BUFFELSDRAAI LANDFILL
A New Regional Landfill
for the eThekwini Council

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SUMMARY

Buffelsdraai Landfill is one of three regional sites planned by the Cleansing and Solid Waste Department of the eThekwini Municipality which will receive waste generated within the Metro Area for the next 50 years. It is located in the northern suburbs approximately 8km west of the small town of Verulam and will serve the area north of the Umgeni River. The site is currently being developed and is due to be commissioned early in 2006.

The identification, planning and design of the site has followed the procedures laid out in the document published by the Department of Water Affairs (DWAF) 'Minimum Requirements for the Disposal of Waste by Landfill' second edition. The process which commenced in 1996 was only completed in 2003, has taken far longer than anticipated.

INTRODUCTION

The Cleansing and Solid Waste Department of the eThekwini Council (DSW) is responsible for the collection and disposal of municipal waste within the Metropolitan area. Before the amalgamation of the Local Councils to form the Metropolitan Council, DSW operated within the old city boundaries, with one landfill at Bisasar Road. Following the amalgamations, DSW's responsibilities were in July 1996, expanded to include the entire Metropolitan Area. They inherited sixteen small sites previously operated by the various local councils.

Only the La Mercy (just outside Tongaat) and Inanda sites were permitted. After reviewing the situation DSW decided to close all the small sites with the exception of the above two. Unfortunately public pressure virtually immediately forced the closure of the Inanda site. The La Mercy landfill was upgraded and investigations initiated for its possible expansion. At the same time a new landfill at Mariannhill near Pinetown was commissioned resulting in three active Municipal landfills serving the Metropolitan Area.

In early 1996 a study was undertaken to determine the landfill requirements of the Metropolitan area. It also confirmed the limitations of potential air space at the existing sites and their relatively short lives. Recognising the difficulties of trying to establish new sites (public reaction and environmental considerations) and the problems of operating several small sites to an acceptable standard, DSW adopted the strategy that three regional sites be identified which would provide sufficient airspace to serve the Metropolitan Area for the foreseeable future (50 to 70 years). The three sites would ideally be located in the north, west and south of the Metropolitan area. Pressure to curtail development of La Mercy landfill due to its proximity to the proposed new airport (King Shaka) and its location in an area identified as a tourist/residential area, meant that the development of a new site in the north was critical.

SITE IDENTIFICATION

The process of identifying the three regional sites ran concurrently. A project team was appointed and a Technical Working Group formed, with representation from the various stakeholders.

In the northern area, following a review of the geology, land use, hydrology and other factors, areas in which potential may exist for the development of a landfill were identified. These areas were studied in more detail and several potential landfill sites were identified. Further investigations revealed only two sites which appeared to fulfill DSW's criteria of:

- Available air space ± 40 to 50 x $10^6 m^3$
- Good access
- Large Buffer area

Both sites were near the small town of Verulam and were on privately owned sugar cane farms.

At this stage Working Groups had been established for each area and representation was expanded to include the local communities. Capacity building workshops were held to educate the community representatives about the waste disposal process and the need for landfills. Despite this, the public participation phase was protracted, with the site selection process being viewed with suspicion by some sections of the community. At times information presented by the various consultants was disbelieved and vested interests became a major factor with a group pressing for the one site over the other.

The technical assessment of the two sites presented to the Working Group revealed that Buffelsdraai was the preferred site and this was only accepted after protracted discussions and meetings.

SITE PERMITTING & REZONING

The establishment of a landfill requires three issues to be addressed:

1. An Environmental Impact Assessment (E.I.A.) must be undertaken and a Record of Decision issued by the Department of Agriculture & Environmental Affairs (DAEA).

2. The land must be rezoned.

3. A permit to operate a landfill must be issued by DWAF.

Work on all three proceeded virtually concurrently. Specialist Environmental and Town Planning consultants were appointed to handle the E.I.A. and rezoning respectively.

The E.I.A. required a variety of specialist studies and experienced sub-consultants were appointed in the following fields:

- Geotechnical/Civil Engineering
- Geohydrology
- Odour Impact
- Social Impact Assessment
- Visual Analysis
- Planning & Land use
- Vegetation Assessment

For the rezoning, it was decided at an early stage to use the Development Facilitation Act (DFA) as it was thought this would streamline the process. During the public hearings required as part of the DFA procedures the same issues and vested interests evident during the public participation stage resurfaced. It was apparent that sections of the community were being manipulated to obstruct the rezoning of the Buffelsdraai site with the intention of making the alternative site more appealing. In terms of the DFA a decision was made to support the rezoning however appeals were submitted which resulted in lengthy delays whilst the Town Planning commission sorted itself out and the rezoning was only approved in 2003.

The Permit application requires that a conceptual design be prepared which confirmed the following:

i) Site area including buffer : 750ha

ii) Landfill footprint : 100ha

iii) Classification : G:L:B+

a) Waste type: The site is intended to handle general municipal solid waste only and hence is classed G.

b) Size: The waste stream is expected to grow from 400 tonnes per day when the site opens to 1500 tonnes per day when Bisasar Road closes in 2013. The daily mass of waste deposited requires that the site be classified as Large (L).

c) Leachate: The mean annual rainfall averaged from five surrounding weather stations is 947mm, generally exceeds the S-pan rate of evaporation. The site will produce leachate and is therefore classified as B+.

