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ZITHOLELE CONSULTING (PTY) LTD

Preliminary Geotechnical Investigation of Preferred Sites B, C, F and H for Proposed Eskom 30 year Kendal Ash Disposal Facility - REVISION 3

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REPORT

Golder Associates

Executive Summary

Golder Associates was appointed by Zitholele Consulting to undertake geotechnical investigations for a proposed new Ash Disposal Facility (ADF) site for the Kendal Power Station to be developed for the long-term planning of its ash deposition facilities.

A two-phase investigation has been carried out on four potential sites, namely Site B, Site C, Site F and Site H for an ash disposal facility with a proposed footprint of the order of 600ha in extent. The investigation has been carried out based on the outcome of a high-level preliminary site selection process involving a number of specialist disciplines, including a preliminary geotechnical (desktop study) assessment. During the first field investigation programme in September 2013, portions of sites B, C and F were investigated. During the second field investigation programme in September 2014, the remainder of site C was investigated, as well as the full footprint of site H.

Site access constraints, resulting in limited access to portions of Sites B and F, are attributed mainly to restrictions imposed by present landowners and users, mostly on account of current and future (planned) opencast coal mining operations, land acquisitions, as well as current land occupation by the Arbor Community.

Fieldwork during the first investigation programme involved the excavation of test pits using a tractor loader backhoe (TLB) across sites B, C and F, restricted to within the areas where access was permitted. Fieldwork during the second investigation programme involved excavation with a TLB, as well as testing will a Dynamic Probe Super Heavy (DPSH).

Discussions, conclusions and recommendations included in this report must therefore be regarded as primarily relevant only to those portions of the sites where field investigations were undertaken, and therefore are not considered as necessarily representative of the remainder of the sites not subjected to field investigations.

Geological mapping shows the general area to be predominantly underlain by rocks of the Vryheid formation of the Ecca Group, Karoo Supergroup including sandstone, shale and coal seams as well as lesser occurrences of Lebowa Suite Granite (Bushveld Complex), Loskop Formation lavas and agglomerates, and Post-Transvaal diabase intrusions (Rooiberg lava Suite). Alluvial deposits along the Wilge and Leeuwspruit are also shown to be present.

During the course of the investigation, sandstone was encountered beneath portions of all four sites. Site B is underlain by sandstone, more extensive lava, minor granite and alluvium. Site C is underlain by sandstone, granule conglomerate, colluvium and alluvium. Site F is underlain by fill associated with past mining rehabilitation and lesser sandstone, while Site H is underlain by sandstone, mudstone, Post-Transvaal intrusives and pedogenic ferricretes (hardpan and nodular).

Based on our interpretations from the investigations, no apparent evidence exists in our opinion, to suggest that the sites are fatally flawed from a geotechnical perspective. Lesser and somewhat more significant challenges have been identified for each of the sites which will give rise to important design and construction expedients which must be adequately addressed in detailed design and project specifications.

Further geotechnical investigations will be required to support/ confirm current assumptions and to advance the current conceptual-level design into a definitive final engineering design suitable to construct the proposed facility.

The key findings of the investigation are as follows:

- Laboratory test results undertaken on selected samples of subsurface soil horizons collected from Sites B, C, F and H, indicate that the majority of material sampled exhibits (or may be blended to provide) reasonably favourable characteristics when considered for the construction of homogeneous starter wall embankments or liner receiving layers associated with the ash facility. In order to improve material workability it will generally prove advantageous to combine (or blend) the generally cohesionless transported horizons with the underlying residual soils for optimal performance;
- Deep seasonally-saturated alluvial (cohesive) soil horizons are envisaged which will affect relatively small portions of Site B and Site C. It is recommended that the proposed ash disposal facility should not extend onto these areas. Alternatively, should these areas be considered to form part of the ash facility footprint, potentially significant pre-cautionary drainage and earthworks mitigation measures may inevitably need to be implemented to prepare these work areas for construction;
- The surface trace of the Ogies dyke as defined from geological maps is located on all four sites. At Site B the trace runs east-northeast/west-southwest through the south eastern third of the site. At Site C the trace runs east/west through the north western corner of the site. At Site F the trace runs east-northeast/west-southwest through the southern third of the site. At Site H the trace runs west/east through the north western corner of the site;
- Site H has been identified by the client as the current "preferred site" for the proposed construction of the ash disposal facility based on the relative distance of the site from the position of the Kendal Power Station, the suitability of the gradient present on site (<2%) and the suitability of the in-situ material with regard to permeability (~10^{-5} cm/sec). The proximity of the site to the Ogies Dyke is also favourable to the proposed development of the new ash disposal facility. The main contributing factor leading to the selection of Site H is the non-availability of the alternate sites which were investigated (i.e. Sites B, C and F);
- A conceptual level slope stability analysis was conducted to determine the slope stability of the ash disposal facility, based on provided information and values found in relevant literature. Two distinct modes of failure have been identified with conceptual level factors of safety for both modes of about 1.2. This is at the bottom end of the industry accepted minimum range of 1.2 to 1.3. These modes comprise a failure at the toe of the facility that would impact the liner system, and the failure mode at the mid-slope bench which would mobilize a large volume of material but would not impact the liner system. Although further work is recommended to analyse the strength of the liner and of the ash material, the geometry upon which the ash disposal facility has been modelled for stability analyses is feasible with the low factor of safety values quoted above to reach a maximum height of 75m; and
- Permeability rates determined from laboratory falling head permeability testing indicate that the subsoil material, subject to proper compaction and treatment (compaction at optimum moisture content and ideal compaction factor), will be suitable to support the installation of the preferred liner system. Permeability rates in the order of 10^{-5} to 10^{-6} cm/sec can be expected from the materials collected from Site H.

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TABLES

1.0 INTRODUCTION AND TERMS OF REFERENCE

Golder Associates Africa (Pty) Ltd (Golder) was appointed by Zitholele Consulting (Pty) Ltd to undertake geotechnical investigations for a proposed new Ash Disposal Facility (ADF) site for the Kendal Power Station to be developed for the long-term planning of its ash deposition facilities.

The current investigation follows-on from earlier studies regarding possible short-term expansion of Kendal's existing ash disposal facility, and has been carried out on four potential sites, namely Site B, Site C, Site F and Site H. As the study has progressed, Site H has become the preferred option.

The sites have been tentatively identified based on the outcome of a high-level preliminary site selection process involving a number of specialist disciplines, including a preliminary geotechnical (desk study) assessment.

The initial (invasive) geotechnical investigation was carried out to determine the general suitability of the potential sites for the placing of an ash disposal facility. The proposed ash facility footprint is anticipated to be of the order of 600ha in extent, depending on the maximum achievable height of the ADF.

The current investigation has been carried out to inform and expand the site selection process and is to be regarded as essentially preliminary in nature and for conceptual design purposes only.

Due to accessibility constraints imposed at the time of the fieldwork commenced, only limited portions of Sites B and C were investigated. These constraints may be attributed to access restrictions imposed by present landowners / users, mostly on account of current and future (planned) opencast coal mining operations, land acquisitions, as well as current land occupation by the Arbor Community.

Any discussions, conclusions and recommendations included in this report must therefore be regarded as primarily relevant only to those portions of the sites where invasive investigations were undertaken, and therefore not to be considered as necessarily representative of the remainder of the sites not subjected to invasive field investigations.

2.0 OBJECTIVES OF THE INVESTIGATION

The investigation has been carried out in order to:

- Establish the general nature and engineering properties of the soils (and shallow rock occurrences) likely to impact on the proposed development, and confined to the (3m) depth limitations of the investigative techniques adopted;
- Provide a general geotechnical appraisal of the underlying founding conditions for ash facility development;
- Determine the general excavation (excavatability) characteristics of the underlying soils and rocks;
- Comment on shallow underlying and perched groundwater conditions, and any perceived impact this may have on the development;
- Comment on the basic chemical corrosiveness of the in-situ soils and rocks towards buried (ferrous) services and exposed concrete foundations; and
- Comment on any further geotechnical issues, identified during the course of the investigation work, which may affect the proposed development.

3.0 INVESTIGATION METHODOLOGY

The scope of work followed during the course of this investigation has included:

- Test-pitting by means of TLB over all four potential sites in order to expose the ground profile to depths generally limited to 3m or machine refusal depth;

- Soil profiling of pits by specialist geotechnical engineer or engineering geologist, in terms of Southern African geotechnical criteria, and representative sampling of the soils for limited (off-site) laboratory testing;
- Testing using a Dynamic Probe Super Heavy (DPSH) rig over Sites C and H in order to determine the machine refusal depth and infer the soil density;
- Laboratory testing of selected samples to determine the general engineering properties of significant horizons in the ground profile succession; and
- Appraisal and interpretation of data, and presentation thereof in a succinct geotechnical engineering report.

4.0 INFORMATION CONSULTED AND REVIEWED

The following sources of information were consulted during the course of the investigation:

- Published 1:250 000 scale Geological Map ï ̃East Randò(2628), Government Printer, 1986;
- Various maps supplied by the Client and online aerial imagery; and
- Falling head permeability test results supplied by the Client, collected from Site H (samples were tested by Civilab with 4% and 8% bentonite blending).

5.0 DESKSTUDY

5.1 Site Locality and Description

The proposed sites are located to the west and north of the existing Kendal Power Station, Mpumalanga and are shown in **Figure 1 Appendix A**.

5.2 Site B

Site B is situated some 3.5km north to north-west of the Kendal Power Station, forming a roughly rectangular shaped site approximately 5km across east-west and 2.5km north-south, covering an area of some 1200ha. The R555 provincial road forms much of the northern border of the site.

The site is host to current coal mining activity (open cast) in the western portion of the site with cultivated farmland in the eastern portion. The site is essentially situated on a local (high-ground) watershed feature, sloping gently to the north and south as well as westwards towards the Wilge River. The Arbor informal settlement is present to the north of the site which is approximately 2km across.

Existing infrastructure affecting the site includes buried Eskom services (water and electrical reticulation), which link Kendal to Kusile Power Station, and which run from the eastern boundary in a north-westerly direction, exiting the site approximately a third of the distance along the northern boundary (from the east). Overhead power lines run east-west through the northern portion of the site and north-south near the eastern boundary.

No significant drainage paths were observed on the site, although the western extremity borders the Wilge River and associated wetlands. Scattered occurrences of boulder rock outcrop and sub-outcrop (shallow bedrock) occur in the eastern portion of the site which is generally under cultivation but was also characterised by patches of uncultivated (non-arable) land attributed to shallow or outcropping rock or ferricrete.

Access for this investigation was limited mostly to the eastern portion of the site and a small portion of the extreme western portion of the site not affected by mining. Current mining activity occupies the central to western portion of the proposed footprint area with the informal Arbor settlements located to the north which, we interpret, will place specific constraints on developing the site for ash disposal as is presently being considered.

5.3 Site C

Site C is located some 4.5km west of the Kendal Power Station and south of Site B and also forms a roughly east-west rectangular shape covering an area of some 950ha, measuring 4.6km across (east-west) and 2km north-south.

The site essentially slopes gently to the east and north to Leeuwspruit, a tributary of the Wilge River. The eastern and western portions of the site are presently under cultivation with maize and some centre-pivot irrigation in evidence. The central portion of the site is occupied by current open-cast coal mining activity (Mbuyelo Group ï Rirhandzu Colliery) which, like Site B, will provide certain constraints on development of this area for ash disposal, if the site is returned for this usage.

Access to the site was limited to the eastern un-mined portion of the footprint. No access was granted to assess the central and western portions of the site which are occupied by the current and potential future mining activity (as directed by the current landowner).

Overhead power lines pass through from the north-eastern corner of the site, heading south-west, affecting the north-western portion of the area investigated. The existing infrastructure affecting Site C includes buried Eskom services (water and electrical reticulation), which link Kendal to Kusile Power Station. The exact position and alignment of the buried services will need to be provided by Eskom.

The Leeuwspruit forms the eastern and northern boundaries of the portion that was investigated and is characterised by a gently-sloping, wide floodplain.

5.4 Site F

Site F is located some 2km north of the Kendal Power Station. The R555 provincial road forms the southern boundary of the site which extends north all the way to the N12 freeway. The site covers approximately 1200ha, measuring roughly 5km from north to south and up to 3km from east to west.

The site is mainly occupied by past and present coal mining (and/or open pit exploration) operations with minor cultivated lands in the northern-most and central / western portion of the site. Current mining occupies the south-eastern and central portions of the site. Overall the site slopes gently to the north.

No rock outcrop or drainage features were observed on the portions of the site where access to investigate was granted for this study. These areas included the previously mined area belonging to Bankfontein Colliery (owned by Shanduka Mining) and a portion of farmland to the west.

The Bankfontein Colliery comprises a range of open cast mine features including two mined-out open cast pits, numerous stockpiles of materials, discard dumps, slurry ponds, a disused wash plant and rehabilitated areas including an old pit which has been backfilled and reshaped to a gently sloping basin feature. Most of the Bankfontein Colliery area is overlain by made ground (i.e. opencast backfill) as a result of the historic mining activity. Our enquiries indicate that further rehabilitation is expected on this site. Mine backfill of this nature is interpreted to have a significant impact on proposed future design considerations, construction preparations and general usage of the site for ash deposition.

Mining pits observed in the Bankfontein Colliery and other open pits on the southern edge of the site show that moderately to relatively shallow sandstone bedrock conditions can be expected across the area, appearing to become shallower to the south.

5.5 Site H

Site H is located some 1.5km west of the Kendal Power Station. The R545 provincial road forms the eastern boundary of the site with the northern boundary lying approximately 1km from the R555 provincial road. The site covers approximately 1150ha, measuring roughly 2.2km from north to south and 2.4km from east to west.

The site is mainly occupied by cultivated lands which are sub-divided into various numbers of agricultural farming portions. Wetland areas have been excluded from the scope of the investigation in order to preserve these areas. The concentration of water in the central portion of Site H is currently being used for irrigation purposes, which are typically central-pivot irrigated lands. Water is being pumped out of the farm dam located to the south west of the Kendal Power Station by means of approximately a 650 - 750mm diameter pipe and is temporarily stored in the natural topographic depression located in the middle of Site H.

The natural depression towards the south of the Site is about 6 to 7m deep, and 325m in diameter. It is expected that the overall size of the pan will increase during the rainy season. The pan is currently unvegetated and approximately 0.5 to 1.0m deep (from visual observations) and surrounded by grasses. No signs of large wildlife were observed near the pan during the site investigation.

Overhead power lines pass through the site, from east to west, then divert to a north westerly/ south-easterly direction. It is expected that these would need to be relocated about in order for Site H to be utilised for the ash disposal facility. Dirt road D1390 will also need to be realigned which cuts across the western portion of Site H.

5.6 Regional Geology

The geological mapping presented in Regional Geology Map 2628 of East Rand, published by the Department of Minerals and Energy in 1986, shows the general area of the four sites under investigation, to be predominantly underlain by the following geological sequences and their lithology in fairly complex disposition with respect to one another:

- Quaternary alluvial deposits associated with the Wilge River and its tributaries;
- Vryheid formation of the Ecca Group, Karoo Supergroup including sandstone, shale and coal seams;
- Lebowa Suite Granite belonging to the Bushveld Complex;
- Loskop Formation lavas and agglomerates, and
- Post-Transvaal diabase intrusions.

Generally, a modest covering of transported soils is expected to cover the sites, (much of which has been previously cultivated), underlain by residual soils derived from the weathering of the respective host rock formations. Further details of the regional geology are provided in Section 7.0.

The very varied and complex geological character of this general locality, and the obvious resulting impact on productive shallow opencast mining (obviously confined to the Vryheid Formation coal seams) in this area, is readily evident by the amount of seemingly discontinuous trial exploration and production mining pits which are scattered over the area and, as a consequence, have created significant disturbance to the land surface.

5.6.1 Ogies Dyke

Figures 2, 3, 4, 4A and 5 represent the regional geology of the sites (**Appendix A**), along with geotechnical test locations excavated for the current study. The surface trace of the Ogies Dyke has been plotted using different source information, as follows:

- Council for Geoscience (unreferenced data).
- Regional Geology Map 2628 of East Rand, Department of Minerals and Energy, 1986 (Scale 1:250,000).
- Core Groundwater Technical Services cc (unreferenced data).

All traces have been placed on the figures in order to provide an indication on the potential variability in the trace position, and since the trace location cannot be confirmed using aerial photography.

Site B

Based on the available inferred traces, the approximate surface trace of the Ogies Dyke crosses east-northeast/west-southwest through the south-eastern third of Site B. Aerial photography does not indicate any geotechnical or geological structures of note. No obvious signs of outcropping rock or dykes are apparent.

Site C

Based on the available inferred traces, the approximate surface trace of the Ogies Dyke crosses east/west through the north western corner of Site C. Aerial photography potential evidence of isolated small areas of outcropping rock toward the northwest of the site. Pans indicating evidence of shallow water are present immediately south of the Site. At least three east-west trending drains have been excavated toward the west of the site, draining into streams west and north of the Site.

Site F

Based on the available inferred traces, the approximate surface trace of the Ogies Dyke crosses east-northeast/west-southwest through the southern third of Site F. Offshoots indicates on the Aerial photography potential evidence of isolated small areas of outcropping rock toward the northwest of the site. Pans indicating evidence of shallow water is present immediately south of the Site. At least three east-west trending drains have been excavated toward the west of the site, draining into creek lines west and north of the Site.

Site H

Based on the available inferred traces, the approximate surface trace of the Ogies Dyke crosses west-east through central north of Site H. Test pits TPH1, TPH2 and TPH15 lay along the inferred trace of the Ogies Dyke. The presence of drainage pans provide potential evidence of shallow water in the vicinity of TPH1 and TPH15 which are situated in the central to west portion of the site. Groundwater seepage at test position TPH1 suggests that the Ogies Dyke could potentially represent a preferential flow path for water; however, this could not be confirmed within the scope of this investigation. No signs of outcropping rock or dykes were observed on site.

6.0 FIELDWORK

6.1 Site Visits

A site visit was undertaken to three sites (B, F and a portion of C) during September 2013 in order to gain a preliminary appreciation of the sites, to assist in investigation planning and determine accessibility for fieldwork. A follow-up site visit to Site H and the remainder of Site C was conducted from 25 to 26 September 2014.

Access for the site visits was arranged by Zitholele, Access to the remaining western and central portion of site C was provided by the landowners under the coordination of Zitholele Consulting. Site F access was limited to the Bankfontein Colliery, where formal mining operations have ceased. Site H access was provided by the majority landowner, Eskom and the lessee Mr Hardu Prinsloo. No access was gained to the central and western portions of Site B, which are currently being mined.

6.2 Test Pitting

During the two fieldwork programmes in September 2013 and September 2014, test pitting by means of a tractor loader backhoe (TLB) was undertaken in order to determine the ground conditions within the depth limit of the investigation.

