head Loss of Inlet, Outlet and Loss of Pressure Pipe is considered.

(6) **Part (E) Critical Depth (Part (E) Downstream Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	-	4/8 Q	4/8 Q
Unit Flow	m^3/s	2.963	4.045
Downstream Water Level	m	**+ 3.860**	**+ 4.110**
Dimension of Waterway	-	$2.0\ m^{W} * 1$ (open)	
Critical Depth	m	0.627	0.771
Bottom Level	m	+ 3.400	
Water Level	m	**+ 4.030**	**+ 4.180**

NOTE:	$h = [1.10 * Q^2 /(g * B^2)]^{1/3}$
	(Critical Depth) $h(QmaxD) = [1.10 * 2.963^2 /(9.8 * 2.0^2)]^{1/3} = 0.627$ m $h(QmaxH) = [1.10 * 4.045^2 /(9.8 * 2.0^2)]^{1/3} = 0.771$ m **(Water Level)** $WL(QmaxD) = + 3.400 + 0.627 + \alpha = + 4.030$ m $WL(QmaxH) = + 3.400 + 0.771 + \alpha = + 4.180$ m

(7) **Part (E) Friction Head Loss (Part (E) Upstream Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	Number	4/8 Q	4/8 Q
Unit Flow	m^3/s	2.963	4.045
Downstream Water Level	m	+4.030	+4.180
Dimension of Waterway	-	$2.0\ m^W * 1$ (open)	
Length	m	About 120	
Water Depth	m	0.630	0.780
Velocity	m/s	2.352	2.593
Bottom Level	m	+3.400	
Friction Head Loss	m	0.398	0.410
Water Level	m	+4.430	+4.590

NOTE:	$V = Q/A$; $I = [(n * V)/R^{2/3}]^2$; $h = I * L$ $R = B * H /(B + 2H)$; $n = 0.013$
	(Velocity) V(QmaxD) = 2.963/(2.0 * 0.630) = 2.352 m/s V(QmaxH) = 4.045/(2.0 * 0.780) = 2.593 m/s **(Hydraulic Radius)** R(QmaxD) = 2.0 * 0.630/(2 + 2 * 0.630) = 0.387 m R(QmaxH) = 2.0 * 0.780/(2 + 2 * 0.780) = 0.438 m **(Hydraulic Slope)** I(QmaxD) = $[0.013 * 2.352)/0.387^{2/3}]^2 = 3.315 * 10^{-3}$ I(QmaxH) = $[0.013 * 2.539)/0.438^{2/3}]^2 = 3.416 * 10^{-3}$ **(Friction Head Loss)** h (QmaxD) = $3.315 * 10^{-3} * 120 = 0.398$ m h (QmaxH) = $3.416 * 10^{-3} * 120 = 0.410$ m **(Water Level)** WL(QmaxD) = $+4.030 + 0.398 + \alpha = +4.430$ m WL(QmaxH) = $+4.180 + 0.410 + \alpha = +4.590$ m

(8) Part (F) Friction Head Loss (Part (F) Upstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	Number	3/8Q	3/8Q
Unit Flow	m^3/s	2.222	3.034
Downstream Water Level	m	+ 4.430	+ 4.590
Dimension of Waterway	-	$2.0\ m^W * 1$ (open)	
Length	m	About 120	
Water Depth	m	1.030	1.190
Velocity	m/s	1.079	1.275
Bottom Level	m	+ 3.400	
Friction Head Loss	m	0.058	0.074
Water Level	m	+ 4.490	+ 4.670

NOTE:	$V = Q/A$; $I = [(n*V)/R^{2/3}]^2$; $h = I*L$ $R = B*H/(B+2H)$; $n = 0.013$
	(Velocity) V(QmaxD) = 2.222/(2.0*1.030) = 1.079 m/s V(QmaxH) = 3.034/(2.0*1.190) = 1.275 m/s **(Hydraulic Radius)** R(QmaxD) = 2.0*1.030/(2+2*1.030) = 0.507 m R(QmaxH) = 2.0*1.190/(2+2*1.190) = 0.543 m **(Hydraulic Slope)** I(QmaxD) = $[0.013*1.079)/0.507^{2/3}]^2 = 4.867 * 10^{-4}$ I(QmaxH) = $[0.013*1.275)/0.543^{2/3}]^2 = 6.202 * 10^{-4}$ **(Friction Head Loss)** h(QmaxD) = $4.867 * 10^{-4} * 120 = 0.058$ m h(QmaxH) = $6.202 * 10^{-4} * 120 = 0.074$ m **(Water Level)** WL(QmaxD) = + 4.430 + 0.058 + α = + 4.490 m WL(QmaxH) = + 4.590 + 0.074 + α = + 4.670 m