iv) Site life : 50 to 70 years

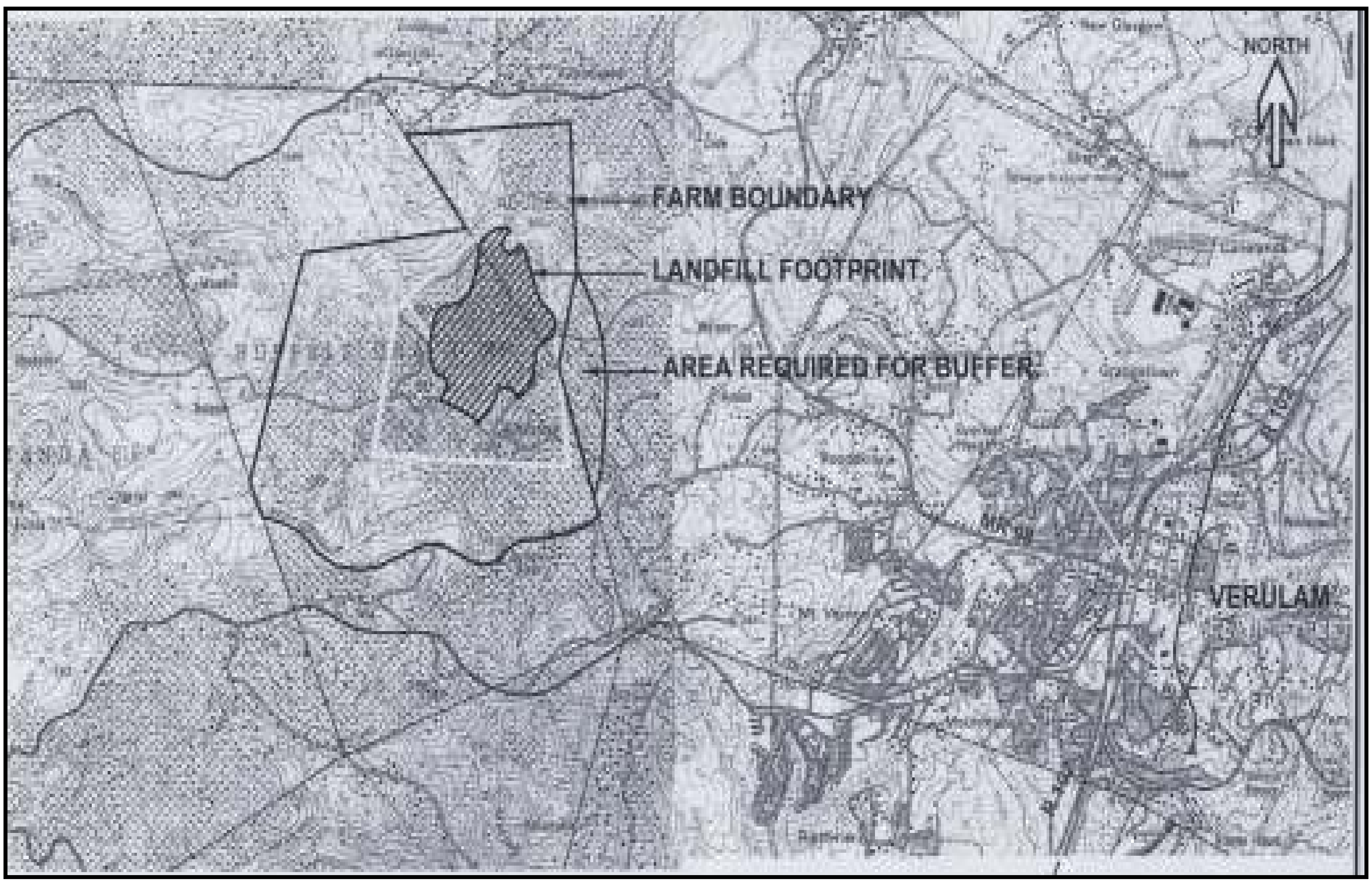
v) Airspace : 45 x $10^6 m^3$

vi) Fatal flaws : none evident

vii) Geotechnically : Geotechnical conditions were favourable in terms of stability and availability of material suitable for use in construction and as cover material.

An application for a permit was submitted in May 2001 supported by the E.I.A., Record of Decision issued by DAEA, Conceptual Design and Operational Plan. A draft Permit was issued in June 2001 followed by the final permit.

