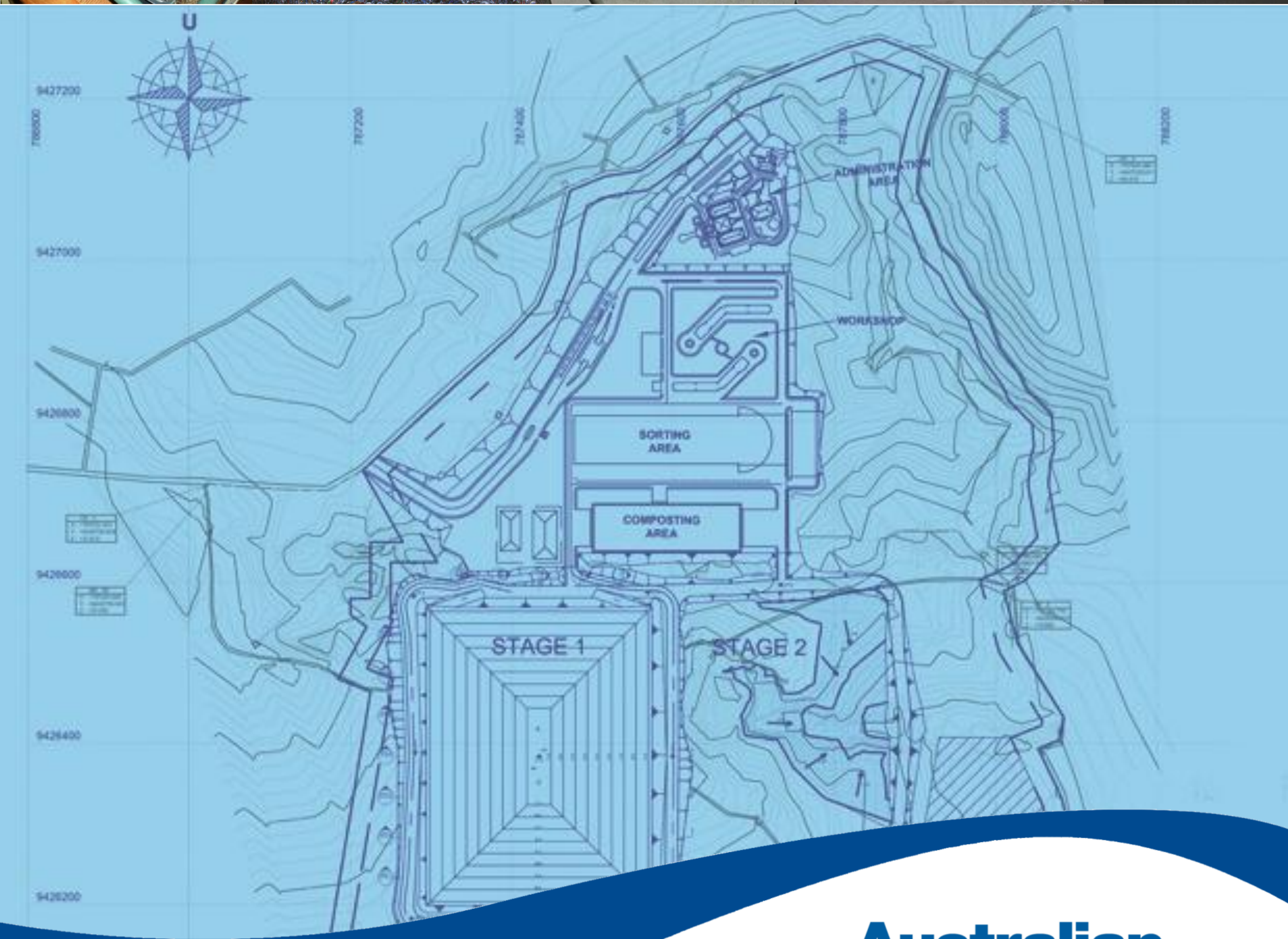
# DETAILED ENGINEERING DESIGN for MAMMINASATA RSWDS

Gowa Regency, South Sulawesi

FINAL DESIGN REPORT

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May 2012

Indonesia
Infrastructure Initiative

## INDONESIA INFRASTRUCTURE INITIATIVE

This document has been published by the Indonesia Infrastructure Initiative (IndII), an Australian Government funded project designed to promote economic growth in Indonesia by enhancing the relevance, quality and quantum of infrastructure investment.

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## ACKNOWLEDGEMENTS

This report has been prepared by EnviroSolutions & Consulting Pte Ltd (ESC) in association with Vista Consulting Engineers Limited (Vista) who were engaged under the Indonesia Infrastructure Initiative (IndII), funded by AusAID, as part of the Activity to prepare the Detailed Engineering Design (DED) for the Regional Solid Waste Disposal Site (RSWDS) Mamminasata, South Sulawesi Province, Indonesia.

Any errors of fact or interpretation are solely those of the author.

Jakarta, May 2012

# TABLE OF CONTENTS

# LIST OF TABLES

# LIST OF FIGURES

# EXECUTIVE SUMMARY

**Introduction**

The Indonesia Infrastructure Initiative (IndII) has appointed EnviroSolutions & Consulting Pte Ltd (ESC) in association with Vista Consulting Engineers Limited (Vista) to prepare the Detailed Engineering Design (DED) for the Regional Solid Waste Disposal Site (RSWDS) Mamminasata, South Sulawesi Province, Indonesia. The detailed scope of work and programmme for the DED was set out in the study's *Inception Report (Revision 1)* (March 2010).

This report represents the amended Final Design Report for the study and presents the final versions of each of the design and study components/tasks, and includes final drawings, descriptive text, bill of quantitiies (BoQ) and specifications for: the landfill, leachate treatment plant (LTP), sorting and composting plant and supporting facilities. The final tender documents, recommendations for the feasibility of a plastics recycling plant and the final access road study, as well as the final version of the training materials are also included.

The final designs have been developed with a view to maximising efficiency, minimising capital and operating costs whilst ensuring environmental provisions are adequate to meet design criteria. ESC seek endorsement of the designs presented to enable completion of this project.

**The Project**

The RSWDS will have an ultimate capacity of over 5 million cubic metres, and will be constructed in two stages, with the first stage covered by this IndII project. The goal of the RSWDS is to contribute to long-term development needs of Indonesia through supporting best practice and sustainable solid waste management for large urban areas. The DED is to employ best practice standards to serve as a model for other Solid Waste Disposal Services (SWDSs) subsequently built in Indonesia.

Further details on the project are detailed in *Section 2.*

**Design Philosophy**

The objective of the detailed design is to develop an integrated solid waste management facility that provides environmental protection, in particular to surface water, groundwater and air quality, while employing fundamental, cost effective technologies that can be constructed, operated and maintained under the constraints of the current local “state-of-the-practice” that demonstrably works in developing countries.

In developing the concept design options, the various available documents previously prepared for this Site; including: the Pre-Feasibility Study; the Preliminary Design (SAPROF); and the Environmental Impact Assessment (AMDAL); were reviewed with a view to;

- Verifying key data (e.g. waste volumes and design capacity);
- Maximising the efficiency of the design;
- Minimising operation & maintenance costs through simplifying the design; and
- Reducing the potential for breakdown by minimising mechanical equipment.

**Landfill Capacity**

Based on the assessment performed herein, the required 5 million cubic metres of airspace volume for 15 years operation indicated in the SAPROF appears to be somewhat conservative; however, it was agreed that it be maintained and that the design of the Landfill Waste Disposal Area be based on this value.

**General Site Details**

Following the movement in site location additional topographical surveys were commissioned to re-survey the entire site. However the site investigation has revealed the general stratigraphy of the site and that the groundwater level is generally between 3m and 5m below the existing ground surface.

**Site Layout & Formation**

The southern landfill layout option was adopted and has been detailed in the design with the leachate treatment plant (LTP) in the southeast, landfill cells west to east across the middle of the site with sorting/ composting facilities immediately north of the landfill on a cut platform. Provision for future landfill gas (LFG) facilities is provided northeast of the landfill. Offices and supporting facilities are located further to the north, for the comfort of occupants. A landscaped 25m buffer zone and vegetated soil stockpiles will provide a visual screen from residences to the south.

Site earthworks will comprise a 'cut & fill' balance over the landfill life of Stage 1 of approximately 0.7 million $m^3$ inclusive of some 213,000$m^3$ cover material, in a 15m – 20m high stockpile along the southern boundary. Slopes designed to 3 horizontal and 1 vertical ensure slope stability and control soil erosion. Only Stage 1 of the entire landfill project is to be constructed at the outset.

**Landfill Systems**

The Stage 1 Landfill will be approximately 360 m by 270 m in plan dimensions; sub-divided into four (4) approximately equal Cells of 180m by 135m, and excavated to a depth of not more than 3 m to 4 m below the existing ground surface so as to remain above the anticipated groundwater level and to provide an adequate elevation head to maintain the gravity flow of leachate to the adjacent treatment facilities. The landfill base lining system will comprise of a geosynthetic clay liner (GCL) overlain by a 1.5mm high density polyethylene (HDPE) textured-surfaced geomebrane.

The leachate collection system comprises a rib & spine network of perforated pipes in trenches overlain by a granular layer with vertical rise pipes to further promote air circulation. Leachate flows to a sump and onwards via gravity to buffer storage and the LTP.

LFG is forecast at up to 5,000 million litres/yr in 2030 with up to 3,000 Ml/yr recoverable for future utilisation via a network of extraction wells.

Cells will be filled sequentially to promote surface water runoff with waste placed to an elevation of 71m (Stage 1) with a final cap comprising 5 layers of soil, compacted soil, drainage layer, soil and topsoil with a vegetated surface.

Surface water management in the landfill and around the site is via a series of surface channels designed to cope with at least a 50-year rainfall event, estimated to be of an intensity of approximately 300mm/hour.

Soil erosion and sediment will be controlled through the use of silt fences, retention pond and through reuse of topsoil to establish a vegetative cover over the disturbed areas as soon as practically possible.

Environmental control is facilitated by a network of monitoring wells and an AMDAL checklist included in the report.

**Leachate Treatment**

A detailed analysis of leachate generation over the life of the landfill has been undertaken inclusive of all contributions, of which rainfall is the primary and most variable component, indicating a maximum of approximately 1,250$m^3$/day in 2021.

Leachate influent quality has been determined based on samples taken from Tamangapa landfill and available literature for similar landfills in Asia. The LTP is designed based on maximum mass loadings of key pollutants over the landfill life and Indonesian leachate discharge standards.

The *Concept Design Report* evaluated the various treatment processes available and concluded that a biological treatment system would likely be required to meet discharge standards. Detailed evaluation shows this is indeed the case, although and the adopted design minimises mechanical parts to maximise reliability. Consideration is also given to maximising, gravity fed, passive treatment in the event of mechanical failure.

The LTP comprises buffer storage, an anaerobic process to address acetogenic leachate, anoxic/aerobic biological treatment to address lower concentrations of $COD/BOD_{5,}$ and also ammonia removal through nitrification-denitrification. A reed bed is included for final polishing by further removal of suspended solids from the treated wastewater before discharging to the river.

**Sorting Plant**

The sorting plant is to sort recyclables such as organics, plastics, paper, metal and glass from incoming waste streams with residues being disposed of within the landfill. The sorting plant (as specified in the SAPROF) is to have a capacity of 1,500 tonnes per day. The Final Design has been developed with a view to maximising efficiency, minimising capital and operating costs whilst ensuring environmental provisions are adequate to meet design criteria.

A flexible staged implementation programme in respect of sorting capacity has been developed to allow decisions to be made in the future as to how to respond to increased waste inputs (increased staff vs increased equipment) and also to allow for any differences in predicted and actual waste inputs. It should be noted that the design drawings have been produced on the basis of the maximum level of mechanical equipment as detailed within 'Alternative Sorting Plant Development – Stage 3' in Section 8.2 of the main report.

**Composting Plant**

The composting plant is designed to have two roles – pretreatment of waste prior to disposal within the landfill and the production of compost. The composting plant is required (as specified in the SAPROF) is to have a capacity of 50 tonnes per day. Per the Preliminary Design the plant has been designed with pre fermentation and fermentation areas for composting with a total area of 150m by 56m (8,400m$^2$)

**Supporting Facilities**

Site supporting facilities for the proposed RSWDS consist of both structures (management office, community facility, workshop, weighbridge, auditorium etc) and key services (water & electrical supply and sewage & sanitation). All of these facilities are described in the report and final design drawings presented in the Annexes.

# CHAPTER 1: INTRODUCTION

## 1.1 BACKGROUND

The Indonesia Infrastructure Initiative (IndII) has appointed EnviroSolutions & Consulting Pte Ltd (ESC) in association with Vista Consulting Engineers Limited (Vista) to prepare the Detailed Engineering Design (DED) for the Regional Solid Waste Disposal Site (RSWDS) Mamminasata, South Sulawesi Province, Indonesia. The detailed scope of work and programmme for the DED was set out in the study's *Inception Report (Revision 1)* (March 2010).

The RSWDS will have an ultimate capacity of over 5 million cubic metres, and will be constructed in two stages, with the first stage covered by this IndII project. The goal of the RSWDS is to contribute to long-term development needs of Indonesia through supporting best practice and sustainable solid waste management for large urban areas. The DED is to employ best practice standards to serve as a model for other Solid Waste Disposal Services (SWDSs) subsequently built in Indonesia.

This report represents the amended Final Design Report for the study and presents the final versions of each of the design and study components/tasks, and includes final drawings, descriptive text, bill of quantitiies (BoQ) and specifications for: the landfill, leachate treatment plant (LTP), sorting and composting plant and supporting facilities. The final tender documents, recommendations for the feasibility of a plastics recycling plant and the final access road study, as well as the final version of the training materials are also included.

The final designs have been developed with a view to maximising efficiency, minimising capital and operating costs whilst ensuring environmental provisions are adequate to meet design criteria. ESC seek endorsement of the designs presented to enable completion of this project.

The main components of the RSWDS DED project are the review of all existing relevant reports and data and the delivery of the following seven key components:

- Detailed Engineering Design (DED) for the landfill and leachate treatment systems;
- DED of the sorting plant and composting plant;
- DED of supporting facilities for the landfill;
- Specifications for the procurement of vehicles and heavy equipment for landfill operations;
- Preparation of Prequalification and Bid Documents;
- Engineering study for Access Road; and
- Preparation of training and operation manuals.

# CHAPTER 2: PROJECT DESCRIPTION

## 2.1 THE INDONESIA INFRASTRUCTURE INITIATIVE (INDII)

The Indonesia Infrastructure Initiative (IndII) is funded by the Government of Australia through the Australian Agency for International Development (AusAID). IndII aims to assist national and sub-national agencies improve infrastructure in Indonesia. IndII, valued at A$50m over three years, is managed by SMEC International Pty Ltd (SMEC). IndII work across a range of infrastructure sectors, including transportation, water and sanitation, and telecommunications.

IndII's program of work is divided into three components:

- *Component 1: Infrastructure Project Management (IPM).* Objective: More efficient and effective infrastructure investment through better identification, coordination, planning and implementation of infrastructure projects that have potential to make a contribution to economic growth.
- *Component 2: Policy and Regulatory (P&R).* Objective: More efficient and effective infrastructure investment through reducing uncertainty in the policy, regulatory and financing environment.
- *Component 3: Infrastructure Enhancement Grants (IEG).* Objective: Enhanced economic and social impact of infrastructure projects through providing financial support to priority infrastructure projects.

The RSWDS DED comes under component 1.

## 2.2 THE REGIONAL SOLID WASTE DISPOSAL SITE (RSWDS), MAMMINASATA

### 2.2.1 Background

Solid waste disposal in urban settings is a significant problem in Indonesia as in other parts of the world. The *Waste Management Law* UU 18 of 2005 states that municipal governments are obliged to close existing open dumping sites within five years and the improvement of solid waste management is a priority in the 2004-2009 medium term development plans.

Under the Second Indonesia Development Policy Loan (IDPL2) a decline in open disposal of solid waste is one of the indicators identified. Regional Integrated Waste Management Centres (RIWMC) will become increasingly important as large suitable disposal sites become more difficult to locate, and as the technology to manage the waste becomes more complex.

The proposed location of the first RSWDS in South Sulawesi Province was selected by BAPPENAS (Ministry of Spatial Planning) and the Director General of Housing and

Settlements (DGHS), Ministry of Public Works (MPW). These agencies expect that it will become an example of a good RIWMC project.

With support from the Japan International Cooperation Agency (JICA), preparatory work progressed to where a detailed engineering design (DED) and bid documents for the solid waste disposal site (SWDS) can be prepared.

### 2.2.2 Background to the Selection of the Mamminasata Project

Mamminasata Metropolitan area comprises Makassar City, Maros Regency, Gowa Regency and Takalar Regency in South Sulawesi Province. The area has been developing as the “engine for development” not only for Sulawesi, but also for the whole of Eastern Indonesia in terms of industry, transportation, trade, social services, etc.

However, because of the absence of proper solid waste management systems, the region is facing serious environmental and sanitation problems such as; harmful environmental conditions at existing dump sites, and the increasing illegal waste dumping in rivers, canals and roads.

With assistance from JICA a Special Assistance for Project Formation (SAPROF) study titled *“The Study on Implementation of Integrated Spatial Plan for the Mamminasata Metropolitan Area”*, was completed in July 2006. This study identified the project scope and the implementation/operation and maintenance framework, necessary to establish feasible, efficient and effective improved solid waste management in the area. The study identified the lack of remaining capacity at the Tamangapa dumping site, (the current dumping site in Makassar City), as the most critical issue, since its remaining capacity is for the next five years only.

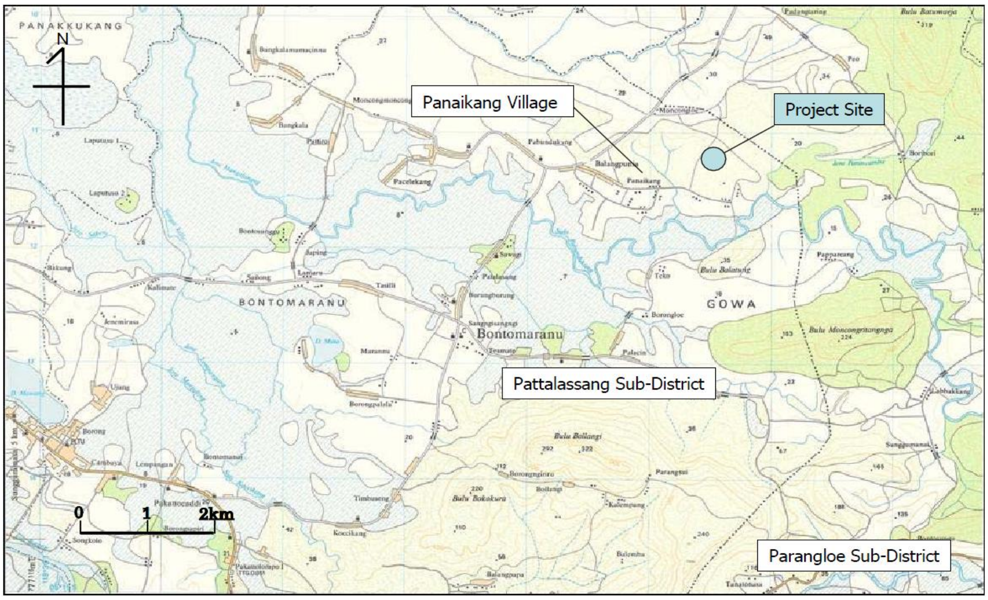
Makassar city produces about 90% of the solid waste discharge amount in the Mamminasata metropolitan area, and, according to forecasts, it is expected that the solid waste discharge amount per day per person will be increased to between 866 and 1.134 grams per person per day in 2027 from the current discharge amount of 477 grams per person per day.

For stable and sustainable development in the Mamminasata Metropolitan Area, the establishment of an RIWMC is critical. Accordingly, the construction of a new landfill site to attain proper disposal for Makassar City, the core city in the region, is an urgent task from the viewpoint of balancing development with environmental sustainability for the whole region.

### 2.2.3 Location of the Mamminasata Project

The site of the RSWDS is in the north-eastern part of Panaikang Village (Desa), Pattalassang Sub-district (Kecamatan), Gowa Regency and is shown in the location map (Figure 1). The area of land is approximately 100 ha and is to include a buffer zone, waste disposal area, cover soil stockpile area, leachate processing area, recycling area and supporting facilities area.

**Figure 1 - Site Location**




### 2.2.4 Battery Limits & Extent of ESC Involvement

The overall Mamminasata RSWDS programme involves a significant number of investigation, design, training and construction activities, with the overall objective of establishing an integrated disposal system for solid waste generated in Mamminasata Metropolitan Area through:

- constructing a RSWDS in Pattalassang, Gowa District; and
- strengthening the operation and management system by establishing Regulatory Body and UPTD/BLU for RSWDS.

The IndII-supported activity, to be completed by ESC, involves preparing the RSWDS design, bid documents and training materials, together with an engineering study of required upgrading works to the last 5.9 km of the access road proposed in the SAPROF. There are, however, other associated design works which are the responsibility of others including DED of a transfer station in Makassar at the current dumpsite and DED of the access road. Further, the IndII/ESC activity does not involve tasks associated with establishing the proposed management body arrangements.

