Section 5

Requirements for Individual Treatment Processes

5.1 Introduction

All new applications for sewage treatment plant approval shall follow the design requirements as stipulated in this section. These requirements have been formulated as a gradual change in sewage treatment methods for Malaysia prior to enforcement of ultimate requirements as stipulated in Sections 3 and 4 of this volume.

Design requirements for each stage of the sewage treatment process, as shown in Figure 5.1 are given in this section.

Figure 5.2 gives an overview of the typical flow diagram and elements of a sewage treatment plant. Figure 5.2 also shows how one facility is closely related to another and thus has an impact upon the overall design.

Sewage treatment plants must be designed to produce an effluent quality that conforms to either Standard A or Standard B or any other special requirements under the provisions of the Environmental Quality Act.

The major indices are those of BOD_5, Suspended Solids, COD, Oil & Grease, Ammoniacal Nitrogen, Nitrate Nitrogen and Total Phosphorus.

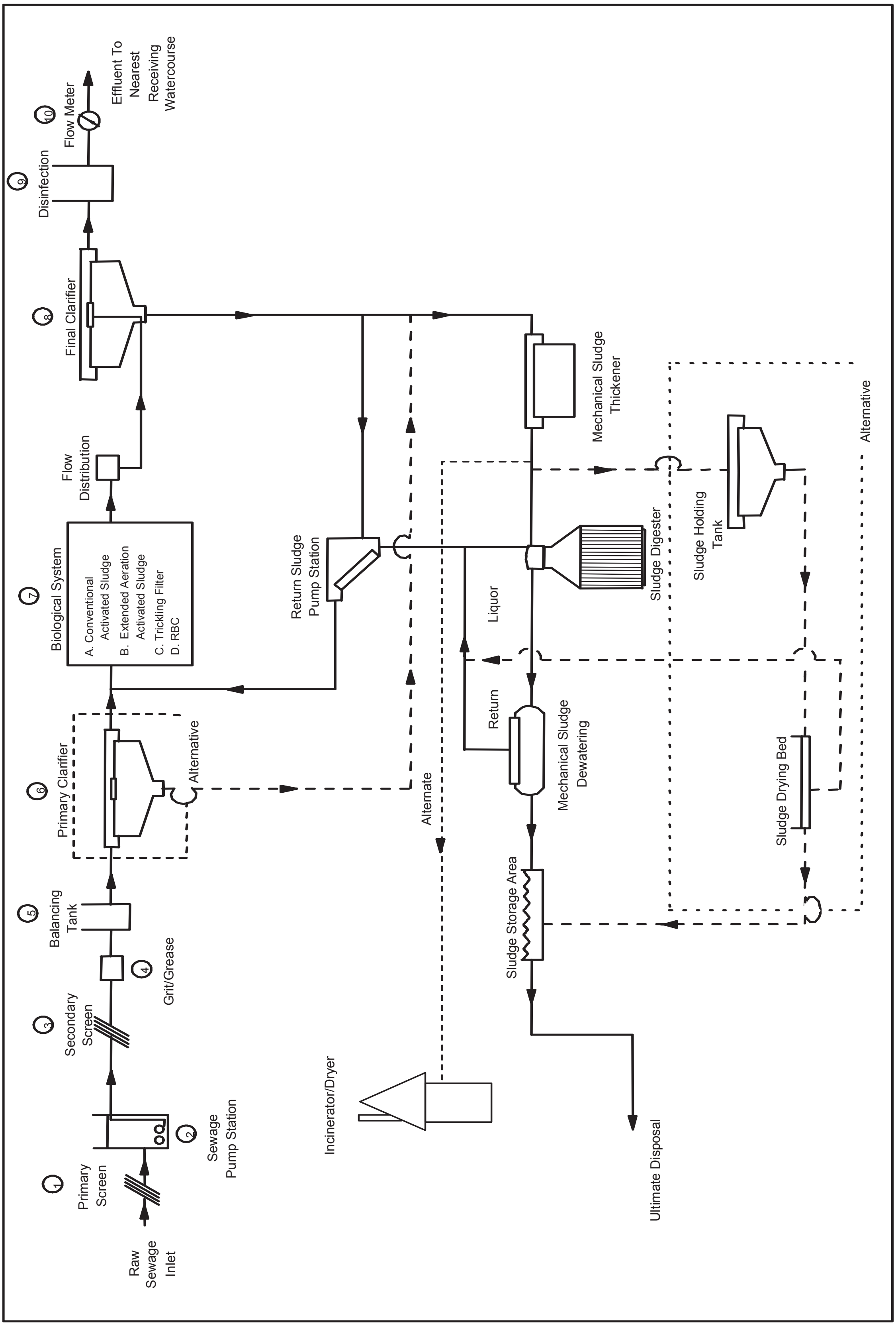
The requirement to comply with absolute standards, where no failures are permitted by law, means that new sewage treatment plants must be designed to produce average effluent qualities well below those permitted by the Standard figures. Design values for final effluent shall be used in the design of new treatment works are given in Table 3.2. These design effluent levels serve as the basis for the design requirement of each unit process given in the following sub sections.

General ventilation systems shall be provided in compliance to the OSHA. The potential for odour generation, its impact and treatment, shall be considered in all aspects of design. Odour treatment equipment shall be selected that such odours be reduced to the lowest possible level and in compliance to the EQA.

Figure 5.1 Typical Treatment Process Flow Chart

Treatment Stage		Treatment Processes	Requirements Mandatory (M) Optional (O)	Design Requirements	Function
Pre Treatment	1	Primary Screen	M	Section 5.2	Removes rocks, roots and rags
	2	Pump Station	M/O	Section 5.3	Lifts sewage and provides consistent flow to the treatment system
	3	Secondary Screen	M	Section 5.4	Removes smaller/finer particles from sewage
	4	Grit/Grease Removal	M	Section 5.5	Removes sand, gravel and other inorganic materials; separates oil & grease
	5	Balancing Tank	M/O	Section 5.6	Balances and equalises flow
Primary Treatment	6	Primary Sedimentation	M/O	Section 5.7	Removes settleable solids/materials
Secondary Treatment	7	Biological Treatment	M	Section 5.8	Remove major polutants (BOD and SS)
	8	Secondary Sedimentation	M	Section 5.9	Separates treated effluent and settled sludge
	9	Disinfection	M	Section 5.10	Destroy disease causing organisms
	10	Flow Meter	M	Section 5.11	Measures and records flows
Bio Solids Handling	11	Thickener Stabilisation Holding Dewatering	M/O	Section 5.12	Reduces potential detrimental effect on the environment and converts sludge to a form suitable for ultimate disposal
		Sludge Disposal			

Figure 5.2 Typical Elements and Process Flow Diagram of a Sewage Treatment Plant



5.2 Design of Primary Screens

5.2.1 Purpose of Primary Screens

Upon reaching the sewage treatment plant, sewage flows through the primary screening facility which is the first stage of treatment. The screens must be provided upstream of all inlet pump stations and shall be designed to protect downstream processes and equipment. The purposes of primary screens are:

a) to protect equipment from rags, wood and other debris

b) to reduce interference with in-plant flow and performance

5.2.2 Inlet Chamber

Provision for inlet chamber before the primary screen channel is necessary for proper operational and maintenance. The summarised requirements for inlet chamber are as follows:

Table 5.1 Requirements for Inlet Chamber

Unit Process		Requirements			Notes
Inlet Chamber		Mandatory			Single and dual penstocks are referring to members of penstocks required. For more than 50 000 PE, the main penstock must be motorised.
	Type	≤ 20 000 PE	> 20 000 PE	> 50 000 PE	
	Single	Yes	n/a	n/a	
	Dual	n/a	Yes	Yes	
	Motorised	n/a	No	Yes	

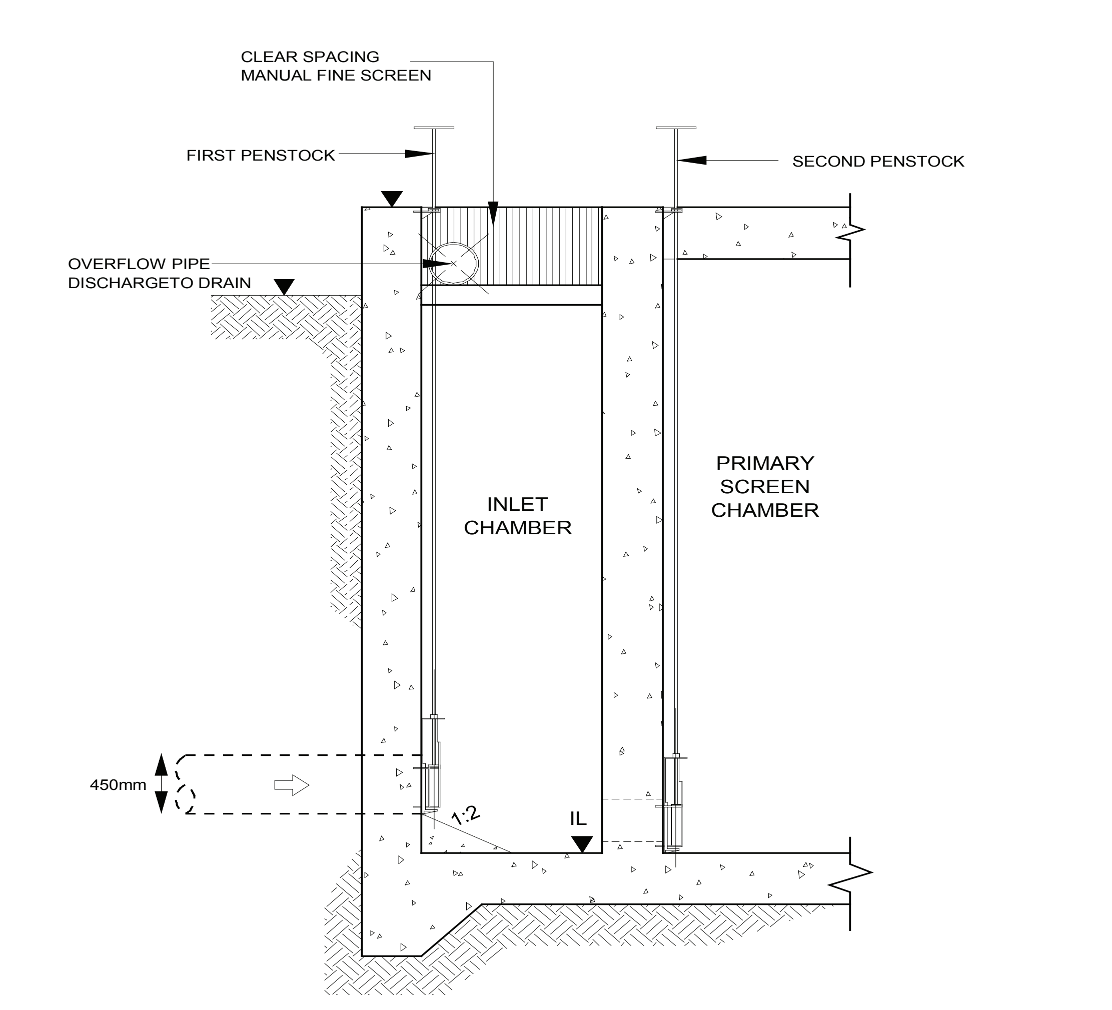
n/a – Not applicable

a) A penstock shall be installed upstream to isolate the pump station in the event of flooding in relation to the bypass and emergency overflow.

b) For safety reasons, a double penstock system shall be provided at the inlet works of all plants with pump station above 20 000 PE.

c) The penstock spindle shall extend to pump station ground level and shall be suitably positioned to allow unrestricted operation of the penstock.

Figure 5.3 Typical Details of Double Penstock



5.2.3 Design Requirements for Primary Screens

Table 5.2 Provision of Primary Screens

Requirements		Numbers of Primary Screen	
		≤ 5000 PE	> 5000 PE
Duty	Manual	1 Unit	-
	Mechanical	-	1 Unit
Standby	Manual	-	-
	Mechanical	-	1 Unit
By Pass		1 Unit	1 Unit

Table 5.3 Design Parameters for Primary Screens

Description	Unit	Design Criteria	
		Manually Raked	Mechanically Raked#
Maximum clear spacing	mm	25	25
Slope to the vertical		$30^o - 45^o$	$15^o - 45^o$
Maximum approach velocity at the feed channel	m/s	1.0	1.0
Maximum flow through velocity at the screen face	m/s	1.0	1.0
Minimum freeboard	mm	150*	150
Estimated volume of screenings per volume of sewage	$m^3 / 10^6 m^3$	30	See Figure 5.4
Screenings skip storage capacity	day	7	7
Minimum channel width	mm	500	500
Minimum channel depth	mm	500	500
RC Staircase with riser detail	1 unit	Anti-skid and non-corrosive	Anti-skid and non-corrosive

Notes:

* Designer shall ensure that with 50% of blockage at the face of screen, sufficient freeboard is provided to prevent the approach channel from overflowing

\# Washing and dewatering of screenings shall be provided.

5.2.4 General Requirements

All plants must include:

a) An emergency manual bypass screen. In the event of system failure and/or power outages, the flow shall be automatically directed to the bypass. It shall also be able to cope with maximum flow.

b) Hand railings, kick plates, standing platforms and other safety features to improve maintenance

Screen chambers must be of open channel construction with proper ventilation. Forced ventilation must be used if chambers are enclosed.

Screens must be designed to withstand the flushing velocity. In the event of the manual bypass screen being blocked, sewage must be able to flow over the top of the screen without causing excessive backup flooding or overflows.

Chambers design must have taken into consideration necessary health and safety aspects. The chamber must also be hydraulically efficient to prevent the settlement of solids in the chamber.

Macerators and communitors as replacements for primary screens are generally not recommended. It may be considered if the consultant is able to provide good engineering reasons for its application.

Reinforced concrete staircase with proper handrailing must be provided to access screen chambers.

Shaftless screw conveyors and belt conveyors may be used where required. The screw conveyor shall be equipped with easy to remove covers. The frame and support of screw conveyor shall be of minimum stainless steel Grade 304 while the screw shall be of high tensile steel. The belt conveyors shall be of heavy duty reinforced rubber belts on a protected mild steel frame. Conveyors shall normally be installed on a very slight grade to allow drainage, with foul water returned to the inlet channel.

All screenings raked from mechanical screen shall be dropped into a skip.

A proper standpipe shall be provided and located within 3 m to the screen chamber.

Figure 5.5 and 5.6 illustrate typical arrangement of screen chambers of various depths.

Refer also to relevant clause of MS 1228 for more details.

