WATER REUSE AND ENVIRONMENTAL CONSERVATION PROJECT

CONTRACT NO. EDH–I–00–08–00024–00 ORDER NO. 04

AS SAMRA BIOSOLIDS MONOFILL FINAL DESIGN REPORT

January 2015

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AS SAMRA BIOSOLIDS MONOFIL
FINAL DESIGN REPORT
JANUARY 2015

Submitted to:

USAID Jordan

Prepared by:

Engicon/AECOM

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LIST OF ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	Standard Test Method
BS	British Standard
CFR	Code of Federal Regulations
DS	Dry Solids
ET	Evapotranspiration
FA	Feasibility Assessment
GCL	Geosynthetic Clay Liner
GoJ	Government of Jordan
HDPE	High Density Polyethylene
HELP	Hydraulic Evaluation of Landfill Performance
JBC	Jordan Biogas Company
kW	Kilowatt
LandGem	Landfill Gas Emissions Model
LFG	Landfill Gas
LGP	Low Ground Pressure
MCM	Million Cubic Meters
MSW	Municipal Solid Waste
MWI	Ministry of Water and Irrigation
NMOC	Non-Methane Organic Compound
O&M	Operation and Maintenance
PE	Polyethylene
PPE	Personal Protection Equipment
PVC	Polyvinyl Chloride
QA	Quality Assurance
RCRA	Resource Conservation and Recovery Act
RCP	Reinforced Concrete Pipe Casing
ROI	Radius of Influence
SDR	Standard Dimension Ratio
SPT	Standard Penetration Test
USAID	United States Agency for International Development
USD	US Dollars
USEPA	United States Environmental Protection Agency
UU	Unconsolidated - Undrained
WWTP	Waste Water Treatment Plant

1 Introduction

The USAID Water Reuse and Environmental Conservation Project works throughout Jordan in institutional capacity building, pollution prevention for industries, solid waste and wastewater management, and water reuse. The project goal is to protect and conserve scarce resources through regulation, education, and coordination with industry, local communities and the private sector. The project is implemented by AECOM and a team of international and Jordanian partner firms. This five-year project has four primary tasks:

- Task 1 – Institutional and Regulatory Strengthening
- Task 2 – Pollution Prevention and Industrial Water Management
- Task 3 – Disposal Sites Rehabilitation and Feasibility Studies
- Task 4 – Water Reuse for Community Livelihood Enhancement , including biosolids

As part of Task 4, this Design Report presents the detailed design for the As Samra biosolids monofill within the As Samra wastewater treatment facility property boundary.

This Design Report addresses and outlines the technical requirements of the project and demonstrates that the design meets the goals for the project. It has been prepared in general accordance with the Terms of Reference (TOR) – As Samra Sludge Management (treatment, reuse and disposal) Feasibility Study" (AECOM, November 2012), the "Conceptual Design Report" submitted in July 2014 and the "As Samara Biosolids Monofill Feasibility Assessment and Environmental Considerations Report" (AECOM, November 2014). More specifically, the purpose of this report is to:

- Summarize disposal requirements based on projected biosolids production from the As Samra Waste Water Treatment Plant (WWTP);
- Present a discussion of the design strategies for As Samra Biosolids Monofill; and,
- Present design analyses and evaluations, in addition to drawings illustrating the components of the design

This report is divided into sections as follows, with appendices providing supplementary information and data:

- Section 1.0 - Introduction
- Section 2.0 - Authorization
- Section 3.0 - Overall Design Strategy
- Section 4.0 - Capacity Requirements
- Section 5.0 - Geotechnical Investigation
- Section 6.0 - Access Control
- Section 7.0 - Support Facilities/Infrastructure
- Section 8.0 - Subgrade Construction
- Section 9.0 - Global Landfill Stability
- Section 10.0 - Landfill Base Liner System
- Section 11.0 - Leachate Collection and Transmission System
- Section 12.0 - Storm Water Management System
- Section 13.0 - Final Cover System
- Section 14.0 - Landfill Gas Management System
- Section 15.0 - Monitoring
- Section 16.0 - Quantity and Cost Estimates
- Section 17.0 - References

1.1 Site Location

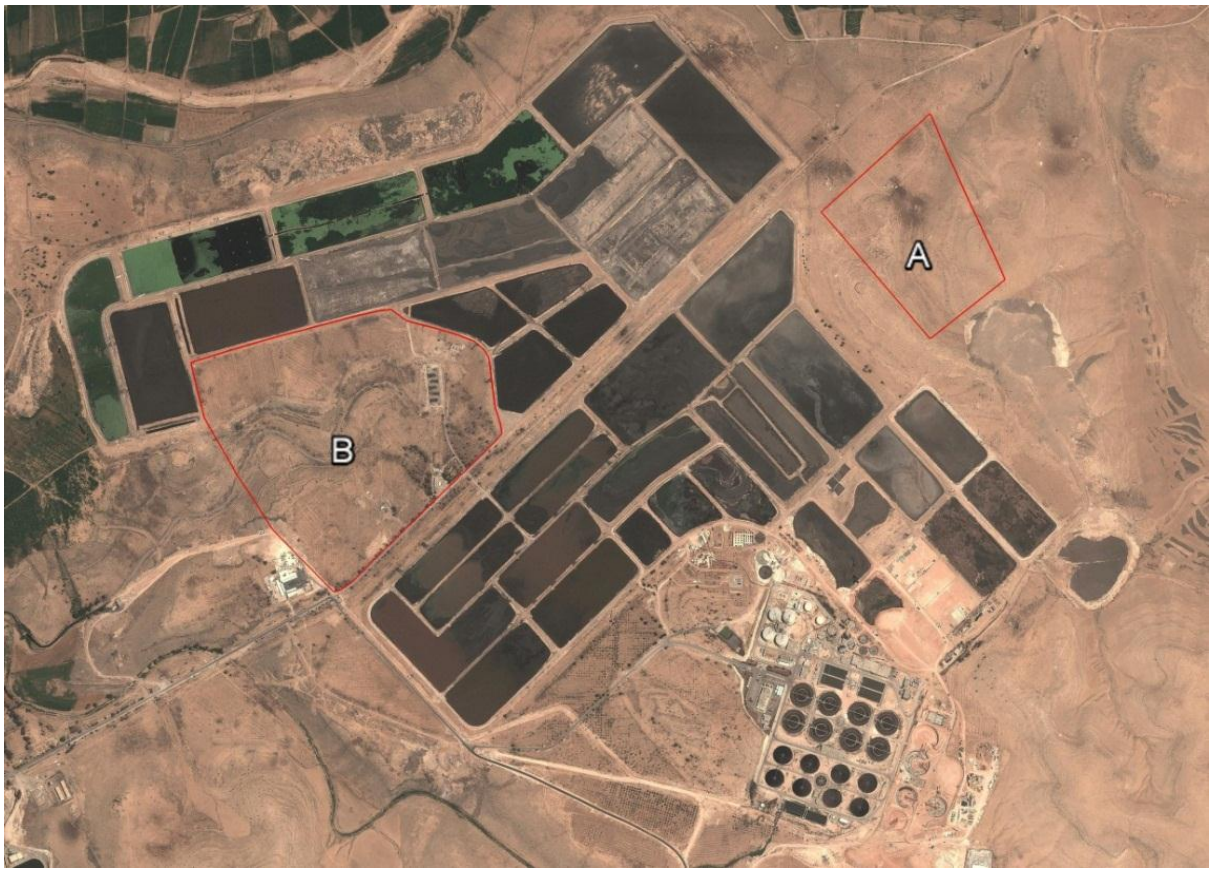
The As Samra Wastewater Treatment Facility (WWTP) services a population of approximately 2.265 million people mainly in Amman and Zarqa, and is located on land owned by the Ministry of Water and Irrigation (MWI) in Al Khirbeh As-Samra area within Al Hashimiyya in Zarqa governorate, 13 km north of Zarqa and 36 km to downtown Amman (ESIA report, 2012) (32°9'15.28"N/36°9'50.38"E).

Figure 1-1: Regional Site Map

Based upon the recommendations within a detailed site selection memorandum dated April 2014, the area designated as "Area B" in Figure 1-2 below was selected as the preferred monofill location and is the subject of this design report. The proposed monofill will be

located to the northwest of the WWTP and the existing lined drying lagoons in an area occupied by a wadi and an abandoned chlorine facility.

Figure 1-2: Site Location



1.2 Site Description

The proposed monofill (Area B as identified within Figure 1-2 above) and its support facilities will encompass approximately 38 hectares of area on barren land that has a serpentine wadi valley initiating from the approximate southeastern edge and sloping generally towards the northeast corner. The land generally slopes northwards from the southern edge of the proposed site towards the wadi and southwards from the northern edge of the proposed site towards the wadi. The steepest slopes (up to approximately 30%) occur in the valley feature incised by the wadi. The proposed site is bound on the north and east by the existing WWTP drying beds and on the south by an existing access road.

Construction of the monofill will require refurbishment/re-use of the former chlorine handling and management building and electrical equipment located along the roadway on the south side of the proposed site. The existing chlorine contact basin and its service pipes and structures near the eastern edge of the proposed site will be removed.

1.3 Climate

The climate of As Samra is that of a dry desert climate, characterized by very hot summers and mild winters. Data from the Al Zarqa Al Jadeida weather station indicate an average maximum temperature of 25.8°C, and average minimum temperature of 13.9°C. Average

monthly rainfall for 2009-2013 was 10.3 mm/month with an average annual rainfall of 123.4 mm/yr.

Table 1-1: Zarqa Mean Maximum Air Temperatures for the years 2009-2013

Station: Zarqa													
Mean Maximum Air Temperature (°C)													
Year	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Yearly**
2009	15.3	16.3	17.9	24.8	28.8	34.0	33.7	34.1	31.0	30.4	20.7	17.2	**25.4**
2010	17.3	18.4	22.3	26.7	30.5	32.8	35.5	37.8	33.9	31.0	26.5	17.9	**27.6**
2011	15.3	15.6	20.1	23.8	28.3	31.4	35.8	33.9	32.2	27.2	18.1	16.3	**24.8**
2012	13.0	14.2	17.3	27.2	29.5	35.2	36.0	34.6	34.0	30.2	22.9	17.0	**25.9**
2013	14.8	17.7	22.8	24.4	31.0	32.3	33.4	34.2	31.7	26.9	23.5	13.7	**25.5**
Average	**15.1**	**16.4**	**20.1**	**25.4**	**29.6**	**33.1**	**34.9**	**34.9**	**32.6**	**29.1**	**22.3**	**16.4**	**25.8**

Table 2-2: Zarqa Mean Minimum Air Temperatures for the years 2009-2013

Station: Zarqa													
Mean Minimum Air Temperature (°C)													
Year	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Year**
2009	4.4	6.2	7.7	11.9	15.7	19.9	21.6	20.4	18.6	18.1	10.1	8.2	**13.6**
2010	7.5	7.9	10.6	13.1	16.6	20.0	21.3	23.4	20.9	18.5	12.5	6.8	**14.9**
2011	6.3	7.4	8.3	11.7	15.5	18.4	22.1	20.8	19.2	15.3	8.1	5.2	**13.2**
2012	5.0	4.9	6.7	13.2	16.7	20.6	22.8	21.9	20.0	17.9	12.9	7.7	**14.2**
2013	5.6	7.9	10.6	12.0	17.3	18.9	20.2	20.9	19.1	13.7	12.3	4.7	**13.6**
Average	**5.8**	**6.9**	**8.8**	**12.4**	**16.4**	**19.6**	**21.6**	**21.5**	**19.6**	**16.7**	**11.2**	**6.5**	**13.9**

Table 3-3: Zarqa Total Rainfall Amounts for the years 2009-2013

Station: Zarqa													
Total Rainfall Amount (mm)													
Year	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Year**
2009	4.3	48.0	17.3	0.6	0.0	0.0	0.0	0.0	0.0	8.3	13.7	26.6	**118.8**
2010	29.8	72.8	1.2	0.0	0.0	0.0	0.0	0.0	0.0	1.5	0.0	26.2	**131.5**
2011	25.8	40.6	5.0	10.9	2.3	0.0	0.0	0.0	0.0	0.0	19.5	9.4	**113.5**
2012	33.6	36.8	32.6	0.0	0.2	0.0	0.0	0.0	0.0	0.0	10.1	14.7	**117.0**
2013	80.5	5.9	0.0	2.0	12.4	0.0	0.0	0.0	0.0	2.2	0.6	32.4	**136.0**
Average	**34.8**	**40.8**	**11.2**	**2.7**	**3.0**	**0.0**	**0.0**	**0.0**	**0.0**	**2.4**	**8.8**	**21.9**	**123.4**

2 Authorization

This Report has been prepared as a sub-task of the USAID Jordan Water Reuse and Environmental Conservation Project (Project) to provide consulting engineering services to the Government of Jordan (GoJ) at specific targets consistent with USAID's Strategic Objective to achieve "Enhanced Integrated Water Resources Management."