The DED works are limited to the approximately 100 ha site reserved for the RSWDS located in Panaikang Village. The SAPROF proposed physical works covering an area of 42 ha for waste disposal, 20 ha for leachate treatment and site for the supporting facilities, recycling plant, and cover soil storage. The first stage will have a disposal capacity 2.36 million cubic metres or approximately 8 years working life. The work to be done by ESC is for the first stage of the facility.

### 2.2.5 Overall Project Implementation Plan & Funding Sources

DED works are schedule to be completed over a 25 week period in 2010. Following this it is envisaged the RSWDS project will be implemented as follows (with funding source in brackets):

- May 2012 - April 2014: Landfill, sorting and composting plants construction (JICA loan).
- October 2013 - April 2014: Heavy equipment procurement (JICA loan).
- October 2013 - April 2014: Heavy equipment procurement (Ministry of Public Works).
- October 2013 - March 2014: Transfer station and sanitary landfill supporting facilities construction (Ministry of Public Works).
- June 2012 - November 2013 Access road construction/ upgrading (South Sulawesi Province).

Budgetary estimates (JICA personal communication/SAPROF) for the costs of the project were approximately IDR 350 billion (USD 38 million) in total or IDR 320 billion (USD 35 million) for works covered in this DED study.

# CHAPTER 3: TERMS & DEFINITIONS

The following terms are defined for use throughout this Concept Design Report. (Note that some terms are also defined within the body of the text of the Report.)

**Airspace**

The total landfill volume, comprised of the disposed waste and the various cover soils (daily, intermediate and final), but exclusive of the lining and leachate collection system and the capping system.

**Airspace Efficiency**

The volume of waste and the various cover soils placed within a landfill (in cubic metres) divided by the plan area over which the waste is placed (in square metres).

**CCL**

The abbreviated form of “Compacted Clay Liner”.

**Facility**

Hereinafter the Mamminasata Regional Solid Waste Disposal Site (RSWDS) Facility, inclusive of the Support Facilities, Sorting and Composting Facilities, Landfill Waste Disposal Area (also hereinafter referred to as the “Landfill”), Leachate Treatment Plant and Soil Stockpile area will collectively be referred to as the “Facility”.

**LAM**

The abbreviated form of “Land Acquisition Map”.

**Landfill**

The Landfill Waste Disposal Facility may herein also be referred to in the abbreviated form as the “Landfill”.

**Leakage**

The flow of leachate through or from the base of a landfill waste disposal area.

**RSWDS**

The Mamminasata Regional Solid Waste Disposal Site; also herein referred to as the "Site".

**Report**

This, the Concept Design Report, will hereinafter also be referred to as the "Report".

**SAPROF**

The Concept Design presented in this document is generally based on review of and referenced to the "FINAL REPORT – Special Assistance for Project Formation for Solid Waste Management Improvement Project in Mamminasata Metropolitan Areas, Republic of Indonesia" prepared by the Japan Bank for International Cooperation (JBIC) and dated February 2008. Hereinafter this document is referred to in the abbreviated form as the "SAPROF".

**Site**

The area within the Site Boundary, inclusive of the Facility, Buffer Zones, and all other areas will collectively be referred to as the "Site".

# CHAPTER 4: GENERAL SITE DETAILS

## 4.1 SITE BOUNDARIES & BUFFER ZONE

The Site Boundary is that as established by the survey conducted in August 2011 by CV. Adi Permata Konsultan and as shown in Drawing No. GEN/002 in Appendix A. .

A 25m wide Buffer Zone has been adopted and will generally be maintained around the entire Site Boundary. Within the Buffer Zone, zero or limited operational activities will be performed but no waste disposal or leachate treatment activities are to be performed (refer to Drawing No. GEN/002 in Appendix A). Some temporary or support features such as sediment basins, soil stockpiles, and maintenance access tracks may be located within the Buffer Zone in select, limited locations.

## 4.2 TOPOGRAPHY

The existing topography of the Site is shown in Drawing No. GEN/002 in Appendix A as established by the survey conducted in August 2011.

## 4.3 SUBSURFACE CONDITIONS

The subsurface conditions of the Site have been investigated, to various extents, in previous studies (Pre-Feasibility and SAPROF). An additional investigation in relation to the proposed landfill facility was commissioned by ESC as part of this contract and was conducted in May 2010 and involved the installation of two (2) boreholes drilled to a depth of 50m and the excavation of nine (9) trial pits excavated to a depth of up to 4m. The general stratigraphy of the Site is summarised below in Table 4.3. The test results obtained from all of the subsurface investigation have been used to establish the geotechnical characteristics that form the basis of the designed presented herein.

It appears from the subsurface investigation performed that the groundwater level (in May 2010) was generally between 3 m and 5 m below the existing ground surface. However, as the landfill excavation requires removal of a localised hilltop, as does the creation of a platform for the Support Facilities and Sorting/Composting facilities etc, it is anticipated that groundwater levels will be reduced below the respective formation levels.

Table 4.3: Summary of the Generalised Site Stratigraphy

| Thickness (m) | Depth [Note 1] (m) | Description |
|---|---|---|
| 0.15 to 0.30 | 0 to 0.3 | Black-brown organic TOP SOIL, with roots |
| 0 to 2.70 | 0.25 to 3.0 | Firm red to light-brown clayey SILT |
| 2.0 to 3.75 | 0.25 to 4.0 | Very weak to weak, fractured, weathered reddish/ light brown/grey TUFF<br>Or<br>Very weak, brown-grey, fractured, weathered BRECCIA |
| 46.0 to 49.0 | 1.0 to 50.0 | Alternating layers of CLAY STONE, TUFF and SANDSTONE |

Notes:

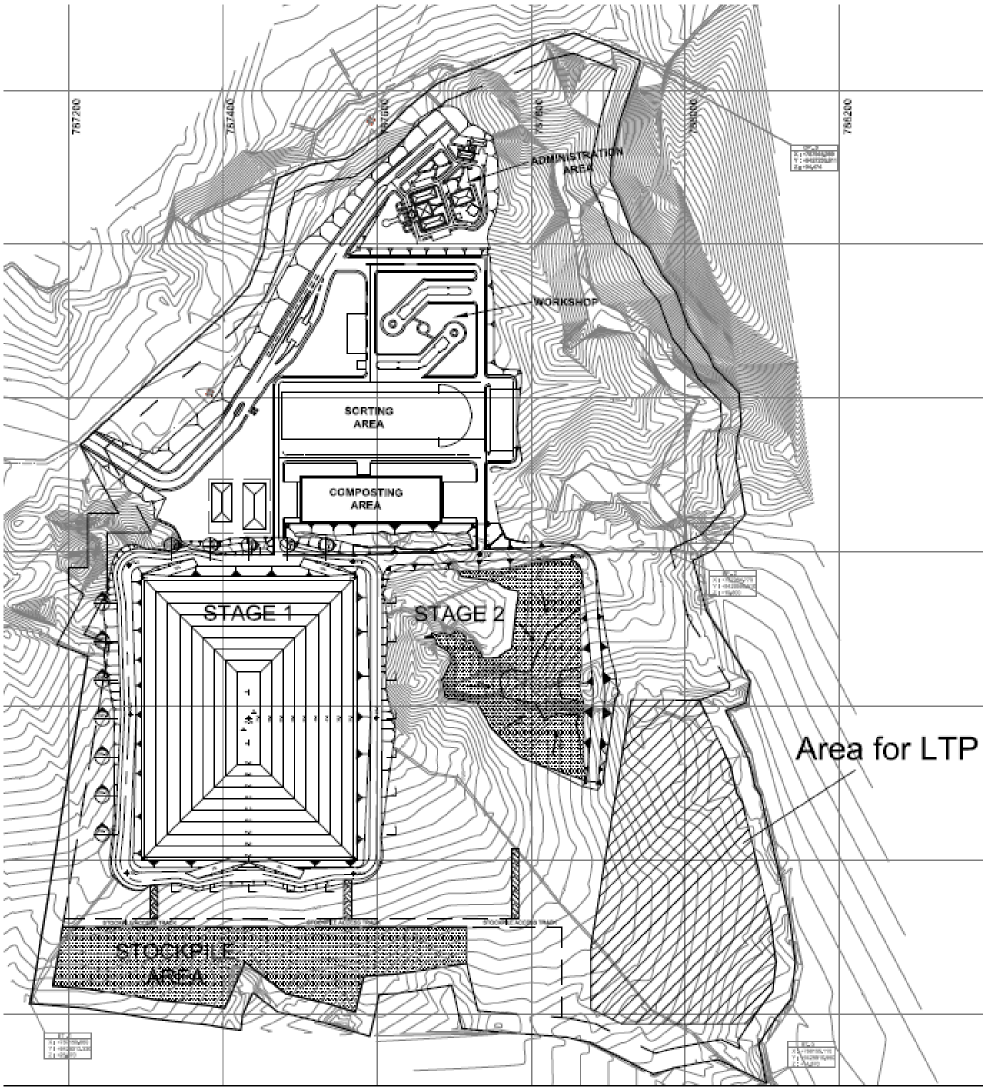
(1) Depth is relative to the existing ground surface.

(2) Below approximately 4.0m, the stratigraphy is determined from only borehole logs.

# CHAPTER 5: SITE LAYOUT & FORMATION

## 5.1 SITE LAYOUT

The figure below indicates the layout for the key operational and support facilities. The layout is based on considerations relating to the smooth operation of the Site whilst minimising amenity impacts to Site staff, visitors and local residents. Detailed layout plans are presented in Appendix B.



## 5.2 SITE FORMATIONS & EARTHWORKS

To a large extent the excavation and filling of soil associated with formation of the Site; which is generally and collectively referred to as “earthworks”; are governed by the volume of material necessary to construct and operate the Landfill. The volumes necessary to form the Stage 1 and ultimately the Stage 2 Landfill area and the facilities area are summarised in Table 5.2.

As it is extremely desirable to obtain the entire volume of soil necessary for construction and operation of the Landfill from on-site sources, it will be necessary to make a number of excavations and then stockpile the material prior to or in association with the initial construction activities. These excavations include formation of a relatively large platform for development of the Site Entrance, the Support Facilities, the Sorting and Recycling Facilities, surface water channels, detention pond the Leachate Treatment Plant and the Stage 1 Landfill area.

The Landfill excavation and fill slopes are generally designed at an inclination of 3 horizontal to 1 vertical (3h:1v), this to control the potential for soil erosion and to insure stability of the slopes, even under seismic conditions. Stability analyses have been performed for the various configurations, with the minimum factor of safety greater than 1.5 (FS>1.5) under static conditions, and greater than 1.2 (FS>1.2) in consideration of a seismic coefficient of 0.1g.

Because the entire Stage 1 Landfill, inclusive of the lining and leachate collection layers, will be constructed as part of the initial works, it will be necessary to excavate a sufficient volume of soil to construct the Landfill, inclusive of the perimeter and inter-cell bunds and stockpile any excess for use in landfill operations or other future use such as the construction of Stage 2 of the landfill or its’ operation. The stockpiled volume after the construction of Stage 1 of the landfill has been completed and prior to operations starting is estimated to be 578,400 $m^3$ (refer to Table 5.2A).

A initial stockpile area is preferred in the area along and generally parallel to the southern boundary of the Site, so as to form a physical buffer, visual screen and noise barrier between the Site and the nearby local residents to the south (Drawing No. GEN/003 in Appendix B) for the plan view location and cross-section, respectively). It is estimated that this stockpile will be a minimum of 15m – 20m in height and store the entire volume of material necessary for the future operation and capping of Stage 1 of the landfill. It is noted, however, that some segregation of materials will be necessary within the stockpile so as to facilitate matching the most appropriate soil to the required application. Examples of such include: topsoil is to be segregated from the other soils and the more clayey soils should be employed for the formation of the perimeter and inter-cell bunds and in construction of the final capping layer. Although it is anticipated that the tuff material will, to a large extent, weather and degrade to a soil-like matrix upon excavation and exposure to the elements, it should generally be reserved for use as daily and intermediate cover within the Landfill. The Stage 2 landfill area may also be available for stockpiling, however this is not preferred as any soil placed there would need to be moved prior to work commencing on Stage 2 of the landfill development.

The stockpile surfaces are to be vegetated to reduce soil erosion. The southern face of the stockpile located along the southern boundary of the Site will be landscaped, inclusive of rapidly growing tree species to further enhance the visual appearance.

The excavation volume to form the platform necessary for the Site Entrance, Administration Area, Sorting Plant and Composting Plant has been determined to bring the overall Site earthworks to a balance. Further in this regard, it is estimated that the level to which this platform is to be formed is approximately elevation +28.0 m.

**Table 5.2A: Summary of Soil Volumes to Construct and Operate Stage 1 Landfill**

| No. | Location | Fill Volume ($m^3$) | Excavation Volume ($m^3$) |
|---|---|---|---|
| 1. | Site Entrance, Support Facilities and Sorting/Composting Areas | 80,000 [Note 2] | 440,000 [Note 7] |
| 2. | Landfill Area | --- | --- |
| 2.1 | Formation (Note 3) | 299,900 | 807,400 |
| 2.2 | Inter-cell Bunds | 86,000 | --- |
| 2.3 | Daily Cover [Note 4] | 213,000 | --- |
| 2.4 | Intermediate Cover (Note 5) | 340,000 | --- |
| 2.5 | Final Capping System | 68,000 | --- |
| 3. | Detention Pond | 4,100 | 8,200 |
| 4. | Surface Water Channels | --- | 95,500 |
| 5. | Leachate Treatment Plant Area | 17,200 | 50,800 |
| 6. | Roads | 24,300 | --- |
| | **Total** | **823,500** | **1,401,900** |
| | | | |
| | Required Stockpile Volume | --- | 578,400 [Note 6] |

Notes:

(1) All volumes are "bank". It is assumed that the bulking factor and reduction with compaction are equal.

(2) Landfill Formation volume includes the Perimeter Bund.

(3) Daily Cover is taken as 10% of the Landfill airspace which is approximately 2,130,000 m3.

(4) Intermediate Cover is taken as a 300mm thick soil layer over the waste at the crest of the perimeter bunds and after reaching the final level.

(5) Required Stockpile Volume is the Total Fill Volume minus the volume of soil used to construct t

(6) Site Entrance, Support Facilities and Sorting/Composting platform estimated to be at elevation +28.0 m

Table 5.2B extends the estimated earthworks for the Stage 2 Landfill area of the Site, and illustrates that the overall earthworks closely balance.

**Table 5.2B: Summary of Soil Volumes to Construct and Operate Stage 2 Landfill**

| No. | Location | Fill Volume ($m^3$) | Excavation Volume ($m^3$) |
|---|---|---|---|
| 1. | Landfill Area | | |
| 1.1 | Formation | 148,400 | 82,000 |
| 1.2 | Inter-cell Bunds | 86,000 | |
| 1.3 | Daily Cover | 310,000 | |
| 1.4 | Intermediate Cover | 29,200 | |
| 1.5 | Final Capping System | 68,000 | |
| | **Total** | **641,600** | **82,000** |
| | | | |
| | Required Stockpile Volume | | (559,600) |

# CHAPTER 6: LANDFILL SYSTEMS

## 6.1 GENERAL LANDFILL DESIGN AND CONSTRUCTION

The Stage 1 Landfill will be approximately 360 m by 270 m in plan dimensions; sub-divided into four (4) approximately equal Cells, each immediately adjacent to one another and sharing a common side in the form of an inter-cell earth bund. Each Landfill Cell will be nominally 180 m by 135 m in plan dimensions, and excavated to a depth of not more than 3 m to 4 m below the existing ground surface so as to remain above the anticipated groundwater level and to provide an adequate elevation head to maintain the gravity flow of leachate to the adjacent treatment facilities. The hilltop within the landfill area is to be excavated through and entirely removed, with the maximum excavation depth in this area at approximately 9 m. Each Cell will be oriented lengthwise from north to south, with the lower elevations of each formed as a sump along both the northern and southern sides (refer to Drawing No. LFS/001 in Appendix C).

The design of the common components of each Landfill Cell (lining system, leachate collection system and final closure system are described in the following sections.

## 6.2 LANDFILL LINING SYSTEM

Various liners have been considered and assessed for construction along the base and side slopes of the Landfill, including: a compacted clay liner using the on-site soils; a geomembrane liner; and a composite liner of soil overlain by a geomembrane. The assessment performed has considered that the groundwater level is relatively shallow beneath the Landfill and is conservatively assumed to flow toward the residents located from 600 m to 800 m south of the Site at a gradient approximately equal to the slope of the ground surface; taken as 1%. In consideration that the groundwater may be used as drinking water, the concentration of ammonia is not to exceed 1.5 mg/L, while the Pre-Feasibility Study reports that the background concentration of ammonia in the groundwater is 0.5 mg/L.

As concluded previously in the SAPROF and Pre-Feasibility Study, the on-site soils are not sufficiently impermeable to be used alone as a compacted clay liner. As a result, geomembrane liners and composite liners of soil overlain by a geomembrane were considered.

In consideration of the site-specific conditions, constraints and practical aspects associated with installation of a geomembrane liner, it was concluded that a geomembrane liner alone is not the preferred containment layer system for the Landfill. As a result, a detailed analysis of a composite liner comprised of a soil layer overlain by an HDPE geomembrane was performed assuming the hydraulic conductivity of the geomembrane to be 2.0 x 10-13 cm/sec and a hydraulic head of 300 mm within the leachate collection layer above. The analysis conservatively assumes that as geomembrane installation is still a developing technology in Indonesia, it will result in one leak (through holes of various sizes) per 800 m2 of installed area. It is also conservatively assumed that the geomembrane may not be in good contact with the underlying soil. The concluded result is that the soil component of a geocomposite liner must have a hydraulic conductivity of not more than 3.4 x 10-10 m/sec. Limited to a minimum hydraulic conductivity of 6 x 10-8 m/sec, the on-site soils will not provide this level of impermeability; and it is concluded that a geosynthetic clay liner (GCL); also sometimes referred to as a “bentonite mat”; with a hydraulic conductivity of 3 x 10-11 m/sec, will be required as the soil layer component of a composite liner, with the overlying component as a 1.5 mm thick, textured-surface (both sides) HDPE geomembrane.

## 6.3 LEACHATE COLLECTION SYSTEM

The leachate collection system is comprised of a number of components designed with various features, including the following:

- A coarse granular blanket layer across the entire landfill area (base and side slopes) draining toward a sump;
- A geotextile cushion between the granular blanket layer and the underlying geomembrane liner;
- Dual “rib” and “spine” networks of trenches in each cell with the trenches containing a perforated collection pipe enveloped by granular material, which ultimately discharges into the sump;
- Two discharge pipes from the sump to an external pipe conveyance system;
- Collection and discharge pipes designed to flow at not more than 50% of their capacity under the 20-year storm event to promote the circulation of air through the waste mass;
- Perforated vertical riser pipes surrounded by granular material to further promote the circulation of air through the waste mass; and
- A pipeline to convey the collected leachate to the Buffer Storage Lagoon located at the Leachate Treatment Plant.

Each of these components and features are described below.

The leachate collection system is comprised of a 500 mm thick granular blanket layer graded toward a sump located at both the northern and southern end of each landfill Cell. This granular blanket layer is composed of non-carbonate material with a nominal particle gradation ranging from 20 mm to 45 mm. This gradation and the loading from the waste to be placed above will require a relatively high-strength geotextile cushion to be placed beneath the blanket layer to protect the underlying geomembrane liner.

The grade of the landfill base is to be nominally 1.0% to the north and south toward the sumps to promote positive flow of leachate.

The coarse granular blanket layer is supplemented with a network of “rib” and “spine” collection trenches containing perforated pipes enveloped by coarse granular material. Each trench is filled with the same material as in the overlying coarse granular blanket layer (gradation ranging from 20 mm to 45 mm). The same relatively high strength geotextile is placed across the base of each trench to protect the underlying geomembrane.

The lateral “rib” collection trenches are spaced at 30 m centre-to-centre horizontally; with each graded at approximately 0.6% toward the main “spine” and containing a minimum 250 mm diameter perforated pipe.

As a form of redundancy, there are two (2) main “spine” collection trenches oriented lengthwise along the base of each Cell. These collection trenches are spaced approximately 50 m apart and contain a 450 mm diameter perforated pipe graded at 1.0 % toward the sump.

It is specifically noted that these pipes are designed to flow at approximately 50% capacity under a 20-year rainfall event; this to further promote the circulation of air through the leachate collection system.

Leachate will flow from each sump under gravity through two (2), 450 mm diameter HDPE pipes to manholes along a leachate conveyance pipeline located beyond the landfill perimeter access road. The leachate from each Cell will discharge into the conveyance pipeline and drain, by gravity, to the Buffer Storage Lagoon of the Leachate Treatment Plant.