Figure 5.4 Quantities of Screenings Collected From Primary Screens

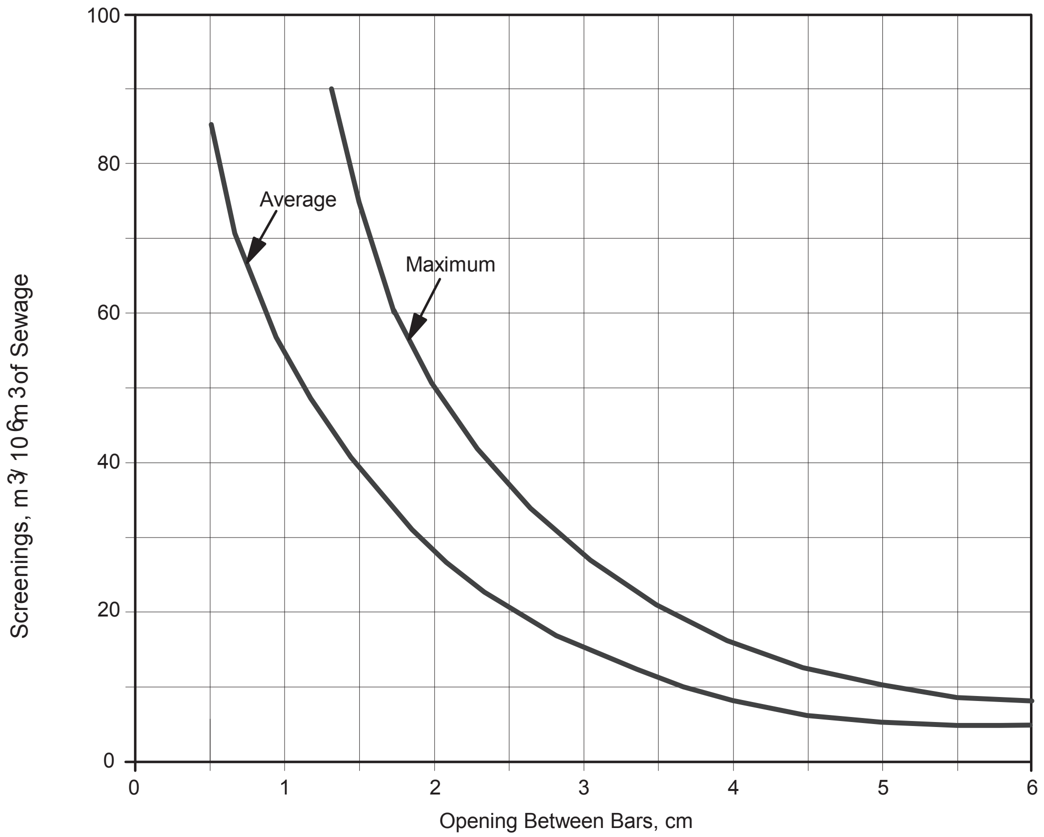


Figure 5.5 Typical Drawing of Screen Chamber based on depth (<5 m for different PE)

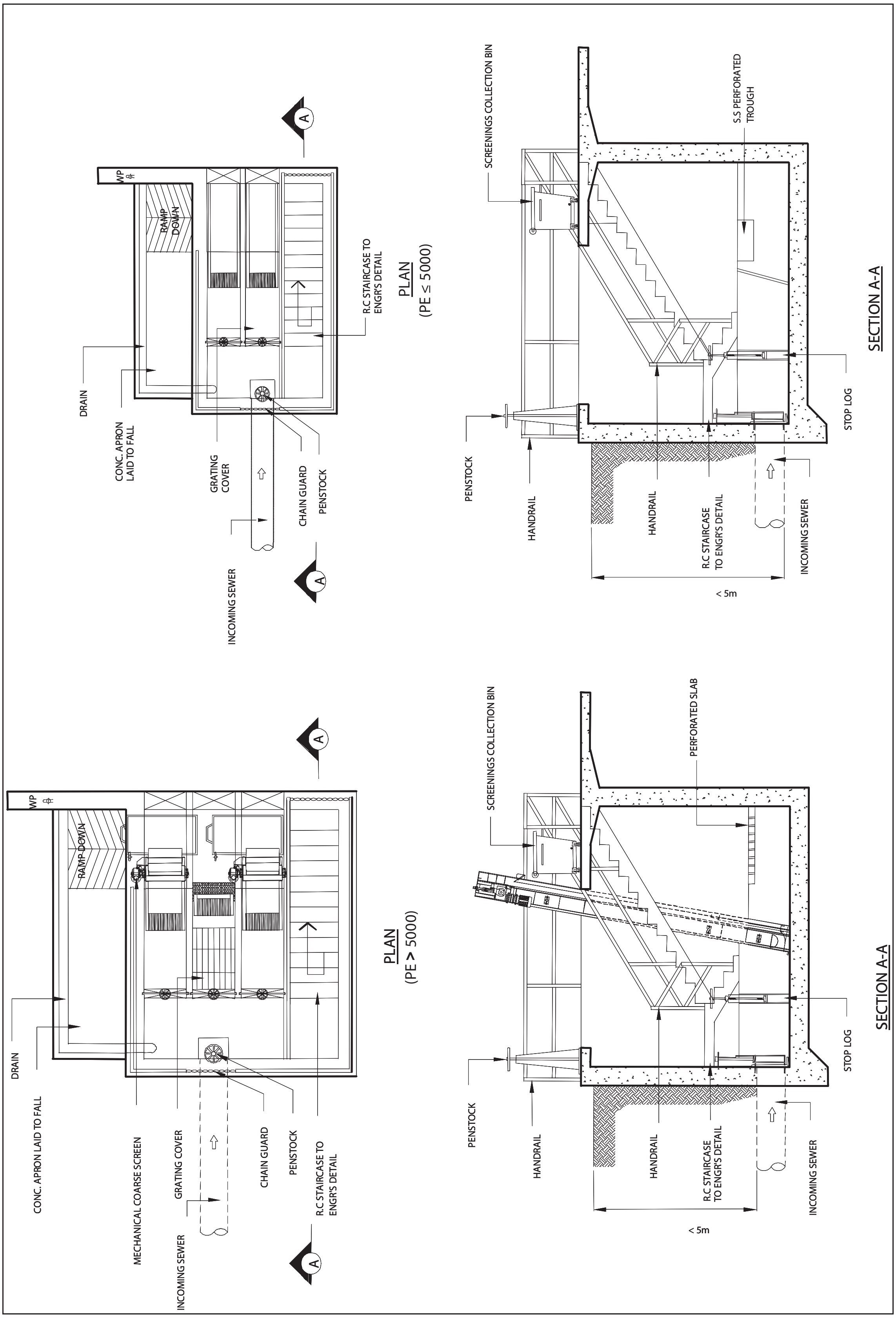
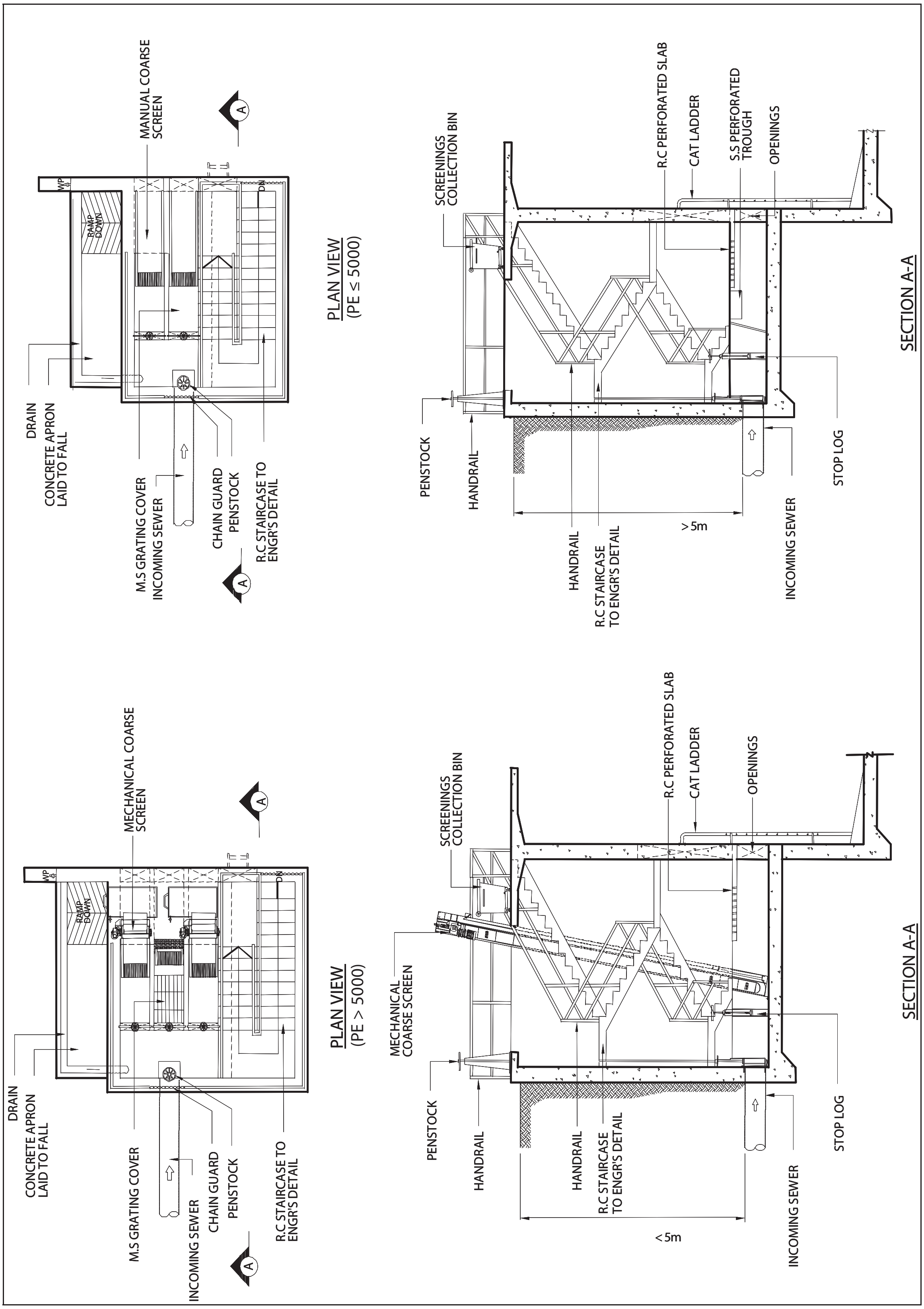


Figure 5.6 Typical Drawing of Screen Chamber based on Depth. (>5 m for different PE)



5.3 Design of Pump Stations

5.3.1 Purpose of Pump Stations

Inlet pump stations must be preceded by primary screens to protect the pumps from being damaged or clogged. The inlet pumps of the treatment works must be capable of handling raw unscreened sewage. Water pumps must not be used as they are not designed to cope with matters that may be found in sewage and the variability and quantity of sewage flow. The purposes of pump stations are:

a) To lift sewage to a higher point for treatment

b) To provide consistent inlet flows to the treatment system

c) To prevent overflow of raw sewage

5.3.2 Design Requirements

(I) Structural Requirements

a) Substructure shall be constructed with reinforced concrete using cement resistant to chemical attack, aggressive soils and groundwater.

b) Safe and suitable access to the wells shall be provided.

c) If cement used is not resistant to the chemical attack, internal walls shall be made resistant to sulphide corrosion by coating with high alumina cement or approved equivalent coating.

d) A controlled overflow from the last manhole upstream of the pump installation shall be provided to allow emergency maintenance works.

e) Access shall be provided at all locations where operation and maintenance works take place.

f) Static screen shall be provided at specific locations where it needs to protect downstream unit processes.

g) Access covers shall be hinged with a lifting weight not exceeding 16 kg.

II) Ventilation Requirements

a) Ventilation shall be provided for all hazardous zones of the pump station.

b) Below ground pits shall have mechanical ventilation.

c) Separate ventilation shall be provided for wet wells and dry wells.

d) Lighting systems shall be interconnected with ventilation.

e) Permanent ventilation rate and air changes shall comply with Section 6 of this Guidelines.

III) Odour Control Requirements

a) Isolate odorous gases from general ventilation by containing identified odour generating sources with a separate local exhaust system.

b) Containment of the odour sources shall be by installing lightweight and corrosion resistant covers/enclosures designed for practical operation and maintenance works.

c) Local exhaust rates for containment shall be designed to provide a negative pressure preventing build up of toxic, corrosive or explosive gases and to include provision for process air or air displaced by changes in the level of liquid inside the covered space.

d) The odourus air in the local exhaust system shall be conveyed through well designed and balanced ductworks by a centrifugal fan to an effective odour treatment system.

e) Odour treatment equipment shall be selected such that odour is reduced to the lowest possible level and in compliance to the EQA.

f) In situation where specific gases such as hydrogen sulphide and ammonia are significantly present, provide a pre-scrubber unit upstream of the main odour treatment equipment.

g) Containment, exhaust and treatment shall be designed as an integrated package.

h) Consideration must be given to the life span of the odour control system and associated costs in operating and maintaining such a system.

IV) Wet Wells Requirements

a) Suction channels shall be designed to avoid “dead zones”, i.e., prevent solids and scum accumulation. All “dead zones” shall be chamfered.

b) Benching shall be such that to minimise deposition of solid matters on the floor or walls of wet wells. The minimum slope of benching shall be 45° to the horizontal.

c) Benching shall preferably be extended up to the pump intake.

d) Minimum hopper bottom slope shall be 1.5 vertical to 1.0 horizontal. Tapered slope shall be provided up to the suction section.

e) Automatic flushing of grit and solids is recommended for plants of PE > 2 000.

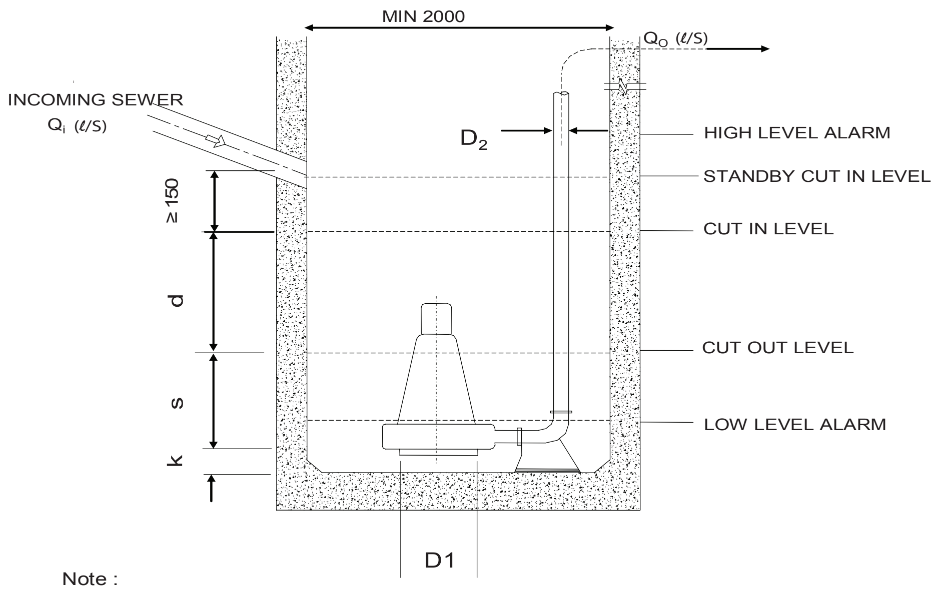
f) The difference between stop and start levels shall be a maximum of 900 mm and a minimum of 450 mm.

g) The difference in level between start or stop of duty and assist pumps shall be greater than or equal to 150 mm.

h) The minimum internal width of wet well shall not less than 2m.

i) Where possible, wet wells shall be open and guarded by a handrail or open mesh grating. The grating shall be easily and safely removed.

Figure 5.7 Typical Dimensions of Wet Well Submersible Pump Station



Note :

Q_1 = Incoming flow rate

Q_O = Forcemain Discharge rate

D_2 = Forcemain Diameter, min 100

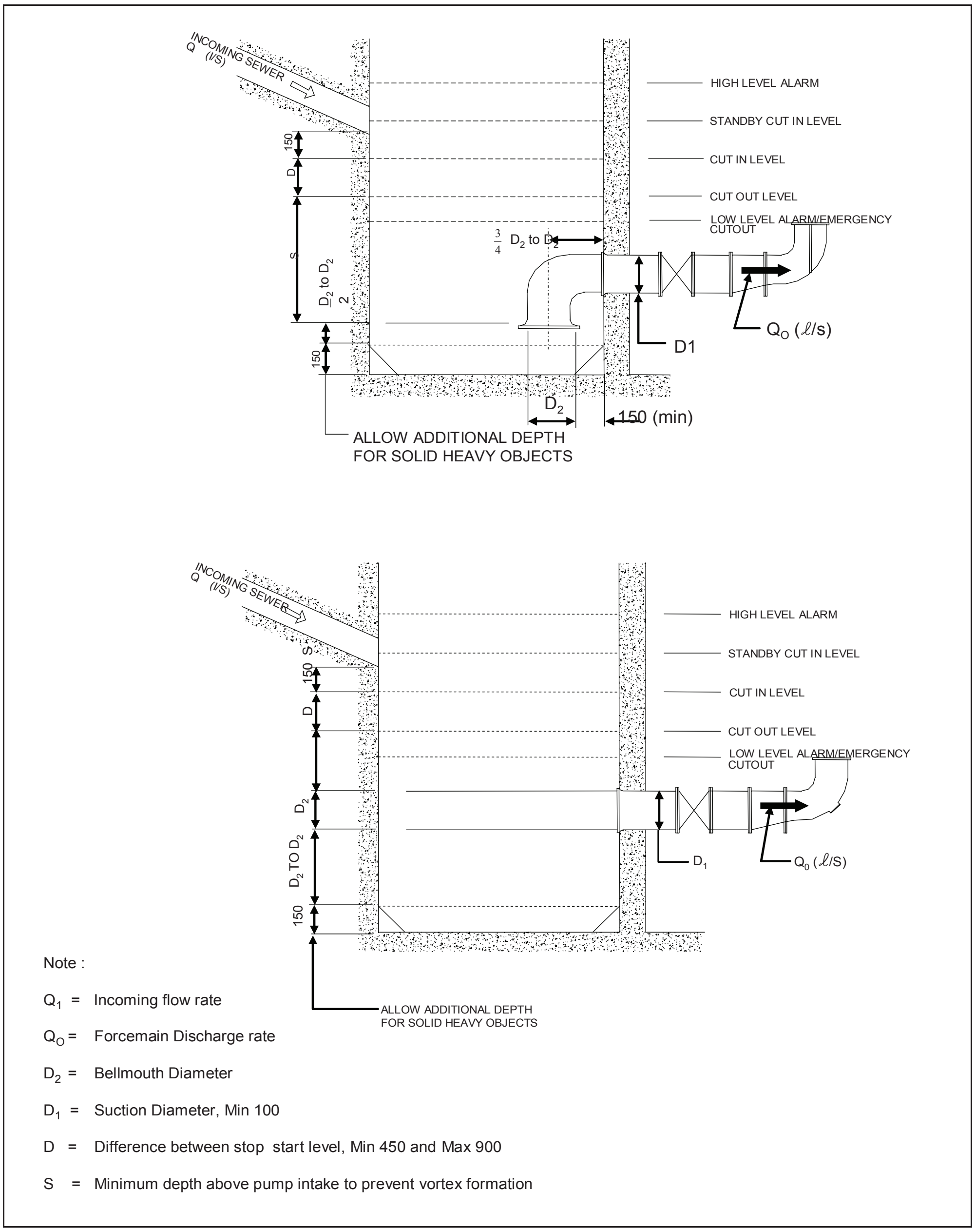
d = Difference between stop & start level, Min 450 Max 900

s = Minimum submergence, depends on manufacturer recommendation

k = Minimum clearance between pump suction and wet-well invert

All dimension are in mm unless otherwise state

Figure 5.8 Typical Dimensions of Dry Well Submersible Pump Station



V) Lighting Requirements

a) Wet wells and dry-wells shall be adequately lit.

b) Electrical installations shall be waterproof and vapour proof or explosion proof.

c) If lights are fitted outside the well, then a spotlight system may be used to provide adequate illumination.

