AP - I

SEWAGE TREATMENT FACILITIES – DESIGN AND DRAWINGS

AP – I1

SMALL SEWAGE TREATMENT SYSTEMS (SSTS) FOR 80 PE

1. 0 DESIGN DATA

Population Equivalent	=	80 PE (maximum allowed PE)
Waste Water Flow Per Person	=	0.225 m^3/ day. PE (or 225L / day)
Average Waste Water flow	=	18.00 m^3/day = 0.21 x 10^{-3} m^3/sec
Peak Flow Factor (p)	=	4.7 x (80 ÷ 1000) $^{-0.11}$
	=	6.21
Peak Hourly Flow (Q_p)	=	111.69 m^3/day = 1.29 x 10^{-3} m^3/sec
Type of waste	=	Domestic
Influent BOD_5	=	250 ppm
Influent SS	=	300 ppm
pH	=	5-8
Influent BOD Loading	=	4.50 kg/day
Influent SS Loading	=	5.40 kg/day

2. 0 DESIGN OF RAW SEWAGE PUMP SUMP

2.1 Raw Sewage Pump Capacity

Effective Volume of Sump (wet well)

Where V	=	TQ ÷ 4
Q	=	Peak Flow = 111.69 m^3/day
T	=	Cycle time (6)
V	=	(6 x 111.694) ÷ (4x24x60) = 0.116 m^3

Provide wet well size of 1.50 m diameter

Required effective depth	=	0.116 m^3 ÷ π $(1.5 / 2)^2$
	=	0.066 ≈ 0.45 m (provided)

2.2 Pipe Sizing

Pump capacity at peak flow	=	1.29 x 10^{-3} m^3 /sec
	=	1.29 L/ sec

Desirable velocity in the discharge pipe at maximum pump discharge ≤ 2.5 m/sec (MS 1228).
Adopt velocity in discharge pipe as 2.0 m/sec

Diameter, D $= \sqrt{\dfrac{1.29 \times 10^{-3}\ m^3/sec \times 4}{\Pi \times 2.0\ m/sec}}$

	=	0.029 m
Provided pipe diameter	=	100 mm (DI material or equivalent)
Actual mean velocity, V	=	$\dfrac{1.29 \times 10^{-3}\ m^3/sec \times 4}{\Pi \times 0.10\ m \times 0.10\ m}$
	=	0.165 m/sec
Other losses	=	pipe + fitting

Losses through the pipe hf (Hazen – William Formula)

V = $0.85 \times C \times R^{0.63} \times (hf/L)^{0.54}$

Losses through the fitting He, (Hazen – William Formula)

He = $K \times V^2 \div 2g$

V = Mean velocity, = 0.165 m/sec

C	=	Hazen – William Coefficient for pipe = 90 for sewage
R	=	Hydraulic Radius = 0.025 m
L	=	Length of pipe = 4.00 m
K	=	Resistance coefficient for fitting & valve = 0.60
g	=	Acceleration due to gravity = 9.81 m/sec
$(h_{f/L})^{0.54}$	=	$\frac{0.165 \text{ m / sec}}{\{0.85 \times 90 \times (0.025)^{0.63}\}}$
$(h_{f/L})$	=	$(0.0220)^{1.85}$
h_f	=	0.0009 x 4.00 m = 0.003 m
He	=	$0.60 \times (0.165)^2 \div (2 \times 9.81)$
	=	0.0008 m / per unit of fitting
Total head	=	static head + other losses
	=	3.00 m +0.003 m +0.0008 m
	=	3.0043 m
Provided total head	=	6.00 m

Provided 2 numbers of pumps, 1 working and 1 standby

∴ Each pump capacity = 5.75 L /s at 6.00 m total head = 0.345 m^3/ min

2.3 Raw Sewage Pump Specification

Type	=	EBARA
Model	=	80 DVS 5.75 T
Power	=	0.75 kW
Total Head	=	6.00 m
Capacity	=	5.75 L/s > 1.29 L/s …… OK

Provided effective volume from 1st pump

Start level to pump stop level	=	$\Pi\ (1.5/2)^2 \times 0.45$ m(D)
	=	0.795 m^3 > 0.116 m^3
Retention time / time to fill	=	0.795 m^3 ÷ 0.013 m^3/ min
	=	63.63 min
Time to empty	=	0.795 m^3 ÷ (0.345 – 0.013) m^3/min
	=	2.39 min
Actual pump cycle at ADWF	=	63.63 min +2.39 min
	=	66.02 min
No. of start / stop	=	60 +66.02 min - 0.91
	=	1 nos.

Note: The pumping cycle 6-15 start /stop per hour cannot be complied due to DGSS requirement; minimum dimension of pump is 1.5 mϕ and minimum cut off level is 450mm.

GOODWATER SYSTEM MODEL GWQ SA50

3.0 DESIGN DATA

DESIGN Process	=	Conventional Activated Sludge Process
Population Equivalent/ Units	=	80 PE (maximum allowed PE)
Waste Water Flow Per Person	=	0.225 m^3/ day. PE (or 225lpcd)
Average Waste Water flow	=	18.00 m^3/day
	=	0.21×10^{-3} m^3/sec

Peak Flow Factor (p)	=	$4.7 \times (80 \div 1000)^{-0.11}$
	=	6.21
Peak Flow (Q_p)	=	111.69 m^3/day
	=	1.29×10^{-3} m^3/sec
Type of waste	=	Domestic
Influent BOD_5	=	250 ppm
Influent SS	=	300 ppm
Efluent BOD_5	≤	10 ppm
Efluent SS	≤	20 ppm

EQA BOD and SS value for Standard A = BOD 20 ppm an SS 50 ppm

Note: The following are not the discharged into the system; chemical of toxic/ corrosive nature, radio active products, metal, oil and grease.

4.0 PRIMARY SEDIMENTATION TANK

Retention Time	=	2.00 hrs
Required Volume V	=	(Q_{peak} x Retention time) ÷ 24
	=	9.31m^3

a) Tank Specification (Refer Appendix 1)

Tank Diameter	=	2.40m	
Tank Radius	=	1.20m	
Tank Height	=	2.40m	
Provided Volume	=	10.86m^3 > 9.31m^3…. OK	*…ok*
Check for HRT	=	2.33 hrs > 2.00 hrs…OK	*…ok*

5.0 AERATION TANK

a) Design Parameter based on DGSS Guideline

a.	F/M Ratio	=	0.25 0.50
b.	HRT/hr.	=	6-16
c.	Sludge Age/day (Q_c)	=	5-10
d.	MLSS/ (mg/l)	=	1500-3000
e.	MLVSS	=	0.80 MLSS
f.	Oxygen Requirement	=	1.50 kgO_2/ $kgBOD_5$

b) Tank Volume (Required)

$V_R = (Q_A \times 8) \div 24$	=	(18.00 x 8) ÷ 24
	=	6.00m^3

c) Tank Specification (Refer Appendix 1)

Tank Radius	=	1.60m	
Tank Height	=	2.35m	
Volume of the provided tank	=	7.53m^3 > 6.00m^3	*…ok*

Check the provided HRT	=	$(7.53 \div 18.00) \times 24$	
(Hydraulic Retention Time)	=	10.04 hrs (6-16 hrs)	*…ok*

d) **Aeration Rate**

Provided Air Rate	=	1.50 kgO_2/BOD_5 removed
Amount of BOD to be removed	=	0.25 kg BOD/m^3 – 0.01 kg BOD/m^3
	=	0.24 kg BOD/m^3 of sewage
Total BOD_5 per day	=	18.00 m^3/day x 0.24 kg BOD/m^3
	=	4.32 kg BOD/day
Amount of O_2 to supply (AOR)	=	4.32 kg BOD/day x 1.50 kg O_2/BOD
	=	6.48 kg O_2/day

e) **Field Conditions**

N (Field)	=	$N_S (C_S - C_L) \div 9.17 \times (1.024)^{T-20} \times \alpha$
N	=	Oxygen transferred in field condition kg O_2/hr
NS	=	Oxygen transfer under standard condition kg O_2/hr
CS	=	Dissolve oxygen saturation value at 25°C :- 8.38 mg/l
CL	=	1-2 mg/l in mixed liquor (assume 2.00 mg/l)
T	=	Temperature (°C), 25°C
α	=	Correlation Factor (0.85)

Therefore:-

6.48	=	$(N_s \times 6.38 \times 1.125 \times 0.85) \div 9.17$		
N_s	=	$(6.48 \times 9.17) \div (6.38 \times 1.125 \times 0.85)$		
	=	9.74 kg O_2/day	=	0.007 kg O_2/min

f) **Air Volume Required**

Assume O_2	=	23.20% of air
Weight of $1m^3$ air	=	1.20kg
Air required normally	=	$(0.007 \times 100) \div (23.20 \times 1.20)$
	=	0.024 m^3/min
But efficiency of diffuser	=	22%
	=	$(0.024 \times 100) \div 22$
Actual volume of air required	=	0.110 m^3/min
	=	110 l/min

g) **Air Diffuser Specification** **Refer Appendix B**

Type	=	Disc Diffuser		
Model	=	UNIFLEX U-330		
Capacity	=	0.10 m^3/min		
Required Air Diffuser	=	$(0.110 \div 0.10)$	=	1.10
No. Provided	=	2		

h) **Total Air Requirement**

Aeration Tank = 0.110 m^3/min

i) **Blower Specification** **Refer Appendix C**

Type = Hiblow Air Pump (2 units)

Model	=	HP120	
Capacity	=	0.120 m^3/min > 0.110 m^3/min	*...ok*
Power	=	115 Watt	

j) **<u>Sludge Return Rate (RAS)</u>**

MLSS	=	3.00 kg/m^3
MLSS ÷ (Cu-MLSS)	=	3.00 kg/m^3 ÷ (7.00 – 3.00) kg/m^3
	=	0.75
	=	0.75 x 18.00 m^3/day
	=	13.50 m^3/day
	=	0.009 m^3/min

Note: Normally Return Sludge Rate will be – 75 – 100% of $Q_{average}$

Recheck the F/M Ratio	=	(18.00 x 0.24) ÷ (2.4 x 7.53)
	=	0.24
Check Organic Loading	=	(18.00 x 0.24) ÷ 7.53 m^3
	=	0.57 kgBOD/m^3day
Sludge Yield	=	0.80 kg/kgBOD removed (Guidelines for Developers Volume IV)
BOD removed	=	0.24 x 18.00
	=	4.32 kgBOD_5/day
Sludge Yield	=	0.80 kg x 4.32 kgBOD_5/day
	=	3.46 kg/day
SS in treated effluent	=	20 mg/l
Solids in treated effluent	=	18.00 x 0.02
	=	0.36 kg/day

Sludge Age, θ_{Sludge} = $\dfrac{\text{Total solids in aeration tank * MLSS}}{\text{Excess sludge wasting/day + solid in effluent}}$

= $\dfrac{(7.53 \times 3.00)\ \text{kg}}{(3.46 + 0.36)\ \text{kg/day}}$

= 6 < 10 days

Waste Activated Sludge (WAS) = $[((VT \times MLSS) \div \theta_{Sludge}) - (Q_{ave} - Sseff)] \div Cu]$

Where:-

VT	=	Volume of reactor (m^3)
MLSS	=	Mixed liquor suspended solids (kg/m^3)
θ_{Sludge}	=	Sludge age (day)
Q_{ave}	=	Average flow (m^3/day)
Sseff	=	Effluent suspended solids (kg/m^3)
Cu	=	Underflow concentration (kg/m^3)
Q_{WAS}	=	$\dfrac{[(7.53 \times 3.00) \div 6] - (18.00 \times 0.02)}{7}$
	=	0.49 m^3/day

6.0 <u>CLARIFIER TANK (SECONDARY SEDIMENTATION TANK)</u>

a) <u>Design Criteria</u>

Hydraulic detention time at Average Flow	>	2.0 hrs
Weir Loading Rate	≤	150 – 180 m^3/m day
Surface overflow	≤	30 m^3/m^2 day
Solid Loading Rate	≤	50 kg/m^2 day

b) <u>Tank Specification</u> (Refer Appendix 1)

Tank Radius	=	0.80 m
Tank Height	=	2.30 m
Volume of Sedimentation Tank	=	3.04 m^3
Surface Area	=	1.25 m^2

Required Capacity (V_{SD})	=	(Q_A x 2.0 hrs) ÷ 24	
	=	(18.00 x 2.0) ÷ 24	
	=	1.50 m^3 < 3.04 m^3	*…ok*
Actual Detention Time (HRT)	=	(3.04 x 2.0 hrs) ÷ 1.50	
	=	4.05 hrs	*…ok*
Surface Overflow Rate (SOR)	=	Q_A ÷ Area	
	=	18.00 m^3/day ÷ 1.25 m^2	
	=	14.40 m^3/m^2.day	
Solid Loading Rate (SLR)	=	Q_A x 2.00 ÷ Area	
	=	(18.00 x 2.00) ÷ 1.25 m^2	
	=	28.80 kg/m^2.day	
Weir Loading Rate	=	Q_A ÷ Weir Length	
	=	(18.00 ÷ 0.60)	
	=	30.00 m^3/m.day	

c) <u>Desludging Interval</u>

Consideration 60% of the Sedimentation Tank volume use for sludge accumulation

Sludge Production Rate = 0.04 m^3/PE.year

Desludging Period = $\frac{\text{Volume for Sludge Accumulation}}{\text{Sludge Production Rate}}$

$= \frac{0.60 \times 10.86 \, m^3}{0.04 \times 80}$

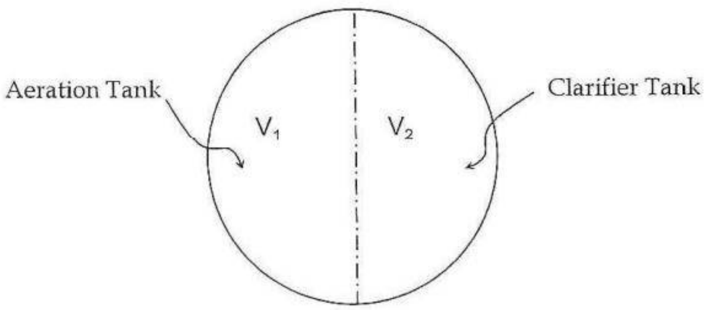
= 2.04 yrs

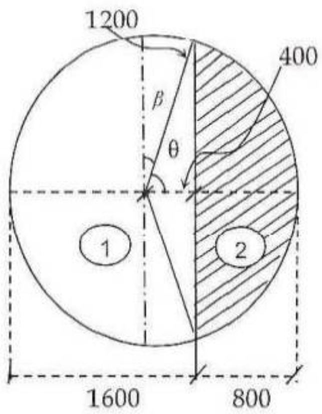
TANK SPECIFICATION

APPENDIX 1

CALCULATION SHEET

Project Title	:	Volume Calculation of Aeration Tank
Drawing Number	:	SA 50 (80 PE)
Date	:	22/3/2013
Page	:	1/2





Radius	=	1.20 m
Depth	=	2.35 m
Diameter of Tank	=	2.40 m Φ

Length of ① = 1.60 m
Length of ② = 0.80 m

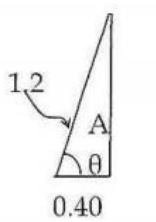
Cos θ = $\frac{0.40}{1.20}$

θ = $\cos^{-1} 0.3333333$

= 70.53^{0}

β = 90 – 70.53

= 19.47^{0}



$1.44 = 0.16 + x^2$

$x^2 = 1.44 - 0.16$

$x = 1.131$ m

CALCULATION SHEET

Project Title	:	Volume Calculation of Aeration Tank
Drawing Number	:	SA 50 (80 PE)
Date	:	22/3/2013
Page	:	2/2

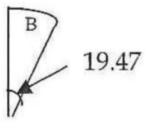
Area of Sector A = $0.50 \times 0.40 \times 1.131$

= $0.226\ m^2$

Area of Sector B = $0.50\ (\theta \times \frac{\pi}{180}) \times r$

= $0.50\ (19.47 \times \frac{3.142}{180}) \times 1.44$

= $0.245\ m^2$



Total Area of Sector A & B = $0.226 + 0.245$

= $(0.471\ m^2) \times 2$

= $0.942\ m^2$

Area of half circle = $\frac{\pi}{2} \times 1.44$

= $2.26\ m^2$

Therefore;

Total area of A, B & half circle; A_1 = $0.942 + 2.26$

= $3.204\ m^2$

Area of shaded segment = $0.50\ [(\theta \times \frac{\pi}{180}) - \sin\theta] \times r^2$

= $0.50\ [(141.06 \times \frac{3.142}{180}) - 0.6285] \times 1.44$

= $1.32\ m^2$

Therefore

Volume, V_1 = 3.204×2.35

= $7.53\ m^3$

Volume, V_2 = 1.32×2.30

= $3.04\ m^3$

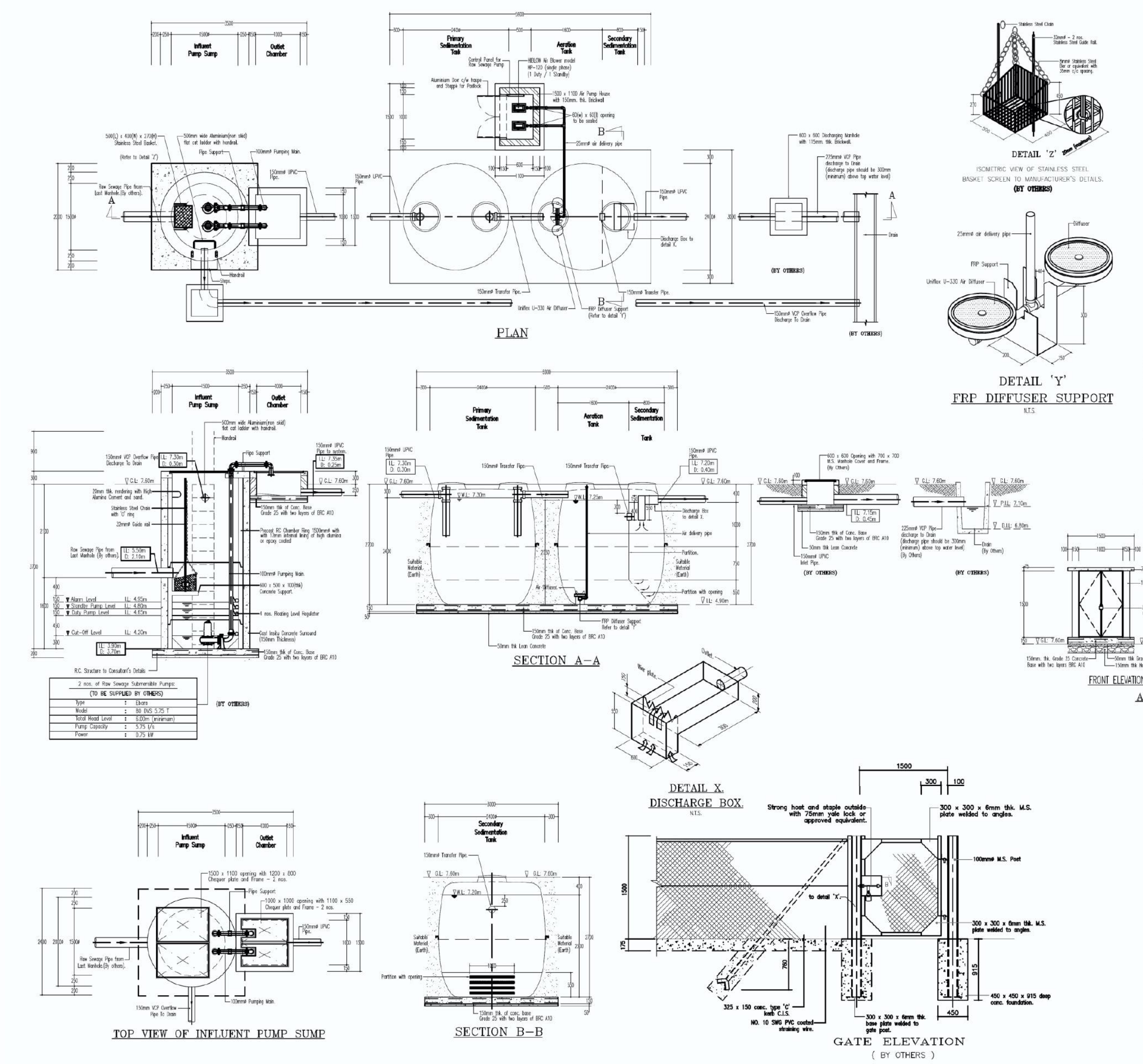


TABLE FOR INFLUENT PUMP SUMP

DAILY FLOW Q (m^3/day)	18.00
PEAK FLOW (m^3/day)	111.69
PUMPING CAPACITY (Required) (l/s)	1.29
PUMP SUMP	1.5øm
PUMP CUT–OFF LEVEL (m)	0.45

DESIGN SPECIFICATION AND DIMENSIONS

DATA	GWS–SA50
PROCESS	ACTIVATED SLUDGE PROCESS
TYPE OF WASTE	DOMESTIC
POPULATION EQUIVALENT (PE)	80
BOD & SS REMOVAL	EQA STANDARD 'A'
NUMBER OF TANK	2
PRIMARY SEDIMENTATION TANK VOLUME (m3)	10.86
AERATION TANK VOLUME (m3)	7.53
SECONDARY SEDIMENTATION TANK (m3)	3.04
TOTAL VOLUME (m3)	21.43
EMPTY TANK WEIGHT (TONNE)	0.55
WEIGHT OF SEWAGE (TONNE)	25.23
TOTAL WEIGHT OF TANK (TONNE)	25.78
HEIGHT (m)	2.70
INLET PIPE LEVEL (m)	0.30
OUTLET PIPE LEVEL (m)	0.40
INLET & OUTLET PIPE ø(mm)	150mmø
AIR BLOWER MODEL/POWER	HP–120 / 115 Watt
AREA REQUIRED (SYSTEM ONLY)	17.70m^2 (5.9m x 3.0m)

NOTES:

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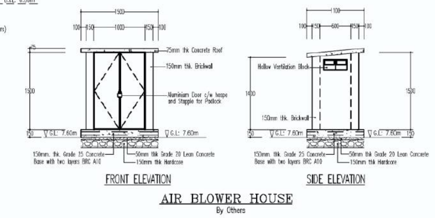
1. MAXIMUM DEPTH OF 'GWS' TANK INSTALLATION BELOW GROUND LEVEL IS 800mm.
2. THE THICKNESS OF INSPECTION CHAMBER SHOULD BE 115mm.
3. THIS DESIGN DOES NOT CATER FOR TRAFFIC LOADING.
4. SHEAR FORCE FROM NEARBY TRAFFIC SHOULD BE TAKEN INTO CONSIDERATION (if any) WHILE DETERMINE THE POSITION OF TANK.
5. WASTEWATER OF OIL AND GREASE NATURE (eg. KITCHEN, RESTAURANT) SHALL BE PRETREATED BY 'GREASE TRAP' BEFORE ENTERING THE 'GWS' SYSTEM.