The first programme of test pitting was carried out between 30th September and 4th October 2013, using a JCB 3CX TLB provided by Transcavators (Pty) Ltd. The second programme of test pitting at sites C and H was carried out between 25th August and 26th August 2014, using a Bell 315SJ TLB provided by Delta Plant and Crane Hire (Pty) Ltd.

The test pits across all sites were excavated to the depth limit of the machine (approximately 3m), except where refusal occurred at shallower depths. In accordance with Golder Health and Safety requirements, no test pits deeper than 1.5m were entered for profiling, unless appropriately battered back to a safe angle. The test pits were excavated to 1.5m entered into and profiled in-situ by a geotechnical specialist and then excavated further to their full depth and profiled from spoil.

The positions of all test pits were recorded using a Garmin, eTrex10, hand held GPS accurate to within about 5 m horizontally.

6.3 DPSH Testing

The second programme of fieldwork which comprised further investigation of Sites C and H also made use of Dynamic Probe Super Heavy (DPSH) testing. This utilises a 63.5kg hammer which is repeatedly dropped over a distance of 760mm along a guide rail onto an anvil, driving a string of rods with a cone attached at the end. The cone has a diameter of 50.5mm and an apex angle of 60º.

A total of eight (No.8) DPSH tests were performed at Site H and a total of five (No.5) tests were performed at Site C. The test results were used to determine the in-situ consistencies of the subsoil materials and also assist when determining the excavatability of the in-situ material. In order to ascertain a better relationship between the DPSH penetration rates and the in-situ subsoil consistency, the DPSH N counts were converted to *Equivalent* SPT N values, after the equation by (MacRobert C., Kalumba D. and Beales P., 2011):

$$\textit{Equivalent SPT N} = \frac{\text{[illegible]}}{\text{[illegible]}}$$

Table 1: Equivalent SPT N Values for Sites C and H (*converted from DPSH N to SPT N values; MacRobert C., Kalumba D. and Beales P., 2011)*

Depth (m)	DPSH C1	DPSH C2	DPSH C3	DPSH C4	DPSH C5	DPSH H1	DPSH H2	DPSH H3	DPSH H4	DPSH H5	DPSH H6	DPSH H7	DPSH H8
0.3	0	0	0	0	0	0	0	0	0	0	0	0	0
0.6	36	6	1	2	14	16	7	10	17	30	18	36	18
0.9	R	12	7	14	12	20	14	12	16	16	20	R	9
1.2		3	6	36	6	13	7	7	7	14	25		3
1.5		7	5	R	3	6	5	3	3	36	36		3
1.8		5	3		2	3	7	3	2	R	R		6
2.1		5	2		3	7	21	30	2				17
2.4		30	2		36	13	28	8	17				36
2.7		30	2		R	13	22	36	36				R
3.0		60	2			13	23	R	R				
3.3		R	12			19	29						
3.6			24			19	36						
3.9			30			25	R						
4.2			30			32							
4.5			36			36							
4.8			R			R							

R = Practical Refusal

Table 2: Relationship between SPT N (blows/ 300mm) and Relative Density/ Consistency of Subsoil Material

SPT N (blows/ 300mm)	Relative Density/ Consistency*
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium dense

SPT N (blows/ 300mm)	Relative Density/ Consistency*
30 - 50	Dense
>50	Very Dense

* Terzaghi and Peckõs (1948)

From the ñcorrectedò DPSH test results, an estimate of the relative density/ consistency for Sites C and H can be made using Terzaghi and Peckõs (1948) classification for sand (this classification can be used regardless of the granular soil type), and is shown in **Table 2** above. Generally, the subsoil material consistency typically ranged from ñvery looseò to ñmedium denseò at varying depths at each test pit position. Refer to **Table 1** above for the actual depths.

The DPSH test results for Sites C and H are described in further detail under sections 8.3.2 and 8.3.4 respectively with the raw data appended under **Appendix E** and the test positions highlighted in **Figure 6**, **Appendix A**.

6.4 Access to Sites and Restrictions Imposed

6.4.1 Site B

Access was initially granted to investigate the eastern portion of the site during the first fieldwork programme, with the western portion of the site (apart from the extreme western portion) being excluded, as this was being mined at the time. During the fieldwork programme, access to the northern and extreme eastern portions of the site was initially declined, but granted later. Test pitting was thus carried out within the eastern, northern and extreme western portions of the site. The western portion of the site was further restricted by the westward advancement of current mining which has reduced the available area considered for investigation.

Approximately 750ha of the site (total 1200ha) was investigated due to limited access and current mining land use, the ultimate extent of which is not presently known.

Twenty one (No.21) test pits, numbered TPB02 to TPB22, were excavated across the site, including TPB10B and TPB13B.

The positions of the test pits are shown in **Figure 2**, **Appendix A**. The test pit profiles are presented in **Appendix B**.

6.4.2 Site C

During the first fieldwork programme in September 2013, access was only granted by the land owner for the eastern-most portion of the site, as the central and western portions of the site were being mined or had been earmarked for future mining, with some of the land apparently already having being sold to the prospective mining company.

Consequently only approximately 200ha of the original 950ha of the site was available for investigation during the first fieldwork programme (**Figure 3**, **Appendix A)**.

Eleven (No.11) test pits, numbered TPC01 to TPC08 and TPC03A, TPC04A and TPC05A were excavated across this portion of the site.

During the second fieldwork programme in September 2014, access was granted to the western and central portions of the Site C. During this phase, an additional five (No.5) test pits and five (No.5) DPSH tests were conducted.

The positions of the test pits are shown in **Figure 5**, **Appendix A** and the detailed test pit profiles presented in **Appendix B**.

6.4.3 Site F

During the first fieldwork programme, access onto Site F was initially limited to the Bankfontein Colliery (Shanduka) which covered the northern portion of the Site F. Nineteen (No.19) test pits numbered TPF02 to TPF 21, excluding TPF03, were excavated in an area which excluded the northernmost agricultural lands.

The area to the south of Bankfontein was being mined by BHP Billiton and Westcoal at the time of the first fieldwork programme. No access was granted to any of these surrounding coal mines.

Towards the end of the first fieldwork programme, permission was granted to investigate a small package of land, estimated to be of the order of 30ha, to the southeast, which was not being mined. This piece of land is separated from Bankfontein by other existing mining operations (possibly Westcoal). Access to this portion of land could not readily be established on site, and it was deemed fruitless to investigate such a small and isolated piece of land inside an area already being mined or possibly designated for future mining.

The Bankfontein area investigated comprises about 200ha of the total 1200ha site.

Positions of the test pits are shown in **Figure 4, Appendix A** and copies of the detailed test pit profiles are presented in **Appendix B**.

6.4.4 Site H

Access onto Site H was granted without limitations or restrictions. Fifteen (No.15) test pits, numbered TPH1 to TPH15 were excavated across the full extent of the site. The area is currently owned by Eskom and is being leased by Mr Hardu Prinsloo. The trace of Site H was moved subsequent to the field programme, thus no test pits are located over the northwestern portion of the Site.

6.5 Laboratory Testing

The following laboratory testing was undertaken:

- Grading, hydrometer and Atterberg Limit tests to determine the basic engineering properties of the in-situ soils and for classification purposes;
- Natural moisture content tests;
- Consolidation (collapse potential) testing to determine the consolidation characteristics under saturation and load;
- Basic chemical tests on in-situ soils to determine aggressiveness and corrosive potential of the soils, including pH and conductivity tests; and
- Falling head permeability testing to determine the permeability rates for liner selection and suitability.

A summary of the laboratory test results is presented in **Table 1** overleaf, whilst the laboratory test certificates are attached in **Appendix C**.

Table 3: Summary of Laboratory Test Results

Hole No	Sample No	Depth (m)			Origin	LL	PI (425)	LS	GM	NMC (%)	PI (whole)	475 (%)	2 (%)	425 (%)	075 (%)	002 (%)	Weston swell (%)	van der Merwe Activity	pH	Conductivity (mS/m)	Resistivity (Ohm/cm)	USC	Collapse potential at 127kPa (%)	Collapse potential at 200kPa (%)
TPB4	1	1.0	to	1.3	Reworked Residual Diabase	29	12	6	1	13.72	8	98	84	64	50	5	0.06	LOW	6	4.1	24390	CL	-	-
TPB3	1	0.5	to	0.7	Transported	NP	NP	0	0.81	10.2	NP	98	94	79	42	3	-	-				SM	7.6	
TPB12	1	0.7	to	1.0	Ferruginised Reworked Residual Diabase	28	11	5	1.7	11.3	5	89	54	44	31	5	0.02	LOW	5.4	8.6	11628	SC	-	-
TPB18	1	1.3	to	1.5	Ferruginised Reworked Residual Diabase	29	11	5	0.95	14.6	8	97	84	70	46	9	0.06	LOW				SC	-	-
TPB20	1	1.0	to	1.5	Transported	68	24	12	0.22	28.6	22	100	98	94	84	21	1.69	MED				MH	-	-
TPC01	1	1.5	to	2.0	Transported	41	30	15	0.4	17.5	26	100	99	86	75	19	0.39	MED	6.3	5	20000	CL	-	-
TPC02	1	1.5	to	2.0	Transported	NP	NP	0	0.84	9.9	NP	100	100	75	36	4	-	-				SM	-	-
TPC04A	1	1.0	to	2.0	Transported	NP	NP	1	0.79	16.4	NP	100	99	76	40	2	-	-				SM	-	-
TPC05A	1	0.6	to	0.8	Ferruginised Reworked Residual Sandstone	26	10	5	1.91	5.8	3	96	47	33	37	5	0.02	LOW	4.8	9.5	10526	SC	-	-
TPC8	1	0.5	to	1.0	Transported	NP	NP	0	0.7	8.4	NP	100	100	79	48	3	-	-				SM	6.9	
TPF05	1	1.5	to	2.0	Transported	33	14	7	0.44	13.7	12	99	97	89	66	13	0.32	LOW-MEDIUM				CL	-	-
TPF08	1	1.5	to	3.0	Ferruginised Reworked Residual Sandstone	35	13	7	0.46	18.9	11	98	96	88	66	12	0.17	LOW-MEDIUM				CL	-	-
TPF10	1	1.5	to	2.0	Transported	NP	NP	0	0.76	7.7	NP	100	99	92	26	2	-	-	4.5	27.6	3623	SM	-	-
TPF11	1	0.8	to	1.0	Slightly Ferruginised Transported	24	11	5	0.73	13	8	91	87	77	58	7	0.07	LOW				CL	-	-
TPF17	1	1.6	to	2.0	Ferruginised Reworked Residual Sandstone	28	15	7	0.79	13	11	92	87	74	57	14	0.08		5.7	5.5	18182	CL	-	-
TPC1	1	0.3	to	3.0	Residual Mudstone	30	16	8	0.42	16.63	13	100	98	85	75	11	0.12	MED				CL	-	-
TPC3	1	1.9	to	3.0	Residual Sandstone	26	10	5	0.58	16.32	8	100	96	78	67	7	0.04	LOW				CL	-	-
TPC5	1	0.4	to	1.2	Residual Conglomerate	NP	NP	0	1.67	5.05	NP	97	62	41	30	2						SM	-	-
TPH1	1	0.5	to	1.9	Residual Mudstone	22	11	5	0.47	21.59	9	100	99	83	71	8	0.02	LOW	7.46	0.479	2088	CL	-	-
TPH3	1	0.0	to	0.8	Nodular Ferricrete Pedogenic	NP	NP	0	1.4	9.25	NP	79	65	55	40	2						SM		
TPH6	1	2.0	to	2.9	Hardpan Ferricrete Pedogenic	24	11	5	1.02	11.54	7	91	80	63	55	8	0.03	LOW	6.75	0.1171	8540	CL	-	-
TPH7	1	0.3	to	2.3	Residual Mudstone	24	12	6	0.51	16.12	10	100	97	83	68	8	0.05	LOW-MEDIUM				CL	-	-
TPH8	1	0.8	to	2.2	Hardpan Ferricrete Pedogenic	33	15	7	0.46	5.29	12	99	95	83	76	11	2.34	LOW-MEDIUM	7.29	0.158	6329	CL	-	0.035
TPH13	1	0.8	to	2.9	Nodular Ferricrete Pedogenic	28	11	5	0.51	11.69	9	95	90	86	74	5	0.20	LOW	7.96	0.211	4739	CL	-	-
TPH15	1	0.0	to	0.9	Colluvium	NP	NP	0	0.98	10.59	NP	87	81	70	51	1			7.78	0.1514	6605	ML	-	-

Key:

LL	- Liquid limit	2 (%)	- Percent passing 2,0mm sieve	Weston Swell	- Empirical estimate of swell based on Weston method
PI (425)	- Plastic index of % passing 0,425mm sieve	425 (%)	- Percent passing 0,425mm sieve	van der Merwe Activity	-Estimated activity potential based on van der Merwe method
LS	- Linear shrinkage	075 (%)	- Percent passing 0,075mm sieve	pH	- Acidity / alkalinity index of soil
GM	- Grading modulus	002 (%)	- Percent passing 0,002mm sieve	Cond (mS/m)	- Electric conductivity of soil
NMC (%)	- Natural moisture content			Resistivity	-Resistivity of soil
PI (whole)	-Plasticity index of whole sample			USC	- Unified soil classification
				Collapse potential	-Collapse potential utilising Schwartz, K problem soils publication (1985)

6.6 Material Properties and Consistency

6.6.1 Site B

Transported Soils (Aeolian and Alluvium)

The samples recovered from the overlying transported horizons exhibited considerable variance.

Sample classification ranged from SM (silty sand) to MH (high plasticity silts) according to the Unified Soil Classification (USC) system. Liquid Limit values varied between 0% (non-plastic) and 68% (high plasticity), whereas Plasticity Indices varied between 0% (non-plastic) and 24% and Grading Moduli ranging between 0.22 and 0.81. Reported van der Merwe potential expansiveness classification ranged between *Low* and *Medium*, whereas the Weston[1] swell potential ranged between 0% and 1.69%.

Residual Soils (Lava)

Laboratory testing undertaken on selected samples revealed that the residual lava encountered across the majority of the portions investigated classify as CL (low plasticity clays) or SC (sand with lesser clay fractions) according to USC. Reported Liquid Limit values varied between 28% and 29%, Plasticity Indices between 11% to 12% and Grading Moduli between 0.95 and 1.7. Van der Merwe potential expansiveness classification of the residual soils generally classified as *Low*, with Weston swell potential ranging between 0.02% and 0.06%.

Residual Sandstone

Laboratory testing of residual sandstone, interpreted as similar in quality to that of Sites C and F, returned a material classification of SC (clayey sand) material with Liquid Limit of 26% (low plasticity), a Plasticity Index of 10 % and Grading Modulus of 1.91. Van der Merwe potential expansiveness classification of the residual sandstone classified as *Low*, with Weston swell potential estimated at 0.02%.

Tactile assessment made in each of the test pits revealed a reasonably consistent and considerable increase in soil consistency with depth where the natural ground profile is encountered. Refusal, where experienced, generally occurred on bedrock, dense/very dense residual soils, competent ferricrete or boulders within the residual matrix.

6.6.2 Site C

Transported Soils (Alluvium and Aeolian)

The samples recovered from the alluvial stratum dominating the low-lying north-eastern portion of the site also exhibited notable variance.

Sample classification ranged from SM (silty sands) to CL (low plasticity clays) according to USC. Liquid Limit values varied between 0% and 41%, whereas Plasticity Indices (of the whole sample) varied between 0% and 30%. Grading Moduli between 0.4 and 0.84 were reported. Van der Merwe potential expansiveness classification ranged between *Low* and *Medium*, whereas the Weston swell potential ranged between 0% and 0.39%.

The aeolian soil samples recovered reported as being non-plastic and having a typical Grading Modulus of approximately 0.7, a *Low* potential expansiveness classification, an SM (silty sand) classification according to USC and a Collapse Potential of 6.9%[2].

Residual Soils (Sandstone)

Aeolian soils on the site are generally underlain by residual sandstone which has been reworked to varying degrees.

Laboratory testing revealed that the residual and reworked residual material classifies as a SC (clayey sand) material with Liquid Limit of 26% (low plasticity), a Plasticity Index of 10% and Grading Modulus of 1.91. Van

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[1] Weston DJ, Expansive road treatment for Southern Africa. Proceedings from the 4th International conference on Expansive soils, Denver, Vol.1, pp339-360

[2] According to the method of Schwartz, K: Problem soils in South Africa ï Collapsible Soils: State of the Art, The Civil Engineer in South Africa, July 1985

der Merwe potential expansiveness classification of the residual sandstone classified as *Low*, with Weston swell potential estimated at 0.02%.

Tactile assessment made in each of the test pits revealed a reasonable increase in soil consistency, in the natural soil profile, with depth. Refusal was not encountered in all instances, but where encountered it generally occurred on sandstone bedrock or well cemented ferricrete. Within the low-lying areas, depth of the alluvial horizon could not be determined owing to its thickness typically exceeding the depth reach of the TLB.

Dynamic Probe Super Heavy (DPSH) Testing

Five (No.5) DPSH tests were performed across Site C in order to determine the in-situ subsoil density and consistency. Refusal to probing was reached when the number of blows to penetrate 300mm exceeded 100, at which point, the test was terminated. The average refusal depth was calculated to be approximately 2.0m below natural ground level.

DPSH tests C1, C2, C3, C4 and C5 were conducted in close proximity to test pit positions TPC5, TPC3, TPC4, TPC2 and TPC1 respectively. The probe results indicate that the subsoil material is generally ñvery looseòto ñlooseòto an average depth of 1.62m, becoming ñmedium denseòto ñdenseòwith increasing depth. **Table 4** overleaf indicates the nearby test pit subsoil material associated with the DPSH test results. Difficult penetration was experienced through the residual conglomerate soil horizons, as well as in areas with shallow bedrock.

Table 4: Summary of Empirical SPT N Values and Nearby Subsoil Material - Site C

Depth (m)	TPC5	DPSH C1	TPC3	DPSH C2	TPC4	DPSH C3	TPC2	DPSH C4	TPC1	DPSH C5
0.3	Fill	0	Coll.	0	Coll.	0	Coll.	0	Coll.	0
0.6	Res. Conglom.	36		6		1	Res. Conglom.	2	Res. M.Stone	14
0.9		R	Coll.	12	Rew. Res. Conglom.	7		14		12
1.2	Conglom.			3		6		36		6
1.5				7		5	Res. Conglom.	R		3
1.8				5		3				2
2.1			Res. M.Stone	5	Res. Conglom.	2				3
2.4				30		2				36
2.7				30		2				R
3.0				60		2				
3.3				R		12				
3.6						24				
3.9						30				
4.2						30				
4.5						36				
						R				

Where:

Res = Residual M.Stone = Mudstone
Rew = Reworked Conglom = Conglomerate
Coll = Colluvium Fill = Fill material
R = Practical Refusal

The DPSH test results can be found in **Appendix E**.

