Figure 8: Test pit locations

The following is a summary of the shallow geology observed in the test pits along the western boundary of the site (Test Pits 1 to 5):

- The upper layer (varying from 0.3 m to 1.3 m depth) generally consists of loose fill such as ash or waste;
- This overlies a moist clayey silt layer (in some areas a transported back clay) approximately 1 m thick;
- Very moist or wet silty sand which varies between pits as residual/weathered sandstone/dolerite is then encountered until refusal on weather dolerite or sandstone;
- Refusal depth varies from 4.9 m in the north to 3.2 m in the south-west corner; and
- The observed seepage depth varies from 3.9 m to 0.4 m – it should also be noted that seepage assumed to be from the slopes of the landfill is present in areas on the surface.

Figure 9: Seepage in Test Pit 1

Figure 10: Test Pit 4

Figure 11: Upper layers of ash in Test Pit 3

The shallow geology observed in the test pits along the southern boundary is as follows (Test Pits 6 to 9):

- The upper layer (varying in depth from 0.6 m to 1 m) consists of moist fill or transported black clay;
- This overlies a soft, silty clay or residual clay in some areas of 1 m to 2 m;
- Weathered sandstone or dolerite is encountered underneath this layer to depths of approximately 3 m;
- Refusal is encountered at depths varying from 2.2 m (Test Pit 9) to 3.2 m (Test Pit 6) on residual sandstone or hard rock dolerite (Test Pit 6); and
- Seepage depths in Test Pits 6 and 7 varied from 1.5 m to 1.7 m below ground level, while no seepage was encountered in Test Pits 8 and 9.

It should be noted that caving was encountered in Test Pit 1 (located in the upper north-west corner of the site) and Test Pit 6 (south-west corner) from about 1 m depth.

Figure 12: Caving of clay layers in Test Pit 6

8.2 Groundwater

The local groundwater associated with Charlie 1 landfill area comprises two aquifer systems, namely an upper weathered Ecca aquifer system and the lower fractured rock Ecca aquifer system. The upper system is characterized by soils and weathered Karoo sediments within which the shallow contaminated seepage from the site will manifest.

The report compiled by the IGS (2008) states that the Karoo Supergroup, which underlies this site, consists of fractured-rock aquifers characterised by sediments with low permeability, implying that groundwater movement occurs mostly along secondary structures such as fractures, cracks and joints in the sediments.

Seepage from recharge into the waste flows along the higher hydraulic conductivity waste to the clay contact along the bottom of the site, resulting in horizontal flow to and along the toe of the landfill. This causes seepage springs on the western portion the site.

The contaminant plume from the landfill site is mostly concentrated within the upper weathered soil and/or clay zone, with shallow water levels within this zone compared to levels at distance from the site. It is suggested that the higher water levels are due to the land fill and that natural conditions no longer exist. The contaminant plume is primarily located within the immediate vicinity of the site.

The average Soil Horizontal Hydraulic conductivity (K) was established as 0.0128 m/d.

8.3 Leachate quantity

The predicted quantity of leachate to be captured in the collection system was derived from Darcy's law for the calculation of flow of liquid through a porous medium:

- Darcy's law is as follows: Q = kiA.

The computed flow rates for each of the two interception drains (north and south) are indicted in Table 8 and Table 9.

Table 8: Leachate seepage rate calculation for North Drain

Description	Value	Unit
Hydraulic gradient (k)	0.0128	m/d
Hydraulic gradient (i)	0.01	m/m
Interception length (L)	550	m
Average trench depth (D)	3.5	m
Interception area (A = L x D)	1925	m^2
Seepage rate (Q = k x i x A)	**0.25**	**m^3/d**

Table 9: Leachate seepage rate calculation for South Drain

Description	Value	Unit
Hydraulic gradient (k)	0.0128	m/d
Hydraulic gradient (i)	0.35	m/m
Interception length (L)	750	m
Average trench depth (D)	3.5	m
Interception area (A = L x D)	2625	m^2
Seepage rate (Q = k x i x A)	**11.75**	**m^3/d**

The total expected seepage rate was therefore assumed to be 12 m^3/day based on the above calculations.

8.4 Leachate interception

The "curtain" drains along the southern, western and northern downslope boundaries of the landfill site will extend to depths varying from two to five metres below surface level, generally extending to refusal depth. The interception drain collects leachate from the landfill into an HDPE pipe which directs flow to a sump located in the north-west corner of the site. The proposed leachate interception drain is shown on Figure 13, comprising the following:

- 600 mm wide, varying depth trench;

- Geocomposite drain within a layer of river sand that intercepts the leachate by creating a low resistance flow path ("curtain drain" concept). The geo-composite extends from the toe of the landfill to the bottom of the trench where it is wrapped around an HDPE perforated pipe;
- Leachate collects in this pipe and drains towards the sump;
- 1.5 mm LLDPE double textured geomembrane over the surface of the drain to prevent stormwater ingress;
- Cleaning/unblocking manholes at 100 m intervals along the drain; and
- Pipe elbows facing the floor within these manholes creates a water seal to prevent air ingress into the drain to reduce the tendency for biochemical clogging.

The above leachate sump is constructed with two sets of manhole rings with concrete infill to prevent leachate leakage and to improve structural integrity. Leachate collected in the sumps is pumped into the leachate pond (refer to section 12.1 for pump details).

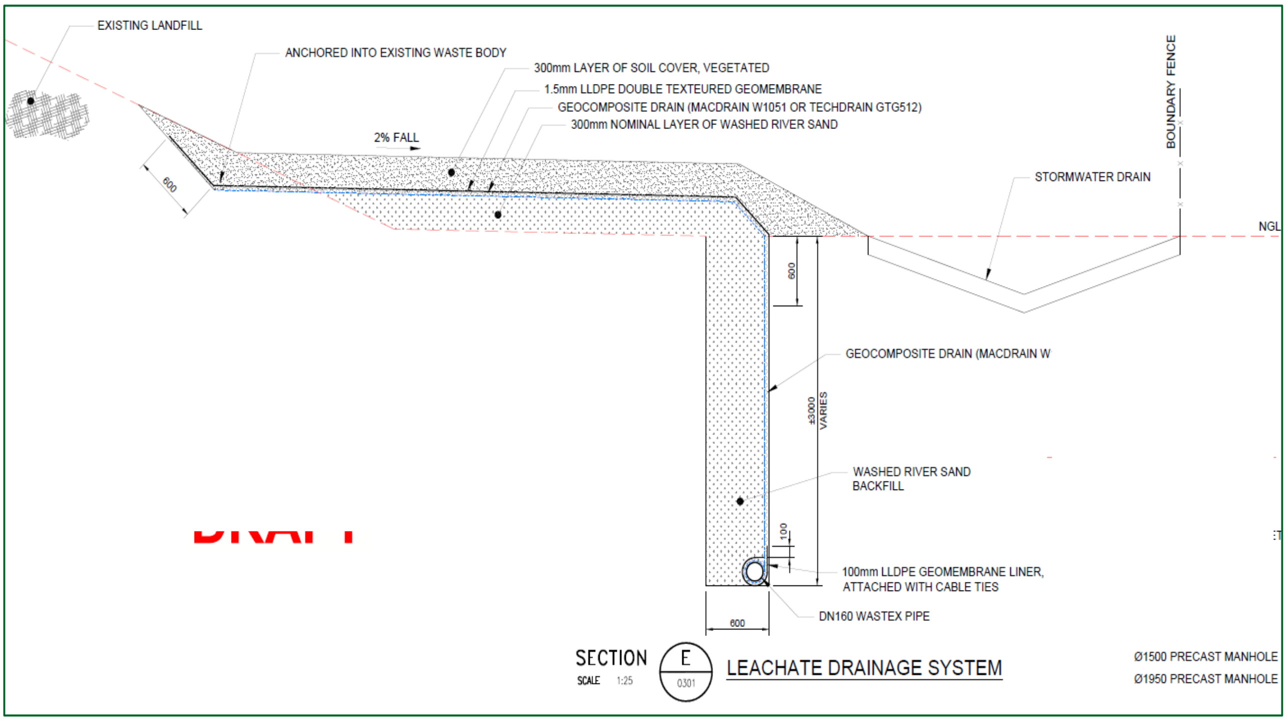

Figure 13: Leachate interception drains (Drawing 0502)

8.5 Leachate collection, impoundment and handling

As test pitting was not conducted for the prefeasibility work, the nature and extent of the onsite leachate generation was not fully appreciated. The test pitting indicated that this requires attention and given the likely computed seepage rate a dedicated leachate pond to impound and handle this flow is required.

The leachate will be collected by means off a dedicated interception system as describe above and routed via the described sump and pump system to a standalone leachate pond that is lined with a geosynthetic liner system meeting regulatory requirements.

The leachate pond has a capacity of 1 500 m^3, designed to maintain a freeboard of at least 0.5 m in accordance with the Waste Act of 2008. To limit the frequency of abstraction from the pond to maintain/manage the in-pond water levels, the pond is equipped with a 12.5 m wide evaporative fringe. The numerical water balance modelling for the pond indicated that about 10 m^3/d of leachate on average (depending on climatic conditions) could be evaporated from this fringe.

This will require periodic abstractions from the pond to maintain the in-pond water levels as well as to ensure that the impounded leachate salt concentrations are maintained within limits that will allow for reasonable ongoing evaporation rates (details on the abstraction requirements are provided in section 11.0).

For abstraction a gooseneck structure and dedicated pump has been included in the design. In the case of an extreme rainfall event, an emergency spillway will release excess water/leachate into the surrounding environment.

To allow for the distribution of the impounded leachate onto the evaporative fringe and also to further enhance evaporation, a micro spray system will be installed and operated on the fringe. The details of this evaporative system are described in detail in section 10.0.

9.0 STORMWATER MANAGEMENT

The existing operation of the landfill in terms of concurrent rehabilitation and associated stormwater routing and control is not optimal. The operation of the landfill has to be adapted in accordance with the development plan and associated stormwater diversion structures (section 15.0) to ensure that areas with defined rehabilitation are created. The runoff from these areas is deemed clean and could be released into the receiving environment, not requiring it to be captured and handled within the onsite contaminated stormwater system.

This will allow the latter system to be more optimally sized. Given this approach, the contaminated stormwater system is designed to cater for about 40% of the expected runoff from the landfill site.

Given the nature and extent of rainfall events on the Mpumalanga Highveld, possible pumping of the collected stormwater runoff was not considered. Allowance has been made for the peak design flows to gravitate freely along the dedicated concrete lined channels to the stormwater pond.