Work on the project is authorized under Order Number 4 in accordance with USAID Contract Number EDH-I-00-08-00024-00 for Global Architect-Engineering Infrastructure Services, as issued to AECOM Technology Corporation (AECOM).

3 Overall Design Strategy

The design of the As Samra biosolids monofill has been based upon the projected WWTP biosolids production, the geometric limitations of the recommended siting area, the stability of the biosolids mass, and the intention of providing an environmentally sensitive and protective means for disposal/management of biosolids (i.e. base liner and closure cover systems).

Jordan does not presently have comprehensive regulations specifically relevant to biosolids monofills that apply for this monofill project. Therefore, regulations in the United States have primarily been used as a basis for design on this project. In the United States, Subpart C of Title 40 of the Code of Federal Regulations (40 CFR), Part 503 provides relevant regulations developed by the Environmental Protection Agency (USEPA) for lining biosolids monofills. Should the biosolids produce leachate with unacceptable levels of contaminants (*arsenic, chromium and nickel*), then a base liner containment system with leachate collection and treatment is required per Part 503. For unlined facilities, the owner/operator must either assure that pollutant concentrations of these contaminants do not exceed Part 503 pollutant limits at specific distance intervals from the boundary of the disposal unit to the site property line (see below) or the owner/operator must meet “site specific” limits set by the relevant permitting authority.

Test results for sludge quality relative to arsenic, chromium and nickel at As Samra are presented in the Terms of Reference – As Samra Sludge Management (Treatment Re-use and Disposal) Feasibility Study (AECOM, November 2012) and are summarized as:

- Arsenic < 7.5 mg/kg of dry weight
- Chromium 67 (37-176) mg/kg of dry weight
- Nickel 37 (24-60) mg/kg of dry weight

These reported values are all currently less than the maximum allowed pollutant concentration limits for all distances per Table 1 and Table 2 of Section 503.23. Part 503 also allows for construction of a single liner system in-lieu of further substantiating if there are unacceptable levels of arsenic, chromium and nickel at the boundary of the facility. Therefore it is recommended that a single geomembrane liner with a leachate collection system be installed at the biosolids facility to ensure into the future that potential unacceptable levels of contaminants from biosolids disposal are not encountered at the facility boundary.

It is currently understood that there is no intent by MWI to utilize this facility for disposal of wastes other than biosolids. Therefore, this design report does not include liner upgrades to conform to USEPA Subtitle D requirements that would include a composite geomembrane liner/geocomposite clay liner and a leachate collection system.

Part 503 also necessitates that measures be taken for closure and post-closure care, leachate collection (if the unit is lined), methane monitoring, and public access restrictions. In addition to these measures, managerial requirements similar to those for municipal solid waste (MSW) monofills must also be met. These include requirements for runoff collection, leachate collection and disposal (if the unit is lined), vector control, methane monitoring, groundwater monitoring or certification, public access restrictions, and restrictions for the growing of crops and grazing of animals.

Based on laboratory testing, the permeability of biosolids is anticipated to be very low (on the order of $k = 2.9 \times 10^{-10}$ cm/sec). While this could conceptually "negate" the need for a closure cap as a cap system is in itself designed primarily as "impermeable" to encapsulate waste and limit infiltration, a closure cap has several other beneficial functions (i.e. nuisance control, gas control, stormwater management, aesthetics, etc.). Additionally, biosolids exposed in arid conditions will desiccate and crack. A closure cap system has therefore been included as part of the As Samra biosolids monofill design.

This Design Report for As Samra biosolids monofill addresses the following components:

- Access control for site;
- Design of individual landfill cells with the bottom of each cell sloping towards leachate collection system;
- Design of a base geosynthetic liner to prevent liquids collected from entering subsurface soils and regional groundwater;
- Design of leachate collection, conveyance and storage/treatment system;
- Design of final grading plan to allow surface water drainage;
- Design of final cap and storm water management system; and,
- Design of landfill gas management system.

4 Capacity Requirements

4.1 Design Strategy

The existing WWTP produces biosolids with a solids concentration on the order of 2%. After application of mechanical pressure within a belt filter press to achieve a solids content of approximately 18%, "solar drying" of the caked solids can be implemented to increase the solids content to approximately 30% using evaporation lagoons. Solar drying can be continued for longer periods of time in the evaporation lagoons to increase the solids content to upwards of 50%. Drying from 18% to 50% solids takes approximately 2 to 3 months at the As Samra facility. It is assumed that solar drying to a solids content of no less than 50% will occur at the treatment plant.

The assumed daily solids production, as provided in Sludge Management Plan 2012/2013, is shown below in Table 4-1. Assuming the compacted dry density of the biosolids material (for a solids concentration of 50%) is 0.56 tonnes/m^3 per O'Kelly (2005b), the required monofill capacity for the targeted 20 years of disposal is approximately 2.4 million m^3.

Table 4.1: ***Assumed dry solids production and equivalent compacted biosolids volume***

Year	Dry solids production (tonnes/day)	Required Volume (m^3/day)	Required Volume (m^3/yr)	Target Volume 5yr Increments (m^3)
1	146.6	262	95,617	
2	152.5	272	99,465	
3	156.4	279	102,009	
4	160.2	286	104,488	
5	164.1	293	107,031	508,610
6	168.0	300	109,575	
7	171.9	307	112,119	
8	176.3	315	114,989	
9	180.7	323	117,858	
10	185.1	331	120,728	575,269
11	189.5	338	123,598	
12	193.9	346	126,468	
13	193.9	346	126,468	
14	193.9	346	126,468	
15	193.9	346	126,468	629,470
16	193.9	346	126,468	
17	193.9	346	126,468	
18	193.9	346	126,468	
19	193.9	346	126,468	
20	193.9	346	126,468	632,340
Total				**2,345,689**

4.2 Design Capacity

The geometry and operational space requirements at the site location have a direct influence on the total life expectancy for the facility. Based on the geometric limitations for the proposed monofill footprint, the anticipated maximum excavation depth without blasting, the proposed base liner and closure cap components, and the slope stability calculations that limit the maximum monofill development slopes to no steeper than 6H:1V due to the shear strength of the biosolids, the phased conceptual capacity of the monofill is summarized as:

Table 4-2: *Monofill Design Capacity*

Cell Designation	Gross Capacity (m^3)	Final ET Cover (m^3)	Daily Cover Required (m^3)	Net Biosolids Capacity (m^3)	Cell Life Expectancy (years)	Cumulative Life Expectancy (years)
Cell 1	578,190	13,144	56,505	508,541	5.00	5.00
Cell 2	659,425	35,987	62,344	561,094	4.88	9.88
Cell 3	708,082	19,436	68,865	619,781	4.93	14.81
Cell 4	677,862	48,256	62,961	566,645	4.48	19.29
Cell 5	779,552	155,183	62,437	561,932	4.44	23.73
Totals	3,403,111	272,006	313,112	2,817,993	23.73	

Notes:

1. *Gross Capacity is calculated from top of closure/intermediate grades to top of protective cover and includes daily cover soil.*
2. *Final ET Cover is 1.5 meters thick (1.35 meters of soil and 0.15 meters of gravel).*
3. *Daily Cover is estimated as 10% of the overall capacity (Gross Capacity – Final ET Cover)*
4. *Net Biosolids Capacity is calculated as Gross Capacity – Final ET Cover – Daily Cover Required*
5. *Cell Life expectancy is based on projected biosolids rates from Table 4-1.*

Actual life will vary (i.e. will be lower) depending on the amount of daily cover installed. We note that the first 4 cells will provide approximately 19.3 years of life expectancy and the 20 year target life expectancy is achieved during early filling of Cell 5.

5 Geotechnical Investigation

A geotechnical investigation was conducted at the site by Triple Corporation in order to provide sufficient geotechnical parameters for the design and construction of the proposed project. The purpose of the geotechnical investigation was to determine the subsurface conditions in addition to the physical and chemical properties of the ground materials. More specifically, the investigation aimed firstly to verify through in-situ exploration the soil-rock strata, water table elevation in the boreholes, and quality and availability of water for the project site. The study can be found in Appendix B.

5.1 Field Exploration and Laboratory Testing

A total of 24 boreholes were drilled to approximate depths ranging from 10.0 to 30.0 meters below the ground surface at the site in June 2014. Borehole locations can be seen in Figure No.1 “General Site Plan” of the Geotechnical Report in Appendix B and is presented below as Figure 5-1.

Figure 5-1: General Site Plan (Figure 1 of the Geotechnical Report)

A summary of the borehole identification, location (coordinates), and drilling depth is presented in Table 5-1 below:

Table 5-1: Boreholes and Corresponding Information

Borehole Number	Elevation (m)	Coordinates		Depth (m)
		Northing	Easting	
BM1	563.00	558447.781	420456.934	10
BM2	562.35	558521.156	420550.900	10
BM3	562.78	558483.917	420343.906	10
BM4	562.30	558561.815	420440.263	10
BM5	559.80	558637.308	420537.945	10
BM6	559.90	558715.403	420635.454	10
BM7	558.10	558806.068	420758.046	10
BM8	560.30	558527.365	420235.402	10
BM9	559.80	558604.143	420333.453	30
BM10	556.70	558757.150	420531.851	10
BM11	551.30	558832.212	420627.045	30
BM12	557.18	558930.919	420752.976	10
BM13	544.00	558720.309	420317.000	10
BM14	547.50	558795.222	420418.451	10
BM15	549.50	558872.706	420514.553	10
BM16	554.00	558948.983	420612.425	10
BM17	541.75	558755.591	402202.537	10
BM18	557.25	558835.499	420304.121	10
BM19	555.75	558918.779	420410.594	10
BM20	557.40	558990.980	420505.150	10
BM21	556.80	558798.635	420095.046	10
BM22	559.00	558877.721	420194.763	10
BM23	561.10	558953.222	420290.620	10
BM24	559.30	559029.396	420391.386	10

Geotechnical Testing

Both In-Situ (field) Testing and Laboratory Testing were conducted. These tests were conducted in accordance to the relevant American Society for Testing and Materials (ASTM) and American Association of State Highway and Transportation Officials (AASHTO) standards. Table 5-2 outlines the tests and relevant standards.

Table 5-2: Laboratory Tests Conducted and Corresponding Standards

Test	Standard No.	Tests Performed
Particle Size by Sieve and Hydrometer	ASTM D422	24
Atterberg Limits	ASTM D4318	24
Moisture Content	ASTM D2216	72
Unconsolidated Undrained Compressive Strength	ASTM D2850	3
Unconfined Rock Compressive Strength	ASTM D2938	20
Flexible Wall Permeability	ASTM D5084	3
pH	AASHTO T289	7
Sulfates	AASHTO T290	7
Chlorides	AASHTO T291	7
Resistivity	AASHTO T288	7

Groundwater was not encountered nor were any caves or cavities encountered in any of the boreholes. The general stratigraphy at the proposed biosolids disposal site consists of the following layers in descending order from the ground surface downwards:

- Light brown, moist silty clay with gravel, cobbles of limestone and basalt;
- Alluvial materials composed of light brown silty clay and dark gray basalt;
- Dark gray moderately strong basalt with air voids and vugs (unconfined rock compressive strength results ranged from q_u = 670 kg/cm^2 to q_u = 1302 kg/cm^2, average q_u = 1007 kg/cm^2); and,
- Wadi deposit composed of light brown silty clay with gravel, cobbles of limestone.

A light brown, fractured, moist, very weak chalky marl (unconfined rock compressive strength result q_u = 122 kg/cm^2) was only identified in boring BH-24 at depths from 2 to 6 meters below ground surface.

Corrected blow counts indicated a minimum corrected standard penetration test (SPT) value of 37.5, with corrected SPT values ranging from 37.5 to 69 to depths up to 18.5 meters below ground surface. Results of the rippability study using seismic methods as an assessment of potential presented within the Geotechnical Report indicate that the rippable zones near the ground surface vary from 3 meters thick to 14 meters thick across the site, followed by an approximate 3 meter thick marginally rippable zone, underlain by non-rippable.