VI) Level Controls

a) Either floats or ultrasonic level controller may be used for the start-stop levels of pumps. Instrument with environmental friendly features are recommended.

b) Ultrasonic level control is recommended due to its clog-free nature.

c) Non-mercury type floats are recommended.

d) Hollow tube electrodes are not acceptable.

e) Level controller shall be placed where they are not affected by the turbulence of incoming flow and where they can be safely removed.

VII) Pump Hydraulic Design

a) The submission of pump hydraulic design and performance shall include:

   i) System curves

   ii) Pump curves

   iii) Operating points of pumps with respect to flow and total dynamic head (TDH)

   iv) Operating characteristics such as efficiency, horsepower and motor rating

b) Pump shall be operating within their best efficiency range at normal operating condition.

c) Pumps are to be equipped with an auto restart mechanism in the event of power failure.

d) Dry-well mounted pumps shall be equipped with auxiliary services such as cooling and gland seal water supply.

e) Pumps equipped with cutting or macerating facilities are not acceptable.

f) Guide rail, lifting device and other wet well fittings must be fabricated of stainless steel that is corrosion resistant. The use of hot dip galvanised iron is not recommended.

g) Horizontal installation of pumps is not allowed. All pumps shall be installed vertical, unless the Consultant is able to provide good engineering reasons for horizontal installation.

5.3.3 General Requirements

a) Drainage of dry-wells and valve pits shall be provided. Drainage lines shall be equipped with back flow protection to ensure that the chamber is not flooded.

b) The wet well shall not be housed with a building structure.

c) Where separate valve pits are used, the connecting pipes shall incorporate at least two flexible joints to allow for differential settlement.

d) Pipework

i) Pipe work shall be of ductile iron or cast iron with cement internal lining. Other approved material by the Commission may be used.

ii) External surface of pipework in chambers and wells shall be epoxy coated.

iii) Buried ductile iron pipe shall have polyethylene sleeving.

iv) Pipe within wells and pits shall have flanged joints, while pipe laid in the ground shall have spigot and socket joints.

v) Pipe work shall be adequately supported on concrete plinths or steel structural supports.

vi) Flanges shall be located at least 150 mm away from structures.

vii) Dismantling joints such as bends shall be provided.

viii) Pumping thrust shall be resisted using pipe supports inside chambers and by mass concrete thrust blocks poured against undisturbed soil in the ground outside chambers.

ix) No welding joints are allowed.

e) Valves

i) Gate valves are preferred with rising spindles operated by a tee piece.

ii) The uses of counterweights are recommended. Tapping (12 mm BSP) shall be located upstream and downstream of check valves.

Also refer to additional requirements in relevant Clause of MS 1228.

Figure 5.9 Typical Details of Wet-well Pump Station

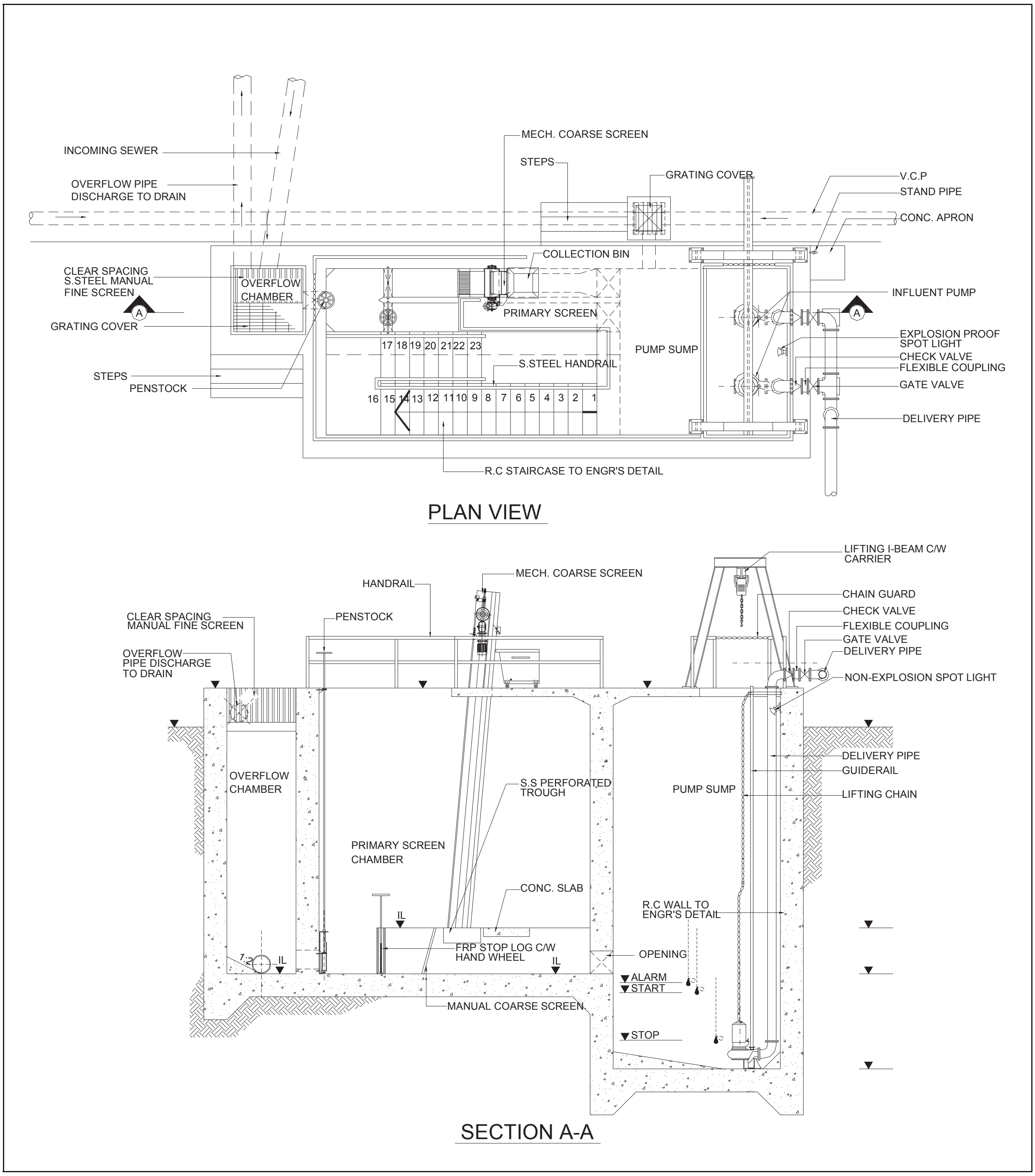


Figure 5.10 Typical Details of Dry-well Pump Station

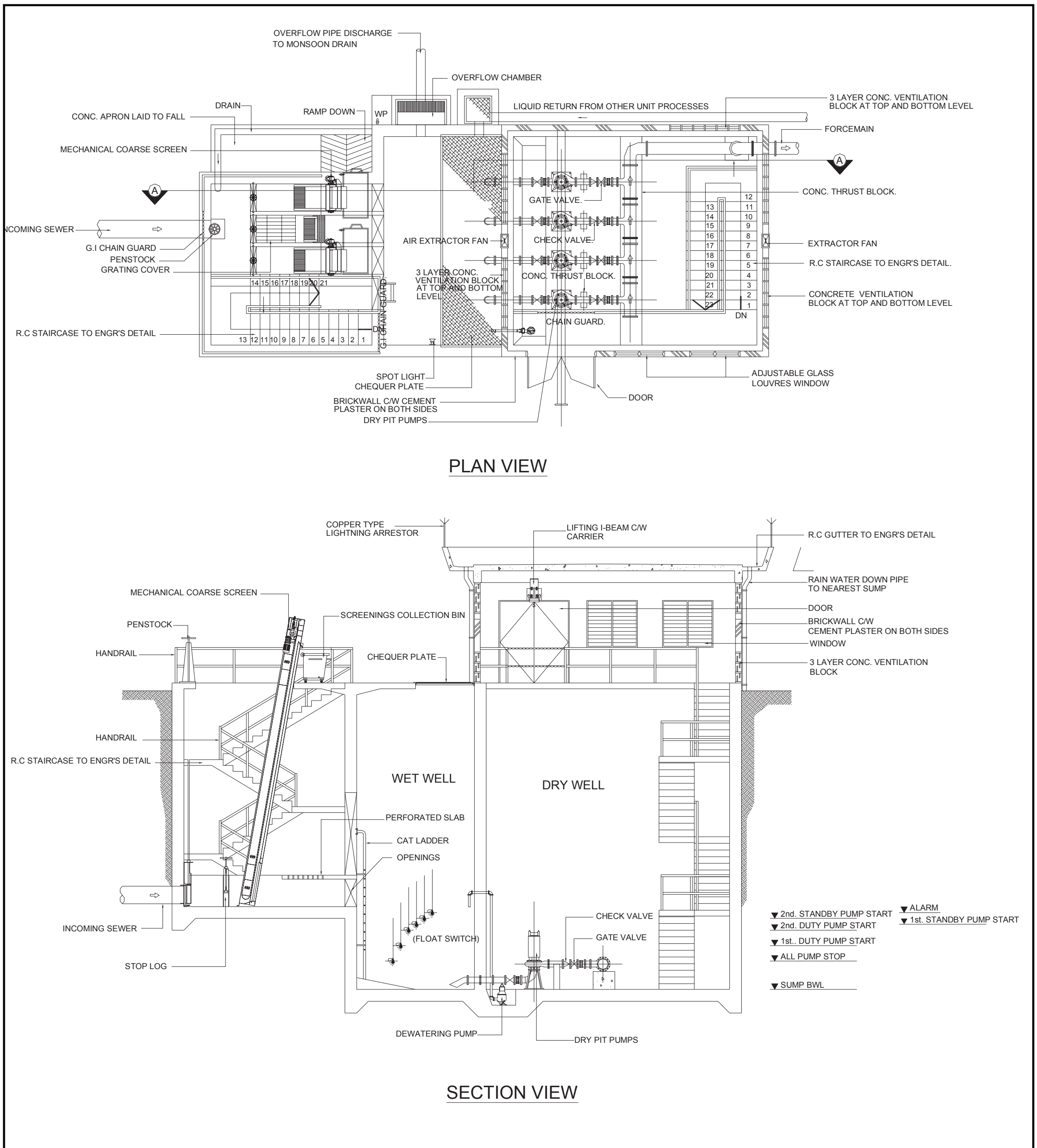


Table 5.4 Recommended Design Parameters for Inlet Pump Stations

		≤ 1000 Design Parameters	
Description	**Unit**	**PE ≤1,000**	**1,000 < PE ≤ 5,000**
Type of station		Wet well	Wet well
Number of pumps (all identical and work sequentially)		2 1 duty, 1 standby (100 % standby)	2 1 duty, 1 stand-by (100 % standby)
Pumps design flow		Each at Q_{peak}	Each at Q_{peak}
Maximum retention time at Q_{ave}	min	30	30
Min pass through openings	mm	75	75
Minimum suction and discharge openings	mm	100	100
Pumping cycle (average flow conditions)	start/ hour	6 min 15 max	6 min 15 max
Lifting device*		Lifting davit	Lifting beam and block
		Design Parameters	
Description	**Unit**	**5,000 < PE ≤ 20 000**	**PE > 20,000**
Type of station		Wet well or dry-well up to 10,000 PE 10,000 PE above – wet well and dry-well	Wet well and dry well
Number of pumps (all identical and work sequentially)		4 (2 sets) 1 duty, 1 assist, per set (100 % standby)	6 (3 sets) 1 duty, 1 assist, per set (50 % standby)
Pumps design flow		Each at 0.5 Q_{peak}	Each at 0.25 Q_{peak}
Maximum retention time at Q_{ave}	min	30	30
Min pass through openings	mm	75	75
Minimum suction and discharge openings	mm	100	100
Pumping cycle (average flow conditions)	start/ hour	6 min 15 max	6 - 15
Lifting device*		Mechanical and block	Mechanical

Note: * Motorised hoists shall be provided when the lifting weight exceeds 100 kg

5.4 Design of Secondary Screens

5.4.1 Purpose of Secondary Screens

After the inlet pump station, further screening is required to reduce the remaining floating matter and finer particles in the sewage that will disrupt the treatment process downstream. The purposes of secondary screens are:

a) to remove material such as plastic, paper, cloth and other particles that may cause problems to the treatment process downstream.

b) to minimise blockages in sludge handling and treatment facilities.

5.4.2 Design Requirements

Plants of all sizes must be installed with secondary screens. The channel shall be designed for Q_{peak} or pump flow whichever is greater. Approach channel shall be design to ensure good contribution of velocity

A minimum of two screens are required for duty and standby. Facility for a screened bypass must be provided in the event of clogging. Where mechanically cleaned screening devices are installed auxiliary manually cleaned screen shall be provided.

Table 5.5 Provision Requirement of Secondary Screens

Requirements		Numbers of Secondary Screen	
		≤5000 PE	>5000 PE
Duty	Manual	1 Unit	-
	Mechanical	-	1 Unit
Standby	Manual	-	-
	Mechanical	-	1 Unit
Bypass	Screen	1 Unit	1 Unit

Table 5.6 Design Parameters for Secondary Screens

Description	Unit	Design Criteria	
		Manually Raked	Mechanically Raked #
Maximum clear spacing	mm	12	12
Slope to the vertical		$30^o – 45^o$	$15^o – 45^o$
Maximum approach velocity at the feed channel	m/s	1.0	1.0
Maximum flow through velocity at the screen face	m/s	1.0	1.0
Minimum freeboard	mm	150*	150
Estimated volume of screenings per volume of sewage	m^3 / 10^6 m^3	See Figure 5.4	
Screenings skip storage capacity	day	7	7
Minimum channel width	mm	500	500
Minimum channel depth	mm	500	500
RC Staircase with riser detail	1 unit	Anti-skid and non-corrosive	Anti-skid and non-corrosive

Notes:

* Designer shall ensure that with 50% of blockage at the face of screen, sufficient freeboard is provided to prevent the approach channel from overflowing

\# Washing and dewatering of screenings shall be provided.

5.5 Design of Grit and Grease Chambers

5.5.1 Purposes of Grit and Grease Chambers

This unit process is important to minimise problems associated with grit and grease. Grit creates problems to pumps and also sludge digestion and dewatering facilities. Grease creates problems at the clarifier and is carried over in the final effluent.

In grit removal system, grit or discrete particles that have subsiding velocities or specific gravities substantially greater than those of organic putrescible solids, e.g. eggshells, sands, gravel are removed by gravitate settlement or centrifugal separation. Same principle apply to oil and grease removal system, where free oil and grease globules lighter than water rise through the liquid and skimmed from the top surface.

The particles must be removed at an early stage of the process because:

a) the grit particles cannot be broken down by any biological treatment.

b) the grit particles are abrasive and wear down equipment.

c) the biological treatment in sewage treatment works is not designed to degrade grease.

5.5.2 General Requirements

A manual bypass shall be provided. In case of grit removal system failure and/or power outages, the flow shall be automatically directed to the bypass.

Where mechanical grit separator is used, they shall be installed at an angle of at least 10° to allow drainage and foul water to be returned to the inlet channel.

Where manual systems are used, allow for safe and easy access to remove grit to a storage bin.

If pump systems are used, the suction pipe shall be short and straight. Tees and short radius bends shall be avoided, if at all possible. Flanges at strategic locations shall be provided so that they can be dismantled to remove any blockages.