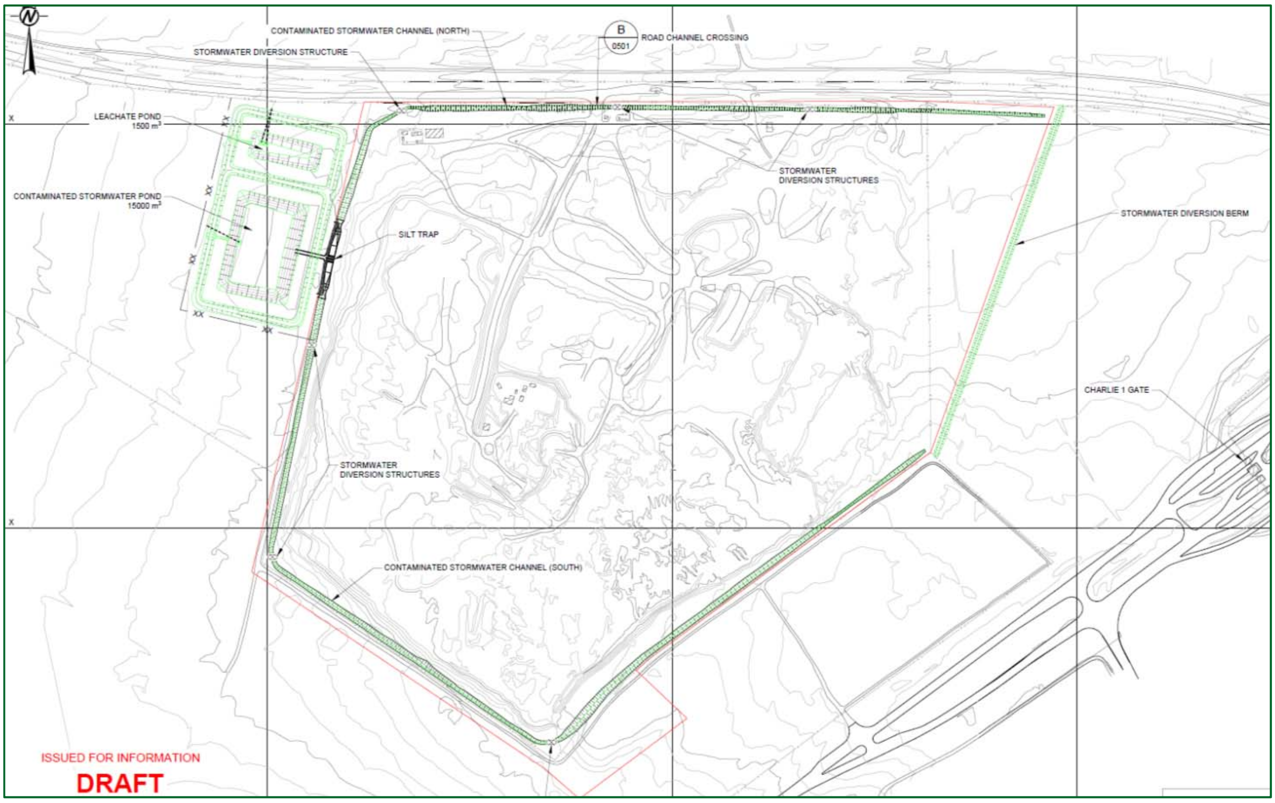

Figure 14: Stormwater system general arrangement (Drawing 0201)

9.1 Stormwater interception

Stormwater falling on the site is intercepted and prevented from leaving the site via a series of concrete v-drains along the boundary. Drains are proposed along the southern, western and northern site boundaries, while a diversion berm is to be constructed along the upslope eastern boundary of the site. This berm serves the purpose of preventing “clean” off-site runoff from entering the site and becoming contaminated.

Construction of the V-drains as shown in Figure 15 will be from concrete filled geocells. This construction technique is proposed due to expected settlement associated with landfill ground conditions and the relative flexibility of the geocells.

Two different sized V-drains are proposed, namely Type 1 and Type 2, based on expected flow rates in different areas.

Surface runoff calculations to calculate the drain requirements were carried out using a PCSWMM model. PCSWMM is a dynamic rainfall-runoff simulation model used for single event or long term simulation of runoff quantity. Based on the modelled scenario and outcomes, the following drain sizes were used in the feasibility designs:

- Type 1: 2.4 m wide, 0.6 m deep, 1:2 side slopes; and
- Type 2: 3.2 m wide, 0.8 m deep, 1:2 side slopes.

Type 1 drains were used for the northern channel and the south-eastern channel while Type 2 was used for the south-western drain.

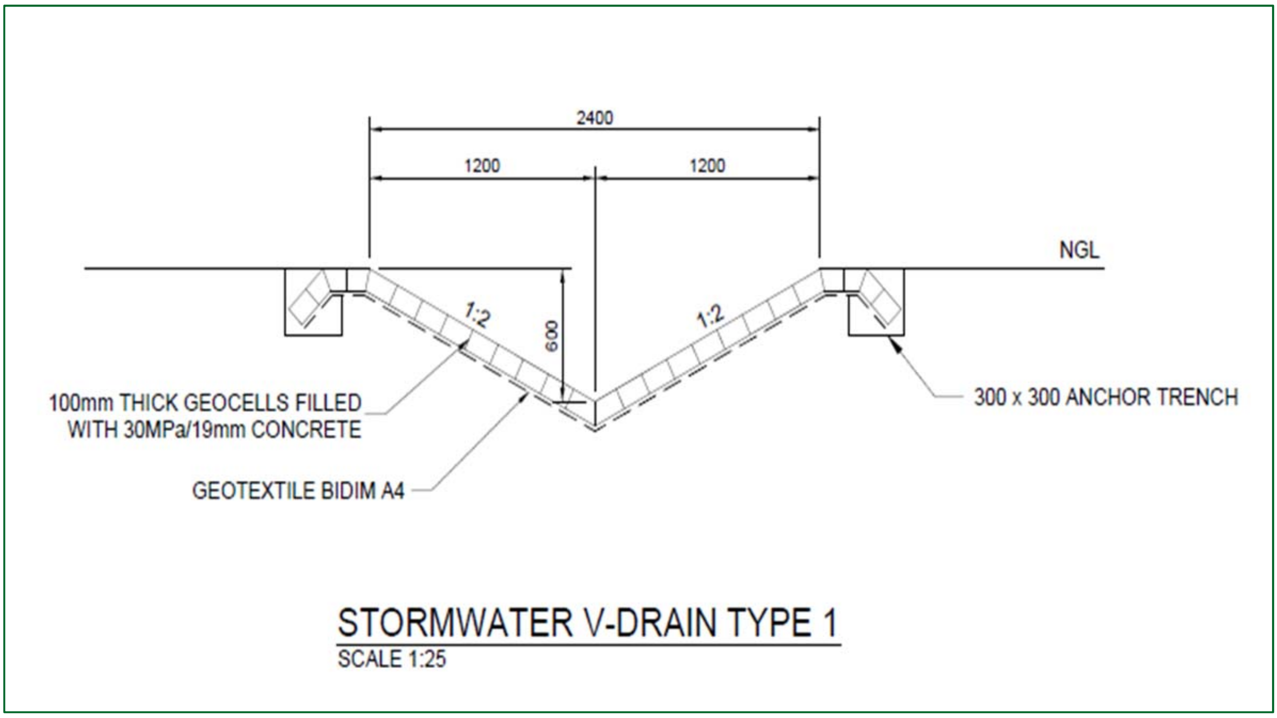

Figure 15: Stormwater v-drain (Drawing 0501)

9.2 Stormwater collection, routing and impoundment

As described above, the contaminated stormwater is collected by a series of concrete lined V-drains. These drains gravitate to a silt trap before spilling into the contaminated stormwater pond. Stormwater diversion structures are to be constructed at strategic locations along these drains to allow the diverting of clean runoff from rehabilitated areas away from the site (details of these structures are provided in section 15.4 of the development plan).

The stormwater pond, which is also lined with a geosynthetic liner system meeting regulatory requirements, has a capacity of 15 000 m^3 while maintaining a freeboard of 0.5 m at all times. As with the leachate pond, this pond is equipped with a 12.5 m wide evaporative fringe. The numerical water balance modelling for the pond indicated that about 30 m^3/d of leachate on average (depending on climatic conditions) could be evaporated from this fringe. This will require periodic abstractions from the pond to maintain the in-pond water levels (details on the abstraction requirements are provided in section 11.0).

For this abstraction a gooseneck structure and dedicated pump has been included in the design. In the case of an extreme rainfall event, an emergency spillway will release excess water/leachate into the surrounding environment.

To allow for the distribution of the impounded stormwater onto the evaporative fringe and also to further enhance evaporation, a micro spray system will be installed and operated on the fringe. The details of this evaporative system are described in detail in section 10.0.

10.0 ENHANCED EVAPORATION SYSTEM

The enhanced evaporation system involves the combination of a 12.5 m evaporative fringe around each of the ponds with a micro sprayer system to spray leachate/stormwater over the fringe for enhanced evaporation. Enhanced evaporation is expected to increase natural evaporation by at least 30%, and drift from the spray system will be largely contained by the extended fringe.

The fringe itself is sloped away from the pond to drain water towards a low point within the fringe from which the water will gravitate towards an inlet point and back into the pond. The water is therefore recirculated through the lined pond area and over the fringe to maximise evaporation. However, under high rainfall conditions the runoff from the respective fringes is directed to the receiving environment after first flush “cleaning” has occurred.

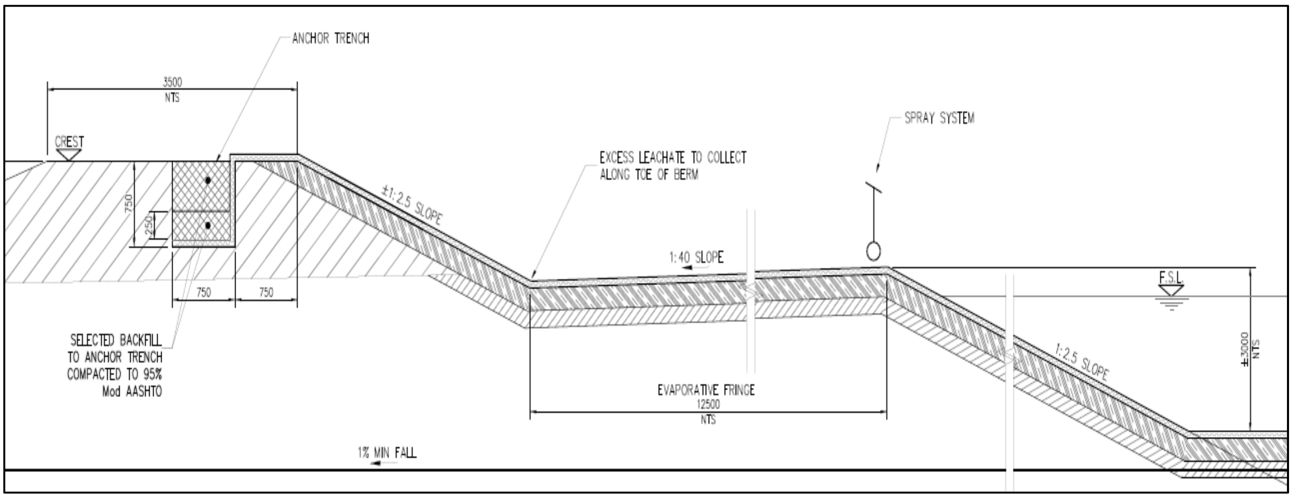

Figure 16: Enhanced evaporation fringe (Drawing 0503)

The system, which is located on the inside of the fringe, sprays water away from the pond through a system of pipes and sprayers linked to a pump system. Each system (stormwater and leachate pond) comprises the following:

- Whirl-jet hollow cone nozzles;
- Spray headers;
- Vertical multistage pumps;

- HDPE piping;
- Pump station (including manifolds, gauges, isolation valves, suction line and foot valve);
- Control panel; and
- Auto-flush strainer bank.