SCOPE OF WORK

GOODWATER ENG. : SUPPLY THE TANK ONLY

CONTRACTOR : INSTALLATION AND PIPE FITTINGS.



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Details of SSTS for 80 PE

APPENDIX I-1

AP – I2

SMALL SEWAGE TREATMENT SYSTEMS (SSTS) FOR 150 PE

‘SSTS Plant Design Calculation for 150 PE

SIZING OF PUMP SUMP

BASIC DESIGN DATA

• POPULATION EQUIVALENT	:	150 PE	
• AVERAGE FLOW RATE	:	33.75 m^3/day	= 0.00039m^3/s (0.0234m^3/min)
• PEAK FACTOR	:	4.7 $(PE/1000)^{-0.11}$	= 5.80
• PEAK FLOW	:	5.80 x 33.75.00	= 195.75 m^3/day = 0.00226 m^3/s (0.1360^3/min)
• BOD CONCENTRATION (Influent)	:	250 mg/l	
• BOD LOADING	:	8.4375 Kg/day	
• MLSS CONCENTRATION (Influent)	:	300 mg/l	
• MLSS Aeration Tank	:	4,250 mg/l	
• MLVSS Aeration Tank	:	3,400 mg/l	

TREATED EFFLUENT

The final effluent shall complied to the Standard A as stipulated by the Environment Quality (Sewerage and Industrial) Regulations 1979, Department of Environment, Ministry of Science, Technology and Environment i.e.

BOD	:	Max 10 mg/l
COD	:	Max 60 mg/l
SS	:	Max 20 mg/l
NH_3N	:	Max 5 mg/l
NO_3N	:	Max 10 mg/l
O&G	:	Max 2 mg/l

1. PRIMARY SCREEN CHAMBER

Average flow	=	33.75 m^3/day
	=	0.00039 m^3/s (0.0234 m^3/min)
Clear Opening	=	25 mm (0.025 m)
Bar Width	=	10 mm (0.010m)
Nos of Opening	=	8
Total Opening Areas	=	8 x 0.025 = 0.20 m
No of Bars	=	9
Total Bar Area	=	9 x 0.010 = 0.09 m^2
Total Screen Width Required	=	0.20 + 0.09 = 0.29 m
Screen width provided	=	0.30 m
Total clear opening area	=	0.025 m (Opening) x 8 Nos x 1.00 m (Width)
	=	0.225 m^3

Up stream velocity flow through at the screen channel at average flow:	=	$33.75\ m^3/day$
	=	$\frac{0.00039 m^3/s}{0.3 \times 1.00}$
	=	$0.0013 m/s$
Up stream velocity flow through at the screen channel at peak flow		
	=	$\frac{0.0026 m^3/s}{0.3 \times 1.00}$
	=	$0.0086/s$
Approach Velocity flow the screen chamber at average flow:		
	=	$\frac{0.0039 m^3/s}{0.225 m^2}$
	=	0.00173 m/s (Less than 0.3 m/s)
Velocity flow through at the screen chamber bar peak flow:		
	=	$\frac{0.00226 m^3/s}{0.225 m^2}$
	=	$0.010\ m/s < 0.8\ m/s$
Head loss through a bar screen:		
H	=	$\frac{v^2 - v^2}{2g} \times \frac{1}{0.7}$
Where;		
h	=	head loss
V	=	velocity through the bar screen
v	=	velocity upstream of bar screen
g		Acceleration due to gravity
Head loss through a bar screen at average flow		
h	=	$\frac{0.00173^2 - 0.0013^2}{2(9.8)} \times \frac{1}{0.7}$
h	=	0.000000095m
Head loss through a bar screen at peak flow		
h	=	$\frac{0.010^2 - 0.0086^2}{2(9.8)} \times \frac{1}{0.7}$
h	=	0.00000189 m
Estimated volume of Screening per volume of waste water	=	$30 m^3/10^6\ m^3$
Design for weekly cleaning (7 days). Thus, quantity of screenings	=	$\frac{30}{10^6} \times 33.75 \times 7$
	=	$0.0070 m^3/week$
Design Volume of basket	=	$0.30\ (L) \times 0.30\ (W) \times 0.50 (D)$

= $0.027m^3 > 0.0070\ m^3$

2. DESIGN OF PUMP SUMP

Peak flow	=	$195.75\ m^3/day$
	=	$0.00226m^3/s$
Pumps design flow	=	$Qpeak$ (As per IWK Guideline)
	=	$0.00226\ m^3/s$ (2.26 L/s)
Allow for 10 start / stop per hours		
Thus volume of pump sump required	=	$\frac{Tq}{4}$
	=	$\frac{\frac{60}{10} \quad \frac{195.75}{24\times60}}{4}$
	=	$0.20\ m^3$
Effective volume provided, using 1500mm dia. Round MH	=	$1.50^2 \times 3.142 \times 0.45\ (D)$
	=	$3.18m^3 > 0.20m^3$
Effective volume provided	=	$3.18m^3$

2.1 PUMPING CYCLE

Average flow	=	$33.75 \frac{m^3}{day} = 0.39l/s$
Peak flow	=	$195.75 \frac{m^3}{day} = 2.26l/s$
Level of Discharge Point	=	18.85 m
Level of Pump Cut-Off Point	=	15.53
Static Head	=	3.32 m
Discharge pipe of each pump	=	100 mm (D.I)
Pipe areas	=	$3.14\ j^2$
	=	$3.14\ (\frac{0.10}{2})^2$
	=	$0.0079m^2$

HEAD LOSS COMPUTATION

Static Head Adopted	=	$4.00m$
Dynamic Head	=	Head loss due to numbers of short elbow 90° bends + Frictional loss in pipe + Velocity head loss
	=	$H_{f1} + H_{fm} + V^2/2g$

Fitting Head loss due to:

Numbers of fittings, valve and short elbow 90°bends:

Item	Description	Qty	K' Value	Amount
1	90 deg elbow	1	0.29	0.29
2	Taper	0	0.09	0
3	Gate valve	1	0.12	0.12
4	Check valve	1	1	1
				1.41

H_{fl} = 'K' value x $V^2/2g$

Flow (l/s)	m^3/s	Velocity (V)	Head Loss
0	0	0.00	0.000
5	0.005	0.64	0.039
10	0.01	1.27	0.155
15	0.015	1.91	0.348
20	0.02	2.55	0.619
25	0.025	3.18	0.968
30	0.03	3.82	1.393

Frictional loss in Pipe by Manning's Formula:

$$H_{fm} = \frac{fm \times L \times V^2}{D\,2g}$$

$$\text{Where } fm = \frac{124.6}{D} \times n^2$$

H_{fm}	=	Frictional Loss Head, m.
Fm	=	Friction Loss Co-efficient.
N	=	Roughness factor = 0.013 for New Ductile Iron Pipe.
L	=	Total pipe length, m.
D	=	Pipe inner diameter, m.
V	=	Average Velocity, m/sec.
g	=	Acceleration of free fall = 9.8 m/sec^2
$V^2/2g$	=	Velocity Head, m.
D	=	0.10 m.
L	=	1.00 m.
n	=	0.013.

Hence H_{fm}

Hence H_{fm} $= \dfrac{124.6 \times n^2 x}{D^{1.333}} \quad \dfrac{L \times V^2}{2g}$

Flow (l/s)	m^3/s	Velocity (V)	Head Loss
0	0	0.00	0.000
5	0.005	0.64	0.009
10	0.01	1.27	0.038
15	0.015	1.91	0.085
20	0.02	2.55	0.150
25	0.025	3.18	0.235
30	0.03	3.82	0.338

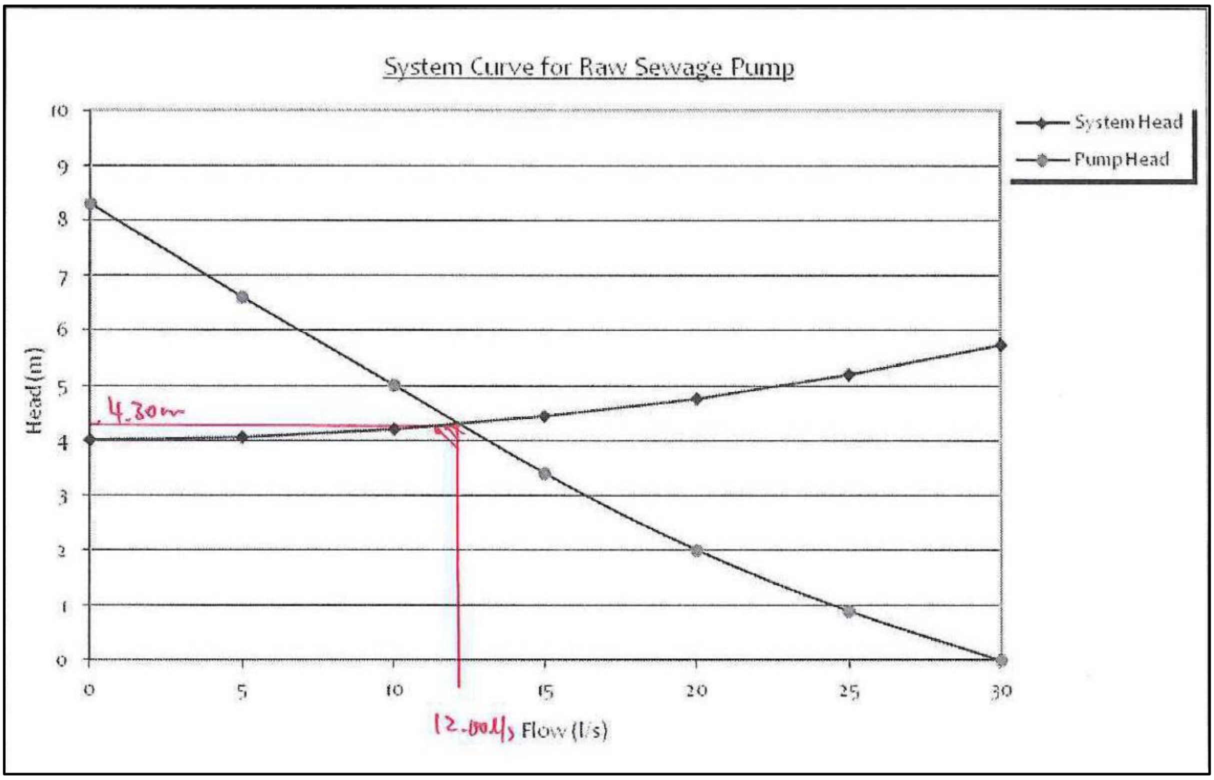
H_{f1} = 'K' value x $V^2/2g$

Total System Head:

Flow (l/s)	Static Head (m)	Fitting Loss due to 100 mm dia. Fittings	Fitting Loss due to 100 mm dia. Pipes	System Head
0	4.00	0.000	0.000	4.000
5	4.00	0.039	0.009	4.048
10	4.00	0.155	0.038	4.192
15	4.00	0.348	0.085	4.433
20	4.00	0.619	0.150	4.770
25	4.00	0.968	0.235	5.202
30	4.00	1.393	0.338	5.731

System Curve:

Flow (l/s)	System Head	Pump Head
0	4.000	8.3
5	4.048	6.6
10	4.192	5
15	4.433	3.4
20	4.770	2
25	5.202	0.9
30	5.731	0



Pump Recommended = 2 Numbers of 'EBARA' Raw Sewage Pump, Model:100 DML51.5; 1.50kW with discharge capacity of 12.00 l/s at operating head of 4.30 m. The peak flow of the system required is 2.26 l/s.

Velocity in 100mm diameter D.I discharge pipe:

Velocity at discharge pipe = $\frac{0.012m^3/s}{0.00785m^2}$

= 1.52 m/s (Less than 2.50 m/s as per SPAN guideline)

TIME CYCLE

- Level of Pump Start Point = 15.98 m
- Level of Pump Cut-Off Point = 15.53 m
- Level Difference in Between Start and Stop = 0.45 m

Based on average flow;

Effective Vol. Provided = $3.18m^3(3{,}180.00\ Liter)$

Filling time = 3.180 / 0.39 l/s

= 8153 s

= 135.90 min

Dewatering time = 3,180.00/(12.00 – 0.39 l/s)

= 273.90 s

= 4.56 min

Total time required = (135.90+4.56) min

= 140.46 min

Cycle = 60/140.46

= 0.42 times/hour

SECONDARY SCREEN CHAMBER

Average flow	=	33.75 m^3 / day
	=	0.00039 m^3/s (0.0234 m^3/min)
Clear Opening	=	12 mm (0.012 m)
Bar Width	=	10 mm (0.010 m)
Nos of Opening	=	29
Total Opening Areas	=	29 x 0.012 = 0.348 m^2
No of Bars	=	30
Total Bar Area	=	30 x 0.010 = 0.30 m^2
Total Screen Width	=	0.348 + 0.30 = 0.648 m
Screen Width provided	=	0.70 m
Screen Length	=	0.5 m
Total clear opening area	=	30 x 0.012 = 0.360 m^2
	=	0.36 m^2 x 0.5 m (Length)
	=	0.180 m^2

Up stream velocity flow thru at screen chamber during Average Flow:

$$= \frac{0.00039 m^3/s}{(0.70 \times 0.50) m^2}$$

= 0.0011 m/s (Less than 0.3 m/s)

Up stream velocity flow thru at screen chamber during Peak Flow:

$$= \frac{0.00226 m^3/s}{(0.70 \times 0.50) m^2}$$

= 0.00645 m/s < 0.8 m/s

Approach Velocity flow thru screen chamber during Average Flow:

$$= \frac{0.0039 m^3/s}{0.18 m^2}$$

= 0.00216 m/s (Less than 0.3 m/s)

Velocity flow thru screen chamber during Peak Flow:

$$= \frac{0.00226 m^3/s}{0.18 m^2}$$

= 0.0125 m/s < 0.8 m/s

Head loss flow through bar screen at Average Flow:

$$h = \frac{0.00216^2 - 0.0011^2}{2(9.8)} \times \frac{1}{0.7}$$

h = 0.000000251 m

Head loss through bar screen at Peak Flow:

$$h = \frac{0.0125^2 - 0.00645^2}{2(9.8)} \times \frac{1}{0.7}$$

h = 0.0032 m

Calculation on half (1/2) blocked during the Peak Flow;

Areas reduce to half (1/2) = ½ x 0.18

= 0.09 m^2

Velocity flow through at the screen chamber bar @ Peak Flow;

$$= \frac{0.0125 m^3/s}{0.09 m^2}$$

= 1.388 m/s

Head loss through bar screen at Peak Flow on half (1/2) block:

$$h = \frac{0.0032^2 - 0.000000251^2}{2(9.8)} \times \frac{1}{0.7}$$

$$h = 0.000000746 \text{ m}$$

Depth of water at 50% clogging = Max. Depth at peak flow + Head loss at 50% clogging
= 0.000000746 + 0.0032
= 0.00320m

The channel depth provided at the secondary screen is 500 mm (0.50m) which is sufficient.

FOR SSTS; MODEL: MFT-150 DETAIL CALCULATION, PLEASE REFER TO ATTACHMENT.

Muifatt mechanical IST Less than 150 PE:

Process: Anaerobic Septic Tank Supplemented with Activated Sludge Process

1. DESIGN DATA

No.	Parameter	Unit	Value/ Description		Remarks
1.	Design population (PE)	PE	150		
2.	Type of waste		Domestic / organic		
3.	System Process		Aeration System for IST using Activated Sludge Process		
4.	Influent BOD (ppm), Si : TSS (ppm), SSi	ppm	250:		300 ppm = g/m^3 or $0.0111 kg/m^3$
5.	Standard of Effluent		A		
6.	Effluent BOD (Average), Se: TSS (Average), SSe	ppm	10	:	20 ppm=g/m^3 or x $0.001 kg/m^3$
7.	BOD: TSS Removal Efficiency (%)	%	96.00	:	93.33
8.	Daily Average Flow, Q_{avg} (m^3/PE.d)	m^3/PE.d	0.225		225 litre/s
9.	Peak Flow Factor, 4.7 x $(PE/1000)^{-0.11}$		5.79		
10.	Total daily average Flow Q_{avg} (m^3/d)	m^3/d	33.8		
		m^3/hr	1.4		
		m^3/min	0.023		
		m^3/s	0.0004		
11.	Total Daily Peak Flow Qpeak (m^3/d)	m^3/d	195.4		
		m^3/h	8.1		
		m^3/min	0.136		
		m^3/s	0.0023		
12.	Dissolved oxygen, DO (Table 5.12)	mg/l (g/m^3)	1.0 (Date from aeration tank)		

13.	Heterotropic yield coef, Yh (Table 5.12)	kg SS/kgBOD	0.80 mg SS/mg BOD (Aeration tank data)
14.	MLSS	mg/l (g/m^3)	2300 (Data from aeration tank)
15.	MLVSS	mg/l (g/m^3)	1840
16.	Influent NH_4-N	mg/l (g/m^3)	30.00 mg/l

1. <u>Anaerobic Single Baffle Reactor Hybrid with Clarifier Settler</u>

Volume provided (6 to 24 hrs residence time) V1	=	m^3	28.92 (M &Eddy Chap.10-4)
BOD Loading	=	$kg/m^3.d$	0.29 <2.2, o.k. M &Eddy Table 10-7
Residence time/Hydraulic Retention Time (HRT)	=	hr	20.56 (M &Eddy Chap.10-4)
Clarifier Settler Compartment Surface Areas at Design TWL, A1	=	m^2	7.50

PROCESS DESIGN SEDIMENTATION

Hydraulic Loading rate (HLR) to be <			4.50 $m^3/m^2.d$
Hydraulic Loading Rate (HLR)	=		Qa/A1
	=		4.5 $m^3/m^2.d$

Design HLR < 4.5m^3/m^2.d (MS1220:1991): Thus design is OK

Average BOD_5 removal efficiency, Re1	=		50.0%(M &Eddy Table 10-7)
BOD_5 removed	=	BOD_5 x Re1	
	=	4.21875 kg/d	
Remaining BOD5 to aeration polishing	=	BOD_5 x (100%-Re1)	
	=	4.21875 kg/d	
Average SS removal efficiency, Re2	=		75%
SS Removed	=	SS x (100%-Re2)	
	=	2.53125 kg/d	
Sludge accumulation rate, Sr	=	0.04 m^3/PE, yr	
Total sludge accumulation, Sd	=	Sr x PE	
	=	6.000 m^3/yr	
Retention time at end of 1st year, t1	=	V1 -Sd x 1	
	=	22.917 m^3 /Qa	
	=	0.679 day	
	=	16.297 hrs	within 4-24hrs range, M &Eddy Chap10-4
Retention time at end of 2nd year, t2	=	V1 -Sd x 2	
	=	16.917 m^3 /Qa	
	=	0.501 day	
	=	12.030 hrs	within 4-24hrs range, M &Eddy Chap10-4

2. Conventional Activated Sludge Secondary Treatment Unit

2.1 AERATION TANK

Note: The Mechanical Aeration Polishing Calculation is carried out for 96.00% BOD Removal

Actual BOD removal efficiency required is	46.00%	after anaerobic digestion		
The aeration volume therefore can be	46.00%	of that calculated here		
The ratio is	46.00% / 96.00%	=	0.479	

Adjust the height of diffusers or location of the inclined plate to proportionate the aeration tank volume calculated here

Volume above the diffusers are effective for aerobic activated sludge biological treatment process.