The following conservative geotechnical parameters for the existing (foundation) soil have been selected for the evaluation of the landfill based on a review of the data and recommendations presented within the geotechnical study included in Appendix B:

- Unit Weight: γ = 2,040 kg/m^3 (20 kN/m^3)
- Internal Shear Strength: Φ = 36°, c = 0 kPa
- Depth to Groundwater: Not encountered

6 Access Control

The monofill will be accessed via the existing roadway and an access ramp adjacent to the former chlorine handling building. This arrangement will utilize the existing roadway and allow immediate visibility of incoming/egressing monofill traffic to operations personnel stationed at the operations building at the facility entrance.

6.1 Access Controls

Access controls that will be implemented to eliminate unauthorized access to the facility will consist of perimeter fencing, gates, and signage. Administrative measures will consist of the education of authorized personnel on site-specific security and health and safety concerns. Furthermore, a health and safety plan must be developed by the site operators.

6.2 Security and Fencing and Gates

Access to the site will be controlled by installing a security fence (minimum height 2000 mm) with a barbed extension section around the entire site with lockable security gates to prevent unauthorized access. Additional fencing may be installed around interior facilities (blower/flare system, fuel dispensary, etc.) in order to restrict access to specific personnel and to reduce potential theft and vandalism. Regular inspection of boundary, gate(s) and fencing must be conducted and damage immediately repaired.

An adequate number of well-trained staff must be available on-site when the facility is open and the entrance shall be closed and locked during non-operating hours.

6.3 Signage

Based on typical landfill signage used across the United States, a facility sign must be permanently posted at the site entrance stating the name and purpose of the facility, the contact information for the responsible Owner/Operator, and the hours of operation. The sign must also specifically prohibit disposal of wastes other than biosolids received from the As Samra WWTP.

Appropriate signage should also be placed around the landfill perimeter, stating the following:

- Unlawful entry and unauthorized scavenging are prohibited
- No smoking, burning or littering is allowed

7 Support Facilities/Infrastructure

7.1 Facility Roadways and Parking

Crushed stone or gravel surfaced roads (150mm minimum thickness and of sufficient width for two-way vehicle traffic) are required around the monofill perimeter and as access roads on the monofill development surface. Parking areas for landfill management, operators, and visitors will also be provided. Separate parking areas will be provided for landfill personnel and authorized visitors so that an accurate assessment of personnel present at the site can be made.

7.2 Utilities

An existing 6m x 11m pad and 500 KVA transformer served by overhead lines along the existing roadway is located to the southwest of the chlorine operations building, providing power to the chlorine operations building via a 380V buried cable. Convenience (sanitary) facilities within the existing building are routed to a septic tank approximately 12 meters away from the southwest building corner that discharges to drain field trenches at some distance behind the building

A common HDPE leachate forcemain will be constructed within a perimeter utility trench along the north side of the monofill during the construction of each monofill cell to connect each cell to the leachate lagoon. An HDPE gas header will be constructed within the perimeter utility trench around the entire monofill in phases to connect the gas collection system piping to the gas generation/flare facility in the fenced infrastructure area adjacent to the operations building. Electrical conduit/cables will also be located within the utility trench to power the submersible leachate pumps within each cell sump.

7.3 Administration Building

The existing chlorine handling building will be upgraded and retrofitted for use as an Administration Building. The architectural, mechanical and electrical drawings for this work are included within Appendix A.

Chlorination equipment had previously been removed. All other damaged, outdated, unusable or otherwise un-necessary equipment and furnishing will be removed. The interior of the building will be retrofitted to provide an entrance hall, site manager's office, first aid area, three additional staff offices, a meeting/conference room, and separate men's and women's lavatory/shower facilities. Further upgrades will include installation of filing/storage and service rooms as well as a mess area and a kitchen. Covered vehicle parking will be provided. New doors and windows will be installed where indicated on the drawings. The existing 20m long by 9m workshop and storage area will continue to be used as such but will be upgraded and/or repaired. The building will be equipped with external/internal lighting, portable fire extinguishers, and appropriate personal protective equipment (PPE) for monofill personnel (hard hats, high-visibility safety vests, hearing protection, etc.) with sufficient reserves for use by authorized visitors. Connection(s) to the existing water supply and electric utilities will be provided.

7.4 Vehicle Maintenance and Equipment Parking Area

The infrastructure area will be of sufficient size to securely accommodate parked construction/operation vehicles within the fenced facility boundary. A sloped concrete pad of sufficient size to accommodate the largest piece of equipment will be located within the infrastructure area with curbing along the perimeter of the pad. This pad will be utilized to contain potential spills during any required engine work, lubrication, or any fluids transfer (other than vehicle fueling) related to maintenance.

7.5 Scale

A modular steel industrial truck scale of sufficient width, length and load capacity will be located adjacent to and in full unobstructed view of the administration building on a flat area near the proposed facility entrance. Smooth and level approaches (3 meters, minimum) will be provided at each end of the scale. It is recommended that the scale, including but not limited to scale modules, load cells, load cell interconnection cables, grounding, all other electrical work and telemetry, accessories be installed with support from the selected scale manufacturer or equipment vendor.

7.6 Fuel Dispensary

An above-ground equipment fuel dispensary (storage tank, manual or electrically operated fuel transfer pump, filling hoses) will be installed within the securely fenced area for use by operations equipment and on-site vehicles. The storage tank will be located within a secondary containment berm or tank to contain potential leaks.

7.7 Leachate Storage/Evaporation Lagoon

A lined leachate evaporation lagoon (equivalent liner system to monofill disposal cells) of sufficient capacity to store the design leachate production volume with sufficient freeboard to contain precipitation will be constructed. The lagoon will receive pumped leachate from each disposal cell and will be located along the northern edge of the monofill where collection sumps are oriented to minimize pumping/piping requirements. The 2-meter-deep lagoon has been designed with approximately 3,613 cubic meters (3,613,000 liters) of storage capacity from its invert to its crest, which exceeds the minimum storage requirements evaluated within the HELP Model leachate generation calculations and the storage capacity required for the ½ of the largest open cell (assumes a temporary rain flap constructed within Cell 1) in the event a large storm event occurs when the cell is initially opened. The lagoon will receive pumped leachate from each disposal cell through the HDPE force main.

For the As Samra site, only passive/natural evaporation from exposure of the contained water surface to solar radiation and wind is proposed. No additional enhanced evaporation proposed by hydraulic/mechanical means such as sprinklers, misters, or aerators is proposed. For added conservatism, evaporation of stored water *has not* been accounted for in leachate lagoon sizing.

The leachate lagoon will be lined with a base liner containment system anchored at the lagoon crest. The liner system will consist of the following components in ascending order over the excavation/subgrade:

- 150 mm (minimum) of compacted select fill/liner cushion soil material; and
- 1.5 mm (60mil) textured HDPE geomembrane liner.

7.8 Stormwater Management Basin

One (1) lined stormwater management basin will be constructed near the northwest corner of the monofill to manage the maximum anticipated runoff from the monofill development area phases as well as runon from limited adjacent areas. Overflow from the basin will be re-directed through a trapezoidal weir back into the existing wadi. For the As Samra site, only passive/natural evaporation from exposure of the contained water surface to solar radiation and wind is proposed. No additional enhanced evaporation proposed by hydraulic/mechanical means such as sprinklers, misters, or aerators is proposed. For added conservatism, evaporation of stored water *has not* been accounted for in stormwater basin sizing. The basin storage capacity from its invert (elevation 537.5) to its crest (elevation 541.0) has been designed as 6,465 cubic meters (6,465,000 liters).

The basin may be equipped with a submersible pump assembly or suction pump connected to a truck load-out facility or facilities so that stored water may be used for landfill operations activities (i.e. dust control, fire protection, etc.). To facilitate storage and retention of water, the stormwater management basin will be lined with a base liner containment system anchored at the basin crest to facilitate retention of water consisting of the following components in ascending order over the excavation/subgrade:

- 150 mm (minimum) of compacted select fill/liner cushion soil material
- GSE "Bentoliner CNSL GCL" (Geosynthetic Clay Liner with a polypropylene geofilm adhered to the upper surface), or approved equivalent
- 300 mm (minimum) thick native soil cover layer with 25 mm maximum particle size

7.9 Landfill Gas Management Facility

A 1200-mm-diameter HDPE landfill gas condensate knockout will be located at the low point of the HDPE landfill gas header as it enters the infrastructure area. Condensate collected within the knockout will be drained via a 110-mm-diameter HDPE drain pipe to a buried condensate holding tank. Condensate will be periodically pumped out of the storage tank for off-site disposal. After the condensate management knockout, the landfill gas header will be routed to a blower/flare station for destruction of landfill gas in the infrastructure area adjacent to the site entrance. Calculations within Appendix F indicate that a blower (estimated differential pressure across the blower of 27 inches or 686 mm W.C.) and flare rated for a maximum capacity of 6,590 scfm (assuming 3,295 scfm collection divided by 50% collection efficiency) are appropriate for this project, with step down/ turndown capabilities for lower flows.

A small monofill gas generation power facility will be constructed with a backup utility flare station in the monofill infrastructure area adjacent to the operations building. Based on the monofill gas generation model (LandGEM) in Appendix F, the maximum annual gas collection rate was estimated as 3,295 scfm. As above, landfill gas generation based upon 50% collection efficiency would indicate a maximum actual production of 6,590 scfm. The maximum annual generating capacity during the active life of the monofill was estimated as 4,800 kW (42,048,000 kW hr). Individual 2,000 KVA generators will be acquired in increments 3 or 4 years apart and phase in and out of operation to meet the variable capacity required over the project life. Engines would be located on a reinforced concrete foundation pad or pads within the fenced infrastructure area approximately adjacent to the

existing transformer. If sale and conveyance of treated or untreated gas to off-site end users is pursued, gas compressors would also be acquired and phased into and out of the gas management infrastructure to meet the variable capacity required over the project life.

8 Subgrade Construction

8.1 Design Strategy

Each phase (or cell) of development will require excavation and/or fill from the design top of subgrade elevations (cell floor and perimeter berm) to the existing topographic elevations. Cut and fill slopes will be no steeper than 3H:1V. Drawing 3 in Appendix A depicts the base grade preparation (top of subgrade prior to installation of liner cushion layer) for the five (5) disposal areas (cells).

8.2 Soil Balance

Drawing 4 in Appendix A depicts a cut/fill isopach indicating the depth of excavation cut and height of fill required to form the landfill subgrade elevations in a grid format. Surplus excavated soils will be loaded and hauled to stockpile area(s) for future use as structural fill, general fill for berms, daily cover, intermediate cover, and potentially evapotranspiration (ET) cap soil. Table 8-1 summarizes the site soil balance based on the design grading plans as presented on this drawing and the monofill phasing plans cut/fill quantities provided in Appendix H.

Table 8.1: ***Monofill Subgrade Excavation/Fill Quantities***

Cell Designation	Gross Excavation (m^3)	Gross Fill (m^3)	Surplus (Deficit) (m^3)
Cell 1	359,302	30,245	329,057
Cell 2	101,639	2,520	99,119
Cell 3	182,607	12,020	170,586
Cell 4	114,586	15,209	99,377
Cell 5	94,540	25,431	69,109
Totals	852,516	85,366	767,151

9 Global Landfill Stability

9.1 Landfill Stability

Four (4) final slope stability cross-section locations were analyzed to determine the stability of the biosolids monofill at its final closure conditions. The desired factors of safety are based on the loading condition and normal engineering practice. For final build out conditions, a safety factor of 1.5 or greater is generally considered acceptable for static analyses.

For the final build out condition and a 500-year return period ground motion, a maximum estimated seismic slope displacement of 150 mm for failure through the geosynthetic liner system and 300 mm for failure not through the geosynthetic liner system is generally considered normal engineering practice for landfills.

Four (4) additional cross-sections were cut to evaluate the stability of the biosolids monofill during construction (i.e., interim stability). For interim conditions, a safety factor criterion of 1.5 or greater was used due to the duration of the expected interim condition (5+ years). Seismic loading was not considered for interim conditions.