It is noted that power points, electrical cable and electrical installation will need to be provided by Sasol for this installation. The specific pumping requirements are covered in section 12.1.

Factors that affect evaporation from these sprayers include wind, humidity, water temperature, water tension, temperature, dwell time and drop size. It is expected that approximately 5% to 15% of the total volume sprayed is evaporated rom the sprayer system alone.

Additional evaporation could be expected from the fringe. Spray headers will be located at approximately 4.5 m intervals around the fringe, with isolation valves to enable or disable portions of the system as required.

Associated with the evaporative system, is the potential issue of spray drift to the adjacent land surrounding the ponds. This drift is expected to be largely mitigated by the 12.5 m wide fringe and 1 m high berm surrounding the ponds. It is also recommended that the system be automated by linking it to an on-site weather station (included in the system costing). This setup should disable the system when wind speeds exceed a certain value to prevent drift. Further to this, the system may be split into “sectors” allowing for certain spray headers to be disabled or isolated if required.

10.1 Stormwater pond evaporation

The enhanced evaporative spray system on the stormwater pond has been designed to evaporate approximately 30 m^3/day during optimal conditions.

The pumping requirements have been split over two pumps to allow for adjustments or partial use of the system. Each pump on this system needs to supply approximately 317 l/min @ 6 bar. It is recommended, and has been included in the pricing, that an extra pump be included in the system as stand-by. A 63 mm diameter HDPE pipe is required around the fringe with a 110 mm HDPE diameter feed pipe.

10.2 Leachate pond evaporation

The enhanced evaporative spray system on the leachate pond has been designed to evaporate approximately 10 m^3/day during optimal conditions.

The pumping requirements have been split over two pumps to allow for adjustments or partial use of the system. Each pump on this system needs to supply approximately 109 l/min at 6.4 bar. It is recommended, and has been included in the pricing, that an extra pump be included in the system as stand-by. A 50 mm diameter HDPE pipe is required around the fringe with a 75 mm HDPE diameter feed pipe.

10.3 First flush system

The evaporative fringes around the leachate and stormwater ponds have the benefit of enhancing evaporation, but could also potentially collect and route additional rainfall into these pond, adversely affecting their respective water balances. Hence, a first flush system had to be considered.

Given that contaminated leachate and stormwater will be sprayed onto the evaporative fringes, salts and other contamination have the potential to precipitate and accumulate on the fringes. During rain events these salts and contamination will be mobilised. To prevent contamination to the immediate receiving environment surrounding the respective ponds, allowance has been made that the first 10 mm of rainfall be allowed to mobilise these accumulated salts and contamination and route it back into the respective ponds. Thereafter, by means of an adjustable weir system, further rainfall will be routed to the receiving environment. Experience has indicated that after the inception of the first 10 mm rainfall depth the runoff following from further rainfall can be deemed clean for release.

The basic details and concept for the first flush system with weir to control the flow is depicted in Drawing 1418079-0510-D.

11.0 EXCESS WATER ABSTRACTION

Allowance has been made for the installation of a dedicated gooseneck (standpipe) near the ponds area for the abstraction of excess water from the ponds as required. A separate pump has also been included in the design to pump either leachate or stormwater from the respective pond to the gooseneck.

The gooseneck structure consists of a reinforced concrete base upon which a water truck or tanker is able to park while filling with water from the elevated pipe, supported by a steel structure.

11.1 Leachate

11.1.1 Quantity

The outcomes of the abstraction options modelled are presented in Table 10. These options assume that leachate will be abstracted from the pond upon reaching 98% capacity until the volume in the pond is reduced to either 80% or 90%.

Table 10: Leachate abstraction requirements

Pond volume (m^3)	Additional take-off activation rules			Spillage frequency return period (years)	Additional take-off average requirement		Average additional take-off (days/year)
	Rate (m^3/d)	On	Off		Occurrences (per year)	Avg duration of occurrence (days)	
1 500	30	98%	80%	250	2.4	10	24
1 500	30	98%	90%	167	5	4.5	22.5

As per Sasol's requirements, the number of abstraction occurrences was limited to two per year to coincide with site wide maintenance/shut down schedules. The most similar option indicates 2.4 occurrences per year.

Given the spill frequency of 1:250 years for this option compared to the regulatory requirement of 1:50 years), it is considered safe and feasible to reduce the abstraction frequency to twice a year as required by Sasol. The option requires approximately 30 m^3/day of leachate to be abstracted for 10 days on each occasion.

Considering the above, it is recommended that Sasol schedules two abstraction periods per year, abstracting a minimum of 35 m^3/day over 12 days. Ideally, these periods should be scheduled immediately before and after the rain season (i.e. September/October and April/May). This schedule may be further refined in later stages of engineering.

It is noted that the leachate volumes are expected to reduce with time due to the improved management of the landfill through the implementation of the deposition plan.

11.1.2 Treatment/discharge

Given the contaminated nature of the leachate, and possible increase in contamination due to enhanced evaporation, the liquid must be treated prior to being released into the environment. It is recommended that this leachate be sent to Sasol's water treatment works (sewage works).

In order to determine the feasibility of sending the leachate to the treatment works, a high level analysis was conducted. This analysis considered the volume of leachate requiring treatment and the quality based on tested leachate samples (refer to section 13.1 and APPENDIX F) or leachate quality details and results).

With regards to the volume of leachate, an expected 30 m^3/day to 35 m^3/day is considered insignificant when compared to the design flow of the facility which is 15 000 m^3/day.

With regards to the leachate quality, no toxic pollutants are noted in the test results. COD, ammonia and phosphorous levels are sufficiently low, with the only concern being associated with the concentration of TDS (total dissolved solids). The leachate quality discharging in the pond has a TDS of approximately 2 500 mg/l that will increase due to the enhanced evaporation system that the pond is equipped with. If allowance is made for a fivefold increase in leachate concentration, the effect of the discharge of the collected leachate by tanker into the local sewage network could increase the TDS concentration of the treated sewage discharge over the days of discharge by approximately 20 mg/l (delta increase). Given that the design flow for the treatment works is 15 000 m^3/day, and the average dry weather flow is approximately 10 000 m^3/day, the increase in TDS under these conditions would be about 34 mg/l.

The Water Use License (WUL) allows for an increase of 70 mS/m which equates to about 47 mg/l, and is therefore in keeping with regulatory requirements.

11.2 Contaminated Stormwater

11.2.1 Quantity

A summary of the modelled scenarios for abstraction requirements associated with the 15 000 m^3 stormwater pond are presented in Table 11. The scenarios assume varying (as indicated) “on” and “off” capacity percentages for which abstraction is required.

Table 11: Stormwater abstraction requirements

Pond capacity (m^3)	Additional take-off			Spill frequency (1 in X years)	Additional take-off average requirements			Average additional take-off (days/year)
	Rate (m^3/d)	On (%)	Off (%)		Occurrences (per year)	(1 in X years)	Avg duration (days)	
15000	300	90	65	56	0.07	15	5.86	0.40
15000	100	80	70	50	0.12	8	13.79	1.63
15000	50	80	50	50	0.08	12	46.43	3.81

As per Sasol’s requirements, the number of abstraction occurrences was limited to two per year to coincide with site wide maintenance/shut down schedules. As is evident from Table 11, abstraction is not required annually from the stormwater pond, although the volumes requiring abstraction are high. It is therefore recommended that abstraction be scheduled once a year, immediately before or after the rain season (i.e. September/October or April/May). Further modelling during later stages of engineering will analyse more specific requirements for abstraction once a year, however for the purpose of feasibility assessments, it is recommended that allowance be made for 50 m^3/day for a duration of 4 days.

It is noted that the contaminated stormwater volumes are expected to reduce with time due to the improved management of the landfill through the implementation of the deposition plan.

11.2.2 Treatment/discharge

Testing/analysis on contaminated stormwater was not conducted as part of the feasibility assessments. It is however expected that this water will be significantly less contaminated than the leachate. The stormwater contained in the pond should therefore be tested to determine how it could be discharged, with the default to the sewage collection network.

12.0 POWER REQUIREMENTS

The power requirements for the planned development at the Charlie 1 landfill focus on pumping requirements. Lighting requirements should be further assessed in later stages of engineering.

12.1 Pumps

12.1.1 Leachate sump

The leachate sump located in the north-western corner of the landfill site requires a single 1.6 kW pump to send leachate from the sump to the leachate pond. It is recommended that a second identical pump be included to serve as a stand-by pump. The following assumptions have been made in the process of sizing this pump:

- 12 m^3/day flow into sump with the option of pumping twice a day or three times a day;
- For pumping three times a day (every 8 hours) an precast manhole with an internal diameter of 1.532 m was selected;
- The manhole/sump is sealed on the inside to ensure water tightness;
- A flow of 1.67 ℓ/s was calculated to size the pump;
- Pipe diameter selection was done as such to have a flow velocity of above 1.1 m/s to ensure no sediment can settle in the pipe. The pipe diameter to be DN50;
- Pipe material selected is HDPE PE100 PN8;
- The pipe will be continuously welded/100m coil;
- The pipe will be buried;
- 100 m length of pipe line;
- The static head is 6 m and a Total Dynamic Head was calculated as 9 m; and
- No air valves or pressure sustaining valves were included and will only be done at final design stage.

Based on the above a KSB Amarex N F 50 to 170 submersible pump was selected using KSB pump selection software. The pump selected rated voltage is 400 V although confirmation from KSB indicates that the motor can be rewound to 525 V, as per Sasol's requirements. The Motor Rated Power for this pump is 1.3 kW, based on all the assumptions above.

It is however recommended that an additional 20% capacity be added to the kW requirement as a safety margin. This will also allow for flexibility during final design stages if larger pumps are required. For this reason, a 1.6 kW power requirement is recommended.

Additional notes on the pump are as follows:

- The pump runs on very low efficiencies which is common with submersible pumps; and
- The pump comes with a float switch that will allow it to stop and start at pre-determined levels which will ensure that pumping only twice a day can be achieved.

12.1.2 Leachate enhanced evaporation

The enhanced evaporation system for the leachate pond recirculates water from the pond over the fringe. The pumps will deliver the required flow rate and pressure to the system discussed in section 10.0. The pumping requirements have been split over two pumps to allow for adjustments or partial use of the system (isolation of certain sectors if required). The system requirements are as follows:

- 217 l/min at 6.4 bar for the overall leachate system;
- Split into 2 pumps delivering 108.5 l/min at 6.4 bar each;

- 1 additional pump included as standby;
- Pumps are 525 V; and
- The system includes a control panel and various other accessories described in section 10.0.

Based on the above information, 3 kW x 3 kW vertical multistage centrifugal pumps are recommended (one of which is required as standby) for a total power requirement of 6 kW.

12.1.3 Contaminated stormwater enhanced evaporation

The enhanced evaporation system for the stormwater pond recirculates water from the pond over the fringe. The pumps will deliver the required flow rate and pressure to the system discussed in section 10.0.

The pumping requirements have been split over two pumps to allow for adjustments or partial use of the system (isolation of certain sectors if required). The system requirements are as follows:

- 634 l/min at 7 bar for the overall stormwater system;
- Split into 2 pumps delivering 317 l/min at 6 bar each;
- 1 additional pump included as standby;
- Pumps are 525 V; and
- The system includes a control panel and various other accessories described in section 10.0;

Based on the above information, 3 kW x 5.5 kW vertical multistage centrifugal pumps are recommended (one of which is required as standby) for a total power requirement of 11 kW. It is however recommended that for contingency purposes, 15 kW be provided for.