6.6.3 Site F

Fill

Large portions of the area investigated on site are overlain by fill that was utilised as cover for the mine rehabilitation operations currently being undertaken. Coal discard was also occasionally encountered. Although no laboratory testing was undertaken on these horizons, the following tactile assessment of the fill material properties is presented below.

The material is deemed to generally classify as SM (silty sand) material. GM (gravel-sand-silt mixtures) and GP (poorly graded gravel-sand mixtures, little fines) may in some instances also be encountered. The majority of these materials are likely to have a low plasticity, and potential expansiveness is therefore also likely to be low.

Should this site be the preferred option for the development of the proposed ash facility, it is recommended that laboratory testing is undertaken to confirm the tactile assessment presented above.

Fill material utilised in rehabilitation of the site generally exhibited medium dense to dense consistency indicating some (but not significant) compaction during fill placement.

Transported Soils (Aeolian)

The material properties of samples recovered from the aeolian and ferruginised aeolian horizon ranged from SM (silty sands) to CL (low plasticity clays). Liquid Limit values varied between 0% and 33%, whereas Plasticity Indices varied between 0% and 14% and Grading Moduli between 0.44 and 0.76. Reported van der Merwe potential expansiveness classification ranged between *Low* and *Medium*, whereas the Weston swell potential ranged between 0% and 0.32%.

Residual Soils (Sandstone)

The upper portions of the residual sandstone underlying the site have in some instances been reworked by ferruginisation.

Laboratory testing revealed that the reworked material classify as a CL material (low plasticity clay) with a Liquid Limit ranging between 28% and 35%, a Plasticity Index of 13% to 15% and a Grading Moduli between 0.46 and 0.79. Van der Merwe potential expansiveness classification of the residual sandstone classified as *Low* to *Medium*, with Weston swell potential estimated at between 0.08 and 0.17%.

Tactile assessment made in each of the test pits revealed a reasonable increase in soil consistency with depth.

Refusal was only encountered in one instance, on sandstone bedrock.

6.6.4 Site H

Transported Soils (Colluvium)

The sample recovered from the overlying transported horizon exhibited the following material properties.

A single disturbed subsoil sample was collected from TPH15 (0.0 ï 0.9m). The sample classified as ML (sandy silt with gravel) according to the Unified Soil Classification (USC) system. Liquid Limit and Plasticity Index values were determined to be non-plastic and the Grading Modulus was determined to be 0.98. Reported van der Merwe potential expansiveness classification and the Weston[3] swell potential both showed that the material is not expansive.

Residual Soils (Mudstone)

Two disturbed subsoil samples were collected from TPH1 and TPH7 from 0.5 ï 1.9m and 0.3 ï 2.3m respectively.

[3] Weston DJ, Expansive road treatment for Southern Africa. Proceedings from the 4th International conference on Expansive soils, Denver, Vol.1, pp339-360

Laboratory testing revealed that the residual material classify as a CL material (sandy lean clay) according to the Unified Soil Classification (USC) System. Liquid Limit values varied between 22% and 24%, whereas Plasticity Indices varied between 11% and 12% and Grading Moduli between 0.47 and 0.51. Van der Merwe potential expansiveness classification of the residual mudstone ranged between *Low* at TPH1 becoming *Low* to *Medium* at TPH7, with Weston swell potential estimated at between 0.02 and 0.05% for TPH1 and TPH7 respectively.

No refusal was only encountered on mudstone bedrock.

Pedogenic Soils (Nodular Ferricrete)

Two disturbed subsoil samples were collected from TPH3 and TPH13 from 0.0 – 0.8m and 0.8 – 2.9m respectively.

Laboratory testing revealed that the pedogenic materials classify as SM material (silty sand with gravel) and CL material (sandy lean clay) according to the Unified Soil Classification (USC) System at TPH3 and TPH13 respectively. Liquid Limit values varied between 0% (non-plastic) and 28%, whereas Plasticity Indices varied between 0% (non-plastic) and 11% and Grading Moduli between 0.51 and 1.4. Van der Merwe potential expansiveness classification of the pedogenic material ranged between *Low* at TPH13 to *negligible* at TPH3, with Weston swell potential estimated at between 0.20 and 0.0% for TPH13 and TPH3 respectively.

Pedogenic Soils (Hardpan Ferricrete)

Two disturbed subsoil samples were collected from TPH6 and TPH8 from 2.0 – 2.9m and 0.8 – 2.2m respectively.

Laboratory testing revealed that the pedogenic materials classify as CL material (sandy lean clay with gravel) and CL material (sandy lean clay) according to the Unified Soil Classification (USC) System at TPH6 and TPH8 respectively. Liquid Limit values varied between 24% and 33%, whereas Plasticity Indices varied between 11% and 15% and Grading Moduli between 0.46 and 1.02. Van der Merwe potential expansiveness classification of the pedogenic material ranged between *Low* at TPH6 becoming *Low* to *Medium* at TPH8, with Weston swell potential estimated at between 0.03 and 2.34% for TPH6 and TPH8 respectively.

Dynamic Probe Super Heavy (DPSH) Testing

Dynamic Probe Super Heavy testing was performed across Site H in order to determine the in-situ subsoil densities and consistencies. The probe results indicate that the subsoils are generally medium dense, becoming dense with increasing depth. Difficult penetration was experienced in areas where nodular and hardpan ferricrete was present, as well as shallow bedrock.

Refusal to probing was reached when the number of blows to penetrate 300mm exceeded 100, at which point, the test was terminated. Eight DPSH tests were performed across site H. The average refusal depth was calculated to be approximately 2.1m below natural ground level.

DPSH tests H1, H2, H3, H4, H5, H6, H7 and H8 were conducted in close proximity to test pit positions TPH14, TPH2, TPH6, TPH8, TPH8, TPH11, TPH14 and TPH15 respectively. The probe results indicate that the subsoil material is generally "medium dense" with areas which can be described as "very loose" to "loose". The average depths, consistencies and associated nearby test pit subsoil material is summarised in **Table 5**. Difficult penetration was experienced through the hardpan ferricrete soil horizons, as well as in areas with shallow bedrock.

Table 5: Summary of Empirical SPT N Values and Nearby Subsoil Material - Site H

Depth (m)	TPH14	DPSH H1	TPH2	DPSH H2	TPH6	DPSH H3	TPH8	DPSH H4	TPH8	DPSH H5	TPH11	DPSH H6	TPH14	DPSH H7	TPH15	DPSH H8
0.3	Nod. Ferr.	0	Coll.	0	Coll.	0	Nod. Ferr.	0	Nod. Ferr.	0	Nod. Ferr.	0	Nod. Ferr.	0	Coll.	0
0.6		16		7	Coll.	10		17		30		18		36		18
0.9	Nod. Ferr.	20	Res. S.Stone	14		12		16		16	Rhyolite	20	Nod. Ferr.	R		9
1.2		13		7		7	Hard. Ferr.	7	Hard. Ferr.	14		25			Hard. Ferr.	3
1.5		6		5		3		3		36		36				3
1.8		3		7		3		2		R		R			Hard. Ferr.	6
2.1		7		21		30		2								17
2.4		13		28	Hard. Ferr.	8	Hard. Ferr?	17								36
2.7	Res. M.Stone	13		22		36		36					Res. M.Stone			R
3.0		13	Res. S.Stone?	23		R		R								
3.3	Res. Mstone?	19		29												
3.6		19	S.Stone?	36												
3.9		25		R												
4.2		32														
4.5	M.Stone?	36														
4.8		R														

Where:

Res = Residual
Ferr = Ferricrete
Coll = Colluvium
R = Refusal
M.Stone = Mudstone
Nod = Nodular
Hard = Hardpan
S.Stone = Sandstone
Rhyolite = Intrusive Rhyolitic Bedrock
? = Inferred

The detailed DPSH test results can be found in **Appendix E**.

7.0 GEOLOGY

7.1 Site Geology and Soil Profile

7.1.1 Site B Overview

During the course of the investigation, sandstone was encountered towards the centre and north-western portions of the site, with occurrences of shallow bedrock and/or bouldery bedrock associated with the (Loskop) lava in the eastern sector.

While the geological mapping shows the extent of the lava to be limited to the areas as shown in **Figure 2**, the coverage offered by the fieldwork programme, of even shallow depth, indicates the extent of the lava to occupy much of the eastern portion of Site B, with a minor occurrence of granite belonging to the Lebowa Suite encountered in the extreme western portion.

The minor occurrence of diabase indicated on the regional geology map was not confirmed during the course of the investigation. This can be attributed to both the probable limited extent of the diabase intrusion as well as the reasonably similar properties of the lava and diabase residual soils, thereby rendering separate identification challenging. The intrusive material has accordingly, been referred to as lava throughout this study and report.

Alluvial deposits associated with the Wilge River were encountered along the western limits of the site.

In general however, the soil profile as exposed during the course of this study is characterised by a thin to moderate layer of transported soils overlying residual (soil-quality) sandstone or lava (and minor granite), which in turn overlie generally shallow to moderate bedrock. Generally the consistency of the soil improves with depth with a moderate to shallow depth to bedrock.

A broad characterisation of each primary profile horizon, identified in the fieldwork programme, is given below, and the reader is referred to the soil profile reports in **Appendix B** to establish the extent of site-to-site variances and range of profile details for test site B. A summary of the soil horizons encountered within each of the test pits is presented in **Table 2, Appendix D.**

Transported Soils

The transported soils across the area investigated comprise a generally thin horizon of loose silty sand, interpreted as hillwash to depths of between 0.2m and 0.5m over most of the site. Soils of apparently aeolian origin generally underlie the hillwash to depths of between 0.75m and 1.8m, likewise comprising loose silty sands.

As a departure from the above characteristic, alluvial soils were encountered within test pit TPB20 in the western portion of the site (located towards the Wilge floodplain), to depths in excess of the 3m depth limit, comprising grey to pale green firm to stiff silty clays.

A distinctive but thin ***pebble marker*** horizon generally occurs beneath the transported soils (which identifies the transition between the overlying transported soils and underlying residual soils). In places the pebble marker was observed either near surface or was not defined/present at all, an occurrence not unusual in Karoo-dominant profiles. It generally comprises loose and friable silty sandy gravels with rounded pebbles occurring within a depth range of between 0.35m and 1.3m.

A degree of ferruginisation within the transported (and underlying residual) soils was generally noted in the soil profile, with some occurrences of well-developed ferricrete observed to occur to depths of between 1.1m and greater than 1.6m, comprising dense and better gravelly silty sand.

Residual Lava

Residual lava was encountered underlying most of the site investigated, represented by test pits TPB02 through TPB15. These soils were encountered to depths of between 1.5m and greater than 3m (test-pit depth limit).

Generally the upper portion has been reworked to depths of between 0.6m and greater than 2.8m comprising loose through dense silty sand, as a consequence of either ferruginisation or pore-producing mechanisms such as possible bioturbation (likely to be attributed to insect activity). The underlying residual lava comprises generally medium dense through dense silty sand.

Residual Sandstone

Residual sandstone was encountered within the central and northern portions of the site represented by test pits TPB16 to TPB19, occurring between depth range limits of 0.7m and 2.9m, comprising generally dense and better silty sand. The upper portion has been altered (mainly by ferruginisation), comprises dense and better silty and gravelly sands.

Bedrock

Bedrock interpreted as being of moderate to high strength, where encountered, occurs at depths of between 0.5m and deeper than 2.9m within the lava, sandstone and granite formations.

Groundwater

Groundwater seepage was only encountered within test pit TPB20 at a depth of 2.8m, i.e. in the alluvial soils along the Wilge River margin.

No other occurrences of groundwater were encountered on this generally high-lying (local watershed) site. The presence of ferricrete horizons identified across the site however, indicates the high probability of a seasonally fluctuating shallow (perched) groundwater table, during wet seasons, and particularly following periods of sustained or heavy rainfall.

7.1.2 Site C Overview

The first fieldwork programme confirmed the presence of sandstone and associated residual soils beneath the investigated (eastern) portions of the proposed site. From the second fieldwork programme in September 2014, the presence of residual mudstone, a granule conglomerate and residual sandstone was intersected.

Deep alluvial and colluvial soils were also encountered alongside the drainage margins of the Leeuwspruit and Wilgerivier as shown in **Figure 5, Appendix A**.

A broad characterisation of each primary profile horizon, identified in the fieldwork programme, is given below, and the reader is referred to the soil profile reports in **Appendix B** to establish the extent of site-to-site variances and range of profile details for the test site C. A summary of the soil horizons encountered at each test pit position is shown in **Table 5, Appendix D**.

Transported Soils

The transported soils across the site investigated comprise a generally thin horizon of loose silty sands, being (1) hillwash/colluvial deposits to depths of between 0.2m and 0.5m comprising loose to medium dense, silty sands, underlain by (2) aeolian soils to depths of between 0.6m and 2.0m, likewise comprising loose silty sands.

Alluvial soils were encountered within test pits TPC01, TPC02, TPC03 and TPC04A to depths in excess of the TLB reach, i.e. 3m (except TPC03A). The alluvium varies from grey loose slightly silty sand to stiff, grey brown sandy clay. Colluvial soils were encountered within test pits TPC1, TPC2, TPC3 and TPC4. The colluvium varies from red to brown, loose to medium dense, silty sand with the presence of roots.

The pebble marker horizon was generally absent, apart from test pit TPC08, where it occurred at a depth of 1.8m comprising a notably gravelly silty sand.

Residual Sandstone

From the first fieldwork programme, it was found that residual sandstone soils generally underlie the site, but were not encountered in areas where deep alluvial soils, shallow bedrock and/or ferricrete were observed. The residual sandstone, where present, occurs to depths ranging between 1.6m and greater than 3m. The upper portion has been reworked to depths of between 1.6m and greater than 2.9m, comprising ferruginised medium dense to dense silty sand to coarse sand. The underlying residual horizon is generally limited in thickness comprising dense and better silty sand.

During the second fieldwork programme over Site C, residual sandstone was only encountered in trial pit TPC3, of the five trial pits excavated. The residual sandstone occurred at a depth of 1.9m and greater than 3.0m, comprising soft, shattered, sandy clay soil.

Residual Mudstone

Residual mudstone was only encountered in trial pit TPC1, of the five trial pits excavated. The residual mudstone occurred from a depth of 0.3m to greater than 3.0m test pit termination depth, and comprised soft, micro-shattered, sandy clay.

Residual Conglomerate

Residual conglomerate soils generally underlie the western portion of Site C. The residual conglomerate occurs to depths from a depth of 0.3m to greater than 3.0m test pit termination depth, and comprised medium dense to dense, coarse sand with rounded to subrounded quartz gravel.

Ferricrete

Ferruginisation to various degrees was observed within the overlying transported soils, aeolian and alluvium, as well as the underlying residual sandstone soils. The ferricrete generally varies from a loose gravelly sand to a well-cemented hardpan ferricrete in places, and is an indicator of the presence of periodic perched groundwater conditions at these locations.

Bedrock Sandstone

Sandstone bedrock was encountered in the south-west sector within test pits TPC06, TPC05 and TPC05A at depths ranging between 0.5m and 2.2m. The sandstone bedrock generally deepens to the north and north-east to depths in excess of the 3m investigation limit, preliminary exposures of which were observed within a trench excavated by others along the western boundary, adjacent to existing temporary open-cast mine stockpiles.

Conglomerate

Granule conglomerate bedrock was encountered at trial pit TPC5, occurring from a depth of 0.95m to greater than 1.2m, comprising sand and rounded to subrounded quartz grains which are 2mm to 4mm in size.

Fill Material

Fill material was encountered in TPC5. The position of TPC5 falls within the mine concession. The fill material which occurs from the surface to approximately 0.35m depth comprises medium dense to dense, sandy fill with crushed shale. The material is used to surface the majority of the haul roads within the mine.

Groundwater

From the first fieldwork programme over Site C, groundwater was intersected within the lower-lying test pits TPC01 and TPC04A at depths of between 2.9m and 2.5m respectively, associated with the alluvial soils of the nearby Leeuwspruit. No groundwater was intersected during the follow-up second fieldwork programmer in any of the five trial pits excavated.

7.1.3 Site F Overview

Fieldwork carried out over the portion of the site accessible to investigation revealed that most of the area is underlain by disturbed ground in the form of general fill and/or opencast backfill, most of which appears to have been placed during recent and on-going rehabilitation activities.

The natural geology and ground profile of the (undisturbed) site comprises sandstone of the Vryheid Formation, (which includes the coal deposits), overlain by residual soils, which in turn are overlain by transported soils (of aeolian origin). In some further areas, the above-mentioned fill, generally associated with rehabilitation of the site, is observed to cover the natural un-mined ground profile.

A broad characterisation of each primary profile horizon, identified in the fieldwork programme, is given below, and the reader is referred to the soil profile reports in **Appendix B** to establish the extent of site-to-site variances and range of profile details for the test site F. A summary of the soil horizons encountered within each of the test pits is presented in **Table 4, Appendix D**

Fill Material

Fill material was encountered across most of the area investigated, apart from the northern-most areas represented by test pits TPF02 and TPF07. The approximate extent of the fill material is shown in **Figure 5, Appendix A.**

The fill, in the exposed horizons, comprises a variety of materials including silty sand with a minor waste rock component comprising gravels through small boulder sized particles, as well as coal discard from prior mining operations, identified to depths of between 0.15m and in excess of 2.9m (test-pit limiting depth).

The mass consistency of the fill material in these exposed upper horizons is generally estimated at medium dense to dense with little or no clearly distinguishable evidence of notable voids, suggesting that a modest degree of compaction has been applied to the rehabilitated surface layers.

Transported Soils

Transported soils of primarily aeolian origin were predominant in the (undisturbed) northern portions of the area investigated, either from surface, as in test pits TPF02 and TPF07, or elsewhere beneath a cover of fill material of between 0.15m and 1.1m thick.

The aeolian soils were encountered to depths ranging between 1.1m and 2.3m at the test locations and, where present, generally comprise loose silty sands.

Residual Sandstone

Residual sandstone soils were generally encountered directly beneath the transported horizon, reaching to depths of greater than 2.7m and 3m. The residual sandstone has been mostly reworked, by partial- to moderate ferruginisation, resulting in a medium dense clayey- and silty sand with a variable gravel fraction.

Bedrock Sandstone

Bedrock sandstone (of soft to medium hard rock quality) was only encountered within test pit TPF11 at a depth of 2.8m. No other occurrences of bedrock were observed within the test pits.

Sandstone bedrock was, however, observed within the two opencast pits located in the central-eastern and south-western portions of the site. The estimated depth to rock there is of the order of 4m from original ground level.