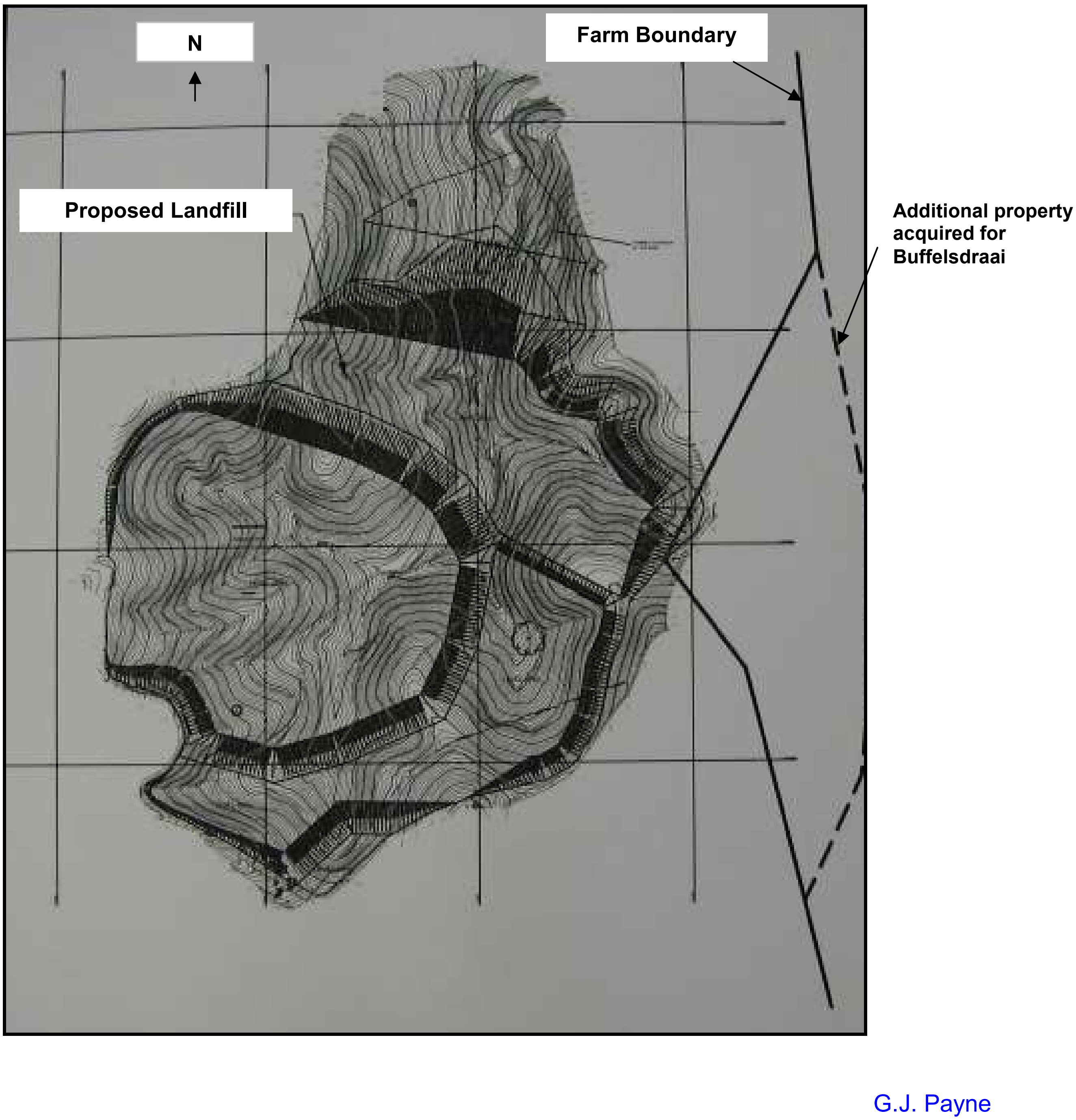
Figure 1 – Locality Plan



SITE DESIGN

The design of the site has been prepared in accordance with the Minimum Requirements and the policies of DSW. The latter have included the concept of a 'closed loop system' for waste disposal, whereby emissions from the decomposing waste are collected/extracted and treated before being used or released into the environment. Examples of this include the Sequence Batch Reactor (SBR) leachate treatment plant recently commissioned at Mariannhill Landfill and the Electricity from gas projects currently being considered by the eThewkini Council.

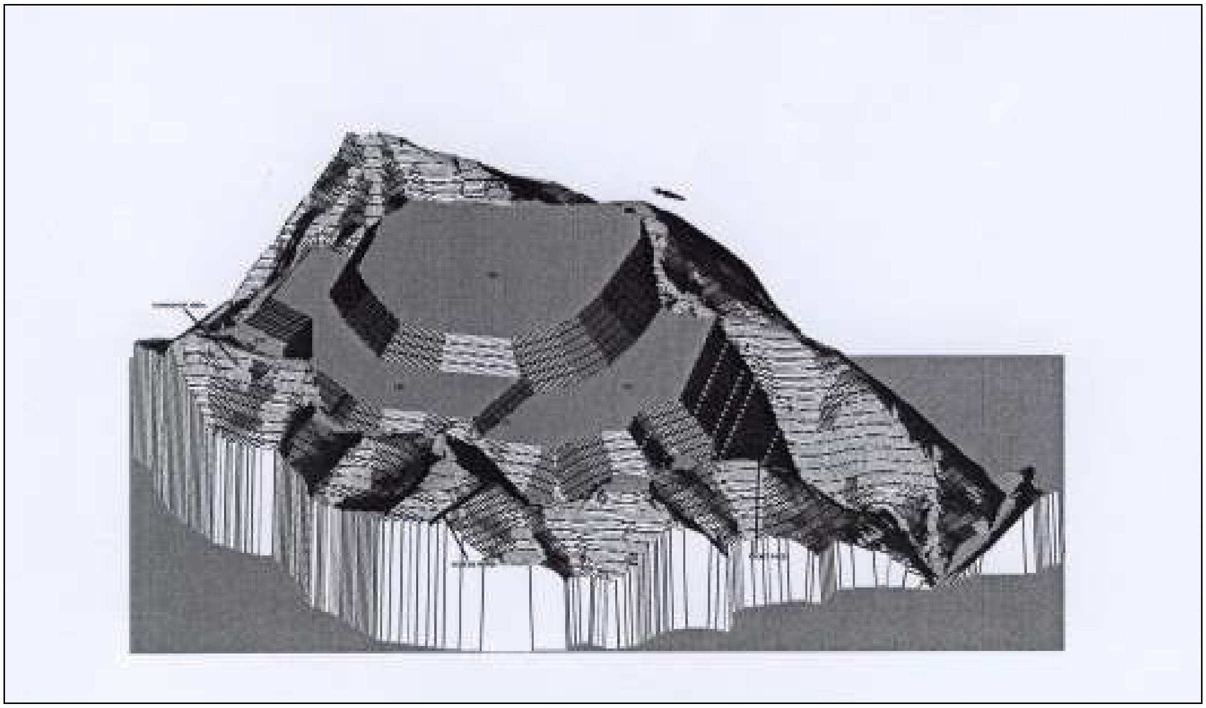
Figure 2 – Site Layout



1. <u>Layout</u>

The site comprises two valleys divided by an east-west ridge. The larger northern valley is designated for landfilling whilst the smaller southern valley will require further investigation if it is to be considered. The original farm comprising 750ha of sugar cane field has been purchased together with portion of the neighbouring farm to the east, to provide a 800m buffer around the footprint.