12.1.4 Abstraction

One additional dedicated pump has been included for abstraction requirements. This pump will be linked to the gooseneck structure and may abstract water/leachate from either pond. A pump, capable of delivering 15 m^3/hour, has been included in the design for this purpose. A vertical multistage centrifugal pump with 2.2 kW, 525 V motor is recommended.

12.1.5 Pumps summary

Table 12 presents a summary of the recommended pumping requirements associated with the feasibility designs of this project.

Table 12: Pumps summary

Pump system description	Flow rate and pressure (l/s)	Power requirement (kW)	Contingency (kW)	Total power requirement (kW)
Leachate sump	1.67	1.3	0.3	**1.6**
Leachate enhanced evaporation	3.6	6	-	**6**
Stormwater enhanced evaporation	10.6	11	4	**15**
Abstraction	4.2	2.2	0.2	**2.4**
TOTAL	**20.1**	**20.5**	**4.6**	**25.1**

12.2 Lighting

Basic lighting in the ponds and landfill area is recommended for routine care and maintenance as well as emergencies. Lighting and basic power supply to the site has not been investigated in this feasibility assessment and was not included in costing estimates. It is recommended that this be investigated by an electrical engineer as part of a separate study or in future project phases.

13.0 BARRIER DESIGN

13.1 Leachate assessment

Leachate samples from three of the test pits down slope of the landfill were sent for analysis at Sasol's laboratory. The analytical results as supplied by Sasol were used as received (refer to APPENDIX F). No verification on the quality of the analytical data was done.

The leachate from the test pits was assessed according to the Waste Classification and Management Regulations (WCMR) which was promulgated on 23 August 2013 (GN R.634 of 2013). In terms of Regulation 8 of the WCMR, waste must be assessed in accordance with the Norms and Standards for Assessment of Waste for Landfill Disposal prior to the disposal of waste to landfill (GN R.635 promulgated on 23 August 2013).

The findings of the classification (refer to APPENDIX G for full report), in terms of informing the barrier design for the leachate pond are as follows:

- The assessment results of the leachate in Test pit 6, based on the risk averse principle in Section 2 of the National Environmental Management Act, 1998 (Act 107 of 1998) (NEMA), are used in selecting the Class of barrier/liner design for the contaminated stormwater pond and the leachate pond;
- The leachate quality indicated potential contamination of the groundwater with Mn, Ni, Cl and SO;
- When assessed according to LCT levels of GN R.635, the leachate is classified as a Type 3 waste;
- Based on this assessment, a Class C/G:L:B^+ (GN R.636 of 23 August 2013) liner will be required; and
- Since GN R.636 prescribes landfill designs final endorsement will have to obtained from the Department of Water Affairs and Sanitation to confirm the acceptability of a Class C design for the leachate pond (in certain instances regarded as a hazardous lagoon by the DWS and not as a landfill in terms of its design).

It should be noted that since leachate samples were taken in test pits at the toe of the Site these can be regarded as representative of actual leachate quality potentially migrating into the water resource and hence defensible as a basis for the barrier design (in the absence of TC which is not available since the required *aqua regia* tests cannot be performed on a liquid sample).

13.2 Proposed barrier design

13.2.1 Leachate pond liner

Although the above assessment of the leachate quality indicates a Type 3 waste, which would typically require a Class C barrier system, the assumption has been made that the quality of the leachate will deteriorate as a result of the enhanced evaporation on the pond. This fact, along with the fact that this is a liquid containment facility and not a solid waste facility to which the Class C barrier system would strictly apply, a triple liner system or equivalent Class A system is proposed for the pond. The proposed liner design, from the bottom up, is as follows:

- Subsoil drainage (Drainex DN110 HDPE slotted pipe set in herringbone trench system backfilled with sand);
- 150 mm base preparation layer, rip and compact to 95% Proctor density;

- 200 mm compacted clay layer, compacted to 95% Proctor density;
- 1.5 mm HDPE geomembrane liner;
- Leakage detection layer (cuspated drainage sheet);
- 1.5 mm HDPE geomembrane liner;
- Leakage drainage layer (cuspated drainage sheet); and
- 2 mm HDPE geomembrane liner.

It is noted that the primary liner of a Class A barrier system would normally consist of an HDPE geomembrane overlying a compacted clay liner or geosynthetic clay liner (GCL) equivalent.

However, in the case of a GCL being used, the GCL requires a permanent confining pressure of at least 35 KPa, which cannot be guaranteed in a liquid pond. Therefore the primary liner has been changed to a double geomembrane system with a leakage drainage layer between the two geomembranes, from which any leakage that occurs through the top geomembrane will be monitored, captured and returned into the leachate pond.

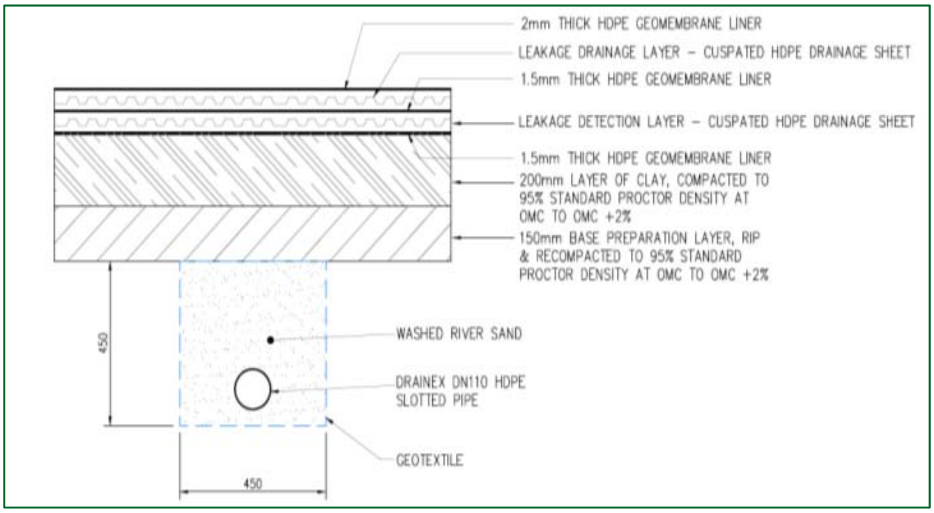

Figure 17: Leachate Pond liner system design

13.2.2 Contaminated stormwater pond liner

Although no testing was possible, based on the assumption that contaminated stormwater is of a significantly lower contamination level than the leachate, the following liner system design is proposed for the contaminated stormwater pond, which is equivalent to a Class B landfill barrier system. The proposed liner system comprises the following layers, from bottom upwards:

- Subsoil drainage (Drainex DN110 HDPE slotted pipe set in herringbone trench system backfilled with sand);
- 150 mm base preparation layer, ripped and recompact to 95% Proctor density;
- 200 mm compacted clay layer, compacted to 95% Proctor density;
- 1.5 mm HDPE geomembrane liner;
- Leakage detection layer (cuspated drainage sheet); and
- 2 mm HDPE geomembrane liner.

Note that the primary liner consists of a double geomembrane system for the same reasons as discussed above for the leachate pond liner system.

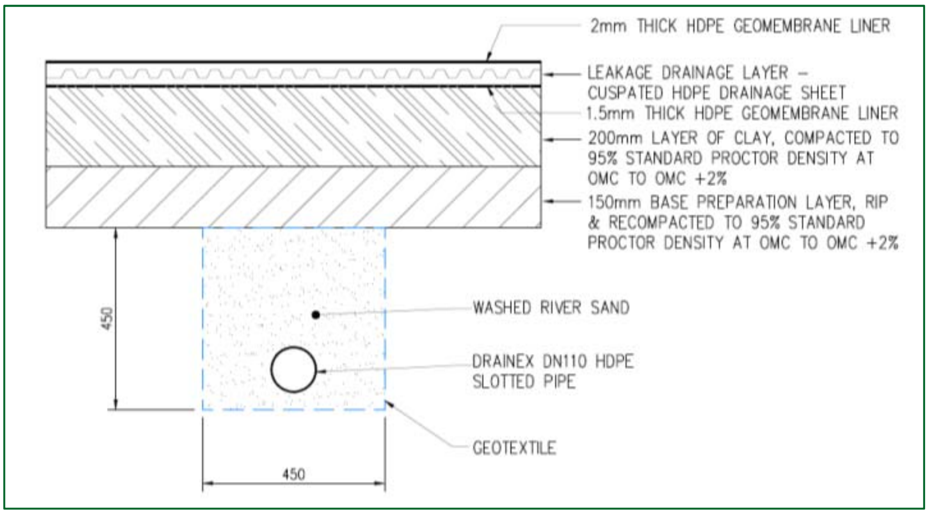

Figure 18: Contaminated Stormwater Pond liner system design

13.3 Leakage detection and drainage sumps

The leakage drainage and leakage detection layers described in the above barrier designs drain to respective sumps in each pond. The stormwater pond has a leakage drainage sump, while the leachate pond has both a leakage drainage sump and a leakage detection sump due to its triple liner system. These sumps allow for collection and monitoring of leakage from the ponds.

The sumps are constructed on the pond floor by excavating an area of approximately 4 m x 4 m, 0.6 m deep. This excavation, which is lined, is filled with stone around an HDPE drainage pipe which leads to a monitoring point. In this case, the pipe penetrates the liner and drains to a manhole located adjacent to the pond.

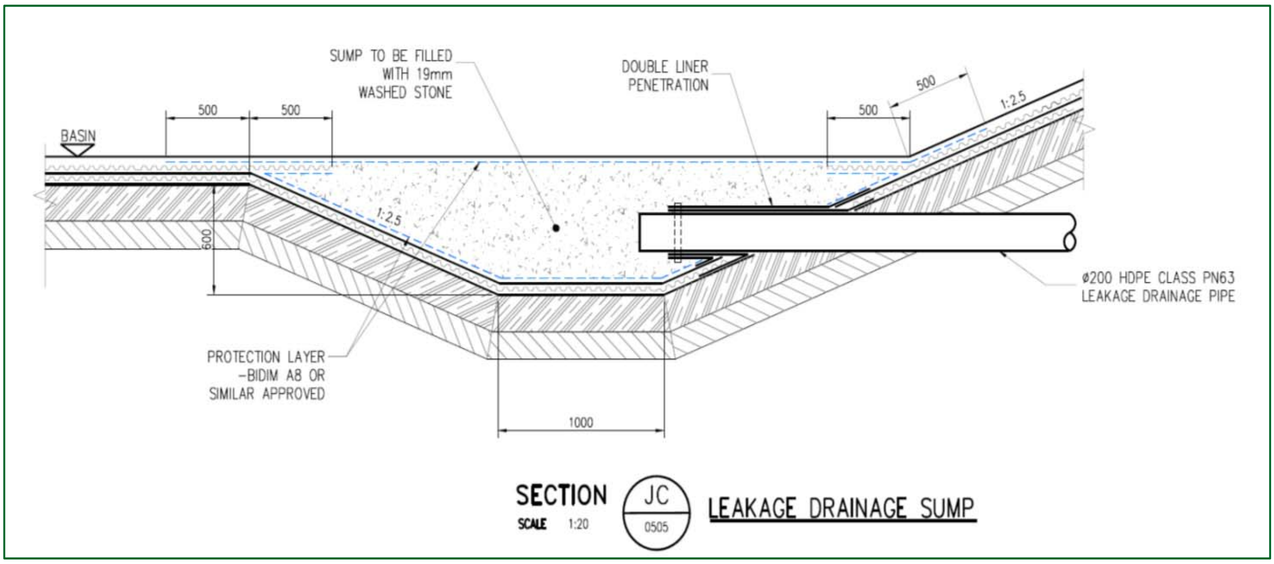

Figure 19: Pond leakage sump

14.0 LANDFILL HEIGHT AND AESTHETICS

Various options for the extension of the Charlie 1 landfill in terms of landfill design parameters and aesthetic considerations were considered as part of the pre-feasibility study.