Groundwater

No groundwater was observed in any of the test pits excavated across the site.

Standing water was however, observed within the opencast pits described above at estimated depths of at least 5m below original ground level.

7.1.4 Site H Overview

Fieldwork carried out over Site H during the second fieldwork programme revealed that most of the area is underlain by pedogenic ferricrete of either nodular or hardpan ferricrete. Various sedimentary units of the Vryheid Formation, Karoo Supergroup, namely sandstone and shale were found to occur at some of the test positions located on Site H. Intrusive rocks of the Rooiberg Suite were encountered in two trial pits on the southern portion of the site.

The natural geology and ground profile of the (undisturbed) site comprises sandstones and mudstones of the Vryheid Formation, overlain by residual soils, which in turn are overlain by transported soils (of colluvial origin). In most areas, soils include some degree of ferruginisation with the presence of nodular ferricrete and hardpan ferricrete. This is evidence of historical groundwater.

A broad characterisation of each primary profile horizon, identified in the fieldwork programme, is given below, and the reader is referred to the soil profile reports in **Appendix B** to establish the extent of site-to-site variances and range of profile details for the test Site H. A summary of the soil horizons encountered within each of the test pits is presented in **Table 5, Appendix D.**

Transported Soils

Transported soils of primarily colluvial origin were predominant in the (undisturbed) northern portions of the area investigated (central portion of Site H), occurring from surface.

The colluvial soils were encountered to depths ranging between 0.0m and 2.4m at the test locations and, where present, generally comprise loose silty sand with partial roots.

Pedogenic (Hardpan and Nodular)

Pedogenic ferricrete was encountered in thirteen of the fifteen test pits excavated during the course of the investigation. At test pit TPH3, sparse subrounded ferricrete was found to occur within loose sand.

Nodular ferricrete was encountered in eight test pits (TPH5, TPH8, TPH9, TPH10, TPH11, TPH12, TPH13 and TPH14). The nodular ferricrete was generally found to comprise medium dense to dense silty sand with rounded and subrounded ferricrete nodules.

Hardpan ferricrete was encountered in eight of the fifteen test pits (TPH4, TPH5, TPH6, TPH7, TPH8, TPH9, TPH10 and TPH15). The hardpan ferricrete was generally found to comprise medium dense to dense, cemented ferricrete nodules with variable gravel fraction.

Residual Mudstone

Residual mudstone soils were generally encountered directly beneath the transported horizon, as noted in TPH1 and TPH7, reaching to depths of greater than 3.0m. The residual mudstone has been mostly reworked, by partial- to moderate ferruginisation occurring in the surrounding soil horizons. The residual mudstone soil generally comprise soft to firm, silty to sandy clay. At position TPH14, mudstone fragments were present within the residual horizon.

Residual Sandstone

Residual sandstone soils were encountered in test pits TPH2 and TPH3. The residual sandstone appeared intact with little reworking and no ferruginisation, comprising soft to very soft sandy clay.

Bedrock Sandstone

Bedrock sandstone (of soft to medium hard rock quality) was only encountered within test pit TPH3 at a depth of 1.0m. No other occurrences of sandstone bedrock were observed within the test pits. The sandstone bedrock was described as medium weathered, fine to medium-grained, massive, very soft rock strength.

Bedrock Shale

Bedrock shale (of soft rock quality) was only encountered within test pit TPH5 at a depth of 2.9m. No other occurrences of shale bedrock were observed within the test pits. The shale bedrock was described as highly weathered, fine-grained, thinly bedded, very soft rock strength.

Bedrock Intrusive

Bedrock intrusive, of rhyolitic composition (of soft to medium rock quality) was only encountered within test pits TPH11 and TPH12 at depths of 2.2m and 1.7m respectively. No other occurrences of intrusive bedrock were observed within the test pits. The intrusive bedrock was described as highly to completely weathered, closely jointed, soft to medium hard rock strength.

8.0 GEOTECHNICAL ASSESSMENT

8.1 Availability of Sites for Ash Facility Development

Golder was appointed to investigate Site B (1200ha), Site C (950ha), Site F (1200ha) and Site H (1150ha) arising from a pre-selection process identified in prior study by others for the intended (future) long-term location of the Kendal Power Station ash deposition facility.

Restrictions were however, placed on our site coverage as the fieldwork investigation programme commenced, by existing landowner arrangements, apparent recent land acquisitions, and currentlyproceeding opencast mining activities, which limited effective coverage of the sites to Site B (750ha) and Site F (200ha), which equates to roughly 60%, and 15% of total surface area, respectively for the three sites.

The full surface area of Site C and Site H was available for investigation, however the trace of Site H was moved after completion of the field programme. Site C is not favoured as a "preferred site" due to land ownership issues as well as the relative distance of the facility to natural water courses; therefore the western portion of Site C was investigated in detail.

As has been noted previously in this report, "*The very varied and complex geological character of this general locality, and the obvious resulting impact on productive shallow opencast mining (confined to the Vryheid Formation coal seams) in this area, is readily evident by the amount of seemingly discontinuous trial exploration and production mining pits which dot the landscape and, as a consequence, have created significant disturbance to the environment".*

As a general observation therefore, this report concludes that the stated objective of the study to characterise sites of approximately 4500ha (total area) has not been possible due to circumstances of land ownership and utilisation. An actual total area of approximately 3050ha was able to be characterised at the end of the investigation.

It is also pertinent to restate that observations, interpretations, conclusions and recommendations made in this report are ***representative specifically of the point locations investigated***, and may not be entirely relevant to unseen portions of the sites for which further study may be required prior to the detailed design phase.

8.2 Site Drainage and Groundwater

8.2.1 Site B

The portion of Site B that was made available for investigation is centred on or close to a local watershed, i.e. on high-lying topography. Observations of groundwater seepage at the time of investigation were therefore limited to the southwest perimeter near the Wilge River floodplain, where the depth of alluvial sediments exceeded the maximum attainable depth of investigation.

Construction in areas underlain by such seasonally saturated alluvial horizons will present significant challenges to design and construction of the starter wall facilities in terms of potential for instability, alternatively place limitations on the available footprint utilisation in these sectors of the site.

Further investigation will be required to delineate appropriate limits to the ash facility coverage or provide site specific parameters for appropriate slope foundation stability and detailed design. Groundwater, most likely from perched water, can however be anticipated.

8.2.2 Site C

The east and northeast portions of the area investigated are underlain by alluvial sediments, which in most instances extend to depths in excess of the limits of investigation. Groundwater seepage was only encountered within two of the test pits excavated at the time of investigation.

The investigations were undertaken at the end of winter (i.e. dry season). The development of a perched water table during periods of sustained or heavy rainfall can therefore not be excluded. The likelihood of seasonal perched water tables is indicated by the presence of the ferricrete observed within the transported and residual horizons, evident in various instances across the investigated portion of the site, including the higher-lying areas.

Profile considerations are such that the deep alluvial fringes of the site will inevitably be saturated during the wet season imposing constraints on practical depth of excavation and ground replacement for starter wall construction in these areas, and potentially limiting the extent of useful utilisation of the site.

8.2.3 Site F

No occurrences of groundwater were observed in any of the test pits excavated on the investigated portion of Site F.

Water (likely to be a combination of storm water run-off and natural groundwater seepages) was however observed generally in the defunct open-cast pits / trialï mining excavations evident on Site F.

Ferricrete was observed in the majority of test pits that intercepted the natural ground profile (i.e. where fill was absent or where test pits could be advanced through fill, into the natural ground profile). As with Site C, it is concluded that the development of a perched water table during periods of sustained or heavy rainfall can be anticipated on Site F.

8.2.4 Site H

Two occurrences of groundwater ingress were observed at two test pit positions during the investigation (TPH1 and TPH14).

At position TPH1 water ingress was observed from 1.6 ï 1.8m. TPH1 was situated along the approximate position of the inferred trace of the Ogies dyke, as well as lying within a ñwetlandò drainage pan. During the field investigation, farming crops had not encroached into the drainage pan. The groundwater water flow rate into TPH1 was not measured; however, the test pit did fill up about 0.1m from the base within about one minute (i.e. a moderate flow). At TPH14 water ingress was observed from 2.8 ï 3.0m. TPH14 is not located close to any drainage features, wetland pans or any structural features and the groundwater flow is much less than that observed in TPH1.

Ferricrete was observed in the majority of test pits. As with Sites F and C, it is concluded that the development of a perched water table during periods of sustained heavy rainfall can be anticipated as probable on Site H. Groundwater seepage can also be expected along the trace of the Ogies dyke.

8.3 Excavation Characteristics

Excavatability has been assessed according to the National Standard Construction Specifications contained in *SANS 1200D: Earthworks,* (1998).

In terms of the Standard, the excavation classes can be summarised as follows:

- *Soft Excavation* is material that can be efficiently removed without prior ripping by 22t bulldozer (such as Cat D7) or front end loader
- *Intermediate Excavation* is material that can be efficiently ripped by a 35t bulldozer (such as Cat D8) with a single ripping tyne
- *Hard Rock Excavation* is material that cannot be efficiently ripped by 35t bulldozer (such as Cat D8), and requires blasting or splitting
- *Boulder Excavation* contains greater than 40% of boulders larger than 0.03m^3 in size (*Class A*), or contains less than 40% boulders larger than 0.03m^3 in size but requires blasting to remove (*Class B*). This excludes rock, which should be classified as *Soft* to *Hard Rock Excavation*.

8.3.1 Site B

Generally *Soft Excavation* is interpreted within the overlying transported and residual soils to depths within a range of 0.5m and greater than 3m.

There is likely to be a relatively narrow zone of *Intermediate Excavation, comprising highly weathered rock. O*n most cases this was not specifically proven, but can nonetheless be anticipated.

Hard Excavation is interpreted within moderately and less weathered rock, and below the zone of *Intermediate Excavation* (where encountered). *Hard Excavation* is anticipated at depths in a range between 0.5m and greater than 3m i.e. within bedrock lava, sandstone and granite.

Boulder Excavation, although not conclusively proven across the site, is anticipated within the residual lava horizon due to its characteristic mode of weathering. It is very noticeable that lava boulders have been previously removed from the profile and stockpiled to promote agricultural activity in the area. Hardpan ferricrete stockpiles which could either classify as *Boulder* or *Hard Excavation* were also observed, mainly in the areas underlain by lava.

8.3.2 Site C

Soft Excavation was encountered across the site within the transported and residual soils to depths of between 0.5m and greater than 3m, but typically deeper than 1.5m.

Intermediate Excavation was generally not proven/encountered, however can be anticipated within a narrow zone of highly weathered rock.

Hard Excavation is anticipated occurring below the soil / sandstone bedrock or soil / hardpan ferricrete interface, and within medium or less weathered rock materials.

8.3.3 Site F

Soft Excavation can be expected generally to depths of the order of 3m and more across the site.

No *Intermediate* was not specifically encountered within the test pits, however can nonetheless be anticipated as a narrow layer within the upper rock profile.

Hard Excavation was only encountered in one test pit at a depth of 2.8m on sandstone bedrock, and can thus be anticipated over the site below the soils and within moderately or less weathered rock.

Boulder Excavation can be expected within the fill/backfill material.

8.3.4 Site H

Soft Excavation was encountered across the site within the transported and residual soils to depths of between 0.9m and greater than 2.8m, but typically deeper than 1.5m.

Intermediate Excavation was generally proven/ encountered between the soil/ hardpan ferricrete interface.

Hard Excavation can be expected below the soil / sandstone bedrock or intrusive rhyolite bedrock interface and within moderate or less weathered rock.

8.4 Soil Compressibility

Fill

Fills encountered in the upper 3m on Site F typically have consistencies ranging between medium dense and dense and therefore expected to be modestly compressible depending on the degree to which backfill compaction was controlled during rehabilitation. Some degree of variability in the consistency of the fill is anticipated.

Backfill material at depth (i.e. beyond the reach of the current investigations) is in an unknown state, may comprise very variable material and particle size, and substantial voiding and can be anticipated to be in a state of on-going settlement. Further investigation will be required during detailed study to properly evaluate the extent and behaviour of deep backfills.

Transported Soils (Hillwash/Aeolian/Alluvial)

The transported hillwash, aeolian and alluvial soils typically exhibited consistencies ranging between loose and medium dense for non-cohesive soils and soft to firm for cohesive soils. The majority of transported soils subjected to loads associated with the ash facility are generally expected to be moderately compressible with some degree of variability.

The uppermost (estimated 0.75m depth) transported soils however, often exhibited loose consistencies, a porous matrix and/or artificial disturbance (from agriculture), and are therefore generally expected to be moderately to highly compressible, requiring appropriate subgrade treatment before placement of raised (engineered) earthworks.

Residual soils (Sandstone/Lava)

The residual lava soils encountered at Site B generally exhibited medium dense consistencies which in most instances increased to dense to very dense within the depth limit of investigation. Occasional pockets of loose to medium dense residual lava were also observed.

These soils are anticipated to be slightly compressible in places by virtue of their variability. Residual sandstone encountered on all three sites exhibited similar or better consistency improvement with depth. These soils are similarly anticipated to be only slightly compressible in places.

8.5 Chemical Characteristics

Limited laboratory testing was undertaken to ascertain the pH, resistivity and conductivity of the soils sampled on sites B, C and F during the initial testing program. Similar chemical testing was conducted on samples collected from Site H during the second testing program. Chemical characteristics of the sampled soils are summarised in **Table 5** overleaf. For a full set of results reference should be made to **Appendix C**.

As noted below Tables 5 and 6, the assessments of water and soil slurry and the results do not directly reflect values of water extracts of the soil samples. The chemical test results are described below:

- At Site B the pH values ranged between 5.4 and 6.0 with resistivity values in the range of 11 628 to 24 390 Ohm/cm. This means that should the subsoil materials become immersed in water, the pH of the resultant mixture would be moderately acidic and the resistivity would indicate that the soil would generally not be corrosive.
- At Site C the pH values ranged between 4.8 to 6.3, with resistivity values in the range of 10 526 to 20 000 Ohm/cm. Therefore, should the subsoil materials become immersed in water, the pH of the resultant mixture would be moderately to strongly acidic and the resistivity would indicate that the soil would generally not be corrosive.
- At Site F the pH values ranged between 4.5 to 5.7, with resistivity values in the range of 3623 to 18 182 Ohm/cm. Therefore, should the subsoil materials become immersed in water, the pH of the resultant mixture would be moderately to strongly acidic and the resistivity would indicate that the soil would range from being very corrosive to generally not corrosive.
- At Site H, the pH values ranged between 6.75 and 7.96, with resistivity values in the range of 2088 to 8540 Ohm/cm. Therefore, should the subsoil materials become immersed in water, the pH of the resultant mixture would be slightly acidic to moderately alkaline and the resistivity would indicate that the soil would range from being mildly corrosive to very corrosive.

These soils may have a corrosive effect on buried steel and concrete based purely on the pH and resistivity results, however more detailed chemical testing, including Langelier Saturation, Ryznar Stability Indices and Basson Index testing, would be required to determine the extent of the activity/ corrosivity towards metals and concrete.

Table 6: Summary of Chemical Test Results (Sites B, C and F)

PARAMETER	RANGE OF VALUES	PRELIMINARY CORROSIVITY ASSESSMENT*
pH	4.5 ~ 6.3	Moderately to strongly acidic
Resistivity, R (q/cm)	Approximately 3600 ~ 24500+	Non- to very corrosive
Conductivity, EC (mS/m)	0.04 ~ 0.56	Non-corrosive to moderately-corrosive

*Relates to assessment on water samples. Assessments may not necessarily apply to water extracts of soils samples

Table 7: Summary of Chemical Test Results (Site H)

PARAMETER	RANGE OF VALUES	PRELIMINARY CORROSIVITY ASSESMENT*
pH	6.75 ~ 7.96	Slightly acidic to moderately alkaline
R Resistivity (q/cm)	Approximately 2088 ~ 8540+	Very- to mildly corrosive
EC Conductivity (mS/cm)	0.117 ~ 0.479	Non-corrosive

*Relates to assessment on water samples. Assessments may not necessarily apply to water extracts of soils samples

The pH and corrosivity of the subsoil samples tested from Site H were found to be slightly acidic to moderately alkaline. This is more favourable to the construction of buried steel and concrete structures, as there is a lower likelihood of corrosive action by the subsoils. This result is corroborated by the relatively high resistivity and low electrical conductivity values.

8.6 Permeability and Potential Requirement for Liner

The installation of the liner for the ash disposal facility requires a minimum permeability value of 10^{-5} cm/sec in the subsoil material. Prior to the commencement of the second program of field testing, preliminary falling head testing was performed by Zitholele Consulting. Material was collected from the upper 300 to 400mm of Site H, and produced results in the range of 10^{-8} to 10^{-11} cm/sec. The samples were also subjected to soil improvement by the addition of 4% and 8% bentonite clay. By adding bentonite, the permeability of the subsoil material was improved, producing permeability rates of up to 10^{-13} cm/sec. The samples were remoulded to 95% Proctor, saturated and tested under a load of 100kPa.

During the second program of field testing, representative subsoil samples were collected from TPH3 and TPH7 and sent for permeability testing. The samples were remoulded to 90% and 95% Mod AASHTO using the Mod value determined in test No. 1583 and 1585 as the 100% values (refer to **Appendix C**). Test results returned permeability rates of 10^{-5} and 10^{-6}cm/sec. The permeability values of the subsoil materials have been shown to be above the required limit of 10^{-5} cm/sec and should, therefore, be suitable for the proposed liner system after the material has been ripped and recompacted to 95% Proctor at +2% OMC.

Large-scale field trials will be required in order to assess the effectiveness of the ripping and recompacting to provide quality control during the earthworks on site. These should comprise double-ring infiltrometer testing, including possibly supplementary Guelph Permeameter testing.

Only basic and preliminary soil testing was carried out for corrosivity. Further testing is required in order to confirm the results.