The mechanical oil and grease skimming device shall be designed to minimise the water being remove while skimming the oil and grease.

Sand pit may be used for further dewatering of the grease removed before ultimate disposal. The drainage from the sand pit shall be returned to the inlet channel for further treatment.

Table 5.7 Provision Requirement of Grit and Grease Removal System

Requirements		Number of Unit Processes	
		≤ 5000 PE	> 5000 PE
i) Grit Removal System			
Duty	Manual	1 Unit @ design flow	-
	Mechanical	-	-1 Unit @ design flow (up to 10 000 PE) -2 unit @ 50% design flow each (>10 000 PE)
Standby	Manual	1 Unit @ design flow	-
	Mechanical	-	-
Bypass		-	Yes
ii) Grease Removal System			
Duty	Manual	1 Unit	-
	Mechanical	-	-1 Unit @ design flow (up to 10 000 PE) -2 unit @ 50% design flow each (>1000 PE)
Standby	Manual	1 Unit @ design flow	-
	Mechanical	-	-
Bypass		-	Yes

5.5.3 Design Criteria

Design criteria are given in Tables 5.8 and 5.9.

Table 5.8 Design Parameters for Grease Chambers

Description	Unit	Design Criteria		
		PE ≤ 5000*	> 5000PE	> 5000PE
Grease removal	-	Simple manual	Manual interceptor	Mechanical
Chamber type	-	Rectangular	Baffled tank	Aerated type
Minimum detention time (Q_{peak})	min	3	3	3
Grit and grease storage period before off-site disposal	day	30	7	7

Note:

* Combined grit & grease chamber is allowed. If combined, then total detention time shall comply to 6 minutes at Q_{peak}.

Table 5.9 Design Parameters for Grit Chambers

Description	Unit	Design Criteria		
		PE ≤ 5000*	>5000PE	>5000PE
Grit removal	-	Manual (tanker)	Mechanical	Mechanical
Chamber type	-	Horizontal flow (2 units required for duty and standby during cleaning) vortex also acceptable	Square, rotary or vortex type simple mechanised grit chamber	Aerated
Minimum detention time at Q_{peak}	minute	3	3	3
Maximum gravity flow through velocity at Q_{peak}	m/s	0.20	0.20	0.20
Maximum centrifugal flow through velocity	m/s	n/a	<1.0	<1.0
Head loss (at parshall flume)	-	35% of depth	-	
Aeration requirement	ℓ/s/meter length of tank	-	-	10.0
Chamber dimension: Depth: Width Length: Width	-	1:2 2:1	Manufacturer's Specification	Manufacturer's Specification
Estimated grit quantity	$m^3/10^3\ m^3$ of sewage	0.03	0.03	0.03
Washing and dewatering of grit	-	No	Yes	Yes

Notes:

Air lift pump for removal of grit is not acceptable.
Water depth in tank to be controlled by weir outlet.

5.6 Design of Balancing Tanks

Balancing tanks are mandatory for all treatment processes that are not designed at peak flow. The tanks are effective means of equalising sewage flow. For extended aeration plants that are designed with a retention time of more than 18 hours and clarifiers designed at peak flow, the use of balancing tanks is not required.

5.6.1 Purposes of Balancing Tanks

The purposes of balancing tanks are to:

a) prevent flow variations entering secondary treatment processes.

b) reduce hydraulic loading into secondary treatment processes.

c) reduce potential overflows that may cause health hazard and pollution.

5.6.2 Design Requirements

The design requirements for balancing tanks are:

a) All balancing tanks must be completely aerated and mixed.

b) Flow control shall be by a non-mechanical constant flow device, such as an orifice, in order to avoid double pumping.

c) Allowance must be made for an emergency overflow.

d) Bypass and drain down facilities as well as suitable access for cleaning shall be provided.

e) A dead water depth of 0.6 - 1.0 m is normally required.

f) For plants with PE > 10 000, where balancing tank is used, minimum one (1) unit of balancing tank shall be provided. The design flow of the upstream and downstream unit processes are recommended as follow:

   i) Where no balancing tanks is provided, design flow of unit process at
      - Upstream = Peak/pumped flow
      - Downstream = Peak/pumped flow

   ii) Where balancing tank is provided, design flow of unit process at
      - Upstream = Peak/pumped flow
      - Downstream = Average flow

Table 5.10 Design Parameters for Balancing Tanks

Description	Unit	Design Criteria
Volume of tanks	m^3	1.5 hr detention at Q_{peak}
Mixing power requirements	W/m^3 of sewage	5 at TWL
Aeration	m^3 air/hour/ m^3 sewage	1 m^3 of air supply for every m^3 of sewage stored per hour at TWL
Overflow bypass to down-stream unit requirement	-	Yes

5.7 Design of Primary Sedimentation Stage

At primary sedimentation stage, the velocity of sewage is reduced to subside settleable suspended organic matters in the sewage. The settled matter is known as primary sludge.

5.7.1 Purposes

The purposes of primary sedimentation are:

a) To remove maximum amount of pollutants such as settleable solids quickly and economically.

b) To separate sewage into sludge and settled sewage, which by being treated separately are normally dealt with more efficiently and economically.

c) When used as a preliminary step for further treatment, the main function of primary sedimentation tank is to reduce the organic loadings on the secondary treatment units and is a essential component of secondary sewage treatment.

5.7.2 Design Requirements

The design requirements of primary sedimentation include the followings:

a) Provide sufficient time for maximum settling under quiescent conditions. Therefore, design factors require careful consideration include: overflow rate, detention period, weir-loading rate, shape and dimensions of the basin, inlet and outlet structures, and sludge removal system.

b) Tanks can either be rectangular, circular or upward flow (square).

c) Provisions for the removal of sludge on a daily basis.

d) Holding tanks for wasted sludge must be provided.

e) V-notch weirs with baffle shall be provided at the outlet. Weirs shall be adjustable with notches 100 mm deep. Typical variation in water level shall be no more than 50 mm under all conditions.

f) Multiple hopper are not permitted.

g) Scum skimming shall be provided to remove both floating materials and scum. These materials can be either discharged to biosolid holding tank or sand drying bed. They shall not be returned to the preliminary treatment units.

h) Flow distribution channel/ chamber shall be provided for flow isolation or equal flow distribution.

i) Rectangular tank

   i) Sludge hopper shall have side slopes of 60° or more from horizontal with the sludge pump located in a pit at hopper invert level. The length of suction pipe shall be minimised. Provision for withdrawal pipe from the tanks shall be provided.

   ii) The capacity of hopper shall be equivalent to 2 hours detention time at Q_{peak}.

   iii) Additional water depth of minimum 400 mm should be provided above the hopper in the vertical side-wall section between the top of the hopper and the top water level. The side-wall height should not be less than 400 mm.

   iv) Equalise flow distribution across the inlet of the tank shall be achieved using a multi-port wall and baffles.

j) Circular Tank

   i) Circular tanks shall be no more than 50 m in diameter and minimum water depth shall be 3.0 m.

   ii) Circular tanks with more than 30 m diameter shall be provided with perimeter walkway for cleaning the weir and shall have appropriate drive system.

(k) The floor slab of the sedimentation tank shall be of reinforced concrete type construct to gradient to enhance the sludge scraping effectiveness.

Table 5.11 Design Parameters for Primary Sedimentation

Description	Unit	Design Criteria
Sedimentation followed by secondary treatment		
Detention time at Q_{peak}	hr	2
Surface overflow rate at Q_{peak} - circular (maximum) - rectangular (maximum)	 $m^3/m^2/d$ $m^3/m^2/d$	 45 45
Weir loading at Q_{peak}	$m^3/m/d$	150
Upward flow rate at Q_{peak}	m/hr	1.2 - 2.0
Sizing of rectangular tanks		
Length : Width		> 3:1
Min water depth	m	2.5
Width : Depth		1 : 1 to 2.5 : 1
Sizing of circular tanks		
Min. side water depth	m	> 3.0
Floor slope wall		1:12

5.8 Design of Biological Treatment Stage

5.8.1 Introduction

Biological treatment is the heart of the sewage treatment process. It is the processes where the dissolved and non- settleable organic material remaining in the sewage are removed by living organisms.

For reasons of long term whole life economics, ease of operation and maintenance, consistent effluent standards and standardisation, the following types of biological treatment processes are recommended for use in Malaysia.

Suspended Growth System

a) Conventional Activated Sludge (CAS) System

b) Extended Aeration (EA)/Oxidation Ditch (OD) System

c) Sequencing Batch Reactor (SBR)/Intermittent Decant Extended Aeration (IDEA)

Attached Growth System

a) Rotating Biological Contactor (RBC) System

b) Trickling Filter (TF) System

c) Hybrid System/Combination Multistage Design

All plants must be strictly designed to meet DOE Standard A/Standard B requirements including, nitrification and denitrification to reduce ammonia and total nitrogen removal level that ensure compliance with the requirement stipulated in Section 3 earlier. Total phosphorus removal must also be taken into account for plants where treated effluent is to be discharged into stagnant water bodies.

Mass balance calculation must be computed and submitted for all biological treatment system and other unit processes proposed for the STP.

5.8.2 Conventional Activated Sludge System (CAS)

5.8.2.1 General Description

The Conventional Activated Sludge process is one of the many versions of the activated sludge process. The activated sludge process is most suitably used where land is limited and expensive, and where large volumes must be treated economically, without creating nuisance to neighbours.

The process involves the production of activated mass of microorganisms capable of stabilising sewage aerobically. This is achieved by introducing organic waste, produced from pre-treatment and primary treatment facilities, into reactors where suspended aerobic bacterial culture oxidises the organic matter into stable matters. These active bacteria cultures are commonly known as activated sludge. During the process, new bacteria cell are also produced.

5.8.2.2 Design Requirements for CAS

For the design of Conventional Activated Sludge system, the aeration tank shall be preceded with primary sedimentation system. An appropriate amount of the bacteria culture, known as activated sludge must be recycled to the upstream of the reactor while the remaining excess sludge must be removed at secondary sedimentation system.

All Conventional Activated Sludge system used at STPs for Class 3, Class 4 and at where requested by the Commission must be designed with anoxic zone to achieve a total nitrogen removal in order to comply with the requirements in Section 3 of this Guidelines, as well as to minimise potential rising sludge at secondary sedimentation system. The anoxic zone must be mixed without inducing dissolve oxygen.

Sludge treatment and dewatering must be available on-site to handle the large quantity of unstable sludge generated.

Table 5.12 Design Parameters for Conventional/ Activated Sludge System

Description	Unit	Design Criteria
Primary Sedimentation System		Must be provided
Minimum number of aeration tanks		2
F/M ratio		0.25 - 0.50
Hydraulic retention time (HRT)	hrs	6 -16 (for system where only ammonia removal is require)
		12 -16 (for plants require total nitrogen removal)
Oxygen requirements (for BOD and ammonia nitrogen removal)	kgO_2/kg substrate	2.0
Mixed liquor suspended solids (MLSS)	mg/ℓ	1500 -3000 Typical: 2500
Dissolved oxygen (DO) level in tank	mg/ℓ	1.0
Aeration device rating		Continuous, 24 hrs
Sludge yield	kg sludge produced/kg BOD_5 consumed	0.8 - 1.0
Sludge age #	day	5 - 10
Waste activated sludge, Q_{WAS}	m^3/d	Refer to equation below †
Return activated sludge flow, Q_{RAS}	m^3/d	$\frac{MLSS}{C_u - MLSS} \times Q_{avg}$ C_u is underflow concentration
Q_{RAS} / Q_{INFLOW}		0.75-1.0
Mixed liquor suspended solids recirculation for denitrification purpose		4 – 6 of Q_{avg}
RAS pump rating	hrs/day	24
Organic loading	kg BOD_5/kg MLSS	0.25 - 0.5
Volumetric loading	kg $BOD_5/m^3.d$	0.3 - 0.6
Minimum mixing requirement	W/m^3	20

Table 5.12 Design Parameters for Conventional Activated Sludge System (continued)

Description	Unit	Design Criteria
Tank dimension		
Water depth	m	3 – 5
Length:Width		3:1
Max width of joined tank	m	< 30

Sludge Age = $\dfrac{\text{total solids in aeration tank}}{\text{excess sludge wasting/day} + \text{solids in effluent}}$

† $$WAS = \frac{\left[\frac{V_T \times MLSS}{\theta_{sludge}}\right] - \left[Q_{avg} \times SS_{eff}\right]}{C_u}$$

Where:

v_T = volume of reactor (m^3)
MLSS = mixed liquor suspended solids (kg/m^3)
θ_{sludge} = sludge age (days)
Q_{avg} = average flow (m^3/day)
SS_{eff} = effluent suspended solids (kg/m^3)
C_u = underflow concentration (kg/m^3)

Refer Table D1 and D2 for aeration equipment duty/standby and also to relevant clause of MS 1228 for more details.

5.8.3 Extended Aeration System (EA)

5.8.3.1 General Discription

The Extended Aeration process is similar to the Conventional Activated sludge process except that it operates in the endogenous respiration phase of the growth curve, which requires a low organic loading and long aeration time. The system produces high MLSS concentration, high RAS pumping rate and low sludge wastage.

The advantage of having long hydraulic retention times is that it allows the plant to operate effectively over widely varying flow and waste loadings. Secondary clarifiers must be designed to the variations in hydraulic loadings and high MLSS concentrations associated with this process.

5.8.3.2 Design Requirements for EA

EA plants shall be designed as either plug flow or completely mixed. Anoxic zone at the head of the reactor must be provided for denitrification. The anoxic zone must be mixed without inducing dissolved oxygen

For Oxidation Ditches, the minimum velocity within the channel shall be sufficient to keep the activated sludge in suspension. The minimum velocity within the channel shall not be less than 0.3 m/s. The tank configuration and aeration and mixing devices shall promote unidirectional channel flow, so that the energy used for aeration is sufficient to provide mixing in a system with a relatively long hydraulic retention time.

Table 5.13 Design Parameters for Extended Aeration

Description	Unit	Design Criteria
Minimum number of aeration tanks		2
F/M ratio		0.05 - 0.1
Hydraulic retention time (HRT)	hrs	18 - 24
Oxygen requirements(for BOD and ammonia nitrogen removal)	$kgO_2/kg_{substrate}$	2.0
Mixed liquor suspended solids (MLSS)	mg/ℓ	2500 - 5000 Typical: 3000
Dissolved oxygen (DO) level in tank	mg/ℓ	2.0
Sludge yield	kg sludge produced/kg BOD_5 consumed	0.4 (at 24 hrs HRT) 0.6 (at 18 hrs HRT)
Sludge age #	day	> 20
Waste activated sludge flow, Q_{WAS}	m^3/d	Refer to equation †
Return activated sludge flow, Q_{RAS}	m^3/d	$\frac{MLSS}{C_u - MLSS} \times Q_{avg}$ C_u is underflow concentration
RAS pump rating	hours/day	24
Recirculation ratio, Q_{RAS}/Q_{INFLOW}		0.5 - 1.0
MLSS recycle ratio		4 – 6 times of Q_{avg}
Volumetric loading	kg $BOD_5/m^3.d$	0.1 - 0.4
Minimum mixing requirement	W/m^3	20
Tank dimension		
Water depth	m	3 – 5
Length:Width	ratio	3:1
Max width of joined tank	m	< 60

Notes:

$$^{\#}\text{Sludge Age} = \frac{\text{total solids in aeration tank}}{\text{excess sludge wasting/day} + \text{solids in effluen}^{\dagger}}$$

$$\text{WAS} = \frac{\left[\frac{V_T \times MLSS}{\theta_{sludge}}\right] - \left[Q_{avg} \times SS_{eff}\right]}{C_u}$$

Where:

v_T	= volume of reactor (m^3)
MLSS	= mixed liquor suspended solids (kg/m^3)
θ_{sludge}	= sludge age (days)
Q_{avg}	= average flow (m^3/day)
SS_{eff}	= effluent suspended solids (kg/m^3)
C_u	= underflow concentration (kg/m^3)

Refer Table D1 and D2 for aeration equipment duty/standby and also to relevant clause of MS 1228 for more details.