Volume below diffusers are effective for anaerobic biological treatment process

The below kinetic coefficient shall be adopted.

Influent BOD loading = 250 ppm x Q_{avg}	=	0.25 kg/m^3 x	33.8 m^3/d (250ppm Influent BOD_5 = 250m^{3}10^{6}m^{3}x1000kg/m^3)
	=	8.44 kg/d	
	=	8.44 kg/d	
MLSS (Mixed Liquor Suspended Solid)	=	2300 mg/L	(2.3 kg/m^3)
Dissolved oxygen	=	1 ppm	(0.001 kg/m^3)
Sludge yield	=	0.80 kg sludge produced / kg BOD5 consumed (at 6hrs hyd.retention time)	
Underflow Concentration	=	4600 mg/l	
Density of Air D_A	=	1.201 kg/m^3	
% of O_2	=	23.2%	

Hydraulic Retention Time

Volume required	=	33.8 m^3/d x 6 hrs/24hrs for system where only ammonia removal is required	
	=	8.44m^3	
Nos. of tank	=	1	
Adopt:		Tank Size	
Type of Aeration	=	Diffuser (Please input "Diffuser" or "Äerator")	
Length without "*l*" Plate (L)	=	2.973 m	equivalent length for 4.028m^2 X-sectn wetted area
Top plan view length of "*l*" Plate (L_p)	=	924 mm	
Total Internal Height = Tank dia (D)	=	2.4 m	
Water Depth (h)	=	2 m	
Freeboard	=	0.4 m	
Width at Water Level (W)	=	1.79 m	refer separate calculations
Nos. of tank	=	1	
Wetted Area of Circular X-sectn, A_c	=	4.028 m^2	
Vol at Circular X-sectn= A_c *L, V_{A11}	=	11.97524 m^3	

Wetted Area of "*l*" X-sectn (A_p)	=	3.532107 m^2
Vol. under "*l*" Plate for Aeration = $A_p/2^* L_p$, V_{AT2}	=	1.631833 m^3
Wetted Area of X-sectn below "*I*" Plate = A_b	=	0.495893 m^2
Vol. below bottom of "*l*" Plate= $A_b{}^*L_p$, V_{AT3}	=	0.458206 m^3
Total Vol. if Aeration Tank V_{AT}	=	14.065 m^3
Actual retention time	=	14.065 m^3 / 33.8 m^3/d x 24 hrs /d
	=	10.002 hrs > 6hrs, O.K.

W;h recommended = 1:1 to 2.2:1

Calculation Notes for Above :

Wetted X-Section area form water level to bottom of "*l*" Plate	=	4.028 m^2^- (pi()*D^2/4) – 4.028 m^2
	=	3.532 m^2
Wetted Area of X-sectn below "*l*" Plate A_b	=	(pi()*D^2/4) – 4.028 m^2
	=	4.524 m^2 – 4.028 m^2
	=	0.496 m^2

<u>F/M Ratio (Food/ Microorganism ratio)</u>

F/M return shall be in the range of 0.25 – 0.50 d^{-1}

F/M = BOD loading x Q_a / V_{AT}xMLSS	=	250 ppm x 33.8 m^3/d / 14.065 m^3 x 2300ppm
		0.26, O.K. 0.25 <F/M<0.5, O.K

It is expected that by providing this volume of aeration tank, with the inflow BOD at 250 ppm, the excess microorganism above 2300 ppm will be staved of food and endogenous decay become the dominant process

<u>Excess Sludge wasting (Sw)</u>

The sludge yield shall be		0.80 kg dry solid / kg BOD consumed
Sw= Qa x BOD loading x sludge yield		33.8 m^3/d x 0.25 kg/m^3 x 0.80 kg sludge /kg BOD consumed
Sw	=	6.75 kg/d

This is a theoretical value. In practice, there is no wasting and all sludge are residence in the aeration cum aerated digester sludge holding tank

<u>Sludge Age θ_s</u>

The sludge age shall be designed for 5~10 days based on daily wasting. There is no actual daily wasting. So, refer to the combined aerated digester sludge holding tank cum aeration tank sludge Age (Solids residence time) calculations

$\theta_s = (V_p)(Mv\text{-}SS_e)/S_w$	=	14.07 m^3 x (2.3kg/m^3-0.02 kg/m^3)/6.75 kg/d
	=	5 days (+/-) To increase MLSS or Volume of Aeration tank OK

Note: V_p = Volume of Primary Aeration tank = V_{AT}, M_v + Mixed Liquor Suspended Solid

Note: If there us daily wasting,

The hungry activated sludge of microorganism is designed to have minimum 5 days' residence time in the aeration tank at a concentration 2300 ppm to eat the food (BOD), grow and actively moving with abundant supply of oxygen from air diffusers and reduce the weight (and hence volume) of its body after growing and vigorous activities.

Theoretical Waste Activated Sludge Q_w (From the underflow of concentration Cu = 4600 ppm)

$Q_w = ([(V_p \times M\theta_s] - (Q_a * SS_e)]/Cu$ = 1.33 m^3/d

Aerator BOD_5 Volumetric Loading

Aerator's BOD5 volumetric loading shall be at 0.3-0.6kg/m^3/d

= Q_{avg} /PE x BOD_{inf} x PE / V_{AT} =

= 0.225 m^3/d.PE x (250ppm x 150 PE/14.06528 m^3)

= (0.05625 kg/d.PE x 150 PE/14.06528 m^3)

= 0.600 kg/d.m^3, 0.3 = Aerator Volumetric Loading = <0.6, OK (Note: 250 ppm = 0.25kg/m^3)

Recirculation Ratio

The allowable re-circulation ratio shall be 0.75 -1 (Table 5.12)

Q_{RAS} = [MLSS/CU-MLSS)] * Q_{AVG},

Where,

Numerator MLSS = Original MLSS in the aeration tank will die off in one day and needed to be replenished.

Denominator MLSS = Original MLSS in the aeration tank will die off in one day and needed to be replenished

CU = Underflow concentration from final sedimentation tank is thicker than MLSS in aeration tank and will be withdrawn a proportion (compared to Q_{avg} rate of flow) to replenish the MLSS in aeration tank that is dying off

CU - MLSS = Excess replenishing MLSS after subtracting the dying off MLSS from the replenishing CU

Therefore,

Q_{RAS} = MLSS /(CU-MLSS)Q_{avg}

Q_{RAS} / Q_{avg} = MLSS /(CU-MLSS)

= 2300 / (4600 – 2300)

= 1.00

The underflow sludge concentration shall be at 4.6 kg sludge/m^3 in order to maintain the MLSS

All requirement Rate

Adopt oxygen requirement = 2 kg O_2 required /kg BOD & Ammonia substrate removal (growing phase)

(i) The critical oxygen transfer, N_F = BOD & Ammonia substrate removal x Qa x Oxygen Requirement

= (0.25 – 0.01) kg/m^3 x 33.8 m^3/d x 2.0 kg O_2/ kg BOD removal

= 16.20 kg O_2/d

= 0.68 kg O_2/hr

(ii) Oxygen transfer under field condition, N_s = $(N_F \times 9.15)/[(C_S-C_L) \times 1.024^{(T20)} \times 0.85]$

$$= \frac{0.68 \text{ kg } O_2 \text{ /hr} \times 9.15}{(8.38-1) \times (1.024)\ (25-20) \times 0.85}$$

	=	0.87 kg O_2/hr
Vol. Air req for coarse bubble diffuser	=	Wt. of O_2/(Da*0.232*Er)
	=	0.87 / (1.201kg/m^3*0.232*0.08)
	=	39.23 m^3/hr
	=	0.65 m^3/min
Vol. Air req for fine bubble diffuser	=	Wt. of O_2/(Da*0.232*Er)
	=	0.65 / (1.201kg/m^3*0.232*0.18)
	=	13.04 m^3/hr
	=	0.22 m^3/min

<u>2.2 Secondary Clarifier (ref. cl. 5.9 & Table 5.18, MS 1227 for HLR for Anaerobic Treatment)</u>

This compartment served to separate the oxidized sludge generated and recycle back to the aeration tank to enhance the growth of the underflow MLSS concentration. The clear treated water is then discharged into the receiving watercourse through the outlet weir and pipe.

No. of tank required NO_T	=	1, Since for PE =<1000, 1 clarifier is acceptable, ref. Table 5.18
No. of tank provided NO_{TP}	=	1
Proposed Geometrical Configurations of The Tanks (S):		Rectangular/Square Surface with Cylindrical Conduit Slopes or 60°

<u>Surface loading or overflow rate (SOR)</u>

Surface loading or overflow rate shall be max 30m^3/m^2/d at Q_{peak}. Ref. Table 5.18 (Note: 40.04 m^3/m^2/d. Table 8-7. Metcalf & Ed)

Surface area req. per compartment = [Q_{peak} m^3/d / (30m^3/m^2/d)]/NO_{TP} (Note: including the anaerobic effluent polishing flow)

	=	195.4 m^3/d / (m^3/m^2/d) / 1
	=	6.514 m^2
Length without "/" Plate (L)	=	3.282 m
Top plan viw length of "/" Plate (Lp)	=	0.24 mm
Total Internal Height = Tank dia. (D)	=	2.40 m
Water Depth (h)	=	2 m (Cl, 3.3.8 MSIG Vol. V, Table 3.2, Liquid depth = 1.22m – 2.6m)
Freeboard	=	0.40m
Width at Water Level (W)	=	1.789 m (refer separate calculations)
Nos. of tank	=	1
Wetted Area of Circular X-sectn, Ac	=	4.028 m^2
Vol. at circular X-sectn = Ac*L, V_{CL1}	=	13.220 m^3
Wetted Area of "/" X-sectn (Ap)	=	3.532 m^2
Vol. above "/" Plate for Clarifier = Ap/2 * Lp, V_{CL2}	=	1.632 m^3
Wetted Area of X-sectn below "/" plate, A_b	=	0.496 m^2
Vol. below bot of "/" Plate = A_b*O, V_{AT3}	=	0 m^3
Total Vol. of clarifier tank, V_{CL}	=	14.852 m^3
Height from top of tank to cover	=	0.15 m
Surface Area req. based on Anaerobic Criteria HLR	=	7.500 m^2
Required Surface Area Based on SORpeak & HLR	=	7.500 m^2
Check Surface Area Provided	=	1.789 m x (3.282 m + 0.924 m) = 7.525 => 7.500 m^2 ok

Re-check actual surface loading: 195.4m^3/d / (7.50m^2 x 1) = 26.06 m^3/m^2/d

<=30M3/M2/d, ok

Weir Overflow Rate (WOR)

The weir loading shall be designed in the range of 150-180m^3/m/d at peak flow

Adopt WOR = 180 m^3/m/d

Weir length required = Qp/WOR

$$= \frac{195.4 \text{ m}^3/\text{d} / 1}{180 \text{ m}^3/\text{m/d}}$$

= 1.086 m per number of clarifier

Solid Loading Rate (SLR)

The SLR shall be 150 kg/m^2/d at Q_{peak}, < 50kg/m^2/d at Q_{avg}

Check SLR peak : Alternative No. 1:

$$\text{SLR peak} = \frac{Q_{peak} \text{ x incoming MLSS/nb VSS} + (Q_{RAS} + Q_{WAS}) \text{ x nb VSS}}{\text{Area of Clarifier}}$$

$$= \frac{195.4 \text{ m}^3/\text{d x 2300 mg/l} + 35.08 \text{ m}^3/\text{d x 4600 mg/l}}{7.50 \text{ m}^2 \text{x}}$$

$$= \frac{195.4 \text{ m}^3/\text{d x 4.6 kg/m}^3 + 35.08 \text{ m}^3/\text{d x 4.6 kg/m}^3}{7.50 \text{ m}^2}$$

= 141.3845 kg/m^2/d

< 150kg/m^2/d, O.K

$$\text{SLRavg} = \frac{33.8 \text{ m}^3/\text{d x 2300 mg/l} + 35.08 \text{ m}^3/\text{d x 4600 mg/l}}{7.50 \text{ m}^2}$$

= 31.87 kg/m^2/d

< 50 kg/m^2/d, O.K.

Scum Removal

The scum removal proposed for the STP is by means of manual scooping. The collected scum shall be returned to the aeration tank for further decomposition.

Selection of Rate of Gravitational Return Sludge

(I) Required Rate of Gravitational Return Sludge Based on QRAS/Q_{avg} = 1

$$\text{Rate of return} = \frac{Q_{ras} + Q_{was}}{\text{1 Nos.}} = \frac{(1.00 \text{ x } 33.8 \text{ m}^3/\text{d}) + 1.33 \text{ m}^3/\text{d}}{1}$$

= 35.08 m^3/d

= 0.024 m^3/min

= 0.00041 m^3/s

3.0 Sizing of air blower specification & Diffuser Specifications (Fine Bubble Diffuser System)

3.1 For Fine Bubble Diffuser System:

Total air required for sewage treatment system for one blower system is:

= air for aeration tank

= 0.217 m^3/min

= 0.217 m^3/min (For 96.00 % BOD removal)

= 0.100 m^3/min (Proportionate For 46.00% BOD removal)

Provision for Mixing and Aerobic Digestion in the Activated Sludge Compartment

(iii) Clause 8.5.2 (d) MS 1228 : 1991

Diffuser = 0.35 l/ m^3.s		
Vol. Air req. for diffuser	=	0.35 l/m^3.s
(per tank of sludge storage)	=	0.3500 * 6.832 m^3 * 60.00 s
	=	0.143468 m^3/min Adopt 0.143/min.

Propose Tube Aerator

3.3.2

Type	:	Uniflex Fine Bubble Air Tube Diffuser
Model	:	UT-550
Material	:	EPDM
Dimension	:	64mm dia. X 580 mm length

Proposed Nos. of Uniflex Fine Bubble Tube Diffuser l/min/diffuser =

Capacity	:	300 l/min/diffuser = 0.3 m^3/min/diffuser
Operating Flow Range	:	150 – 250 l/min/diffuser, use 150l/min (170 mm H_2O Pressure Loss) = 0.15 m^3/min/diff.
Discharge	:	0.143 m^3/min
Nos. of diffuser required	:	0.143 m^3/min / (0.150 m^3/min/diffuser) = 0.96 Diff.
Calculated Nos. of diffuser	:	1
Provided No. of diffuser	:	1 (Pressure Loss = 1*170 mm 170 mm, H_2O
Diffuser submergence Depth	:	1000 mm (Discharge Pressure min 1500 mm AQ. A1 Blower, Density of sludge 1020 kg/ m^3)

3.12

Type	:	EDI FlexAir T-Series Fine Bubble Air Tube Diffuser
Model	:	62 x 762
Material	:	EPDM
Dimension	:	66mm dia. X 762mm length

Proposed EDI FlexAir T-Series Fine Bubble Air Tube Diffuser

Capacity (Peak Air-flow)	:	24 m^3/hr/diffuser = 0.4 m^3/min/diffuser
Design Air Flow Range	:	5-17 m^3/hr/diffuser, use 8.5 m^3/hr (340mm H_2O Pressure Loss) = 0.142 m^3/min/diff.
Discharge	:	0.143 m^3/min
Nos. of diffuser required	:	0.143 m^3/min / (0.142 m^3/min/diffuser) = 1.01 Diff.
Calculated Nos. of diffuser	:	2
Provided No. of diffuser	:	2 (Pressure Loss = 1 * 300 mm 300 mm, H_2O
Diffuser submergence depth	:	1000 mm (Discharge Pressure min 1500 mm AQ. At Blower, Density of sludge 1020 kg/m^3

3.1.3

Type	:	HEXA Tubular EPDM Micro Pores Fine Bubble Air Tube Diffuser
Model	:	T-550
Material	:	EPDM
Dimension	:	550 mm length

Proposed HEXA Tubular EPDM Micro Pores Fine Bubble Air Tube Diffuser

Capacity (Peak Air-flow)	:	24 m^3/hr/diffuser = 0.4 m^3/min/diffuser
Design Air Flow Range	:	1-12 m^3/hr/diffuser, use 6 m^3/hr (300mm H_2O Pressure Loss)
Discharge	:	0.143 m^3/min

Nos. of diffuser required	: 0.143 m^3/min /(0.100 m^3/min/diffuser) = 1.43 Diff.
Calculated Nos. of diffuser	: 2
Provided No. of diffuser	: 2 (Pressure Loss = 1*300mm 300 mm.H_2O
Diffuser submergence Depth	: 1000 mm (Discharge Pressure min 1500 mm AQ. AI Blower, Density of sludge 1020kg/ m^3)

3.1.4

Type	:	Uniflex Fine Bubble Air Disc Diffuser
Model	:	U330
Material	:	ABS/EPDM
Dimension	:	10″

<u>Proposed Nos. of Uniflex Fine Bubble Air Disc Diffuser</u>

Capacity	: 10 m^3/hr/diffuser = 0.1167 m^3/min/diffuser
Operating Flow Range	: 1-10 m^3/hr/diffuser, use 5 m^3/hr (87mm H_2O Pressure Loss) = 0.083 m^3/min/diff
Discharge	: 0.143 m^3/min
Nos. of diffuser required	: 0.143 m^3/min /(0.083 m^3/min/diffuser) = 1.72 Diff.
Calculated Nos. of diffuser	: 2 Adopt
Provided No. of diffuser	: 2 (Pressure Loss = 1*87mm 87 mm.H_2O
Diffuser submergence Depth	: 1000 mm (Discharge Pressure min 1500 mm AQ. AI Blower, Density of sludge 1020kg/ m^3)

<u>Proposed Disc Aerator</u>

3.1.5

Type	:	EDI Fine Bubble Air Disc Diffuser
Model	:	9″ Disc Diffuser High Cap
Material	:	EPDM
Dimension	:	277mm Dia.

<u>Proposed Nos. of EDI Fine Bubble Air Disc Diffuser</u>

Capacity	: 18.7 m^3/hr/diffuser = 0.312 m^3/min/diffuser
Operating Flow Range	: 1.7-8.5 m^3/hr/diffuser, use 4.25 m^3/hr (232mm H_2O Pressure Loss) = 0.071 m^3/min/diff
Discharge	: 0.143 m^3/min
Nos. of diffuser required	: 0.143 m^3/min /(0.071 m^3/min/diffuser) = 2.03 Diff.
Calculated Nos. of diffuser	: 1 Not to adopt
Provided No. of diffuser	: 1 (Pressure Loss = 1*232mm 232 mm.H_2O
Diffuser submergence Depth	: 1000 mm (Discharge Pressure min 1500 mm AQ. AI Blower, Density of sludge 1020kg/ m^3)

<u>Proposed Disc Aerator</u>

3.3.7

Type	:	HEXA Disc EPDM Micro Pores Fine Bubble Air Diffuser
Model	:	D250
Material	:	EPDM
Dimension	:	250mm Dia.