A viewshed analysis modelling *(Sasol Charlie 1 Dump: Viewshed analysis to determine visibility of various dump heights and initial screening options*; number 12614891_Techmemo_002; Golder, 2013) was conducted for unscreened landfill heights of 5 m, 10 m, 15 m and 20 m.

The results for all the options were largely similar, with the landfill expected to be visible from more than 80% of the target area for all unscreened options, including from the casino, mall and most of the surrounding residential neighbourhoods.

The follow-up visual assessment of 2015 concluded that Charlie 1 Gate is arguably the most sensitive visual receptor location in terms of the planned expansion of the landfill, as it is located directly adjacent to the existing landfill site (see Figure 20). Three key receptors (Charlie 1 Gate, Graceland Casino & Hotel and Secunda Mall) were identified by Sasol that would likely be impacted by the aesthetics of the landfill in general.

It is expected that the visual impact can be significantly mitigated by planting a tree and shrub screen along the perimeter of the site (see Figure 21), regardless of the final height of the landfill. Conversely, the extent to which the landfill can be screened is somewhat limited from the other receptor locations (Hotel and Mall), as in most instances the viewer will be able to see over a vegetative screen that is planted alongside the landfill. However, the severity of the visual impact is reduced due to the distance between the landfill and these receptor locations; and is expected to be further mitigated once the landfill has been re-profiled and vegetated. In addition, the landfill will in some instances actually serve to screen parts of the Sasol complex from view.

For detailed information on the above mentioned follow-up viewshed analysis, refer to APPENDIX H *(*Sasol Synfuels Charlie 1 landfill: visual assessment modelling to determine potential screening effectiveness of vegetative barriers; number 1418079_Techmemo_006; Golder, 2015).

Figure 20: Existing scenario, from Charlie 1 Gate

Figure 21: 15 m landfill height with tree and shrub screen, from Charlie 1 Gate

14.1 Landfill height

In consultation with Sasol, it was decided that for the purpose of this feasibility assessment, a landfill height of 15 m would be assumed that is motivated as follows:

- The large area of the landfill footprint, combined with the size of the incoming waste stream, does not warrant a landfill height in excess of 15 m;
- The large footprint, with relatively low overall height of 15 m facilitates gentle final landfill side slopes; and
- The generally flat topography associated with the surrounding area will result in the landfill being visible from most surrounding locations. A relatively low height of 15 m coupled with the implementation of the vegetative screen will largely mitigate this situation.

15.0 DEVELOPMENT PLAN AND LANDFILL SITE LIFE

The development plan describes the proposed waste placement/deposition strategy to optimise operations and site life at the Charlie 1 Landfill Site. It also proposes some initial steps to shape the existing landfill waste body to reduce leachate and contaminated stormwater generation. The development plan steps are depicted in Drawings Set 1418079-0600-D, with a summary of the airspace information provided in Table 13. Although the Charlie 1 Landfill has no clearly defined waste cells, the development plan has split the landfill site into six separate areas.

15.1 Site life

Two scenarios have been analysed in calculating the expected life of the landfill, namely the best case scenario and the worst case scenario. These are based on low and high records of actual waste volumes entering the Charlie 1 Landfill, as supplied by Sasol (APPENDIX J). The following assumptions were made for both scenarios:

- Incoming waste volumes (m^3) are quoted as uncompacted volumes;

- Estimated densities of the incoming waste streams have been assumed as follows, based on experience and the literature:
  - Domestic waste (delivered by compactor vehicle) = 450 kg/m^3;
  - Builder's rubble = 1 500 kg/m^3;
  - Garden waste = 300 kg/m^3;
  - Cover material (mainly soil) = 1 400 kg/m^3; and
  - Insulation waste and spent catalyst are combined with domestic waste to assume 450 kg/m^3.
- All landfilled waste is compacted to an average of 1 tonne/m^3, including daily cover (additional cover material is assumed to be compacted to 1.8 tonne/m^3);
- Cover material is stockpiled separately and used for daily cover and/or interim capping;
- A cover to waste (V:V) ratio of 1:5 is maintained;
- Cover material not used for daily cover and/or interim capping is landfilled (compacted to 1.8 tonne/m^3);
- Maximum landfill height is 15 m above NGL; and
- Recycling accounts for approximately 3.5% of incoming waste (based on 2012 data used in Golder's pre-feasibility study).

The best case scenario (longest site life, lowest waste stream), based on 2014 incoming waste volumes provided by Sasol and the above assumptions, gives an average airspace utilisation of 93 000 m^3 per annum. The worst case scenario (shortest site life, highest waste stream), based on 2012 incoming waste volumes and the above assumptions, gives an average airspace utilisation of 152 000 m^3 per annum. It is noted that the 2012 increased waste quantities were largely due to a significant increase in cover material received by the landfill during that year. This is probably due to increased building activities at the Sasol plant during this time and therefore this quantity has been used for the upper estimate.

In accordance with the *Minimum Requirements* the site life calculation takes into account a cover to waste ratio of 1:5. Cover material is placed on a daily basis to ensure sanitary operational procedures.

The phases of development presented in Table 13 are described in detail the deposition sequence (section 15.5). Each phase represents the landfilling of a section of the landfill to a certain height.

Table 13: Capacity and site life

Step	Airspace (m^3)	Cumulative airspace (m^3)	Site life (years) 93 000 m^3/year	Cumulative life (years) 93 000 m^3/year	Site life (years) 152 000 m^3/year	Cumulative life (years) 152 000 m^3/year
Phase 1	269 412	269 412	2.9	2.9	1.8	1.8
Phase 2	199 848	469 260	2.1	5.0	1.3	3.1
Phase 3	107 270	576 530	1.2	6.2	0.7	3.8
Phase 4	202 663	779 193	2.2	8.4	1.3	5.1
Phase 5	327 980	1 107 173	3.5	11.9	2.2	7.3
Phase 6	367 148	1 474 321	3.9	15.9	2.4	9.7
Phase 7	296 347	1 770 668	3.2	19.0	1.9	11.6
Phase 8	433 420	2 204 088	4.7	23.7	2.9	14.5

Step	Airspace (m^3)	Cumulative airspace (m^3)	Site life (years) 93 000 m^3/year	Cumulative life (years) 93 000 m^3/year	Site life (years) 152 000 m^3/year	Cumulative life (years) 152 000 m^3/year
Phase 9	315 075	2 519 163	3.4	27.1	2.1	16.6
TOTAL	**2 519 163**			**27.1 years**		**16.6 years**

15.2 Sanitary landfilling

The following general rules of sanitary landfilling (from the *Minimum Requirements* Series) should be followed during each of the steps described in the deposition sequence (section 15.4):

- **Waste must be compacted**. This is best achieved by spreading waste into thin layers and compacted with a landfill compactor; and
- **Daily cover is to be applied** at the end of each working day. The compacted thickness of this layer should be approximately 150 mm. Current incoming quantities of cover material at the Charlie 1 Landfill are more than sufficient to maintain a 1:5 (cover : waste) ratio. Surplus material is to be stockpiled on site for use in interim/final capping.

15.3 Methods of landfilling

The **standard cell operation method**, as per the *Minimum Requirements*, should be used at the Charlie 1 Landfill. The standard cell operation method requires the formation of smaller cells within the larger cell or area being utilised. The smaller cells are to be constructed by forming 1.5 m to 2 m high berms, constructed from soil, rubble or waste (covered). The working face (active part of the landfill where waste is deposited) is to be kept as small as possible for control and covering purposes, however, it should be wide enough to avoid traffic congestion. There should always be sufficient cell capacity to accommodate at least one week's waste. Waste should be deposited at the bottom of the working face and worked up a 1 in 3 slope. Cover material may then be deposited at the top of the cell during the day and extended to cover the working face at the end of the day.

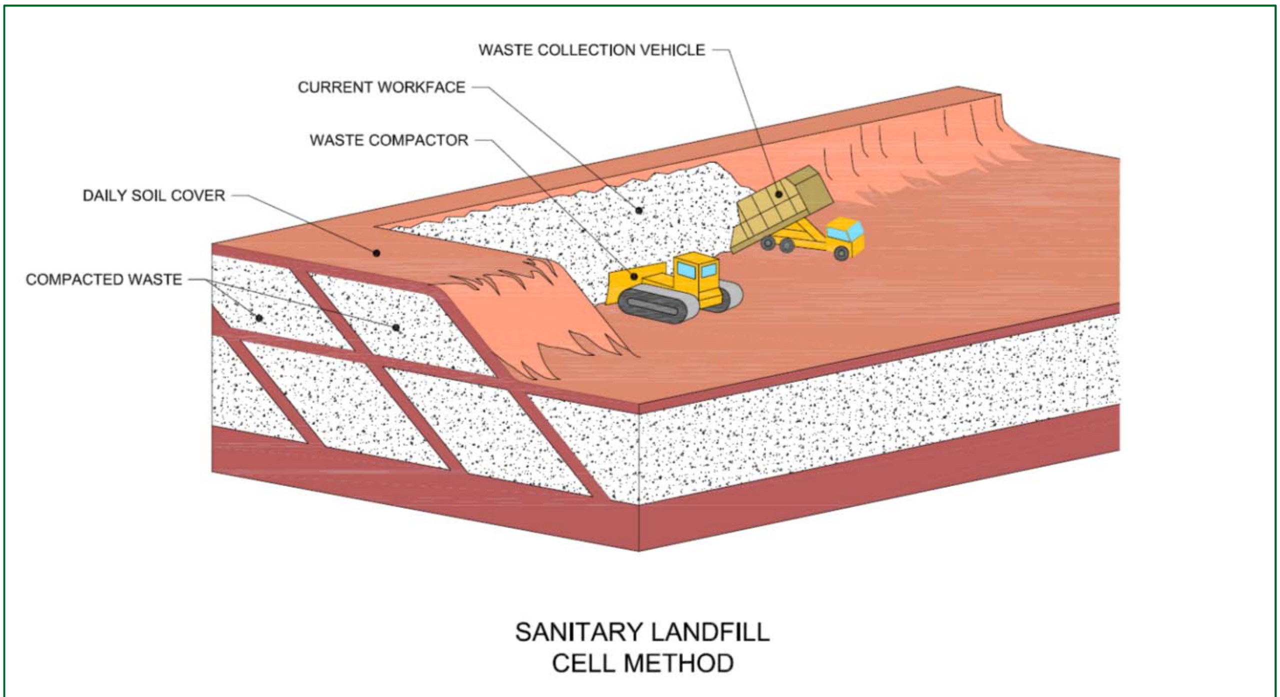

Figure 22: The standard cell operation method

A **wet weather cell** is to be formed close to the entrance of the larger cell/area in use. This cell is for use during or immediately after abnormally wet conditions and must have sufficient capacity to accommodate one week's waste. The wet weather cell should be operated in the same manner as the standard cell operation method.