Figure 5.11 Fine Bubble Diffuse Air Extended Aeration System

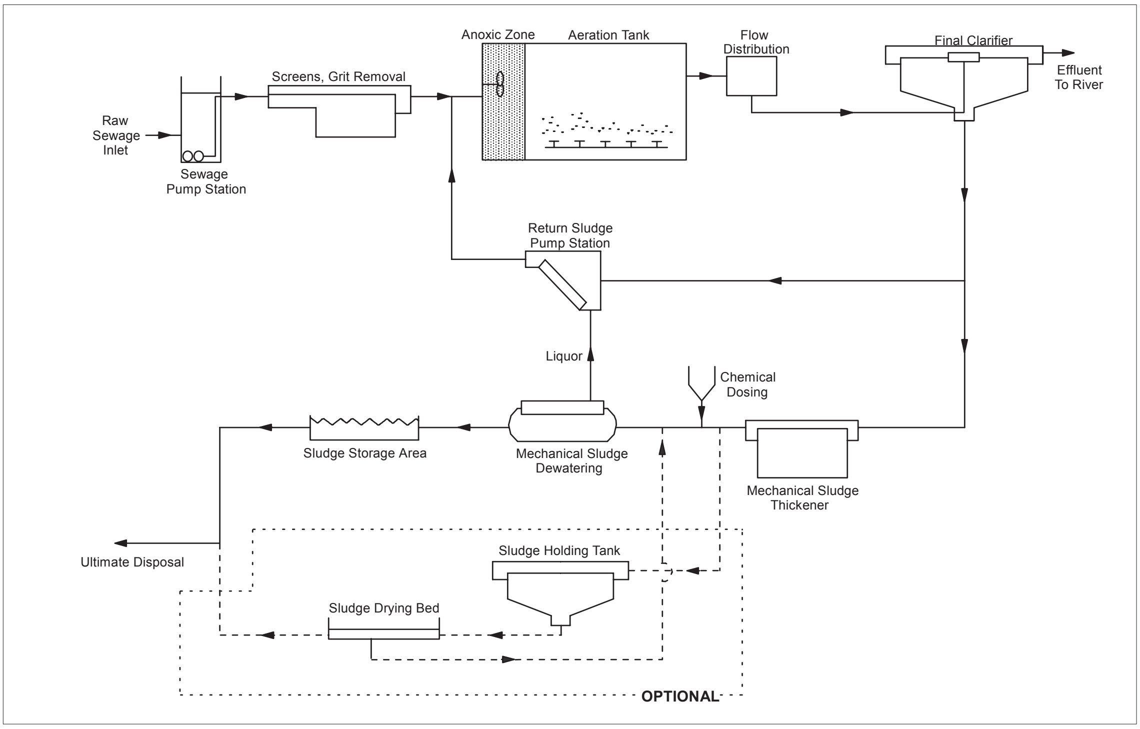
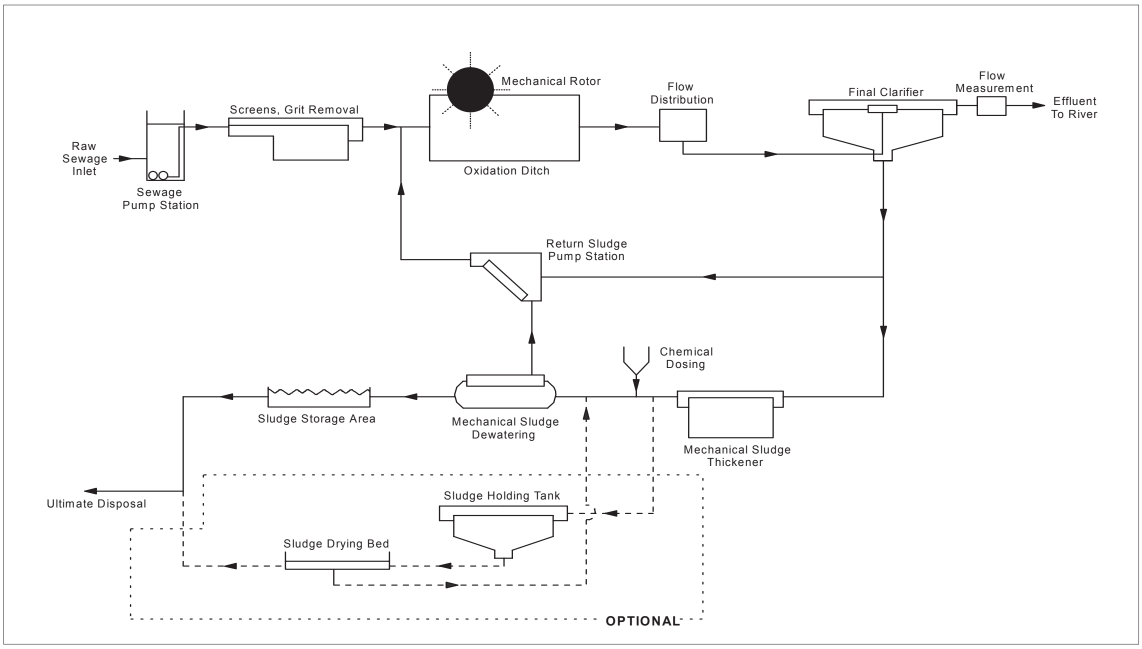


Figure 5.12 Oxidation Ditch Activated Sludge System



5.8.4 Rotating Biological Contactors (RBC)

5.8.4.1 General Description

Rotating Biological Contactors use a series of rotating media for biological treatment. The rotating medium, typically made from sheets of high quality plastic, provides a surface on which organisms grow. As the media rotate, the fixed film biomass is in contact with organic pollutions in sewage and oxygen in atmosphere alternately. Layers of biomass are sheared from the surface of the media during the rotation to prevent overgrown of the fixed film.

RBCs are conventionally submerged to 40% of disc diameter. Increased submergence of discs up to about 90% is also acceptable if sufficient air supply is provided at the base of the tank. This system is normally called the submerged biological contactor (SBC).

5.8.4.2 Design Requirements for RBC Plants

Preceding the RBC must be a primary sedimentation tank or a secondary screening with < 6 mm opening. A flow balancing tank must also be provided unless the plant is designed to peak flow.

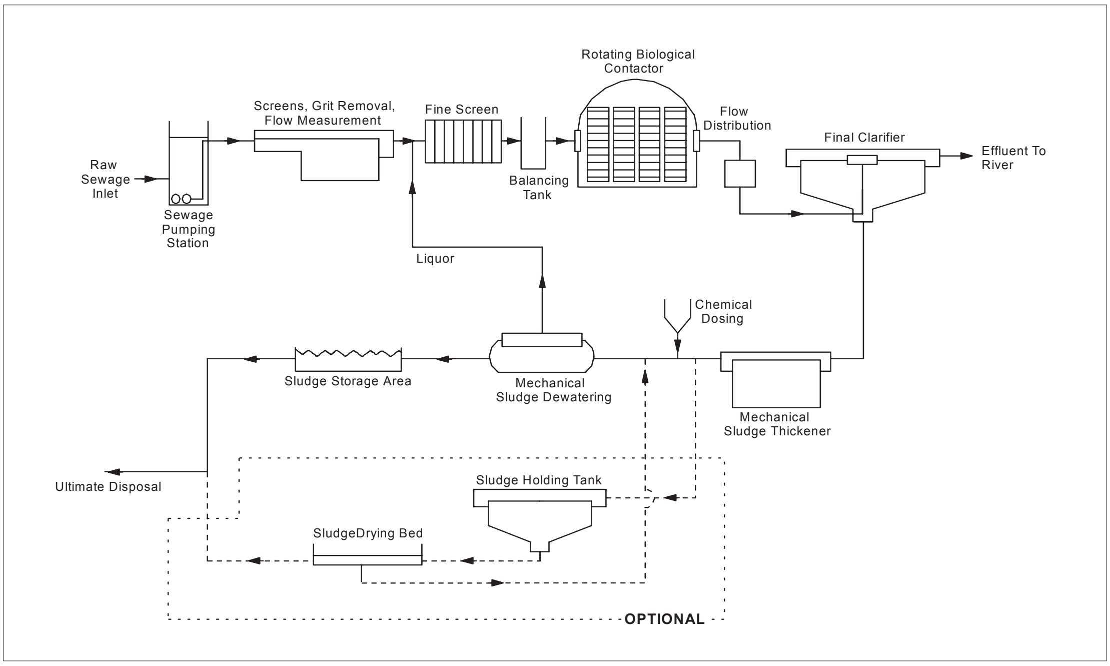
Units must be covered for aesthetics and odour control, and only approved media types are accepted.

Table 5.14 Design Parameters for RBC Plants

Description	Unit	Design Criteria
Minimum number of stages		3
Total BOD_5 specific loading	$g/m^2/d$	5 - 10
Total tank volume		Based on 2 hrs at Q_{avg}
Sludge yield	kg excess sludge/ kg BOD_5 consumed	0.9
Disc diameter	m	2.5 - 3.5
Speed of rotation	rev / min	0.5 - 1.0
Maximum peripheral velocity	m/s	0.3
Depth of disc submergence	%	40 - 90

Refer also to Table D.3 for duty standby requirements and relevant clause of MS 1228 for more details.

Figure 5.13 Rotating Biological Contactor (RBC) Systems



5.8.5 Trickling Filter

5.8.5.1 General Description

The Trickling Filter is an established biological treatment process removing 65 to 85% BOD_5 and suspended solids. The process consists of a bed of highly permeable medium. An overhead rotating distributor applies sewage to the media. The flow trickles over and flows downward to the underdrain system.

The media provides a large surface area to develop biological slime growth which is also known as zoogleal film. The film contains living organisms that break down organic material in the sewage.

Many variations of the Trickling Filters have been constructed, however the recommended designs are given in Table 5.15.

5.8.5.1 Design Requirements for Trickling Filters

Secondary screens (< 6 mm) and flow balancing tanks to equalise the flow must be provided before trickling filters.

Provisions shall be available for even distribution to achieve complete wetting of the filter media.

Figure 5.14 Trickling Filter Systems

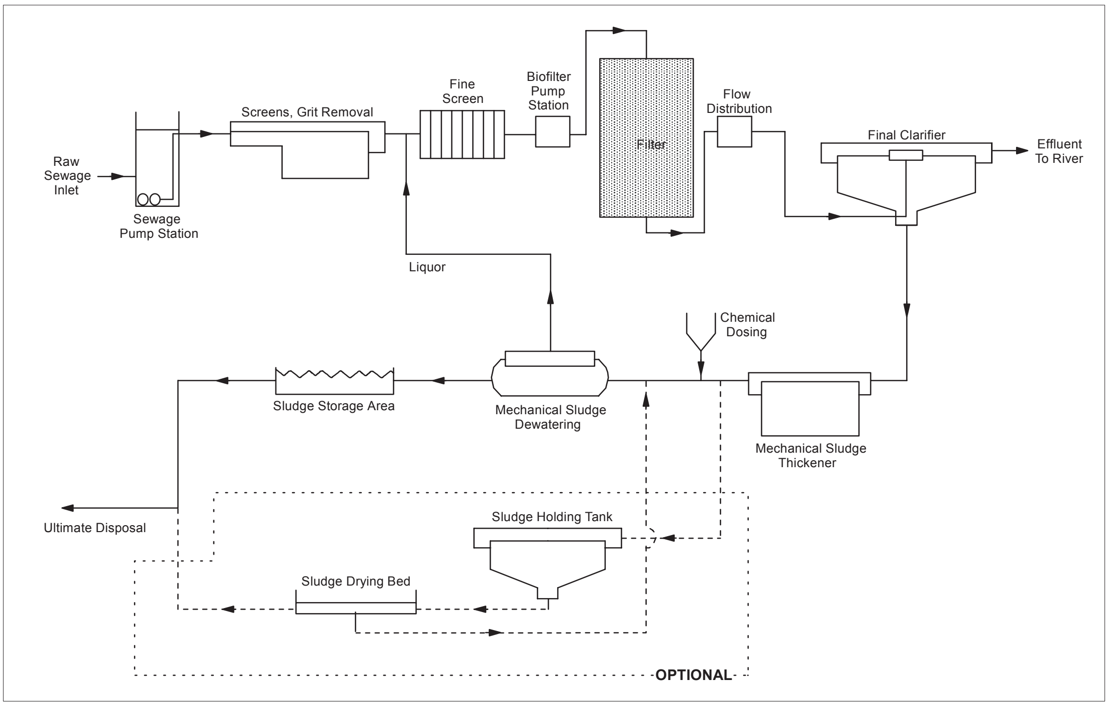


Table 5.15 Design Parameters for Trickling Filter

Description	Unit	Design Criteria
Organic loading **(depending on filter type)** Low rate Intermediate rate High rate	kg BOD_5/day/m^3	 0.08 - 0.15 0.15 - 0.5 0.5 - 2.0
Recirculation of flow to head of plant $\frac{Q_{recycle}}{Q_{inflow}}$ (to maintain wetting rate and improve flow)		> 1.0
Acceptable media		HDPE, PVC, stone, slag, coke, etc. (random or standard arrangement)
Hydraulic loading Low rate Intermediate rate High rate	m^3/day/m^2	 1 - 4 4 - 10 10 - 40
Sludge Yields Low-rate filters Intermediate filters High-rate filters	kg sludge / kg BOD_5 influent	 0.5 0.6 - 0.8 1.0
Minimum depth of media	m	1.5

Refer also to Table D.4 for duty standby requirements and relevant clause MS 1228 for more details.

5.8.6 Sequencing Batch Reactors (SBR) System

5.8.6.1 General Description

Sequencing Batch Reactors system is suspended activated sludge system. In this system, sewage flows into one or more reactors where biological oxidation and clarification of sewage take place within the same reactors sequentially on cyclical mode. There are five (5) basic sequences in a cycle, namely:

1. Fill
2. React (Aeration)
3. Settle
4. Decant
5. Idle

Typically, all actions in the reactor occur at different sequence of time. In other words, the system is intermittently fill and intermittently decant. Typical SBR plant consists of a minimum of two (2) reactors in a plant. When one unit of the reactors is in fill mode, the other reactor(s) may be in the stage of react, settle, decant or idle. Recent development of SBR system leads to the emergence of variation in the operating sequences. Continuous fill and intermittently decant system is one of the variations of this system, where feeding into all rectors are continuous but the other phases (react, settle, decant, idle) are run in sequence.

In the reaction stage, oxygen supplied to the system shall be in accordance to the load to the system within the time frame of reaction cycle. This generally requires higher oxygen capacity per unit time than a continuously aerate system.

In the decant stage, there shall be sufficient time to allow for mixed liquor suspended solids (MLSS) to settle before effluent decanting begins. Decanting time is normally much shorter than fill time. Consequently, the effluent flow rate will also be much higher than influent flow rate. Hence the design of the decanting weir must be capable to handle high over-flow rate without scouring the settled sludge. Therefore, sufficient clear water depth between the minimum water level after decant and the top of the settled sludge blanket must be allowed for to minimise sludge carry over. Hence the depth of water decanted must be restricted to prevent scouring of solids.

5.8.6.2 Design Requirements for SBR Plants

All SBR plants must be designed to cater for peak flows. A minimum of a two (2) tanks system is required. Proven control system in the form of Programmable Logic Controller (PLC) with complete instruction, and operational and training manuals must be submitted together with the design. All SBR systems must be preceded with complete preliminary works. Allowance shall be provided to completely empty a tank for maintenance purposes without interrupting the operating sequence of the plant.

Table 5.16 highlights the key design requirements for an SBR plant.

Table 5.16 Design Requirements for SBR System

Parameter	Unit	Continuous Fill and Intermittently Decant	Intermittently Fill and Intermittently Decant
No. of Reactors	unit	Minimum 2	Minimum 2
Hydraulic retention time at Qavg (at average water level)	hr	18 – 24	18 – 24
F/M ratio	d^{-1}	0.05 – 0.08	0.05 – 0.30
Sludge age	d	20 – 30	10 – 30
Sludge Yield	$\frac{\text{kg Sludge}}{\text{kg BOD}_5\text{ load}}$	0.75 – 0.85	0.75 – 1.10
MLSS (End of decant)	mg/ℓ	3000 – 4500	3000 – 4500
Cycle Time	hr	4 – 8	4 – 8
DO (Reactor) DO (Effluent)	mg/ℓ mg/ℓ	0 ~ 6.5 2.0	0 ~ 6.5 2.0
Oxygen Requirement	$\frac{\text{kg O}_2}{\text{kg Substrate}}$	$\frac{\text{Cycle time}}{\text{Aeration Time}} \times \frac{2.0\text{ kg O}_2}{\text{kg substrate}}$	$\frac{\text{Cycle time}}{\text{Aeration Time}} \times \frac{2.0\text{ kg O}_2}{\text{kg substrate}}$
Decant time	hrs	≥ 1.0	≥ 1.0
Decant depth	m	Max 0.5	max 1.0
Decant volume	%	Not more than 25% of volume of Biological Reactor at TWL	Not more than 30% of volume of Biological Reactor at TWL
Decanting device loading rate*	$m^3/m/hr$	≤ 20 for decant draw-down from TWL	≤ 20 for decant draw-down from TWL
Minimum number of decanter		2 nos. independent decanter per tank	2 nos. independent decanter per tank
Max. pecanter length	m	4.0	4.0
WAS	kg sludge/d	$\text{WAS} = \frac{\text{Total solids in system}}{\text{Slude age}}$	$\text{WAS} = \frac{\text{Total solids in system}}{\text{Slude age}}$
Fill volume	m^3	$V_{fill} = (Q_P\ m^3/hr \times 1.5hr) + (T_{fill} - 1.5) \times Q_{AVG}$ (if no balancing tank) $V_{fill} = Q_{AVG} \times T_{fill}$ (if preceded by balancing tank)	$V_{fill} = (Q_P\ m^3/hr \times 1.5hr) + (T_{fill} - 1.5) \times Q_{AVG}$ (if no EQ) $V_{fill} = Q_{AVG} \times T_{fill}$ (if preceded by balancing tank)

* For continuous fill, length to width ratio shall be based on 3 : 1

* Decanting device loading rate shall be based on V_{fill}/decant time during decanting.