<u>Proposed HEXA Disc EPDM Micro Pores Fine Bubble Air Diffuser</u>

Capacity	: 35 m^3/hr/diffuser = 0.583 m^3/min/diffuser
Operating Flow Range	: 6.00 m^3/hr/diffuser, use 6 m^3/hr (232mm

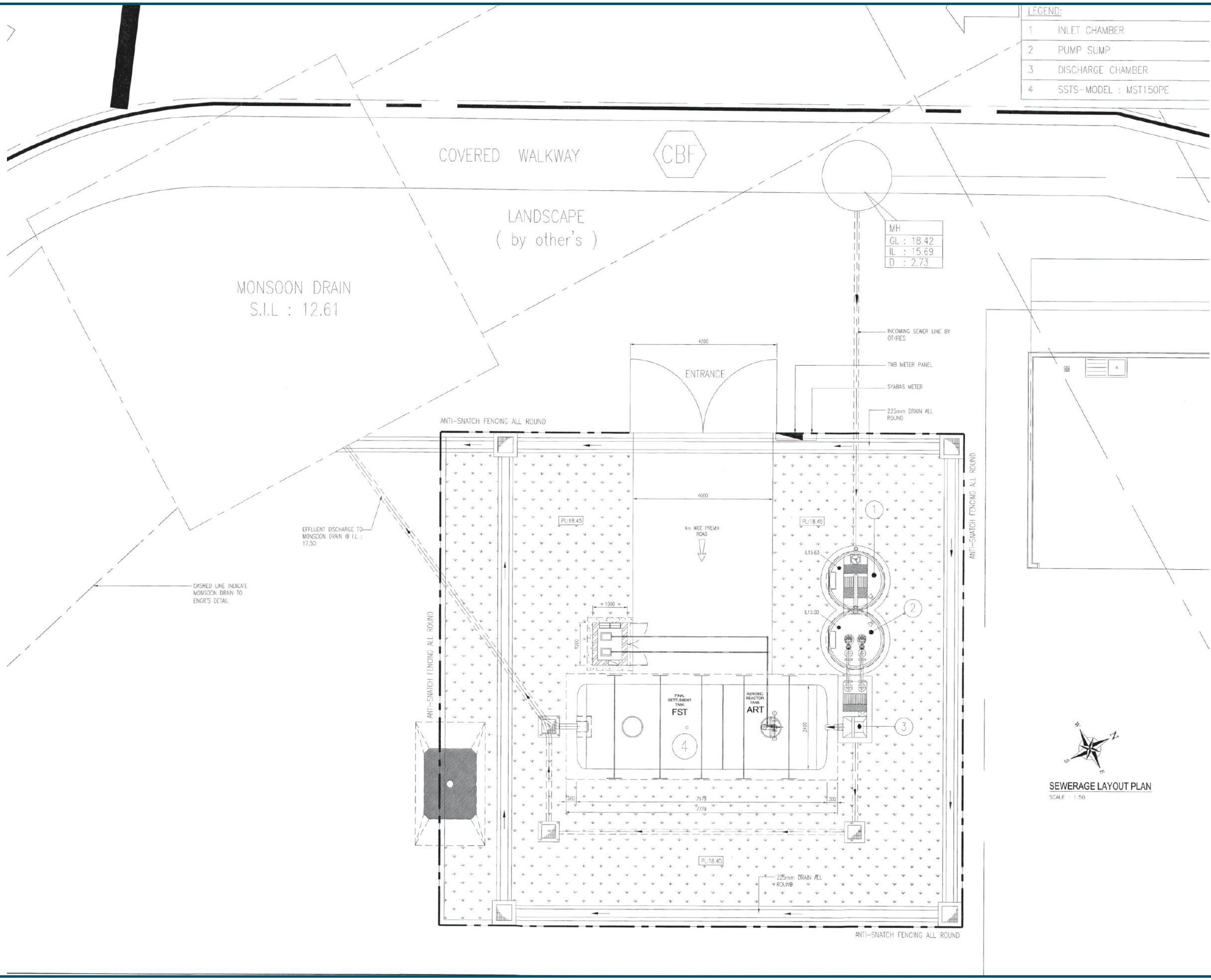
H_2O Pressure Loss) = 0.100 m^3/min/diff

Discharge	: 0.143 m^3/min
Nos. of diffuser required	: 0.143 m^3/min /(0.100 m^3/min/diffuser) = 1.43 Diff.
Calculated Nos. of diffuser	: 2 Adopt
Provided No. of diffuser	: 2 (Pressure Loss = 1*232mm 232 mm.H_2O
Diffuser submergence Depth	: 1000 mm (Discharge Pressure min 1500 mm AQ. AI Blower, Density of sludge 1020kg/ m^3)

3.1.7

<u>Proposed Air Blower Specification for Coarse Bubble Diffuser System</u>

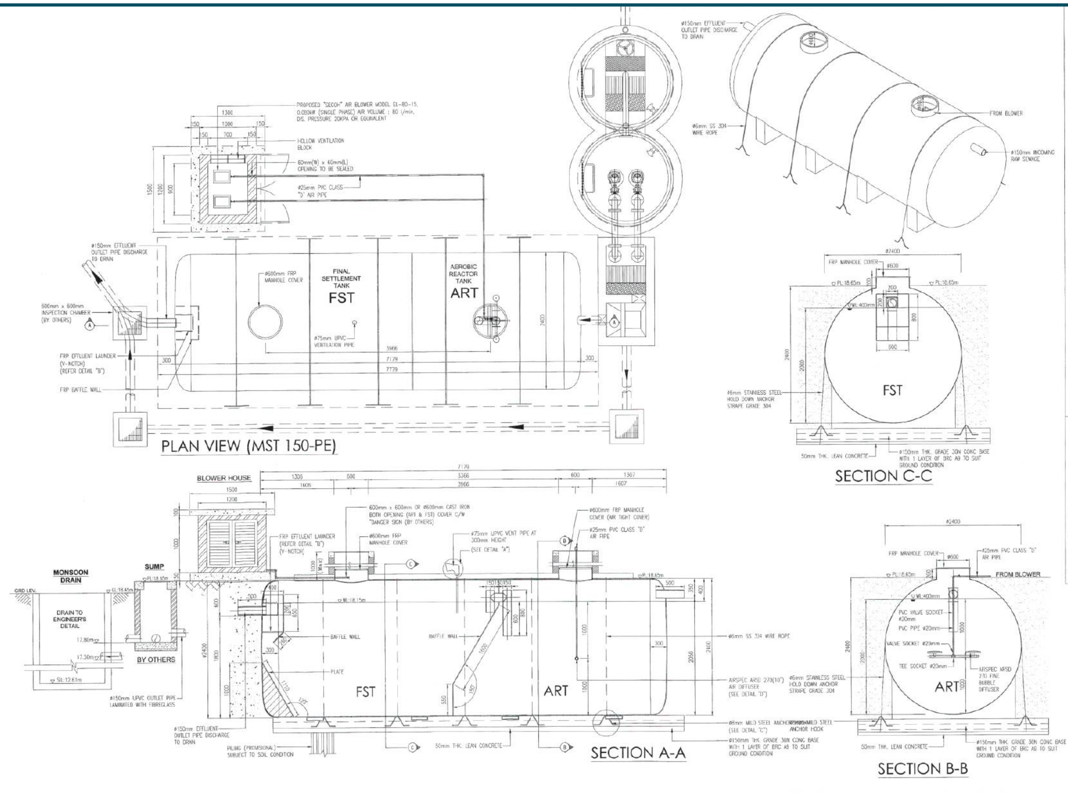
Model	:	TSB-50 Bore 50A	RSR-50K (50)
Capacity	: 0.143	0.78 m^3/min @ 770 rpm	1.26 m3.min @ 1240
Power	:	0.64 kW	1.5kW
Discharge Pressure	:	0.15 kgf/cm^2	2 kgf/cm^2
Nos. of Unit	:	2 units (1 duty, 1 standby)	2 units (1 duty, 1 standby)



Date	16-03-2017
Project No	EJ 610
Produced by	FCZ
Revision	A

Sewerage Layout Plan for SSTS (150PE)

APPENDIX I-2



DESIGN SPECIFICATION & DIMENSION OF MST150-PE
(HORIZONTAL TYPE)

DESIGN CRITERIA

MODEL	MST150-PE
PROCESS	UP FLOW ACTIVATED SLUDGE
TYPE OF WASTE	DOMESTIC
POPULATION EQUIVALENT	150 (MAXIMUM)
NUMBER OF TANK	1
BOD5 AND TSS REMOVAL	EQA STANDARD "A"
PRODUCT MATERIAL	FIBREGLASS
AEROBIC REACTOR TANK: VOL (L)	14065
FINAL SETTLEMENT TANK: VOL (L)	14852
TOTAL VOLUME (L)	32477
TANK DIAMETER, (MM)	2400
TANK HEIGHT, (MM)	2400
LIQUID HEIGHT, (MM)	2000
TANK LENGTH, (MM)	7179
INLET PIPE LEVEL (MM)	350
OUTLET PIPE LEVEL (MM)	600
INLET & OUTLET PIPE SIZE Ø (MM)	150
VENTILATION PIPE	Ø 75MM
SIRIM STANDARD	CAN/CSA-B66-05

IMPORTANT

THIS DRAWING IS VALID FOR CONSTRUCTION PURPOSES ONLY WHEN STAMPED 'CONSTRUCTION DRAWING' BY GEMASEK FEBRITECH (M) SDN. BHD. ANY RELIANCE OTHERWISE IS AT ONE OWN RISK & COST

LEGEND:

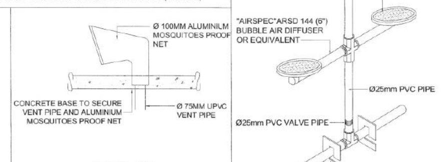
ART = AEROBIC REACTOR TANK
FST = FINAL SETTLEMENT TANK

ADDITIONAL NOTES:

- ALL INTERNAL Ø 150MM PIPE CONNECTIONS ARE FRP OR EQUIVALENT
- ALL AIR PIPES ARE PVC CLASS "D" OR EQUIVALENT IF NOT STATED. THE AIR PIPES Ø 25 MM FOR MST SYSTEMS FOR 40 PE TO 150 PE
- ALL MANHOLE/MANWAY COVER ARE FRP OR EQUIVALENT.
- ALL WIRE STRAP MUST BE SS 304 WIRE ROPE Ø12MM OR HIGHER IF NOT STATED
- ALL ANCHOR HOOK MUST BE SS Ø12MM HOOK OR HIGHER IF NOT STATED
- ALL WIRE STRAP MUST BE CLIP WITH SS304 CLIP AT Ø20MM. (BOTH SIDE)

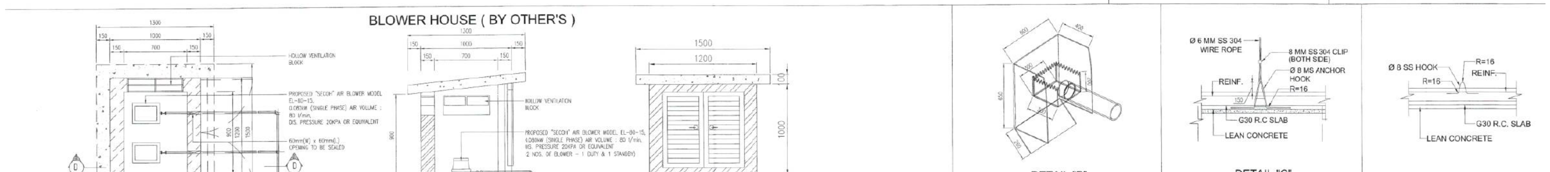
NOTES:

1. THIS DRAWING IS ISSUED BY GEMASEK FEBRITECH (M) SDN BHD (GFSB) SUBJECT TO THE CONDITION THAT IS NOT COPIED, REPRODUCED OR DISTRIBUTED EITHER IN WHOLE OR IN PART, OR USED IN ANY WAY DETRIMENTAL TO THE INTEREST OF GEMASEK FEBRITECH (M) SDN. BHD. LOADING AND PILING REQUIREMENTS
2. THE DESIGN OF THE FRP TANK DOES NOT CATER FOR TRAFFIC LOADING.
3. DISTRIBUTED LOAD FROM NEARBY TRAFFIC SHOULD BE TAKEN INTO CONSIDERATION (IF ANY) WHILE DETERMINING THE POSITION OF THE TANK.
4. THE WEIGHT OF "MST SYSTEMS" (FRP TANK AND SEWAGE) IS 4.3 TONNE/METER FOR Ø 2.4 METER TANK EXCLUDING WEIGHT OF R.C BASE SLAB AND SAND BACKFILLING. PILING
5. DESIGN SHALL BE DETERMINED OR OMITTED BY THE PROJECT CONSULTING ENGINEER BASED ON ACTUAL SOIL BEARING CAPACITY AND BOUYANCY OF THE FRP TANK. THE TANK MAY BE SUBJECT TO VERY STRONG BOUYANCY DUE TO HIGH WATER LEVEL. THE TANK MUST BE FILLED WITH WATER IMMEDIATELY AFTER POSITIONING TO AVOID ANY DAMAGE TO THE TANK.
6. THE BOUYANCY FORCE FOR :- Ø 2.4m TANK - 9 TONNE/METER - RUN
7. ANCHOR STRAPS ARE TO SECURE THE TANK IN POSITION WHILE BACKFILLING IS IN PROGRESS. THE STRAPS ALSO ASSIST IN ANCHORING THE TANK AGAINST UPLIFT BOUYANCY FORCE.
8. THE EXCAVATION FOR THE TANK INSTALLATION SHOULD BE PROPERLY BACKFILLED WITH SAND TO PROVIDE AN EVEN SUPPORT TO THE TANK.
9. ALL PIPES IN CONTACT WITH SEWAGE IN THE TANKS SHALL BE NON-CORROSIVE (GRP/FRP/PVC/UPVC CLASS D). WASTEWATER OF OIL AND GREASE NATURE (E.G. KITCHEN, RESTAURANT) SHALL BE PRETREATED BY "GREASE TRAP" BEFORE ENTERING THE "MST SYSTEM".
10. THE CONTRACTOR MUST CHECK ALL DIMENSIONS ON SITE. ONLY FIGURED DIMENSIONS ARE TO BE WORKED FROM. DISCREPANCIES MUST BE REPORTED IMMEDIATELY TO THE APPOINTED ENGINEER.
11. BEFORE PROCEEDING WITH WORK ALL DIMENSIONS ARE IN MM UNLESS OTHERWISE STATED.
11. ALL PILING FOUNDATION AND REINFORCED CONCRETE DRAWINGS MUST BE REFERED TO C&S CONSULTING ENGINEER'S DETAIL DRAWINGS.



DETAIL "A"
DETAIL OF VENTILATION PIPE
(BY OTHER'S)

DETAIL "D"
DETAIL OF DIFFUSER SUPPORT



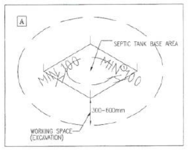
DETAIL "B"
DETAIL OF EFFLUENT LAUNDER
(V-NOTCH)

DETAIL "C"
STEEL ANCHOR HOOD AND CONCRETE BASE DETAIL
(BY OTHER'S)

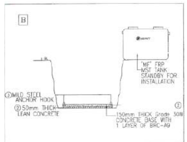
ELEVATION

INSTALLATION INSTRUCTIONS

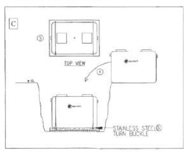
1. Allow for working space of 300m to 600m all around the FRP "MST" Tank when excavating the pit



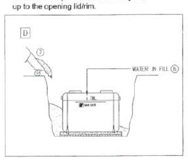
2. Construct a 150mm thick grade 30 N concrete base reinforced with one (1) layer of BRC-A9, wire mesh on top of a 50mm thick lean concrete.
3. Four (4) nos. of mild steel anchor hook coasted in concrete base.



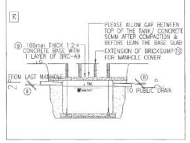
4. Unload the FRP "MST" Tank on a well leveled and cured 150mm thick Grade 30N Concrete base reinforced with one (1) layer of BRC-A9 wire mesh.
5. Tie the Ø 6mm stainless steel anchor strap over FRP "MST" Tank and tighten using stainless steel turnbuckle.



6. Fill FRP "MST" Tank with water up to top water level.
7. Back fill the space around the "MST" Tank with sand compacted in 150mm layers, up to the opening lid/rim.

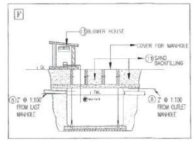


8. Make sure the backfills at the inlet and outlet are well compacted and allow for a drop gradient of two (2) degress for both the inlet and outlet pipes.
9. Lay over the FRP "MST" Tank a 100mm thick slab, with manhole openings of 1:2:4 concrete reinforced with one layer of BRC-A9 wire mesh.
10. Construct the manhole neck connections with bricks sump up to the specified design platform level.



11. Backfill with sand up to the designed platform level and install the manhole cover.
12. Fill the FRP "MST" with water to full capacity.
13. Blower installation in Blower House.

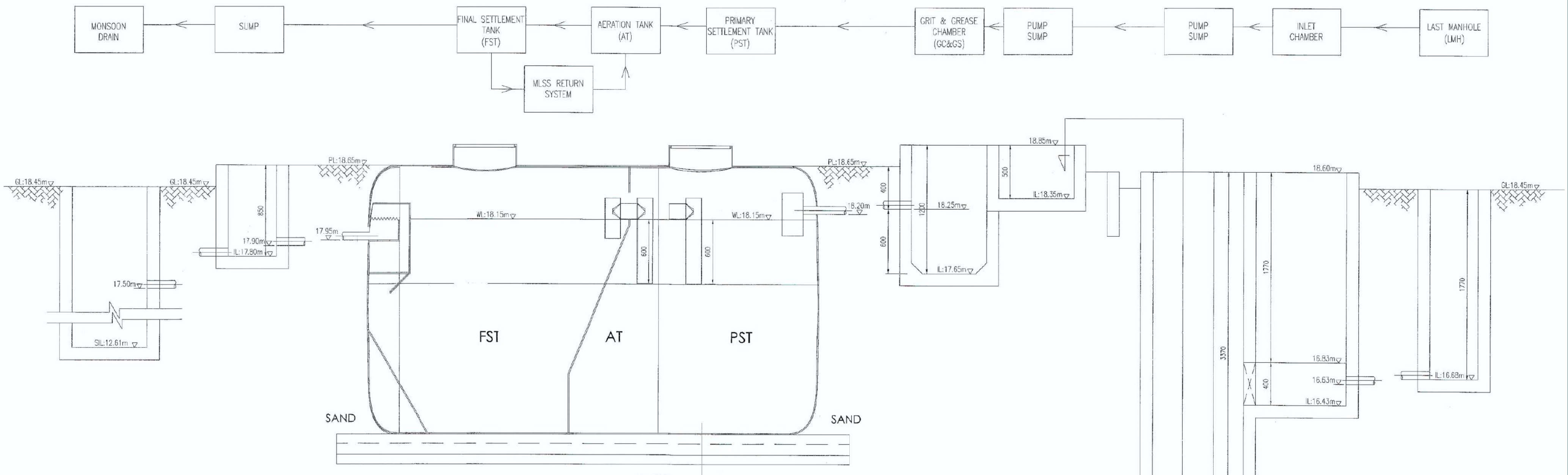
Note: Also do so every time after desludging later on. Please refer to the relevent drawings for the proper way of installation.



Date	16-03-2017
Project No	EJ 610
Produced by	FCZ
Revision	A

Details of SSTS (150 PE)

APPENDIX I-3



BLOWER (HI-BLOW OR EQUIV.)