9.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the geotechnical assessments and interpretations, the following conclusions and recommendations are presented:

9.1 Groundwater, Drainage and Site Preparation

- Portions of Site B and Site C underlain by alluvial soils (as outlined in the figures included in **Appendix A**) as well as any other low-lying areas subject to seasonal inundation may pose challenges as far as constructability and serviceability of the ash facility is concerned, or lead to curtailment of the footprint coverage in such areas;
- The alluvial zones identified in the first testing program study constitute a reasonably small proportion of sites B and C. It is therefore recommended that such areas should be delineated and preferably not be included as part of the designated ash disposal facility development footprint;
- Should the ash facility be required to extend onto these alluvial areas, a combination (but not necessarily all) of the following mitigating measures may need to be considered in the design and construction strategy, depending on extent and severity of groundwater, and any associated negative ground profile characteristics:
  - Appropriate measures to control groundwater to address impacts on the constructability and serviceability of the proposed ash facility and its initial ground preparation works, including:
    - pre-drainage measures which will be exacerbated if construction commences during the wet season.
    - sub-surface- and cut-off drains to be constructed in advance of the main earthworks operations.
  - Trafficking and preparation of work areas / access routes for earthmoving and other construction equipment may prove difficult in sectors bordering on the margins of the alluvial floodplains;

    - Importation of pioneering layers (and possible geotextile separation layers) in areas where aforementioned mitigation proves to be inadequate or need to be supplemented;

- Based on the confirmed presence of ferruginisation in some areas across sites, B, C, F and the distinct presence of ferruginisation at site H, periodic occurrences of localised seasonal groundwater seepage can generally be anticipated across the sites, the impacts of which should be likewise appropriately considered. Owing to the porous, friable nature of the uppermost soils, it is recommended that appropriate sub-grade treatment of the in-situ soils is undertaken subsequent to initial topsoil and vegetation removal to ensure the integrity of raised engineered fills/ starter walls, composite liner receiving layers and the like, which might be negatively impacted by yielding founding material.

  This is likely to entail ripping and re-compaction of upper horizons. Deeper impact compaction may also be required in areas where deeper aeolian and unconsolidated fill is present;

- Coarse materials (such as a ferricrete horizon or mining related waste) are deemed unsuitable to receive the envisaged composite geosynthetic liner system;

  In all instances where coarse materials are encountered at the sub-grade level, an appropriate receiving layer of compacted soil (complying with the liner manufacturerõs specifications) will be required to provide necessary protection of the composite liner;

- Where deep opencast exploration or mining pits have been established, backfilling undertaken or other significant ground disturbance has occurred, special rehabilitation measures will be an essential pre-requisite to establishment of a secure lined ash disposal facility;

- Based on Golderõs Technical Memo, ñ*Kendal 30yr*ò, we believe Sites B, C and H to be unaffected by mining operations. This information has been provided to Golder under a confidentiality agreement between Golder and the mining companies. Mining at Site F was evident during the first testing program. Unfortunately, detailed underground mine plans, either historic or present, were unavailable during the investigation and compilation of this report;

- The proposed ash disposal facility and liner system require permeability rates of 10^{-5} cm/sec (based on the recommendation made by the Department of Environmental Affairs), or better, of the underlying soil material. From falling head permeability tests conducted at positions TPH3 and TPH7 at site H, average permeability rates of 10^{-5} cm/sec, 10^{-6} cm/sec and 10^{-10} cm/sec are possible. The required rates of permeability may therefore be achievable by ripping and recompacting the upper 400 to 500mm (nodular ferricrete and residual mudstone) material between 90% and 95% Mod AASHTO. These values were obtained by plotting a curve (test 1583 and 1585 in **Appendix C**). We anticipate the in situ density of the soil to range between 1814 and 1851 kg/m^3;

- From the preliminary investigation of Sites B, C, F and H, Site H has been identified as the ñpreferred siteò for the construction of the ash disposal facility, provided that the recommendations for design and construction are adhered to;

- The slope stability analysis conducted for the construction of a lined ash disposal facility was conducted for Site H, as this was the preferred site based on the geotechnical investigation and the findings outlined within the scope of this report. A factor of safety of 1.2 was achievable for the overall design which is feasible in terms of the required outcomes for the Client;

- We understand that in order to design for sufficient airspace, the proposed ADF will be partly located over the surface trace of the Ogies Dyke, as indicated from regional geological maps. As mentioned previously, we were unable to confirm presence of the Ogies Dyke during the field investigation. We consider that the ADF may be constructed over the Ogies Dyke provided that the site preparation and design procedures as described in this report, or as may be recommended as the design progresses or during construction. This is further described in Section 9.4 below;

9.2 Construction Materials

The following general comments, considered relevant to the investigated portions of all four sites, apply:

- Proposed future borrow investigations (refer next section) will be needed to provide reasonable estimates / quantities and a representative range of design parameters for the available materials for construction of the proposed ash facility and/or in-situ compaction as receiving layers for geomembrane liners;
- The soils encountered within the footprint of Site H require ripping and recompaction at optimum moisture content to comply with the requirements for the anticipated geomembrane liner (Class C) configuration. As such, stringent earthworks quality control and quality assurance must be in place during the construction phase.

  Additional subsoil samples collected from Sites C and H by Zitholele Consulting, were treated with increasing percentages of bentonite (4% and 8%) and subjected to falling head permeability testing. Although the bentonite treated samples provided rates in the order of 10^{-12} cm/sec, the permeability test results for the untreated samples also provided marginal to suitable permeability rates for the proposed construction;
- Provision needs to be made for double handling, stockpiling, drying-out, re-importation and compaction of saturated soils that are otherwise suitable as construction material (subject to outcomes of future borrow investigations); and
- Similarly provision needs to be made for removal and temporary stockpiling or spoiling (subject to outcomes of future borrow investigations) of upper soft/loose soils which may be necessary.

Specific recommendations relevant to the individual sites are presented below.

9.2.1 Site B

The majority of the transported soils (excluding alluvium), as well as the residual lava soils, classify as reasonably favourably for construction of homogeneous (blended material) starter walls envisaged for the proposed development.

It is interpreted that the upper (predominantly cohesionless aeolian) SM soils will need to be combined with the low-plasticity clayey (CL) and sandy (SC) residual soils thereby providing a material with the plastic fines fraction required to attain a reasonably workable material. Furthermore, the alluvial soils sampled on this site are deemed to have a too high a silt fraction to be considered as suitable for starter walls and homogeneous embankments without blending.

9.2.2 Site C

The majority of sampled alluvial soils comprising low plasticity clay (CL) and/or predominantly sandy soils with a notable silt fraction(SM) encountered on this site, exhibited favourable characteristics for the construction of starter wall embankments. In some instances the potential swell classification of the alluvial soils exceeded acceptable levels and would require appropriate blending to be considered for usage. Typically alluvial soils with Van der Merwe activity classifications of medium or higher should preferably not be considered for construction material without blending.

The residual sandstones classify as predominantly sandy soils with a modest clay fraction (SC) or low plasticity clays (CL) and are also deemed suitable for starter wall embankments. This horizon was limited in extent (i.e. thickness) in some of the test pits and may therefore yield limited construction material if considered as a potential material source.

9.2.3 Site F

Predominantly aeolian and ferruginised derivatives found in undisturbed profiles on this site exhibited similar characteristics to those encountered on Sites B and C. As such these materials are considered reasonably suitable for the construction of starter walls but should ideally be combined with the clayey (CL) residual sandstone in order to attain a combined material with an adequate fines fraction to meet acceptable workability requirements.

Tactile assessment of the mine waste fill encountered on this site, indicates that this material may, subject to appropriate testing, prove to be suitable source of material to construct starter walls, provide it is combined with suitable cohesive material, such as the underlying residual sandstone.

9.2.4 Site H

The majority of sampled soils comprising low plasticity clay (CL) and/or predominantly sandy soils with a notable silt fraction encountered on this site, exhibited favourable characteristics for the construction of starter wall embankments. In some instances the potential swell classification of the pedogenic and residual mudstone soils exceeded acceptable levels and would require appropriate blending to be considered for usage. Transported soils with Van der Merwe activity classifications of medium or higher should preferably not be considered for construction material without dilution.

The residual mudstones classify as predominantly sandy to silty clayey soils with low to medium plasticity clays (CL) and are not deemed suitable for starter wall embankments. This material will require blending to be considered for usage. This horizon was limited in extent (i.e. distribution across site H) and present in only two test pits, TPH1 and TPH7 and may therefore yield limited construction restrictions.

9.3 Construction and Operational Constraints

- Potential for excessive settlement and slope failure are visualised in the deeper, predominantly cohesive alluvial horizons present on the western portion of Site B and the eastern and north-eastern portions of Site C. These constraints need to be formally addressed in subsequent studies if the proposed ash disposal facility is envisaged to encroach onto these areas. Mitigation measures necessary to address these risks may include:
  - Soil improvement: Removal and replacement of the alluvium to the fullest extent possible, while maintaining necessary drainage to minimise the impact of poor foundations on slope stability;
  - The composite liner system will need to demonstrate satisfactory strength characteristics so that it may effectively accommodate differential settlements and resulting elongation, whilst remaining functional;
  - Basal reinforcement of the ash facility perimeter, utilising geosynthetic grids or other suitable means (to be determined following conclusion of future geotechnical investigation, materials testing and slope stability analyses); and
  - Application of appropriate operational procedures in such sectors of the proposed ash facility, including limitations on allowable heights, greater set-backs to reduce effective slope angles, and the like.
- Potential concerns may be escalated by the Department of Water Affairs with the Client, regarding the presence of the natural pan located in the southern portion of Site H. The proper procedures must be followed in order to construct over the pan and facilitate the reworking of the underlying material to meet the specific design requirements of the proposed liner system for the ash disposal facility.

9.4 Construction over Ogies Dyke

We understand that airspace requirements necessitate the construction of the ADF over the possible trace of the Ogies Dyke, as defined using regional geological information. To this end, we recommend that the following procedures are implemented in order to safely build over the Dyke:

- Further geotechnical and hydrogeological investigations should be undertaken in order to assess the nature and strength of the weathered near surface properties of the dyke, as well as the hydrogeological properties in terms of the potential for and volume of water production. The studies should be phased where appropriate. Investigations should at least extend to the residual horizon (below the transported horizon), and may include trenches across the likely trace of the dyke, geophysical surveys, laboratory testing, etc.
- The detailed design should mitigate the risks associated with construction over the Dyke, including risks associated with potentially weak or reactive soils, differential settlement, and potential water production along the dyke. The following are potential mitigating design elements which may be required below the footprint of the ADF:

- Excavate all residual dyke material to ensure material strength and clay properties are at least similar or better than the surrounding ADF subgrade material;
- Install a robust subsurface dewatering/drainage system to ensure that all water produced as a result of preferential flow along the dyke margins, daylighting of groundwater against the dyke, or due to the increased ADF surcharge, is effectively and permanently drained. The system may incorporate subsurface drainage elements within natural soils, as well as elements in the lowermost ADF (pioneering) layers.

9.5 Related Issues

Based on information made available, none of the sites investigated appears to be underlain by historical underground mining activity which would preclude it from consideration for the long-term proposed ash deposition facility.

The potential effects of recently proclaimed mining operations bordering all of the investigated sites need careful considerations. Operations such as major open-pit blasting within close proximity of the proposed ash disposal facility may have significant bearing on its stability.

In an Environmental Impact Assessment Report compiled by Blast Management and Consulting for the Heuvelfonetein Colliery on the 23/05/2013 relating to the ground vibration and air blast effects, it is explicitly outlined within the executive summary that the, "*ground vibration yielded from blasting is expected to be high and could contribute to damages of structures*". Specific mitigation of the blasting operations will be required to ensure the stability of the ash disposal facility.

The majority of sampled materials collected from Sites B, C and F during the first program of field testing exhibited potential for corrosion on buried services such as steel and reinforced concrete. Samples collected from Site H during the second program suggest that the soil is slightly acidic to moderately alkaline and mildly corrosive to very corrosive. Further testing should be undertaken in future studies to determine the potential corrosivity towards steel and concrete or where deemed appropriate by the design engineers or where key material properties may be compromised.

Consideration should also be given in future studies to the chemical and thermal compatibility between the ash (to be stored on the facility) and the various constituents of the envisaged composite liner system, which has not been assessed in this study.

9.6 Slope Stability

Preliminary slope stability analyses have been briefly considered, based on prior and equivalent studies undertaken for the Continuous Ashing Site, which is intended to serve as the interim extension for the current ash facility at Kendal Power Station.

Owing to the generally non-cohesive nature of soils dominating much of the currently-considered four sites, it is anticipated that the likelihood of slope instability attributed to deep seated failures is very limited. With the exception of the specific deep alluvial areas identified during the current study, along the margins of the existing alluvial floodplains, slope stability considerations will largely be attributed to operational issues surrounding the ash quality and strength parameters, with particular emphasis on the introduction of the composite liner which will tend to act as the weakest potential slip surface.

Two distinct modes of failure have been identified with conceptual level factors of safety for both modes of about 1.2. This is at the bottom end of the industry accepted minimum range of 1.2 to 1.3. These modes comprise a failure at the toe of the ADF that would impact the liner system, and the failure mode at the mid-slope bench which would mobilize a large volume of material but would not impact the liner system.

In this regard reference is made to Golder's earlier technical memorandum on this subject, which considered the issues surrounding slope stability of the ash facility in some detail. Relevant recommendations in the aforementioned documentation should also be referred to in addressing the provisional requirement for the four sites considered in this report.

Measures to address the occurrence of liner instability were nonetheless addressed in Golder's earlier technical memorandum and would also apply to areas considered in this report where steeper surface topography is present. The geometry upon which the ash disposal facility has been modelled for stability analyses is feasible with the low factor of safety values quoted above. The effects of prevailing steeper surface topography (measured to be up to 6% in some areas) on the internal stability of the composite liner was specifically considered in this instance and will warrant further consideration during detailed studies to ensure that internal stability of the composite liner is attained.

A detailed discussion on the slope stability analyses conducted for the proposed construction on Site H can be found within the Technical Memorandum attached under **Appendix F**.

10.0 CONCLUDING RECOMMENDATIONS

The geotechnical characteristics of the four sites, based on information derived from preliminary investigations, are defined in this report along with preliminary geotechnical recommendations.

Based on our interpretations from the current investigations, no apparent evidence exists in our opinion, to suggest that the sites are fatally flawed from a geotechnical perspective. Significant challenges have been identified for each of the sites which will give rise to important design and construction expedients which must be adequately addressed in detailed design and project specifications.

Further geotechnical investigations (as outlined in the next section) will be required to support / confirm current assumptions and to advance the current conceptual-level design into a definitive final engineering design suitable to construct the proposed facility.

The key findings of the investigation have been determined as follows:

- Owing to access constraints imposed by current landowners on Sites B, C and F investigated during the first program, as well as a change in the Site H outline, only a limited portion of each site could be accessed to undertake the planned invasive field investigations. All discussions, conclusions and recommendations included in this report must therefore be considered as relevant only to those portions of the sites where investigations were undertaken, and not deemed representative of the remainder of each of the respective sites unless and until equivalent (invasive) corroborating studies are implemented to prove such validity;

- Laboratory test results undertaken on selected samples of surface soil horizons indicate that the majority of material sampled exhibits (or may be blended to provide) reasonably favourable characteristics when considered for the construction of homogeneous starter wall embankments or liner receiving layers associated with the ash facility. In order to improve material workability it will generally prove advantageous to combine (or blend) the generally cohesionless transported horizons with the underlying residual soils for optimal performance;

- Deep seasonally-saturated alluvial (cohesive) soil horizons are envisaged which will affect relatively small portions of Site B and Site C. It is recommended that the proposed ash disposal facility should not extend onto these areas. Alternatively, should these areas be considered to form part of the ash facility footprint, potentially significant pre-cautionary drainage and earthworks mitigation measures may inevitably need to be implemented to prepare these work areas for construction;

- The recommended geomembrane liner system comprises a 2mm, textured geomembrane placed on soil horizons with a permeability rate of 10^{-5} cm/sec, or better. Here, reference is made to Golder's earlier technical memorandum on the matter of general slope stability and the internal stability of envisaged composite liner system. Relevant recommendations are made in this document and are also relevant to addressing the provisional requirement for Site H (the preferred site) considered in this report; and

- Additional work is recommended to establish detailed design parameters for the preferred ash deposition facility, preliminary details of which are outlined in the section below.

11.0 RECOMMENDATIONS FOR FURTHER GEOTECHNICAL INVESTIGATION

Detailed investigations are recommended to support construction materials evaluations, stability analyses, liner strength, engineering and liner design of the proposed development and will need to be undertaken prior to- and feed in to the detailed engineering design phase. Such investigations are expected to include:

- A combination of drilling, probing and/or additional (deeper) test pitting and materials testing, the details and extent of which are subject to the preferred site selected and undertaken to determine the depth and properties for analysis of foundations which will impact on ash disposal facility design;
- Further geotechnical and hydrogeological studies should be carried out to investigate the position and nature of the Ogies Dyke, and in order to provide design recommendations. This may include geophysical surveys, drilling, test pitting, trenching, as well as on-site and laboratory testing.
- Comprehensive borrow and materials investigations may prove essential to confirm on-site resources and identify additional off-site sources required for any earthworks components, including the soil buffer anticipated to be required to protect the geomembrane liner from pozzolanic activity of the ash;
- Investigations on areas of concern on the preferred site which have not yet been investigated in great detail owing to constraints imposed during the current investigations. Inferences have been made for areas between the investigation points using professional judgement;
- In view of planned and currently on-going mining operations for Sites B, C and F, and the extent to which the Sites are likely to be physically transformed between now and the onset of detailed design of the ash facility, it is recommended that the extent to which the preferred site will potentially be transformed, be ascertained and detailed studies be planned accordingly; and
- From discussions and ongoing assessment of the suitability of Sites B, C, F and H for the construction of the ash disposal facility, Site H has been identified as the preferred site. Detailed investigations on Site H will be required during the detailed engineering design phase.

GOLDER ASSOCIATES AFRICA (PTY) LTD.