Figure 3 – Perspective View of the Landfill Looking West vertically exaggerated



The front face of the waste body rises from the toe at 160msl to a terrace at 240msl. A second terrace set back 50m rises to 260msl and a final terrace also set back 50m rises to 280msl. The positions of the terraces have been considered in conjunction with the need to restrict the visual impact. The slope of the front face and terraces are graded at 1 in 3. Stability analysis of the overall waste body revealed satisfactory factors of safety and against the three possible modes of failure (i.e. block, circular & wedge).

Originally it was anticipated that the site would be developed in seven cells, however to reduce initial expenditure, development of the site has been split into two phases. The first covering the original cells 1-3 is located at the head of the valley. Interim facilities for leachate storage and stormwater contaminant will be provided immediately below these cells with the ultimate facilities constructed when required at the toe of the site. The development of Phase 1 has been reviewed in the latest development plan and the anticipated cell development programme is indicated in Table 1 below. Cell 1A is located in a side valley and covers about 40000m². Interim stability consideration limits the height of the waste to 20m. As other cells are developed they will buttress the toe allowing the height to be raised.

Table 1 Buffelsdraai Landfill Development Plan

Description	Approximate Volume		Landfilling Life		Construction	Estimated Costs
	Sub Cell	Cumulative	Life (yrs)	Period		
1A	236,000		1.75	2006/07	Current	R 1,170,000
2A	790,000	1,026,000	2.25	2007/08/09	2006/07	R 14,205,000
2B	1,200,000	2,226,000	4	2010/11/12/13	2008/09	R 18,202,800
1B	942,000	3,168,000	1	2014	2012/13	R 7,400,000
3A	648,000	3,816,000	1	2015	2013/14	R 11,500,000
4A	700,000	4,516,000	1	2016	2014/15	R 14,784,000
3B	700,000	5,216,000	1	2017	2015/16	R 9,367,400
2C	825,000	6,041,000	1.25	2018/19	2016/17	R 9,585,000
Leachate Treatment Plant					2006/07	R 4,928,000

* Costs exclude VAT & Escalation

2. Geology

The entire site is underlain by Dwyka Tillite the minimum thickness of which is 13m. Dwyka Tillite is a poor aquifer and the existence of this type of soil/rock was a major factor in favour of the site. The geology is complicated by a dyke which cuts across the valley forming a barrier to the movement of groundwater. A stress fracture cuts through the eastern edge portion of the valley and acts as an aquifer and additional measures will be included in the liner design over this area.

Other important factors considered were:

- Layers of soft material which can be used for cover and in construction.
- Availability of clay for liner construction.
- Rock for possible use in drainage layers.
- Gravel for haul roads, etc.

The impact of the geology on site development costs must not be underestimated and significant savings can be achieved if material is available for use in road and liner construction. As an example, at least 400000m³ of crushed rock will be required for the construction and closure of the site. Suitable rock is available within the site area and the potential saving if it is from the quarry could amount to R30 million.

3. Surface Drainage

Although the site is located at the head of a valley the control of surface drainage was a major consideration in the planning. Wherever possible stormwater flowing into the landfill footprint is diverted into adjacent valleys. The perimeter road has been used as a cut off drain where possible. Stormwater drainage facilities have been based on 50 year return period storms and rainfall data extracted from intensity curves published by the eThekwini Engineers Department. No major drainage structures are required.

4. Leachate Collection, Containment & Treatment

Effluent draining from the cell area will comprise concentrated leachate released from the waste body mainly as a result of infiltration of rainwater and runoff from the surface of the waste body during heavy storms which will probably contain contaminates. The latter is referred to as contaminated stormwater. The design and operational plans for the site must attempt to reduce these volumes as much as possible, the first by restricting infiltration and the second by diverting uncontaminated runoff away from the waste body.

It was mentioned previously that the policy of DSW is based on a ‘closed loop system’ plan involving containment and treatment of the effluent. As a result of the phased approach, interim storage and containment facilities will be provided below the first three cells.

During heavy storms, contaminated stormwater will flow on the surface and will drain into a containment dam in the valley invert below the cells. Based on rainfall charts, the 24 hour duration, 50 year return period storm event will result in a depth of precipitation of 246mm. The capacity of the dam has been based on the largest of the three cells being landfilled and requires a storage capacity of 10500m^3. A bypass channel around the storage dam has been provided to direct uncontaminated stormwater from future landfill areas around the dam discharging into the valley below.

The contaminated stormwater dam has a valve control outlet and stored effluent will be monitored and if the quality of the effluent is acceptable it will be released into the environment. Alternatively dependant on the level of contamination it will be used for irrigation of sugar cane or removed for treatment at the plant or off site.

Leachate is collected by a stone drainage layer between the waste body and the liner. The layer is drained by two 160mm dia pipes which extend through the stability berm into a leachate contaminant area adjacent to the contaminated stormwater pond. A two pipe drainage system in the basal area (refer Figure 4) has been adopted as a safeguard against blockages and to facilitate the separation of leachate and stormwater during the initial filling of the cell.