9.1.1 Geotechnical Parameters

Foundation Soils / Engineered Fill /ET Cover Soil
The following geotechnical parameters for the existing (foundation) soil, which also may be used as engineered fill, have been selected for the evaluation of the monofill based on a review of the data and recommendations presented within the geotechnical study included in Appendix B:

- Internal Shear Strength: $\Phi = 36°$, c = 0 kPa
- Depth to Groundwater: Not encountered

For cover design feasibility, a potential ET cover was assumed to have the following geotechnical parameters:

- Internal Shear Strength: $\Phi = 30°$, c = 0 kPa

Biosolids
Unit weight and drained and undrained shear strength parameters were assigned to the biosolids based on the results of laboratory shear testing on actual dried sludge from the existing As Samra WWTP. These results are presented and discussed within Appendix C.
A summary of the unconfined unconsolidated (UU) triaxial shear test results is as follows:

Table 9-1: *As Samra WWTP Biosolids UU Test Results*

Test Identification	Test Date	Solids Content (%)	Water Content (%)	Unit Weight (kN/m^3)	Confining Stress (kPa)	Shear Strength, Cohesion (kPa)
UU-1	4/10/14	50	100.1	5.61	75	42.3
UU-3	4/10/14	50	101.3	5.58	300	46.4
UU-4	5/06/14	50	99.5	5.61	150	40.7
UU-5	5/22/14	55	80.1	6.12	150	45.1
UU-6	5/22/14	60	64.1	6.28	150	66.2
UU-7A	5/22/14	62.5	57.2	6.36	150	85.9

For undrained slope stability analysis, the design assumed that biosolids will be installed at approximately 50% solids content (i.e., 100% water content) with internal shear strength of biosolids represented by Su = 43 kPa, which corresponds to the average shear strength reported for the UU tests at about 100% water content.

Effects of consolidation, wetting, long-term drainage, and stress path were evaluated using consolidated isotropic undrained triaxial compression (CIUC), consolidated isotropic drained triaxial compression (CIDC), consolidated isotropic undrained triaxial extension (CIUE), and consolidated undrained direct simple shear (DSS) tests. These tests were primarily conducted at target compacted moisture contents of 100%, with some at 60% moisture content. These results, along with detailed interpretation, are presented within Appendix C. A summary of these test results is as follows:

Table 9-2: *WWTP Biosolids Consolidated Triaxial and Direct Shear Test Results*

Test Identification	Test Date	Solids Content (%)	Water Content (%)	Unit Weight (kN/m^3)	Confining Stress (kPa)	Shear Strength, Cohesion (kPa)
CU-6-1	5/31/14	50	103.5	5.6	75	20.5
CU-6-2	5/31/14	50	101.8	5.7	150	25.9
CU-6-3A	6/05/14	50	103.5	5.6	300	20.5
CU-1a	4/9/14	50	100.1	5.6	75	23.1
CU-5	5/01/14	50	100.1	5.5	150	20.1
CU-4	5/01/14	50	100.1	5.5	300	23.6
CD-1	5/04/14	50	100.1	5.4	150	24.5
CU-EX-2	5/31/14	50	103.5	5.6	150	25.0
CU-EX-1	5/4/14	50	100.1	5.4	150	27.5
DSS-1	5/04/14	50	100.1	5.7	150	39.0
DSS-6	7/03/14	50	106.7	5.5	75	31.7
DSS-3	6/03/14	50	103.5	5.6	150	40.3
DSS-4	6/12/14	50	103.5	5.6	300	31.4
DSS-8	7/08/14	62.5	66.8	6.1	75	21.2
DSS-5	7/03/14	62.5	62.8	6.2	150	18.6
DSS-7	7/16/14	62.5	66.0	6.1	300	43.3

For drained (i.e., long-term condition after consolidation) slope stability analysis, the internal shear strength of the biosolids is represented with effective-stress shear strength parameters of Φ' = 13° and c' = 6 kPa

Geosynthetic Components

The base liner system was assumed to act as a single material. The critical interface was determined by comparing the residual strength of the three geosynthetic interfaces (from top to bottom): granular soil and nonwoven needle punched geotextile (NW-NP-GT); NW-NP-GT and textured high density polyethylene (HDPE-T); and HDPE-T and granular soil. These shear strengths were estimated based upon data presented in Koerner and Narejo (2005). The critical residual interface for final build out stability (confining stresses greater than 100 kPa) was determined to be the NW-NP-GT and HDPE-T interface. The critical residual interface was estimated to have a typical residual shear strength represented by a drained friction angle of 17 degrees with no adhesion. The critical peak interface for temporary liner stability (confining stresses less than 20 kPa) was determined to be the NW-NP-GT and granular soil interface. The critical peak interface was estimated to be 33 degrees and no adhesion.

Water Table

According to the site investigation report, no groundwater was encountered in any of the borings at the drilled depths during or after completion of the drilling activities. Therefore, no groundwater piezometric surface was modeled within the foundation soils. As the monofill will include a drainage layer and leachate collection piping to limit head above the base liner, no water surfaces (leachate) have been evaluated within the biosolids waste mass.

Seismic

The site seismic parameters were based on a 500-yr return period ground motion with a controlling magnitude of Mw 7.5, a peak ground acceleration of 0.13g, a short-period spectral acceleration of 0.325g at 0.2 seconds, and a spectral acceleration of 0.13g at 1 second. A shear wave velocity of 200 m/s was used for biosolids to provide a conservative estimate for seismic displacement. Undrained biosolids shear strength was used during seismic stability calculations due to the short-term nature of seismic loading.

Summary of Parameters

Table 9-3 summarizes the parameters selected for the foundation/berm/ET cover soils, composite liner, and biosolids.

Table 9-3: Geotechnical Parameters

Soil Type	Total Unit Weight (kN/m^3)	Undrained Strength "Su" (kPa)	Drained Strength Cohesion "c" (kPa)	Drained Friction Angle "Φ" (degrees)
On Site Soil	20	NA	0	36
ET Cover	20	NA	0	30
Base Liner, Residual	NA	NA	0	17
Base Liner, Peak	NA	NA	0	33
Biosolids	11	43	6	13

9.1.2 Global Landfill Stability

The stability of the landfill at final build out was evaluated to determine the factors of safety considering the foundation soil, berms, base liner system, and biosolids waste mass. The cross-sections analyzed were selected based on the combination of subgrade excavation depth, perimeter berm fill height, biosolids thickness, and slope angle at the final build out configuration which result in the most critical cross-sections in terms of stability. Minimum factor of safety values of 1.5 for static condition and maximum estimated seismic displacement of 15 cm for a failure surface through the liner and 30 cm otherwise were evaluated.

Slope/W was used to analyze each cross-section to determine the critical slip surface (those with the lowest factors of safety). Estimated seismic displacements were evaluated by applying a pseudo-static horizontal acceleration to the slide mass to achieve a factor of safety of 1.0 to determine the yield acceleration for a given slope. The Bray and Travasarou (2007) method was used to estimate the expected seismic slope displacement for a combination of yield acceleration (k_y), slope failure height, and the site seismic parameters.

The results of the stability analyses are provided in Appendix C and are summarized in Table 9-4.

Table 9-4: *Landfill Slope Stability at Final Build-out Results*

Stability Cross Section	Static Factor of Safety, Undrained	Static Factor of Safety, Drained	Estimated Seismic Displacement (cm)
A-A' left side	2.2	1.9	<1 cm
A-A' right side	1.9	2.0	<1 cm
B-B' left side	2.0	1.9	<1 cm
C-C' left side	1.5	1.8	1.4 cm
D-D' left side	1.7	2.0	<1 cm
D-D' right side	2.3	2.0	<1 cm

Based on the analyses, the minimum static factor of safety (1.5) and maximum estimated seismic displacement of 15 cm for a failure surface through the liner and 30 cm otherwise are achieved for each of the cross-sections and conditions analyzed.

9.1.3 Minimum Base Liner Strength

The residual interface shear strength of the base liner system was varied (i.e., an iterative process) for the critical section C-C' (see Section 9.1.2) to determine the minimum shear strength needed to achieve the required factor of safety for static conditions and seismic displacement, as described above. The minimum residual friction angle required for the base liner system was calculated to be 14 degrees. This is compared to the estimated shear strength of the base liner system of 17 degrees.

9.1.4 Temporary Liner Stability

The minimum peak interface shear strength for placement of protective cover along the entire 3:1 perimeter berm slope is represented by a friction angle of approximately 28.1° as indicate within the "liner veneer stability calculations" in Appendix C.

9.1.5 Interim Stability

Four additional cross-sections were cut to evaluate the stability of the biosolids monofill during construction (i.e., interim stability). The proposed construction will be conducted in 5 stages (4 intermediate stages, and 1 final stage). Cross section locations were chosen for each four intermediate stages of construction based upon the likely critical slope stability sections. Each cross section was analyzed for both undrained and drained biosolids shear strength parameters. For interim conditions, a safety factor criterion of 1.5 or greater was used due to the duration of the expected interim condition (5+ years). Seismic loading was not considered for interim conditions

The results of the stability analyses are provided in Appendix C and are summarized in Table 9-5.

Table 9-5: *Landfill Slope Stability at Interim Conditions Results*

Stability Cross Section	Static Factor of Safety, Undrained	Static Factor of Safety, Drained
1-1'	2.2	1.8
2-2'	1.9	1.7
3-4'	1.8	1.6
4-4'	1.7	1.6

Based on the analyses, the minimum static factor of safety (1.5) is achieved for each of the cross-sections and conditions analyzed. The sequence of biosolids placement within each cell was assumed to be first placed at the lowest elevation of the cell with biosolids at higher elevations placed subsequently in order to remain stable.

9.1.6 Final Cover Stability

A feasibility evaluation for an evapotranspiration (ET) cover was conducted. Stability of the cover under three conditions was assessed: (1) failure at the cover/biosolids interface assuming the interface strength is equal to the biosolids undrained shear strength; (2) failure at the cover/biosolids interface assuming the interface strength is equal to the biosolids drained shear strength; (3) failure through the cover itself, assuming a friction angle of 30° and no cohesion. The resulting static factors of safety were greater than 1.5 for each analysis. Additional discussion and results for these evaluations are included in Appendix C.

9.1.7 Liquefaction Evaluation

Liquefaction occurs when saturated coarse grained soils are subjected to cyclic loading induced by an earthquake. Based on the lack of groundwater at the monofill site, liquefaction is not expected. The liquefaction evaluation is provided in Appendix C.

9.1.8 Construction Quality Assurance

The minimum residual friction angle of 14 degrees calculated for final build out stability refers to the interface strength that may be compared to future lab testing on site/project-specific samples. The minimum peak friction angle of 28.1 degrees calculated for temporary liner stability refers to the interface strength that may be compared to future lab testing on site/project-specific samples. These requirements for liner shear strength are provided in **Table 9-6**.

Interface testing for all soil-to-geosynthetic and geosynthetic-to-geosynthetic interfaces should be conducted prior to construction to verify that the minimum shear strength is met. Different combinations of adhesion and friction angle can also meet the shear strength requirements, as approved by the Engineer.

Table 9-6: ***Minimum Required Liner Shear Strength***

Type	Drained Strength Adhesion c_a' (kPa)	Drained Friction Angle Φ' (degrees)
Peak	0	28.1
Residual	0	14

9.1.9 Monitoring, Instrumentation, and Operations

As discussed in Appendix C and while As Samra WWTP biosolids have been studied, there is uncertainty in the shear strength of the biosolids due to the lack of case histories of field shear strengths of similar biosolids, construction of similar 30m high monofills, and the complicated time, biological, gas generation, and moisture sensitive properties of the biosolids. This risk can be mitigated by placing the biosolids drier than 100% water content which should significantly increase the undrained and drained shear strengths of the compacted biosolids.

However, if that is not possible, we recommend geotechnical instrumentation and regular geotechnical monitoring of the monofill. Geotechnical instrumentation should be used to monitor the field effectiveness of the biogas extraction system to prevent the potential buildup of gas pressure within the monofill and the effectiveness of the surface water control systems to keep the biosolids from absorbing an appreciable amount of moisture. If cracking, bulging, or other signs of slope distress are observed during geotechnical monitoring, placement of the biosolids should be stopped and the stability of the monofill re-

evaluated. The geotechnical instrumentation and monitoring programs should be conducted by a qualified geotechnical engineer.

Placement of the biosolids should be carefully monitored for moisture and compaction to reduce the risk of slope instability. Monofill drainage should be designed to control surface water to limit infiltration of rainwater into the monofill to limit wetting of the biosolids in order to maintain the monofill shear strength. If biosolids are wetted during a rain event, biosolids placement should not continue in that area until the biosolids have been dried and re-compacted. Placement of biosolids during rain events should be avoided. During construction of each cell, the biosolids should be first placed at the lowest elevation of the cell with biosolids at higher elevations placed subsequently in order to meet stability requirements.

Gas production within the biosolids has been identified as a trigger of slope instability in biosolids failure case histories. The gas collection system should be properly maintained and operated to ensure that gas does not build up in the landfill and cause slope instability. This recommendation is consistent with recommendations provided in the literature for addressing this type of instability.

10 Landfill Base Liner System

10.1 Design Strategy

As indicated above, a 60-mil (1.52 mm) textured high density polyethylene (HDPE) geomembrane liner (minimum of 60-mil) is proposed to prevent leachate from the biosolids facility from impacting the groundwater. The liner will be overlain with a protective geotextile and a protective cover/leachate collection layer designed to maintain leachate head at <= 300mm.