MODEL	HP-120
POWER	0.095kW
NOS	1 DUTY

SUBMERSIBLE PUMPS (EBARA)

MODEL	100 DML 51.5
CAPACITY	12.00 L/S
POWER	1.5kW
HEAD	4.25 meter
QTY	2 UNITS

LEGEND:	
IC	INLET CHAMBER
PS	PUMP SUMP
DC	DISCHARGE CHAMBER
GC&GS	INLET CHAMBER
PST	PUMP SUMP
AT	DISCHARGE CHAMBER
FST	DISCHARGE CHAMBER
BR	INLET CHAMBER
CP	PUMP SUMP
∞	LIGHTING POST
LEGEND:	
	COMPRESSED AIR LINE (GREEN)
	SEWAGE / EFFLUENT LINE (GREY)
	SLUDGE LINE (BROWN)

ere consulting group

Process and Instrumentation Diagram with Hydraulic Profile (150 PE)

Date	16-03-2017
Project No	EJ 610
Produced by	FCZ
Revision	A

APPENDIX I-4

AP – I3 | OIL & GREASE TRAP DESIGNS

Perangkap Minyak - GTS02

SPESIFIKASI

2016 Model Baru,
Tidak lagi masalah sumbat, bau busuk.

5 year WARRANTY

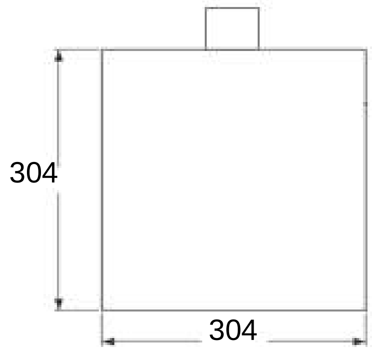
SIRIM
R018/15

SUCCESSFUL DESIGN 最成功设计

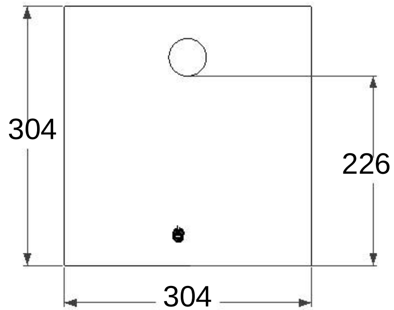
Model: GTS02
Kadar Aliran (minit) : 12 GPM
Hidangan Harian : 40-150
Saiz Paip (mm) : 40
Saiz Perangkap Minyak : 17" (L) x 12" (W) x 12" (H) inchi
Kapasiti Maksimum sisa dan minyak diperangkap : 20 liter
Kapasiti Maksimum sehingga limpahan : 40 liter
Material: Keluli 304 Tidak Berkarat 0.8 mm
Kesesuaian Kegunaan : Kopitiam/ Cafeteria/ Perumahan/ Restaurant / Bawah sinki

Cap Dagangan:

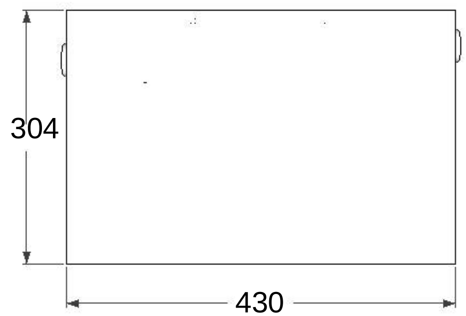
GreenVirox®



Front view - Inlet



Back view - outlet

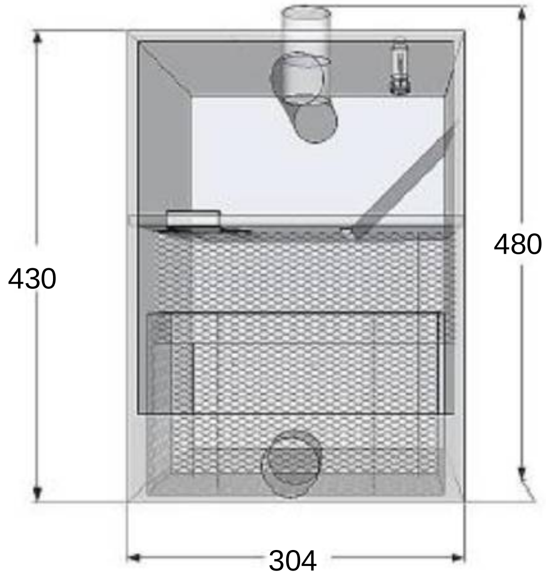


Left view

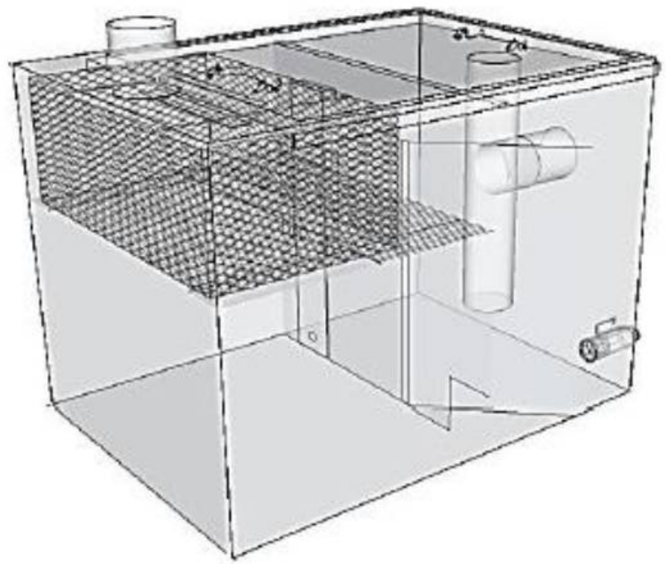
Specifications

Model: **GTS02**
Material: Stainless Steel 304
Thickness: 0.8 mm
Inlet pipe size: 1 ½ inch length 3”
Outlet pipe size: 1 ½ inch length 3”
Flow rate: 12 GPM
Capacity: 40 Liter

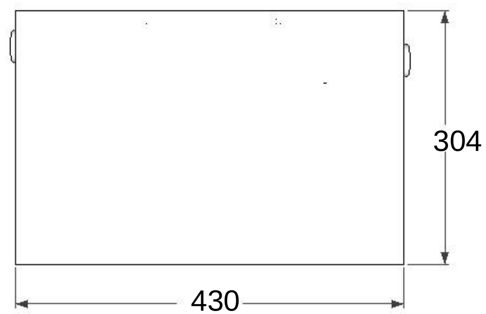
SIRIM: YES
Patent: YES
Trade Mark: EJAU
Warranty: 5 Years
Ready Stock: YES
Delivery: Between 3 days
Own Factory: YES, In Selangor



Top view



ISO view



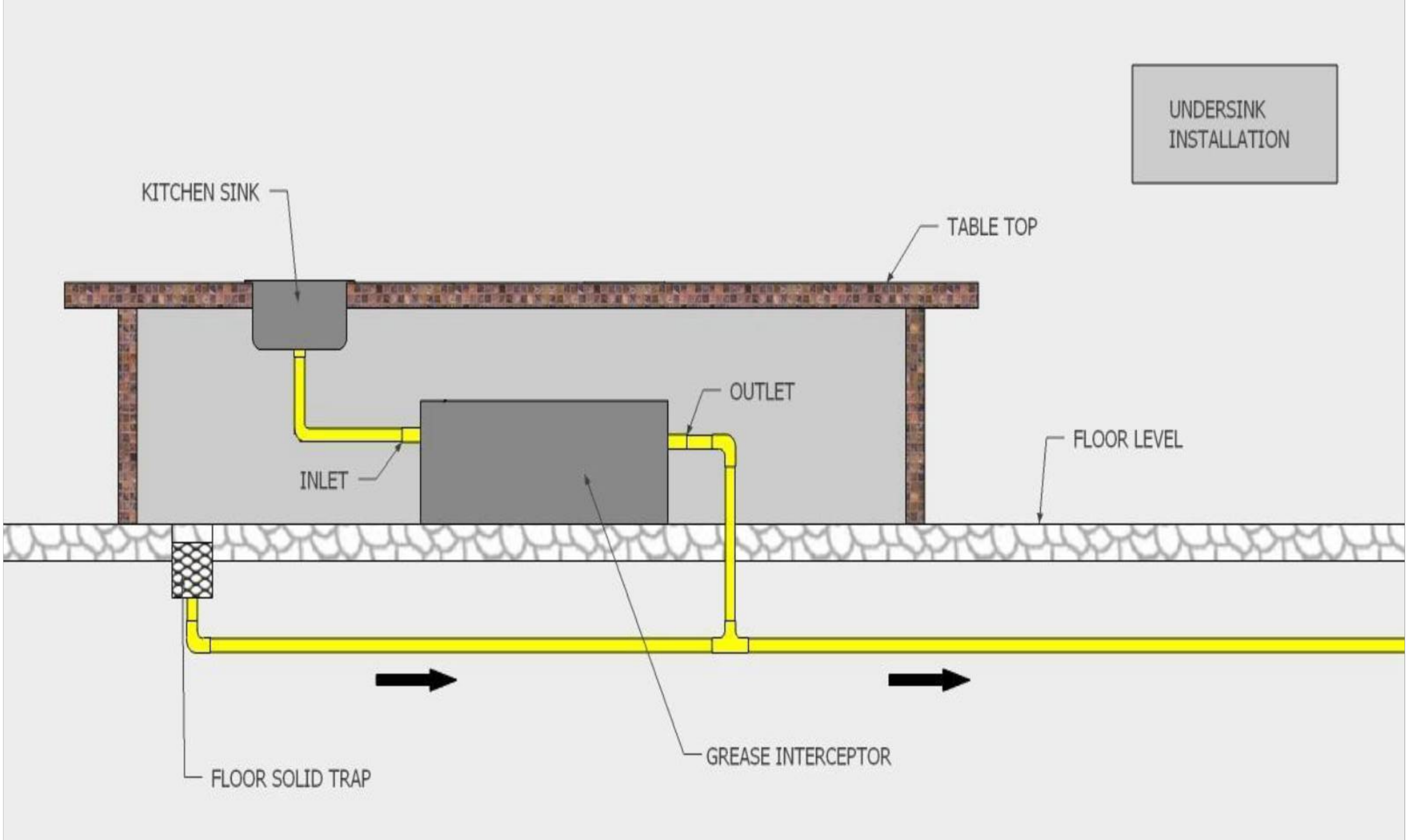
Right view

* In keeping with our policy of updating products we reserve the right to make improvements or changes.

KUALITI ALAM HIJAU (M) SDN BHD

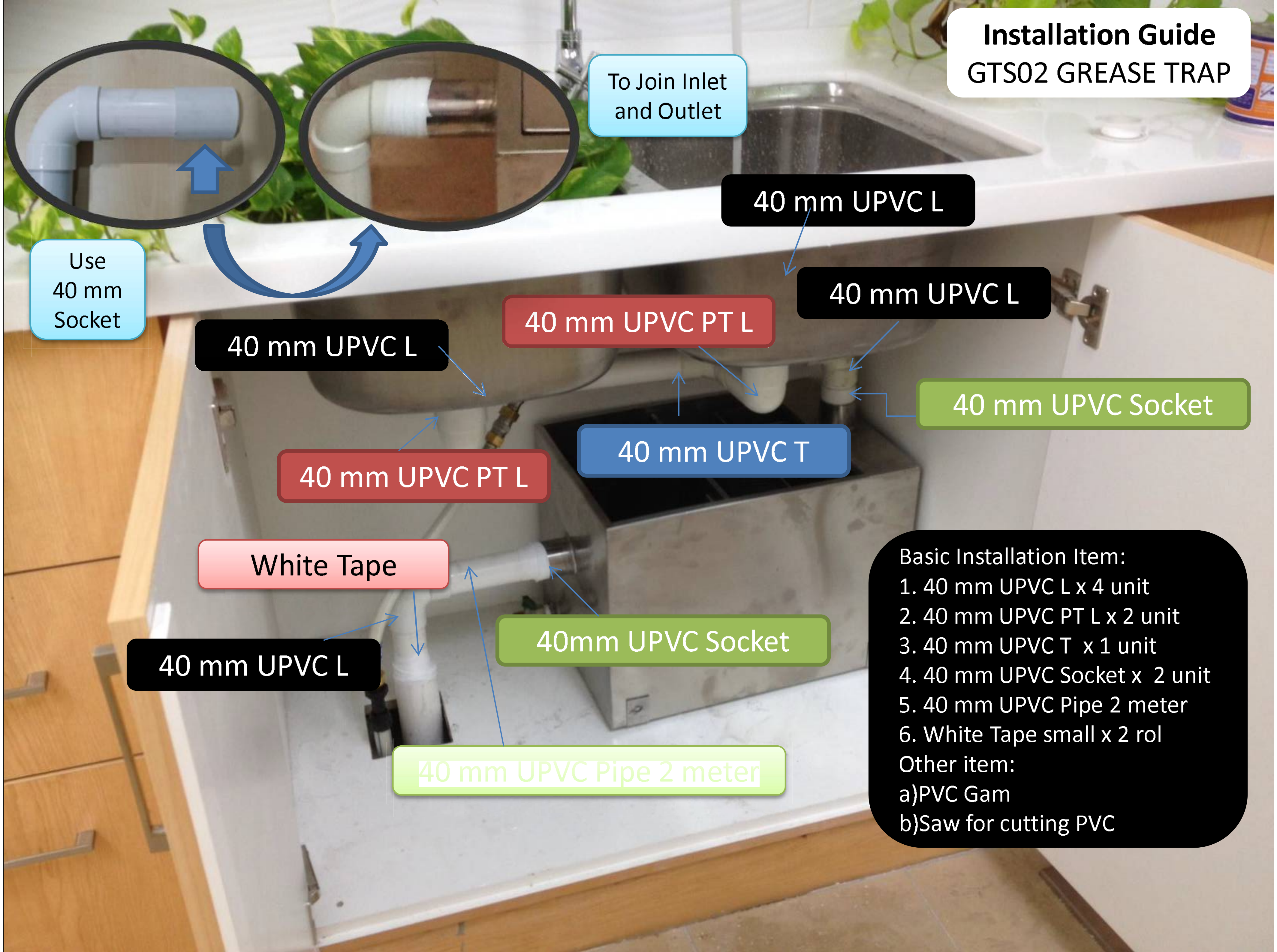
DRAWING FOR :	**DRAWING DESCRIPTION :** **Grease Interceptor** **Model: GTS02** **Material: Stainless Steel 304**	**DATE :** 1/6/2015 **NOTE :** All dimension in MM unless otherwise specified	**Drawn by :** Syharul **Checked by :** Tan Kok Hua **Approved by :** Tan Kok Hua

GREASE INTERCEPTOR INSTALATION GUIDE



KUALITI ALAM HIJAU (M) SDN BHD

DRAWING FOR :	DRAWING DESCRIPTION : Material : Stainless Steel 304 Non Rusted	DATE : 30/4/2014 NOTE : All dimension in MM unless otherwise specified	Drawn by : Syahrul Checked by : Jackson Wong Approved by : TAN

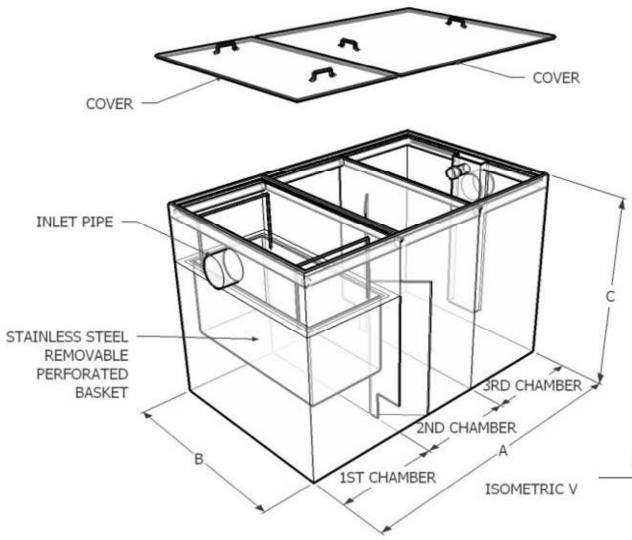


Installation Guide
GTS02 GREASE TRAP
To Join Inlet and Outlet
Use 40 mm Socket
40 mm UPVC L
40 mm UPVC L
40 mm UPVC PT L
40 mm UPVC L
40 mm UPVC Socket
40 mm UPVC T
40 mm UPVC PT L
White Tape
40mm UPVC Socket
40 mm UPVC L
40 mm UPVC Pipe 2 meter
Basic Installation Item:
1. 40 mm UPVC L x 4 unit
2. 40 mm UPVC PT L x 2 unit
3. 40 mm UPVC T x 1 unit
4. 40 mm UPVC Socket x 2 unit
5. 40 mm UPVC Pipe 2 meter
6. White Tape small x 2 rol
Other item:
a)PVC Gam
b)Saw for cutting PVC

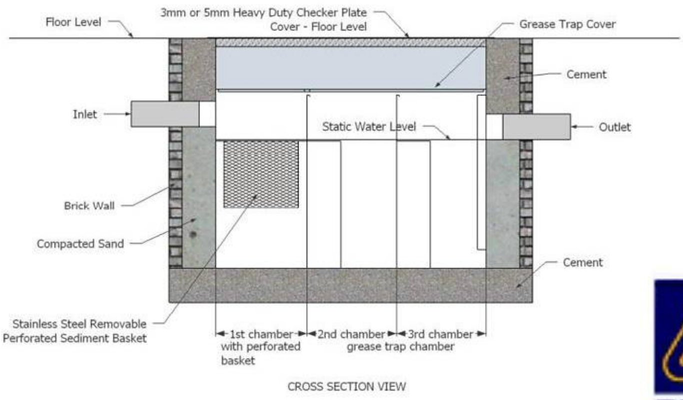
Perangkap Minyak - GTA350

SPESIFIKASI

5 year WARRANTY



SUCCESSFUL
DESIGN
最成功设计

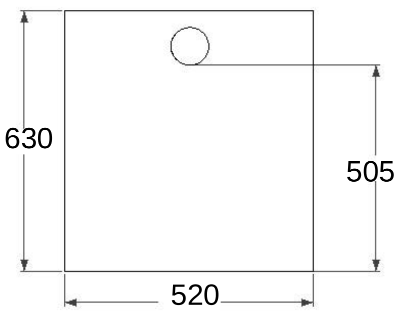


Model: GTA350
Kadar Aliran (minit) : 50GPM
Hidangan Harian : 400 to 600
Saiz Paip (mm) : 3"
Saiz Perangkap Minyak : 32" (L) x 22" (W) x 25" (H) (Inch)
Kapasiti Maksimum sisa dan minyak diperangkap : 176 Liter
Kapasiti Maksimum sehingga limpahan : 262 Liter
Material: Keluli 304 Tidak Berkarat
Kesesuaian Kegunaan : R.Mamak / Workshop /

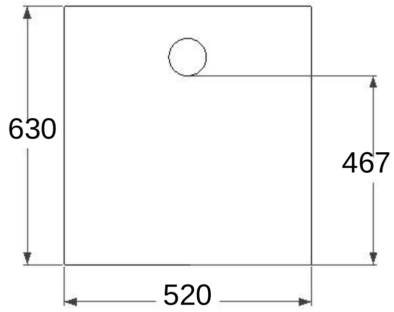
SIRIM
SIRIM TESTED

Cap Dagangan:

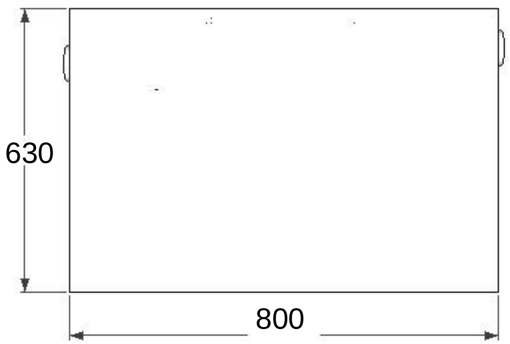
GreenVirox®



Front view - Inlet



Back view - outlet

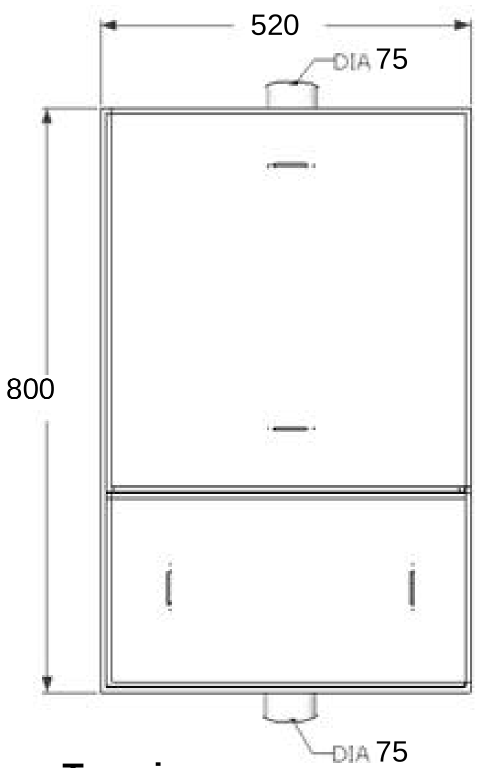


Left view

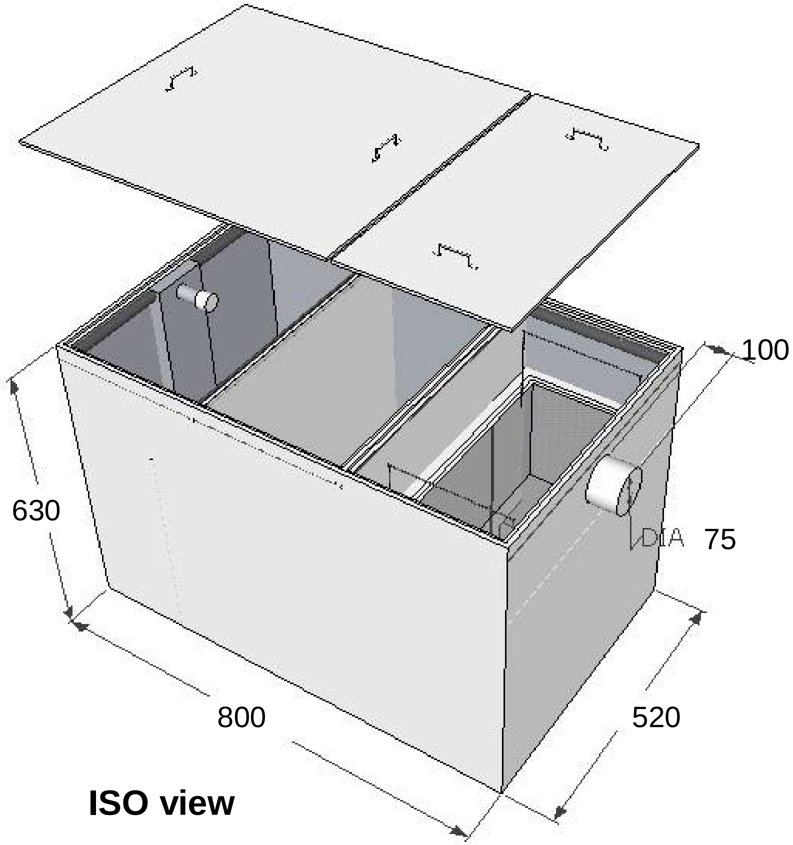
Specifications

Model: **GTA350**
Material: Stainless Steel 304
Thickness: 0.8 mm
Inlet pipe size: 3 ” inch length 4”
Outlet pipe size: 3 ” inch length 4”
Flow rate: 50 GPM
Capacity: 262 Liter