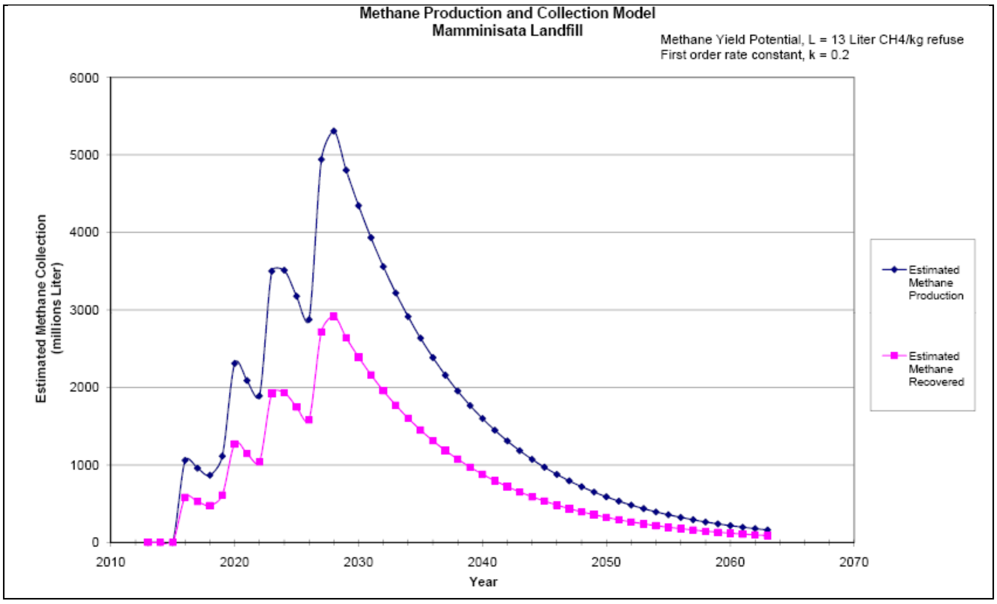
The relatively large-sized granular material and the collection pipes sized to flow at 50% of their capacity are to allow air to circulate through the leachate collection system and the surrounding waste mass. Another feature to further promote such air flow is a series of vertical 150mm diameter HDPE riser pipes surrounded by gabions and spaced nominally 70m on-centre across the Landfill base; each hydraulically connected to the leachate collection layer beneath (refer to Drawing No. LFS/014 in Appendix C). The potential adverse effect of the drag-down forces acting around these vertical risers will be addressed through a concrete foundation and/or the necessary protective cushioning.

### 6.4 LANDFILL GAS GENERATION & EXTRACTION

Although the volume of methane produced within the landfill is not expected to be significant, as a contingency, if methane is produced, it can be extracted from the waste through a series of vertical wells installed from the surface of the landfill final cover.

The designed location of these wells, as shown on Drawing No. LGM/001 and Drawing No. LGM/001 for Landfill Stage 1 and Stage 2, respectively1800 in Appendix C, is based on the estimated potential rate of landfill gas production and a "radius of influence" (ROI) of approximately 38 m, and a spacing of approximately 67 m. Each well is designed to a depth of approximately 20 m, with 150 mm diameter perforated high density polyethylene (HDPE) vertical pipes, connected by a network of 150 mm diameter HDPE lateral pipes and a ring of 250 mm diameter HDPE header pipes routed to a flare and/or utilisation facility to the northwest of the Landfill area. (Detailed design of the landfill gas flare and/or utilisation system is not included in the scope of the engineering services provided under this Contract).

Each well is accessible from the Landfill final cover surface by a well-head chamber (refer to Drawing No. LGM/003 in Appendix C); while the network of extraction lateral and header pipes are buried within the final cover profile.




In order to maximise the volume of methane extracted from the landfill, it is proposed that the wells and the associated piping be installed sequentially with waste disposal activities and installation of the final cover.

## 6.5 FINAL COVER CONFIGURATION & PROFILE

As shown on Drawing No. LFC/001 and Drawing No. LFC/002 in Appendix C the waste is to be placed above the surrounding ground surface at a slope of 3h:1v; then at nominally a 5% gradient up to a maximum of up to 71 m for Landfill Stage 1 and ultimately to 81 m.

A minimum 300 mm thick layer of soil obtained from on-site sources is to be placed over the waste immediately after the final level is achieved. This layer is a temporary measure to reduce odours and prevent access to rodents; and serves to provide a relatively firm foundation over which the overlying compacted soil cover can be placed.

The final cover profile design comprises of four (4) layers across both the side slopes (at 3h:1v) and the upper platform (at 5% gradient), as shown on Drawing No. LFC/003 each serving a specific function. A description of each layer, is provided as follows:

### 6.5.1 Compacted Soil Cover

A 500 mm thick compacted soil cover layer will serve as a barrier to reduce the infiltration of rainfall into the underlying waste mass. This compacted soil cover is constructed of "controlled" fill (controlled for moisture content and compacted density) to minimum infiltration into the underlying waste.

### 6.5.2 Drainage Layer

A drainage layer, either 300 mm thick granular or a geocomposite drainage layer, will be placed over the compacted soil cover layer to intercept infiltration and divert it from the final cover; thus reducing the hydraulic head and infiltration into the underlying waste mass.

### 6.5.3 Soil Cover

A 500 mm thick layer of site-derived soil will be placed and compacted as protection for the underlying drainage layer and compacted soil layer.

### 6.5.4 Topsoil

A minimum 150 mm thick layer of topsoil will be placed over the entire surface of the final cover to support vegetative growth. The topsoil is to be the same material as initially stripped from the area over which the Landfill and the other facilities of the Site are constructed.

### 6.5.5 Vegetation

Vegetation over the surface of the final cover will be limited to grasses so as to control soil erosion whilst restricting the depth of root penetration.

## 6.6 SEQUENTIAL LANDFILL OPERATIONS

All four (4) Cells of the Stage 1 Landfill, inclusive of the site formations, lining system and leachate collection systems, will be constructed during the initial works, but operated sequentially. The general sequence of operations is recommended to be from west to east, with initially the first four Cells filled to a level only slightly higher than the perimeter bund (to promote surface water runoff, but to prevent such from discharging into an adjacent non-active Cell). The subsequent filling sequence will be above the crest of the perimeter bund to the final level.

After each Cell has achieved its final level, the final capping system can be placed and the landfill gas wells can be sequentially installed and made operational.

## 6.7 SURFACE WATER MANAGEMENT

### 6.7.1 Construction & Operation Phases

The Landfill surface water management system will generally comprise of a series of grass lined channels constructed around the Site Entrance, Support Facilities, Sorting/Composting Facilities and the Landfill area. In some instances, particularly immediately adjacent to the larger roofed structures (i.e. the Sorting Plant and the Composting Plant) pipes and concrete channels are employed. These channels are designed to cope with at least a 50-year rainfall event, estimated to be of an intensity of approximately 300 mm/hour.

Around the Landfill at the external toe of the perimeter bund, a small grass-lined channel will collect run-off from the Landfill access road and the lower portion of the adjacent bund slopes. This water will be routed through pipes to perimeter channels located beyond the perimeter access road; of which two (2) are main channels oriented east-west along the northern and southern sides of the Landfill, respectively. These channels will generally be grass-lined, however with the increased flow volume and the increased gradient; portions are designed of concrete, and some require energy dissipation.

During construction and operation of the Landfill, Cells that do not yet contain waste will collect rainfall. This water will be discharged from the Cell by gravity through the leachate collection system piping within the Landfill (before commissioning to transmit leachate) to the main channels along to the north and south sides of the Landfill area to be eventually discharged to the adjacent stream courses to the north and in the southeast corner of the Site.

### 6.7.2 Final Cover Phase

The surface water management system across the final closure is also designed to cope with at least a 50-year rainfall event (estimated to be 300 mm/hour) and will be comprised of a “rib” and “spine” drainage system (sometimes referred to as a “chevron” or “herringbone” pattern). The “ribs” of this system will be a network of “porous” pipes partially embedded in the landfill cover surface with an exposed mesh to intercept rainfall runoff and a solid-wall invert to transmit the collected water laterally. These pipes will be spaced at nominally 35 m centre-to-centre along the slope vertically, and discharge into downslope ‘spine’ drainage features.

The downslope “spines” are to be plastic-reinforced concrete trapezoidal channels spaced nominally 75 m centre-to-centre around the Landfill perimeter. The plastic reinforcement will be a geocell product, first serving temporarily as formwork, and then permanently as plastic-reinforcement. The trapezoidal channel is to be nominally 1m or less across the bottom, with 2 horizontal to 1 vertical (2h:1v) side slopes and a depth of not less than 0.5 m. Both the bottom and sides of the channel will contain baffle blocks to dissipate the energy of the flowing water down the 3h:1v side slopes and discharging into the perimeter channels.

This drainage system is designed to be constructed progressively as the landfill is sequentially developed. This will minimise soil erosion and thus establish the final cover of the landfill as quickly as possible after waste placement is completed in each of the respective cells.

### 6.7.3 Sediment Control

Soil erosion and sediment will be controlled through the use of three (3) methods: temporary silt fences (typically at the toe of slopes and along other lines of constant elevation); through reuse of topsoil to establish a vegetative cover over the disturbed areas as soon as practically possible and a surface water detention pond located in the valley to the northeast of the Landfill which will serve as a sediment basin for the northeast portion of the Site. The details of each of these methods are described as follows.

Silt fences will be used in strategic locations on a short-term and as-needed basis from the time fill soil is placed until vegetation can be established on permanent slopes. The silt fences will require cleaning from time to time to remove captured sediments.

Topsoil is to be stripped at the initial stage of construction and reused to cover exposed excavation and fill soil slopes across the Site. Vegetation will be established through seeding and select planting to establish growth and control erosion as soon as practically possible.

A surface water basin, constructed in the valley north of the Landfill will detain water for potential on-site use. This basin will also serve to capture soil (sediment) that has eroded from excavation and fill surfaces and not trapped by silt fences.

## 6.8 ENVIRONMENTAL MONITORING

A series of six (6) wells; with three (3) up-gradient of the overall Facility and/or the Landfill and three (3) down-gradient of the Site will be installed to monitor the groundwater quality on a regular basis throughout the life of the Facility. Proposed locations for such wells are shown on Drawing No. ENV/001 for the proposed location of these monitoring wells in Appendix C.

Landfill gas monitoring probes will be placed on nominally 150 m centres and in select additional locations around the perimeter of the Landfill area to monitor for the potential for lateral migration. Refer to Drawing No. ENV/001 for the proposed locations of these monitoring probes.

One (1) leachate effluent quality monitoring station is to be established immediately prior to the confluence of the southern surface water channel and the receiving stream. Refer to Drawing No. ENV/001 for the plan view location of this monitoring point.

## 6.9 AMDAL REPORT

It is a requirement for the consultant to ensure that the requirements laid down in the approved AMDAL study are incorporated into the DED wherever appropriate. ESC has prepared a checklist of features from the AMDAL documents that require attention in the DED including indication of where these requirements have been incorporated into the design. The checklist is presented in Appendix D.

# CHAPTER 7: LEACHATE TREATMENT SYSTEM

## 7.1 LEACHATE GENERATION

A detailed analysis of the potential leachate generation rate has been performed, with consideration given to the various contributing factors, including: rainfall, evaporation, infiltration, surface water runoff, "free" liquids delivered with the waste, waste consolidation, the on-site sorting and composting operations, the waste intake rate, typical landfill operational areas (open waste, daily cover, intermediate cover and final cover) anticipated landfill operation methods; and the preferred operational sequence employed to place waste in the Landfill. Of these, rainfall and the operational sequence are the primary contributing factors; with rainfall a variable subject to nature.

A detailed assessment has been performed of the potential variation of annual rainfall with various sources of data consulted and statistical analyses performed; the conclusion being a conservatively assumed a total rainfall depth of 2,900 mm for a 1 in 20-year return period for a 2-month duration.

The rainfall value was used in a proprietary model that estimates the leachate generation rate resulting from infiltration through the various landfill operational areas (open waste, daily cover, intermediate cover and final cover), and the relative location of each within the Landfill cells (below the perimeter bund, at or near the top of the perimeter bund, above the perimeter bund, and near the final level). The model also considers the sequential placement of waste within the Landfill; both horizontally from cell-to-cell; and vertically, first below the perimeter bund and then above. It has been assumed that the waste placement sequence will proceed in such a manner that all four cells of Stage 1 will be filled to the level of the perimeter bund before waste is placed above. It is further considered that the Stage 2 Landfill will be constructed and operated in the same manner. The result is a conservatively estimated maximum leachate generation rate of 3,190 m3/day for the wettest 2-month duration of the "rainy season"; to be handled by a combination of treatment capacity (1,800 m3/day) and temporary storage (approximately 83,000 m3).

Modelling of the leachate generated as a function of time results in an estimated maximum flow rate of approximately 1,250 m3/day. In consideration of the seasonal variation and relatively intense rainfall that occurs through particularly the period from December through February each year, the estimated maximum leachate generation rate increases by a factor of 4 times with a 1 in 20-year monthly rainfall of about 1,450 mm.

## 7.2 LEACHATE QUALITY

Table 7.2A presents the different data sources of leachate pollutant concentrations considered in the design of the leachate treatment plant (LTP). Table 7.2B presents the nominated influent concentrations used in the design of the LTP. Effluent discharge standards used for the design are also shown and taken from Indonesia's *General Technical Operational Guidance of Landfill Management* (Prof. Enri Damanhuri, FTSB ITB).

For COD and $BOD_5$, the values presented by Robinson et. al. derived from measured values for similar landfills in Asia (refer to Table 7.2A) are the highest and therefore the safest to use for designing a leachate treatment plant. However, considering dilution due to heavy rainfall in the area, it was determined that 10,000 mg/l below the conservative values presented by Robinson et. al. would still allow for a conservative design of the proposed LTP. These 'reduced' values were used for determining the dimensions of the system at the early acidogenic phase of landfill operations when COD and $BOD_5$ values of the leachate are expected to be high (Appendix E). The COD and $BOD_5$ values from the Tamangapa landfill (sampled as part of this study) indicate methanogenic (later stage) leachate and were disregarded. The values presented in the SAPROF and AMDAL are significantly lower than the values presented by Robinson et. al.. Use of these values may result in an under-designed leachate treatment plant. For ammonia, the value used for design of 1,500 mg/l was the extrapolated value for the first 18 months when COD and $BOD_5$ are at their peak. For ammonia, the results from leachate samples taken from the Tamangapa Landfill in Makassar were used as this landfill had been operating for quite a while and the COD and $BOD_5$ results are low indicating methanogenic leachate (Appendix E). The Tamangapa Landfill results for ammonia were used as they are consistent with the nominated values for COD and $BOD_5$ which are approximately 20% lower than Robinson's extreme values.

Figure 7.3B and Figure 7.3C further describes the design basis.

**Table 7.2A: Data-Sources of Leachate Pollutant Concentrations**

| Parameter | SAPROF | AMDAL | Robinson et. al (Extreme Values) | Tamangapa Landfill – Makassar Sampling Results | |
|---|---|---|---|---|---|
| | | | | Sample 1 | Sample 2 |
| COD mg/l | No data | 8,000-10,000 | 500-50,000 | 490 | 583 |
| $BOD_5$ mg/l | 6,900 | 4,000-5,000 | 2,000-30,000 | 383 | 334 |
| Ammoniacal-N mg/l | No data | No data | 750-3,000 | 2,360 | 2,470 |
| pH | No Data | No Data | 7.0-8.5 | 8.49 | 8.48 |
| TSS mg/l | No Data | No Data | No Data | 10 | 7 |

**Table 7.2B: Design Considerations for Leachate Influent Quality**

| Parameter | Acetogenic Stage | Methanogenic Stage | Indonesia Effluent Standard of Leachate (Note 1) |
|---|---|---|---|
| Flow Rate m3/day | **900** | **1,800** | NA |
| COD mg/l | **40,000** | **1,000** | 300 |
| $BOD_5$ mg/l | **20,000** | **500** | 150 |
| Ammonia-N mg/l | **1,500** | **2,570** | 5 |
| pH | **8.5** | **8.5** | 6-9 |
| TSS mg/l | **3,00** | **3,00** | 400 |

Notes

(1) From "GENERAL TECHNICAL OPERATIONAL GUIDANCE of LANDFILL MANAGEMENT"

## 7.3 LEACHATE TREATMENT PLANT DESIGN BASIS

Figure 7.3A presents the projected leachate generation per year. Using the design considerations and projected leachate generation per year, main pollutant concentrations (COD, $BOD_5$ and ammonia) were extrapolated for a 30-year period taking into consideration acetogenic leachate within the first two years of the landfill and its conversion to methanogenic leachate for the rest of the life of the landfill. The main pollutant concentrations extrapolated are presented in Figure 7.3B. The COD and $BOD_5$ results from the Tamangapa Landfill are below even the minimum values presented by Robinson et al. which are 2,000 mg/l and 500 mg/l respectively. Designing at these levels may again result in an under-sized leachate treatment plant, therefore twenty percent (20%) to 25% below the minimum was used.

**Figure 7.3A: Leachate Generation Rate**

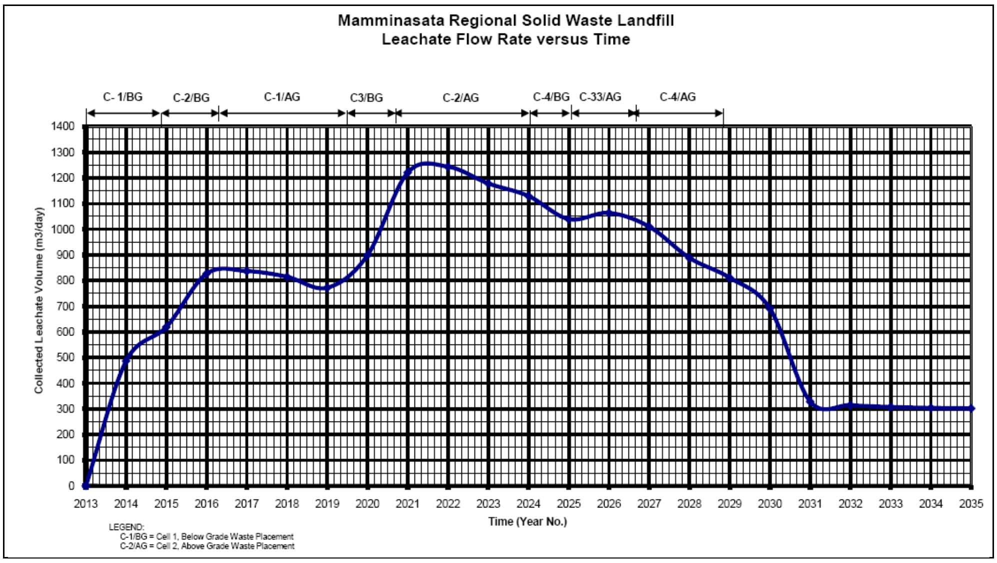
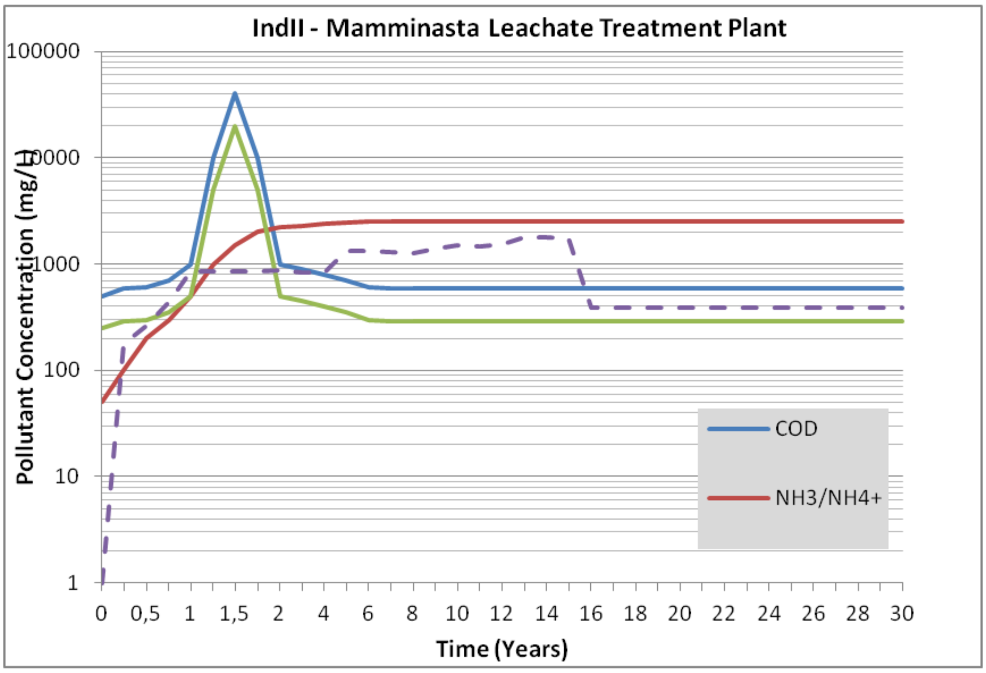
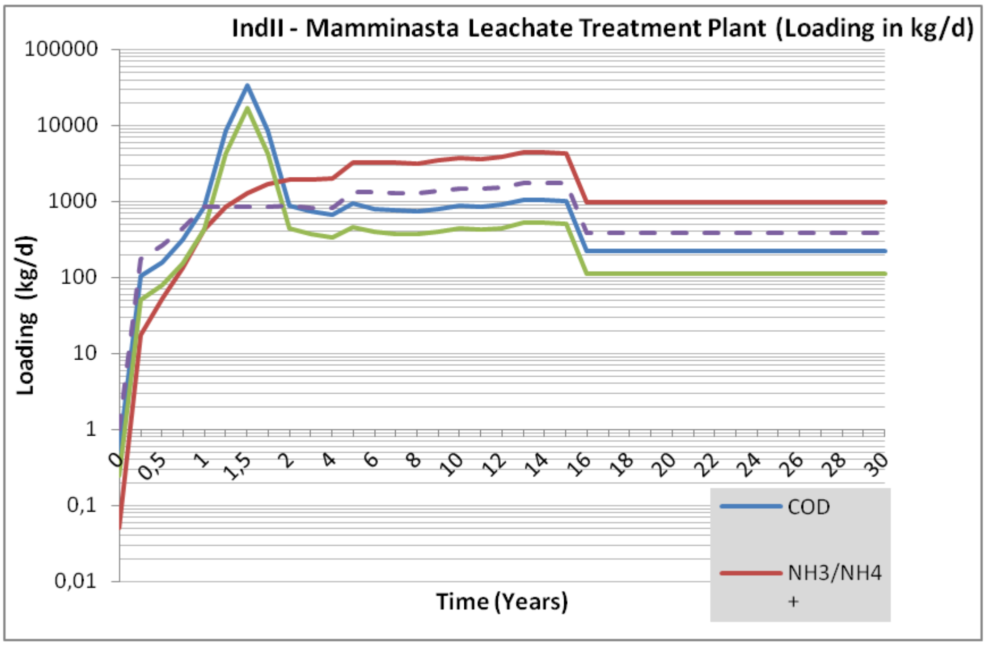



**Figure 7.3B: Leachate Pollutant Concentration Curves**




The corresponding mass loadings were then calculated from the pollutant concentrations. Figure 7.3C presents the mass loadings. From these mass loadings, two scenarios indicated by the red circles were compared in dimensioning the tanks of the LTP. The more conservative values were used.