(9) **Part (G) Friction Head Loss (Part (G) Upstream Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	Number	2/8Q	2/8Q
Unit Flow	m^3/s	1.482	2.023
Downstream Water Level	m	+4.490	+4.670
Dimension of Waterway	-	2.0 m ^{W} * (open)	
Length	m	About 120	
Water Depth	m	1.090	1.270
Velocity	m/s	0.680	0.796
Bottom Level	m	+3.400	
Friction Head Loss	m	0.022	0.028
Water Level	m	+4.520	+4.700

NOTE:	$V = Q/A$; $I = [(n*V)/R^{2/3}]^2$; $h = I*L$ $R = B*H/(B+2H)$; $n = 0.013$
	(Velocity) V(QmaxD) = 1.482/(2.0*1.090) = 0.680 m/s V(QmaxH) = 2.023/(2.0*1.270) = 0.796 m/s **(Hydraulic Radius)** R(QmaxD) = 2.0*1.090/(2+2*1.090) = 0.522 m R(QmaxH) = 2.0*1.270/(2+2*1.270) = 0.559 m **(Hydraulic Slope)** $I(QmaxD) = [0.013*0.680)/0.522^{2/3}]^2 = 1.859*10^{-4}$ $I(QmaxH) = [0.013*0.796)/0.559^{2/3}]^2 = 2.325*10^{-4}$ **(Friction Head Loss)** $h(QmaxD) = 1.859*10^{-4}*120 = 0.022$ m $h(QmaxD) = 2.325*10^{-4}*120 = 0.028$ m **(Water Level)** $WL(QmaxD) = +4.490 + 0.022 + \alpha = +4.520$ m $WL(QmaxH) = +4.670 + 0.028 + \alpha = +4.700$ m

(10) Part (II) – Friction Head Loss (Part (II) - Upstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Flow Rate	Number	1/8 Q	1/8 Q
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	**+ 4.520**	**+ 4.700**
Dimension of Waterway	-	2.0 m ^{W} * 1 (open)	
Length	m	About 120	
Water Depth	m	1.120	1.300
Velocity	m/s	0.331	0.389
Bottom Level	m	+ 3.400	
Friction Head Loss	m	0.005	0.007
Water Level	m	**+ 4.530**	**+ 4.710**

NOTE:

$V = Q/A$; $I = [(n * V)/R^{2/3}]^2$; $h = I * L$
$R = B * H/(B+2H)$; $n = 0.013$

(Velocity)
V(QmaxD) = 0.741/(2.0*1.120) = 0.331 m/s
V(QmaxH) = 1.011/(2.0*1.300) = 0.389 m/s

(Hydraulic Radius)
R(QmaxD) = 2.0 * 1.120/(2.0 + 2 * 1.120) = 0.528 m
R(QmaxH) = 2.0 * 1.300/(2.0 + 2 * 1.300) = 0.565 m

(Hydraulic Slope)
$I(QmaxD) = [0.013*0.331)/0.528^{2/3}]^2 = 4.339 *10^{-5}$
$I(QmaxH) = [0.013*0.389)/0.565^{2/3}]^2 = 5.475 *10^{-5}$

(Friction Head Loss)
$h(QmaxD) = 4.339 * 10^{-5} * 120 = 0.005$ m
$h(QmaxH) = 5.475 * 10^{-5} * 120 = 0.007$ m

(Water Level)
$WL(QmaxD) = + 4.520 + 0.005 + \alpha = + 4.530$ m
$WL(QmaxH) = + 4.700 + 0.007 + \alpha = + 4.710$ m

3.8 Final Sedimentation Tank, Outflow Trough Head Loss (Trough Upstream Water Level): Free Overflow

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	20*8*2=320	320
Unit Flow	m^3/s	0.019	0.025
Downstream Water Level	m	+ 4.530	+ 4.710
Trough Width	m	0.500	
Trough Bottom Level	m	+ 4.800	
Trough Bottom Slope	%	i = 0	
Trough Head Loss	m	0.095	0.113
Water Level	m	+ 4.900	+ 4.920