Several factors affect the volume of leachate including:

- Landfilling method (i.e. operations).
- Type and quantity of cover material.
- Planning/design of the site.

Consequently estimation of leachate volumes is difficult and has been the subject of many papers. The fact that this site is being developed in phases allows some leeway in that the leachate produced during the initial phase can be monitored to allow more accurate sizing of the final facilities. Determination of the storage facilities must take account of the fact that the site will be treating the leachate and storage volume is critical mainly in the initial cells.

Simple water balance calculations based on Cells 1A & 2A indicate leachate volumes of 33,7m^3/day. This calculation takes cognaissance of the typical DSW mode of operation in terms of working face and covering.

Alternatively sites with similar climatic conditions, topography and operations can be expected to generate leachate at similar rates. Mariannhill landfill is in many respects very similar to Buffelsdraai in that:

- Both are operated by the same organisation (i.e. similar operations, planning & design).
- Topography and climate are similar.
- Cover material is essentially clayey and normally applied liberally.

Leachate generated from Mariannhill landfill was measured over a three year period. Average daily volumes varied from 62,25m^3 (December 2000) to 3,0m^3 (August 1999). The average daily volumes over the entire period is 18m^3/day.

It is noted that the cell development of Mariannhill landfill has often resulted in relatively large open areas which have not been diverted to stormwater and this has possibly contributed to the very high peak volumes during the summer rainfall months.

Based on the above, projected average daily flows from Cells 1A & 2A will be:

- Peak monthly 127,7m^3
- Average 36,9m^3

As discussed above it is proposed that the leachate volumes will be monitored carefully during the first few years and if necessary the final leachate containment facilities adjusted to suit measured flows.

A SBR leachate treatment plant is being planned which should be commissioned shortly after the site is commissioned in July 2006. For the first few months of the site life, leachate will be removed from the storage tanks taken and either disposed of at a Waste Water Treatment Works or utilized on site for dust suppression, etc.

LANDFILL GAS

It is DSW's intention to extract gas probably through the installation of wells into the gas body. The gas extraction system has not yet been designed although provision has been made in the cell construction for gas drains to be installed up the side slopes in the stone drainage layer.

The depth of the waste will exceed 100m in places and the extraction of gas from these depths will be difficult and alternatives need to be considered. An option could be to commence the gas extraction at interim stages before the landfilling reaches final levels.

Whether the gas will be flared or used (i.e. gas for electricity) still has to be finalised.

LINER DESIGN CELL 1

The purpose of the liner is to provide a barrier between the waste body and the environment to prevent the ingress of groundwater and contain emissions from the waste. Various options were considered including Compacted Clay, Geosynthetic Clay (GCL) and Geomembranes as well as composite liners comprising a combination of the above.

The most significant factors affecting the liners are:

1. Stability

   Being located in a valley the liner will have to be able to perform as required laid on a slope. The slope of the valley invert although relatively gentle affects the overall stability of the waste body whilst the side slopes affect both overall stability and localised stability particularly during construction.

   The design was based on the concept that the residual friction angle Ør of the weakest layer in the liner system was to be greater than 15° as determined by the stability analysis.

2. Barrier Protection

   The effectiveness of the liner to contain leachate. The Minimum Requirements are based on a 600mm compacted clay liner (GCL) with a permeability of not less than 10^{-7}m/sec. Alternative liner options must at least provide similar properties.

3. Cost

   The estimated cost of the liner supplied and installed which also takes account of the materials general handling workability.

Table 2 Liner Options Considered

LINER OPTION	STABILITY	BARRIER PROTECTION	COSTS
Clay Liner (CCL)			
Compacted clay from site or imported	Good (ØR >15°)	Medium	Low - if clay is from site High - if clay is imported
Geosynthetic Clay Liner (GCL)			
GCL with soil support and protection layers	Good (ØR probably >15°)	Medium to poor if protection soil is sandy Good if protection soil is clayey	Medium if protection soil from site High if protection soil imported
Geomembrane Liners (GM)			
Geomembrane with soil support and protection layer	Medium (if protection soil is sandy, ØR possibly >15°) Poor (if protection soil is clayey, ØR <15°)	Medium to poor if protection soil is sandy Good if protection soil is clayey	Low if protection soil from site Medium if protection soil imported

LINER SELECTION

The compacted clay liner was considered feasible however the geotechnical investigation revealed insufficient volumes of on-site clay to line the entire cell and the importation of a clay was considered to be too costly. The compact clay liner could therefore only be considered for part of this liner (i.e. valley basal portion).

The GCL with soil support and protection layers was considered feasible provided:

- shearbox testing revealed a Ør >15° for the weakest liner layer;
- the majority of the soil protection layer was obtainable from site to keep costs acceptable.

The GM with soil protection layers was also considered feasible provided the soil protection layers were sandy and the geomembrane type was FPP which is likely to result in a Ør >15° for the weakest liner layer. This would need to be confirmed by shearbox testing.

Stability analysis required that the liner layer with the weakest peak shear parameters have a minimum residual friction (Ør) of 15°, (i.e. this assumes that movement will occur along the liner layer with the lowest (weakest) peak shear strength, and that movement can increase until residual shear strength conditions are reached). It is also good practice to have this weakest liner layer above the impermeable liner so as not to disturb the liner should movement occur. The liner options therefore required confirmation of the peak and residual shear parameters of the various layers in the liner system by shearbox testing.

Permeability and shearbox testing of the on site clay revealed it to be suitably impermeable and to have a residual friction angle Ør >15°. Due to the sensitivity of the overall stability of the waste body down the valley the higher shear strength of the compound clay made it the preferred liner option in the basal area.