† RAS maybe necessary where length to width ratio poses dilution affect into the inlet.

5.8.7 Design Requirements for Hybrid Systems

A hybrid system is a recent development in biological treatment combining suspended solids and fixed film growth processes. The treatment system may be considered if the design criterion to be adopted has proven performance and result.

5.8.8 Design for Nutrient Removal for Sensitive Receiving Water

Nutrient removal is required for effluent discharge to lakes and stagnant water bodies to prevent eutrophication or other potential impacts that may impede the sensitivity of the receiving water. Nutrient removal can be achieved via:

a) Biological treatment.

b) Physical treatment.

c) Chemical treatment.

It has been emphasised in the beginning of this chapter, all biological treatment system shall be designed to achieve ammonia reduction and where necessary anoxic zone/stage to be added to encourage denitrification for total nitrogen removal.

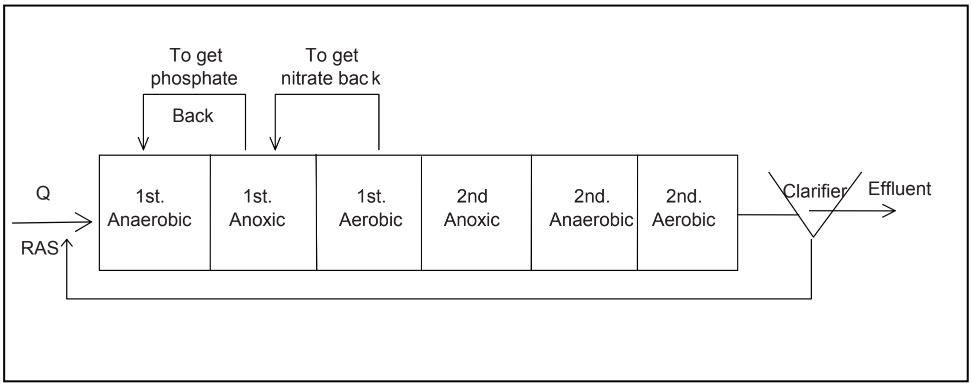
The biological phosphorus removal mechanism is based on the fact that bacteria are capable of storing excess phosphorus as polyphosphate and removing simple fermentation substrates produced in the anaerobic zone and assimilating them into storage products within their cells. Hence, the design for of the biological treatment shall follow the following for plants where nutrient removal (nitrogen and phosphorus) is required, design parameters:

Table 5.17 Design Requirement for Biological Nutrient Removal System

Item	Design Parameters	HRT (hrs)	MLSS (mg/l)	Internal Recycle
1	1st stage anaerobic	1 - 2	2 000 – 6 000	RAS from clarifier
2	1st stage anoxic	2 - 4	2 000 – 6 000	2 to 1 (MLSS recirculation ratio)
3	1st stage aerobic (oxic)	8 - 12	2 000 – 6 000	3 to 2 (MLSS recirculation ratio)
4	2nd stage anoxic	2 - 4	2 000 – 6 000	Mixing
5	2nd stage anaerobic	1 - 2	2 000 – 6 000	Mixing
6	2nd stage aerobic (oxic)	4	2 000 – 6 000	-

It has been emphasised in the beginning of this chapter, all biological treatment system shall be designed to achieve ammonia reduction and where necessary anoxic zone/ stage to be added to encourage denitrification for total nitrogen removal.

Figure 5.15 Typical Process Flow Diagram for Biological Nutrient Removal System



Alternatively, both physical and chemical treatment may be used to remove phosphorus in wastewater.

The designer shall take all necessary consideration in the design in relation to the specific requirements of the receiving water in determining the actual nutrient removal requirement on the case by case basis.

5.9 Design of Secondary Clarifiers

5.9.1 Purpose

Effluent from biological processes contains large populations of micro-organisms (MLSS). Secondary clarifiers provided after the biological system allow the mixed liquor organisms/ solid to settle. Clear supernatant is discharged as the effluent, while some of the settled microorganisms are returned to the biological treatment system to maintain the MLSS concentration and excess microorganisms are removed as biosolid to the sludge treatment facility.

5.9.2 Design Requirements

The design requirements shall include:

I) General

a) Minimum retention time for settlement.

b) Maximum settling velocity for settlement.

c) Sludge hopper to collect settled sludge.

d) All clarifiers must be equipped with sludge scrapers to skim sludge from the bottom unless they are designed with a 60^o hopper bottom.

e) All clarifiers must be equipped with scum skimmer to remove scum from the surface. The scum collected must be drained (where necessary) and disposed off. Returning the scum to the preliminary system or the biological system is not permitted.

f) Multiple hopper are not permitted.

g) Stilling basin to prevent hydraulic shock circuiting.

h) Bottom slope at clarifier floor.

i) Facilities to dispose scum and sludge.

j) Appropriate feed and outlet pipe with hydraulic consideration.

k) Effluent collection channel to be of glazed finish/tiles.

l) Proper maintenance access to all components.

m) Properly designed air lift pumps are only permitted for PE less than 1000.

II) Weirs

a) If insufficient length is available, then considerations shall be given for the use of double weir.

b) Cascading V-notch is preferred over rectangular weirs.

c) Slots in the weir shall be provided to allow for level adjustment during the installation stage.

d) Broad crested weirs are not encouraged.

e) All parts of the weirs must be visible and accessible for regular cleaning.

f) Type of weir and the hydraulic calculation for the weir proposed must be submitted.

III) Circular Clarifiers

a) The maximum diameter permissible is 50 m and a reasonable allowance between tanks shall be provided for vehicle access.

b) The minimum side water depth shall be 3.0 m. Greater side water depths may be used if it can be shown that the mixed liquor is well denitrified in the aeration tank.

c) Flow distribution channel/chamber shall be provided for flow isolation or equalise flow distribution.

d) The scraper tip travelling speed shall not exceed 0.03 rpm. A multiple stage reduction unit must be incorporated to achieve such speed.

IV) Rectangular clarifiers

a) Shall not be wider than 6 m per tank to allow for scraper removal, unless other approved scraper units are available.

b) Multiport wall and baffled inlet shall be provided.

c) Slide gates shall be used to isolate each tank.

d) Allowance also shall be provided for vehicle movement between unit processes for maintenance purposes.

e) Scraper travelling speed shall be between 0.3 – 0.6 m/min.

Refer also to relevant Clause of MS 1228 for more details.

5.9.3 Multiple Hoppers

Multiple hoppers are not accepted. This is due to the settling characteristics of the particles in the flow. Larger and heavier particles settle faster than smaller and lighter particles, creating difference in the distribution of sedimentation in different hoppers. This will present operational difficulties because sludge removal from the hoppers is unequal. To avoid the non-uniform withdrawal of sludge, each hopper in the multiple hopper configuration needs a separate pipe and pump or valve on each outlet.

Table 5.18 Design Parameters for Secondary Clarifiers

Description	Unit	Design Criteria	
		PE ≤ 5,000	PE > 5,000
Minimum number of tanks		2 *	2
Tank configuration		Square Circular Rectangular	Circular Rectangular #
Minimum side water depth	m	3**	3
Minimum hydraulic retention time (HRT) at Q_{peak}	hrs	2	2
Surface overflow rate at Q_{peak}	$m^3/d/m^2$	≤30	≤30
Solids loading rate at Q_{peak}	$kg/d/m^2$	<150	<150
Solids loading rate at Q_{avg}	$kg/d/m^2$	<50	<50
Weir loading rate at Q_{peak}	$m^3/day/m$	<180	<180
Return activated sludge (RAS) pumping rate		Continuous	Continuous
Waste activated sludge (WAS) pumping rate		Continuous or batch	Continuous or batch
Sizing of rectangular tanks			
Length : Width		3:1 or greater.	
Maximum side water depth	m	3.0	
Width : Depth		1 : 1 to 2.5 : 1	
Sizing of circular tanks			
Side water depth, minimum	m	3.0 **	
Floor slope wall		1:12	

Notes:

* For PE less than or equal to 1000 a single clarifier is acceptable.

\# Rectangular tanks are acceptable if equipped with automatic scraping and desludging devices.

** For square clarifier with 60^0 slope minimum 1 m side water depth shall be provided.

5.10 Disinfection

Disinfection refers to the selective destruction of disease causing organisms in sewage effluent.

Methods of disinfection can be physical, chemical or radiation.

Continuous disinfection is required for those areas where the discharge from the sewage works will cause detrimental effect onto the receiving water course, such as bathing beaches, lakes, etc.

The Commission reserves the right to determine the need for the provision of a continuous disinfection facility.

The common forms of disinfection that are available for wastewater applications are:

a) Chlorination

b) Ultra-violet (UV)

c) Others

Chlorination is by far the most common type of disinfection used world-wide. This is due to its effectiveness in providing a good pathogen kill with relative simplicity in operation and maintenance. However, chlorination using chlorine gas, requires a higher degree of operational skills and poses potential health and safety hazards in the shipping and handling aspects. Therefore, to reduce these hazards, only liquid or solid hypochlorite (sodium or calcium) shall be used.

Ultra-violet (UV) disinfection differs from chemical disinfection in that it uses irradiation to induce photobiochemical changes within the micro-organisms. To ensure effective photochemical reaction taking place, one of the conditions is that such radiation must be absorbed by the target molecule (organism). The other condition is that sufficient radiation energy to alter chemical bonds is made available. Given the conditions above, it is critical that the effluent prior to disinfection must be relatively clear of suspended solids. As such, for UV disinfection to be highly effective in wastewater applications, filtration of upstream of the UV unit must be made available.

Other forms of wastewater disinfection that are available are maturation ponds and ozonation. Maturation ponds have been used widely and successfully in Malaysia. However, the drawback is that a relatively large area of land is required to provide sufficient retention time in the pond for the decay of pathogens. Ozone disinfection involves the direct ozone oxidation or by reaction with the radical by-products of ozone decomposition. However, due to ozonation's relatively new status in wastewater applications and higher costs at small scale facilities, its usage for disinfection is still limited.

5.10.1 Design Requirements

All new STPs must have provision for disinfection infrastructure such as contact tank and other necessary structures. The provision of the disinfection facilities and equipment shall be in accordance to Table 5.19 below. The disinfection shall be carried out in continuous or intermittent mode.

Table 5.19: Requirements for Disinfection Facility

Description	Continuous	Intermittent
Class of STP*	Class 4	Class 1 Class 2 Class 3
Type of Disinfection	▪ Chlorination ▪ Ultra-violet (UV) ▪ Ozone	Chlorination
Facility	(1duty/1standby) for equipment	Basic facility structure.

* The Commission may impose separate requirements on case by case basis.

Figure 5.16 Schematic illustration of ultraviolet disinfection system with stilling plate for flow conditioning and elongated weir for level control

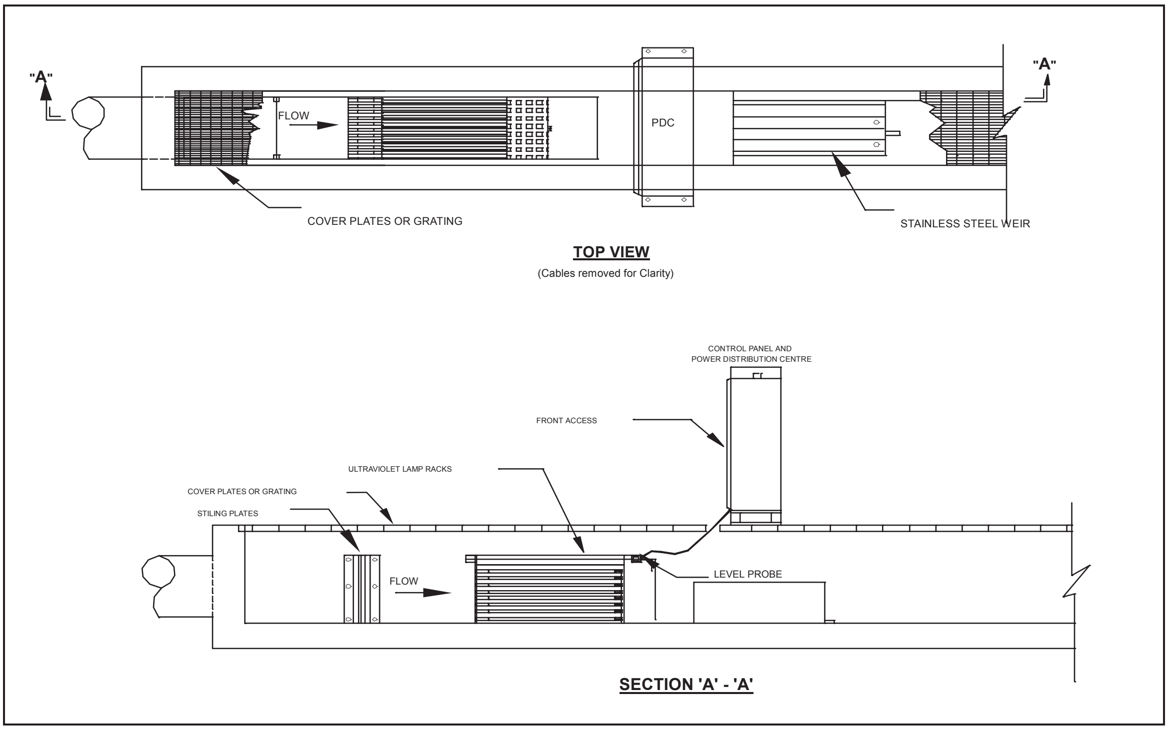


Figure 5.17 Profile schematic of lamp modules relative to inlet and outlet structure