NOTE:	$h_c = [1.10 * Q^2/(g * B^2)]^{1/3}$, $h_o = \sqrt{3} * h_c$ (Free Overflow)
	(Critical Depth) h_c (QmaxD) = $[1.10 * 0.019^2/(9.8 * 0.5^2)]^{1/3} = 0.055$ m h_c (QmaxH) = $[1.10 * 0.025^2/(9.8 * 0.5^2)]^{1/3} = 0.065$ m **(Trough Head Loss)** h_o(QmaxD) = $\sqrt{3} * 0.055 = 0.095$ m h_o(QmaxH) = $\sqrt{3} * 0.065 = 0.113$ m **(Water Level)** WL(QmaxD) = + 4.800 + 0.095 + α = + 4.900 m WL(QmaxH) = + 4.800 + 0.113 + α = + 4.920 m

3.9 Final Sedimentation Tank, Triangular Notch Head Loss (Final Sedimentation Tank, Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	20*8=160	160
Unit Flow	m^3/s	0.037	0.051
Downstream Water Level	m	**+ 4.900**	**+ 4.920**
Triangular Notch Form	-	Wear Plate (Triangular Weir)	
Notch Degree	No/m	7 (Trough L = 27.0 m/waterway)	
Number of Notch	Number/waterway	7 * 27.0 = 189	
Bottom Level	m	+ 5.000	
Overflow Depth	m	0.029	0.032
Water Level	m	**+ 5.030**	**+ 5.040**

NOTE:

$h = (q/1.42)^{2/5}$

(Quantity per Notch)

V(QmaxD) = 0.037/189 = 1.958 * 10^{-4} m^3/s

V(QmaxH) = 0.051/189 = 2.698 * 10^{-4} m^3/s

(Overflow Depth)

h(QmaxD) = $[(1.958 * 10^{-4}) / 1.42]^{2/5}$ = 0.029 m

h(QmaxH) = $[(2.698 * 10^{-4}) / 1.42]^{2/5}$ = 0.032 m

(Water Level)

WL(QmaxD) = + 5.000 + 0.029 + α = + 5.030 m

WL(QmaxH) = + 5.000 + 0.032 + α = + 5.040 m

(Final Sedimentation Tank Bottom Level) H = 3.5 m

BL = + 5.030 – 3.500 = + 1.530 (most shallow point)

3.10 Final Sedimentation Tank, Inflow Gate Head Loss (Final Sedimentation Tank, Upstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	1,024,000	1,211,000
	m^3/s	11.852	14.016
Establishment Number	Number	10*8=80	80
Unit Flow	m^3/s	0.148	0.175
Downstream Water Level	m	**+ 5.030**	**+ 5.040**
Dimension of Gate	-	500mm$^{(W)}$ * 500mm$^{(H)}$ * 80 pcs	
Velocity	m/s	0.592	0.700
Gate Head Loss	m	0.027	0.038
Water Level	m	**+ 5.060**	**+ 5.080**

NOTE:	**Return Sludge** = QmaxD * 100% (Max) $h = 1.5 * V^2/2g$, $V = Q/A$
	(Velocity) V(QmaxD) = 0.148/(0.5 * 0.5) = 0.592 m/s V(QmaxH) = 0.175/(0.5 * 0.5) = 0.700 m/s **(Gate Head Loss)** h(QmaxD) = 1.5 * $0.592^2/2g$ = 0.027 m h(QmaxH) = 1.5 * $0.700^2/2g$ = 0.038 m **(Water Level)** WL(QmaxD) = + 5.030 + 0.027 + α = + 5.060 m WL(QmaxH) = + 5.040 + 0.038 + α = + 5.080 m

3.11 Aeration Tank, Overflow Weir Head Loss (Aeration Tank, Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	1,024,000	1,211,000
	m^3/s	11.852	14.016
Establishment Number	Number	10*8=80	80
Unit Flow	m^3/s	0.148	0.175
Downstream Water Level	m	**+ 5.060**	**+ 5.080**
Overflow Weir Form	-	Suppressed Rectangle Weir, B = 8.25m	
Weir Level	m	+ 5.200	
Overflow Depth	m	0.046	0.051
Water Level	m	**+ 5.250**	**+ 5.260**