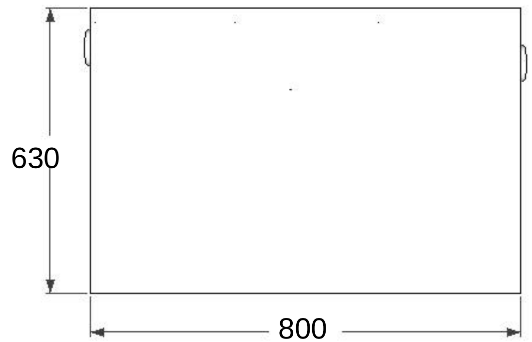
SIRIM: YES
Patent: YES
Trade Mark: EJAU
Warranty: 5 Years
Ready Stock: YES
Delivery: Between 3 days
Own Factory: YES, In Selangor



Top view



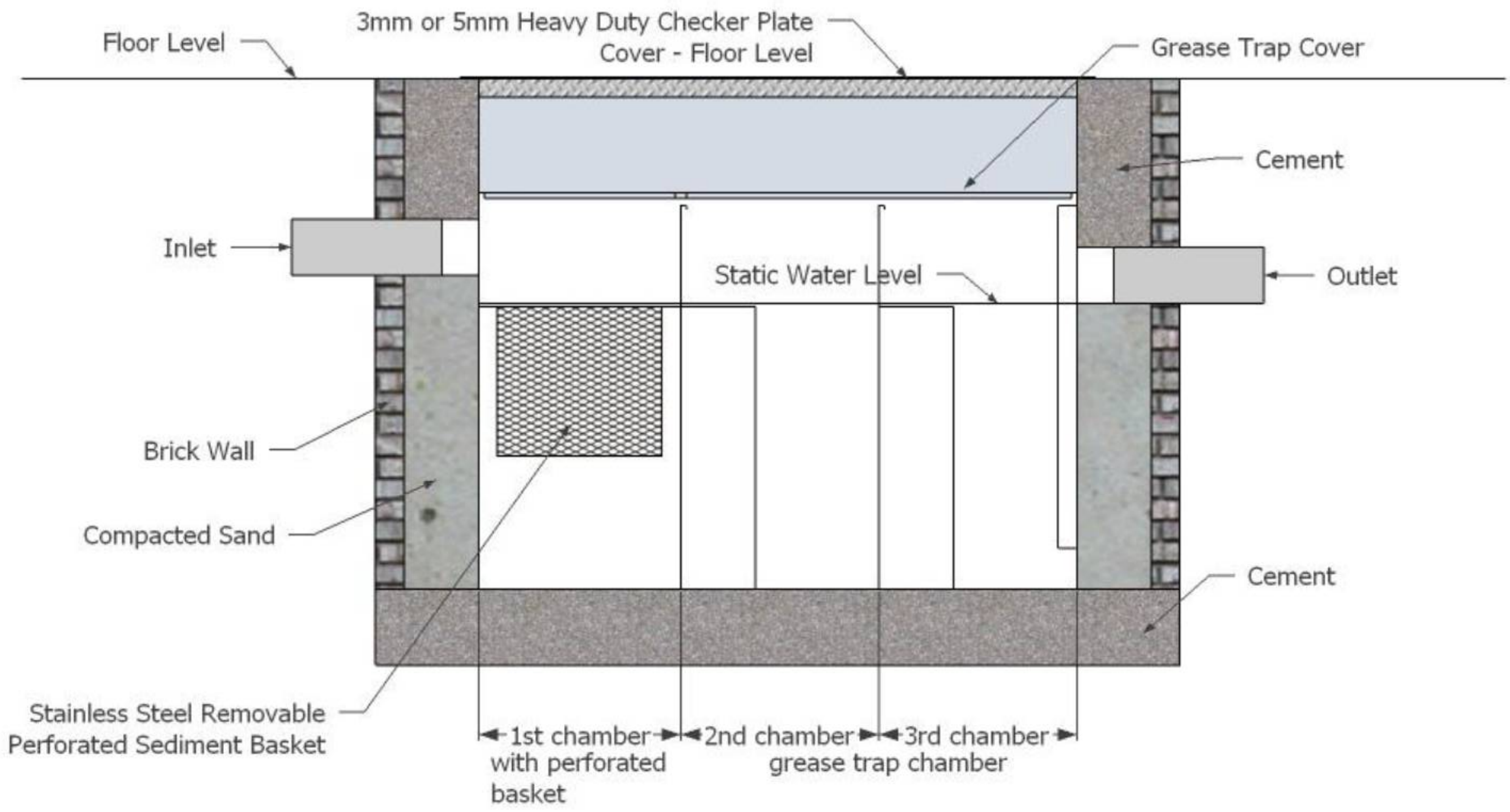
ISO view



Right view

KUALITI ALAM HIJAU (M) SDN BHD

DRAWING FOR :	**DRAWING DESCRIPTION :** Material : Stainless Steel 304 Non Rusted	**DATE : 30**/4/2014 **NOTE :** All dimension in MM unless otherwise specified	**Drawn by :** Liza **Checked by :** Jackson **Approved by :** Marizan



CROSS SECTION VIEW

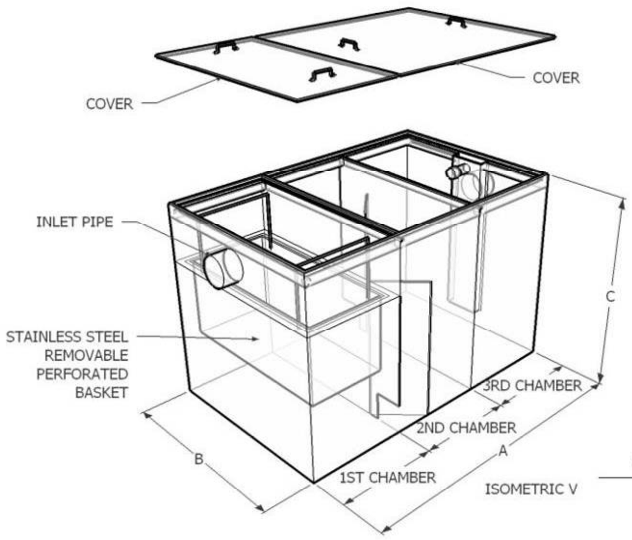
KUALITI ALAM HIJAU (M) SDN BHD

DRAWING FOR :	DRAWING DESCRIPTION :	DATE : 30/4/2014	Drawn by : Liza
North South Green Resources	Material : Stainless Steel 304 Non Rusted	**NOTE :** All dimension in MM unless otherwise specified	**Checked by :** Jackson Wong **Approved by :** Marizan

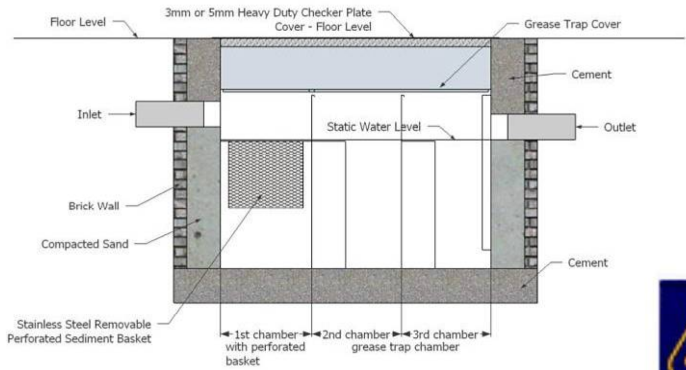
Perangkap Minyak - GTA3500

SPESIFIKASI

5 year WARRANTY



SUCCESSFUL
DESIGN
最成功设计



CROSS SECTION VIEW

Model: GTA3500

Kadar Aliran (minit) : 500 GPM

Hidangan Harian : 12000 to 15000

Saiz Paip (mm) : 6"

Saiz Perangkap Minyak : 87" (L) x 56"(W) x 62"(H) (Inch)

Kapasiti Maksimum sisa dan minyak diperangkap : 3877 Liter

Kapasiti Maksimum sehingga limpahan : 4839 Liter

Material: Keluli 304 Tidak Berkarat

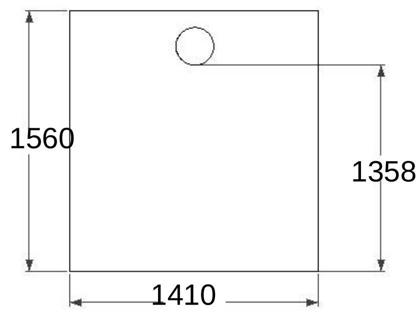
Kesesuaian Kegunaan : Kilang / Kanteen / Hotel

SIRIM

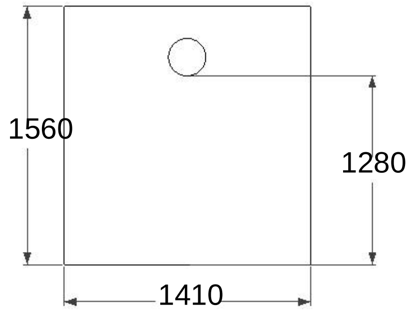
SIRIM TESTED

Cap Dagangan:

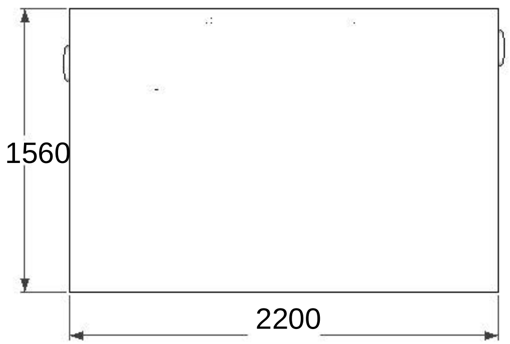
GreenVirox®



Front view - Inlet



Back view - outlet

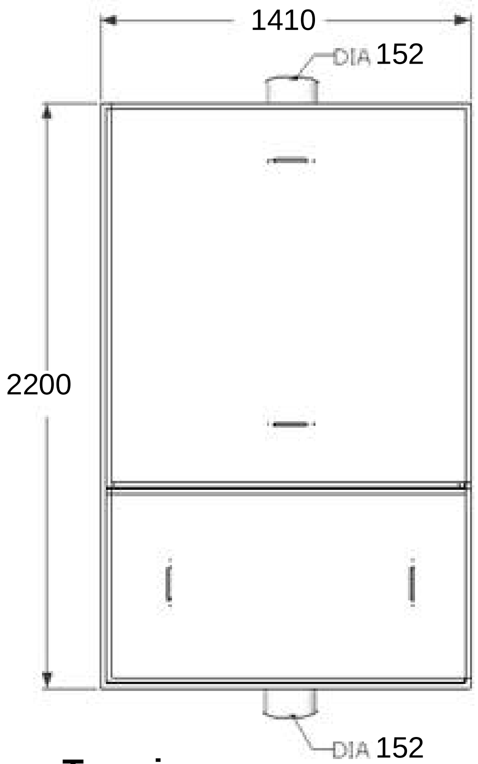


Left view

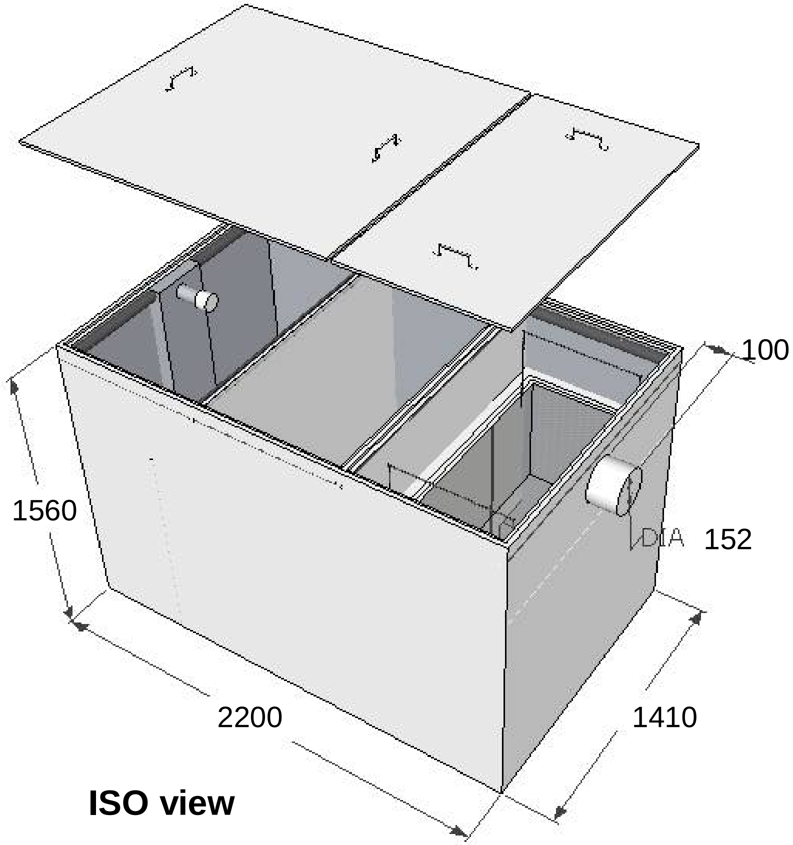
Specifications

Model: **GTA3500**
Material: Stainless Steel 304
Thickness: 1.0 mm
Inlet pipe size: 6” inch length 4”
Outlet pipe size: 6” inch length 4”
Flow rate: 500 GPM
Capacity: 4839 Liter

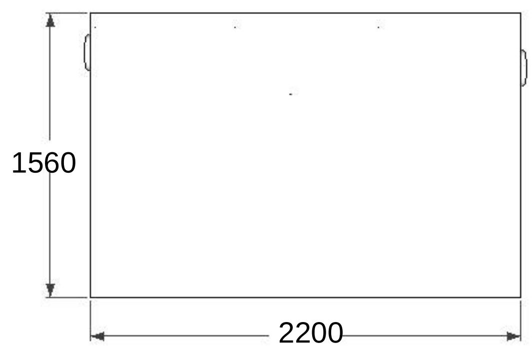
SIRIM: YES
Patent: YES
Trade Mark: EJAU
Warranty: 5 Years
Ready Stock: YES
Delivery: Between 3 days
Own Factory: YES, In Selangor



Top view



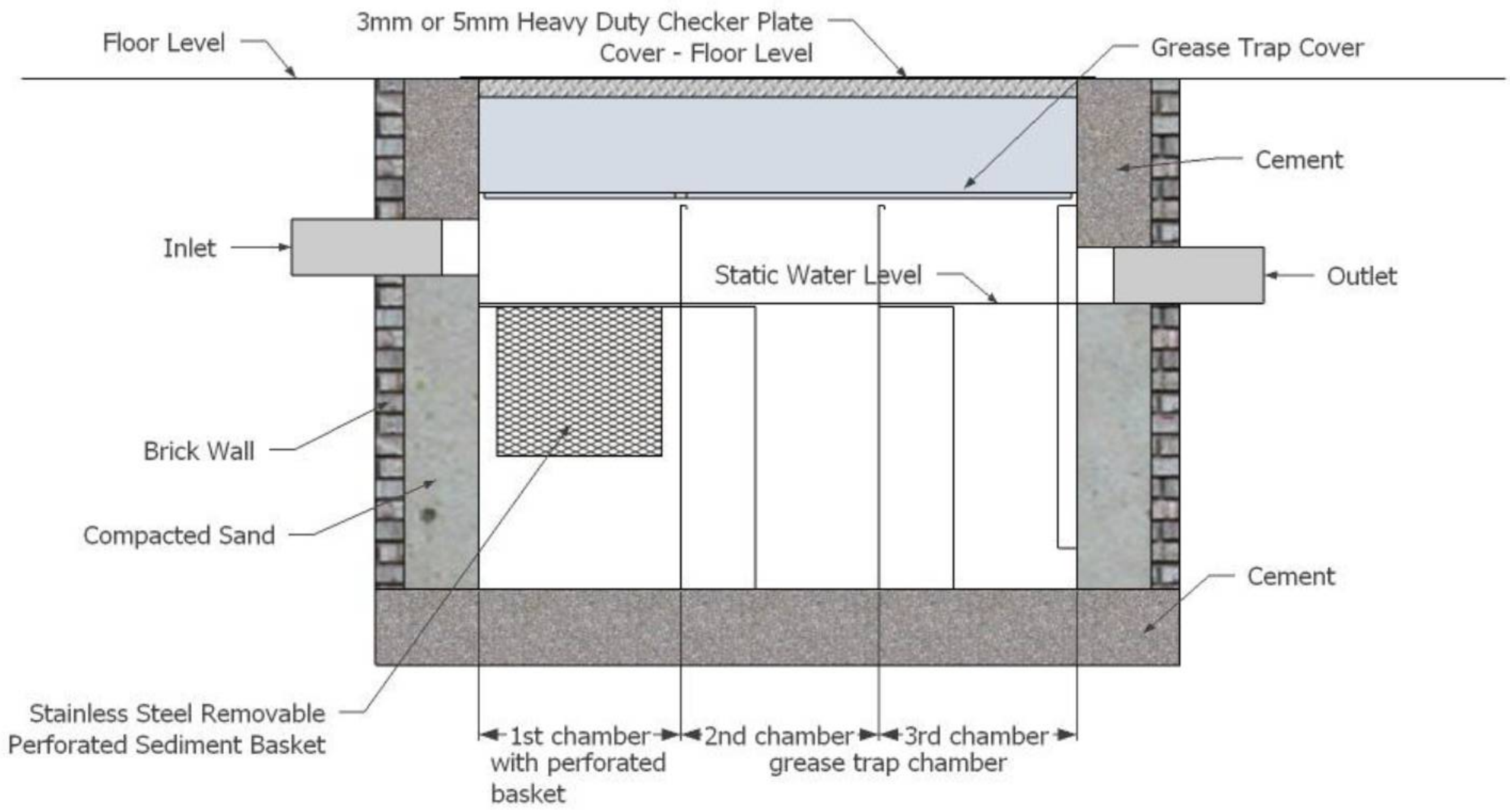
ISO view



Right view

KUALITI ALAM HIJAU (M) SDN BHD

DRAWING FOR :	**DRAWING DESCRIPTION :** Material : Stainless Steel 304 Non Rusted	**DATE : 30**/4/2014 **NOTE :** All dimension in MM unless otherwise specified	**Drawn by :** Liza **Checked by :** Jackson **Approved by :** Marizan



CROSS SECTION VIEW

KUALITI ALAM HIJAU (M) SDN BHD			
DRAWING FOR :	**DRAWING DESCRIPTION :** Material : Stainless Steel 304 Non Rusted	**DATE : 30**/4/2014 **NOTE :** All dimension in MM unless otherwise specified	**Drawn by :** Liza **Checked by :** Jackson Wong **Approved by :** Marizan

AP - J

QUAL2K

Appendix J: Water Quality Modelling

1. QUAL2K MODEL

The water quality model for Segment 2B (Sg. Serendah to Sg. Selangor) was established using the QUAL2K software developed by Chapra *et al.* (2006). QUAL2K is a stream water quality model that is a modernized version of the QUAL2E model that was developed by Brown and Barnwell (1987). QUAL2K is :

- One dimensional. The channel is assumed to be well-mixed vertically and laterally.
- The system supports branching - A mainstem river with branched tributaries.
- Steady state hydraulics. Non-uniform, steady flow is simulated.
- Diel water-quality kinetics. All water quality variables are simulated on a diel time scale.
- Heat and mass inputs. Point and non-point loads and withdrawals are simulated.