The wet weather cell shall operate on a surface which is of coarse material and well drained, such as builders' rubble, to ensure vehicle access in extreme wet conditions.

15.4 Cover material stockpile

High volumes of topsoil and other soils are received by the Charlie 1 Landfill; however the regularity of this inflow is unpredictable. For this reason it is proposed that separate stockpiles of cover material and topsoil be maintained on site. In this case, topsoil and cover material are defined as follows:

- Topsoil – clean, uncontaminated soil that is able to support vegetation growth and can therefore be used as final or interim cover; and
- Cover material – mixed soil and construction rubble that would not normally support vegetation growth, but could however be used for daily cover operations as well as for interim cover material. Potential cover material should be scrutinised to ensure that it is not contaminated with hazardous substances such as hydrocarbons or the like, which would render it unsuitable for use on a general waste landfill.

The locations of these stockpiles should be strategically placed in order to optimise operations. It is recommended that the cover material stockpile be a "moving" stockpile located on or immediately adjacent to the active landfill area (refer to the areas specified in the deposition sequence), in close proximity to the working face. The topsoil stockpile should be located on a "clean" area of the site for use as interim of final cover.

The approximate volumes of interim/final cover required for each "Area" described in the deposition sequence are presented in Table 14. This volume should be maintained in the stockpile when the relevant landfill area approaches capacity (in this case, capacity refers to the end of the relevant phase of development as indicated in the deposition sequence).

Table 14: Interim/final cover volumes

Area	Required volume (m^3)
1	20 000
2	18 000
3	20 000
A	18 000
B	15 000
C	10 000

15.5 Deposition sequence

The deposition sequence initially considers the shaping and interim covering of the southern half of the landfill while deposition continues on the northern half. As areas are covered, stormwater diversions (refer to Drawing Figure 23) may be implemented to divert clean runoff away from the site. Operational or uncovered areas will direct the "contaminated" runoff to the contaminated stormwater pond.

Eventually, areas will become operational in an anti-clockwise direction, with the previously landfilled area being temporarily or permanently capped. For further clarity on this matter, refer to the descriptions of the individual development phases below.

Shaping

Note: It should be noted that the timing of the shaping activities proposed in Phases 1 to 3 may take place before waste deposition in the relevant areas associated with these phases is complete. Ideally, the shaping of these areas (A, B and C) should take place as soon as possible and should not be held up by the waste deposition sequence. Quantities and costing for this initial shaping have been determined and are included in the capital cost estimate.

Stormwater diversion

The stormwater diversions are implemented at various locations along the northern and southern stormwater drains (3 diversions on each drain, refer to Drawing 1418079-0601-D) corresponding to the boundaries of the development plan "areas". The diversion consists of an area where the v-drain becomes flat, with brick walls guiding the water either along the concrete v-drain or out of the concrete v-drain to drain freely into the environment. Figure 23 indicates which of the walls are permanent and which are temporary. The temporary walls may be constructed or removed as required during the various phases of development.

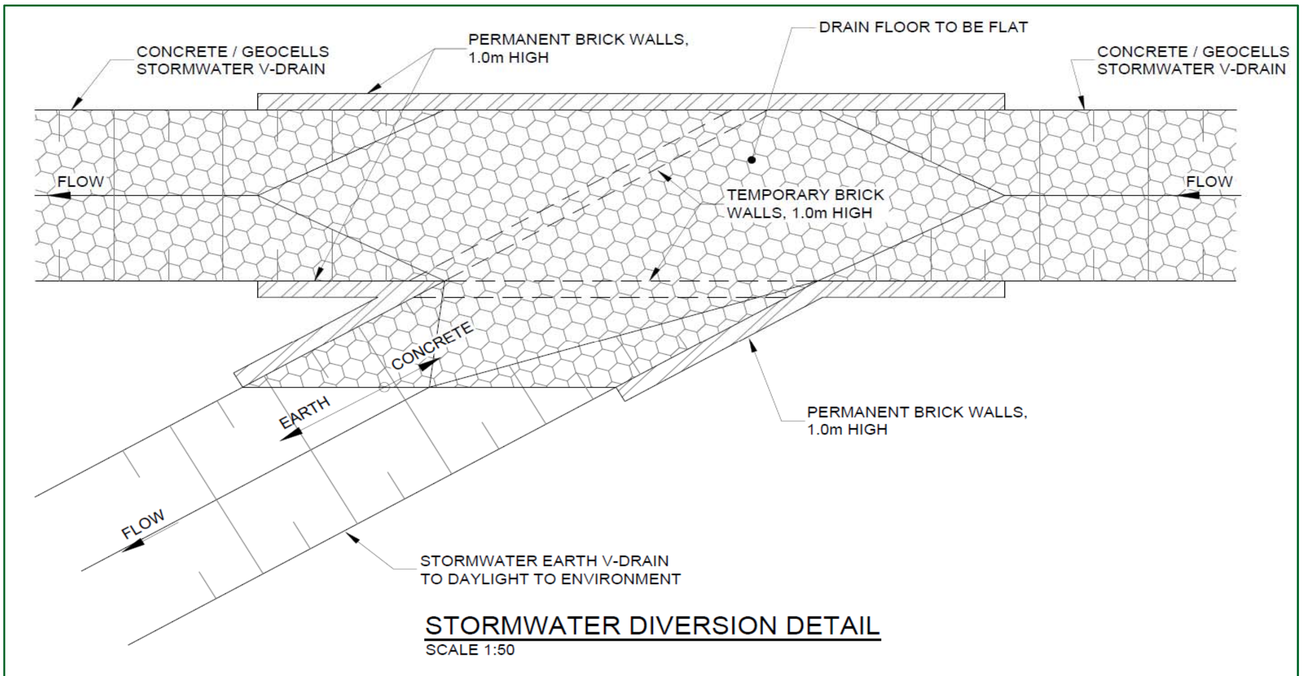

Figure 23: Stormwater diversion detail

Capping

It should be noted that interim capping of the various areas of "development" once complete is applied in Phases 1 to 4, while final capping is to be applied from Phases 5 to 9. This is based on the final landfill height of 15 m above ground level. Interim capping is implemented by means of a 300 mm thick layer of soil/ cover material while final capping is depicted in Drawing (for further details on final capping, please refer to section 17.0).

15.5.1 Phase 1

The first phase of development of the landfill involves the landfilling of "Area 1" in the north-east to a total height of 1 615 mamsl (see Figure 24). The standard cell method of operation is to be implemented in this area and side slopes are to be maintained at 1:4. Approximate values associated with this Phase are as follows:

- Airspace = 269 400 m^3; and
- Life = 1.8 to 2.9 years.

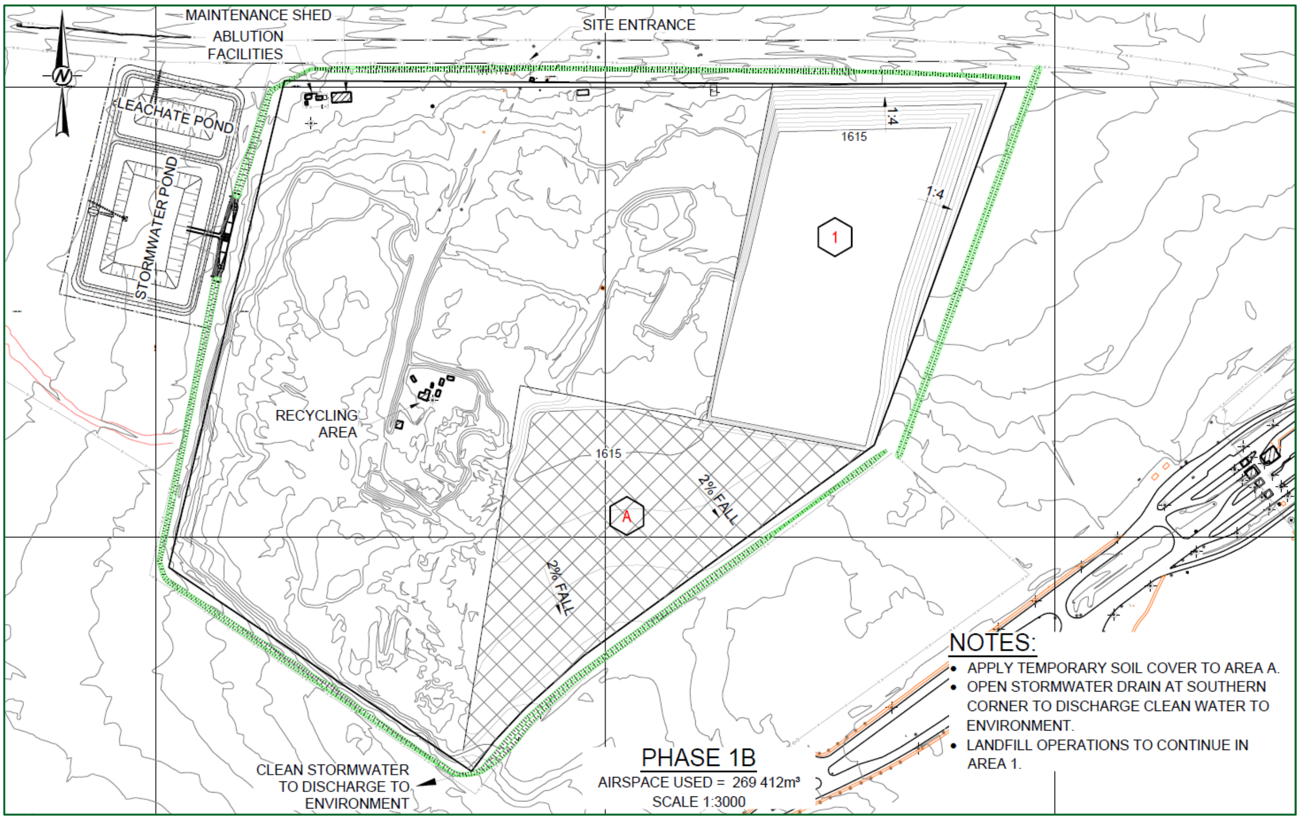

Figure 24: Development plan - Phase 1

Shaping "A"

Concurrently or prior to this taking place, "Area A" is to be shaped from a height of 1 615 mamsl centrally in the landfill, with a 2% fall towards the site boundary and stormwater drains. Once shaped, the area is to receive interim capping by means of a 300 mm soil/cover material layer. The stormwater diversion structure located in the south-east corner of the site is to route clean water off site once interim capping has been applied. It should be noted that the Phase is split up into 1A and 1B for description purposes only, with timing of the shaping operations to be considered separately.