NOTE:	$h = [Q/(1.84 * B)]^{2/3}$
	(Overflow Depth) $h(QmaxD) = [0.148/(1.84 * 8.25)]^{2/3} = 0.046$ m $h(QmaxH) = [0.175/(1.84 * 8.25)]^{2/3} = 0.051$ m **(Water Level)** $WL(QmaxD) = + 5.200 + 0.046 + \alpha = + 5.250$ m $WL(QmaxH) = + 5.200 + 0.051 + \alpha = + 5.260$ m **(Aeration Tank, Bottom Level)** H = 5.5 m BL = + 5.250 – 5.500 = - 0.250 m

3.12 Aeration Tank, Inflow Movable Weir Head Loss (Aeration Tank, Inflow Waterway Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	10*8=80	80
Unit Flow	m^3/s	0.074	0.101
Downstream Water Level	m	**+ 5.250**	**+ 5.260**
Dimension of Inflow Movable Weir	-	Movable Weir, B = 500mm	
Weir Level	m	+ 5.500	
Overflow Depth	m	0.186	0.229
Water Level	m	**+ 5.690**	**+ 5.730**

NOTE:	$h = [Q/(1.84 * B)]^{2/3}$
	(Overflow Depth) $h(QmaxD) = [0.074/(1.84 * 0.5)]^{2/3} = 0.186$ m $h(QmaxH) = [0.101/(1.84 * 0.5)]^{2/3} = 0.229$ m **(Water Level)** $WL(QmaxD) = + 5.500 + 0.186 + \alpha = + 5.690$ m $WL(QmaxH) = + 5.500 + 0.229 + \alpha = + 5.730$ m

3.13 Aeration Tank, Step Movable Weir Head Loss (Aeration Tank, Step Waterway Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	80*4=320	320
Unit Flow	m^3/s	0.019	0.025
Downstream Water Level	m	**+ 5.250**	**+ 5.260**
Dimension of Step Movable Weir	-	Movable Weir, B = 500mm * 4 places/tank (4 Point Step)	
Weir Level	m	+ 5.500	
Overflow Depth	m	0.075	0.090
Water Level	m	**+ 5.580**	**+ 5.590**

NOTE:	$h = [Q/(1.84 * B)]^{2/3}$
	(Overflow Depth) $h(QmaxD) = [0.019/(1.84 * 0.5)]^{2/3} = 0.075$ m $h(QmaxH) = [0.025/(1.84 * 0.5)]^{2/3} = 0.090$ m **(Water Level)** $WL(QmaxD) = + 5.500 + 0.075 + \alpha = + 5.580$ m $WL(QmaxH) = + 5.500 + 0.090 + \alpha = + 5.590$ m

3.14 Primary Sedimentation Tank, Outflow Trough Head Loss (Primary Sedimentation Tank, Outflow Trough Upstream Water Level): Free Overflow

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	20*8*2=320	320
Unit Flow	m^3/s	0.019	0.025
Downstream Water Level	m	**+ 5.690**	**+ 5.730**
Trough Width	m	0.500	
Trough Bottom Level	m	+ 5.800	
Trough Bottom Slope	%	i = 0	
Critical Depth & Trough Head Loss	m	0.094	0.113
Water Level	m	**+ 5.900**	**+ 5.920**

NOTE:	$h = \sqrt{3} * [1.10 * Q^2/(g * B^2)]^{1/3}$ = Free Overflow
	(Critical Depth and Trough Head Loss) $h_o(QmaxD) = \sqrt{3} * [1.10 * 0.019^2/(9.8 * 0.5^2)]^{1/3} = 0.094$ m $h_o(QmaxH) = \sqrt{3} * [1.10 * 0.025^2/(9.8 * 0.5^2)]^{1/3} = 0.113$ m **(Water Level)** WL(QmaxD) = + 5.800 + 0.094 + α = + 5.900 m WL(QmaxH) = + 5.800 + 0.113 + α = + 5.920 m