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NR/SOC/mfb

Reg. No. 2002/007104/07
Directors: RGM Heath, MQ Mokulubete, SC Naidoo, GYW Ngoma

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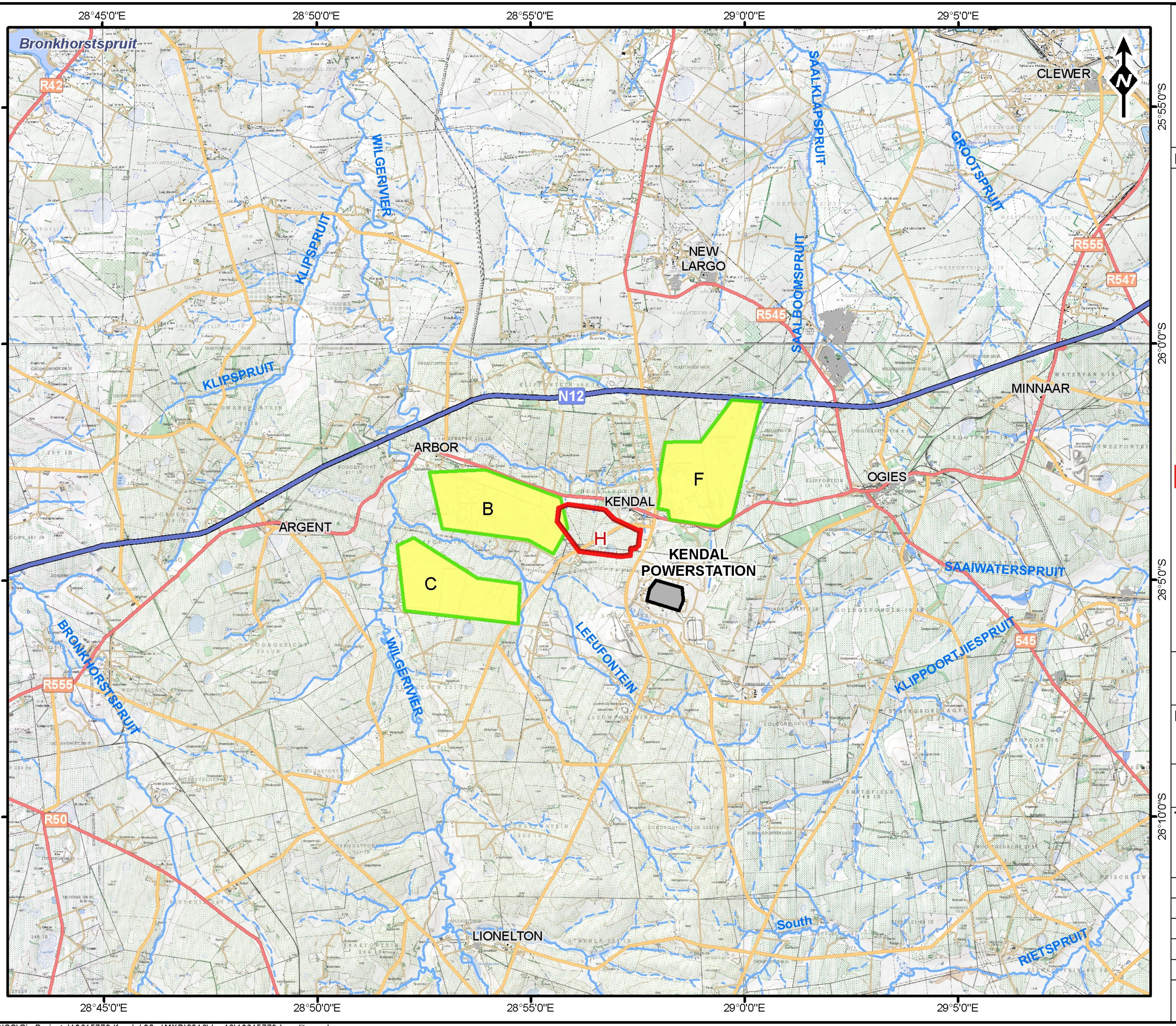
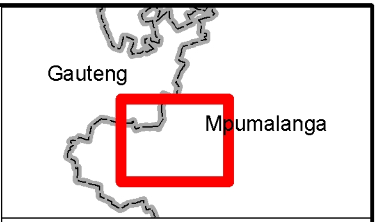
APPENDIX A

Figures





LEGEND

- National Route
- Main road
- National Route
- Main road
- Secondary road
- Access road
- Street
- Rivers - Perennial
- Rivers - Non perennial
- Dams
- Kendal Powerstation
- Site Alternatives
- Site Alternative H

0 0.9 1.8 3.6 5.4 7.2 9
Kilometers

RESOURCES\NOTES

Coordinate System: WGS84 LO29
Projection: Transverse Mercator

PROJECT

KENDAL 30 YEAR

TITLE

FIGURE 1:
LOCALITY MAP

PROJECT No. 13615779		REV 0
SCALE 1:200,000		A4
GIS	CS	09/06/2016
CHECK	NR	09/06/2016
REVIEW	WK	09/06/2016

Golder Associates
Johannesburg, South Africa

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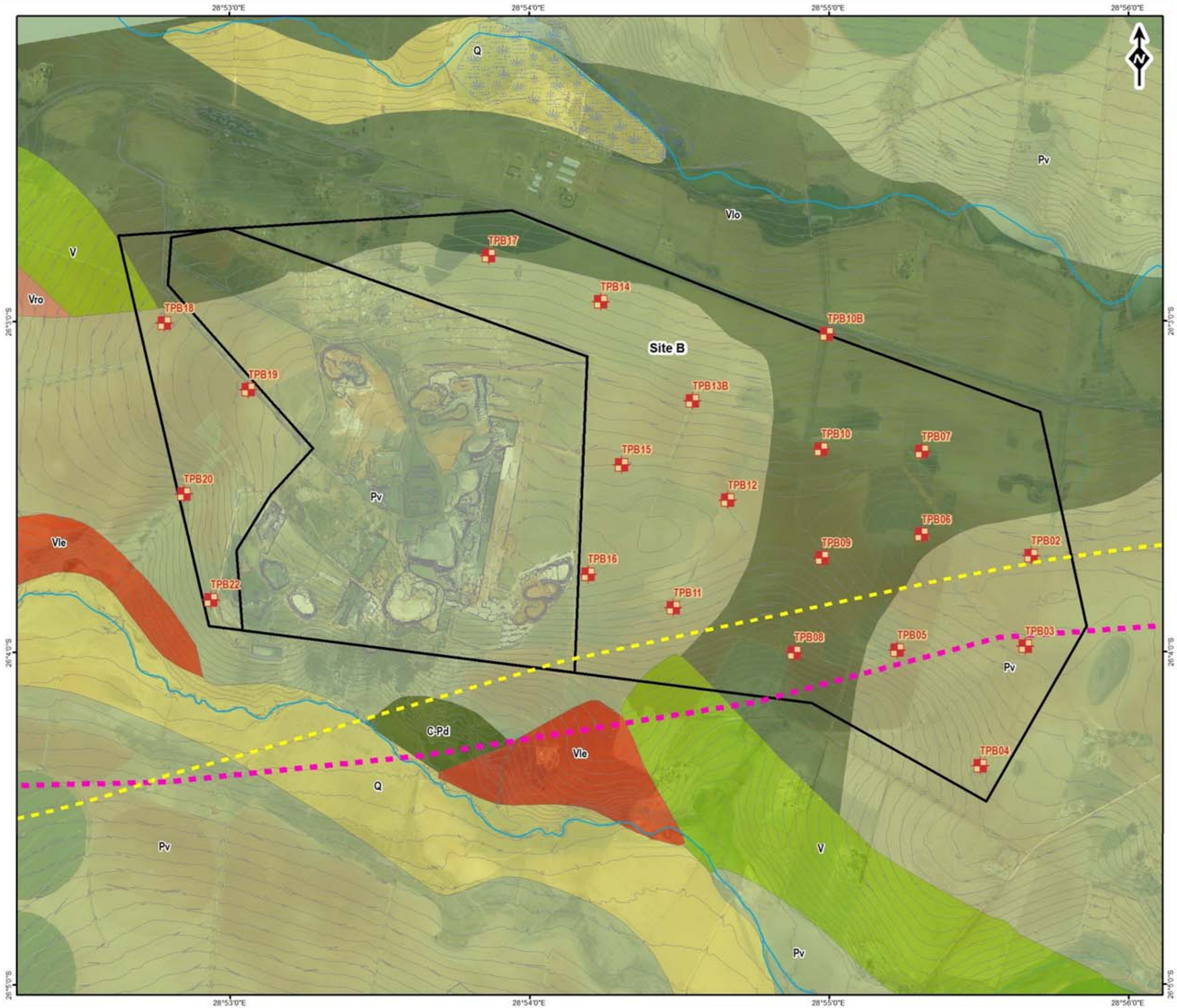

LEGEND

- Test Pits - First Program
- Ogies Dyke (Core Groundwater Technical Services cc - unreferenced data)
- Ogies Dyke (Geology Map 2628 of East Rand, Department of Minerals and Energy, 1986)
- Rivers - Perennial
- Rivers - Non perennial
- Contours (2m)
- GeoTech Sites
- Marsh vlei
- Alluvium
- Diabase
- Diamictite (polymictic clasts, set in a poorly sorted, fine-grained matrix) with varved shale, mudstone with dropstones and fluvioglacial gravel common in the north
- Fine- to coarse-grained sandstone, shale, coal seams
- Granite
- Massive porphyritic rhyolite, pyroclastic rocks
- Pyroclasts, lava

RESOURCES\NOTES

250 125 0 250 500 750
Meters

REFERENCE

Coordinate System: WGS84 LO29

PROJECT

GEOTECHNICAL INVESTIGATION: KENDAL 30 YEAR ASHING

TITLE

FIGURE 2: SITE B

PROJECT No. 13615331		REV 4
SCALE 1:20 000		A3
GIS	SS	17/09/2014
CHECK	NR	17/09/2014
REVIEW	SOC	17/09/2014

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Johannesburg, South Africa

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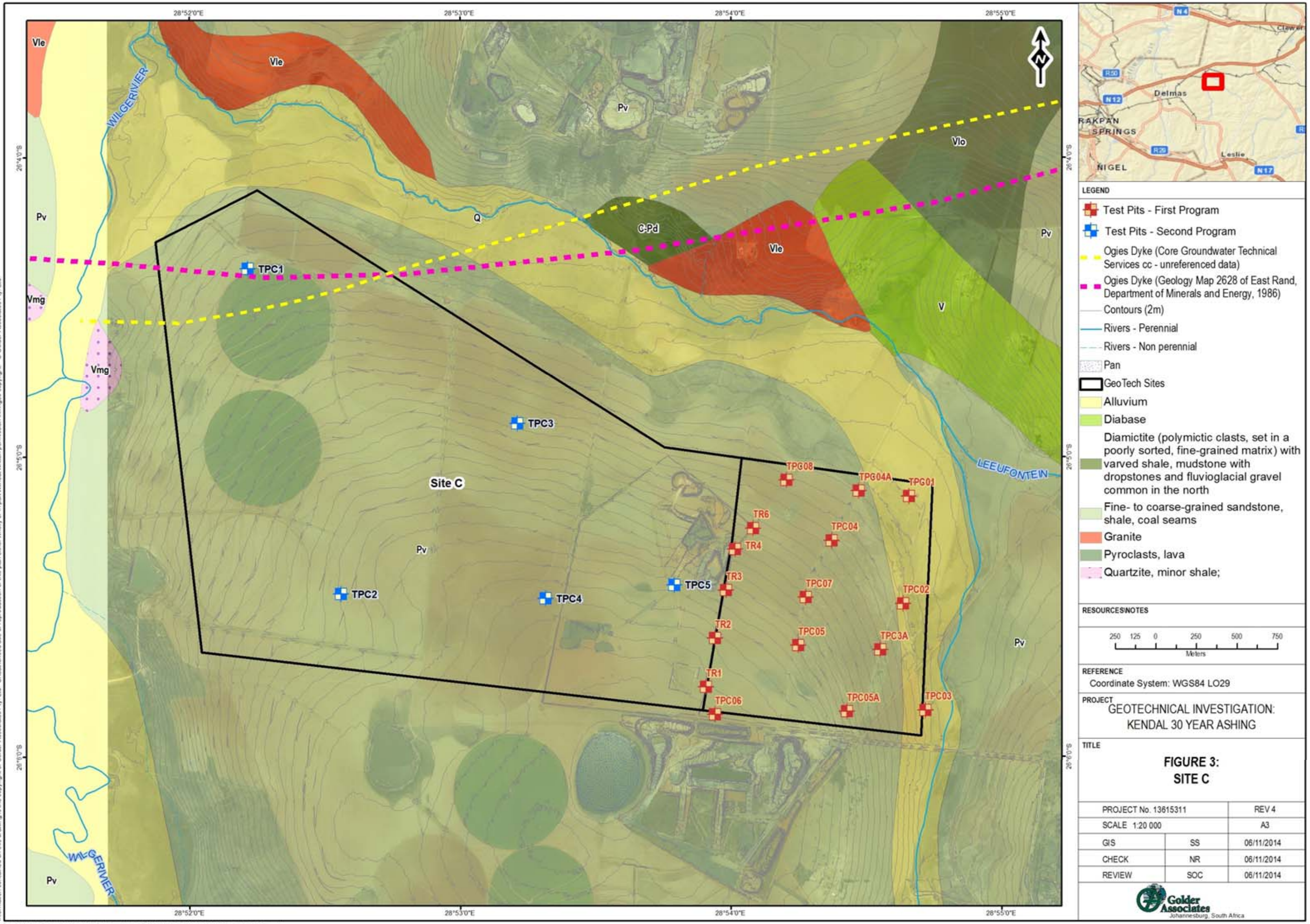

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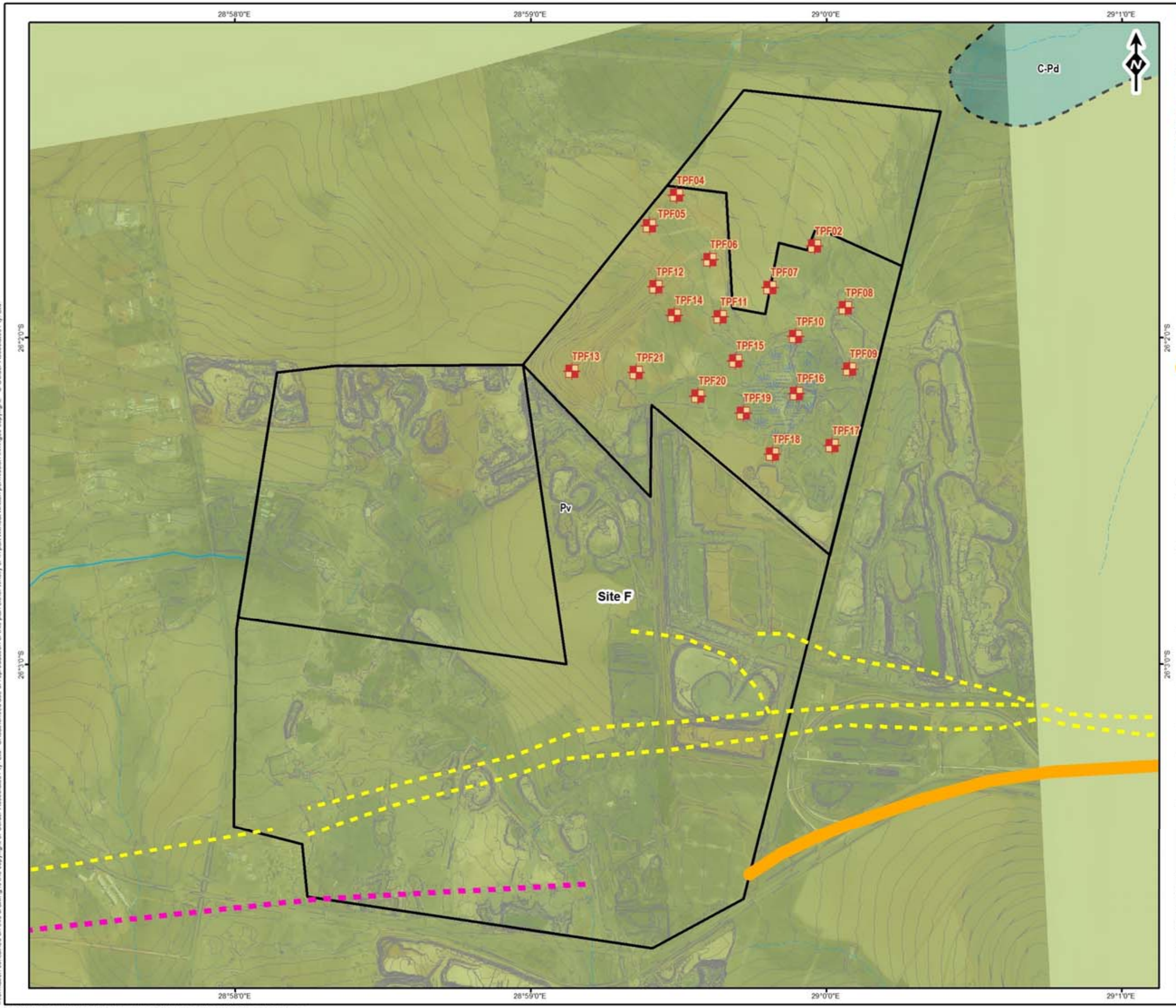

LEGEND

- Test Pits - First Program
- Ogies Dyke (Core Groundwater Technical Services cc - unreferenced data)
- Ogies Dyke (Geology Map 2628 of East Rand, Department of Minerals and Energy, 1986)
- Dolerite Dyke (Council for Geoscience - unreferenced data)
- Marsh vlei
- Rivers - Perennial
- Rivers - Non perennial
- Contours (2m)
- GeoTech Sites
- Diamictite (polymictic clasts, set in a poorly sorted, fine-grained matrix) with varved shale, mudstone with dropstones and fluvioglacial gravel common in the north
- Fine- to coarse-grained sandstone, shale, coal seams

RESOURCES/NOTES

250 125 0 250 500 750
Meters

REFERENCE

Coordinate System: WGS84 LO29

PROJECT

GEOTECHNICAL INVESTIGATION:
KENDAL 30 YEAR ASHING

TITLE

FIGURE 4:
SITE F

PROJECT No. 13615311		REV 4
SCALE 1:20 000		A3
GIS	SS	17/09/2014
CHECK	NR	17/09/2014
REVIEW	SOC	17/09/2014

Golder Associates
Johannesburg, South Africa

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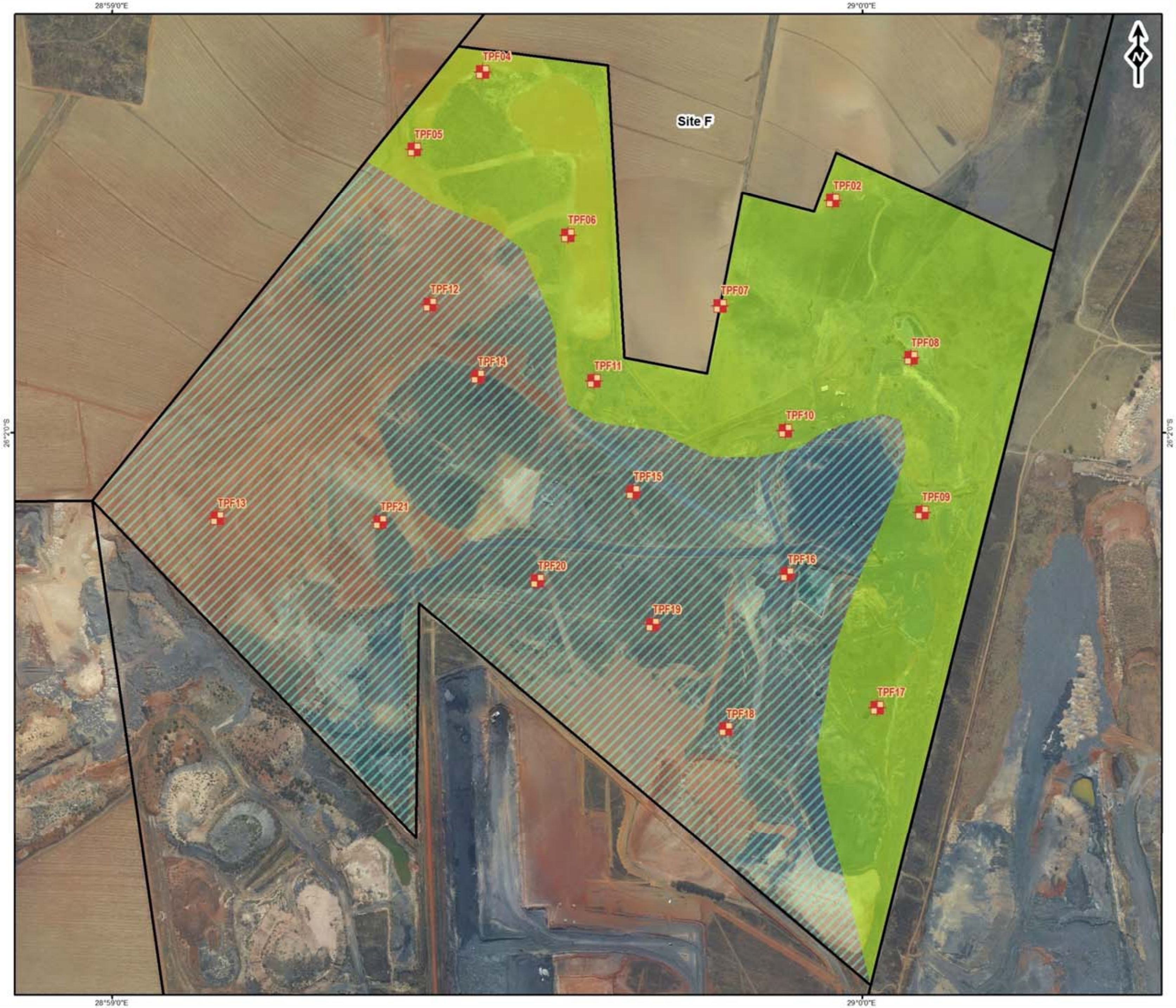