10.2 Description of Base Liner System

The proposed As Samra monofill liner system will consist of the following components in ascending order over the excavation/subgrade:

- A 150 mm (minimum) of compacted select fill/liner cushion soil material;
- A 1.5 mm (60mil) textured HDPE geomembrane liner;
- A 540 (minimum) g/m^2 nonwoven geotextile cushion; and,
- A 600 mm (minimum) thick layer consisting of 300 mm protective cover aggregate overlain by 300 mm of filter sand.

An anchor trench will be constructed along the perimeter of the proposed liner area as a means of securing the liner in place.

10.3 Material Evaluation and Selection

10.3.1 Select Fill / Liner Cushion

After the facility subgrade (excavation/fill) grades have been constructed, a 150 mm (minimum) thick layer of select fill will be installed as a liner cushion layer, protecting the underside of the liner system from potential protrusions or irregularities near the surface of the subgrade. This sub-cushion layer will also provide a smooth surface for installation of the subsequent, overlying geosynthetics, reducing the possibility of puncturing the geosynthetics from above during installation. The soil will be comprised of clean, locally available soil, free of potentially deleterious material, with a maximum particle size of 19 mm (3/4 inch).

10.3.2 High Density Polyethylene (HDPE) Geomembrane

HDPE is a chemically inert polymer used to manufacture geomembranes for waste containment applications. The inertness of HDPE is attributable to its molecular structure and morphology. HDPE has a simple molecular structure containing a six-member repeating unit consisting of two carbon and four hydrogen atoms. The hydrogen atoms are covalently bonded to the carbon atoms. Each carbon-hydrogen bond is equivalent and has the same bond energy. Most degenerative reactions in hydrocarbons occur through displacement of a hydrogen atom. However, due to covalent bonding, hydrogen atoms on the HDPE repeating unit are stable, and HDPE is not reactive under normal conditions.

Manufacturers' product compatibility testing shows that HDPE is unaffected by typical municipal solid waste leachate and is resistant to most compounds including inorganic acids, organic acids, volatile organics, petroleum based products, and poly-chlorinated biphenyls. HDPE geomembranes are widely used throughout the United States in the solid waste industry because they not only have acceptable physical properties, such as tensile strength,

strain characteristics, and chemical compatibility, but also are relatively easy to deploy, seam, and test for defects. For these reasons, HDPE geomembranes are considered the industry standard for the geomembrane component in a landfill composite liner.

Manufacturers' data show little variation in HDPE physical properties over a range of temperatures from -40°F (-40°C) to 180°F (82.2 °C). Therefore, HDPE should not be affected by temperature variations. During construction, any loose ends or edges of the HDPE geomembrane will be secured in backfilled anchor trenches, tack welded, or weighted with sand bags to prevent wind-induced movement.

HDPE is resistant to ultraviolet degradation by the addition of carbon black and anti-oxidants during the manufacturing process. Long-term durability tests conducted by geomembrane manufacturers indicate that no surface cracks were observed even under significantly harsher conditions than those which occur in a landfill. In addition to its inherent resistance to ultraviolet degradation, the HDPE geomembrane will be protected by the geotextile cushion placed directly above it.

10.3.3 Nonwoven Geotextile

A nonwoven geotextile will be placed over the HDPE geomembrane to serve as a cushion against the overlying granular drainage layer in the base, trench, and sump liner systems. The burst resistance, grab tensile strength, and puncture resistance of the geotextile cushion were evaluated for loaded construction vehicles and the maximum anticipated landfill waste load to assure that the geotextile will provide adequate protection for the underlying HDPE geomembrane. The analyses were performed for gradations equivalent to AASHTO No.8 and AASHTO No.57 materials, which are common in protective cover / leachate collection layer installations in the United States. The results of the analyses are provided in Appendix C and are summarized in Table 10-1.

Table 10-1: Minimum Nonwoven Geotextile Specifications

Drainage Layer Material	Required Burst Resistance	Required Grab Tensile Strength	Required Puncture Strength	Minimum Mass per Unit Area
Sub-Rounded AASHTO No.8	31.5 psi	4.2 lbs	11.5 lbs	7 g/m^2
Sub-Angular AASHTO No.8	31.5 psi	4.2 lbs	11.5 lbs	14 g/m^2
Sub-Rounded AASHTO No.57	42.0 psi	7.4 lbs	15.4 lbs	99 g/m^2
Sub-Angular AASHTO No.57	42.0 psi	7.4 lbs	15.4 lbs	198 g/m^2

To allow flexibility in selection of the drainage layer during construction and to assure protection of the liner, a 540 g/m^2 (minimum) nonwoven geotextile will be installed.

10.3.4 Protective Cover / Granular Drainage Layer

To provide a highly permeable and transmissive protective cover layer for the liner system, a 600 mm thick layer, consisting of 300 mm protective cover aggregate overlain by 300 mm of sand, will be installed over the protective geotextile. The sand layer will serve as a separation and filter layer between the biosolids material and the porous protective cover aggregate. The protective cover aggregate will be sub-rounded to sub-angular aggregate (generally with a gradation equivalent to AASHTO No. 8 or AASHTO No. 57).

During installation of the protective cover, temporary access roads with a minimum of 1.0 meter of aggregate will be installed in the disposal cell to maintain a protective cushion between non-low ground pressure (LGP) equipment and the liner system. Non-LGP equipment (> 5 psi contact pressure) will only be allowed to operate on these temporary

access roads within the disposal cell. After all the protective cover aggregate and filter sand is delivered to the disposal cell, the temporary roads are removed (spread out) and the 600 mm thick sand/protective cover layer is completed

10.4 Settlement

Soil settlement is a result of the combination of elastic (or instantaneous) settlement of granular soils and primary consolidation and secondary consolidation of fine-grained soils. Settlement due to secondary consolidation of fine-grained soils is typically very small and is typically neglected.

The general stratigraphy encountered within the twenty four (24) borings consists of the following layers, generally in descending order from the ground surface downwards:

- Light brown, moist silty clay with gravel, cobbles of limestone and basalt;
- Alluvial materials composed of light brown silty clay and dark gray basalt;
- Dark gray moderately strong basalt with air voids and vugs (unconfined rock compressive strength results ranged from q_u = 670 kg/cm^2 to q_u = 1302 kg/cm^2, average q_u = 1007 kg/cm^2); and,
- Wadi deposit composed of light brown silty clay with gravel, cobbles of limestone.

Corrected blow counts indicated a minimum corrected SPT value of 37.5, with corrected SPT values ranging from 37.5 to 69 to depths up to 18.5 meters below ground surface. Results of the rippability study using seismic methods as an assessment of potential presented within the Geotechnical Report indicate that the rippable zones near the ground surface vary from 3 meters thick to 14 meters thick across the site, followed by an approximate 3 meter thick marginally rippable zone, underlain by non-rippable material.

It is anticipated that most of the settlement along the base of the monofill will occur during construction due to re-orientation of coarse-grained soils and reduction in void ratio. The maximum anticipated load for the final build out of the landfill (including a factor of safety of 1.5) has been estimated within the pipe loading calculations in Appendix C as 11,030 kg/m^2 (or 15.7 psi). As this value is much less than typical maximum heavy equipment wheel loads (70,307 kg/m^2 or 100 psi), settlement due to loading will primarily occur during cell construction. Therefore, detailed settlement calculations have not been performed.

10.5 Veneer Stability

Stability of the proposed liner system on the side slope was analyzed to represent short-term conditions during placement of the granular protective cover / leachate collection layer. The analysis assumes 0.600 meters of stone/aggregate/sand on a 3H:1V side slope. A transient load is applied by typical construction equipment (Caterpillar D6N LGP dozer). The minimum acceptable interface shear strength for the liner on the side slopes must maintain a minimum factor of safety of 1.3, since this represents a short term condition.

For stability of the liner on the <u>entire</u> side slope during placement of the 300 mm thick stone layer (to be overlain with 300 mm sand filter layer) with a LGP dozer, a minimum <u>peak</u> interface friction angle of 28.1^o is required (worst case evaluation for Cell 5). Protective cover should also be staged up side slopes in shorter slope length intervals as determined

by the Engineer if a peak interface friction angle of 28.1^{o} is not achieved during interface testing of the geosynthetics prior to installation. The maximum stage length should be computed by the engineer based upon actual interface friction testing results as required.

Veneer Stability calculations are included in Appendix C.

10.6 Specifications and Quality Assurance

The specifications for each of the geosynthetic liner system components are included within the "Quality Assurance Manual for Base Liner System Installation at As Samra Waste Management and Landfill" in Appendix G.

11 Leachate Collection and Transmission System

11.1 Design Strategy

EPA regulations (Subpart D) require a leachate collection system that is designed and constructed to maintain less than 300 mm depth of leachate above the liner. The proposed design for leachate collection is perforated PVC pipes incised into the protective cover layer which will be spaced to maintain leachate head of less than 300 mm. The leachate collection pipes within each cell will drain to a sump near the perimeter, where the collected leachate will be pumped out to an onsite leachate evaporation lagoon. The landfill base grade slopes will be 1% (minimum) along the leachate collection pipes and 2% (minimum) perpendicular to the pipes.

11.2 Description of Leachate Collection System

The proposed design for the leachate collection system will consist of perforated pipes incised into the protective cover / leachate collection layer at regular intervals and connected to a common central trunk line draining to a sump near the perimeter of the cell, from which the collected leachate will be pumped out and stored/treated. While actual grades may be steeper, the landfill base grade slopes will be a minimum 1% along the leachate collection pipes and a minimum of 2% perpendicular to the pipes. The maximum length for cleaning out leachate pipes will be 365 meters (1,200 feet), which is the limit of most modern jetting equipment.

The leachate collection pipes will be 150 mm diameter perforated SCH40 PVC lateral pipes (or an equivalent HDPE pipe as approved by the engineer) connected to a 225 mm diameter perforated trunk line along the centerline of the cell. Perforations will be drilled along the lower half of the pipe at 60° angles from the perpendicular. The holes will be alternately staggered and will have a diameter of 15 mm unless alternate perforation size is required by the engineer for compatibility with the proposed coarse aggregate surrounding the pipes.

11.3 HELP Model Analysis, Leachate Head, and Pipe Spacing

HELP Model leachate generation calculations are included within Appendix D. The model has been used to evaluate leachate head on the liner system and to quantify leachate generation/collection rates for the maximum anticipated perpendicular drainage length in each cell and typical cell liner base grades.

While leachate pipe spacing is typically designed to complement the permeability of the leachate collection/protective cover layer in order to allow no greater than 300 mm of head buildup on the base liner system, results of the HELP Model runs indicate that lateral pipes are not required to maintain head at less than or equal to 300 mm. The longest perpendicular flow path to the central trunk line (170 meters) coupled with the permeability of the overlying sand filter layer and protective cover layer is sufficient without laterals. However, laterals have been included for contingency purposes at the approximate midpoint of each cell on either side of the collection trunk.

Pipe loading calculations within Appendix D indicate that SCH40 PVC (or thicker) is appropriate for the leachate collection piping within each cell.

11.4 Description of Leachate Transmission System and Pump Calculations

The perforated leachate collection trunk lines will terminate at the low point of the leachate collection system over an excavated sump within each cell, allowing collected liquid to “drop out” into the sump. The perforated pipe will transition to solid piping up the perimeter berm for use as a cleanout riser. The sump will be lined with additional non-woven geotextile and will be filled with coarse aggregate. A perforated 610 mm diameter HDPE pipe section will be installed within the sump area to house the submersible pump and will be connected to a solid 610 mm diameter riser pipe. The side slope riser will terminate at its upper end with a face plate assembly above ground level. The pump discharge for the leachate collection system will be connected through the riser termination to 100 mm (minimum) diameter HDPE perimeter force main, which will discharge to the lined leachate lagoon.

A 3.0 horsepower (HP) submersible pump (EPG model WSD 12-5 or an equivalent as approved by the engineer) and discharge piping will be lowered through the solid riser into the perforated sump section by support cables. Based on the proposed force main system and system head curve, it is anticipated that the pump will operate at about 227 liters per minute and 32.9 meters of total head. Pump Calculations are provided in Appendix D.

11.5 Description of Leachate Storage/Treatment Facilities

As described in Section 4.6, a 2.0 meter deep lined leachate storage/evaporation lagoon will be constructed. The lagoon will receive pumped leachate from each monofill disposal cell through the HDPE force main.