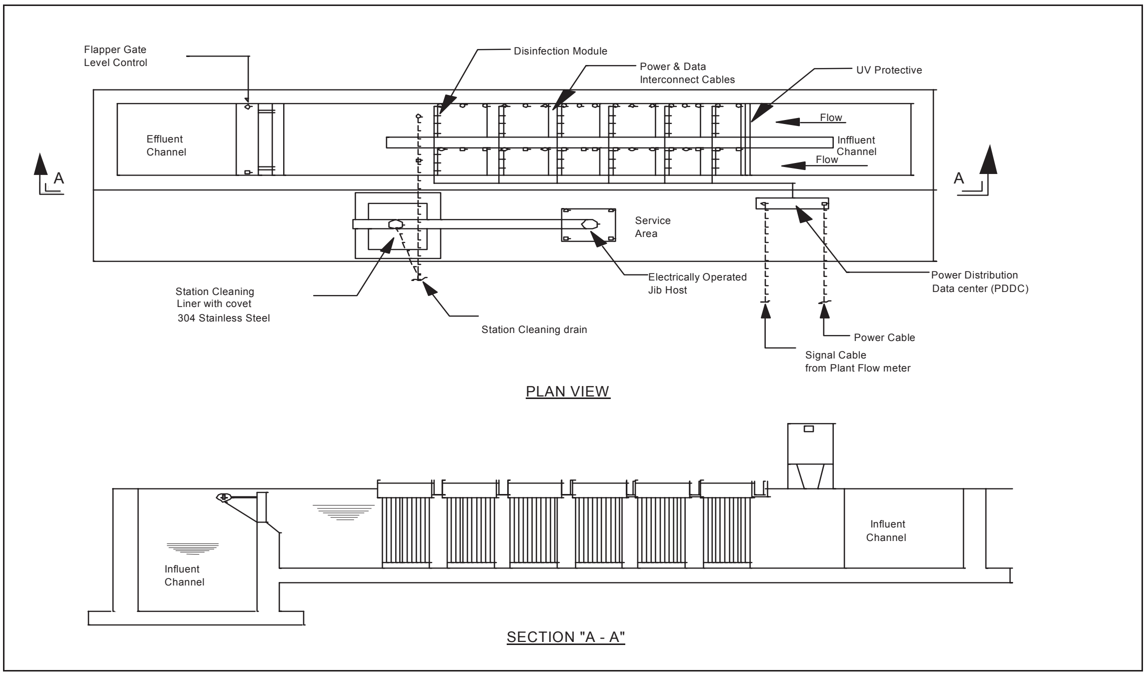
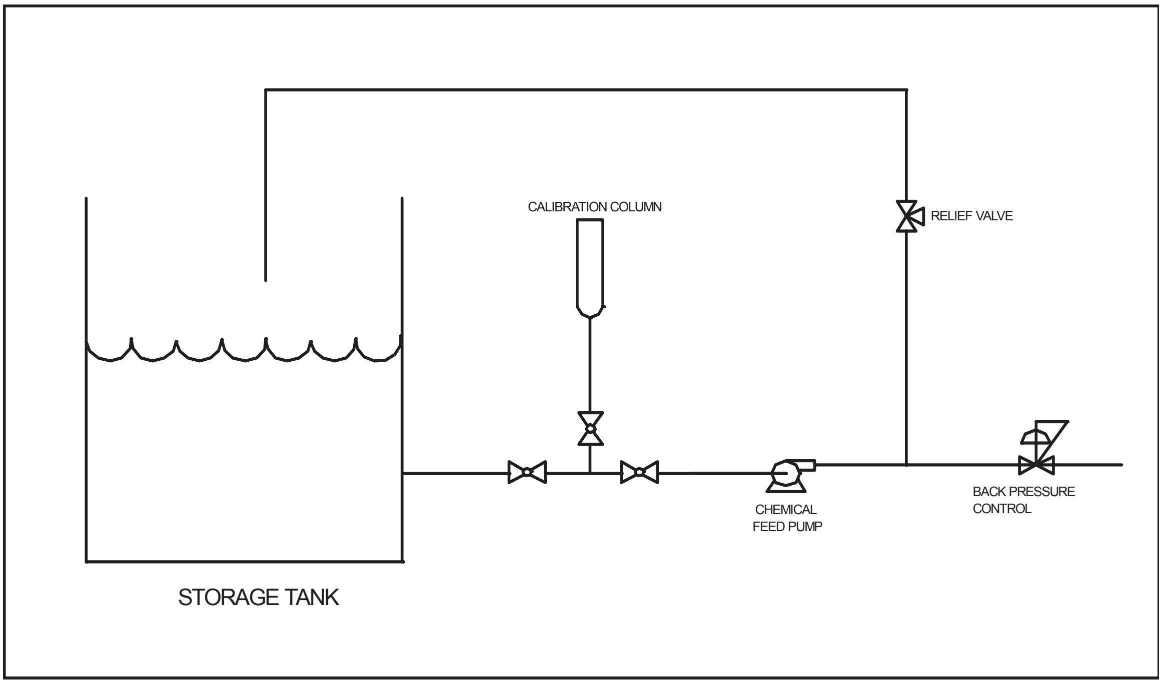


Figure 5.18 Chemical-feed system schematic



5.10.1.1 Continuous Disinfection

I) Disinfection with Ultra-Violet (UV)

a) Filtration ahead of the ultra-violet disinfection is a must in order to ensure consistent and reliable disinfecting performance, as well as, to reduce maintenance, such as, fouling of the UV lamps.

b) The performance of the UV unit shall meet a UV transmittance level of secondary effluent of at least 60% on a filtered basis. If a lower transmittance level is encountered, it shall be compensated with the reduction in the spacing of lamps and/or usage of an advanced higher intensity system to compensate the lower tranmissibility of the sewage effluent.

c) The channel for the UV shall be open, long and narrow to encourage plug flow and avoid short circuiting.

d) As a guide, the average sizing is roughly 10 conventional 1.5 meter lamps per 1,000 m^3/day at peak flow. However, the actual sizing shall be site specific and subjected to effluent quality desired as well as the level of upstream treatment provided.

e) In the design to house the UV modules, it is important to include proper inlet and outlet structures and consider the approach and exit flow conditions.

f) A stilling well is required to distribute the flows and equalise the velocities across the cross-section of the channel. The stilling plate shall be placed at least 5 meters in front of the first lamp bank. Otherwise, the channel should have an undisturbed straight line of two (2) or three (3) lamps length.

g) Sufficient distance shall be allowed between lamp banks (0.5 m to 1.0 m) and two (2) to three (3) lamp lengths between the last bank and the downstream level control device.

h) In large system applications, a multichannel configuration is required. This is to allow the inlet structure to satisfy the dual requirements of inducing flow and to allow even distribution of flow among operational channels. Channel inlet structures shall allow for hydraulic isolation of individual channels during low flow and routine maintenance. In operation, the multichannel design shall be controlled to maintain a minimum velocity through any one channel.

i) Wastewater within the channel must be maintained at a constant level with little fluctuations. This shall be accomplished by using a mechanical counter balance gate downstream of the lamp batteries.

j) It is crucial to avoid a dryness state in the channel during low or no flow conditions to prevent the fouling of the quartz jackets surrounding the lamps and/or causing damage to the UV modules. To alleviate this, for a small STP, a fixed or adjustable weir should be used.

However, sufficient weir length shall be provided to avoid water level fluctuations. In larger STPs using the multichannel design, these flow fluctuations can be attenuated by the opening and closing of channels as needed.

k) Control systems should be simple. Its objective is to ensure that the system loading can be maintained and disinfection accomplished, while conserving the operating life of the lamps. In small STPs, the control system shall consists, of a full duty unit in operation at all times with a similar redundant unit on standby. Manual control and flexibility should be made available to enable the operator to bring portions of the systems in and out of operation, as needed, to adjust for the changes in flow or water quality. In larger STPs, a complete automation is warranted for plants using the multi channel design.

l) Safety aspects of an UV disinfection facility involve mainly the electrical hazards and protection from the exposure to UV radiation. The exposure risks could be minimised as long as the operating lamps are submerged and the lamp batteries are shielded. The UV lamps shall not be operated in air and unshielded. All systems must be equipped with safety interlocks that shut down the modules if they are moved out of their operating positions or the wastewater level falls, leaving any or all lamps exposed to air. Electrical hazards can be minimised by the inclusion of ground-fault-interruption circuitry with each module. This feature is a requirement for all UV systems.

m) The design of the UV system shall allow for easy access to the lamp modules for cleaning and other maintenance tasks. The installation shall have adequate working area for maintenance and servicing of the modules when taken out of the channels. Cleaning of the lamps shall be accomplished using mechanical wipers which may be fitted with chemical injectors and with chemical baths when taken out of the channels.

n) A drainage system back to the head of the treatment works shall be provided to drain back water from the reactors, channels and other related tankage. In addition, a permanent clean-water system is to be made available to allow for rinsing and cleaning needs. A bypass around the UV disinfection facility is to be made available in the event the system is shut down completely for maintenance.

Table 5.20 Design Guides for Disinfection with Ultra-Violet(UV)

Description	Design Criteria
UV system	Mounted in open channel type* Must be preceeded with filtration.
Minimum UV Dose (at 254 nm) at end of lamp life	30 mJ/cm^2 (30,000 $\mu Ws/cm^2$) at Qpeak
Maximum Total Suspended Solids in effluent to UV system	≤ 10 mg/l
Maximum Mean Particle Size in effluent	20 microns
UV Transmittance at 254 nm	65%
Lamp life	≥ 12 000 hours
Minimum UV output at end of lamp life	80%
Operating Temperature	18 – 40 ^oC
Relative Humidity	> 95% at 40 ^oC
UV detection System	UV sensor, transmittance, dose pacing
Lamp Sleeve Cleaning system	a. Mechanical wipers and out of channel chemical cleaning. b. Additional 25% more lamps shall be provided for mechanical wipers.
Standby lamps	25% to be provided with min 2 lamp banks
Disinfection Standards	In accordance with receiving water requirement or effluent usage

Note:
*Enclosed system shall be permitted under special circumstances

II) Disinfection with Hypochlorite

a) Calcium and sodium hypochlorite are hazardous chemicals to handle and use. Calcium hypochlorite is classified as a corrosive and rapid oxidant, while sodium hypochlorite is a corrosive agent. Eye protection, access to an emergency eyewash system and showers must be made available to the operators.

Also, direct contact with the undiluted hypochlorite is likely to cause burns to the skin and clothing. Therefore, it is imperative that protective clothing which includes rubber gloves, must be worn by operators when working with these chemicals. When calcium hypochlorite is being transported in the powder form and mixed in water to form solutions, the operator should always wear eye protection and dust masks. All areas exposed to hypochlorite should be washed thoroughly.

b) Chlorine dosage ranges from 6 to 10 mg/ℓ for effluent. The dosage rate is affected by the suspended solids and ammonia present in the effluent, mixing employed, contact time and the control strategies used for dosing.

c) Proper mixing is important for effective disinfection.

d) If a hydraulic jump is employed as a mixing device, the submergence of the diffuser shall not be less than 230 mm (9 in.) below the water surface and placed before the hydraulic jump at the minimum flow. The hydraulic jump is effective in mixing when the head loss exceeds 0.6 m (2 ft). To ensure adequate mixing is achieved, the evaluation of the flow characteristics should be carried out. As a minimum, the Reynolds number shall be 2.1×10^4 for pipe flow and Froude numbers between 4.5 and 9 for open channels is recommended.

e) Hypochlorinators are chemical-feed pumps used for feeding sodium or calcium hypochlorite. The basic components are a storage reservoir or mixing tank for the hypochlorite solution; a metering pump that consists of a positive displacement pumping mechanism, motor or solenoid and a feed rate adjustment device; and an injection device. Depending on the size of the system, a plastic or fiber-glass vessel may be used to hold a low-strength hypochlorite solution. It is not acceptable to use metals commonly used in the construction of storage tanks to hold the hypochlorite solutions because of the corrosive nature of the chemical which will also expedite the decomposition of the liquid hypochlorite.

f) Feeding of calcium hypochlorite will require a mixing device, usually a motorised propeller or agitator located in the tank. Also, in the tank is a foot-valve and suction strainer connected to the suction inlet of the hypochlorinator.

g) The hypochlorinator shall be feed rate adjustable.

h) The injection fitting shall be similar to that used in the gas chlorinator.

i) A chlorine contact tank shall take the form of rectangular serpentine chamber. Circular chambers for disinfection are not acceptable.

j) Local supply of water shall be made available for the dilution of powder calcium or sodium hypochlorite. A breaker tank for supply of water is required for the proper running of the hypochlorinator. If deemed necessary, the water pressure maybe increased with the provision of water booster pump.

k) Pipe works should be suitable for chlorination applications and well supported.

l) Provision for draining the chlorine contact chamber is required for cleaning and maintenance purposes. This shall include a drain valve located at the bottom of the downstream end of the chamber. The point of discharge shall ensure that adequate treatment is given and this could be done through pumping the content from the chamber and return to the head of the treatment plant.

m) A bypass around the chlorine contact chamber shall also be provided to enable flows to be bypassed during maintenance or servicing. There shall be penstocks upstream and downstream of the chlorine contact chamber for isolation purposes.

n) Adequate access with sufficient turning radius for the vehicle to carry in the chemicals to the disinfection system shall be provided.

o) A small housing structure shall be provided to house hypochlorinator, associated chemicals and ancillaries. Some important consideration have to be given in the design of adequate space for the operators to replace and fill the chemicals, washing facility, eyewash, record keeping of chemical dosing, effluent flowmeter data among others. Due to the hazardous nature of the disinfection system housed, a locking system shall be made available to deter vandalism and promote safety of the plant.

The structure housing the hypochlorinator and the chemicals shall be bunded to prevent the possibility of spillage. The sizing of bunds shall correspond to the total volume of the storage/solution tanks.

Table 5.21 - Design Guide for Disinfection with Hypochlorite

Type	Calcium or Sodium
Dosage	6 – 10 mg/ℓ
Mixing	Mechanical, baffles or hydraulic jump.
Hypochlorinator system	Feed rate adjustable
Equipment	1 duty/ 1 standby
Contact Tank	
Contact Period	15 minutes at Q_{peak}
Maximum depth	3 m
Depth : Width	2 : 1
Min no. of passes	4
Length : Width at each pass	6 : 1
Wetted Depth : Width	< 2:1

5.10.1.2 Intermittent Disinfection

The design requirements for intermittent disinfection facility shall comply to the following:

a) Due to the infrequent usage and other health and safety considerations in an intermittent disinfection system, **ONLY** liquid hypochlorite, either calcium or sodium, shall be used.

b) A chlorine contact chamber shall be provided with a minimum of 15 minutes hydraulic retention time at peakflow. This chamber shall be of a rectangular configuration with aspect ratios optimised to promote plug flow conditions. The recommended aspect ratios are as follows:

   i) Length to width (each "Pass"): 6:1

   ii) Minimum number of passes: 4

   iii) Height to width of the cross-section of the wetted section: < 2:1

   iv) Depth of chlorine contact chamber is typically 2 – 3 m. The corners shall be rounded to reduce the dead flow areas and the velocity through the contact chamber shall be sufficient to minimise solids deposition.

v) horizontal baffles shall be used to ensure the plug flow conditions are maintained and to minimise the possibility of short circuiting.

vi) Upstream of the contact chamber, a dosing coupling for easy hook-up of the mobile hypochlorite disinfection system shall be an inherent part of the chamber design.

c) A breaker tank for supply of water adjacent to the contact tank is required to provide water for the mixing of powder hypochlorite and for cleaning or cleansing of the contact chamber. If deemed necessary, a water booster system shall be provided to increase the water pressure for the intended application.

d) Draining provision must be made available to allow for complete drainage of the chlorine contact tank. For this purpose, a drain valve shall be provided at the bottom of the downstream end of the chamber.

e) Penstock/slide gate shall be provided at the upstream and downstream end of the contact chamber. This allows for the effluent flow from the treatment plant to bypass this chamber when its service is not required.

f) Adequate access shall be provided for a portable hypochlorinator unit mounted on a skid to be brought by a truck to the contact chamber area when disinfection is required. A concrete pad adjacent to the contact chamber shall be provided for the skid mounted hypochlorinator to be situated when in use.

g) Power supply shall be adequately provided and located close to the contact chamber to run the intermittent disinfection system.

Table 5.22 Design Guide for Intermittent Disinfection

Type	Calcium or Sodium
Dosage	6 – 10 mg/l
Mixing	Mechanical, baffles or hydraulic jump.
Hypochlorinator system	Feed rate adjustable
Equipment	1 duty/ 1 standby

Table 5.22 Design Guide for Intermittent Disinfection (cont.)

Type	Calcium or Sodium
Contact Tank	
Contact Period	15 minutes at Q_{peak}
Maximum depth	3 m
Depth : Width	2 : 1
Min no. of passes	4
Length : Width at each pass	6 : 1
Wetted Depth : Width	< 2:1

5.11 Design of Flow Measurement Devices

5.11.1 Purpose of Flow Measuring Devices

Flow measuring devices are necessary for monitoring of plant operation and process control continuously. The purposes of flow devices are:

a) to maintain flow records periodically for future reference, especially when plant expansion is needed.

b) to identify the flow pattern which may be due to population growth or infiltration.

c) Statutory requirement by the DOE to maintain flow records at all sewage works.

d) To establish operational cost for treatment of sewage.

5.11.2 Design Requirements for Flow Devices

Flow devices are mandatory for all STP, regardless of size.

Table 5.23 Design Parameters for Flow Devices

Description	Design Criteria	
	PE ≤ 5,000	PE > 5,000
Location of flow meter	Inlet or Outlet	Inlet and Outlet
Type of Flow Measurement	Closed Conduit or Open Channel	Closed Conduit or Open Channel
Type of flow meter	V- notch (Outlet Only) Rectangular Weir (Outlet Only) Flumes Electromagnetic Ultrasonic	V-notch (Outlet Only) Rectangular (Outlet Only) Flumes Electromagnetic Ultrasonic
Method of measurement	• Automated or manual measurement of Staff gauge to measure height of crest with calibration curves / tables	Automated devices linked to data logging with integrator and transmitted to chart recorder (minimum 7 days chart time)
Measurement times	Continuous or Intermittent	Continuous

5.12 Sludge Holding, Treatment and Disposal

5.12.1 Introduction

All treatment processes are capable of producing significant quantities of sludge which requires to be further treated. The sludge comprises essentially inert and organic matters that are biodegradable and non-biodegradable present in sewage, and bacterial cells generated by the biological treatment processes. The treated sludge, often referred as biosolids is ready for safe disposal or reuse.

The importance of sludge management increases with the increase in the amount of sewage treated. Space has to be allowed within the premises of an STP to accommodate sludge treatment, handling and storage facilities.

All sludge need to be treated for safe disposal back to the environment. The minimum requirement for sludge treatment is to achieve stabilize sludge with a 20% dry solid content.

For large scale development whereby the full sludge generation will only be achieved over a certain time period, proper sizing/modulation of sludge treatment facilities need to be provided in order to achieve the immediate needs for sludge treatment.