3.15 **Primary Sedimentation Tank Triangular Notch Head Loss (Primary Sedimentation Tank Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	20*8=160	160
Unit Flow	m^3/s	0.037	0.051
Downstream Water Level	m	**+ 5.900**	**+ 5.920**
Triangular Notch Form	-	Wear Plate (Triangular Weir)	
Notch Degree	Pitch/m	7 (Trough. L = 13.0m/waterway)	
Number of Notch	Number/waterway	7 * 13.0 = 91	
Bottom Level	m	+ 6.000	
Overflow Depth	m	0.038	0.043
Water Level	m	**+ 6.040**	**+ 6.050**

NOTE:

$h = (q/1.42)^{2/5}$

(Overflow Rate per Notch)
$q(QmaxD) = 0.037/91 = 4.066 * 10^{-4}\ m^3/s$
$q(QmaxH) = 0.051/91 = 5.604 * 10^{-4}\ m^3/s$

(Overflow Depth)
$h(QmaxD) = (4.066 * 10^{-4}/1.42)^{2/5} = 0.038\ m$
$h(QmaxH) = (5.604 * 10^{-4}/1.42)^{2/5} = 0.043\ m$

(Water Level)
$WL(QmaxD) = + 6.000 + 0.038 + \alpha = + 6.040\ m$
$WL(QmaxH) = + 6.000 + 0.043 + \alpha = + 6.050\ m$

(Primary Sedimentation Tank Bottom Level) H = 3.0 m
BL = + 6.040 – 3.000 = + 3.040 m (most shallow point)

3.16 Primary Sedimentation Tank, Inflow Movable Weir Head Loss (Primary Sedimentation Tank, Upstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Number	Number	10*8=80	80
Unit Flow	m^3/s	0.074	0.101
Downstream Water Level	m	**+ 6.040**	**+ 6.050**
Dimension of Inflow Movable Weir	-	Movable Weir, B = 500mm	
Weir Level	m	+ 6.200	
Overflow Depth	m	0.186	0.229
Water Level	m	**+ 6.390**	**+ 6.430**

NOTE:	$h = [Q/(1.84 * B)]^{2/3}$
	(Overflow Depth) $h(QmaxD) = [0.074/(1.84 * 0.5)]^{2/3} = 0.186$ m $h(QmaxH) = [0.101/(1.84 * 0.5)]^{2/3} = 0.229$ m **(Water Level)** $WL(QmaxD) = + 6.200 + 0.186 + \alpha = + 6.390$ m $WL(QmaxH) = + 6.200 + 0.229 + \alpha = + 6.430$ m

3.17 Bypass Waterway Critical Depth
(Part of Final Sedimentation Tank, Downstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	8	8
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	**+ 4.530**	**+ 4.710**
F.S.T. Waterway Bottom Level	m	+ 3.400	
Dimension of Waterway	-	1.5 m ^{W} * 1 (open)/train	
Critical Depth	m	0.301	0.371
Bypass Waterway Bottom Level	m	+ 4.700	
Water Level	m	**+ 5.010**	**+ 5.080**

NOTE:	$h = [(1.10*Q^2)/(g * B^2)]^{1/3}$;
	(Critical Depth) $h(QmaxD) = [1.10 * 0.741^2/9.8*1.5^2)]^{1/3} = 0.301$ m $h(QmaxH) = [1.10 * 1.011^2/9.8*1.5^2)]^{1/3} = 0.371$ m **(Water Level)** $WL(QmaxD) = +4.700 + 0.301 + \alpha = +5.010$ m $WL(QmaxH) = +4.700 + 0.371 + \alpha = +5.080$ m

3.18 **Bypass Waterway Friction Head Loss (Part of Final Sedimentation Tank, Upstream Water Level)**

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	8	8
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	**+5.010**	**+5.080**
Dimension of Waterway	-	1.5 m ^{W} * 1 (open)/train	
Length	m	About 30	
Water Depth	m	0.310	0.380
Velocity	m/s	1.594	1.774
Bottom Level	m	+4.700	
Friction Head Loss	m	0.098	0.100
Water Level	m	**+5.110**	**+5.180**

NOTE:

$V = Q/A$; $I = [(n * V)/R^{2/3}]^2$; $h = I * L$
$R = B*H/(B+2H)$; $n = 0.013$

(Velocity)
V(QmaxD) = 0.741/(1.5*0.310) = 1.594 m/s
V(QmaxH) = 1.011/(1.5*0.380) = 1.774 m/s

(Hydraulic Radius)
R(QmaxD) = 1.5*0.310/(1.5+2*0.310) = 0.219 m
R(QmaxH) = 1.5*0.380/(1.5+2*0.380) = 0.252 m