15.5.2 Phase 2

Upon completion of Phase 1, interim capping by means of a 300 mm thick layer of soil/cover material is to be applied to Area 1. The north-eastern stormwater diversion structure is to divert clean runoff away from the site. The second phase of development of the landfill involves the landfilling of "Area 2" to a total height of 1 615 mamsl, joining Area 1. The standard cell method of operation is to be implemented in this area and side slopes are to be maintained at 1:4. Approximate values associated with this Phase are as follows:

- Airspace = 199 800 m^3; and
- Life = 1.3 to 2.1 years.

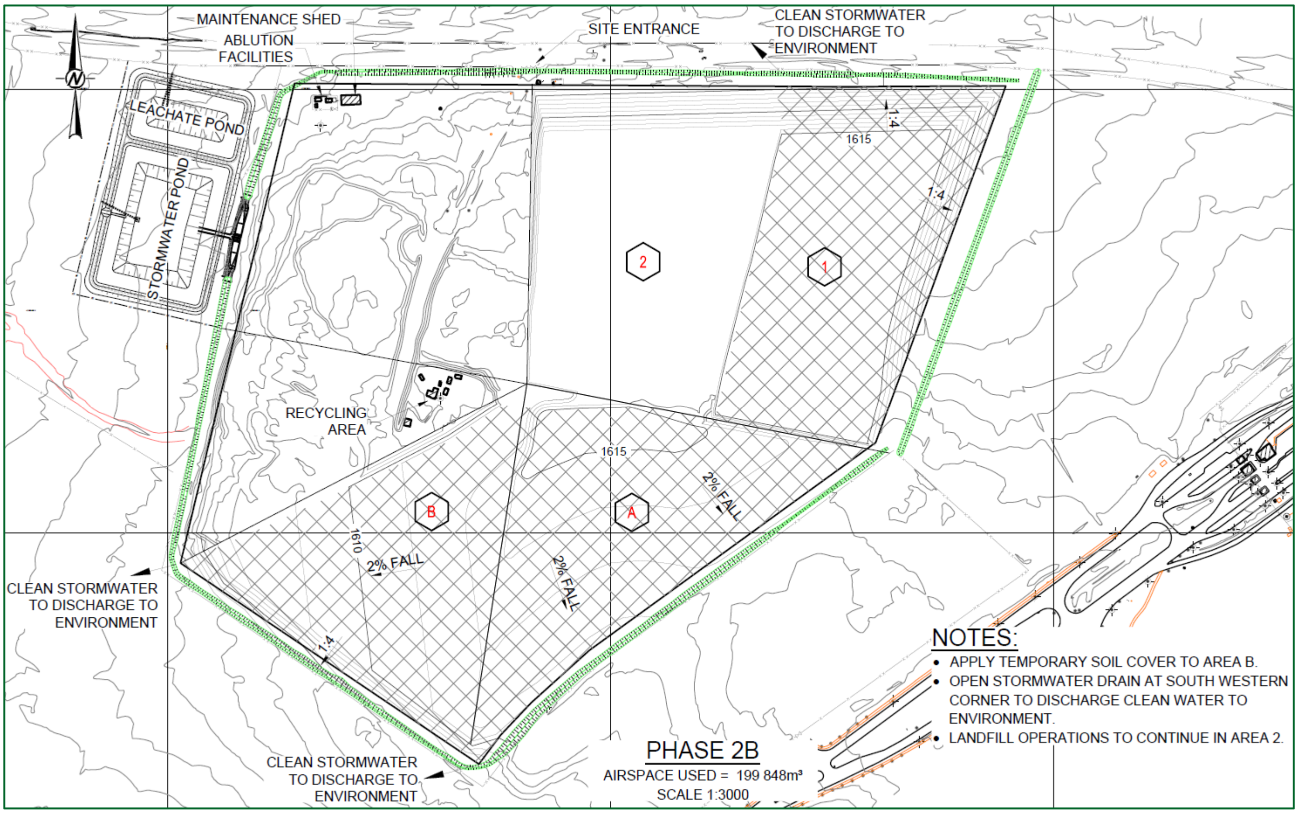

Figure 25: Development plan - Phase 2

Shaping “B”

Concurrently or prior to this Phase, “Area B” is to be shaped from a height of 1 615 mamsl centrally in the landfill, with a 2% fall towards the site boundary and stormwater drains. Once shaped, the area is to receive interim capping by means of a 300 mm soil/cover material layer. The stormwater diversion structure located in the south-west corner of the site is to route clean water off site once interim capping has been applied. The Phase is split up into 2A and 2B for description purposes only, with timing of the shaping operations to be considered separately.

15.5.3 Phase 3

Upon completion of Phase 2, interim capping by means of a 300 mm thick layer of soil/cover material is to be applied to Area 2. The north-central stormwater diversion structure is to divert clean runoff away from the site. The third phase of development of the landfill involves the landfilling of “Area 3” to a total height of 1 615 mamsl, joining Area 2. The standard cell method of operation is to be implemented in this area and side slopes are to be maintained at 1:4. Approximate values associated with this Phase are as follows:

- Airspace = 107 300 m^3; and
- Life = 0.7 to 1.2 years.

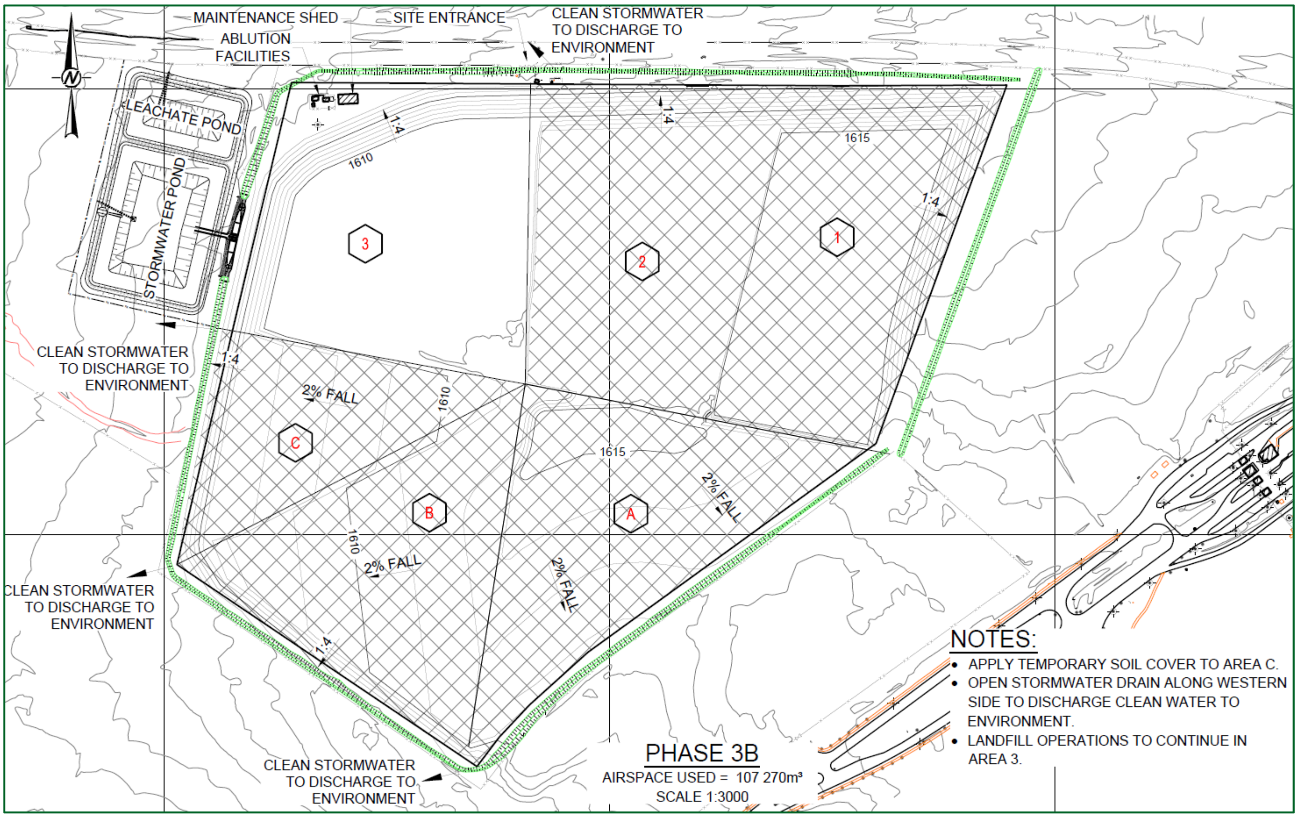

Figure 26: Development plan - Phase 3

Shaping "C"

Concurrently or prior to this Phase, "Area C" is to be shaped from a height of 1 615 mamsl centrally in the landfill, with a 2% fall towards the site boundary and stormwater drains. Once shaped, the area is to receive interim capping by means of a 300 mm soil/cover material layer. The stormwater diversion structure located immediately south of the ponds area is to route clean water off site once interim capping has been applied. The Phase is split up into 3A and 3B for description purposes only, with timing of the shaping operations to be considered separately.

15.5.4 Phase 4

Upon completion of Phase 3, interim capping by means of a 300 mm thick layer of soil/cover material is to be applied to Area 3. The north-western stormwater diversion structure is to divert clean runoff away from the site. The fourth phase of development involves the landfilling of "Area C" to a total height of 1 620 mamsl, shaping the top surface to form a 2% fall towards the western landfill side slope. The standard cell method of operation is to be implemented in this area and side slopes are to be maintained at 1:4. All stormwater diversions, except the diversion immediately south of the ponds, are to direct runoff away from the site. The diversion located immediately south of the ponds is to direct stormwater into the drain system towards the ponds. From Phase 4 until closure after Phase 9, the operational areas become active/closed in an anti-clockwise direction. Approximate values associated with Phase 4 are as follows:

- Airspace = 202 700 m^3; and
- Life = 1.3 to 2.2 years.