LEGEND

- Test Pits - First Program
- GeoTech Sites
- Fill
- Residual (Sandstone)

RESOURCES\NOTES

150 75 0 150 300
Meters

REFERENCE

Coordinate System: WGS84 LO29

PROJECT

GEOTECHNICAL INVESTIGATION:
KENDAL 30 YEAR ASHING

TITLE

FIGURE 4A
SITE F

PROJECT No. 13615311		REV 3
SCALE 1:8 000		A3
GIS	SS	29/10/2014
CHECK	NR	29/10/2014
REVIEW	SOC	29/10/2014

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Johannesburg, South Africa

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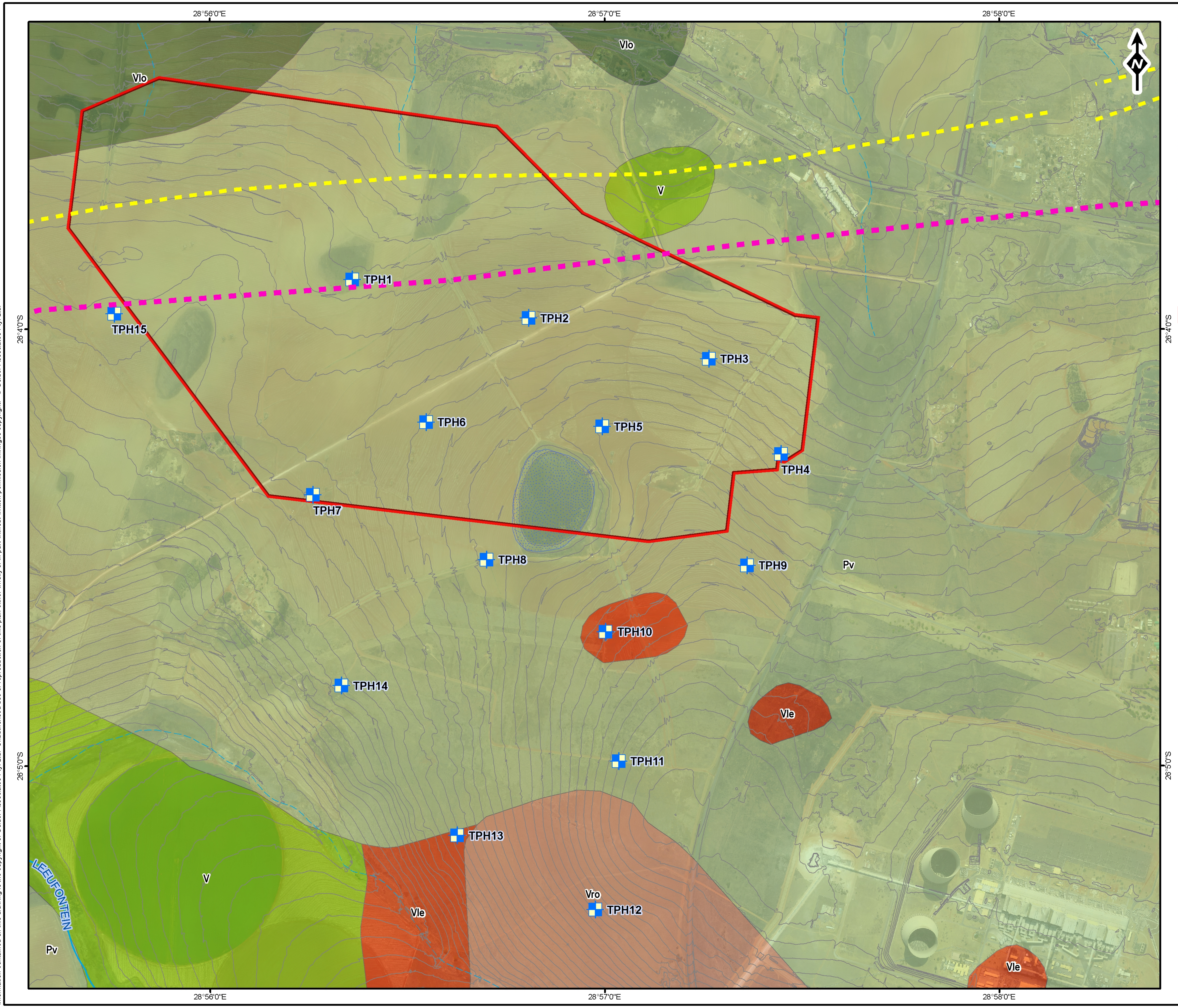

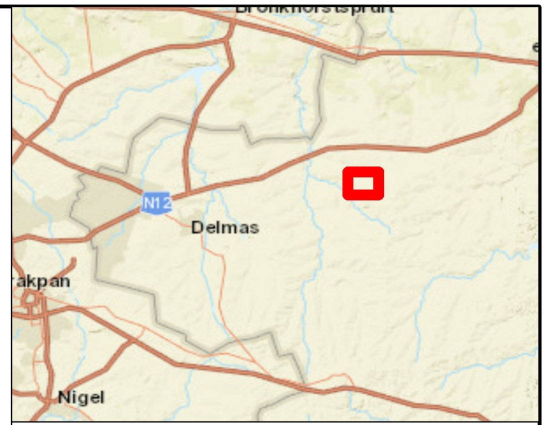

LEGEND

- Ogies Dyke (Core Groundwater Technical Services cc - unreferenced data)
- Ogies Dyke (Geology Map 2628 of East Rand, Department of Minerals and Energy, 1986)
- New test pits
- Site Alternative H
- Pan
- Rivers - Perennial
- Rivers - Non perennial
- Contours (2m)
- Diabase
- Fine- to coarse-grained sandstone, shale, coal seams
- Granite
- Massive porphyritic rhyolite, pyroclastic rocks
- Pyroclasts, lava

RESOURCES\NOTES

200 100 0 200 400 600
Meters

REFERENCE
Coordinate System: WGS84 LO29

PROJECT
GEOTECHNICAL INVESTIGATION:
KENDAL 30 YEAR ASHING

TITLE
FIGURE 5:
SITE H

PROJECT No. 13615311		REV 4
SCALE 1:15,000		A3
GIS	CS	09/06/2016
CHECK	NR	09/06/2016
REVIEW	SOC	09/06/2016

Golder Associates
Johannesburg, South Africa

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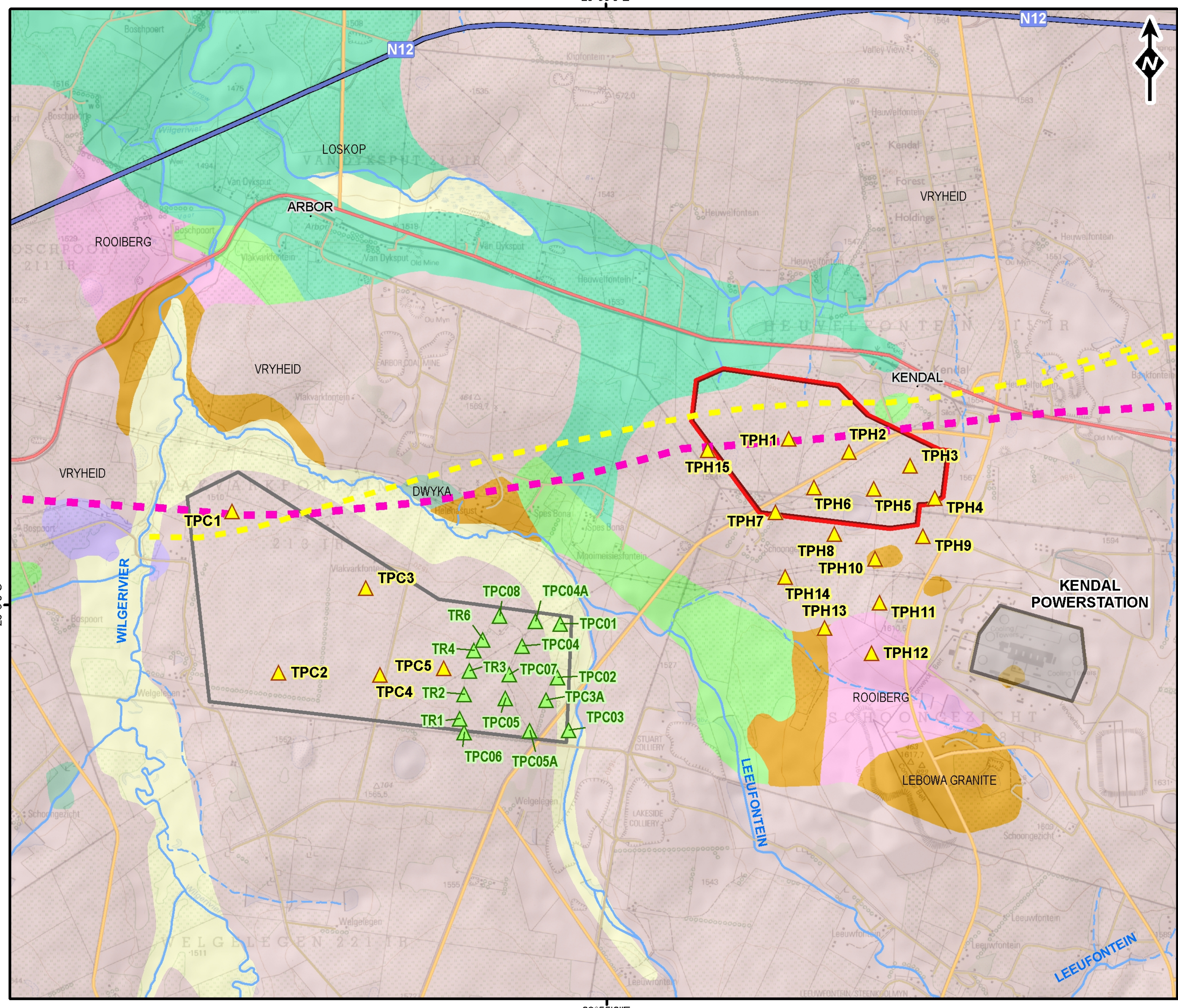

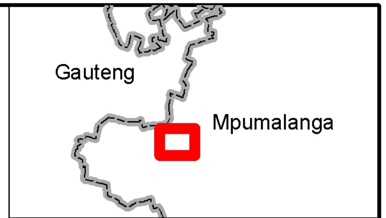

LEGEND

- Test Pits (Sept 2013)
- Test Pits (Sept 2014)
- Ogies Dyke (Core Groundwater Technical Services cc - unreferenced data)
- Ogies Dyke (Geology Map 2628 of East Rand, Department of Minerals and Energy, 1986)
- National Route
- Main road
- Secondary road
- Access road
- Street
- Rivers - Perennial
- Rivers - Non perennial
- Site Alternative C
- Site Alternative H
- Kendal Powerstation

Geology

- Magaliesberg
- Rooiberg
- Alluvium
- Vryheid
- Lokop
- Dwyka
- Diabase
- Lebowa Granite

0 0.5 1 2 3
Kilometers

RESOURCES\NOTES
Coordinate System: WGS84 LO29
Projection: Transverse Mercator

PROJECT
KENDAL 30 YEAR

TITLE
FIGURE 6:
SITE C AND SITE H

PROJECT No. 13615779		REV 0
SCALE 1:65,000		A4
GIS	CS	09/06/2016
CHECK	NR	09/06/2016
REVIEW	WK	09/06/2016

Golder Associates
Johannesburg, South Africa

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APPENDIX B

Test Pit Soil Profiles

Test Profiles available on CD

APPENDIX C

Laboratory Test Results

Laboratory test results available on CD

APPENDIX D

Tables

Tables available on CD

APPENDIX E

Dynamic Probe Super Heavy (DPSH) Test Results

DPSH Test results available on CD

APPENDIX F

Slope Stability Technical Memo

TECHNICAL MEMORANDUM

DATE 7 November 2014 **PROJECT No.** 13615779

TO Nevin Rajasakran
Zitholele Consulting

CC Simon Owens-Collins, Francois Marais

FROM Andrew Fuggle **EMAIL** afuggle@golder.co.za

SLOPE STABILITY ANALYSIS FOR PROPOSED KENDAL 30 YEAR ASH DUMP – SITE H

1.0 INTRODUCTION AND OBJECTIVE

Zitholele Consulting (Pty) Ltd. (Zitholele) commissioned Golder Associates Africa (Pty) Ltd. (Golder) to perform a geotechnical study for a site being considered for the proposed 30 year Ash Dump for the Kendal power station. The site is known as “Site H”. Golder understands that the geotechnical information from this investigation feeds into regulatory and licensing processes being handled by Zitholele.

This technical memorandum is focused on slope stability analysis of the proposed dump on Site H. In particular, this aspect of the scope of work was to:

- Perform a high level stability analysis
- Comment on potential stability problems that may be experienced or expected

The approach, assumptions, result and conclusions are described in this technical memorandum.

2.0 MODELLING APPROACH AND ASSUMPTIONS

The design of the proposed ash dump slopes was an iterative process between Zitholele and Golder. The final geometrical configuration for the ash dump was decided upon by Golder and Zitholele, and the stability analysis was performed using this configuration.

Golder understands that the ash type and composition that may be deposited at Site H is the same as that being deposited on the existing Kendal facility. Golder also understands that the mechanism of ash transport and placement (conveyor and stacker system) is the same for Site H as is currently being used.

LiDAR survey data of Site H and an existing ash dump facility at the Kendal power station was provided by Zitholele. The survey data indicated that the average slope of the natural ground at Site H was approximately 1.5%. The survey also showed that the bench face angle of the existing ash dump facility was in the range of 35 to 38 degrees. This is consistent with values reported in the literature (Fourie *et al.*, 1997).

We understand that the Eskom, and industry accepted minimum Factor of Safety for a similar waste facility comprising monitored, operational slopes is in the range of 1.2 to 1.3.

2.1 Ash Dump Configuration

The following configuration parameters were defined to model the ash dump:

- Bench face angle 38 degrees
- Natural ground slope 1.5%

- Horizontal distance between successive benches 100 m

The top surface of the ash dump was assumed to be parallel to the natural ground, i.e. at a slope of 1.5%. The total height of the modelled ash dump was 78.5 m, at an overall average slope angle of approximately 11 degrees.

The modelled ash dump configuration is shown in Figure 1.

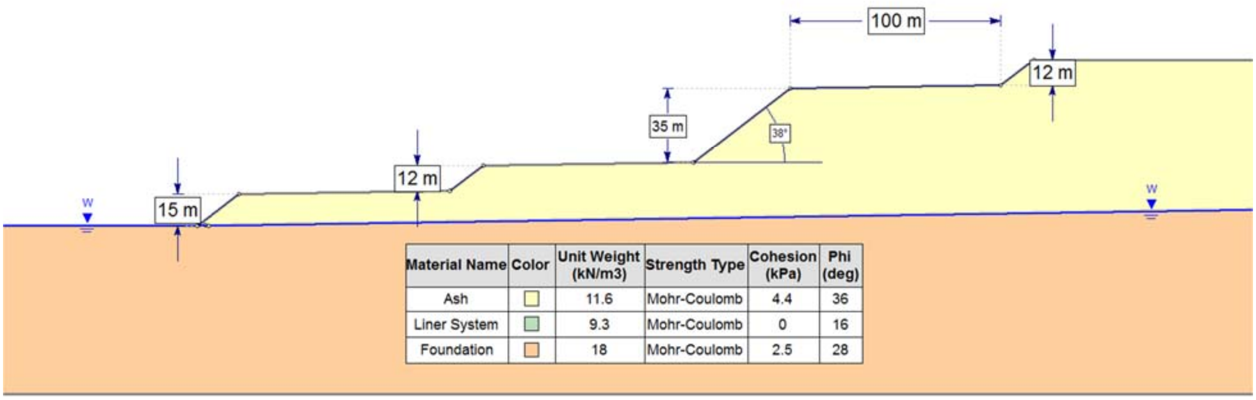

Figure 1: Modelled Site H Ash Dump Configuration

2.2 Material Properties

For stability modelling purposes, the ash dump is comprised of essentially three materials, namely:

- Ash
- Foundation (natural ground)
- Liner system

A summary of the material properties used for slope stability modelling is provided in Table 1.

Table 1: Summary of Material Properties used for Slope Stability Modelling

Material	Property	Value
Ash	Friction Angle	36 degrees
	Unit Weight	11.6 kN/m^3
	Cohesion due to Matric Suction	4.4 kPa
	Material Strength Model	Mohr-Coulomb
Foundation	Friction Angle	28 degrees
	Unit Weight	18 kN/m^3
	Apparent Cohesion	2.5 kPa
	Material Strength Model	Mohr-Coulomb
Liner System	Friction Angle	16 degrees
	Unit Weight	9.3 kN/m^3
	Apparent Cohesion	0 kPa
	Material Strength Model	Mohr-Coulomb

2.2.1 Ash

The critical state friction angle of the ash was assumed to be 36 degrees and the unit weight to be 11.6 kN/m^3. These values are based on data reported by Fourie *et al.* (1997), which includes experimental data for ash from the Kendal power station.