(Hydraulic Slope)
I(QmaxD) = $[0.013*1.594)/0.219^{2/3}]^2 = 3.253 * 10^{-3}$
I(QmaxH) = $[0.013*1.774)/0.252^{2/3}]^2 = 3.341 * 10^{-3}$

(Friction Head Loss)
h(QmaxD) = $3.253 * 10^{-3} * 30 = 0.098$ m
h(QmaxH) = $3.341 * 10^{-3} * 30 = 0.100$ m

(Water Level)
WL(QmaxD) = $+5.010 + 0.098 + \alpha = +5.110$ m
WL(QmaxH) = $+5.080 + 0.100 + \alpha = +5.180$ m

3.19 Bypass Waterway, Critical Depth
(Part of Aeration Tank, Downstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	8	8
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	**+ 5.110**	**+ 5.180**
Dimension of Waterway	-	1.5 m ^{w} * 1 (open)/ train	
Critical Depth	m	0.301	0.371
Bypass Waterway Bottom Level	m	+ 5.200	
Water Level	m	**+ 5.510**	**+ 5.580**

NOTE:	$h = [(1.10*Q^2)/(g * B^2)]^{1/3}$;
	(Critical Depth) $h(QmaxD) = [1.10 * 0.741^2/9.8*1.5^2)]^{1/3} = 0.301$ m $h(QmaxH) = [1.10 * 1.011^2/9.8*1.5^2)]^{1/3} = 0.371$ m **(Water Level)** WL(QmaxD) = + 5.200 + 0.301 + α = + 5.510 m WL(QmaxH) = + 5.200 + 0.371 + α = + 5.580 m

3.20 Bypass Waterway Friction Head Loss (Part of Aeration Tank, Upstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	8	8
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	**+ 5.510**	**+ 5.580**
Dimension of Waterway	-	1.5 m ^{W} * 1 (open)/ train	
Length	m	About 30	
Water Depth	m	0.310	0.380
Velocity	m/s	1.594	1.774
Bottom Level	m	+ 5.200	
Friction Head Loss	m	0.098	0.100
Water Level	m	**+ 5.610**	**+ 5.680**

NOTE:

$V = Q/A$; $I = [(n * V)/R^{2/3}]^2$; $h = I * L$

$R = B * H/(B+2H)$; $n = 0.013$

(Velocity)

V (QmaxD) = 0.741/(1.5*0.310) = 1.594 m/s

V (QmaxH) = 1.011/(1.5*0.380) = 1.774 m/s

(Hydraulic Radius)

R(QmaxD) = 1.5*0.310/(1.5+2*0.310) = 0.219 m

R(QmaxH) = 1.5*0.380/(1.5+2*0.380) = 0.252 m

(Hydraulic Slope)

I(QmaxD) = $[0.013*1.594)/0.219^{2/3}]^2 = 3.253 * 10^{-3}$

I(QmaxH) = $[0.013*1.774)/0.252^{2/3}]^2 = 3.341 * 10^{-3}$

(Friction Head Loss)

h(QmaxD) = $3.253 * 10^{-3} * 30 = 0.098$ m

h(QmaxH) = $3.341 * 10^{-3} * 30 = 0.100$ m

(Water Level)

WL(QmaxD) = $+ 5.510 + 0.098 + \alpha = + 5.610$ m

WL(QmaxH) = $+ 5.580 + 0.100 + \alpha = + 5.680$ m

3.21 Bypass Waterway Critical Depth
(Part of Primary Sedimentation Tank, Downstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	8	8
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	**+ 5.610**	**+ 5.680**
Dimension of Waterway	-	1.5 m ^{W} * 1 (open)/ train	
Critical Depth	m	0.301	0.371
Bypass Waterway Bottom Level	m	+ 5.700	
Water Level	m	**+ 6.010**	**+ 6.080**

NOTE:	$h = [(1.10*Q^2)/(g * B^2)]^{1/3}$;
	(Critical Depth) $h(QmaxD) = [1.10 * 0.741^2/9.8*1.5^2)]^{1/3} = 0.301$ m $h(QmaxH) = [1.10 * 1.011^2/9.8*1.5^2)]^{1/3} = 0.371$ m **(Water Level)** WL(QmaxD) = + 5.700 + 0.301 + α = + 6.010 m WL(QmaxH) = + 5.700 + 0.371 + α = + 6.080 m