The QUAL2K framework includes the following elements:

- Software Environment and Interface. QUAL2K is implemented within the Microsoft Windows environment. Numerical computations are programmed in Fortran 90. Excel is used as the graphical user interface. All interface operations are programmed in the Microsoft Office macro language: Visual Basic for Applications (VBA).
- Model segmentation. QUAL2E segments the system into river reaches comprised of equally spaced elements. QUAL2K also divides the system into reaches and elements. However, in contrast to QUAL2E, the element size for QUAL2K can vary from reach to reach. In addition, multiple loadings and withdrawals can be input to any element.
- Carbonaceous BOD speciation. QUAL2K uses two forms of carbonaceous BOD to represent organic carbon. These forms are a slowly oxidizing form (slow CBOD) and a rapidly oxidizing form (fast CBOD).
- Chemical Oxygen Demand (COD) cannot be modeled.
- Anoxia. QUAL2K accommodates anoxia by reducing oxidation reactions to zero at low oxygen levels. In addition, denitrification is modeled as a first-order reaction that becomes pronounced at low oxygen concentrations.
- Sediment-water interactions. Sediment-water fluxes of dissolved oxygen and nutrients are simulated internally rather than being prescribed. That is, oxygen (SOD) and nutrient fluxes are simulated as a function of settling particulate organic matter, reactions within the sediments, and the concentrations of soluble forms in the overlying waters.
- Bottom algae. The model explicitly simulates attached bottom algae. These algae have variable stoichiometry.
- Light extinction. Light extinction is calculated as a function of algae, detritus and inorganic solids.
- pH. Both alkalinity and total inorganic carbon are simulated. The river's pH is then computed based on these two quantities.
- Pathogens. A generic pathogen is simulated. Pathogen removal is determined as a function of temperature, light and settling.

- Reach specific kinetic parameters. QUAL2K allows the user to specify many of the kinetic parameters on a reach-specific basis.

2. RIVER SEGMENTATION

The QUAL2K model requires the river to be represented as a series of reaches. These represent stretches of river that have constant hydraulic characteristics (e.g., slope, bottom width, etc.). The reaches are numbered in ascending order starting from the headwater of the river's main stem. Both point and non-point sources and withdrawals (abstractions) can be positioned anywhere along the channel's length.

For systems with tributaries, the reaches are numbered in ascending order starting at reach 1 at the headwater of the main stem. When a junction with a tributary is reached, the numbering continues at that tributary's headwater. Both the headwaters and the tributaries are also numbered consecutively following a sequencing scheme similar to the reaches. The major branches of the system (that is, the main stem and each of the tributaries) are referred to as segments. This distinction has practical importance because the software provides plots of model output on a segment basis.

2.1 River Representation

A 33.31 km stretch of the rivers - from Sg. Serendah (approx. 0.18 km upstream of the Serendah Komuter Station) to about 0.87 km downstream of the water intake of SSP1 along Sg. Selangor – traversing through Sg. Garing and Sg. Sembah. The hydraulic schematic model of rivers modelled is shown in **Figure 1.**

2.2 Headwaters

The model has one headwater. A headwater is the upper limit of the model but do not necessarily represent the upper limit of the catchment. Each headwater has an upper catchment area which provides the inflows that drive the model. The headwater of the model is labeled as HW0.

The details of the headwater are as follows:

Head-water ID	: 0
Headwater name	: HW0
River name	: Sg. Serendah
Reach No.	: 1
Coordinates	: 3°22'36.75"N 101°36'57.53"E
Catchment area (km^2)	: 13.5 km^2

2.3 Reaches

Reaches are sections of the river that have uniform hydraulic characteristics. The three main factors governing the flow discharge (Q_o) in a reach of a river are the flow cross-sectional area governed by the flow depth (y_o) and geometry of the river cross-section, the river channel bed slope (S) and the Manning's roughness coefficient (*n*) (**Table 1**), which is a measure of the channel boundary resistance to water flow.

$$Q_0 = f(y_0, Slope, Manning'n', Geometry)$$

Manning's Equation:

$$Q_0 = \frac{M}{n} A_0 R_0^{2/3} S_0^{1/2}$$

$M = 1.49$ imperial
$\quad 1.00$ metric
A = flow area
R = hydraulic radius
S = bed slope

Each segment of the model was classified and divided into reaches according to its hydraulic characteristics. A separate reach was established whenever a significant change in the hydraulic characteristics was observed. The channel slopes were calculated from the measured elevation at each reach.

The model has 9 reaches (**Table 2**). The determination of reaches was made by field surveys of project site, secondary data and satellite images wherein the following features were noted :

- Bottom width to determine the flow area (A) during low flow. On-site measurements of the channel width were made, where possible. Otherwise, they were estimated from survey drawings
- Channel shape (side slope)
- On-site GPS measurement of the elevations at the boundary of river reaches, to estimate the channel bed slope of the reach.

The Manning's roughness coefficient (*n*) was estimated based on observations during site visits to the project site and by comparing the conditions of the channel bottom and wetted perimeter at the channel sides with the values given in the QUAL2K Documentation Manual by Chapra *et. al* (2006) (**Table 1**).

Table 1 The Manning's Roughness Coefficient for Open Channel Surfaces

Material	***n***
Man-made channels	
Concrete	0.012
Gravel bottom with sides:	
Concrete	0.020
Mortared stone	0.023
Riprap	0.033
Natural stream channels	
Clean, straight	0.025-0.04
Clean, sinding and some weeds	0.03-0.05
Weeds and pools, winding	0.05
Mountain streams with boulders	0.04-0.10
Heavy brush, timber	0.05-0.20

Source: (from Chow *et al.* (1988), cited in QUAL2K Documentation Manual)

Table 2 Reaches in the Water Quality Model

Reach Label	Reach No	Headwater Reach	Length (km)	Upstream (km)	Down-stream (m)	Manning n
R1	1	Yes	1.59	33.31	31.72	0.035
R2	2	No	5.92	31.72	25.80	0.035
R3	3	No	5.24	25.80	20.56	0.035
R4	4	No	3.44	20.56	17.12	0.035
R5	5	No	2.18	17.12	14.94	0.035
R6	6	No	3.21	14.94	11.73	0.035
R7	7	No	3.39	11.73	8.34	0.035
R8	8	No	2.94	8.34	5.40	0.035
R9	9	No	5.40	5.40	0.00	0.035

Figure 1a Hydraulic Schematic for Segment 2B (Construction Phase)

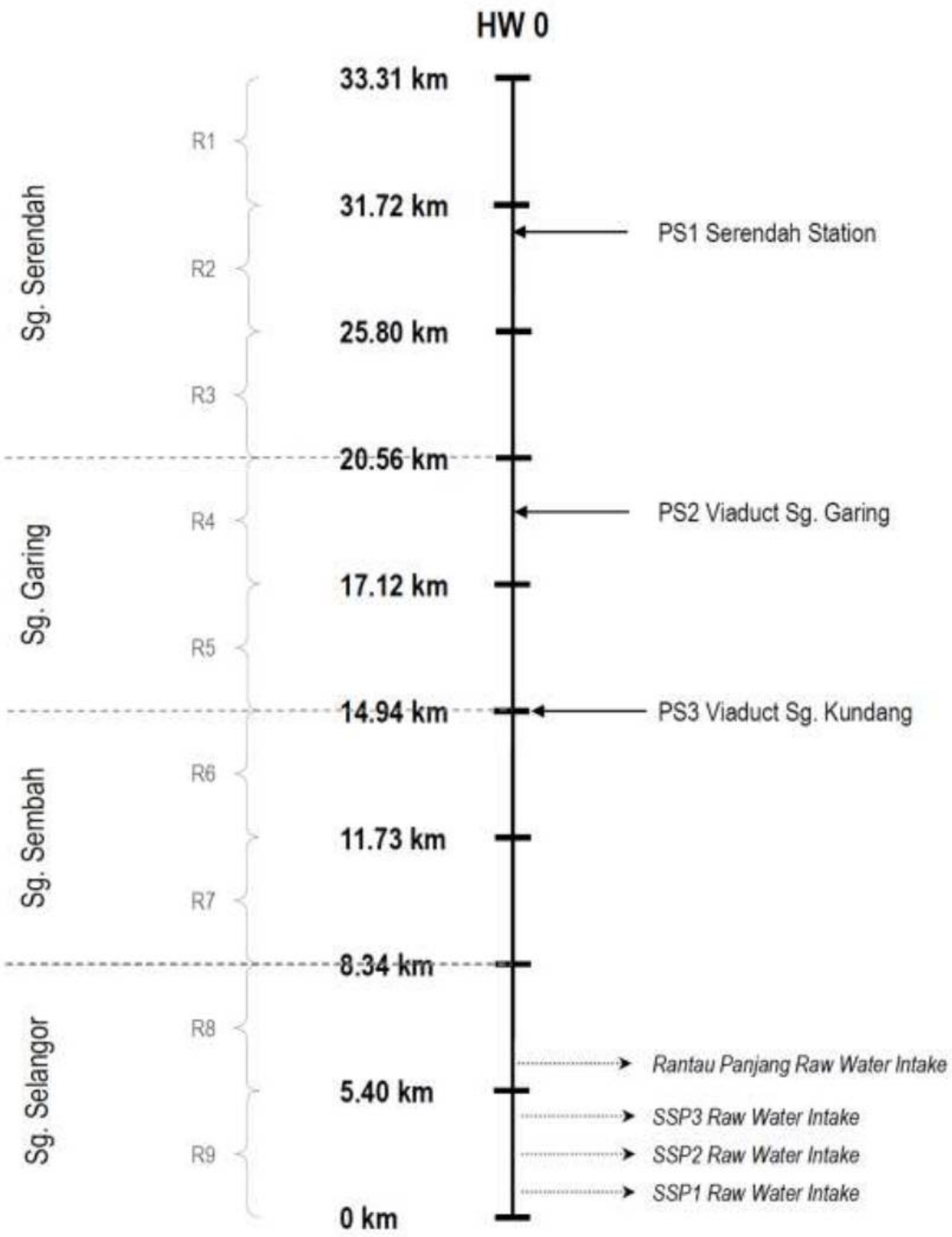
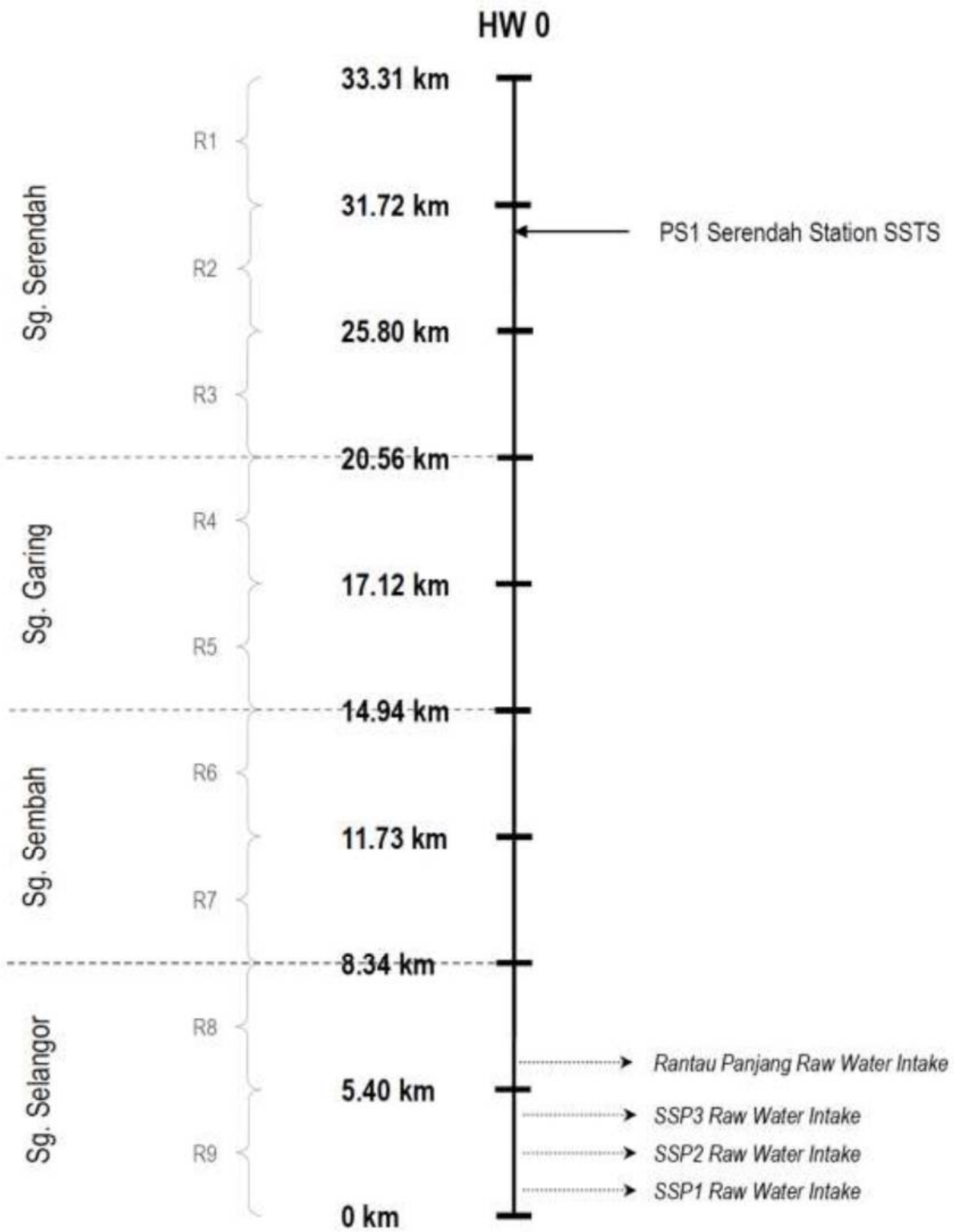


Figure 1b Hydraulic Schematic for Segment 2B (Operation Phase)



3. Flows

Three types of flow values were used in the model:

a) flow obtained from field measurements to establish the baseline model;
b) peak discharge of 2-year-return-period calculated using DID's Hydrological Procedure No. 4 for Scenario 2 during the construction phase;
c) 7Q10 low flow calculated using DID's Hydrological Procedure No. 12 for Scenario 2 during the operation phase.

4. Pollution Sources

4.1 Point sources

Tributaries (streams and drains) were modeled as point loads (see Section 2.1 above). The quantity of discharge for the tributaries was calculated using their drainage areas and the average annual low flows while the quality of discharge was determined from field sampling**.** In total, 7 point sources were included in the model during Construction Phase and 1 point source was included during Operation Phase. The details of the point sources modeled are given in Table 3.

Table 3 Point Sources input for the Model

Point source	Description	Headwater ID	Location (km)	Inflow (m^3/s)
Construction Phase				
PS1	ROW Sediment 1	0	29.81	5.9383
PS2	S13 – Serendah Station	0	29.47	1.0700
PS3	ROW Sediment 2	0	25.06	2.4127
PS4	ROW Sediment 3	0	22.41	2.5896
PS5	ROW Sediment 4	0	19.61	2.4230
PS6	Hotspot S15	0	18.50	0.4700
PS7	Hotspot S16	0	14.94	8.8790
Operation Phase				
PS1	Serendah Station SSTS	0	29.93	0.0023

4.2 Diffused sources

Diffused sources of pollution were input into the entire stretch of the model. Diffused sources entered the reaches directly. The locations and area of diffuse sources were determined from land use maps and satellite images.

The diffused pollution loadings were determined based on the Event Mean Concentration (EMC) method. Generally, the estimation of pollutant input is based on the concept that land use may be regarded as a direct indicator of the amount of pollutant released annually or, that the concentration of the pollutant which is discharged out of the catchment may be estimated. For calculating pollution loads, the formula below is applied:

$$L = R \times EMC \times A \times C_v / 100$$

where,

L = Annual pollutant load (kg/year)
R = Mean annual rainfall (mm/year)
EMC = Event mean concentration (mg/L)
A = Catchment area (ha)
C_v = Area-weighted volumetric runoff coefficient for the whole catchment.

Each type of land use will however generate different pollution characteristics and loads and therefore EMC values will need to be determined for the individual land use type.

Table 4 shows the recommended mean EMC value for various pollutants and land uses (MSMA 2nd Edition, DID, 2012) while Table 5 shows the input of diffuse sources into the model. Diffuse flows are established from flow balance calculations.

Table 4 Mean EMC Values for Selected Land Uses

Pollutant		Land Use			
Parameter	**Unit**	**Residential**	**Commercial**	**Industry**	**Highway**
TSS	mg/L	128.00	122.00	166.00	80.00
Turbidity	NTU	122.00	96.00	147.00	69.00
TDS	mg/L	131.00	43.00	137.00	38.00
pH	-	6.46	6.77	6.66	6.57
BOD	mg/L	17.90	22.90	19.30	14.90
COD	mg/L	97.00	134.00	140.00	81.00
AN	mg/L	0.73	0.85	1.00	0.44

Source: MSMA 2nd Edition, DID, 2012

Table 5 Diffuse Sources input for the Model

Diffuse Load Name	Reach no.	Location upstream (km)	Location downstream (km)	Diffuse Abstraction (m^3/s)	Diffuse Inflow (m^3/s)
D1	1	33.310	31.720	-	6.5207
D2	2	31.720	25.800	-	-
D3	3	25.800	20.560	-	0.2020
D4	4	20.560	17.120	-	1.5975
D5	5	17.120	14.940	-	-
D6	6	14.940	11.730	-	1.4358
D7	7	11.730	8.340	-	9.3494
D8	8	8.340	5.400	0.5263	-
D9	9	5.400	0.000	18.1950	-

5. Reaction Rates

The default values for water quality kinetics and reaction provided in the QUAL2K model were used for the Perwaja Drainage model (**Table 6**).