**Figure 7.3C: Leachate Mass Loadings**




## 7.4 AMMONIA TOXICITY

Free ammonia (NH3-N) and ionised-ammonia (NH4+-N) represent two forms of reduced inorganic nitrogen which exist in equilibrium depending upon the pH and temperature of the waters in which they are found. Of the two, the free ammonia NH3-N is considerably more toxic to both higher and lower level organisms than the ionised form; we have therefore paid considerable attention to the relative concentration of this particular contaminant.

Ammonia varies in toxicity at different pH and temperature of the water. Table 7.4 shows the varying levels of ammonia against the comparatively harmless ammonium as a function of the temperature and pH. The calculation method was developed by James E. Alleman from Purdue University.

Using this table and the data from the leachate analysis previously conducted where Total Ammonia is 2,470 mg/L, free ammonia would be around 500 mg/L. There are no straight forward levels for ammonia toxicity mentioned for plants in literature but there have been reports of free ammonia inhibition on algal growth at 34mg/L. The indications suggest that the leachate, because of its high ammonia content, will only disable non-mechanical passive through-flow type process units such as facultative aerobic ponds where algae are an important source of oxygen supply.

**Table 7.4: Free Ammonia Versus pH and Temperature**

| % Percent of free ammonia from 'total ammonia' | | | | | | |
|---|---|---|---|---|---|---|
| **Temp C/F** | **pH 6.5** | **pH 7.0** | **pH 7.5** | **pH 7.7** | **pH 8.0** | **pH 8.5** |
| **20C (68F)** | 0.125 | 0.395 | 1.239 | 1.950 | 3.810 | 11.150 |
| **25C (77F)** | 0.179 | 0.565 | 1.766 | 2.770 | 5.380 | 15.242 |
| **28C (82F)** | 0.221 | 0.696 | 2.170 | 3.396 | 6.550 | 18.156 |
| **30C (86F)** | 0.253 | 0.798 | 2.482 | 3.780 | 7.450 | 20.292 |

## 7.5 LTP LAYOUT

The LTP comprises two identical main process lines (Drawing No. MRSWDF-2008 in Appendix F). They include an anaerobic treatment step for treatment of acidogenic leachate, an anoxic/aerobic biological treatment step for treatment of low concentrations of $COD/BOD_{5,}$ and ammonia removal through a biological nitrification-denitrification process. A reed bed is included for final polishing of the biologically treated leachate by further removal of suspended solids before discharge to the river. Non-mechanical systems were considered but not used as main treatment units for the following reasons:

- Ammonia toxicity from the high ammonia concentration; and
- Design feasibility – considering an ammonia concentration of 2,570 mg/l.

The following are the main considerations in laying out the LTP process units:

- Minimisation of pipe lengths;
- Site elevations; and
- Access.

The LTP process units are arranged in such as way that their inlets and outlets are adjacent. The discharge from the reed bed is closest to the river where the effluent shall be discharged. Site elevations show a gradient from the north of the LTP down towards the south east. The flow direction from the arrangement of the process units follows the natural gradient of the Site making construction less difficult. Proper operation of the LTP would require at least one operator at any given time as complex automation equipment is not included in the design. The operator will be responsible for maintaining the operation parameters such as sludge volume and the solids concentration of the anoxic/aerobic treatment system. Also, flowrate from the pump at the buffer/storage may have to be adjusted from time to time. The LTP units that require more attention such as the drying beds and clarifiers are proximal to the control room where the operators usually spend most of the time when performing scheduled LTP maintenance activities. The location of the sludge holding tank anticipates an access road for easier handling and transport of sludge.

## 7.6 DESCRIPTION OF MECHANICAL EQUIPMENT IN LTP PROCESS UNITS

Table 7.6 summarises the mechanical equipment in the LTP. The purpose of each is described in the following sub-sections.

**Table 7.6: Mechanical equipment in the LTP.**

| Process Units | Mechanical Equipment | Quantity | No. of Operational Units | No. of Stand-by Units |
|---|---|---|---|---|
| Buffer/Storage Pond | Feed Pumps | 4 | 3 | 1 |
| Anaerobic Tank | Recirculation Pumps | 3 | 2 | 1 |
| Aeration Tanks | Surface Aerators | 6 | 6 | 0 |
| Aeration Tanks | Recirculation Pumps | 4 | 2 | 2 |
| Anoxic Tanks | Mixers | 6 | 6 | 0 |
| Clarifiers | Sludge Pumps | 4 | 2 | 2 |
| Recirculation Pit | Recycle Pumps | 2 | 1 | 1 |

### 7.6.1 Buffer/Storage Pond

The main purpose of the buffer storage tank is to protect the downstream process units from surge flows. This is achieved by its large volume (buffering capacity) and feed pumps that controls outflow to the downstream processes. The volume of the pond is around 68,000 $m^3$ and is HDPE lined to prevent seepage. It is equipped with 4 x 1.9 kW feed pumps each with a flowrate of 28 $m^3$/hour at 10 m head.

### 7.6.2 Anaerobic Tank

The purpose of the anaerobic tank is to address acidogenic leachate when COD and $BOD_5$ concentrations are high.

The active micro organisms involved in the anaerobic conversion process, belong to the group of anaerobic bacteria. This group has a great variety of species that are able to, and in most cases only can, exist in an environment that excludes oxygen.

The anaerobic degradation of organic material is a stepped process (see Figure 7.6); each step is mediated by distinct groups of anaerobic bacteria.

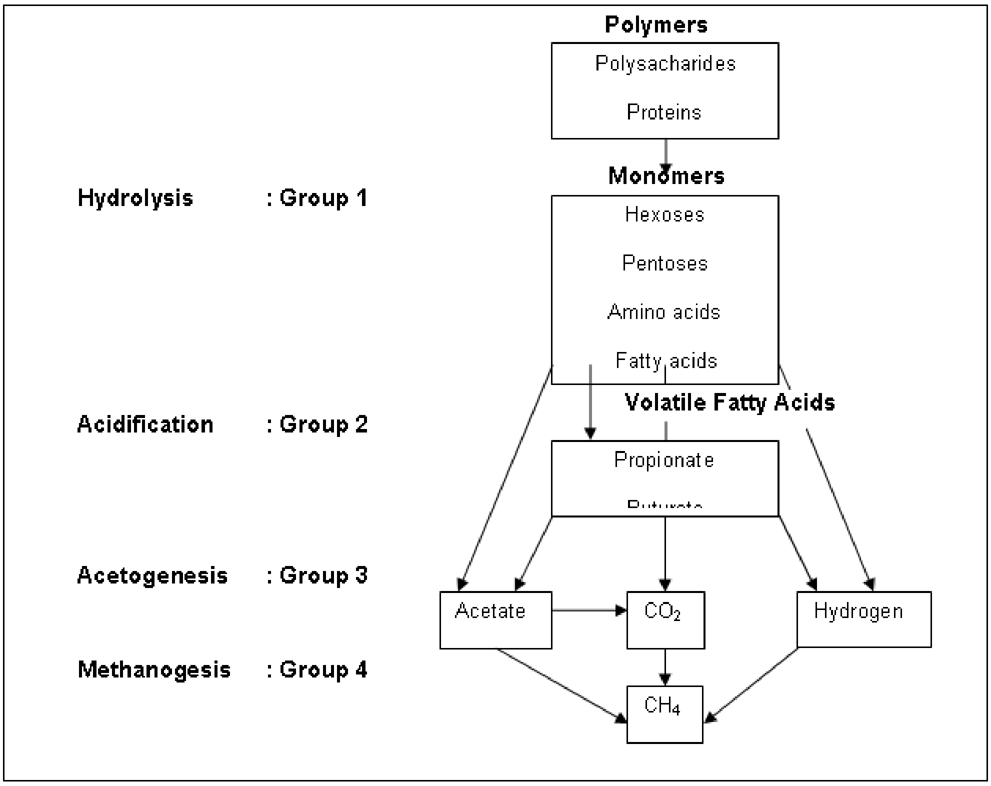
All biological degradable material is, through various intermediates, converted into biogas and additional cell material. Only during the last step (the methane and carbon dioxide generation stage) is pollution (measured as COD) removed from the waste water. The organically bound carbon will leave the water as methane $CH_4$ and carbon dioxide $CO_2$. In this context the methanogenic bacteria play a key role in the total conversion process as they are responsible for this last step.

Over 70 % of the methane production originates from bacteria that use acetic acid, the remaining 30 % of the methane is produced by bacteria which are utilising hydrogen and carbon dioxide.

The conversion rate depends on:

- The nature of the organic material (waste water composition);
- Anaerobic biomass quantity and it's adaptation and activity;
- The intensity of the contact between organic material and biomass, mixing and contact time (design);
- Environmental factors such as temperature, pH and alkalinity; and
- Availability of macro- and micro-nutrients.

**Figure 7.6: Anaerobic degradation of organic material**



### Hydrolysis

In the first process step large organic molecules like proteins, fats and carbohydrates are converted by hydrolytic micro-organisms that secrete exo-enzymes to break down large molecules (polymers) outside their cells into smaller soluble molecules like amino acids, fatty acids and sugars (monomers) which can be degraded in the next process step (acidification) by the acidogenic bacteria.

Hydrolysis will proceed rapidly with starch, however the breakdown of fats and suspended solids (cell walls of plants) is slow and hydrolysis will then be the rate limiting step. It should be noted that an increased temperature has a positive effect on hydrolysing conditions.

During the hydrolysis, the COD-concentration of the waste water hardly changes.

### Acidification (Acidogenesis)

The second process step is the conversion of the smaller soluble molecules like amino acids, fatty acids and sugars (monomers) by acidifying bacteria (they are also called fermentation organisms) into simple products like a mixture of Volatile Fatty Acids (VFA), lactic acid, alcohols, $CO_2$ and $H_2$. By-products like $H_2S$ and $NH_3$ may also develop in this phase.

Main products are Volatile Fatty Acids:

- Formic acid $C_1$
- Acetic acid $C_2$
- Propionic acid $C_3$
- (iso)Butyric acid (i)$C_4$
- (iso)Valeric acid (i)$C_5$

As an example the acidification of glucose to acetic acid is given:

$C_6H_{12}O_6 \rightarrow 3\ CH_3COOH$ ( – 206 kJ)

With this reaction, energy is released for the bacteria and they will use this energy for reproduction (growth).

### Acetic Acid Production (Acetogenesis)

In the third step the (mixture of) Volatile Fatty Acids (VFA) like propionic and butyric acid, lactic acid and alcohols are converted into acetic acid. The acetogenic bacteria are responsible for this reaction.

During this acetogenesis process a by-product ($H_2$, hydrogen) is formed, which needs to be removed in order to allow the continuation of the conversion step.

Propionic Acid $\rightarrow$ Acetic Acid + $H_2$ ( + 76.1 kJ)

As this conversion normally doesn't deliver energy for bacterial growth, this reaction only can only take place under very strict conditions of hydrogen uptake (by methane producing bacteria in the reactor) and for this reason the presence of propionic acid in anaerobic effluent indicates unfavourable process conditions.

**Methane Production (Methanogenesis)**

The group of methane forming micro-organisms converts the "end products" of the preceding processes (like hydrogen, formic acid, methanol, methylamine and acetic acid) into methane ($CH_4$) according to the following reaction equations.

Different types of methane bacteria are responsible for the different conversion mechanisms:

Hydrogen
$4\ H_2 + CO_2 \rightarrow CH_4 + 2\ H_2O$ ( – 135.6 kJ)

Acetic Acid
$CH_3COOH \rightarrow CH_4 + CO_2$ ( – 31.0 kJ)

The tank is reinforced concrete with a volume of around 7,100 $m^3$. It is equipped with 3 x 15 kW recirculation pumps. These pumps recirculate the leachate in the anaerobic tank through a piping network. This recirculation enhances contact of the anaerobic microorganisms with substrate (leachate) in the tank and leachate through mixing. Enhanced contact significantly increases the COD/$BOD_5$ removal efficiency. The high removal efficiency will prevent overloading of the downstream anoxic/aerobic biological treatment system. Production of methane from the anaerobic tank will only take place during the early stages of landfill operation, since the COD and $BOD_5$ concentration of the leachate generated during the methanogenic phase of landfill operation will drop considerably. Sophisticated covered solutions were not included as part of the anaerobic treatment system to collect methane as the production is expected to be temporary only during the acidogenic phase of landfill operation.

### 7.6.3 Aeration Tanks

The sludge in the aeration tanks will convert COD/$BOD_5$ to biomass, carbon dioxide, and water through an aerobic biological process. The tanks are made of reinforced concrete with a volume of about 6,400 $m^3$each. The tanks are equipped with 3 x 90 kW floating type mechanical aerators for each tank. The mechanical aerators are sized not only to supply the oxygen required to convert COD/$BOD_5$ but also to convert ammonia to nitrate through a process called nitrification by nitrifying microorganisms in the sludge. The tanks are also equipped with 2 x 15 kW pumps each rated at 375 $m^3$/hr at 10 m head each for recirculation back to the anoxic tanks.

The efficiency of nitrification is dependent on the ratio between organic carbon and nitrogen components (COD/N ratio). Before nitrogen can be converted into nitrates ($NO_3^-$), nitrogen containing components, have to be hydrolysed in order to enable ammonification which is described in the following equations:

(1) $C_5H_7O_2N + 5\ O_2 \rightarrow 5\ CO_2 + NH_3 + 2\ H_2O$

(2) $NH_3 + CO_2 + H_2O \leftrightarrow NH_4^+ + HCO3^-$

In anaerobic treatment nitrogen is also converted to ammonia/ammonium.

The equilibrium of reaction (2) depends on temperature and pH value. Under normal process conditions the balance will be on the right side of the equation. The produced ammonia can now be further oxidised to nitrite ($NO_2^-$) and nitrate ($NO_3^-$) the conversion to nitrite is mainly executed by the bacteria *Nitrosomonas*:

(3) $NH_4^+ + 1.5\ O_2 \rightarrow NO2^- + H_2O + 2\ H^+$

The next oxidation step from nitrite to nitrate is mainly executed by the bacteria *Nitrobacter*:

(4) $NO_2^- + 0.5\ O_2 \rightarrow NO3^-$

Of the two bacterial species *Nitrobacter* are more sensitive to toxic components than *Nitrosomonas*. In the presence of toxic compounds the accumulation of nitrite may take place; in cases where the de-nitrification process is allowed to reach equilibrium, this will not affect the overall treatment efficiency.

Favourable conditions for the nitrification process are:

- High sludge retention times (> 10 days)
- Dissolved Oxygen (DO) concentration > 1 mg $O_2$ /l
- Temperature > 10 ºC

### 7.6.4 Anoxic Tanks

The anoxic tanks convert the nitrate generated by the nitrification process to nitrogen gas through de-nitrification. De-nitrification is performed by facultative aerobic organisms who are able, under the condition of low Dissolved Oxygen concentrations (approx. 0.2 mg/l), to use nitrite and nitrate as an oxygen source for their metabolism. These bacteria will (conditionally) develop in every aerobic process; so de-nitrification processes are a common feature of all biological treatment installations.

Favourable conditions for the de-nitrification process are:

- Dissolved Oxygen (DO) concentration < 0.2 mg O2 /l
- pH values between 6.5 and 7.5
- Sufficient organic carbon source

The tanks are reinforced concrete with baffles. The baffles provide additional mixing in the tank by providing a series of overflows and underflows. Three (3) x 11 kW floating type mechanical mixers are installed for each tank in order to facilitate contact of the leachate and microorganisms and also release of the nitrogen gas.

### 7.6.5 Clarifiers

The clarifiers produce a clear overflow and concentrated sludge. Quiescent conditions must be maintained to achieve a clear overflow and a concentrated sludge. Quiescence is achieved through proper design of the tank particularly the overflow structures of the tank. A hydraulic surface loading rate of 0.7 $m^3/m^2$/hour is used to achieve quiescence. The tanks are made of reinforced concrete with 60 degrees sloped bottoms. The sloped bottom helps to promote sludge concentration and compression. Baffles and weir plates installed help in preventing the escape of residual sludge through the overflow structure. Two (2) x 2.2 kW pumps with a flowrate of 38 $m^3$/hour at a 10 m head for each tank return the sludge to the anoxic tanks to maintain the required concentration of biomass in the system. They also provide a means to bring the sludge to the sludge thickener for further processing and disposal.

### 7.6.6 Sludge Thickener

The sludge thickener reduces the volume of the sludge to be handled by further concentrating settled sludge. The supernatant from this tank will overflow to the buffer pond transfer pit. The sludge settles at the bottom of the thickener. When needed, the operators will open the valve connecting the sludge thickener with the sludge holding tank. When this valve is opened, the sludge overflows to the sludge holding tank.

### 7.6.7 Hydraulic Profile

The LTP's hydraulics are designed to allow the leachate to flow by gravity in the event of pump failure.

## 7.7 LTP DESIGN CALCULATIONS

The influent loads of the two treatment lines can be determined using the parameters shown in the table below:

**Table 7.7A: Leachate Pollutant loading in each process line**

| Parameters | Unit | Acidogenic Stage | Methanogenic Stage |
|---|---|---|---|
| Flow rate per line | [m3/d] | 450 | 900 |
| Influent COD load | [kg/d] | 18,000 | 900 |
| Influent BOD load | [kg/d] | 9,000 | 450 |
| Influent TSS load | [kg/d] | 135 | 270 |
| Infkuent TKN load | [kg/d] | 675 | 2,313 |

Because the pumps regulating the flow rate from the buffer pond to the anaerobic tank are unlikely to be maintained, a 50 % flow rate contingency has been applied to the volume calculated. In case of power or pump failure the LTP must control a higher flow rate during the rainy season.

**Influent Transfer Pit**

The influent transfer pit is connected to the storage buffer pond via an underwater pipe to ensure a steady level at all times.

The lift pumps are designed so that they can cope with the maximum design flow rate (1800 $m^3$/d, 75 $m^3$/hr). The flow rate of each pump is 28 $m^3$/hr so that only three pump are needed at the same time, hence, maintenance can be performed on the remaining pump without treatment interruption.

It is assumed that the inlet pipe DN 300 is big enough to maintain a steady level in the transfer pit and that there is no risk for the pumps to run dry.

**Anaerobic Tanks**

The anaerobic tank is designed and considered to be useful for the acidogenic stage because the $BOD_5$ concentration is too low during the methanogenic stage to perform anaerobic treatment. The removal efficiency comes after a retention time of 5 days. A design load of 8 kgCOD/$m^3$ (Kayombo, 2005) is stated in the table below.

**Table 7.7B: Anaerobic tank design calculation and removal efficiencies**

| Design parameters | Unit | Acidogenic Stage |
|---|---|---|
| Flow rate | [m3/d] | 900 |
| Design COD load of the ABR | [kg COD/(m3.d)] | 8 |
| Retention time | [days] | 5 |
| Required volume | [m3] | 4,500 |
| **Removal efficiencies** | | |
| COD removal | [%] | 70 |
| BOD removal | [%] | 70 |
| TSS removal | [%] | 70 |
| TKN | [%] | 0 |

In the long run , the anaerobic tank will not be used, but is necessary to treat the COD / $BOD_5$ during the first stage (acidogenic phase). The volume needed is calculated using the formula:

$$t = \frac{V}{Q}$$

$t = retention\ time$

$V = volume\ of\ the\ anaerobic\ tank$

$Q = leachate\ flow\ rate$

A 50 % volume contingency has been applied to reach a volume of 6,750 $m^3$.