For the sideslopes, initial tests indicated that liner systems comprising either the GCL or geomembrane were feasible and the tender document required contractors to price both. The tender results confirmed the GCL liner configuration to be more cost effective than the FPP liner option and interface and internal shear tests were carried out on the GCL the tenderer indicated he may use (namely the Enviromat X2000 supplied by Kaytech) to determine whether or not the shear parameters met the stability requirements of the landfill cell.

Figure 4 – Valley Basal Liner Detail

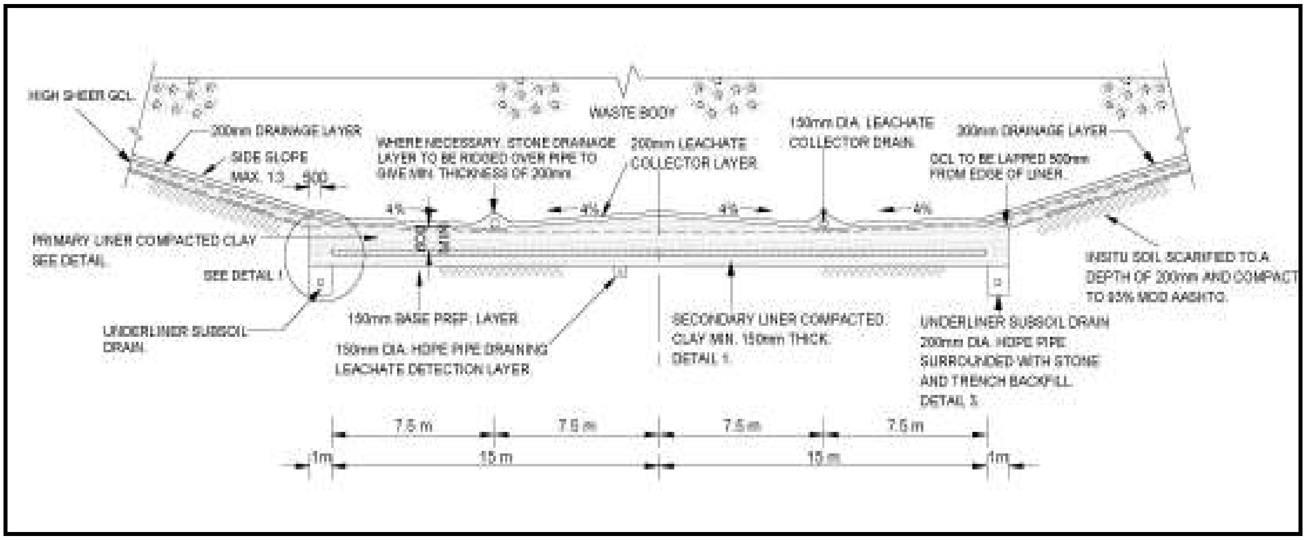
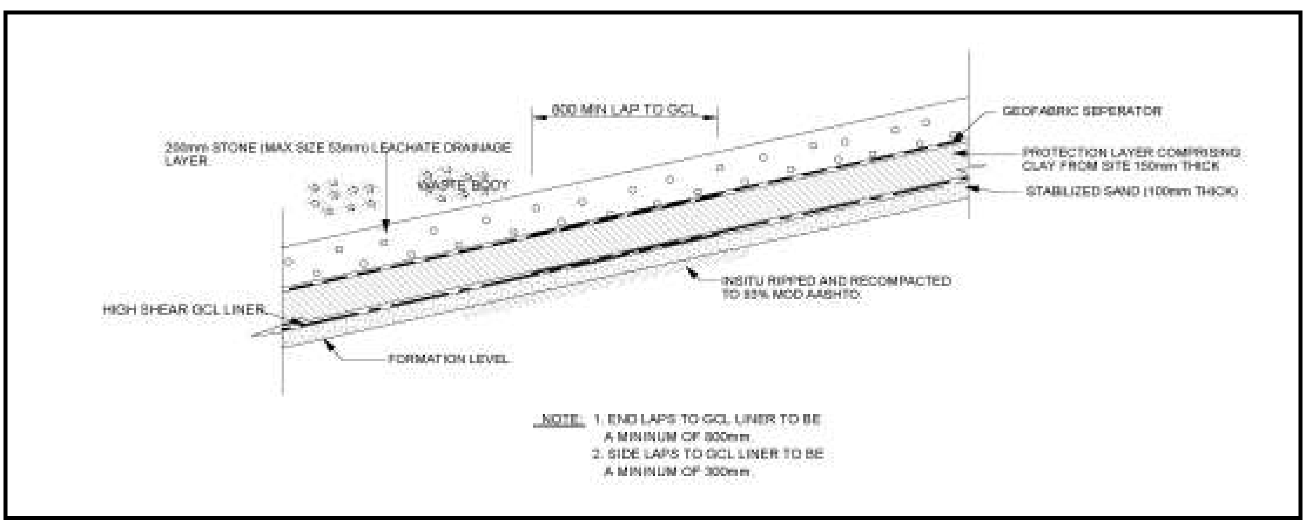


Figure 5 – Side Slope Liner



MATERIALS TESTED

Shear testing was undertaken on the liner configuration (refer Figure 5) to determine the interface and GCL internal shear parameters.

The purpose was to confirm that the weakest layer occurred on the interface between the upper clay protection layer and the GCL and that the residual shear strength of this layer had a residual friction angle (Ør). 15°.