3.22 Bypass Waterway, Friction Head Loss
(Part of Primary Sedimentation Tank, Upstream Water Level)

Item	Unit	QmaxD	Hmax
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	8	8
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	**+ 6.010**	**+ 6.080**
Dimension of Waterway	-	1.5 m " * 1 (open)/ train	
Length	m	About 15	
Water Depth	m	0.310	0.380
Velocity	m/s	1.594	1.774
Bottom Level	m	+ 5.700	
Friction Head Loss	m	0.049	0.050
Water Level	m	**+ 6.060**	**+ 6.130**

NOTE:

$V = Q/A$; $I = [(n * V)/R^{2/3}]^2$; $h = I * L$

$R = B * H /(B+2H)$; $n = 0.013$

(Velocity)

V (QmaxD) = 0.741/(1.5*0.310) = 1.594 m/s

V (QmaxH) = 1.011/(1.5*0.380) = 1.774 m/s

(Hydraulic Radius)

R (QmaxD) = 1.5*0.310/(1.5+2*0.310) = 0.219 m

R (QmaxH) = 1.5*0.380/(1.5+2*0.380) = 0.252 m

(Hydraulic Slope)

I (QmaxD) = $[0.013*1.594)/0.219^{2/3}]^2 = 3.253 *10^{-3}$

I (QmaxH) = $[0.013*1.774)/0.252^{2/3}]^2 = 3.341 *10^{-3}$

(Friction Head Loss)

h(QmaxD) = $3.253 * 10^{-3} * 15 = 0.049$ m

h(QmaxH) = $3.341 * 10^{-3} * 15 = 0.050$ m

(Water Level)

WL(QmaxD) = $+ 6.010 + 0.049 + \alpha = + 6.060$ m

WL(QmaxH) = $+ 6.080 + 0.050 + \alpha = + 6.130$ m

3.23 Primary Sedimentation Tank, Inflow Waterway Friction Head Loss (Primary Sedimentation Tank, Inflow Waterway Upstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	8	8
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	**+ 6.390**	**+ 6.430**
Dimension of Waterway	-	1.0 m ^{W} * 1 (open) / train	
Length	m	About 115	
Water Depth	m	0.590	0.630
Velocity	m/s	1.256	1.605
Bottom Level	m	+ 5.800	
Friction Head Loss	m	0.175	0.275
Water Level	m	**+ 6.570**	**+ 6.710**

NOTE:

$I = [(n * V)/R^{2/3}]^2$; $h = I * L$; $V = Q/A$

$R = B * H/(B+2H)$; $n = 0.013$

(Velocity)

V(QmaxD) = 0.741/(1.0*0.590) = 1.256 m/s

V(QmaxH) = 1.011/(1.0*0.630) = 1.605 m/s

(Hydraulic Radius)

R(QmaxD) = 1.0*0.590/(1.0+2*0.590) = 0.271 m

R(QmaxH) = 1.0*0.630/(1.0+2*0.630) = 0.279 m

(Hydraulic Slope)

I (QmaxD) = $[0.013*1.256)/0.271^{2/3}]^2 = 1.520 * 10^{-3}$

I (QmaxH) = $[0.013*1.605)/0.279^{2/3}]^2 = 2.388 * 10^{-3}$

(Friction Head Loss)

h(QmaxD) = $1.520 * 10^{-3} * 115 = 0.175$ m

h(QmaxH) = $2.388 * 10^{-3} * 115 = 0.275$ m

(Water Level)