Based on the removal efficiency stated in Table 4, the concentrations in the leachate from the anaerobic biological reactor are as follows:

**Table 7.7C: Effluent from anaerobic tank to anoxic reactor**

| Effluent from ABR and to aeration ponds | | | |
|---|---|---|---|
| **Parameter** | **Unit** | **Acidogenic Stage** | **Methanogenic Stage** |
| COD | [mg/l] | 12,000 | 1,000 |
| $BOD_5$ | [mg/l] | 6,000 | 500 |
| TSS | [mg/l] | 240 | 300 |
| TKN | [mg/l] | 1,500 | 2,570 |
| **Effluent load from ABR and to aeration ponds** | | | |
| **Parameter** | **Unit** | **Acidogenic Stage** | **Methanogenic Stage** |
| COD | [kg/d] | 5,400 | 900 |
| $BOD_5$ | [kg/d] | 2,700 | 450 |
| TSS | [kg/d] | 108 | 270 |
| TKN | [kg/d] | 675 | 2,313 |

Based on the sludge production, the calculated amount of sludge from the anaerobic tank to the sludge thickener is calculated based on the formula below:

$$m_{sludge} = \frac{[TSS] \times TSS_{removal}}{Dry\ Solid\ Content}$$

The results are shown in Table 5. The sludge flow rate is used for designing the sludge thickener.

**Table 7.7D: Sludge production from anaerobic reactor**

| ABR sludge to sludge thickener: | | Acidogenic Stage |
|---|---|---|
| Sludge from TSS | [kg/d] | 27 |
| Dry solids content | [%] | 5 |
| Sludge flow rate | [m3/d] | 0.54 |

## Nitrification/Denitrification System

A summary of the results are shown in Table 7.7E.

**Table 7.1E: Anoxic/aerobic tank system**

| | | Acidogenic Stage | Methanogenic Stage |
|---|---|---|---|
| **Design parameters** | | | |
| MLSS content | kg/m3 | 4.5 | 4.5 |
| Sludge age | days | 12 | 12 |
| Operation DO | mg/l | 1 | 1 |
| O2 transfer efficiency | kg O2/kWh | 1.8 | 1.8 |
| **Results** | | | |
| Total required volume | m3 | 4,640 | 4,368 |
| Required aeration capacity | kW | 186 | 290 |
| Aerator capacity for each aeration pond | kW | 186 | 290 |
| Anoxic volume ponds | m3 | 805 | 3,196 |
| Aerobic volume ponds | m3 | 3,834 | 1172 |
| Recirculation ratio | | 5 | 5 |

A 50 % volume contingency has been applied to obtain a volume of 5100 $m^3$ for the anoxic tank and 6,400 $m^3$ for the aerobic tank.

The maximum capacity required for the floating aerators is 290 kW. It has been chosen to install three floating aerators of 90 kW each. Also, three aerators are installed so that the power required can gradually increase with the flow rate and the $BOD_5$ concentration, resulting in no waste of energy during the first years of the landfill.

The specification of the floating aerator is as follows:

**Floating Mixer for Anoxic Tank**

| | |
|---|---|
| Manufacturer: | Aquaturbo or similar |
| Quantity: | 6 units |
| Type: | floating mixer |
| Mixing Power: | 21 kW |
| Motor: | 22 kW, 3x400 V, 50 Hz |

The recirculation rate is calculated to be in a ratio of five.

$$r = \frac{recirculation\ flow\ rate}{effluent\ flow\ rate}$$

This ratio should be calculated based on the concentration of nitrogen required for the effluent and the influent according to the following formula:

$$r = \frac{(NH_4^+ - N)_{influent} - (NH_4^+ - N)_{effluent}}{(NO_3^- - N)_{effluent}}$$

The concentration of both total nitrogen and nitrate are not known accurately. It has been chosen to take a recirculation ratio of five, which is, based on experience, a good trade-off between nitrogen removal efficiency and electricity consumption of the recirculation pumps.

Therefore, the capacity of the recirculation pump must be five times the flow rate.

$$Q_{recirculation} = r \times Q_{flow\ rate} = 5 * 75 = 375\ m^3/hr$$

The formula for calculating Actual Oxygen Requirement (AOR) is:

$$AOR = \frac{OD_{COD\ removal} + OD_{dN} + OD_N}{24}$$

$$AOR = Actual\ Oxygen\ Requirement\ in\ kg/hr$$

$$OD_{COD\ removal} = Oxyden\ Demand\ for\ removal\ of\ COD\ in\ kg/d$$

$$OD_{dN} = Oxyden\ Demand\ for\ denitrification\ in\ kg/d$$

$$OD_N = Oxyden\ Demand\ for\ nitrification\ in\ kg/d$$

With:

$$OD_{COD\ removal} = COD_{sol} + COD_{vss}$$

$$COD_{sol} = Chemical\ Oxygen\ Demand\ of\ the\ soluble\ fraction\ in\ kgO_2/d$$

$$COD_{vss} = Chemical\ Oxygen\ Demand\ of\ the\ solid\ fraction\ in\ kgO_2/d$$

And

$OD_N = Q_N \times OD_{ni}$

$OD_{ni} = Oxygen\ Demand\ for\ nitrification\ of\ one\ kg\ of\ Nitrogen = 4.6\ kgO_2/kgN$

$Q_N = Daily\ flow\ rate\ of\ nitrogen\ in\ kgN/d$

And

$OD_{dn} = \left((T_{Nnit}) \times Spont\ denit\ Rate\right) \times OD_{dni}$

$OD_{dni} = \ Oxygen\ demand\ for\ denitrification = 2.85\ kgO_2/kgN$

For the calculation of Standard Oxygen Requirement (SOR) and Power Requirement the following ratio is used:

$$R_{SOR/AOR} = \alpha \times 1.019^{T_{Op} - T_{ref}} \times \frac{T_{ref}}{101.2}\left(\beta \times \frac{O_{2sat} - O_{2op}}{O_{2n}}\right)^{-1}$$

At 30°C and 101.3kPa, the calculated ratio is 1.4. Therefore, the SOR can be calculated using the AOR.

The power requirement is calculated considering a transfer efficiency of 1.8 $kgO_2/kW$.

The volumes of the anoxic and aerobic tanks have been determined according to the following formulas:

$$V_{anoxic\ tank} = \frac{Q_N}{r \times V_{SS} \times S_{SS} \times MLSS}$$

$$V_{aerobic\ tank} = \frac{V_{sludge}}{MLSS}$$

The excel sheets used for the calculations are provided in Appendix F.

The results for the clarifier are based on the calculations detailed in Appendix F..

**Table 7.7F: Sludge production in the clarifier and quality of the effluent leachate**

| Efficiency of the activated sludge system: | | Acidogenic Stage | Methanogenic Stage |
|---|---|---|---|
| COD removal | [%] | 97.5 | 70 |
| BOD removal | [%] | 97.5 | 70 |
| TSS removal | [%] | -25 | 0 |
| TKN removal | [%] | 97 | 98 |
| Clarifier size: | | | |
| Hydraulic overflow rate (HOR) | [m/hr] | 0.7 | 0.7 |
| Side depth | [m] | 4.5 | 4.5 |
| Required surface | [m2] | 27 | 54 |
| Diameter | [m] | 5.8 | 8.3 |
| Clarifier effluent to each monitoring tanks: | | | |
| COD | [mg/l] | 300 | 300 |
| BOD | [mg/l] | 150 | 150 |
| TSS | [mg/l] | 300 | 300 |
| TKN | [mg/l] | 38 | 38 |

Based on the sludge production, the calculated amount of sludge from the anaerobic tank to the sludge thickener is based on the formula below:

$$m_{sludge} = \frac{[TSS] \times TSS_{removal}}{Dry\ Solid\ Content}$$

The results are shown in Table 8. The sludge flow rate is used to design the sludge thickener.

**Table 7.7G: Sludge production in the clarifier**

| Clarifier sludge to sludge thickener: | | Acidogenic Stage | Methanogenic Stage |
|---|---|---|---|
| Sludge from TSS | kg/d | 1,303 | 169 |
| Dry solids content | | 1 % | 1 % |
| Sludge flow rate | m3/d | 130 | 17 |

The sludge pumps are designed so that each pump is able to reach the daily required flow rate in only 7h.

$$Q = \frac{130}{24} \times 7 = 38\, m^3/hr$$

## Sludge Thickener

The sludge volume is calculated using the sludge from the ABR and the clarifier. The sludge thickener size is based on a 2-day retention time.

**Table 7.7H: Sludge thickener design parameters and sizing**

| Each sludge thickener line | | Acidogenic Stage | Methanogenic Stage |
|---|---|---|---|
| Sludge from clarifier and ABR: | | | |
| Flow rate | [m³/d] | 131 | 17 |
| Retention time | [day] | 2 | 2 |
| Required volume | [m³] | 262 | 34 |

The sludge thickener is designed with a 60 % volume contingency to reach the size of a box of 7.5*7.5*7.5m so that the clarifier would be identical to the sludge thickener, easing the construction process.

## Reed Beds

The reed beds are produced with a retention time of 3.5 days and are 0.5 m in width at the maximum flow rate.

**Table 2I: Reed beds design parameters**

| | | |
|---|---|---|
| **Flow rate** | **[m3/d]** | **1,800** |
| **Retention time** | **[day]** | **3.5** |
| **Volume required** | **[m³]** | **6,300** |
| **Depth** | **[m]** | **0.5** |
| **Area** | **[m²]** | **12,600** |

The calculation made is theoretical, as the available area is not enough to reduce the volume to 3,000 $m^3$. The reed beds shall be divided in 4 trains of 5 units each, one unit being 10m * 30m * 0.5m for a total area of 3,000 $m^3$.

The reed beds have no important influence to the design as the previous processes should be enough to treat all the pollutant. The reed beds are present only for polishing the effluent.

**Recirculation Loop**

The flow rate going through the recirculation loop is designed based on a conservative approach, referred to as the "one-and a half" approach (Maier, 1998); the initial drainage of the leachate being the first pass and the reinjection of leachate that is then stored in the waste being the half pass through the waste and the landfill.

Following this rule, leachate is reinjected to the waste to the extent that the waste can absorb it without creating potential undesirable effects such as pressure drop or slope stability failure.

Leachate will be recirculated only during the first phase of leachate production (acidogenic phase) to stabilise the waste and make sure that the LTP stays "wet".

According to the "one-and-a-half" approach and the maximum acidogenic flow rate being 900 m3/d, the recirculated flow rate must be 450 $m^3$/d.

The recirculation pumps are designed so that one of them is able to recirculate the entire flow rate, while the other is in stand-by. The capacity of each pump is 20 $m^3$/hr (480 $m^3$/d).

Two things have been considered for designing the reinjection into the landfill (Meir, 1998):

(i) the maximum allowable hydraulic head on the landfill liner; and

(ii) the rate at which leachate will percolate from the injection trench into the waste mass.

*Hydraulic Head on Liner*

It is assumed that the liner is considered to be able to withdraw a head of 300 mm, considering a leachate flow rate of 20 $m^3$/hr, the recirculated leachate has a negligible impact on the head of the liner

*Waste Absorptive Capacity and Pore Pressure*

The calculation below has been performed to determine the required size of the recirculation trenches. The leachate flow rate that can be infiltrated through the trenches is calculated with the formula below.

$$Q_t = k\left(1 + \frac{(h - P_0)}{z_f}\right)(B + h) \times L$$

The results are presented in the table below:

**Table 7.7J: Recirculation trench details**

| Recirculation Flow rate | | |
|---|---|---|
| Influent leachate flow rate of the LTP | [m3/d]<br>[m3/hr] | 900<br>37.5 |
| Maximum allowable recirculation FR | [m3/d]<br>[m3/hr] | 450<br>18.75 |
| Waste moisture content | | 30% |
| Field capacity | | 46% |
| Absorptive capacity | | 16% |

| Iteration: design of a trench - Injection capacity | | |
|---|---|---|
| Number of trench | | 3 |
| hydraulic conductivity (k) | [m/d] | 1.5 |
| depth of the wetting front below trench (zf) | [m] | 46 |
| Width of trench (B) | [m] | 0.9 |
| waste suction | [m] | 0 |
| high of trench (h) | [m] | 1.2 |
| Length of trench (L) | [m] | 45 |
| Individual Infiltration flow rate | [m3/d] | 150.8003 |
| Total Infitration flow rate ($Q_t$) | [m3/d] | 452.4009 |

The waste moisture content, field capacity, hydraulic conductivity is assumed based on similar landfill in Indonesia.

Based on these parameters, three trenches of 45m long, 1.2m high and 0.9 width are necessary. They should be parallel and located 30m away from each other. It is assumed that the leachate flow rate is too low to create pore pressure.

## Structural Calculations

The structural calculations are detailed in Appendix F.

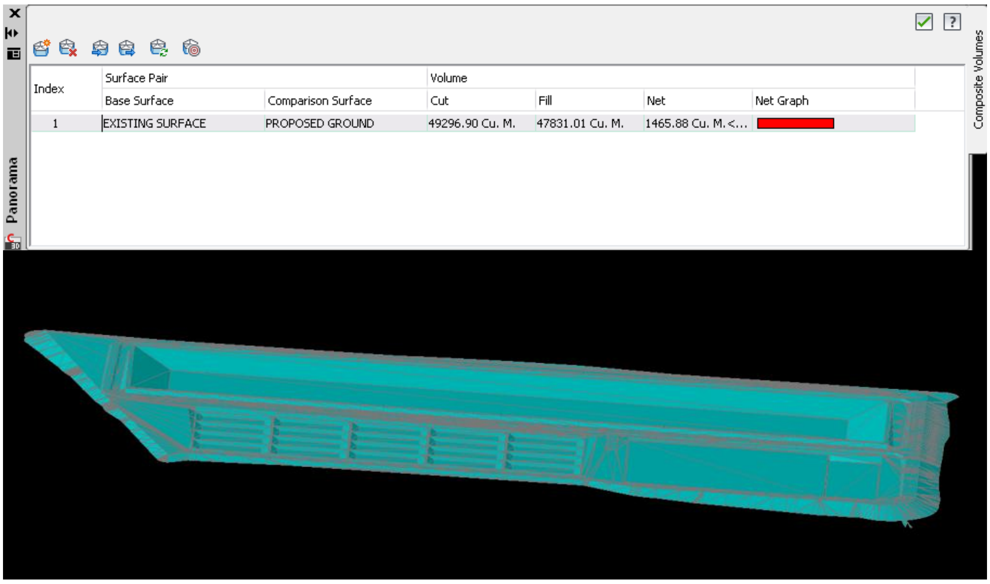
## Material Balance

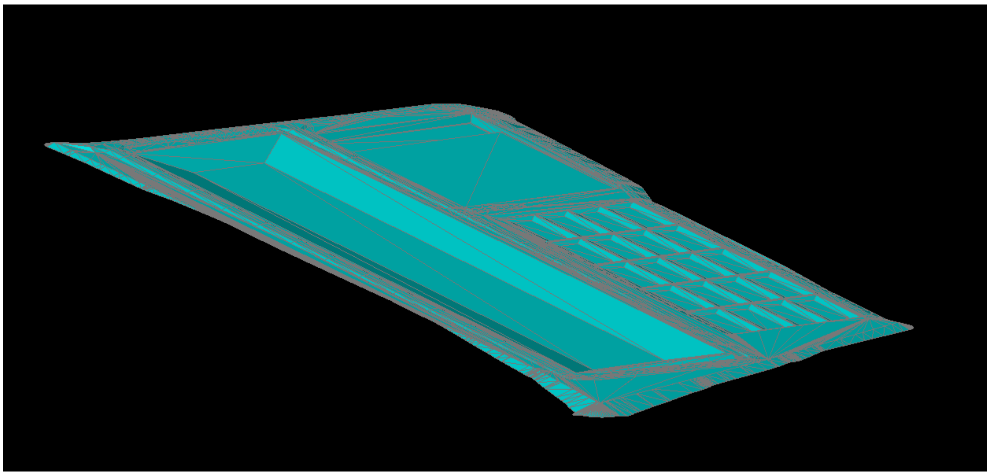
The material balance calculations have been determined using an AutoCAD Civil-3D model. The results are presented below:

**Table 7.7L: Volumes of Materials**

| Volumes | Unit | |
|---|---|---|
| Cut | [m3] | 49,297 |
| Fill | [m3] | 47,831 |
| Net | [m3] | 1,466 |

Views extracted from the model are shown below:





## Pipe Length

The length of each pipe is detailed in the table below:

**Table 7.7L: Pipe list**

| Pipe tagging | Length (m) | Diameter (mm) | Material | Nominal pressure (bar) |
|---|---|---|---|---|
| 01-01 | 57.75 | 300 | HDPE | 6 |
| 02-01 | 1.2 | 200 | HDPE | 6 |
| 02-02 | 9.3 | 300 | HDPE | 6 |
| 03-01 | 89.2 | 400 | HDPE | 6 |
| 03-A02 | 2.35 | 300 | HDPE | 6 |
| 03-B02 | 2.35 | 300 | HDPE | 6 |
| 04-A01 | 1.6 | 500 | HDPE | 6 |
| 04-B01 | 1.6 | 500 | HDPE | 6 |
| 05-A01 | 71.8 | 400 | HDPE | 6 |
| 05-B01 | 71.8 | 400 | HDPE | 6 |
| 05-A02 | 7.5 | 300 | HDPE | 6 |
| 05-B02 | 7.5 | 300 | HDPE | 6 |
| 06A-01 | 110 | 200 | HDPE | 6 |

| Pipe tagging | Length (m) | Diameter (mm) | Material | Nominal pressure (bar) |
|---|---|---|---|---|
| 06-B01 | 110 | 200 | HDPE | 6 |
| 06A-02 | 33.2 | 200 | HDPE | 6 |
| 06B-02 | 33.2 | 200 | HDPE | 6 |
| 06A-03 | 16.5 | 400 | HDPE | 6 |
| 06B-03 | 16.5 | 400 | HDPE | 6 |
| 06A-04 | 5 | 200 | HDPE | 6 |
| 06B-04 | 5 | 200 | HDPE | 6 |
| 06A-05 | 5.5 | 300 | HDPE | 6 |
| 06B-05 | 5.5 | 300 | HDPE | 6 |

## 7.8 PERFORMANCE ASSESSMENT OF THE CONSEQUENCES OF MECHANICAL EQUIPMENT FAILURE

### 7.8.1 Pump failure

The pumps of the buffer/storage pond regulate the flowrate going to the downstream unit processes. Failure of these pumps will eventually cause the buffer/storage pond to fill up and overflow to the downstream process units at an overflow rate dependent on the flowrate of the leachate coming into the LTP. It is estimated that the system can tolerate pump failure for no longer than three days. The LTP is expected to function normally during flows within the design flowrate. At flowrates significantly exceeding the design flowrate, such as during stormwater events, it is expected that the downstream processes will be overloaded. During such events, sludge may be carried over into the final effluent, significantly exceeding the Total Suspended Solids (TSS) limit of 400 mg/l.

Failure of the pumps in the anaerobic tank will just decrease its removal efficiency due to lack of mixing/contact. Failure of the pumps in the lamella settler and clarifiers may cause a reduction of solids in the anaerobic and aeration tanks from anaerobic sludge carry over to the aeration tank thus lowering their removal efficiencies.

### 7.8.2 Mixer Failure

Failure of the mixers in the anoxic tanks would reduce the efficiency of dentirification. The effluent is expected to have a high nitrate concentration. Failure of the mixer in the degasifier unit will reduce the solids in the anaerobic tank. Sludge carryover is expected but only up to the aeration tank.

### 7.8.3 Aerator Failure

Partial aerator failure would mean inadequate supply of oxygen and mixing in the aeration tanks. COD/$BOD_5$ and ammonia levels are expected to be lowered but not enough to meet the effluent discharge standards. Sludge carryover is also expected due to the favorable growth of filamentous microorganisms which have poor settling properties.

Total aerator failure would reduce COD/$BOD_5$ concentrations of the final effluent only slightly since only a few microorganisms will be mixed with the incoming leachate and only limited oxygen transfer from the atmosphere will take place through passive diffusion. The ammonia concentration is expected to remain high as the microorganisms are not able to mediate the nitrification process. The high ammonia concentrations may subsequently kill the reed bed plants in the reed bed and ultimately aquatic organisms in the river.

### 7.8.4 Total Mechanical Failure

In the case of a total mechanical equipment failure, the LTP is expected to undergo a combination of all the symptoms stated in the previous sub-sections. The removal efficiencies will not be more than 50% for COD and $BOD_5$ and only up to 20%-25% for ammonia.

# CHAPTER 8: SORTING & COMPOSTING FACILITIES

## 8.1 INTRODUCTION

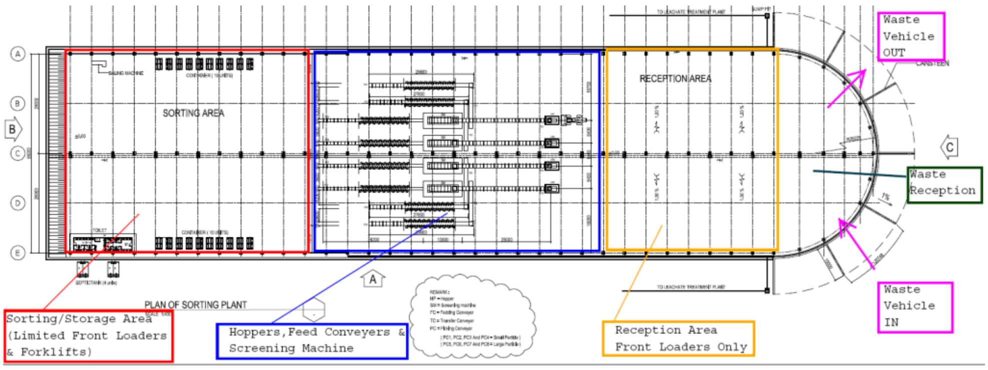
The DED requires the design of sorting & composting plants and a feasibility study for a plastic manufacturing factory.