TESTING METHOD

In order to provide a reliable statistical group, GCL interface and internal shear tests were performed with two different apparatus, namely:-

- A 300 mm square shearbox performed at the University of the Witwatersrand (WITS).
- A ring shear device (180mm OD/25mm sample width) performed at the University of Natal, Durban (UND).

The 300mm square shearbox is the industry standard and generally provides accurate determination of the peak shear strength parameters however, as the shear distance is limited, residual conditions are not usually reached. The ring shear device generally provides more accurate determination of the residual shear strength parameters as the shear length is unlimited thereby allowing the residual conditions to be reached.

Testing was performed in accordance with ASTM D6243-98 "Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liners by the Direct Shear Method".

Testing was performed at the following normal loads in each device:-

- Wits 300mm square shearbox - Normal loads 100, 200, 300 kPa
- Ring shear device (180mm OD) - Normal loads 50, 100, 200, 300 kPa

RESULTS OF THE SHEAR TESTING

The table below summarises the results of the shearbox testing performed at Wits and the UND on the clay and GCL configuration:

Table 3 Summary of Shear Testing

	Normal Loads (kPa)				Resultant Shear Parameters
	50	100	200	300	
Square Shearbox *(300 x 300mm)* Peak Shear Strength		85 kPa	118 kPa	145 kPa	Peak friction Ø P = 16.7° c = 56 kPa
Ring Shear Device *(180mm OD)* Peak Shear Strength	31 kpa	50 kPa	78 kPa	143 kPa	Peak friction Ø P = 17.15° c = 17 kPa
Residual Shear Strength	30 kpa	50 kPa	76 kPa	32 kPa	Residual friction Ø P = 16.7° c = 17 kPa

The following was noted from the results of the shear testing:

- Only the peak shear strengths are quoted for the square shearbox due to the limited displacement travel of the square shearbox, the peak and residual (or limited displacement) conditions are almost identical.

- For the square shearbox the failure surface occurred on the interface between the upper clay and GCL for all the normal loads 100, 200, 300 kpa.

- For the ring shear device, the failure surface occurred received between the upper clay and GCL for all normal loads below 200 kPa, however, above 200 kPa the failure plane went internally through the GCL, hence the significant decrease in shear strength to a residual condition for the 300 kPa normal load testing.

It is thus concluded that up to a normal load of 200 kPa (equivalent to ± 20m of landfill), shearing is likely to occur between the upper clay and the GCL which has an acceptable residual friction angle of Ør = 16,7° (refer Table 3). The stability analysis showed that a residual friction angle of 15° is sufficient for a stable landfill (refer Table 3). However, above 200 kPa (> 20m landfill height) there is the possibility that shearing would occur internally within the GCL itself which is known to have a residual friction angle of the order of only 5°. It is thus important that the landfilling over the GCL be limited to a maximum height of 20m until such time that the toe of the waste body can be supported.

The above summarises the shear test results as numerous additional tests were performed to determine a suitable clay protection soil, i.e. more granular clays were initially tested however this resulted in the failure plane passing internally within the GCL at very low normal loads (± 100 kPa) in both the square and circular shear apparatus.

BARRIER PROPERTIES

Assuming a 300m head on the liner the potential performance of the compacted clay and GCL compare as follows:

Description	cm/s
Minimum Requirements Clause 8.4.3	1×10^{-6}
Compacted Clay 600mm	$1{,}5 \times 10^{-6}$
GCL (Enviromat X4000)	$1{,}83 \times 10^{-6}$

COSTS

Based on the tender results the cost of the liner options compare as follows:

Compacted Clay	R 66-70/m²
GCL	R130-75/m²
Geomembrane	R137-85/m²

Costs exclude contractors preliminary & general cost, VAT and contingencies.

LINER OPTION SELECTED

Based on the above, it was decided to use the GCL liner configuration shown in Figure 5 as the valley sideslope liner (i.e. the valley sideslopes of Cell 1 don't have in excess of 20m landfill height) and to use a compacted clay as the valley basal liner (i.e. there are portions of the valley base where landfill heights exceed 20m). The valley basal liner configuration is also included in Figure 4.

Further reasons for adopting a GCL sideslope liner and compacted clay basal liner are:

- The better workability of a GCL on the sideslopes.

- The limited clay source and costs of importing clay.

Prior to construction the contractor requested the use of an alternate GCL product, namely Bentofix B4000. Internal and interface shear testing of this alternate product also met with the required design specification and hence this product was accepted as the GCL for Buffelsdraai Cell 1.

INFRASTRUCTURE

To function as a regional landfill capacity of receiving 1500 tonnes of waste per day significant infrastructure is required to support the site. Infrastructure required included:

- 2,5km access road
- Administration & Control buildings
- Workshops
- Washbay facilities for landfill plant and trucks
- 22m long Weighbridge 2 №
- Weighbridge operator kiosks
- Perimeter road (4,8km) and fencing

Electrical supply exists at the farm building, however it will have to be upgraded to supply the increased demand. There is no potable water supply to the site and originally the farms borehole was taken over, however following several problems with the supply a pump has been inserted into one of the monitoring boreholes and water is being extracted.

In line with DSW's policy establishment at Mariannhill landfill, a plant rescue nursery (PRUNIT) has been established and prior to areas being cleared, topsoil and indigenous vegetation is removed and either stored at the nursery or replanted directly. Already trees have been relocated into areas requiring visual barriers albeit only in 30 years time.

IMPLEMENTATION

Implementation of the development of the landfill has commenced. Initial estimates covering the infrastructure and the construction of Cell 1 were R33,0 million. As the funds were spread over three financial years the site works were divided into several contracts.