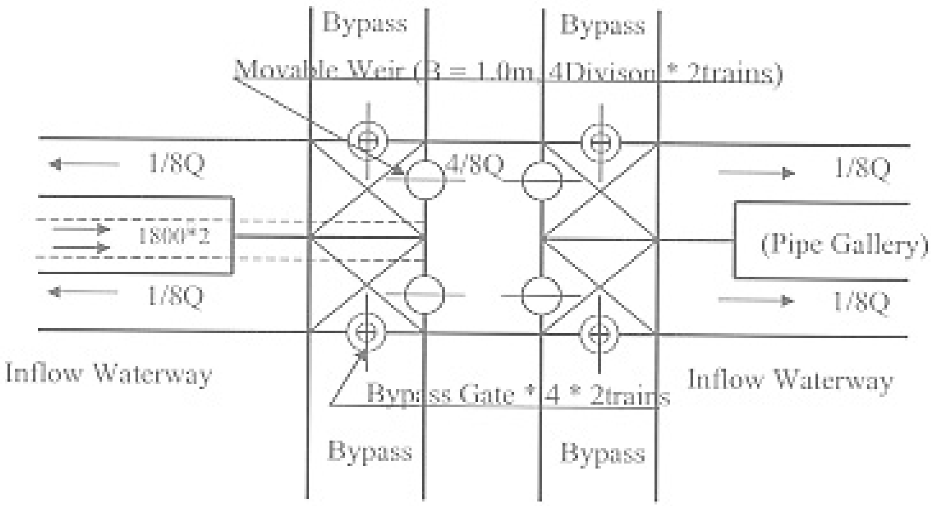
WL(QmaxD) = + 6.390 + 0.175 + α = + 6.570 m

WL(QmaxH) = + 6.430 + 0.275 + α = + 6.710 m

3.24. Distribution Tank Movable Weir Head Loss (Distribution tank, Upstream Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	8	8
Unit Flow	m^3/s	0.741	1.011
Downstream Water Level	m	+ 6.570	+ 6.710
Weir Form	-	Movable Weir B = 1.0m (4 Division * 2 Trains)	
Weir Level	m	+ 6.900	
Overflow Depth	m	0.545	0.671
Water Level	m	+ 7.450	+ 7.580

Distribution Tank Plan



NOTE:	$h = [Q/(1.84 * B)]^{2/3}$
	(Overflow Depth) $h(QmaxD) = [0.741/1.84*1.0)]^{2/3} = 0.545$ m $h(QmaxH) = [1.011/1.84*1.0)]^{2/3} = 0.671$ m **(Water Level)** $WL(QmaxD) = + 6.900 + 0.545 + \alpha = + 7.450$ m $WL(QmaxH) = + 6.900 + 0.671 + \alpha = + 7.580$ m

3.25. Distribution Tank – Discharge Tank Connection Pipe Head Loss (Discharge Tank Water Level)

Item	Unit	QmaxD	QmaxH
Quantity	m^3/d	512,000	699,000
	m^3/s	5.926	8.090
Establishment Number	Number	2	2
Unit Flow	m^3/s	2.963	4.045
Downstream Water Level	m	+ 7.450	+ 7.580
Dimension of Connection Pipe	-	1,800 mm * 2 trains	
Section Area	m^2	3.240 / train	
Length and Bend	m	About 300, 90° bend * 1	
Velocity	m/s	0.915	1.248
Connection Pipe Head Loss	m	0.084	0.156
Bend/Inlet/Outlet Loss	m	0.107	0.199
Total	m	0.191	0.355
Water Level	m	+ 7.650	+ 7.940

NOTE:

$h = (n^2 * V^2 * L)/R^{4/3}$; $R = B * H / (B + 2H)$, $n = 0.013$

$h = 1.0 * V^2/2g * n$; $n = 1$ (90° Bend); $V = Q/A$, $h = 1.5 * V^2/2g$

(Section Area)

$A = 1.8 * 1.8 = 3.240\ m^2$ / train

(Velocity)

V(QmaxD) = 2.963/3.240 = 0.915 m/s

V(QmaxH) = 4.045/3.240 = 1.248 m/s

(Hydraulic Radius)

R = 1.8 * 1.8/(1.8 + 2*1.8) = 0.600 m

(Connected Pipe Head Loss)

h(QmaxD) = $(0.013^2 * 0.915^2 * 300) / 0.600^{4/3} = 0.084$ m

h(QmaxH) = $(0.013^2 * 1.248^2 * 300) / 0.600^{4/3} = 0.156$ m

(Bend Loss) + (Inlet Loss) + (Outlet Loss)

h(QmaxD) = $2.5 * 0.915^2 / 2g * 1 = 0.107$ m

h(QmaxH) = $2.5 * 1.248^2 / 2g * 1 = 0.199$ m

(Water Level)

WL(QmaxD) = $+ 7.450 + 0.191 + \alpha = + 7.650$ m

WL(QmaxH) = $+ 7.580 + 0.355 + \alpha = + 7.940$ m