- The sorting plant is to sort recyclables such as organics, plastics, paper, metal and glass from incoming waste streams with residues being disposed of within the landfill.
- The sorting plant (as specified in the SAPROF) is to have a capacity of 1,500tonnes per day.
- The composting plant is proposed to have two roles – pretreatment of waste prior to disposal within the landfill and the production of composts.
- The composting plant is required (as specified in the SAPROF) is to have a capacity of 50 tonnes per day.

### 8.1.1 Vehicle Flow

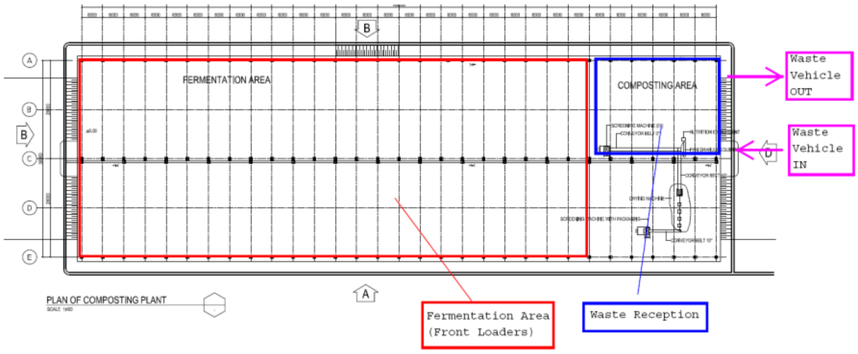
Vehicles carrying materials for sorting will enter the Site and be weighed at the weighbridge before travelling to the Sorting Plant. Waste loads will be deposited in the Sorting Plant Reception Area after which the vehicles will be required to leave the Site after being weighed in again at the weighbridge.

Deposited wastes will be picked up using Front Loaders and deposited into the Sorting Line Hoppers. Vehicle movements in the vicinity of the Sorting Line will be restricted to the removal of residues/organics at the end of day/shift (using Front Loaders) and/or the movement of Recycled Material Bins by forklift.




Delivery of Organic Wastes to the Composting Plant will either be via truck from the Sorting Plant or via truck bringing organic waste from outside the Site. Organic waste will be deposited in the Composting Plant waste Reception Area.

Deposited waste will be picked up using Front Loaders and be mechanically screened before being moved by Front Loader to the Fermentation Area. The fermentation Area has been designed as an open space to provide a flexible operational area and also allow windrows to be turned (using Front Loaders) approaching from the side of the windrow to facilitate quicker and more effective turning that allowed with a series of 'windrow boxes).




Process flows for both Sorting & Composting Plants are provided in Appendix G.

## 8.2 FINAL DESIGN FOR SORTING PLANT

### 8.2.1 Introduction

The DED requires the design of a sorting plant based on the following key criteria - .

- The sorting plant is to sort recyclables such as organics, plastics, paper, metal and glass from incoming waste streams with residues being disposed of within the landfill.
- The sorting plant (as specified in the SAPROF) is to have a capacity of 1,500 tonnes per day.

In addition, following the Preliminary Design presentaion held in Jakarta on 30th July 2010, specific requests were made to ESC that have incorporated into the Final Design. The requests were that –

*"The waste sorting plant facility should be planned for the design capacity specified in the SAPROF report, but designed for implementaion in stages."*

*"The sorting facility should be enhanced with the inclusion of basic mechanical equipment such as conveyors to improve efficiency."*

The Final Design has been developed with a view to maximising efficiency, minimising capital and operating costs whilst ensuring environmental provisions are adequate to meet design criteria.

A flexible staged implementation programme has been developed to allow decisions to be made in the future as to how to respond to increased waste inputs (increased staff vs increased equipment) and also to allow for differences in predicted and actual waste inputs. It should be noted that the design drawings have been produced on the basis of the maximum level of mechanical equipment as detailed within 'Alternative Sorting Plant Development – Stage 3' in Section 8.2.2 below.

### 8.2.2 Review of Waste Generation Rate

As previously presented in the Concept Design Report, Tables 8.4.2a & 8.4.2b below summarise the results of empirical sampling and analysis of waste disposal volumes conducted during 2006/2007 as represented in the SAPROF report. Table 8.2.2a indicates the calculated waste disposal figures (tonnes per day) where as Table 8.2.2b represents the same data in terms of waste generation per capita (kg per day per head of population).

**Table 8.2.2a**

| Total Waste Disposal 2006/2007 Empirical (SAPROF) | | |
|---|---|---|
| City/Regency | Tonnes per day | % of Total |
| Makassar | 358 | 90 |
| Gowa | 19 | 5 |
| Maros | 16 | 4 |
| Takalar | 3 | 1 |
| **TOTAL** | **396** | |

**Table 8.2.2b**

| Waste Generation Per Capita (kg/day/head) – 2006/2007 Empirical (SAPROF) | | | | |
|---|---|---|---|---|
| City/Regency | Urban Population | Collection Coverage (%) | Total Waste (kg/day) | Waste Generation (kg/day/head) |
| Makassar | 1,244,000 | 60 | 358,000 | 0.48 |
| Gowa | 256,000 | 30 | 19,000 | 0.25 |
| Maros | 144,000 | 52 | 16,000 | 0.21 |
| Takalar | 115,000 | 19 | 3,000 | 0.14 |
| **TOTAL** | | | **396,000** | |

The SAPROF report goes on to provide estimations of waste generation (tonnes per day & kg/day/head) based on two scenarios. The first scenario assumes a low level of waste generation and a high waste collection coverage (as a % of total population) where as the second scenario assumes a high level of waste generation and a low waste collection coverage. These estimations are summarised below in Tables 8.4.2c and 8.4.2d respectively.

**Table 8.2.2c**

| Total Waste & Generation Per Capita (kg/day/head) – 2027 Estimated (SAPROF) Waste Generation (Low) & Collection Coverage (High) | | | | |
|---|---|---|---|---|
| City/Regency | Urban Population | Collection Coverage (%) | Total Waste (kg/day) | Waste Generation (kg/day/head) |
| Makassar | 1,639,337 | 89 | 1,167,000 | 0.80 |
| Gowa | 362,664 | 82 | 124,000 | 0.42 |
| Maros | 189,391 | 76 | 54,000 | 0.37 |
| Takalar | 166,418 | 82 | 27,000 | 0.20 |
| **TOTAL** | | | **1,372,000** | |

**Table 8.2.2d**

| Total Waste & Generation Per Capita (kg/day/head) – 2027 Estimated (SAPROF) Waste Generation (High) & Collection Coverage (Low) | | | | |
|---|---|---|---|---|
| City/Regency | Urban Population | Collection Coverage (%) | Total Waste (kg/day) | Waste Generation (kg/day/head) |
| Makassar | 1,639,337 | 60 | 1,034,000 | 1.05 |
| Gowa | 362,664 | 30 | 59,000 | 0.55 |
| Maros | 189,391 | 52 | 48,000 | 0.49 |
| Takalar | 166,418 | 19 | 8,000 | 0.26 |
| **TOTAL** | | | **1,149,000** | |

Having reviewed the SAPROF estimations for future waste production (2027) against detailed waste arising surveys conducted in Indonesia the Consultant believes that both SAPROF scenarios represent over estimations of future waste generation rates and has calculated alternative scenarios based on likely and maximum waste generation volumes as presented below in tables 8.2.2e & 8.2.2f respectively.

**Table 8.2.2e**

| Total Waste & Generation Per Capita (kg/day/head) – 2027 Estimated (ESC)<br>Likely Anticipated Waste Generation & Collection Coverage Increases | | | | |
|---|---|---|---|---|
| City/Regency | Urban Population | Collection Coverage (%) | Total Waste (kg/day) | Waste Generation (kg/day/head) |
| Makassar | 1,675,488 | 75 | 879,630 | 0.70 |
| Gowa | 344,795 | 50 | 68,960 | 0.40 |
| Maros | 193,947 | 60 | 46,550 | 0.40 |
| Takalar | 154,888 | 30 | 13,940 | 0.30 |
| **TOTAL** | | | **1,009,080** | |

**Table 8.2.2f**

| Total Waste & Generation Per Capita (kg/day/head) – 2027 Estimated (ESC)<br>Maximum Anticipated Waste Generation & Collection Coverage Increases | | | | |
|---|---|---|---|---|
| City/Regency | Urban Population | Collection Coverage (%) | Total Waste (kg/day) | Waste Generation (kg/day/head) |
| Makassar | 1,675,488 | 80 | 1,072,310 | 0.80 |
| Gowa | 344,795 | 60 | 103,440 | 0.50 |
| Maros | 193,947 | 70 | 67,880 | 0.50 |
| Takalar | 154,888 | 40 | 24,780 | 0.40 |
| **TOTAL** | | | **1,268,410** | |

As can be seen from the above tables, there are significant variations between the SAPROF and Consultant predicted waste generation figures with the SAPROF predicting 1,149 – 1,372 tonnes per day and the Consultant predicting 1,009 – 1,268 tonnes per day.

Regardless of the differences between predicted waste inputs to the Site, the ultimate capacity of the sorting plant has been set at 1,500 tonnes per day. However, the proposed method of achieving this capacity is recommended by ESC to be based on a flexible approach so as to be able to respond to all eventualities.

### 8.2.3 Key Assumptions

The following key assumptions have been made as part of the sorting plant design process:

- Upon entering the sorting plant, waste trucks will deposit their loads onto the ground in designated areas;
- The reception area should be segregated into 6 bays to allow waste brought in at similar times to be processed as one batch;
- The waste will be visually inspected in order to identify any unsuitable wastes;
- The waste will be loaded into the hopper(s) by front loaders;
- The distance between the waste reception bays and the hoppers should be big enough to allow vehicle movements, but minimised so as to allow front loaders to collect waste from the bays and feed into the hoppers as quickly as possible;
- Each front loader can deliver 20 loads per hour into the hopper(s);
- Each hopper will drop waste onto a conveyer travelling to a screening machine;
- Each screening machine will have a capacity of 70 cubic metres per hour;
- Each screening machine will output 50% of screened waste as small particles and 50% as large particles;
- Small and large particle wastes will be released onto separate conveyers. Therefore each screening machine will feed two conveyers (one small particle & one large particle);
- Each conveyer will have a capacity of 35 cubic metres per hour and each member of the waste picking staff can pick and segregate two cubic metres of waste per hour;
- Each conveyer will have 20 picking stations (10 on each side of the conveyer) where recyclable wastes will be picked by hand ;
- The first picking station should be a minimum of 4 metres away from the start of the conveyer after the screening machine;
- Each conveyer will extend beyond the picking stations to allow residues to be transported to the end on the sorting plant for later disposal to landfill – this should be a minimum of 10 metres from the location of the last picking station on each conveyer belt;
- Washroom etc facilities should be based on maximum staffing levels indicated in Section 8.2.6.

### 8.2.4 Staged Implementation of Sorting Plant Capacity

It is recommended that the capacity of the sorting plant is designed and implemented in 3 stages. This will reduce the amount of redundant equipment installed at the facility and also allow flexibility to deal with the actual waste generation and collection rates experienced rather than those currently predicted.

- Stage 1 will cover the period from commissioning of the Site until approximately 2016/2017 and the maximum required capacity of the facility is calculated to be 400 tonnes per day based upon SAPROF estimated waste generation rates from 2006/2007.
- Stage 2 will cover the period between 2016/2017 and 2020/2021 when the maximum required capacity of the facility is calculated to be 1,000 tonnes per day based upon the predicted growth of waste generation assumed from the SAPROF report and the ESC Likely Anticipated figure for 2027.
- Stage 3 will cover the period from 2020/2021 to 2027 when the maximum required capacity of the facility is assumed to be 1,500 tonnes per day based on the requirement to design according to SAPROF predicted waste generation/collection figures. It should be noted that if the ESC Likely Anticipated figures listed in Table 2.2e prove to be more accurate that the SAPROF 2027 predictions then there will be no need for Stage 3 to be implemented.

**Recommended Sorting Plant Development – Stage 1**

This first stage assumes a maximum daily waste input of 400 tonnes which can be sorted during a single daily shift of eight hours utilising two sorting lines. The number of required mechanical components and their daily capacities is presented below.

| STAGE 1 SORTING PLANT – RECOMMENDED OPTION | | | | |
|---|---|---|---|---|
| PREDICTED WASTE INPUT | | TONNES | 400 | |
| | | CUBIC METRES | 910 | |
| NUMBER OF WORKING SHIFTS | | | 1 | |
| NUMBER OF HOURS PER WORKING SHIFT | | | 8 | |
| ITEM | NUMBER | TOTAL CAPACITY (M3 HR) | TOTAL CAPACITY (M3 SHIFT) | TOTAL CAPACITY (M3 DAY) |
| FRONT LOADER (FEED HOPPERS) | 3 | 150 | 1,050 | 1,050 |

| | | | | |
|---|---|---|---|---|
| HOPPER | 2 | | | |
| FEED CONVEYER | 2 | 140 | 980 | 980 |
| SCREENING MACHINE | 2 | 140 | 980 | 980 |
| SMALL PARTICLE CONVEYER | 2 | 140 | 980 | 980 |
| LARGE PARTICLE CONVEYER | 2 | | | |

**Recommended Sorting Plant Development – Stage 2**

The second stage assumes a maximum daily waste input of 1,000 tonnes which can be sorted during two daily shifts of eight hours utilising three sorting lines. The number of required mechanical components and their daily capacities is presented below.

| STAGE 2 SORTING PLANT – RECOMMENDED OPTION | | | | |
|---|---|---|---|---|
| PREDICTED WASTE INPUT | | TONNES | 1,000 | |
| | | CUBIC METRES | 2,276 | |
| NUMBER OF WORKING SHIFTS | | | 2 | |
| NUMBER OF HOURS PER WORKING SHIFT | | | 8 | |
| ITEM | NUMBER | TOTAL CAPACITY (M3 HR) | TOTAL CAPACITY (M3 SHIFT) | TOTAL CAPACITY (M3 DAY) |
| FRONT LOADER (FEED HOPPERS) | 4 | 200 | 1,400 | 2,800 |
| HOPPER | 3 | | | |
| FEED CONVEYER | 3 | 210 | 1,470 | 2,940 |
| SCREENING MACHINE | 3 | 210 | 1,470 | 2,940 |
| SMALL PARTICLE CONVEYER | 3 | 210 | 1,470 | 2,940 |
| LARGE PARTICLE CONVEYER | 3 | | | |

**Recommended Sorting Plant Development – Stage 3**

This third stage assumes that the SAPROF predicted maximum daily waste input of 1,500 tonnes will be partially or fully achieved. It is recommended that any increase in waste inputs above 1,000 tonnes per day is managed via the employment of additional manpower. The table below indicates how three shifts of seven hours can cope with 1,500 tonnes per day. The second stage three sorting lines can thus cope with the extra waste input without any additional expenditure on plant and machinery. The number of required mechanical components and their daily capacities is also presented below.

<table>
<tr><th colspan="5">STAGE 3 SORTING PLANT – RECOMMENDED OPTION</th></tr>
<tr><td colspan="2" rowspan="2">PREDICTED WASTE INPUT</td><td>TONNES</td><td colspan="2">1,500</td></tr>
<tr><td>CUBIC METRES</td><td colspan="2">3,414</td></tr>
<tr><td colspan="3">NUMBER OF WORKING SHIFTS</td><td colspan="2">3</td></tr>
<tr><td colspan="3">NUMBER OF HOURS PER WORKING SHIFT</td><td colspan="2">7</td></tr>
<tr><td>ITEM</td><td>NUMBER</td><td>TOTAL CAPACITY (M3 HR)</td><td>TOTAL CAPACITY (M3 SHIFT)</td><td>TOTAL CAPACITY (M3 DAY)</td></tr>
<tr><td>FRONT LOADER (FEED HOPPERS)</td><td>4</td><td rowspan="2">200</td><td rowspan="2">1,200</td><td rowspan="2">3,600</td></tr>
<tr><td>HOPPER</td><td>3</td></tr>
<tr><td>FEED CONVEYER</td><td>3</td><td>210</td><td>1,260</td><td>3,780</td></tr>
<tr><td>SCREENING MACHINE</td><td>3</td><td>210</td><td>1,470</td><td>2,940</td></tr>
<tr><td>SMALL PARTICLE CONVEYER</td><td>3</td><td rowspan="2">210</td><td rowspan="2">1,260</td><td rowspan="2">3,780</td></tr>
<tr><td>LARGE PARTICLE CONVEYER</td><td>3</td></tr>
</table>

**Alternative Sorting Plant Development – Stage 3**

This alternative third stage again assumes that the SAPROF predicted maximum daily waste input of 1,500 tonnes will be partially or fully achieved. If the recommendation that any increase in waste inputs above 1,000 tonnes per day be managed via the employment of additional manpower is not accepted, then an additional sorting line will be required. The table below indicates how two shifts of eight hours can cope with 1,500 tonnes per day, utilising four sorting lines and also indicates the number of required mechanical components and their daily capacities.

| STAGE 3 SORTING PLANT – ALTERNATIVE OPTION | | | | |
|---|---|---|---|---|
| PREDICTED WASTE INPUT | | TONNES | 1,500 | |
| | | CUBIC METRES | 3,414 | |
| NUMBER OF WORKING SHIFTS | | | 2 | |
| NUMBER OF HOURS PER WORKING SHIFT | | | 8 | |
| ITEM | NUMBER | TOTAL CAPACITY (M3 HR) | TOTAL CAPACITY (M3 SHIFT) | TOTAL CAPACITY (M3 DAY) |
| FRONT LOADER (FEED HOPPERS) | 5 | 250 | 1,750 | 3,500 |
| HOPPER | 4 | | | |
| FEED CONVEYER | 4 | 280 | 1,960 | 3,920 |
| SCREENING MACHINE | 4 | 280 | 1,960 | 3,920 |
| SMALL PARTICLE CONVEYER | 4 | 280 | 1,960 | 3,920 |
| LARGE PARTICLE CONVEYER | 4 | | | |

### 8.2.5 Staff Roles

The table below summarises the suggested responsibilities of the different staff roles that will be required to operate the sorting plant.

| STAFF ROLE | RESPONSIBILITIES |
|---|---|
| PLANT MANAGER | Overall responsibility for the smooth operation of the sorting plant. |
| SHIFT MANAGER | Day to day responsibility for the smooth operation of each shift. |

| STAFF ROLE | RESPONSIBILITIES |
|---|---|
| WASTE RECEPTION SUPERVISER | Day to day responsibility for the smooth operation of the reception area. |
| WASTE RECEPTION STAFF | Direct vehicles as to where to deposit waste loads, visual inspection of waste for non permissible items and direction of front loader drivers as to which waste to feed into the hoppers. |
| FRONT LOADER DRIVER | Load hoppers with waste in a timely manner such that a smooth and consistent delivery of waste to the sorting line is achieved |
| SORTING LINE SUPERVISR | Day to day responsibility for operation of sorting line from the hoppers to the deposit of residues. |
| FEED CONVEYER STAFF | Visual inspection of waste for non permissible items and manual cutting of bagged waste. |
| WASTE PICKING STAFF | Picking of recyclable wastes and placement into segregated containers. |
| FORKLIFT DRIVER | Movement of segregated wastes from picking lines and operation of recyclable processing and storage area. |
| FRONT LOADER DRIVER | Removal of residues from sorting plant and placement in vehicles for deposit at landfill. |
| TECHNICIAN | Routine repair and maintenance of sorting plant equipment. |

### 8.2.6 Staff Numbers

The table below summarises the predicted number of staff required to operate the sorting plant during the development stages detailed in Section 8.2.4.

**Table 3**

| STAFF ROLE | STAFF NUMBERS PER DAY | | | |
|---|---|---|---|---|
| | STAGE 1 | STAGE 2 | STAGE 3 (RECOMMENDED) | STAGE 3 (ALTERNATIVE) |
| PLANT MANAGER | 1 | 1 | 1 | 1 |
| SHIFT MANAGER | 1 | 2 | 3 | 2 |

| STAFF ROLE | STAFF NUMBERS PER DAY | | | |
|---|---|---|---|---|
| | STAGE 1 | STAGE 2 | STAGE 3 (RECOMMENDED) | STAGE 3 (ALTERNATIVE) |
| WASTE RECEPTION SUPERVISER | 1 | 2 | 3 | 2 |
| WASTE RECEPTION STAFF | 2 | 4 | 6 | 4 |
| FRONT LOADER DRIVER | 3 | 8 | 10 | 12 |
| SORTING LINE SUPERVISR | 1 | 2 | 3 | 2 |
| FEED CONVEYER STAFF | 4 | 12 | 16 | 18 |
| WASTE PICKING STAFF | 64 | 168 | 240 | 288 |
| FORKLIFT DRIVER | 1 | 2 | 3 | 2 |
| FRONT LOADER DRIVER | 1 | 2 | 3 | 2 |
| TECHNICIAN | 1 | 2 | 3 | 2 |
| **TOTAL** | **80** | **205** | **291** | **335** |

### 8.2.7 Sorting Plant Design Calculations

The Sorting Plant consist of Reception and Sorting Areas contained within a Steel structure building, without brick walls, so as to make it easier for the circulation of waste transportation vehicles and heavy equipment.