For the As Samra site, only passive/natural evaporation from exposure of the contained water surface to solar radiation and wind is proposed. No additional enhanced evaporation proposed by hydraulic/mechanical means such as sprinklers, misters, or aerators is proposed. For added conservatism, evaporation of stored water *has not* been accounted for in leachate lagoon sizing.

The leachate lagoon will be lined with a base liner containment system generally equivalent to the landfill cells anchored at the lagoon crest, except that the sand filter layer will not be installed. The liner system will consist of the following components in ascending order over the excavation/subgrade:

- 150 mm (minimum) of compacted select fill/liner cushion soil material (19mm maximum particle size); and,
- 1.5 mm (60mil) textured HDPE geomembrane liner.

12 Storm Water Management System

12.1 Design Strategy

EPA regulations (Subpart C) requires that surface water run-off from the active portion of the landfill be collected and controlled for at least the water volume resulting from a 25-year storm. Stormwater conveyance structures (channels, culverts, and basins) for the As Samra Landfill have been sized using the peak flow calculated using the rational equation and the 20-year return interval. Additional capacity has been provided in drainage channels and the stormwater management basins for larger storms by incorporating a minimum 250 mm of freeboard above the 20-year storm flow depth.

12.2 Stormwater Run-On

Surface water catchment areas draining from outside the landfill development area into/onto the site (run-on) were identified so that diversion channels (and culverts) could be designed to convey this surface water around the development area to the stormwater (runoff) management basin. Run-on towards the monofill from the south and east will be diverted north and west around the landfill through lined diversion channels to the basin.

12.2.1 Run-On Diversion Channels

The diversion channel has been designed with a trapezoidal cross-section and the Manning's equation to convey the peak flow with at least 250 mm of freeboard above the 20-year storm event. The channel has been designed with two separate design/construction alternatives to provide flexibility for use of more readily available and/or less expensive materials at the time of construction. Alternative 1 is riprap channel lining with average stone diameter (d_{50}) indicated below underlain by a woven geotextile filter fabric. Alternative 2 is long-term (permanent), UV-resistant synthetic erosion control matting with manufacturer's shear strength rating for bare earth equivalent to or greater than the values indicated. Detailed calculations are included within Appendix E of this Report. Table 12-1 summarizes the design alternatives.

Table 12-1: Run-on Diversion Channel Summary

Channel Reach	Design Flow (m^3/sec)	Slope (m/m)	Bottom Width (mm)	Side Slope (H:V)	Alternative 1 Riprap d50 (mm)	Alternative 2 Erosion Mat Minimum Shear Strength (Pa)	Minimum Channel Depth (mm)
A	0.1614	0.007	2000	2:1	75	9.0	397
B	0.1815	0.020	2000	2:1	75	20.2	371
C	0.2935	0.020	2000	2:1	75	22.0	402
D Reach 1	0.4444	0.005	2000	2:1	75	10.1	517
D Reach 2	0.4444	0.010	2000	2:1/3:1	75	16.7	470
E	1.2832	0.019	2000	2:1/3:1	75	47.1	570
F Reach 1	1.3655	0.019	2000	2:1/3:1	75	48.8	581
F Reach 2	1.3655	0.031	2000	2:1/3:1	75	70.5	540
F Reach 3	1.3655	0.050	2000	2:1	150	101.5	552
G Reach 1	1.6372	0.050	2000	2:1	150	112.2	580
G Reach 2	1.6372	0.090	2000	2:1	150	171.1	537
G Reach 3	1.6372	0.050	2000	2:1	150	112.2	580

12.3 Stormwater Run-Off

Surface water catchment areas draining from within the monofill development area and draining off of the site (run-off) were identified so that conveyance channels and culverts could be designed to convey this surface water to the stormwater (runoff) management basin. Runoff from a portion of the landfill final cover area will be collected within the Landfill Access Road Channel 4. Channel 4 and Perimeter Channel 3 will convey surface water through Culvert No.2 to Perimeter Channels 5 and 6, which will discharge through Culvert 3 to Diversion Channel E. Perimeter Channel 7 will convey surface water through Culvert 4 to Diversion Channel F. Perimeter Channel 8 will convey surface water through Culvert 5 to Diversion Channel G. Perimeter Channel 9 will convey surface water through Culvert 6 to the stormwater management basin. Perimeter Channels 1, 2 and 10 will discharge through Culvert 7 to the stormwater management basin.

12.3.1 Runoff Diversion Channels

Trapezoidal runoff diversion channels have been designed to collect runoff from the monofill final development surface and the perimeter access road and convey it to the stormwater management basin. Each channel has been designed with the Manning's equation to convey the peak flow with at least 250 mm of freeboard above the 20-year storm event. Each channel has been design with two separate design/construction alternatives to provide flexibility for use of more readily available and/or less expensive materials at the time of construction. Alternative 1 is riprap channel lining with average stone diameter (d_{50}) indicated below underlain by a woven geotextile filter fabric. Alternative 2 is long-term (permanent), UV-resistant synthetic erosion control matting with manufacturer's shear strength rating for bare earth equivalent to or greater than the values indicated. Detailed calculations are included within Appendix E of this Report. A summary of the design alternatives is provided in Table 12-3.

Table 12-3: Run-off Diversion Channel Summary

Channel Reach	Design Flow (m^3/sec)	Slope (m/m)	Bottom Width (mm)	Side Slope (H:V)	Alternative 1 Riprap d50 (mm)	Alternative 2 Erosion Mat Minimum Shear Strength (Pa)	Minimum Channel Depth (mm)
1	0.2952	0.010	1500	2:1	75	15.6	547
2 Reach 1	0.3908	0.080	1500	2:1	75	98.8	392
2 Reach 2	0.3908	0.020	1500	2:1	75	30.0	451
3	0.3700	0.010	1500	2:1	75	17.7	482
4	0.4004	0.025	1500	2:1	75	35.5	442
5	0.7545	0.010	1500	2:1	75	26.4	581
6	0.8007	0.020	1500	2:1	75	44.9	537
7	0.2154	0.020	1500	2:1	75	26.3	400
8	0.3638	0.020	1500	2:1	75	28.8	444
9	0.1075	0.020	1500	2:1	75	17.4	357
10	0.0734	0.020	1500	2:1	75	13.9	340

12.3.2 Runoff Diversion Culverts

Culverts have been designed to convey the design flow from the 20-year storm event without overtopping at the inlet constraint (top of channel, adjacent roadway, etc.), with sufficient soil cover over the pipe crest to withstand maximum anticipated vehicle loads, and with outlet protection to prevent scour in the downstream channel. Culverts will be either reinforced concrete pipe (RCP, for cover depth less than 600mm) or smooth-bore corrugated

polyethylene (CPE, for cover depths >= 600mm). Multiple circular culverts were selected if required to pass the design flow without overtopping the inlet, with sufficient cover over the pipe crest. Alternative designs (box culverts, arch pipes, etc.) may be reviewed and approved by the engineer prior to construction. Detailed calculations are included within Appendix E of this Report. A summary of the run-off diversion culvert design is provided in Table 12-4.

Table 12-4: Run-on Diversion Culvert Summary

Culvert	Design Flow (m^3/sec)	Invert In (m)	Invert Out (m)	Plan View Length (m)	Culvert Slope (m/m)	Culvert Type	Number Of Culverts	Culvert Diameter (mm)
1	0.4444	552.09	551.75	220	0.0155	CPE	1	610
2	0.7483	561.85	561.75	15.5	0.0064	RCP	2	762
3	0.8007	552.70	551.75	14.0	0.0679	CPE	1	762
4	0.2154	550.75	550.00	13.0	0.0577	CPE	1	457
5	0.3638	548.10	545.90	23.0	0.0957	CPE	1	457
6	0.1075	546.45	540.50	32.5	0.1831	CPE	1	457
7	0.4418	545.60	540.50	29.5	0.1729	CPE	1	457

12.3.4 Lined Surface Water Management Basin

As described in Section 4.7, a lined surface water management basin will be located at the northwest corner of the site and will ultimately receive runoff from the monofill. To facilitate storage and retention of water and to prevent potential undermining of the adjacent monofill perimeter berm, the stormwater management basin will be lined with a base liner system anchored at the basin crest consisting of the following components in ascending order over the excavation/subgrade:

- 150 mm (minimum) of compacted select fill/liner cushion soil material
- GSE Bentoliner CNSL GCL (Geosynthetic Clay Liner with a polypropylene geofilm adhered to the upper surface), or approved equivalent
- 300 mm (minimum thickness) native soil cover layer with 25 mm maximum particle size

For the As Samra site, only passive/natural evaporation from exposure of the contained water surface to solar radiation and wind is proposed. No additional enhanced evaporation proposed by hydraulic/mechanical means such as sprinklers, misters, or aerators is proposed. For added conservatism, evaporation of stored water *has not* been accounted for in stormwater basin sizing.

The basin may be equipped with a suction pump connected to a truck load out facility or facilities so that stored water may be used for landfill operations activities such as dust control or fire protection.

13 Final Cover System

13.1 Design Strategy

Jordan does not presently have comprehensive regulations specifically relevant to biosolids monofills that apply for this monofill project. Therefore, regulations in the United States have primarily been used as a basis for design on this project. In addition to providing requirements for the lining of biosolids monofills, 40 CFR Subpart C Part 503 also necessitates that measures be taken for closure and post-closure care.

Based on laboratory testing, the permeability of biosolids is anticipated to be very low (on the order of k = $1x10^{-10}$ cm/sec). While this could conceptually "negate" the need for a closure cap as a cap system is in itself designed primarily as "impermeable" to encapsulate waste and limit infiltration, a closure cap has several other beneficial functions (i.e. further minimization of infiltration, nuisance and odor control, minimize air intrusion into the waste, minimize fugitive gas emissions, stormwater management, aesthetics, etc.). Additionally, biosolids exposed in arid conditions will desiccate and crack. A closure cap system has been included as part of the As Samra biosolids monofill design. To promote LFG collection, the landfill shall be capped intermittently as waste filling reaches design elevations in each phase.

13.2 Description of ET Cover

For the As Samra landfill, an evapotranspiration (ET) cover system is proposed as a more economical alternative to commonly used clay and/or geosynthetic liners. The choice of using an ET cover was also influenced by the fact that the region is considered arid, with an average annual precipitation of 111.11 mm/yr and an average mean temperature of 19.5°C. ET covers rely on storing moisture within the cover system itself until the water either transpires or evaporates. Typical ET cover designs are either monolithic (single fine-grained soil layer) or include a capillary break. The capillary break allows the ET cover to retain more moisture especially under unsaturated conditions. The design of such covers depends on climate conditions of the landfill area, ET soil properties, and type of vegetation to be used in the cover.

After final biosolids placement, a monolithic ET cover will be installed. The surface of the ET cover will then be covered with an aggregate layer for erosion control. The ET cover was chosen due to its proven suitability in arid and semi-arid areas, its limited long-term maintenance requirements, as well as its economic feasibility. The ET cover design for the As Samra monofill will consist of a minimum of 1350 mm (1.35 m) of ET native soil cover overlain by 150 mm (minimum) of aggregate for erosion control.

Calculations within Appendix C evaluate potential percolation through evaporative cover soils of varying hydraulic conductivity (varied from a minimum of $5.0x10^{-6}$ cm/sec to a maximum of 1.7 x 10^{-3} cm/sec) and varying thickness (varied from a minimum of 300mm to a maximum of 1350 mm). Results generally indicate that evapotranspiration improves and percolation (infiltration into the biosolids) begins to be impeded significantly with ET cover soil hydraulic conductivity is reduced to approximately $1x10^{-5}$ cm/sec. While ET cover soil thickness plays less of a role in reducing infiltration, the proposed thickness will improve landfill gas collection efficiency.

14 Landfill Gas Management System

14.1 Design Strategy

The landfill (monofill) gas (LFG) Management System at the As Samra monofill has been designed to minimize the release of LFG to the atmosphere and to maximize the potential capture and destruction or re-use of gas at the on-site LFG management facility.

Vertical gas extraction wells (perforated PVC piping connected to surface accessible well-heads) are proposed with connections to a flare station, monofill gas electric generator, and/ or other utilization facility(ies) via a network of HDPE collection laterals and header piping. Perforated collection pipe within sloped horizontal trenches may also be installed at lower elevations in the waste mass in order to collect produced gas before final waste heights are reached and vertical wells are installed. As the gas cools in the collection system, moisture in the gas condenses. Gas collection piping will be sloped towards condensate traps (as applicable) to prevent pipes from becoming obstructed by accumulated condensate water. The system must also be designed to accommodate differential settlement by using flexible connections and pipe joints where practical/applicable and by using 2% (minimum) slopes for transmission piping on the monofill development surface.