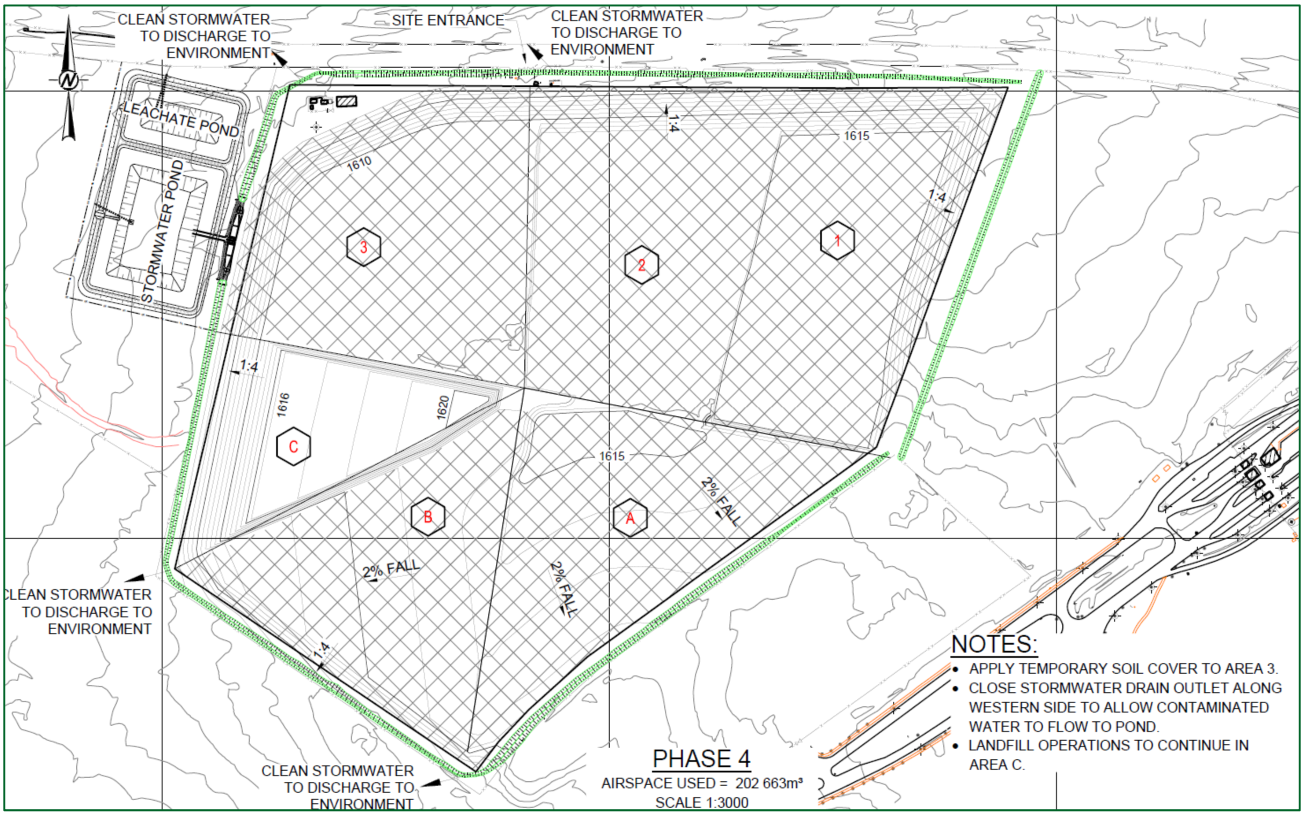

Figure 27: Development plan - Phase 4

15.5.5 Phase 5

Upon completion of Phase 4, final capping (refer to Drawing 1418079-0605-D) is to be applied to Area C. All of the northern stormwater diversion structures, along with the south-eastern diversion are to divert stormwater away from the site. The fifth phase of development involves the landfilling of “Area B” to a total height of 1 620 mamsl, shaping the top surface to form a 2% fall towards west, joining Area C. The standard cell method of operation is to be implemented and side slopes are to be maintained at 1:4. Along the southern boundary of the site, the two western-most stormwater diversion structures are to divert runoff along the drains to the ponds. Clean runoff from the capped surface of Area C is to be directed onto Area 3 so as to reduce the amount of water entering the contaminated water pond. Approximate values associated with this Phase are as follows:

- Airspace = 328 000 m^3; and
- Life = 2.2 to 3.5 years.

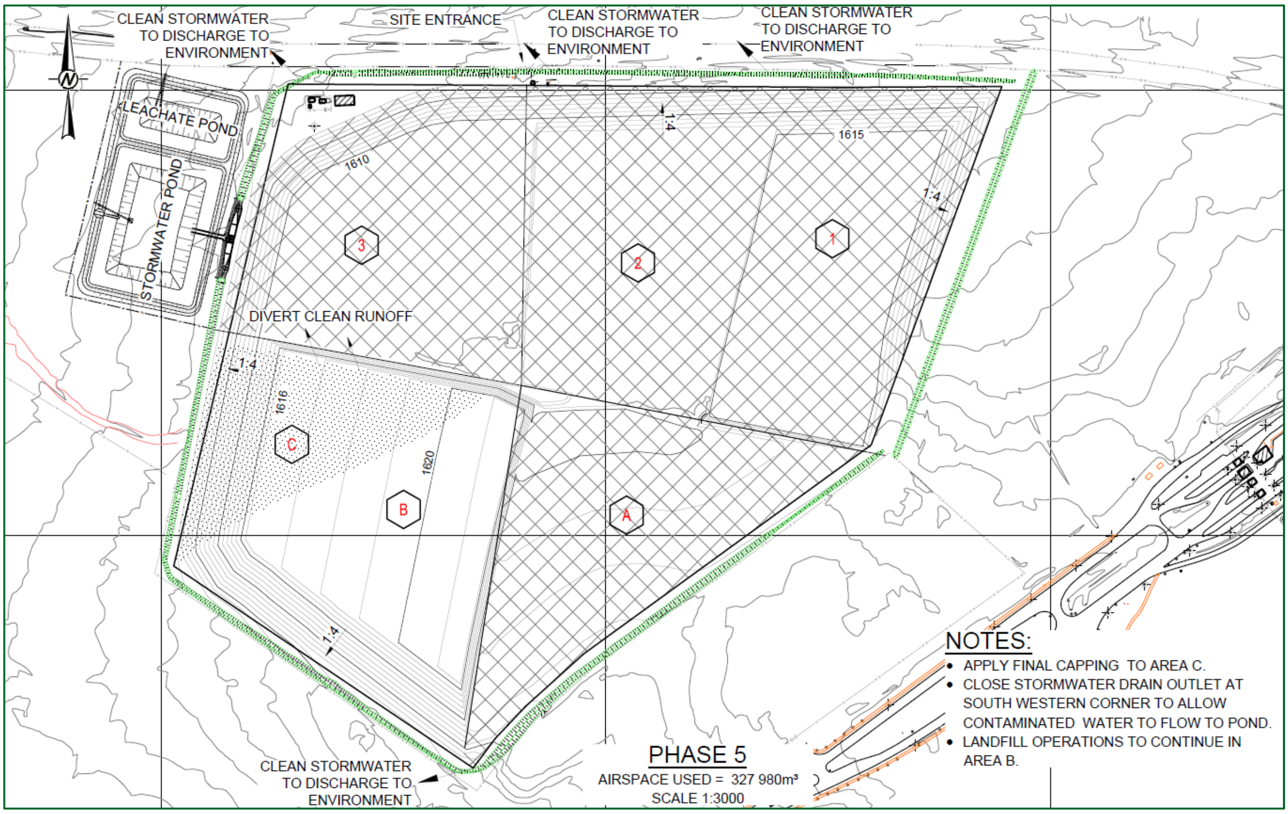

Figure 28: Development plan - Phase 5

15.5.6 Phase 6

Upon completion of Phase 5, final capping is to be applied to Area B. All of the northern stormwater diversion structures are to divert stormwater away from the site, while the southern diversion structures are to send water to the ponds. The sixth phase of development involves the landfilling of “Area A” to a total height of 1 625 mamsl, shaping the top surface to form a 2% fall towards the west and joining Area B. The standard cell method of operation is to be implemented and side slopes are to be maintained at 1:4. Clean runoff from the capped surface of Areas B and C is to be directed onto Area 3 by means of surface berms so as to reduce the amount of water entering the contaminated water pond. Approximate values associated with this Phase are as follows:

- Airspace = 367 100 m^3; and
- Life = 2.4 to 3.9 years.

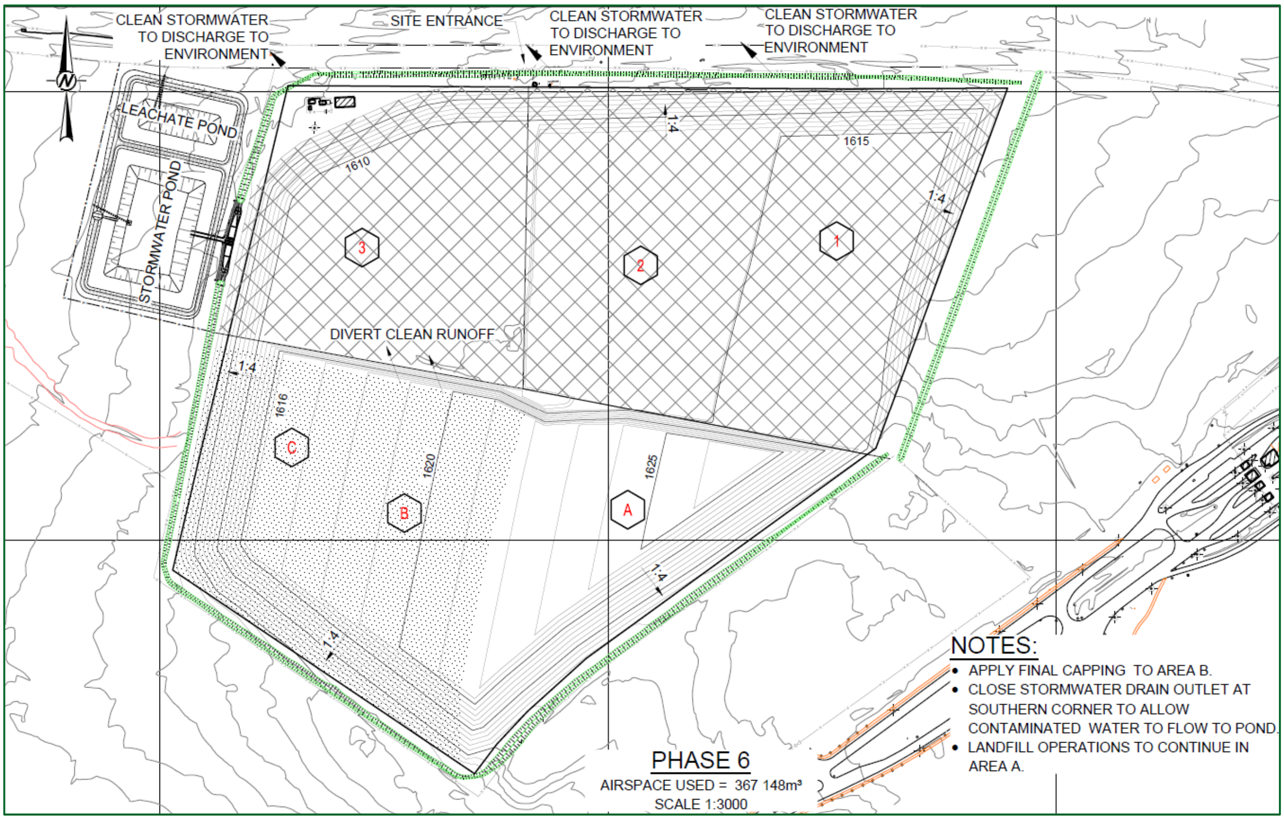

Figure 29: Development plan - Phase 6

15.5.7 Phase 7

Upon completion of Phase 6, final capping is to be applied to Area A. All of the southern stormwater diversion structures are to divert stormwater away from the site, while the northern diversion structures are to send water to the ponds. The seventh phase of development involves the landfilling of “Area 1” to a total height of 1 625 mamsl, shaping the top surface to form a 2% fall towards the north-west and joining Area A. The standard cell method of operation is to be implemented and side slopes are to be maintained at 1:4. Runoff from the final capped surfaces of Areas A, B and C is to be directed into the southern stormwater drain, and diverted to the environment at the various diversion structures. Runoff from the interim capped surface of Areas 2 and 3 is to be directed to the south western corner of Area 3 by means of surface berms, to discharge into the environment. Runoff from the landfilling operations in Area 1 is to be collected in the northern stormwater drain to discharge into the contaminated water pond. Approximate values associated with this Phase are as follows:

- Airspace = 296 300 m^3; and
- Life = 1.9 to 3.2 years.