Plants Dimension Calculation:

**Reception Area:**

Total waste = 1,500 ton/day

= 3,414 $m^3$/day (source SAPROF section 16.5).

**Assumptions:**

Td (time detention) = 2.0 days (anticipate the holiday)

T (waste pile height) = 1.5 metres

A (area requirement) = Volume/T

Total waste volume = 2 x 3,414

= 6,828 $m^3$/2 days

For manoeuvring and circulation of heavy equipment (front loader, etc)

Additional 20% area requirement:

Area requirement = 1.2 x 6.28 /1.5

= 5,462 $m^2$/2 days

The dimensions of Reception Area:

Length = 78 metres

Width 2 x 28 = 56 metres

= 4,368 $m^2$

**Sorting Area:**

Based on empirical experience in Pusat Daur Ulang Kompos (PDUK) Cakung-Cilincing, each sorting staff can finished sorting approximately 2.0 $m^3$/person/hour using a conveyor belt system.

Based on empirical experience in Pusat Daur Ulang Kompos (PDUK) Cakung-Cilincing (previous work conducted by ESC's Indonesian design subcontractor), each member of the sorting staff can finish sorting approximately 2.0 $m^3$ in 1 hour (2.0 $m^3$/person/hour).

The area requirement:

For sorting staff work space ≈ 1.2 $m^2$ x 170 = 204 $m^2$

Machinery & Equipment = 3,507 $m^2$

Vehicle Circulation = 1,465 $m^2$20 cm

Recylable Storage = 1,099 $m^2$

Total Area Requirement = 6,275 $m^2$

So the dimension of Sorting Area :

Length = 120.0 metres

Width 2 x 28 = 56.0 metres

= 6,720 $m^2$

The sorting plant includes, six toilets and 6 basins for female staff and four toilets, seven urinals and 6 basins for male staff, and 4 septic tanks.

## 8.3 FINAL DESIGN FOR COMPOSTING PLANT

Flowcharts for the process are presented in Appendix G along with the design drawings.

### 8.3.1 Composting Plant Design Calculations

This Plant consists of Fermentation Area and Composting facilities enclosed within a steel structure building with 5 staff employed for a single 8 hour daily shift

The anticipated input of organic waste from local markets is 10 ton/day, which will be combined with an anticipated 40 ton/day of small particle organic waste from the Sorting Plant. The anticipated maximum throughput of organic waste is therefore 50 ton/day which equates to 250 $m^3$/day of organic waste according to the SAPROF report and the required area of the Composting Plant has been based on that figure.

The td (time detention) for Pre Fermentation is 14 days, and td for Fermentation is 35 days.

**Fermentation Plant calculation:**

Small Organic waste = 40 ton/day ≈ 200.0 $m^3$/day

Market waste = 10 ton/day ≈ 50.0 $m^3$/day

Organic for Compost = 50 ton/day ≈ 250.0 $m^3$/day

**Assumptions:**

| | |
|---|---|
| Td1 (Pre Fermentation) | = 14 days |
| Decomposition in Pre-Fermention | = 60% |
| Td2 (Fermentation) | = 35 days |
| Decomposition in Fermentation | = 60% |
| W (pile width) | = 2 metres |
| T (compost pile height) | = 1.2 metres |
| A (pile cutting area) | = 2.4 $m^2$ |

Total Volume:

| | |
|---|---|
| Pre Fermentation | = 1,400.0 $m^3$/14 days |
| Fermentation | = 3,500.0 $m^3$/35 days |

Area Requirement

| | |
|---|---|
| Pre Fermentation | = 1,166.7 $m^2$/14 days |
| Fermentation | = 2,916.7 $m^2$/35 days |

For manoeuvring and circulation of heavy equipment (front loader,etc) an additional 100% is required for the plant area due to the need to revolve the piles (windrow composting method) both in Pre and Fermentation Plant.

So

| | |
|---|---|
| Pre Fermentation Area requirement | = 2,333.3 $m^2$/14 days |
| Fermentation Area requirement | = 5,833.3 $m^2$/35 days |
| Total Area requirement | = 8,166.7 $m^2$/total days |

The dimension of Pre Fermentation and Fermentation Plant:

| | |
|---|---|
| Length | = 150.0 metres (per 6 metres) |
| Width 2 x 28 metres | = 56.0 metres |
| Area | = 8,400.0 $m^2$ |

### 8.3.2 Approved Additional Recommendations From Preliminary Design

The Preliminary Design of the Composting Plant included two optional components, namely the inclusion of nutrient enrichment and granulation equipment and a compost drying machine. These two recommended additions were approved at the Preliminary Design Meeting held in Jakarta on 30$^{th}$ July 2010 and have been included within the Final Design based on the justifications detailed below.

Effectively the drying machine is necessary as a result of the inclusion of the granulation equipment, as the drying machine is essential to retain the compost powder in a granular form after the enrichment nutrition process.

The drying process can be done naturally by laying powder compost in an open area inside the fermentation plant but a disadvantage is that the compost produced would only be in a powder from as opposed to granules (which my attract a higher sales price in the marketplace). Additional acknowledged benefits are that the drying process can be completed quicker within a smaller area by use of the drying machine but the additional procurement, operational and maintenance costs will obviously be higher as compared to manual drying.

# CHAPTER 9: SUPPORTING FACILITIES

Site supporting facilities for the proposed RSWDS consist of both structures (management office, community facility, workshop, weighbridge, auditorium etc) and key services (water & electrical supply and sewage & sanitation). All of these facilities are described below in the relevant sections – please note that buffer zones have been covered previously in this document.

## 9.1 SITE STRUCTURES

The following table summarises the key design criteria for the site buildings and structures. Final Design drawings are presented in Appendix G.

| Structures | Area ($m^2$) | Description & Staffing | Sanitary Facilities |
|---|---|---|---|
| Main Gate | | Gates will be hinged rather than sliding to remove the potential for sliding parts to become clogged and therefore unusable. | 0 |
| Fencing | | The option selected for the boundary fencing was a concrete panel fence which provides greater security and durability but at a higher cost.<br>Chain link fencing is included around areas such as the leachate treatment plant where there are health &safety risks present. | 0 |
| Site Roads | | Hard paving has been included within the DED for routes linking the site entrance with site buildings, sorting/composting plants, LTP, soil stockpiles, landfill cells and leachate treatment plant and also around the entire perimeter of the landfill. The general profile of the hard paving is provided in detail in Drawing No. MRSWDF-1002 in Appendix C. | 0 |
| Landscaping | | Landscape planting is to be provided in three (3) general areas/applications around the Site: amenity planting in the areas of the Site Entrance and Support Facilities; tree planting within the Buffer Zone; and grasses and trees (selected locations) on the soil stockpiles. The final cover of the Landfill will be planted with grasses to prevent root penetration through the final cover soils. | 0 |

| Structures | Area ($m^2$) | Description & Staffing | Sanitary Facilities |
|---|---|---|---|
| Guard Post | 24 | Single storey building<br>2 staff per shift (3shifts per day) | Toilet:1<br>Septic Tank:1 |
| Weighbridge & Post | 64 | Single story building<br>2 staff per shift (3shifts per day) | Toilet: 1<br>Septic Tank: 1<br>Weighbridge: 2 |
| Wash Bay | 317 | Single storey building<br>2 staff<br>It is assumed that each of the 114 trucks will be washed once every 3 days | 0 |
| Management Office | 2,160 | Double storey building<br>24 staff consisting of:<br>Team Leader: 1<br>Deputy Team Leader: 1<br>Administration Section<br>• Section Head: 1<br>• Clerk: 1<br>• Admin & Financial Officer: 1<br>• Security: 2<br>• Head of Storage: 1<br>Maintenance & Supervision Section<br>• Section Head: 1<br>• General Supervisor: 1<br>• Repair Station/workshop Officer:2<br>• Water Laboratory Analyst: 1<br>Operation Section:<br>• Section Head: 1<br>• Heavy Equipment Drive: 4<br>• Leachate Processing Installation Organiser: 1<br>• Incineration Operator: 4<br>• Composting Implementation Officer: 1 | Toilet: 9<br>Urinal: 8<br>Basin: 9<br>Septic Tank: 3 |

| Structures | Area ($m^2$) | Description & Staffing | Sanitary Facilities |
|---|---|---|---|
| Auditorium | 416 | Single storey building<br>347 person capacity | Toilet: 2<br>Septic Tank: 1 |
| Dormitory (Mess) | 688 | Double storey building | Bedroom: 16<br>Toilets: 8<br>Basins: 8<br>Septic Tank: 2 |
| Community Facility | 144 | Single storey building<br>84 person capacity | Toilet: 2<br>Wudhu Tap: 8<br>Septic Tank: 1 |
| Workshop and Garage | 615 | Single storey building<br>2 staff<br>Parking capacity for 30 trucks | Toilets: 2<br>Septic Tanks: 2 |

## 9.2 WATER SUPPLY

Detailed data regarding the calculation of required water supply is presented in Appendix G along with design drawings.

The clean water calculations are in accordance with the following standards:

- SNI 03-6481-2000 Sistem Plambing; and
- Peraturan Menteri PU No. 18 Tahun 2007 tentang Penyelenggaraan Pengembangan Sistem Penyediaan Air Minum No. 18/PRT/M/2007

### 9.2.1 Water Consumption Calculations

Water consumption per employee per day = 50 litres/person/day (refer to SNI 03-6481-2000 System Plumbing). If staff at a location work in three 8 hour shifts (24 hours total) the water consumption multiply by 3, so the water consumption is 150 litres/employee/day. This calculation has been applied to the Guard Post, Weighbridge, and Sorting Plant.

Water consumption for the Community Facility is based upon 10 litres/employee/pray time, and it has been assumed that a normal work day will involve three prayer sessions giving a water consumption figure of 30 litres/employee/day.

Water consumption for the Mess has been calculated as 120 litres/bedroom/day in accordance with SNI 03-6481-2000 System Plumbing).

Q maximum day= C1 x Q average

C1 = 1.2 (refer to Peraturan Menteri PU No. 18 Tahun 2007 tentang Penyelenggaraan Pengembangan Sistem Penyediaan Air Minum No. 18/PRT/M/2007 page 30)

Q peak hour = C2 x Q average

C2 = 1.75 (refer to Peraturan Menteri PU No. 18 Tahun 2007 tentang Penyelenggaraan Pengembangan Sistem Penyediaan Air Minum No. 18/PRT/M/2007 page 55)

The Head Loss Calculation (HL) for critical point in metres (refer to Hazen Williams formula, and Peraturan Menteri PU No. 18 Tahun 2007 tentang Penyelenggaraan Pengembangan Sistem Penyediaan Air Minum No. 18/PRT/M/2007 page 63):

HL = (Q /(0.2785 x 110 x (D)^2,63 ) )^1.85 x L

HL = metres

Q = water flow (debit in $m^3$/detik)

D = pipe diameter in metres

L = pipe distance in metres

Total water consumption = 63.44 $m^3$/day

Water loss = 15%, so the Q average = 72.96 $m^3$/day.

(refer to Peraturan Menteri PU No. 18 Tahun 2007 tentang Penyelenggaraan Pengembangan Sistem Penyediaan Air Minum No. 18/PRT/M/2007 page 27).

The Q maximum day = 87.55 $m^3$/day

The Q peak hour = 127.67 $m^3$/day

### 9.2.2 Source of Clean Water

Clean water will be obtained from an on-site deep well, water being pumped from the deep well straight into the elevated storage tank prior to distribution into each building.

The required depth of the well has been estimated as 100 metres depth (assumption, field adjustment) to avoid any potential contamination of the source by RSWDS activities. The well water will be pumped using a submersible pump. The required volume of the elevated tank has been calculated as 15% of the Q maximum per day (refer to Peraturan Menteri PU No. 18 Tahun 2007 tentang Penyelenggaraan Pengembangan Sistem Penyediaan Air Minum No. 18/PRT/M/2007 page 57).

| Water Supply Design Criteria | | |
|---|---|---|
| **Base Data** | | |
| Q = 87.55 $m^3$/day or 60.8 litres per minute | | |
| H = 80 Metres + 20 Metres + 10% (Head Loss) = 110 Metres | | |
| P = ρ x Q x H | | |
| **Formula for Water Supply Pumping Calculation** | | |
| P | = | $\dfrac{\rho.g.Q.H}{\text{Pump Efficiency}}$ |
| P | = | Power, Watt |
| ρ | = | Water density, 1,000 Kg/$M^3$ |
| g | = | Gravity, 9.8 M/Second |
| Q | = | Water consumption, $m^3$/second = 0.00101 $m^3$/Second |
| H | = | Submersible pump to Elevated Reservoir Level |
| Pump Efficiency | = | 60% |
| P= (1000 x 9.8 x 0.00101 x 110)/0.6 = 1,814.6 Watt | | |
| Therefore the submersible pump should have a requirement of 2.2 KWatt. | | |

### 9.2.3 Plumbing Design Criteria

A summary of the plumbing design criteria is provided in the table below.

| PLUMBING DESIGN CRITERIA (refer to SNI 03-6481-2000 System Plumbing) | |
|---|---|
| Plumbing Fixture | Minimum PVC Pipe Diameter for Cold Water (mm) |
| Kitchen sink | 15 |
| Lavatory | 10 |
| Urinal | 20 |

| PLUMBING DESIGN CRITERIA (refer to SNI 03-6481-2000 System Plumbing) | |
|---|---|
| Closet with low down flush tank | 25 |
| Water Tap for Shower Bath | 15 |
| Water Tap for Community Facility | 10 |
| Water Tap for Toilets | 15 |
| Water Tap for Gardening | 15 |
| Minimum Clean Water Pipe for Stack Pipe | |
| If Flush Valve = 0 | 15 |
| If Flush Valve = 1 - 2 units | 32 |
| If Flush Valve > 2 units | 40 |
| NB - If clean water quality is not high enough due to high Total Solids concentration the minimum pipe diameter required will be one size greater than listed above. | |

## 9.3 ELECTRICAL SUPPLY

The Electrical Supply will be derived from Diesel Generator Sets. Two locations for generators have been planned with the first location providing power to all supporting facilities and the second location providing power to the LTP. For each location the predicted electrical consumption requirements have been calculated. These calculated figures have then been increased by 25% - 30% to provide spare capacity for any future Site developments and also to ensure that generators do not have to operate at maximum output.

Again, at each of the two locations, the required size of diesel tanks (to feed the generators) have been calculated so as to have the capacity to feed the generators for a two week period without the diesel tanks needing to be refilled from delivery trucks. This is to ensure that the number of fuel deliveries is minimised whilst also providing sufficient capacity to cover for any potential disruptions to diesel deliveries to the Site.

Finally, a backup generator has been provided for at each location to ensure that electrical supplies can be maintained at required levels even if one generator at each location were to fail.

Full tabulated details of the calculated generator and diesel tank capacities are provided in Appendix G, with a brief summary provided below –

**Supporting Facilities**

Two operational generators will be installed together with two distribution panels (MDP 1 & MDP 2). MDP 1 will provide power for all lighting and equipment plugged into standard electrical sockets. MDP 2 will provide power for all sorting and composting plant machinery. MDP 1 & MDP 2 will be served by separate diesel generators. The total power requirement for MDP 1 has been calculated at 206 kVa and it will be served by a 260 kVa genset (capacity approximately 26% above calculated requirement). The total power requirement for MDP 2 has been calculated at 200 kVa and it will be served by a 260 kVa genset (capacity approximately 20% above calculated requirement). The third (backup) genset will also have a capacity of 260 kVa.

Both gensets have been calculated to have diesel consumption rates of 14.7 litres per hour giving a combined rate of 29.4 litres per hour. Extrapolated to a 2 week period this gives a combined predicted consumption of 9,878.4 litres. The diesel supply tank has therefore been designed with a volume of 10,000 litres.

**LTP**

Two operational generators will be installed together with two distribution panels (MDP 3 & MDP 4). Both MDP 3 & MDP 4 will provide for designated elements of the LTP as indicated in Appendix G. MDP 3 & MDP 4 will be served by separate diesel generators. The total power requirement for MDP 3 has been calculated at 510 kVa and it will be served by a 675 kVa genset (capacity approximately 32% above calculated requirement). The total power requirement for MDP 4 has been calculated at 555 kVa and it will be served by a 675 kVa genset (capacity approximately 22% above calculated requirement). The third (backup) genset will also have a capacity of 675 kVa.

Both gensets have been calculated to have diesel consumption rates of 45 litres per hour giving a combined rate of 90 litres per hour. Extrapolated to a 2 week period this gives a combined predicted consumption of 30,240 litres. Two diesel supply tank have therefore been designed each with a volume of 16,000 litres, giving a combined volume of 32,000 litres.

### 9.3.1 Electrical Supply Design Criteria

The electrical supply calculations are in accordance with the following standards:

PUIL 2000

- Planning of electrical installations;
- Cable type;
- Equipments of electrics.

SNI,IEEE and BSN.

- Installation AC for 1.5 pk = 24 $m^2$
- Installation Street Lighting = 30 m
- Installation illuminators amount. = 15 Watt/$m^2$

A summary of the electrical supply design criteria is provided in the table below.

| Electrical Supply Design Criteria |
|---|
| **Formula for AC** |
| ( L x W ) x 500 Btu/hr<br>L = Length ( m )<br>W = Wide ( m )<br>500 Btu/hr = Set of AC |
| **Formula for Street Lighting** |
| S = $\dfrac{FxUxnxM}{ExW}$<br>S = Distance<br>F = Flux (lumen) 6,300<br>U = Utility Factor 0.297<br>N = Lamps amount.<br>M = Factor of Maintenance 0.9<br>E = Lighting strength. ( Lux )<br>W = Street Width ( m ) |
| **Formula for Illumination** |
| N = $\dfrac{ExA}{\Phi xLLFxCU}$<br>Φ = Lumen Lamps<br>E = Lighting strength.<br>A = wide area work<br>( LLF )Light Loss Factor = 0.7-0.8. LLF<br>( CU ) Coefficient of Utilisation = 50-65 % |

| Formula for Ampere |
| --- |
| $$I = \frac{p}{Vx\sqrt{3}x\cos\phi}$$ I = Ampere<br>V = Volt<br>P = Energy<br>Cos φ = 0.8 |

### 9.3.2 Electrical Supply Requirements – Future Mains Connection

As part of the DED, the potential future connection of the Site to electricity from the distribution grid is to be considered and the termination points of such a connection considered.

As part of the DED the four distribution panels MDP 1 – MDP 4 have been designed with Automatic Change Over Switches (ACOS) with 500 Amp MDP 1 & MDP 2) and 1,400 Amp capacities (MDP 3 & MDP 4).

## 9.4 SEWERAGE & SANITATION

Detailed data regarding the calculation of the required sewerage and sanitation capacity is presented in Appendix G together with design drawings.

The sewerage system involves on-site treatment, with septic tanks provided for each building.

The sewerage calculations are in accordance with the following standards:

- SNI 03-6481-2000 Sistem Plambing;
- Standar Teknis MCK dan Tangki Septik, Dep PU, 1985; and
- Buku Referensi: Pilihan Sistem dan Teknologi Sanitasi, Tim Teknis Pembangunan Sanitasi, 2010

### 9.4.1 Septic Tank Design Criteria

A summary of the septic tank design criteria is provided in the table below.

| Septic Tank Design Criteria |
|---|
| The Ratio of Length : Wide for the foursquare concrete structure septic tank is 2:1 to 3:1.<br>The water level in septic tank at least 1.00 metre and 2.10 metres maximum depth.<br>Free board at least 20-40 cm.<br>Minimum septic tank width = 0.75 metres with 1.50 metres minimum length.<br>Sewage quantity for office staff = 80% x clean water consumption/day<br>Black Water = 20% x sewage quantity<br>Grey Water = 80% x sewage quantity<br>Sludge accumulate quantity = 35 liter/person/year |
| Septic Tank Volume (V):<br>Va = Q x O x T<br>Va= Sewage Volume in septic tank, $m^3$<br>Q = 20% x 80% x clean water consumption, liter/person/day<br>O = Septic tank user quantity, person<br>T = time detention, 3 days<br>Formula above excludes free board and sludge tank |
| Sludge Tank:<br>V1 = O x L x P<br>V1= Sludge quantity, $m^3$<br>O = Septic tank user quantity, person<br>L = 0,035 $m^3$/person/year<br>P = 3 year |
| Anaerobic Filter detention time = 12 Hours<br>Anaerobic Filter Compartment = Q x td<br>The filter consists of a wooden box frame with pieces of coconut shell inside. The operation of the septic tank is normal, and the effluent from the septic tank enters the anaerobic filter compartment and is treated by contact with the biological growth on the surface and inside the piece of coconut shell.<br>Septic Tank Volume V = Va + V1<br>Minimum diameter pipe for sewerage = 100 mm for PVC pipe, with 2% slope<br>Minimum distance between septic tank with well = 10 metres.<br>Composition of Untreated Domestic wastewater based on Metcalf & Eddy, Wastewater Engineering, 1991 |

# CHAPTER 10: ACCESS ROAD

A final report on the Site access road is presented in Appendix H.