14.2 Landfill Gas Generation

The annual future potential of the landfill to generate LFG was calculated using the EPA landfill gas modeling equation presented in the USEPA Landfill Gas Emissions Model (LandGEM). The equation is as follows:

$$Q_{CH_4} = \sum_{t=1}^{n} \sum_{j=0.1}^{1} kL_0 \left(\frac{M_t}{10}\right) e^{-kt_{ij}}$$

Where,

Q_{CH4}= Annual methane generation in the year of calculation (m^3/year),
i= 1 year time increment,
n= (year of the calculation) - (initial year of waste acceptance),
j= 0.1 year time increment,
k= methane generation rate ($year^{-1}$),
L_0= potential methane generation capacity (m^3/Mg),
M_i= mass of waste accepted in the ith year (Mg),
t_{ij}= age of the jth section of waste mass M accepted in the ith year (decimal years, ex. 3.2 years)

The LandGEM model assumes a six-month time delay between the placement of waste and the generation of LFG (EPA Central America LFG Model User's Manual, 2007). After six months, the model assumes that the LFG generation per unit of waste will decrease exponentially as the organic fraction is consumed. It also uses a first-order exponential decay function that assumes that the LFG generation is at its peak following a time lag representing the period prior to methane generation (EPA Central America LFG Model User's Manual, 2007). It is worth noting that maximum LFG generation usually occurs in the first and second years after closure.

Of the above equation components, all require site-specific data to produce generation estimates except for k and L_0 which are given constants. These two constants are defined as follows:

- **Methane Generation Rate Constant (k):** a model constant that determines the estimated rate of landfill gas generation. The first-order decomposition model assumes that k values before and after peak landfill gas generation are the same. k is a function of moisture content in the landfill waste, availability of nutrients for methanogens, pH and temperature. The unit used for k is 1/year. A typical value for k is 0.04 yr^{-1} for municipal solid waste landfills in the United States. The k value is expected be lower in dry climate conditions but higher if the waste has high moisture contents and nutrients (i.e. greater water and organic content). Essentially, a higher k value means that the methane would be produced faster and could be utilized faster. The k value for the As Samra model was set as 0.044 yr^{-1} based upon SWANA methodology

- **Potential Methane Generation Capacity (L_0):** L_0 is a model constant that represents the potential capacity of a landfill to generate methane. It is dependent on the amount of cellulose in the waste (i.e. it increases as organic content increases) and is measured in m^3/Mg. The L_0 value for the As Samra model was set to 281 m^3/Mg based on SWANA methodology and consideration of other factors such as anticipated volatile solids (VS) content in the dried biosolids.

The above data were used to run the EPA LandGEM model, which is included within Appendix F of this Report. The AS Samra LandGEM model indicates a maximum gas "generation" rate of 3,295 cfm. As the model used k and L_o modified by the SWANA method, the "collection rate" was actually calculated as opposed to "generation rate". Therefore, the peak generation from the LandGEM Model is divided by a collection efficiency factor in order to conservatively size gas headers and "control" devices (i.e. blowers and flares) for the adjusted generation rate.

The generation rate for various efficiencies may be calculated as follows:

- For potential collection efficiency of 50%, generation = 3,295 cfm / 0.50 = 6,590 cfm

The gas collection efficiency was assumed to be 50% (it is noted that this is entirely possible at the As Samra monofill, especially if daily cover application is sporadic and other standard operational practices vary from ideal situations over the project life) and was converted to potential electric power generation as presented in Appendix F. If the landfill gas system is constructed as designed and operated as intended, the LFG collection efficiency should be more than 70%. However, a conservative 50% collection efficiency was assumed.

The maximum annual generating capacity during the active life of the monofill was estimated as 4,800 kW (42,048,000 kW hr). Individual 2,000 KVA generators will be acquired in increments 3 or 4 years apart and phase in and out of operation to meet the variable capacity required over the project life. Engines would be located on a reinforced concrete foundation pad or pads within the fenced infrastructure area approximately adjacent to the existing transformer. If sale and conveyance of treated or untreated gas to off-site end users is pursued, gas compressors would also be acquired and phased into and out of the gas management infrastructure to meet the variable capacity required over the project life.

14.3 Landfill Gas Collection, Transmission and Utilization System

At a minimum, the proposed LFG management system will consist of 97 permanent vertical extraction wells, transmission piping, condensate collection and handling facilities, a blower to draw the LFG from the well field, and a backup flare to reduce atmospheric emissions. Landfill gas utilization (pretreatment, storage, transmission piping, power generation, etc.) were evaluated as options within the Feasibility Study.

14.3.1 Horizontal Gas Collection Trenches

Horizontal gas collection trenches will be installed within the monofill to allow for the collection and transmission of LFG from the lower elevations (depths) prior to installation of the ET cover system and/or when vertical drilling is impractical due to shallow biosolids depths. Horizontal trenches will be excavated into the biosolids and sloped away from the extraction location into the waste at slopes no shallower than 1% (0.01 meter/meter). The trench will be lined with a geotextile filter. A perforated collection pipe will be installed within the trench and then the trench will be backfilled with granular drainage layer. The perforated pipe will be transitioned to a minimum five (5) meter long solid piping segment near the extraction point and then to a vertical “trench wellhead” equivalent to the well heads installed for vertical gas extraction wells. To help minimize air intrusion, it is recommended that biosolids reach a minimum height of 5 meters above the trench crest prior to activating the trench.

14.3.2 Vertical Gas Collection Wells

The layout and density of the vertical gas wells has been designed for efficient extraction and transmission of landfill gas from all portions of the landfill. While well spacing (horizontal distance between the wells) may be standardized in a grid pattern, well spacing may also be determined by calculating a radius of influence (ROI) of each well. The ROI defines an area from which gas can be extracted without drawing excessive air into the landfill. The layout configuration for the As Samra Landfill employs an approximate 38 m of equidistant spacing between wells, with a design maximum ROI of 23 meters such that their radii of influence overlap. It is noted that minimum well spacing for “typical” North American municipal solid waste industry LFG is about 1 well per hectare.

The minimum LFG well borehole diameter will be 1000 mm (1m) and the pipe casing for the wells will have a diameter of 225 mm (0.225m). The well depth will vary with waste depth but will be no deeper than 30 meters, with a minimum clearance of 4.5 meters between the base of the well and the protective cover layer of the base liner system. The wellheads will be equipped with quick change orifice plate assemblies and precision control valves, as can be seen in the Design Details.

14.3.3 Gas Transmission Piping

The LFG laterals and header piping will be installed within a compacted backfill layer with a minimum of 150 mm of compacted backfill above and below each pipe. This facilitates the collection of LFG and provides structural support to the pipe. A minimum 600 mm (0.6m) thick layer of general backfill with an embedded utility identification tape will be installed above the compacted backfill, followed by the final cover soil.

All LFG pipes shall be PE 100 polyethylene, standard dimension ratio 17 (SDR 17) unless noted otherwise. Changes in horizontal alignment and vertical PE LFG pipes shall be accomplished by taking advantage of the flexural properties of the PE pipes whenever possible. The minimum bend radius for these shall be 27 times that of the pipe’s outer diameter or 100 times that of the pipe’s outer diameter when fittings fall on the bend.

Gas conveyance piping is selected to the appropriate minimum diameter that will convey flow at no greater than the selected maximum velocities for concurrent and countercurrent flow. Based on experience, the following maximum LFG flow velocities were used: 6.1 m/s when LFG and condensate flow in opposite directions (counter-current flow) and 12.2 m/s when LFG and condensate flow in the same direction (concurrent flow). The minimum lateral pipe size (gas well connection to main header) shall have a minimum nominal diameter of 100 mm. The minimum header pipe size on the landfill (connecting well field to the perimeter header) shall have a minimum nominal diameter of 150 mm. The minimum nominal diameter of the perimeter header pipe to the flare station will be 300 mm. The contractor shall ensure that the LFG pipe slope is maximized while pipe depth is minimized. The LFG collection header system at As Samra is designed with a minimum slope of 2.0% to provide for gravity flow of condensate to designed low points.

The LFG header pipe system will facilitate efficient LFG control by employing a looped piping system to provide an even distribution of blower vacuum to the vertical extraction wells. LFG header valves (butterfly valves or equivalent as approved by the engineer) will be distributed along the system as indicated on the Design Drawings.

14.3.4 Gas Condensate Management

Landfill gas condensate will be removed from the LFG header at a condensate knock-out (or knock-outs). Knock-outs may be designed to allow the condensate to drain back into the refuse mass and ultimately to the landfill's leachate collection system, or to a temporary holding tank outside the limits of waste. Based on assumed temperature, the amount of condensate generated at the proposed As Samra Landfill was calculated to be an average of approximately 10,771liters/day (collection estimated as 5,386 liters/day). The permanent LFG system piping will be sloped at a minimum of 2% to convey condensate to the engineered low point in the system adjacent to the landfill gas management area.

Due to the relatively small size of the landfill, the proposed geometry and alignment of the gas collection pipes have been oriented such that one low-point is required along the perimeter berm near the northwest corner and an additional low point near the landfill gas management area in the proposed infrastructure. An in-line condensate knockout with an interior baffle plate and sump will be installed at each low point along the main header and prior to the flare station. Condensate from the knockouts may then be pumped to a vacuum truck, or pumped or drained by gravity to a condensate storage tank.

The condensate knock-out near the gas management facility will be connected to an underground condensate holding tank via 110 mm SDR 17 PE piping. The tank will have at least 2500 mm (2.5 m) of native soil cover over the crest. The tank will be double-walled steel construction in accordance to the US EPA Act 100 specifications or other similar specifications. It will be selected to operate under earth loads with a minimum of 150% safety factor. All the pipe connections on the tank shall be flanged and piping shall be PE unless otherwise noted and exposed metallic piping and metallic piping in the tanks shall be primed and coal tar epoxy painted. The holding tank will to be emptied by manual pumping through a suction pipe via a vacuum truck. The specified tank is 37,850 liters (10,000 gallons), making the actual storage capacity approximately 7 days based on an anticipated maximum condensate collection rate of 5,386 liters/day near the end of the facility life.

14.3.5 Blower and Flare System

The extracted LFG will be transferred via the header pipe to a Blower/Flare station which will be located within the infrastructure area as indicated and detailed within the Design Drawings. The flare stack will rest on a concrete foundation and will include a flame arrestor, a condensate drain port and an orifice plate between the blower and flare to measure the

flow rate. The operational vacuum is to be a minimum of 0.254 m (10 inches) of water column at each well. The maximum blower/flare capacity is to be 6,590 scfm with step down/turndown capability for lower flows. The blower calculation is the sum of the required minimum vacuum at the furthest well, the pressure losses in the LFG piping and pressure losses through the flare and flame arrestor. For the proposed system, this adds up to the minimum approximate blower vacuum of 686 mm (27 inches) of water column.

14.3.6 Conveying Gas to JBC Facility

If a third party is identified and contracted with, the LFG would be extracted from the landfill and delivered to a pipeline with a standard commercial blower system. The landfill gas delivered to the pipeline system will be initially passed through a filtering system to remove any filterable impurities, which could damage equipment. The filtered landfill gas will be directed to a single stage blower where its pressure will be raised to approximately 15 psig. The compressed landfill gas will be directed into a knockout tank where a large portion of the entrained water will be removed. The knockout tank has no vents; therefore, it has no air pollution potential. At this point in the fuel gas compressor process, the landfill gas will have a temperature of approximately 230°F and will be directed to a cooler (air to air) to reduce its temperature. The temperature reduction will force the condensation of water vapor in the gas. After the cooler, the gas will pass through a second coalescent filter to remove the condensed water vapor.

A dehydration system is then used to further remove entrained liquids. The condensate will be directed to a storage tank and then discharged to the facility's on-site leachate storage facility.

Once the landfill gas is compressed in the gas compression system and the condensate is removed, the landfill gas will be moved into the pipeline system. This pipeline will be constructed of 406mm (typical) diameter HDPE pipe. It will be placed underground in a trench that is three feet wide. The top of the pipe will be 1000mm (typical) below the ground surface. This option is generally limited to a user facility within approximately 8 km of the As Samra landfill facility.

15 Monitoring

15.1 Design Strategy

In the United States, environmental monitoring in the vicinity of landfill is required by law as part of EPA regulations for landfill design, construction, and operation. The final set of monitoring requirements is site specific and depends on the risk of contamination. Landfill gas detection monitoring and groundwater monitoring are discussed below for further consideration prior to implementation of the project. Geotechnical monitoring of the biosolids monofill is discussed within Section 9.