Contract №	Description	Value	Status
WS.5546	Access Road	R 6 143 321-45	Complete
WS.5547	Earthworks for Admin Area	R 4 214 324-40	Complete
WS.5555	Fencing	R 713 050-00	Complete
WS.5578	Boreholes	R 203 396-00	Complete
WS.5618	Infrastructure Building & Paving	R 7 065 304-30	In progress
WS.5591	Cell Construction	R 9 734 920-00	In progress

* Prices exclude VAT

The last two contracts are due to be complete in November 2005 and January 2006 respectively.

Planning has commenced for the leachate treatment plant and for Cell 2.

CONCLUSION

The development of the Buffelsdraai Landfill has been undertaken in line with a strategic decision taken by DSW to identify sufficient airspace to accommodate the eThekwini Metropolitan Council waste disposal required for the foreseeable future. The need for a landfill to serve the northern areas of the Metro has resulted in Buffelsdraai site proceeding. The site select process design has been undertaken strictly in accordance with the Minimum Requirements for the Disposal of Waste by Landfill which together with the Record of Decision issued by DAEA and the land being rezoned has resulted in DWAF issuing a permit for the operation. Implementation of the site works has commenced and the site is due to be commissioned early in 2006.

ACKNOWLEDGEMENTS

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REFERENCES

DWAF (Department of Water Affairs & Forestry) (1998). Minimum Requirements for Waste Disposal by Landfill. Second edition.

Strachan L, Rolando A & Wright M (2002). Rescue, Reinstate & Remediate - Landfill Engineering Methods that Conserve the Receiving Environment. Proceedings Wastecon 2002, International Congress. IWMSA pp 443-451.

Lombard J.E., Parkin J, Butler G (1998). A Streamlined Procedure for Landfill Site Selected and Related Environmental and Land use Planning Processes in KwaZulu-Natal : A Model for the Durban Metro Area. Proceedings Wastecon 1998 fourteenth Congress. IWMSA pp 621.

Figure 6 – Layout of Cell 1

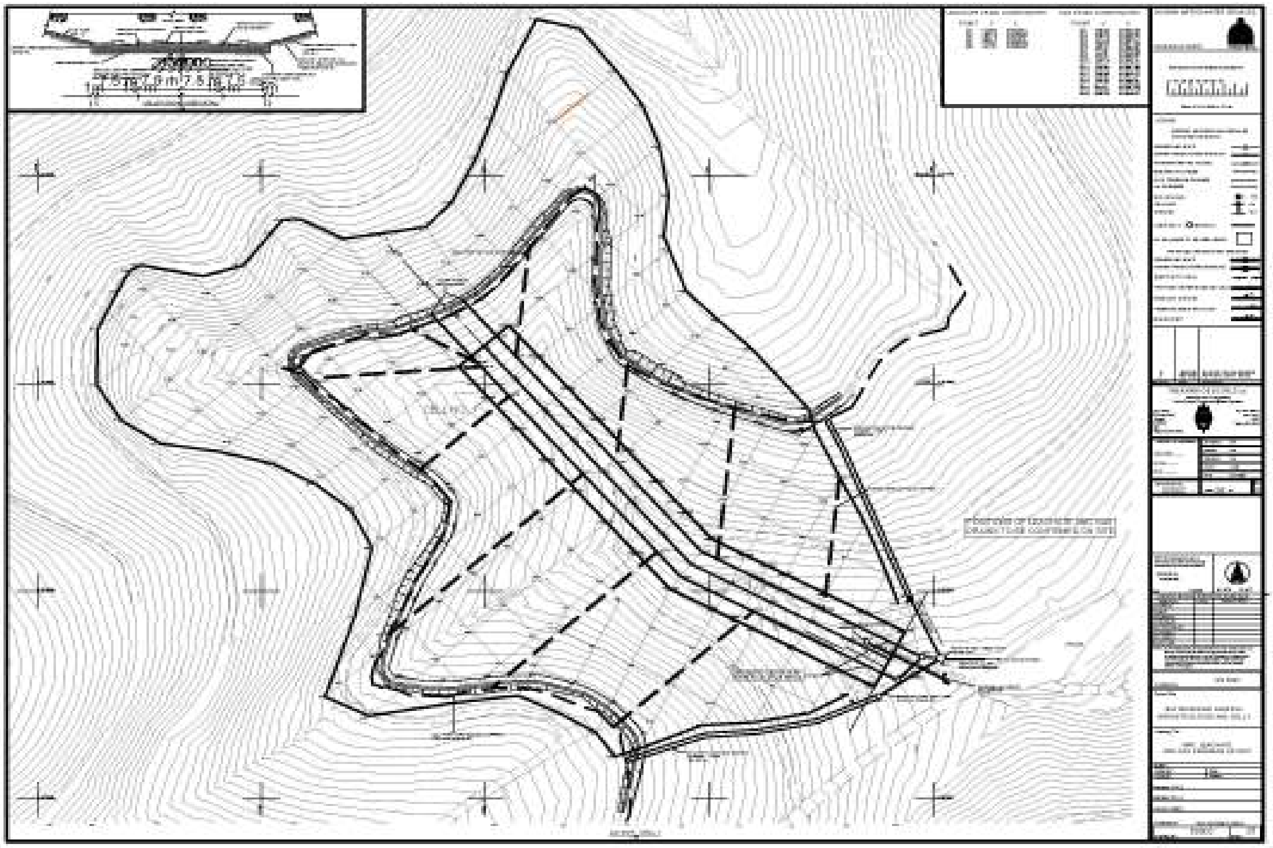

Figure 7 – Overall Layout