# CHAPTER 11: TRAINING MATERIALS

Training materials have been prepared in accordance with the key principles set out below and are presented in Appendix I.

- Manuals are brief and written in a non-technical manner so that they can be clearly understood with zero or minimal pre-training;
- Manuals are practical to the situation(s) that staff will encounter and not written on a theoretical level;
- Manuals are based on examples known to work elsewhere with a particular emphasis on successful best practice adoption in Indonesia;
- Manuals include appendices that contain more detailed practical reference materials, but again these reference materials will be predominantly related to Indonesian examples and experiences so as to make them relevant to the target audience;
- The structure of all documents is based upon simple and intuitive icons that will serve as effective navigation and reference tools common to the suite of documents and presentation materials.

# CHAPTER 12: PLASTICS RECYCLING FEASIBILITY REPORT

The Feasibility Report (Appendix J) represents the full ESC findings in relation to the potential for establishing a plasic factory/imndustry at the Site and/or selling raw plastic to established factories in the region. A summary of the main findings is provided below for ease of reference.

## 12.1 INTRODUCTION

The Indonesia Infrastructure Initiative (IndII) has appointed EnviroSolutions & Consulting Pte Ltd (ESC) in association with Vista Consulting Engineers Limited (Vista) to prepare the Detailed Engineering Design (DED) for the Regional Solid Waste Disposal Site (RSWDS) Mamminasata, South Sulawesi Province, Indonesia. The detailed scope of work and programme for the DED was set out in the study's *Inception Report (Revision 1)* (March 2010).

The RSWDS will have an ultimate capacity of 5 million cubic metres, with a first stage of 2.36 million cubic metres, which is covered by this IndII project. The goal of the RSWDS is to contribute to long-term development needs of Indonesia through supporting best practice and sustainable solid waste management for large urban areas. The DED is to employ best practice standards to serve as a model for other solid waste disposal services subsequently built in Indonesia.

This report represents ESC's findings in relation to the potential for establishing a plastic factory/industry at the Site and/or selling raw plastic to established factories in the region.

## 12.2 PLASTICS AND PLASTIC RECYCLING

Generally, the polymers from which plastics are made are immiscible, that is to say they cannot be mixed together to obtain a homogeneous material. This finding has a serious consequence: It is necessary to sort the plastics before recycling based on the base polymer from which the plastic waste is manufactured. Plastics fall into two broad families: thermoplastics and thermosetting polymers. Comparing these types, thermoplastics are much easier to adapt to recycling as they can be melted and reformed into new products. Thermoplastics are also by far the most widely used polymers. There are several hundreds of types, but only fifteen of them are used for common applications and by far the greatest proportion of plastic wastes are thermoplastic wastes from the packaging sector.

Four thermoplastics have a share over 70% of all the polymers in the global market: polyethylene (PE - various types; Polyethylene terephtalate (PET), high density (HDPE) and low density (LDPE) being the most common), polypropylene (PP), polystyrene (PS) and polyvinylchloride (commonly known as PVC). Their ease of transformation and the large quantities that are produced mean that they are good candidates for recycling.

## 12.3 CURRENT LIMITATIONS TO PLASTIC RECYCLING

The main barriers to recycling plastic are not related to technical issues or processing capacity. The main barriers are related to the economics of recycling plastics. For example:

- Transportation costs -plastics have a high volume to weight ratio making them bulky and therefore expensive to transport and store; and
- Segregation costs - there is a large variety of types of plastic containing different dyes and additives. If these are not separated poor quality recyclate can be produced which limits end uses.

This means it can be cheaper and easier to use new plastic rather than recycled plastic. Also plastic can generally only be recycled once as the quality degrades each time it is reheated. This does not mean plastics should not be recycled. Recycling plastic reduces the amount of waste sent to landfill, fossil fuel use and energy consumption.

Recently there has been a push to produce higher quality recyclate with increased economic value and a greater variety of end uses. This means ensuring that recyclate is not contaminated with other types of plastic.

### 12.3.1Processing & Recycling Opportunities at the RSWDS

For the RSWDS, opportunities identified include those targeted at increasing the resale value of recyclate through sorting, washing, crushing/ bailing, flaking (grinding/ shredding) and potentially pelletisation (i.e. melting, extruding and chipping plastic to make pellets/ granules). Each step from sorting to pelletisation increases product purity and/or increases recyclate density to reduce transport costs to markets in Java or abroad.

If actual products are to be made, then the best opportunities are likely to comprise products that have a use either in the regional waste management system itself (e.g. waste bins or bin liners) or that have a ready local market (e.g. string for use by the agricultural sector) where the production costs can be offset against the costs of importing virgin product to Sulawesi.

## 12.4 SUMMARY OF CURRENT MARKET CONDITIONS

The following provides a brief summary of key current market and waste conditions with figures estimated on a conservative basis -

- The scale of existing scavenger and junk shop operations in Mamminasata are not insignificant at greater than 4,000 people;
- There is an existing plastic flake factory in Maros that exports its product to Surabaya, where as other export destinations for processed and unprocessed plastic wastes are known to be Jakarta and Surabaya with significant export of plastic resins overseas for further processing in specialist chemical plants most notably in China;
- It is estimated a of total of approximately 547,000 tonnes of plastics (13% of the waste stream) will be delivered to the RSWDS from 2012-2027;
- Some 90% or 492,000 tonnes will comprise mixed plastic wastes of LDPE, HDPE, PVC, PP, PS and others, whilst 10% comprise PET, HDPE or PVC bottles;
- It is estimated that 85% of hard plastics/ bottles and 20% of plastic bags/ films or approximately 238,000 tonnes of plastics (43% of the total) can be recovered in the sorting plant;
- The price of virgin plastic is influenced by the price of oil as oil is the principle feedstock for plastic production. As the quality of recovered plastic is typically less than that of virgin plastics, the price of virgin plastic effectively sets the ceiling for the prices paid for recovered plastic;
- Although the prices paid for virgin and waste plastic are known to vary in relation to the price of oil, the percentage differences paid for the various grades of waste plastic remain fairly constant. Thus, the most valuable plastic wastes are (from high to low values) PP, PET, HDPE, PVC and LDPE;
- Processing sorted plastic wastes results in significant increases in recyclate sale value: washing/ flaking by a factor of 2 and pelletisation by a factor of about 3; and
- It is estimated that unprocessed sorted plastics from the RSWDS would fetch 30% of the market prices in China, processed plastics 50%of market value and that any plastic products produced would fetch 1.5 times the value of processed plastic pellets.

## 12.5 ASSESSMENT OF OPPORTUNITIES

The discussion of options focuses on four key differing approaches.

1. Status Quo: The first approach assumes that there is effectively no change in the 'status quo'. That is to say that the RSWDS will, in relation to plastic waste, operate only in a sorting role, with sorted plastic waste bales being sold to the existing industry network of brokers and exporters. This scenario represents ‘baseline’ conditions against which other options involving plastic processing or manufacturing are assessed.

2. Flaking Factory: The second approach assumes that RSWDS will include a plastic flaking factory utilising collected plastic wastes from the sorting plant and/or other sources to produce plastic flakes via washing and grinding/ shredding for sale and/or export.
3. Pelletisation Factory: The third approach assumes that RSWDS will include a plastic pelletisation factory utilising collected plastic wastes from the sorting plant and/or other sources to produce plastic pellets via washing, flaking and extrusion for sale and/or export.
4. Shaped Product Manufacturing Factory: The fourth approach assumes that RSWDS will include a plastic manufacturing factory utilising produced plastic pellets in the production of new shaped plastic products via injection and/or extrusion moulding.

The assessment of options considers the relative costs of implementation & operations, potential impacts (positive & negative) on existing plastic recycling networks, the ability to provide high value (clean & homogenous) plastic recyclate, and, on a conservative basis, potential revenues.

## 12.6 ASSESSMENT OF RELATIVE ADVANTAGES & DISADVANTAGES OF OPTIONS 1 – 4

| Predicted Profits, RoI & Summary of Perceived Advantages & Disadvantages for Options 1 - 4 over the period 2012 - 2027 with 13,725 & 21,354 te/yr Plant Capacity | | | | |
|---|---|---|---|---|
| | Predicted Profit & RoI Above Baseline | | Advantages | Disadvantages |
| | Profit (Rp Billion) | RoI (%) | | |
| 1 | 0 (108.040 Baseline) | NA | No extra CAPEX or OPEX; Uses existing trade network; | Poor quality recyclate (-Revenue); Fluctuating market sales prices; Long distance from source/markets |
| 2 | a) 56.865 | 7.97 | Good quality recyclate (+ Revenue); Uses existing trade network; | Extra CAPEX & OPEX; Fluctuating market sales prices; Long distance from source/markets; Resentment from existing network? |
| | b) 58.797 | -14.23 | | |

| Predicted Profits, RoI & Summary of Perceived Advantages & Disadvantages for Options 1 - 4 over the period 2012 - 2027 with 13,725 & 21,354 te/yr Plant Capacity | | | | |
|---|---|---|---|---|
| | Predicted Profit & RoI Above Baseline | | Advantages | Disadvantages |
| | Profit (Rp Billion) | RoI (%) | | |
| 3 | a) 103.153 | 59.23 | Good quality recyclate (+ Revenue);<br>Uses existing trade network; | Extra CAPEX & OPEX;<br>Fluctuating market sales prices;<br>Long distance from source/markets;<br>Resentment from existing network? |
| | b) 100.777 | 21.85 | | |
| 4 | a) 112.514 | 43.90 | Produces new recycled products;<br>Product sales price less +/- than raw materials;<br>Products could be used to enhance waste collection and expand 3Rs concept;<br>Close proximity of sources/market | Extra CAPEX & OPEX;<br>Unknown/fluctuating market demand for products;<br>Resentment from existing network? |
| | b) 102.665 | 4.04 | | |

The table below presents the predicted profits and return on investment (RoI) of each option over the life of the RSWDS depending on whether a) only the most valuable recovered plastics are processed (LDPE from waste plastic bags, PP, PET & HDPE - plant capacity 13,725 tonnes/year), or b) all recyclable plastics recovered are processed (plant capacity 21,354 tonnes/year), together with a summary of the main advantages and disadvantages of each option.

Given that the price of recycled plastics fluctuates significantly with the market, the profit figures below should be viewed in comparative, rather than absolute terms.

The assessment suggests revenues from sales of unprocessed or processed plastics could significantly subsidise the operation costs of the RSWDS (estimated at Rp 261 Billion in the SAPROF) to 2027.

The RoI figures suggest processing options should focus on higher value plastic products in a plant of lower capacity with pelletisation (Option 3) giving the highest return on investment.

## 12.7 CONCLUSIONS & RECOMMENDATIONS

The report has demonstrated the financial potential to implement any of the options identified. Option 1 will anyway be implemented (without any additional expenditure) with the construction of the RSWDS sorting plant. Hence there would appear to be no urgency to build, equip and operate an additional plastic recycling factory purely on financial grounds.

The decision to make further investments to increase revenues from plastics recycling via processing to flakes or pellets (Options 2 & 3) should be taken on the basis of demonstrating the administration's commitment to the 3Rs concept, retaining more of the value of the recovered materials in Sulawesi and/or a desire to create livelihoods in waste management.

This study also finds that the decision to manufacture recycled plastic products (Option 4) should be taken with consideration of whether such products could be used to enhance waste collection in the region and thus assist in the justification to increase household waste collection 'retribution' fees needed to subsidise the operation of the RSWDS in the long term. Such products could include household waste bins and bin liners as well as products that could be sold to the existing local markets (e.g. string for the agricultural sector).

It is recommended that some additional analysis and consultation be conducted prior to any decision being made about constructing a plastic recycling factory and/or the capacities and capabilities of such a facility. Ares of focus and other recommendations are detailed below –

- Consultation with current informal and formal plastic recycling networks in the area to gauge their opinions about perceived weaknesses and benefits of constructing a plastic recycling factory with particular emphasis placed on how it may impact upon their livelihoods. This will help to ensure an informed and amicable integration with existing networks;
- Scavengers approved to work at the RSWDS and/or other waste sites that the RSWDS may replaced should be offered paid employment in the sorting plant or plastic factory to compensate them for any actual or perceived loss of income;
- Detailed analysis of different types of plastics within existing waste collection and disposal networks. This will help to ensure there is a detailed understanding of the types and quantities of plastic wastes that will be encountered;
- Consideration of what recycled plastic products could assist promotion of the 3Rs concept within communities and/or what recycled plastic products could be used as a component of waste collection and disposal networks (waste sacks from recycled LDPE & waste bins from recycled HDPE). This would help to ensure a guaranteed market for any recycled plastic products;
- In relation to the above point, an assessment should be made as to how much it would cost to purchase products such as waste sacks and bins against the cost of manufacturing a similar product from recycled plastic;

If it were a government run plastics factory, it is probable that players in the existing marketplace may well perceive that the RSWDS is interfering with their livelihoods. As previously stated (and supported by SAPROF recommendations), to counteract such criticisms, ESC recommends that consideration be given to operating the plastics factory as a private business.

# CHAPTER 13: BILL OF QUANTITIES

BOQs for all elements of the DED are provided in Appendix K.

# CHAPTER 14: BID DOCUMENTS

The tasks of the assignment that specifically require preparation of tender documents are the landfill and leachate treatment facilities, the recycling facilities, the support facilities and the heavy equipment. Of these, bid documents for two components (the landfill/leachate treatment and the recycling facilities) have been prepared in ICB format, whilst bid documents for the remaining two components (the support facilities and the heavy equipment) have been prepared in LCB format. All bid documents are presented in Appendix L.

Following the Draft Final Design Presentation in Makassar on 4th November 2010, ECS wrote to IndII to seek confirmation of a number of issues including approval of the preparation of 1 ICB and 2 LCB packages. Copies of this correspondence and related minutes and presentation slides are presented in Appendix M.

# CHAPTER 15: CONCLUSIONS & RECOMMENDATIONS

## 15.1 DESIGN PHILOSOPHY

The objective of the detailed design is to develop an integrated solid waste management facility that provides environmental protection, in particular to surface water, groundwater and air quality, while employing fundamental, cost effective technologies that can be constructed, operated and maintained under the constraints of the current local "state-of-the-practice" that demonstrably works in developing countries.

In developing the concept design options, the various available documents previously prepared for this Site; including: the Pre-Feasibility Study; the Preliminary Design (SAPROF); and the Environmental Impact Assessment (AMDAL); were reviewed with a view to;

- Verifying key data (e.g. waste volumes and design capacity);
- Maximising the efficiency of the design;
- Minimising operation & maintenance costs through simplifying the design; and
- Reducing the potential for breakdown by minimising mechanical equipment.

## 15.2 LANDFILL CAPACITY

Based on the assessment performed herein, the required 5 million cubic metres of airspace volume for 15 years operation indicated in the SAPROF appears to be somewhat conservative; however, it was agreed that it be maintained and that the design of the Landfill Waste Disposal Area be based on this value.

## 15.3 GENERAL SITE DETAILS

Following the movement in site location additional topographical surveys were commissioned to re-survey the entire site. However the site investigation has revealed the general stratigraphy of the site and that the groundwater level is generally between 3m and 5m below the existing ground surface.

## 15.4 SITE LAYOUT & FORMATION

The southern landfill layout option was adopted and has been detailed in the design with the leachate treatment plant (LTP) in the southeast, landfill cells west to east across the middle of the site with sorting/ composting facilities immediately north of the landfill on a cut platform. Provision for future landfill gas (LFG) facilities is provided northeast of the landfill. Offices and supporting facilities are located further to the north, for the comfort of occupants. A landscaped 25m buffer zone and vegetated soil stockpiles will provide a visual screen from residences to the south.

Site earthworks will comprise a 'cut & fill' balance over the landfill life of Stage 1 of approximately 0.7 million $m^3$ inclusive of some 213,000$m^3$ cover material, in a 15m – 20m high stockpile along the southern boundary. Slopes designed to 3 horizontal and 1 vertical ensure slope stability and control soil erosion. Only Stage 1 of the entire landfill project is to be constructed at the outset.

## 15.5 LANDFILL SYSTEMS

The Stage 1 Landfill will be approximately 360 m by 270 m in plan dimensions; sub-divided into four (4) approximately equal Cells of 180m by 135m, and excavated to a depth of not more than 3 m to 4 m below the existing ground surface so as to remain above the anticipated groundwater level and to provide an adequate elevation head to maintain the gravity flow of leachate to the adjacent treatment facilities. The landfill base lining system will comprise of a geosynthetic clay liner (GCL) overlain by a 1.5mm high density polyethylene (HDPE) textured-surfaced geomebrane.

The leachate collection system comprises a rib & spine network of perforated pipes in trenches overlain by a granular layer with vertical rise pipes to further promote air circulation. Leachate flows to a sump and onwards via gravity to buffer storage and the LTP.

LFG is forecast at up to 5,000 million litres/yr in 2030 with up to 3,000 Ml/yr recoverable for future utilisation via a network of extraction wells.

Cells will be filled sequentially to promote surface water runoff with waste placed to an elevation of 71m (Stage 1) with a final cap comprising 5 layers of soil, compacted soil, drainage layer, soil and topsoil with a vegetated surface.

Surface water management in the landfill and around the site is via a series of surface channels designed to cope with at least a 50-year rainfall event, estimated to be of an intensity of approximately 300mm/hour.

Soil erosion and sediment will be controlled through the use of silt fences, retention pond and through reuse of topsoil to establish a vegetative cover over the disturbed areas as soon as practically possible.

Environmental control is facilitated by a network of monitoring wells and an AMDAL checklist included in the report.

## 15.6 LEACHATE TREATMENT

A detailed analysis of leachate generation over the life of the landfill has been undertaken inclusive of all contributions, of which rainfall is the primary and most variable component, indicating a maximum of approximately 1,250$m^3$/day in 2021.

Leachate influent quality has been determined based on samples taken from Tamangapa landfill and available literature for similar landfills in Asia. The LTP is designed based on maximum mass loadings of key pollutants over the landfill life and Indonesian leachate discharge standards.

The *Concept Design Report* evaluated the various treatment processes available and concluded that a biological treatment system would likely be required to meet discharge standards. Detailed evaluation shows this is indeed the case, although and the adopted design minimises mechanical parts to maximise reliability. Consideration is also given to maximising, gravity fed, passive treatment in the event of mechanical failure.

The LTP comprises buffer storage, an anaerobic process to address acetogenic leachate, anoxic/aerobic biological treatment to address lower concentrations of COD/$BOD_{5,}$ and also ammonia removal through nitrification-denitrification. A reed bed is included for final polishing by further removal of suspended solids from the treated wastewater before discharging to the river.

## 15.7 SORTING PLANT

The sorting plant is to sort recyclables such as organics, plastics, paper, metal and glass from incoming waste streams with residues being disposed of within the landfill. The sorting plant (as specified in the SAPROF) is to have a capacity of 1,500 tonnes per day. The Final Design has been developed with a view to maximising efficiency, minimising capital and operating costs whilst ensuring environmental provisions are adequate to meet design criteria.

A flexible staged implementation programme in respect of sorting capacity has been developed to allow decisions to be made in the future as to how to respond to increased waste inputs (increased staff vs increased equipment) and also to allow for any differences in predicted and actual waste inputs. It should be noted that the design drawings have been produced on the basis of the maximum level of mechanical equipment as detailed within ‘Alternative Sorting Plant Development – Stage 3’ in Section 8.2 of the main report.

## 15.8 COMPOSTING PLANT

The composting plant is designed to have two roles – pretreatment of waste prior to disposal within the landfill and the production of compost. The composting plant is required (as specified in the SAPROF) is to have a capacity of 50 tonnes per day. Per the Preliminary Design the plant has been designed with pre fermentation and fermentation areas for composting with a total area of 150m by 56m (8,400$m^2$)

## 15.9 SUPPORTING FACILITIES

Site supporting facilities for the proposed RSWDS consist of both structures (management office, community facility, workshop, weighbridge, auditorium etc) and key services (water & electrical supply and sewage & sanitation). All of these facilities are described in the report and final design drawings presented in the Annexes.

# APPENDIX A - SITE BOUNDARY

# APPENDIX B - SITE LAYOUT

# APPENDIX C - DED DRAWINGS BUNDLE 1 – LANDFILL, SITE ROADS & LANDSCAPING

# APPENDIX D - AMDAL CHECKLIST

# APPENDIX E - LEACHATE QUALITY

# APPENDIX F - DED DRAWINGS BUNDLE 2 - LEACHATE TREATMENT PLANT

# APPENDIX G - DED DRAWINGS BUNDLE 3 – SORTING, COMPOSTING & SUPPORT FACILITIES

# APPENDIX H - ACCESS ROAD

# APPENDIX I - TRAINING MATERIALS

# APPENDIX J - PLASTIC RECYCLING FEASIBILITY REPORT

# APPENDIX K - BOQS

# APPENDIX L - BID DOCUMENTS

# APPENDIX M - DOCUMENTS RELATING TO DRAFT FINAL DESIGN PRESENTATION

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