15.2 Landfill Gas Detection Monitoring

A LFG monitoring system typically consists of gas probes installed around the site perimeter that measure the gas concentrations at regular intervals. Probes are constructed to a minimum diameter of 100 mm with threaded pipe connections. A minimum of 3000 mm (3m) of the 32 mm diameter PN16 PVC-U is perforated with 5mm diameter holes. The borehole depths will vary and must be field verified prior to drilling.

15.3 Groundwater Monitoring

At a minimum, a groundwater monitoring network consists of one upgradient and two groundwater monitoring wells. If wells are installed, general parameters that should be obtained during a monitoring event consist of:

- Static water level (field parameter)
- Specific conductivity (field parameter)
- pH (field parameter)
- Dissolved oxygen (field parameter)
- Turbidity (field parameter)
- Temperature (field parameter)
- Color and sheens (by observation)

USEPA Subtitle D recommendations for groundwater monitoring recommend at a minimum the sampling and analysis of the following Constituents for Detection Monitoring:

From Appendix I to Part 258—Constituents for Detection Monitoring

	Common name	CAS RN2
Inorganic Constituents:		
(1)	Antimony	(Total)
(2)	Arsenic	(Total)
(3)	Barium	(Total)
(4)	Beryllium	(Total)
(5)	Cadmium	(Total)
(6)	Chromium	(Total)
(7)	Cobalt	(Total)
(8)	Copper	(Total)
(9)	Lead	(Total)
(10)	Nickel	(Total)
(11)	Selenium	(Total)

(12)	Silver	(Total)
(13)	Thallium	(Total)
(14)	Vanadium	(Total)
(15)	Zinc	(Total)

Organic Constituents:

(16)	Acetone	67–64–1
(17)	Acrylonitrile	107–13–1
(18)	Benzene	71–43–2
(19)	Bromochloromethane	74–97–5
(20)	Bromodichloromethane	75–27–4
(21)	Bromoform; Tribromomethane	75–25–2
(22)	Carbon disulfide	75–15–0
(23)	Carbon tetrachloride	56–23–5
(24)	Chlorobenzene	108–90–7
(25)	Chloroethane; Ethyl chloride	75–00–3
(26)	Chloroform; Trichloromethane	67–66–3
(27)	Dibromochloromethane; Chlorodibromomethane	124–48–1
(28)	1,2-Dibromo-3-chloropropane; DBCP	96–12–8
(29)	1,2-Dibromoethane; Ethylene dibromide; EDB	106–93–4
(30)	o-Dichlorobenzene; 1,2-Dichlorobenzene	95–50–1
(31)	p-Dichlorobenzene; 1,4-Dichlorobenzene	106–46–7
(32)	trans-1, 4-Dichloro-2-butene	110–57–6
(33)	1,1-Dichlorethane; Ethylidene chloride	75–34–3
(34)	1,2-Dichlorethane; Ethylene dichloride	107–06–2
(35)	1,1-Dichloroethylene; 1,1-Dichloroethene; Vinylidene chloride	75–35–4
(36)	cis-1,2-Dichloroethylene; cis-1,2-Dichloroethene	156–59–2
(37)	trans-1, 2-Dichloroethylene; trans-1,2-Dichloroethene	156–60–5
(38)	1,2-Dichloropropane; Propylene dichloride	78–87–5
(39)	cis-1,3-Dichloropropene	10061–01–5
(40)	trans-1,3-Dichloropropene	10061–02–6
(41)	Ethylbenzene	100–41–4
(42)	2-Hexanone; Methyl butyl ketone	591–78–6
(43)	Methyl bromide; Bromomethane	74–83–9
(44)	Methyl chloride; Chloromethane	74–87–3
(45)	Methylene bromide; Dibromomethane	74–95–3
(46)	Methylene chloride; Dichloromethane	75–09–2
(47)	Methyl ethyl ketone; MEK; 2-Butanone	78–93–3
(48)	Methyl iodide; Idomethane	74–88–4
(49)	4-Methyl-2-pentanone; Methyl isobutyl ketone	108–10–1
(50)	Styrene	100–42–5
(51)	1,1,1,2-Tetrachloroethane	630–20–6
(52)	1,1,2,2-Tetrachloroethane	79–34–5
(53)	Tetrachloroethylene; Tetrachloroethene; Perchloroethylene	127–18–4
(54)	Toluene	108–88–3
(55)	1,1,1-Trichloroethane; Methylchloroform	71–55–6
(56)	1,1,2-Trichloroethane	79–00–5
(57)	Trichloroethylene; Trichloroethene	79–01–6
(58)	Trichlorofluoromethane; CFC–11	75–69–4
(59)	1,2,3-Trichloropropane	96–18–4
(60)	Vinyl acetate	108–05–4
(61)	Vinyl chloride	75–01–4
(62)	Xylenes	1330–20–7

It is worth noting here that as per the USEPA regulations these monitoring requirements should be site-specific and may be suspended if the owner/operator of the landfill demonstrates that there is no potential migration of hazardous constituents from that landfill to the uppermost aquifer during the active life of the landfill and the post-closure care period.

For the proposed As Samra project, the ground water table is deep under the landfill and anticipated rain fall is low. Therefore, it may be possible to waive some of these monitoring requirements.

16 Quantity and Cost Estimates

This section includes the quantity estimates for infrastructure needed at the monofill, phased cell construction, and phased closure and permanent gas management installation. Capital costs for landfill gas treatment, temporary gas management, electric power generation, gas connection, staff salary and training, and overhead are not included.

16.1 Infrastructure

As discussed above, infrastructure and other facilities will be required in support of security, access, operations, management, and monitoring functions. Existing facilities will be re-furbished and new facilities will be constructed in advance of, or in conjunction with the initial cell and are included within the quantity estimates for the first cell.

- Perimeter security fencing with a lockable access gate(s);
- Crushed stone surfaced access roads;
- Refurbished chlorine handling building to serve as site office/maintenance building;
- Fuel dispensary;
- Lined leachate evaporation lagoon;
- Monofill gas/electricity generation facility with a backup flare station;
- Lined stormwater management basin;
- Runon/Runoff Drainage Channels;
- Three (3) groundwater monitoring wells; and,
- Utility extensions/improvements.

The costs for the construction of the infrastructure/support areas were estimated within Appendix H Table 1 (not including any contingency factor) to total:

- Infrastructure and Support = JD 1,137,226 (USD 1,603,729)

16.2 Cell Construction

The monofill will be designed with five (5) cells, each draining to its own sump from which collected leachate will be pumped to the leachate storage/evaporation lagoon. Each cell will be segregated from adjacent cells with a 1.5 meter (minimum height) lined interim/intercell berm. Quantity elements included in the construction of each of the five cells include:

- Subgrade excavation;
- Subgrade fill;
- Perimeter access road and concrete or stone lined stormwater channel;
- Liner cushion layer (150 mm thick);
- 60 mil HDPE liner layer;
- Nonwoven geotextile cushion layer;
- Protective cover/leachate collection layer (450mm protective cover aggregate overlain by 300mm sand filter layer);
- PVC leachate collection piping system;
- Leachate sump/risers/pumping system;
- Perimeter utilities (leachate forcemain, gas header and electrical); and,
- Quality assurance during construction.

Major cell construction quantities are summarized in Table 16-1.

Table 16-1: Major Construction Quantities

Cell Identification	Subgrade Cut (m^3)	Subgrade Fill (m^3)	Liner Area (m^2)	PVC Piping (m)	Perimeter Forcemain (m)	Perimeter Electric (m)	Perimeter Gas (m)
Cell 1	359,302	30,245	62,092	730	220	850	850
Cell 2	101,639	2,520	32,122	530	105	100	100
Cell 3	182,607	12,021	44,104	620	130	130	255
Cell 4	114,586	15,209	28,522	470	65	65	145
Cell 5	94,541	25,432	32,461	505	na	na	510
Totals	**852,517**	**85,366**	**199,301**	**2,855**	**520**	**1145**	**1,860**

The costs for the construction of the each cell were estimated within Appendix H Tables 2 through 6 (not including any contingency factor) to total:

- Cell 1 Construction = JD 2,851,622 (USD 4,019,853)
- Cell 2 Construction = JD 1,119,692 (USD 1,578,594)
- Cell 3 Construction = JD 1,700,891 (USD 2,397,785)
- Cell 4 Construction = JD 1,142,895 (USD 1,611,201)
- Cell 5 Construction = JD 1,133,864 (USD 1,880,685)

16.3 Phased Closure and Permanent Landfill Gas Management System

The monofill will be closed in phases with an ET cover to minimize potential infiltration of stormwater into the waste mass, to control odors and limit other nuisances such as mosquitos and vermin, and to allow for earlier and more efficient gas collection/utilization. Quantity elements included in closure and gas management system estimates include:

- 1.35 m (maximum thickness) ET soil cover;
- 0.15 m (minimum) gravel layer for erosion control;
- Permanent gas extraction wells;
- Permanent gas piping; and,
- Quality assurance during construction.

Minimum phased closure and gas management system quantities are summarized in Table 16-2 as quantities constructed by the end of filling in each phase as indicated on the design drawings.

Table 16-2: Closure Cap and Permanent Landfill Gas Management Quantities

Cell/Closure Phase	Capping Area (ha)	Trench Wellheads (each)	Perforated Piping Trenches (m)	Permanent Wellheads (each)	Drilling Depth (meters)	Gas Piping (meters)
Phase 1	0.9736	8	530	0	0	206
Phase 2	2.6657	7	755	13	168	618
Phase 3	1.4397	5	585	9	115	480
Phase 4	3.5745	6	695	24	417	1,048
Phase 5	11.4950	10	640	51	860	2,348
Totals	**20.1485**	**36**	**3,205**	**97**	**1,560**	**4,700**

It is noted that trench wellheads and permanent wells may be installed at any time during development of the cell depending on the need for active gas management prior to phased

installation of the ET cover. The estimates above represent minimum anticipated requirements.

The costs for the construction of the each phase of closure capping (including the temporary LFG management system) as shown on the design drawings were estimated within Tables 7 through 12 in Appendix H (not including any contingency factor) to total:

- Cell 1 Closure= JD 132,059 (USD 186,194)
- Cell 2 Closure = JD 328,888 (USD 445,691)
- Cell 3 Closure = JD 197,643 (USD 278,662)
- Cell 4 Closure = JD 444,504 (USD 626,703)
- Cell 5 Closure = JD 1,247,283 (USD 1,758,590)

16.4 Operation and Maintenance Costs

Operations and Maintenance (O&M) cost have been developed considering the vehicle operations and maintenance, fuel and electricity costs needed to operate the monofill over a 33-year period. O&M cost estimates are included in Appendix H Table 19 (not including any contingency factor) to total:

- Annual O&M = JD 257,951 (USD 363,824)

Appendix H - Table 20 calculates the one-time closure/post-closure cost associated with decontamination and removal of equipment and support facilities after waste disposal has ceased.

- Close/Post Close = JD 202,065 (USD 285,000)

Appendix H - Table 21 calculates the annual post-closure monitoring and maintenance cost for the facility (inspections, cap maintenance, groundwater monitoring/analysis/reporting/, gas well field balancing, stack maintenance, leachate monitoring, pumping, Quality Assurance and reporting, etc.). Annual post closure costs are projected to be incurred for at least the first 5 years after closure has been completed.

- Annual Post Clos. = JD 156,689 (USD 221,000)

17 References

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APPENDIX A
DESIGN DRAWINGS

- MONOFILL DESIGN DRAWINGS
- ADMINISTRATION BUILDING DESIGN DRAWINGS

APPENDIX B
GEOTECHNCIAL INVESTIGATION

APPENDIX C
GEOTECHNICAL AND GEOSYNTHETIC MATERIALS CALCULATIONS

1. Biosolids Geotechnical Testing
2. Slope Stability Analyses
3. Liquefaction Analysis
4. Liner Veneer Stability
5. Nonwoven Geotextile Design
6. PVC Leachate Collection Pipe Strength Analysis

APPENDIX D
LEACHATE MANAGEMENT CALCULATIONS

1. Infiltration through Evapotranspiration (ET) Cap
2. Leachate Generation Estimate
3. Leachate Collection Pipe Spacing Evaluation
4. Leachate Forcemain and Pump Design

APPENDIX E
STORMWATER MANAGEMENT CALCULATIONS

APPENDIX F
LANDFILL GAS SYSTEM CALCULATIONS

APPENDIX G
QUALITY ASSURANCE MANUAL FOR GEOSYNTHETIC LINER SYSTEM INSTALLATION

APPENDIX H
QUANTITY AND COST ESTIMATES