5.12.2 Sludge Strategy in General

Figure 5.20 shows the typical sludge treatment and disposal strategy which consists of three main stages.

a) Stage 1 - Preliminary treatment and digestion

b) Stage 2 - Conditioning and dewatering

c) Stage 3 - Utilisation and disposal

I) Stage 1 - Preliminary Treatmentand Digestion

Preliminary treatment may include reception or holding facility for screened sludge, primary thickening and digestion facilities.

For imported sludge the reception facility may comprise of an unloading area, screen chamber, reception tank and transfer pump(s).

Thickening equipment, such as, centrifuge, drum thickener or gravity belt thickener is provided to thicken the raw screened sludge from about 1% dry solids content to about 6% dry solids content. To assist the thickening process, an 'in-line' polymer dosing system or chemical conditioning shall be provided.

Two types of digestion facilities are available for digestion after the thickening: aerobic and anaerobic digestion.

Secondary thickening is recommended to reduce the volume of digested sludge, which will then reduce the size and the number of the next treatment process unit, i.e., dewatering equipment.

II) Stage 2 - Conditioning and Dewatering

Dewatering can be achieved by two (2) methods : mechanical dewatering and non-mechanical dewatering.

a) Mechanical dewatering such as belt filter press, centrifuge or filter press is provided for sludge dewatering purposes. To assist the mechanical dewatering equipment in achieving optimum level of cake dryness, an 'in-line' polymer dosing system or chemical conditioning shall be provided.

b) Non-mechanical dewatering, e.g. drying beds or sludge lagoons is often used

For a facility serving ≥ 10,000 PE, the drying beds must be designed to support mechanical/machine lift for the purpose of clearing the dried sludge.

Sludge lagoons of about 2 m depth are also used for sludge stabilisation and drying. The sludge lagoons shall be sized to receive sludge for a period of at least 6 months and are allowed to undergo stabilisation through evaporation and drying for another 6 months period. The lagoons shall be lined with either PVC lining, concrete or 600 mm thick clay lining. An access ramp shall be provided to allow mechanical equipment access to clean dried sludge.

III) Stage 3 - Utilisation and Disposal

After the dewatering process, an on-site storage for 30 days of the treated bio-solid shall be provided. The storage structure shall be covered with roof and provided with partly opened walls to allow for proper ventilation.

Finally, the bio-solid is either composted and/or applied directly for land reclamation (i.e., for ex-mining land), land application (i.e., for certain types of agriculture land and forest land/reforestation) or used as top soil cover at land fill site. The ultimate disposal of bio-solid is the responsibility of the plant operator.

5.12.3 Provision of Sludge Holding, Treatment and Disposal

The Service Licensee will advise on current capacity in its existing sludge treatment facilities, suitable sludge stabilisation, dewatering and final disposal of the sludge shall be provided.

If the Service Licensee has the capacity to receive sludge generated from the development, then the project proponent has the option to negotiate with the Service Licensee to dispose off the sludge at the existing facility. In this case, a sludge storage tank with a minimum capacity to hold for 30 days with the sludge thickened to 1% solids is acceptable. Otherwise, the sludge shall be stabilised, dewatered and prepared in a suitable form for disposal.

Different types of STPs produce different quantities of bio-solid. The principal assumptions adopted on waste generation rates are summarised in Table 5.24.

Table 5.24 - Sludge Generation Rates

Treatment System	Unit Generation Rates	Comments
Primary Sludge		
Primary Clarifier	0.5 kg sludge/kg solids input	Based on continuous sludge withdrawal
Imhoff Tank	0.15 kg sludge/kg SS input	Based on average 6 month desludging period
Secondary Sludge		
Conventional Activated Sludge System	0.8 to 1.0 kg sludge/kg BOD_5 removed	Standard A/B
Extended Aeration or Oxidation Ditch	0.4 to 0.6 kg sludge/kg BOD_5 removed	Standard A/B
RBC/SBC/High Rate Trickling Filter System	0.8 kg sludge/kg BOD_5 removed	Standard A/B
Hybrid System	0.4 kg sludge/kg BOD_5 removed	Standard A/B

Note:
Based on the above assumptions, the quantity of waste sludge requiring treatment and disposal can be estimated. Refer also to design guides related to each of the above individual processes.

5.12.4 Design Criteria

The ultimate aim of sludge treatment is to achieve at a minimum stabilised sludge with dry solids content of 20% for final disposal. A combination of various unit processes may be used to achieve this minimum requirement.

I) Sludge Reception/Sludge Holding

a) An unloading area is normally provided to receive sludge tankers delivering imported sludge to the facility, if necessary. It should also includes a parking area for sludge tankers.

b) A mechanically raked screen with 12 mm opening together with a manually raked by-pass screen shall be provided where necessary.

c) Connection fitted female coupling with ball valve shall be provided at the reception facility for the connection of desludging tanker's hose.

d) Minimum three (3) days sludge holding capacity of between 1 to 4% dry solids content sludge (depending on the types of sludge) shall be provided after the screening process.

e) Overflow pipe shall be provided at sludge holding tank to aeration tank to avoid overflowing.

f) Adequate ventilation/air extraction fan shall be provided at the thickening/dewatering house.

II) Solid Thickening

Thickening is a process used to increase the solids content of sludge by removing a portion of the liquid fraction. It is generally accomplished by physical means, including co-settling, gravity settling, flotation, centrifugation, gravity belt and rotary drum. The design parameters for sludge thickening equipment shall follow Table 5.25 below:

Table 5.25 - Design Parameters for Sludge Thickening

Type of Thickening	% Dry Solids	Polymer System	Speed of Sludge feed pump	Backwash water system
Picket Fence Gravity Thickener	1.5	n/a	n/a	n/a
Dissolved Air Flotation	2			
Belt Thickener	4	Yes with appropriate polymer turndown ratio	< 300 rpm	Yes
Drum Thickener	4			
Centrifuge	4			

Note:

a) Mechanical thickener shall be designed for 8 hrs/day and 5 days/week operation.

b) For belt, drum and centrifuge thickener, three polymer injection points shall be provided

c) Potable water to be provided for polymer mixing system.

III) Solid Digestion

Sewage biosolids in its natural state (raw) is rich in pathogenic organisms, easily putrescible and rapidly developing unpleasant smells. Stabilization processes were developed with the purpose of stabilizing the biodegradable fraction of organic matter present in the bio-solids, thus reducing the risk of putrefaction as well as diminishing the concentration of pathogens. The stabilization processes can be divided into:

a) Biological stabilization – specific bacteria promote the stabilization of the biodegradable fraction of the organic matter.

b) Chemical stabilization – chemical oxidation of the organic matter accomplishes sludge stabilization.

c) Thermal stabilization – heat stabilizes the volatile fraction of sludge in hermetically sealed containers.

The most widely used stabilization process is biological stabilization via anaerobic and aerobic digestion.

Table 5.26 - Design Parameters for Aerobic and Anaerobic Digestion

Description	Unit	Design Criteria	
		Aerobic Digestion	Anaerobic Digestion
Number of Tank, Minimum	No.	2	2
Min. Solids Retention Time	Days	10	18
Organic Loading Rate	$KgVS/m^3.d$	1.6 – 4.8	0.8 – 1.6
Typical Feed Solids Concentration	%	2	2 - 6
Type of Mixing		Aerators Diffusers	Gas Injection Mechanical Stirring Mechanical Pumping
Min. Water Depth, minimum	m	3	7.5
Tank Shape		Cylindrical Rectangular	Cylindrical Egg-Shaped
Tank Dimension, maximum	m	25 diameter 25 length	25 diameter
Dissolved Oxygen	mg/L	1 - 2	-

IV) Sludge Dewatering

Table 5.27 Recommended Design Parameters for Sludge Stabilisation and Dewatering

Descriptions	Unit	Design Considerations	
		PE ≤ 2,000	PE > 2,000
Sludge Stabilisation			
Type of stabilisation process		Simple anaerobic or aerobic digestion	Ambient anaerobic digestion with good mixing facility
Hydraulic retention time (HRT) minimum	Days	30	30
Dewatering			
Type of device		Belt press Centrifuge Filter press Drying bed	Belt press Centrifuge Filter press Drying bed *
Minimum dry solids (content after dewatering)	%	20	20
Operating period of mechanical thickening and dewatering facility		5 days/week # 8 hours/day 250 days/year	5 days/week # 8 hours/day 250 days/year
Handling capacity of drying bed		4 weeks cycle on 450 mm thick feed †	4 weeks cycle on 450 mm thick feed †
Covered storage area		1 month holding	1 month holding

Notes:

a) Access ramp of at least 1.5 m wide shall be provided at all sludge drying beds

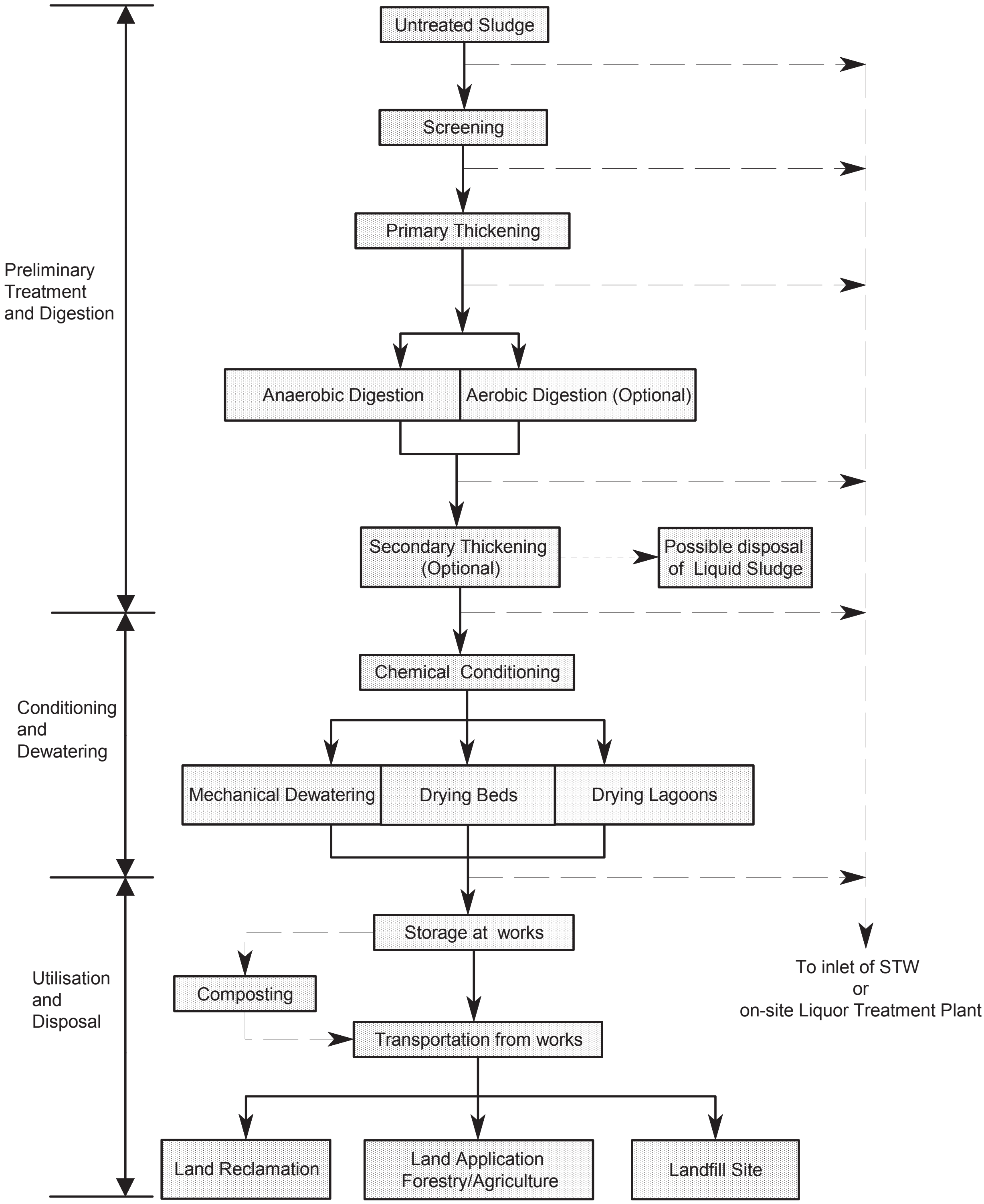
* Drying beds must be designed to support mechanical/machine lift for more than 10 000 PE.

Design to be based on one full-time working shift only.

† In computing the area requirements of a sludge drying bed, the designer may assume a maximum 450 mm depth of sludge feed to the bed. The actual quantity of sludge from the upstream unit processes needs to be computed before sizing the bed. Each bed may be designed to handle a maximum of 7 days continuous feed. The next feed to the same bed shall only be after a minimum of 21 days from the last feed. A one-third (1/3) reduction in actual land area requirement will be acceptable if fully covered drying beds are provided. Reduction shall only apply to the total surface area of drying

bed. No reduction is allowed for the drying bed thickness. Structures and materials used for the drying bed covers shall be designed to an acceptable structural strength and of acceptable quality to withstand local weather conditions.

Figure 5.20 Sludge Treatment and Disposal Strategy



V) Ultimate Disposal

The treatment plant project proponent has to indicate in the proposal the ultimate disposal options and actual volume for disposal throughout the life time of the plant.

5.13 Tertiary Treatment

5.13.1 Introduction

Tertiary treatment is associated with the requirements to further reduce or remove pollutants beyond the levels achieved by common secondary treatment processes. Such requirements can be in the forms of the removal of nutrients, such as, nitrogen and phosphorus; lower BOD_5 or SS levels; or trace elements of toxic constituents, such as, heavy metals or refractory organics.

The various methods of tertiary treatment include simple maturation ponds, adsorption, chemical treatment and filtration; air stripping, mambrane or reoxygenation.

Tertiary treatment is required before discharging to very sensitive receiving waters. The Commission will specify the need for such treatment on a case-by-case basis, depending upon the sensitivity of the project.

5.13.2 Design Requirement

I) Filtration system

a) Filtration is the most common tertiary treatment system used to remove suspended or colloidal matter in the effluent.

b) Backwashing shall be limited to once per day. The volume of backwash water shall not exceed 10% of plant throughout. Backwash water shall be stored in a buffer tank before being return to the inlet of the plant.

c) Where used, the facility for dosing conditioners shall be provided at the inlet of the filter system.

d) On-line turbidity meter, level detector and flow measurement shall be utilized to measure filter performance.

e) If the filters are housed in a building, adequate and safe access shall be provided for maintenance purposes.

f) The filters shall have automated backwash features and sized adequately to allow continuous filtration.

g) For package plants, it is preferred to use FRP as the filter vessel. However, fabricated steel is also acceptable provided that protective coatings are included. For larger plants, the use of reinforced concrete is encouraged.

II) Adsorption (Activated Carbon)

a) Activated carbon is used to remove small quantities of refractory organics, as well as inorganic compounds, such as nitrogen, sulficles and heavy metals.

b) Adsorption shall be preceded by filtration using granular media to ensure a consistant feed quality, which is affected by pH, temperature and flow rate.

c) Uniform feedwater to avoid any surges that might adversely affect the carbon adsorption.

d) Clarity of feedwater is important to avoid restriction of pores or build up of materials within the pore structure.

e) Backwashing rate and the frequency required depend on the hydraulic loading and operational method. Typical duration of backwashing is 10-15 minutes.

III) Chemical Treatment

a) Chemicals can be used as tertiary treatment for acid-base neutralisation and precipitation of phosphorous.

b) Phosphorous precipitation requires the addition of coagulants, which usually are lime, alum, sodium aluminate, ferric chloride and ferrous sulfate.

c) Dosing systems and safety features to be provided to assure the operation and maintenance of the systems can be carried out in a safe and healthy environment.

IV) Air Stripping

a) This method is used to remove ammonia nitrogen (NH_4 – N) from effluent.

b) The design features shall depend on the required level of nitrogen removal with the critical parameters being tower packing, quantity of air supply, air and liquid temperatures and process control measures.

V) Reoxygenation

a) This method is used to increase the dissolved oxygen (DO) levels in the effluent.

b) The different types of reoxygenation systems are cascade, reoxygenation and mechanical reoxygenation

c) Cascade reoxygenation is achieved using on the hydraulic design of structures, such as, weir overflows, flumes, spillways, etc.

d) Mechanical reoxygenation is achieved using mechanical equipment such as surface aerators, jet diffusers or diffused air (coarse, fine bubble, etc.)

e) Design of structures or mechanical equipment is based on the amount of DO required for the effluent.

VI) Maturation ponds

a) Pond systems are normally not encouraged because it requires large land area and the inherent difficulty in controlling algal growth. In special cases, where land is in abundance, the project proponent may choose to use this system.

Figure 5.21 Typical Roof Details for Covered Sludge Drying Bed