Table 6 Default Values for Water Quality Kinetics and Reaction Rates

Parameter	***Value***	***Units***	***Symbol***
Stoichiometry:			
Carbon	40	gC	gC
Nitrogen	7.2	gN	gN
Phosphorus	1	gP	gP
Dry weight	100	gD	gD
Chlorophyll	1	gA	gA
Inorganic suspended solids:			
Settling velocity	1.304	m/d	v_i
Oxygen:			
Reaeration model	O'Connor-Dobbins		
User reaeration coefficient α	0		α
User reaeration coefficient β	0		β
User reaeration coefficient γ	0		γ
Temp correction	1.024		θ_a
Reaeration wind effect	None		
O2 for carbon oxidation	2.69	gO_2/gC	r_{oc}
O2 for NH4 nitrification	4.57	gO_2/gN	r_{on}
Oxygen inhib model CBOD oxidation	Exponential		
Oxygen inhib parameter CBOD oxidation	0.60	L/mgO2	K_{socf}
Oxygen inhib model nitrification	Exponential		
Oxygen inhib parameter nitrification	0.60	L/mgO2	K_{sona}
Oxygen enhance model denitrification	Exponential		
Oxygen enhance parameter denitrification	0.60	L/mgO2	K_{sodn}
Oxygen inhib model phyto resp	Exponential		
Oxygen inhib parameter phyto resp	0.60	L/mgO2	K_{sop}
Oxygen enhance model bot alg resp	Exponential		
Oxygen enhance parameter bot alg resp	0.60	L/mgO2	K_{sob}
Slow CBOD:			
Hydrolysis rate	4.999	/d	k_{hc}
Temp correction	1.047		θ_{hc}
Oxidation rate	5	/d	k_{dcs}
Temp correction	1.047		θ_{dcs}
Fast CBOD:			
Oxidation rate	5	/d	k_{dc}
Temp correction	1.047		θ_{dc}
Organic N:			
Hydrolysis	0	/d	k_{hn}
Temp correction	1.07		$\square_{hn}$

Settling velocity	0	m/d	v_{on}
Ammonium:			
Nitrification	1.649	/d	k_{na}
Temp correction	1.07		$\square_{na}$
Nitrate:			
Denitrification	0	/d	k_{dn}
Temp correction	1.07		$\square_{dn}$
Sed denitrification transfer coeff	0	m/d	v_{di}
Temp correction	1.07		$\square_{di}$
Organic P:			
Hydrolysis	0	/d	k_{hp}
Temp correction	1.07		$\square_{hp}$
Settling velocity	1.999	m/d	v_{op}
Inorganic P:			
Settling velocity	0	m/d	v_{ip}
Inorganic P sorption coefficient	0.073	L/mgD	K_{dpi}
Sed P oxygen attenuation half sat constant	1.831	mgO_2/L	k_{spi}
Phytoplankton:			
Max Growth rate	2.5	/d	k_{gp}
Temp correction	1.07		$\square_{gp}$
Respiration rate	0.1	/d	k_{rp}
Temp correction	1.07		$\square_{rp}$
Excretion rate	0	/d	k_{ep}
Temp correction	1.07		$\square_{dp}$
Death rate	0	/d	k_{dp}
Temp correction	1		$\square_{dp}$
External Nitrogen half sat constant	15	ugN/L	k_{sPp}
External Phosphorus half sat constant	2	ugP/L	k_{sNp}
Inorganic carbon half sat constant	2.00E-05	moles/L	k_{sCp}
Light model	Half saturation		
Light constant	57.6	langleys/d	K_{Lp}
Ammonia preference	25	ugN/L	k_{hnxp}
Subsistence quota for nitrogen	0	mgN/mgA	q_{0Np}
Subsistence quota for phosphorus	0	mgP/mgA	q_{0Pp}
Maximum uptake rate for nitrogen	0	mgN/mgA/d	$\square_{mNp}$
Maximum uptake rate for phosphorus	0	mgP/mgA/d	$\square_{mPp}$
Internal nitrogen half sat constant	0	mgN/mgA	K_{qNp}
Internal phosphorus half sat constant	0	mgP/mgA	K_{qPp}
Settling velocity	0.15	m/d	v_a
Bottom Algae:			
Growth model	Zero-order		
Max Growth rate	999.991	$mgA/m^2/d$ or /d	C_{gb}
Temp correction	1.07		$\square_{gb}$

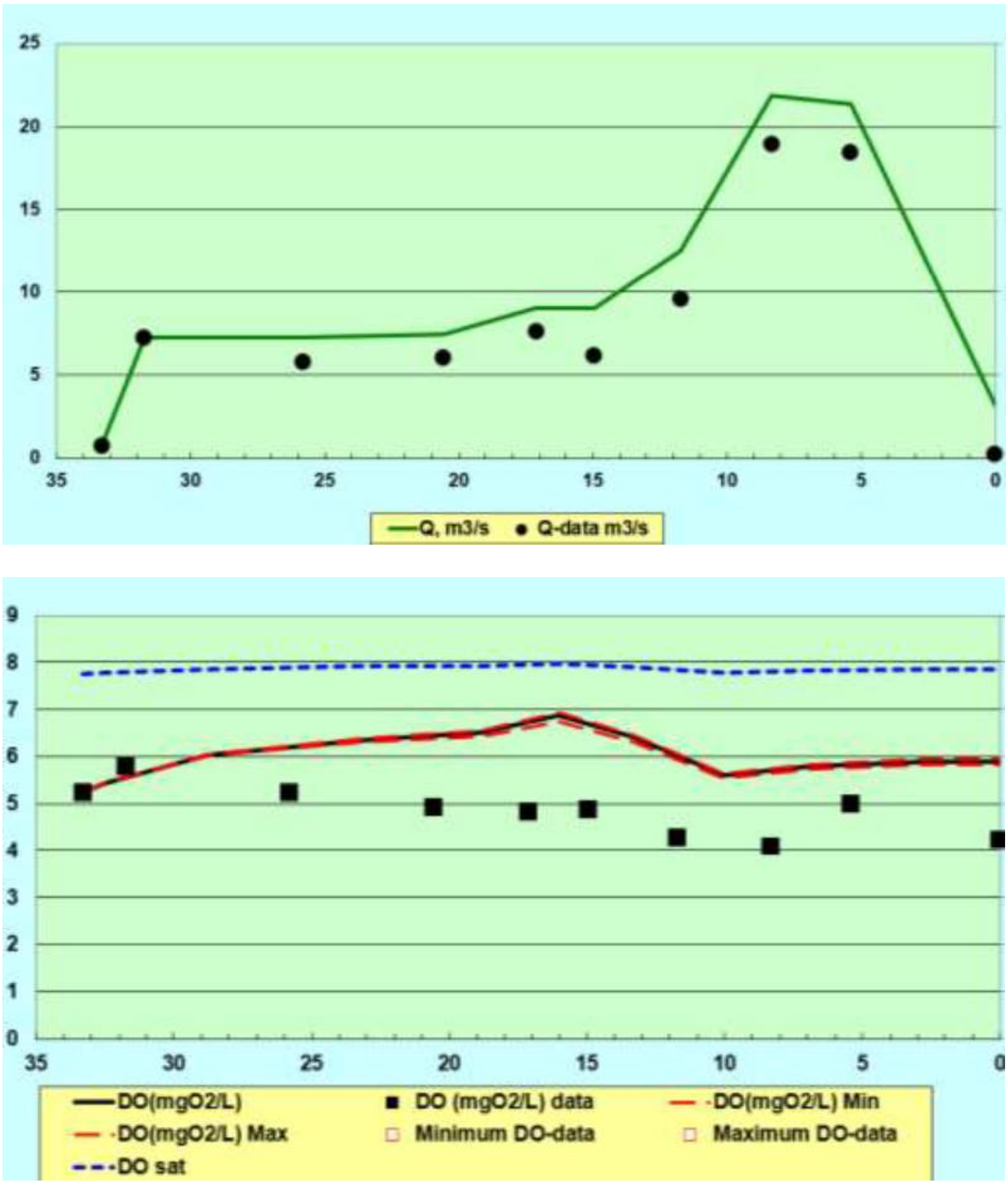
First-order model carrying capacity	1000	mgA/m^2	$a_{b,max}$
Respiration rate	1	/d	k_{rb}
Temp correction	1.07		□$_{rb}$
Excretion rate	0.5	/d	k_{eb}
Temp correction	1.05		□$_{db}$
Death rate	0.09	/d	k_{db}
Temp correction	1.07		□$_{db}$
External nitrogen half sat constant	0.052	ugN/L	k_{sPb}
External phosphorus half sat constant	96.379	ugP/L	k_{sNb}
Inorganic carbon half sat constant	1.00E-05	moles/L	k_{sCb}
Light model	Half saturation		
Light constant	76.319	langleys/d	K_{Lb}
Ammonia preference	99.982	ugN/L	k_{hnxb}
Subsistence quota for nitrogen	2.524	mgN/mgA	q_{0N}
Subsistence quota for phosphorus	0.002	mgP/mgA	q_{0P}
Maximum uptake rate for nitrogen	149.913	mgN/mgA/d	□$_{mN}$
Maximum uptake rate for phosphorus	5.009	mgP/mgA/d	□$_{mP}$
Internal nitrogen half sat constant	0.384	mgN/mgA	K_{qN}
Internal phosphorus half sat constant	0.102	mgP/mgA	K_{qP}
Detritus (POM):			
Dissolution rate	7.179	/d	k_{dt}
Temp correction	1.07		□$_{dt}$
Fraction of dissolution to fast CBOD	1.00		F_f
Settling velocity	0.236	m/d	v_{dt}
Pathogens:			
Decay rate	0.8	/d	k_{dx}
Temp correction	1.07		□$_{dx}$
Settling velocity	1	m/d	v_x
Light efficiency factor	1.00		□$_{path}$
pH:			
Partial pressure of carbon dioxide	347	ppm	p_{CO2}
Constituent i			
First-order reaction rate	0	/d	
Temp correction	1		□$_{dx}$
Settling velocity	0	m/d	v_{dt}
Constituent ii			
First-order reaction rate	0	/d	
Temp correction	1		□$_{dx}$
Settling velocity	0	m/d	v_{dt}
Constituent iii			
First-order reaction rate	0	/d	
Temp correction	1		□$_{dx}$
Settling velocity	0	m/d	v_{dt}

6. Model Calibration

The water quality model was calibrated using field data (flow and dissolved oxygen) collected in August 2017.

Figure 2 shows the predicted and the observed water quality along the rivers modelled. Some variance in the correlation between the predicted and observed data can be expected, as the model predicts a steady-state water quality whereas the observed data captures the water quality at a particular point in time. The limitations of the model are due to estimations of hydraulic and water quality data along sub-tributaries and incomplete pollution source data, such as inflows and effluent quality of the pollution sources. Reaction rates and kinetics also play an important role in determining the outcome of the model.

Figure 2 Model calibration of the water quality model for Flow and Dissolved Oxygen



7. SCENARIO MODELED

For both Construction and Operation Phases, in addition to the baseline scenario which simulates the changes in the water quality without the proposed project, two scenarios were modeled for this study. During the Construction Phase, the scenarios are:

i. Scenario 1 (with Mitigation Measures)

   The first scenario with the proposed project models the changes in water quality as a result of sediment contribution from the ROW and station construction sites of the Project. The maximum sediment contribution from the Project is 50 mg/L with implementation of mitigation measures.

ii. Scenario 2 (Worst Case with Mitigation Measures):

   The Worst Case Scenario simulates the outcome when rainfall event occurs and sediment runoff increases drastically. The sediment contribution from the Project is 1,000 mg/L with implementation of mitigation measures at peak discharge of 2-year-return-period.

During the Operation Phase, the scenarios are:

i. Scenario 1 (Normal)

   Sewage treatment systems will discharge effluent that meets Standard A limits of the Environmental Quality (Sewage) Regulations 2009 at discharge flow of 0.0023 m^3/s.

ii. Scenario 2 (Worst Case):

   Sewage treatment systems will discharge effluent that meets Standard A limits of the Environmental Quality (Sewage) Regulations 2009 at discharge flow of 0.0023 m^3/s but during 7Q10 low flow conditions.

8. MODELING RESULTS

The impact of the Project to the water quality of the river during the Construction Phase is shown in **Chart 1** while during Operation Phase the results are shown in **Charts 2 to 4** below.

9. ASSUMPTIONS AND LIMITATIONS OF THE MODEL

The water quality model is assumed to be reflective of the actual conditions based on calibration of selected parameters only, i.e. dissolved oxygen, BOD, pH, and TSS. Default values of the water quality kinetics, reaction rates and constants have been used, as local data is very limited.

Chart 1 Impact of Proposed Project on Suspended Solids Concentration during Construction Phase

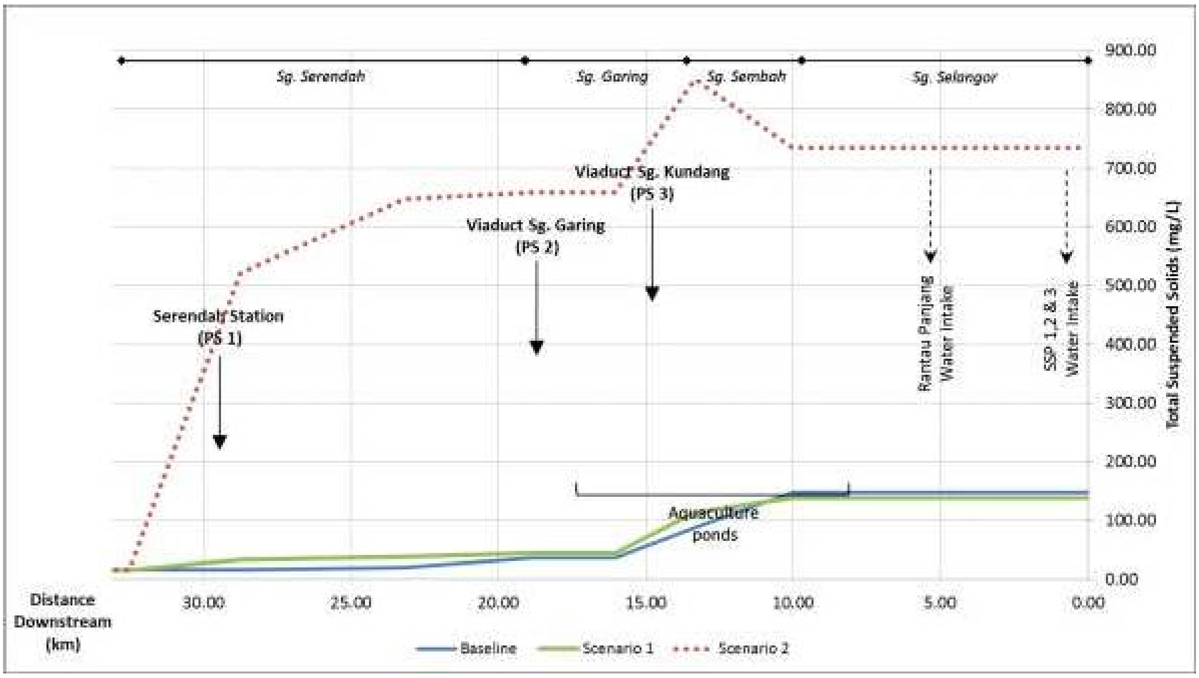


Chart 2 Impact of Proposed SSTS Effluent Discharge on Suspended Solids Concentration during Operation Phase

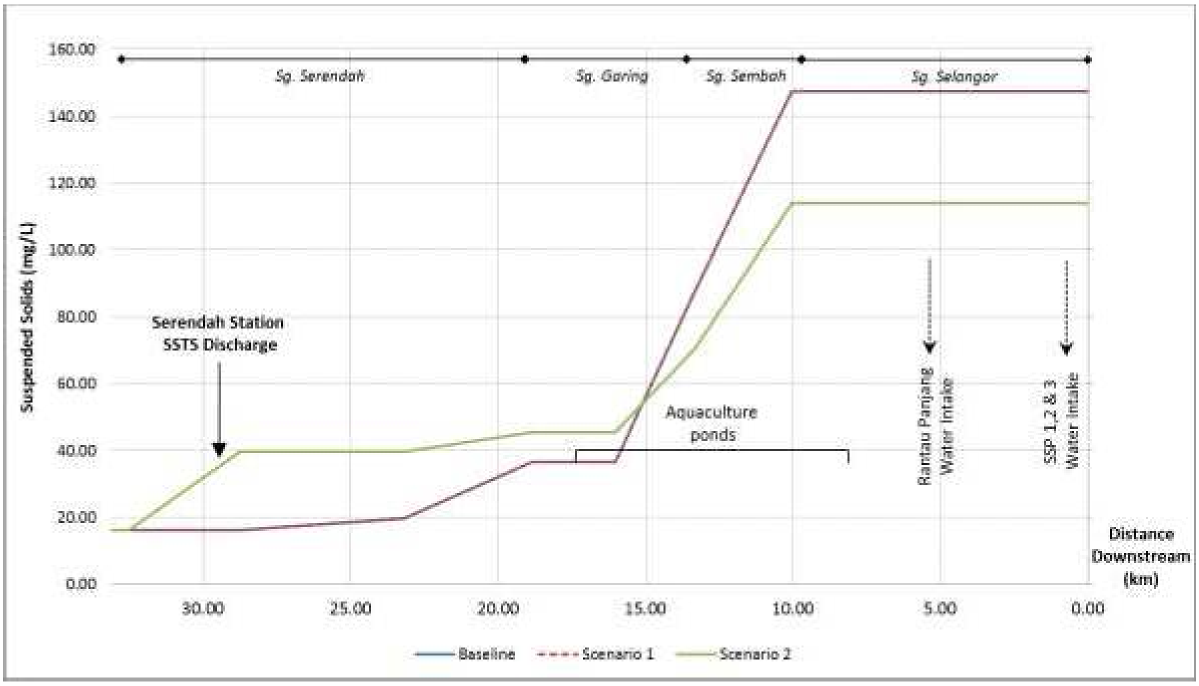


Chart 3 Impact of Proposed SSTS Effluent Discharge on Biochemical Oxygen Demand (BOD) Concentration during Operation Phase

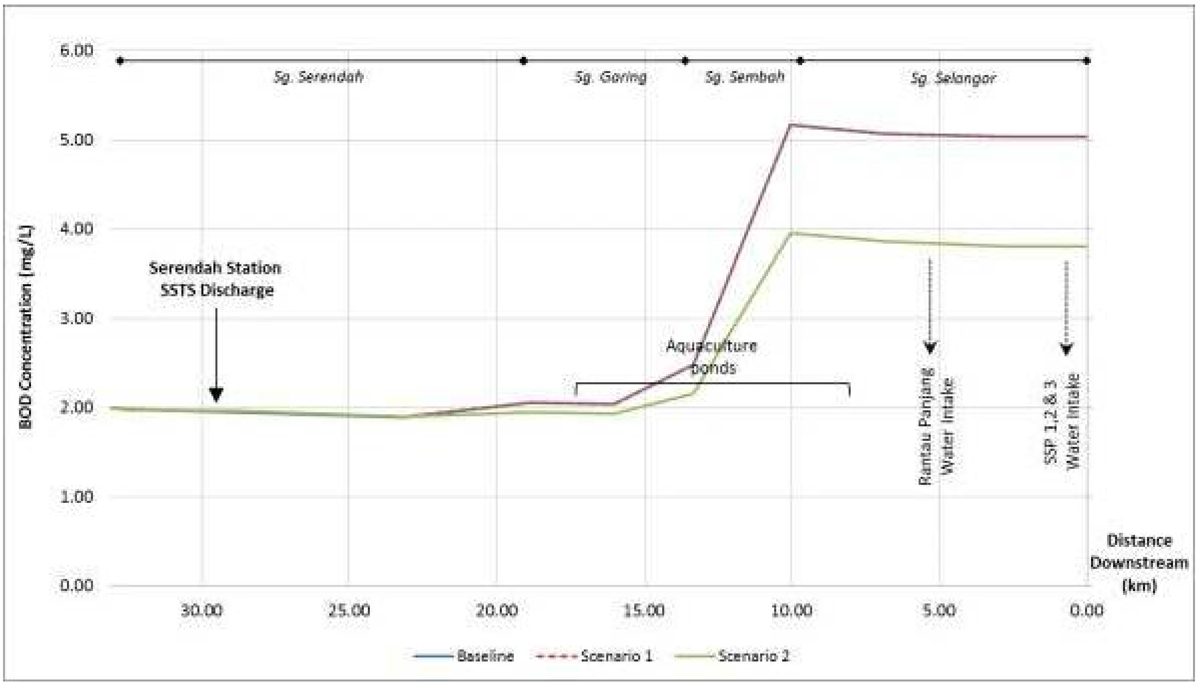
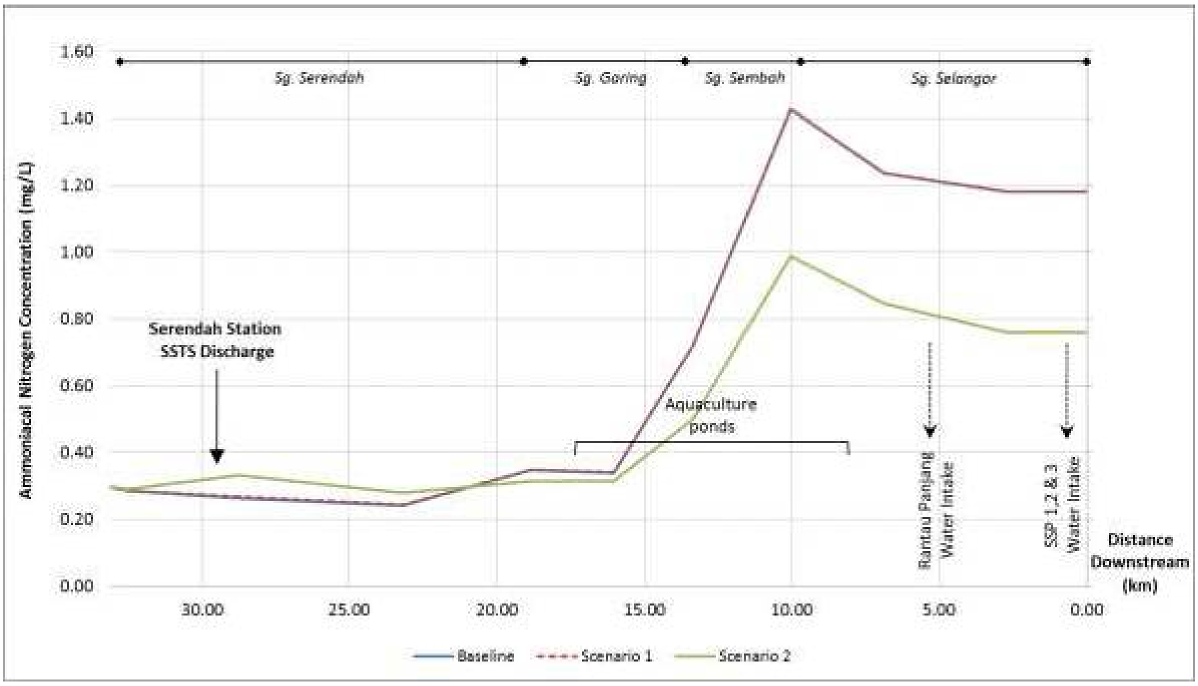


Chart 4 Impact of Proposed SSTS Effluent Discharge on Ammoniacal Nitrogen Concentration during Operation Phase



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