The ash friction angle is less than the angle at which the bench faces form. Dry-dumped ash exhibits a phenomenon known as matric suction. This matric suction increases the strength of the soil above what it would be under fully saturated (or completely dry) conditions. Field measurements of the Lethabo ash dump indicate suction values of between 30 and 40 kPa (Fourie *et al.*, 1997). Golder has assumed that similar suction values would be present in the proposed facility at Kendal. This assumption is based on the similarities between the ash from the Lethabo and Kendal power stations.

Fourie *et al.* (1997) further report that the rate of increase in the shear strength relative to suction (ϕ^b) is 20% of the value of the friction angle (ϕ). The contribution of suction to the shear strength can be included in stability modelling as an apparent cohesion value (Rahardjo and Fredlund, 1995) using the total cohesion method. This method can be implemented in conventional limit equilibrium analysis methods.

The total cohesion used in the analysis is given by the following equation:

$$c = c' + MS \times \tan(\phi^b)$$

where c = total cohesion
c' = effective cohesion (assumed to be zero)
MS = matric suction (assumed to be 35 kPa)

The value of total cohesion due to matric suction was calculated to be 4.4 kPa.

2.2.2 Foundation

Engineering properties of the foundation were assumed based on data obtained from Golder's field investigation.

2.2.3 Liner System

Golder understands that the proposed facility at Site H will be lined with a geomembrane liner system. The strength of this liner system (including any internal interfaces, as well as external interfaces with surrounding materials) was not assessed as part of this study. Typical values for a textured HDPE geomembrane in contact with a cohesive soil, as reported in published literature (Koerner and Narejo, 2005), were used to model the liner system. A residual friction angle of 16 degrees and zero residual cohesion were used in the modelling.

2.3 Perched Water and Groundwater Elevations

Based on Golder's field investigation, the groundwater table was assumed to be below the proposed geomembrane liner system.

No perched water table within the proposed ash dump was modelled. This is a very significant assumption.

Should porewater pressures (excess, seepage, or hydrostatic) be present within the ash dump for whatever reason, this analysis will not be applicable. The presence of water in the ash dump will reduce the factors of safety calculated in this analysis.

2.4 Modelling Approach

The stability of the proposed ash dump was evaluated using the computer software program Slide (version 6.0) using a two-dimensional generalized limit equilibrium method that satisfies both force and moment equilibrium. An in-built algorithm was used to search possible circular and non-circular failure surfaces to find the critical failure surface with the lowest factor of safety. Analysis was performed for static loading conditions only (loading cases including seismically-induced ground motions were not evaluated).

3.0 RESULTS AND DISCUSSION

Analysis showed two distinct modes of failure, as illustrated below:

- Block-type failure at the toe of the dump, passing through the liner system (Figure 2)
- Circular failure through the mid-slope bench (Figure 2)

Both modes of failure have factors of safety of 1.2. Failures of a global nature passing from the top crest through to the toe of the dump have factors of safetya greater than 1.5.

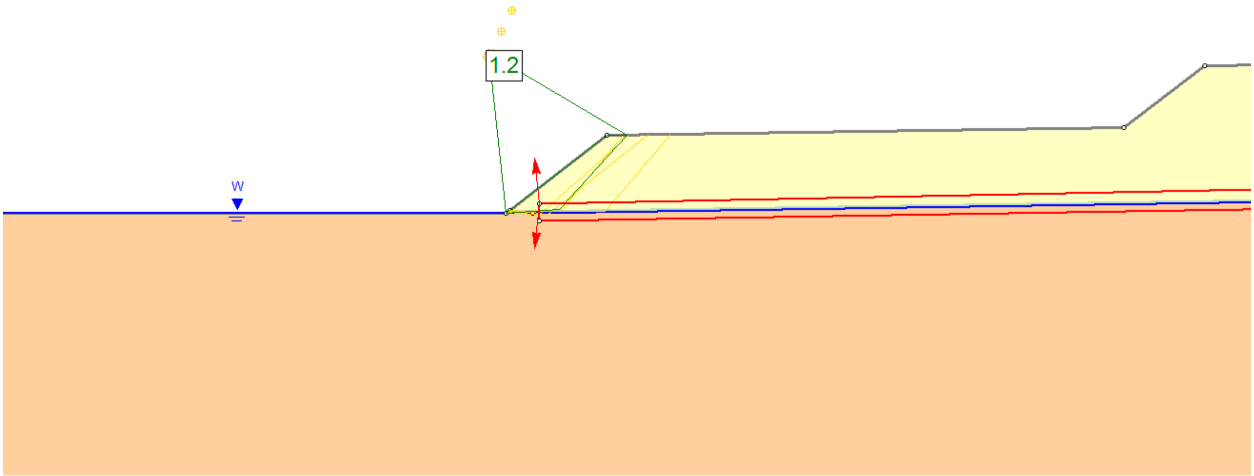

Figure 2: Block-Type Failure at Dump Toe

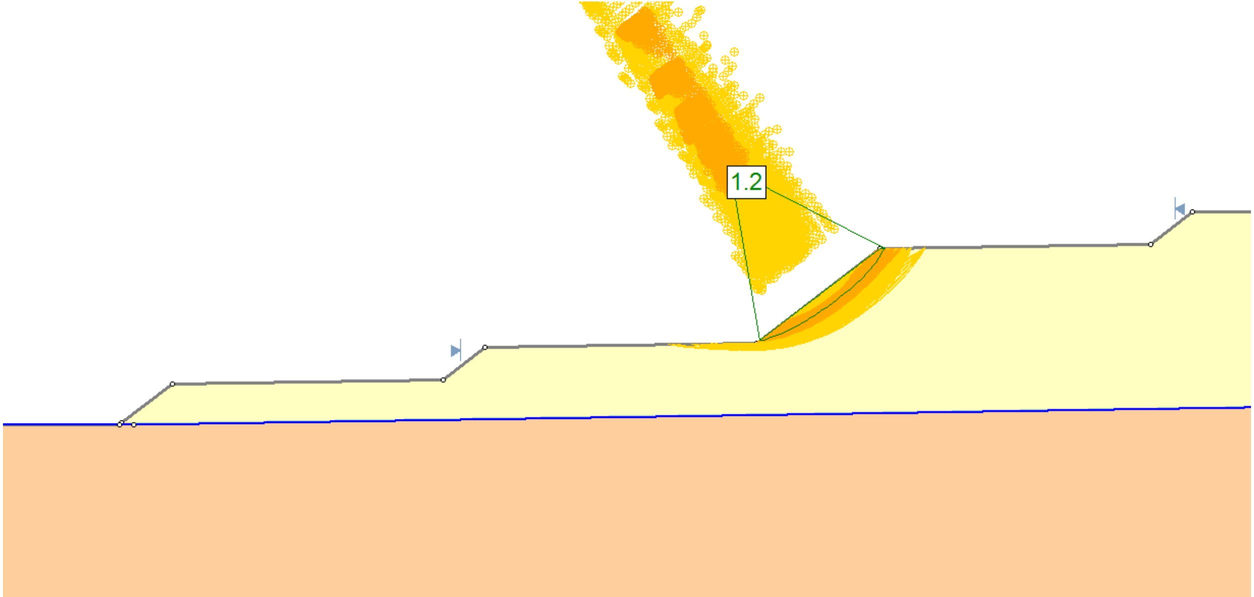

Figure 3: Circular Failure at Mid-Slope Bench

3.1 Toe Failure Mode

The results indicated the following with regards to the failure mode at the toe:

- Factor of safety of 1.2, which is at the low end of the recommended minimum range of 1.2 to 1.3 for monitored, operational slopes
- Potential failure surface will impact the first bench (15 m high bench)
- Potential failure surface intersects the liner system
  - Damage to the liner system may occur if such a failure occurs
  - The strength of the liner system is an important consideration and must be measured directly
- Potential failure may trigger instability in other portions of the slope

3.2 Mid-Slope Failure Mode

The results indicated the following with regards to the mid-slope failure mode:

- Factor of safety of 1.2, which is at the low end of the recommended minimum range of 1.2 to 1.3 for monitored, operational slopes
- Potential failure surface will impact the mid-slope bench (35 m high bench)
- Potential failure surface does not intersect the liner system
- Potential failure may trigger instability in other portions of the slope

3.3 Possible Mitigation Measures

Possible practical mitigation measures include:

- Maintain a buffer zone at the toe of the slope to reduce the consequences of a failure
- Maintain a buffer zone at the crest of the slope to reduce the consequences of a failure
- Active monitoring of the slope and operations

Through discussion with Zitholele, Golder understands that flattening bench face slopes is not a practical mitigation measure during dump operations. This option should be considered for dump closure or if unstable slope conditions during operations warrant such a measure.

Other options such as construction of a toe berm may improve the stability, but the likely improvement is considered to be small. Likewise, reducing the bench height or increasing the spacing between benches is likely to have a small effect.

4.0 RECOMMENDATIONS AND CLOSING

Golder recommends that additional laboratory and in-situ testing be performed to determine the properties required for slope stability analysis of an unsaturated slope. Such testing would include, amongst others, tests to determine the shear strength, unsaturated behaviour, and infiltration rates.

Golder also recommends laboratory testing of the proposed liner system to determine a composite shear strength failure envelope.

In accordance with the scope of work, Golder has performed a high level slope stability analysis based on the information provided and values found in relevant literature. Likely modes of failure, the scale and location of the likely critical failure surfaces, and factors of safety have been determined based on the information available and assumptions made.

Two distinct modes of failure have been identified and factors of safety for both modes is 1.2. The geometry upon which the ash dump facility has been modelled for at Site H is feasible with the low factor of safety values quoted above. The failure mode at the toe of the dump would impact the liner system. The failure mode at the mid-slope bench would mobilize a large volume of material but would not impact the liner system.

Please do not hesitate to contact the undersigned if you have any questions.

GOLDER ASSOCIATES AFRICA (PTY) LTD.

Andrew Fuggle
Geotechnical Engineer

Simon Owens-Collins
Senior Engineering Geologist

AF/SOC/af

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APPENDIX G

Risk Matrix Analysis of Sites B, C, F and H

TECHNICAL MEMORANDUM

DATE 7 November 2014 **PROJECT No.** 13615779

TO Nevin Rajasakran

CC Francois Marais

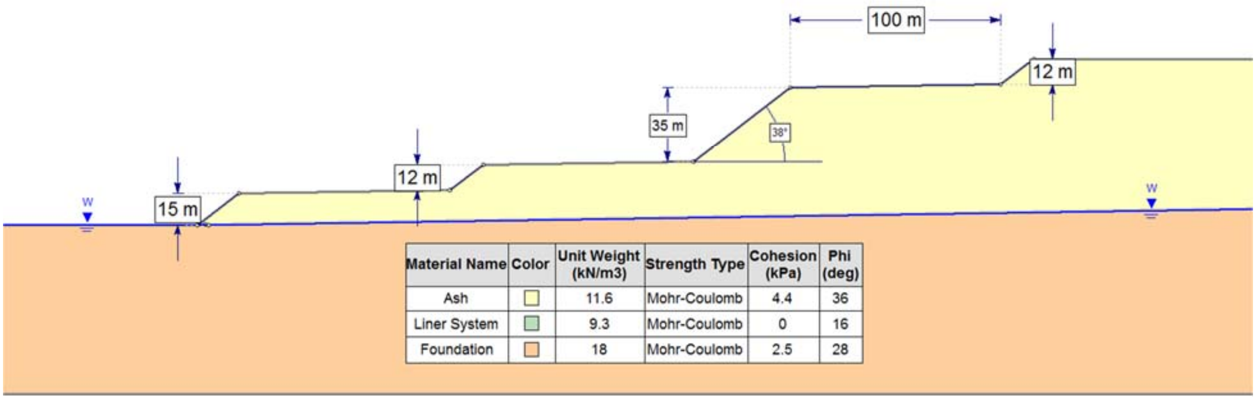
FROM Simon Owens-Collins **EMAIL** sowens-collins@golder.co.za

GEOTECHNICAL SITE RANKING OF SITES B, C, F & H FOR PROPOSED 30 YEAR KENDAL ASH STORAGE FACILITY

Dear Nevin

Tabulated below is the geotechnical screening assessment, rating and ranking of the suitability of Sites B, C, F and H for the proposed Kendal 30 Year Ash Disposal Facility. This technical memorandum should be read in conjunction with our geotechnical report number 13615779-12331-2 dated November 2014; title “Preliminary Geotechnical Investigation of Preferred Sites B, C, F and H for Proposed 30 year Kendal Ash Storage Facility”.

The assessment is based on the typical dump configuration and operations as shown below



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Ash		11.6	Mohr-Coulomb	4.4	36
Liner System		9.3	Mohr-Coulomb	0	16
Foundation		18	Mohr-Coulomb	2.5	28

Figure 1: Modelled Site H Ash Dump Configuration

The rating is based on the system shown in Table 1, as measured on a Scale of 1 to 5:

Table 1: Proposed Table Rating System

Scale	Description
1	Poor / severe / negative (unfavourable)
2	Poor to moderately severe (unfavourable to fair)
3	*Not used*
4	Moderate to good / positive (fair to favourable)

Scale	Description
5	Good / very positive (favourable)
F	Fatal flaw

Based on the rating system given in Table1, each of the sites are rated in Table 2 below according to key geotechnical factors which will influence the selection of the sites and the performance of the ash facilities. Table 2 should be read in conjunction with Figures 2 to 5 in the aforementioned Golder report.

Table 2: Kendal 30 Year Ash Disposal Options – Site Rating

Item	Geotechnical Factors which could Influence Site Selection	High Level Rating of Geotechnical Factors Impacting on Suitability of Site for Ash Disposal			
		Site B	Site C	Site F	Site H
1	**Site Suitability for Ash Disposal based on Invasive Geotechnical Investigation** (and desktop evaluation):				
	- General slope	2	4	4	4
	- Marshes/ Wetlands/ Alluvial floodplain / or flooded open-cast mine workings	2	3	2	3
	- Cultivation	2	1	4	4
2	**Knowledge (level of certainty) of Geotechnical Ground Conditions based on:**				
	- Intrusive work by others	n/a	n/a	n/a	n/a
	- Test pitting by Golder	4	4	1	5
	- Judgement based on desk/ reconnaissance study	4	5	2	4
3	**Undermined**	1	1	1	4
4	**Geology complexity** (Dykes, Sills, Dolomites, Faults, Fill)	2	4	1/F?*	4
5	**Geo-Hydrological Features; includes perceived impact of prior opencast mining**				
	- Present (2014) condition of site	3	4	1	5
	- Future (mined-out) condition of site	1	1	1	4
6	**Level of Engineering Required to Construct Ash Dump if site is a**:				
	- Greenfield site with no possibility of being exploited / mined	4	5	1	5
	- Brownfield site (Rehabilitated open cast site)	2	2	2	2
	- Brownfield site (Current open-cast operations or other mining activities)	1	1	1	1
	- Currently a Greenfield site but with future plans for open-cast mining	1	1	1	2
7	**Perceived impact of DWA-imposed regulations to deal with**				

Item	Geotechnical Factors which could Influence Site Selection	High Level Rating of Geotechnical Factors Impacting on Suitability of Site for Ash Disposal			
	groundwater contamination occurrence during prior open-cast mining:				
	- Present (2014) condition of site	4	5	1	4
	- Future (mined-out) condition of site	1	1	1	1
8	**Perceived impact on liner design**				
	8.1 Present (2014) condition of site	1	3	1	4
	8.2 Future (mined-out) condition of site	1	1	1	1
9	**Access to construction materials for earthworks / starter walls / return water dams etc:**				
	9.1 Present (2014) condition of site	3	3	1	3
	9.2 Future (mined-out) condition of site	1	1	1	1
10	***Proposed 60m/75m height of dump as per sketch above, is*** **readily achievable; if**				
	10.1 Footprint area is constrained; i.e. spatial constraints on achieving stepped bench slopes required for stability	1	1	1	1
	10.2 Entire site available for dump, i.e. without impact from mining; is possible	3	4	2	5
	10.3 Only portions of site exploited in prior opencast mining, i.e. a differential backfill settlement problem is envisaged	2	4	1	5
11	**Extent of rehabilitation required to site**; i.e. perceived negative impact of prior open-cast mine workings not being currently rehabilitated by operator	3	5	1	5

* Pre-Karoo dolomites of the Malmani subgroup may be present below the lowermost coal seams at Site F

Rating

No total scoring of the individual sites has been carried out at this stage as the final scoring will depend on a number of combinations of the geotechnical factors, for example such as the actual positioning of the ash dump on each of the sites, in particular the extent of the dump, if any, placed on re-habilitated mining land.

The total scoring for the site will be done once all the sub-items in Table 2 have been considered and preliminary optimisation of the positioning of the dumps on the sites have been done.

Ranking of sites

Based on our judgement as informed by our investigations undertaken at this stage, Site H is the preferred site followed by Site C. Site F is not favoured or recommended because the mitigation costs to prepare the site to support the dump could prove uneconomical.

Conclusions and Recommendations

Based on our interpretation from the current investigations, no apparent evidence exists, to indicate that, from a geotechnical perspective, Sites B, C and H are fatally flawed. Pre-Karoo dolomites of the Malmani subgroup may be present below the lowermost coal seams at Site F which needs to be verified as this may be prove to be a fatal flaw in respect of development of this site.

Significant engineering challenges have been identified for each of the sites, particularly in respect of Site F, that will give rise to important design and construction considerations which will have to be adequately addressed in detailed design and project specifications.

From the current investigation, there is also a possibility that future blasting at the Heuvelfontein Colliery may negatively impact the development of the ash facility on Site H. Further investigation on the anticipated blasting is required to determine if the effects can be mitigated against to promote the development of the proposed ash facility on the preferred site, Site H.

Owing to access constraints imposed by current landowners on the sites investigated, only a limited portion of Site B and F could be accessed to undertake the planned test pitting. All discussions, conclusions and recommendations included in this report must therefore be considered as relevant only to those portions of the sites where investigations were undertaken, not deemed representative of the remainder of the respective sites unless further collaborating field investigations are implemented to prove such validity.

Deep seasonally-saturated alluvial/clay soils are envisaged which will affect portions of Site B, C and H. It is recommended that the proposed ash facility should not extend on these areas or, if it is needed in terms of dump capacity, it is likely that significant precautionary drainage and earthworks mitigation measures may need to be implemented to prepare these areas for construction.

The Ogies dyke extends onto portions of Sites B, C, F and H, which will increase the permeability of the adjacent soils and increase ingress of water. Where possible, the footprint of the dump should be moved off the probable trace of the dyke.

Detailed investigations are recommended during the detailed design phase to support construction material evaluations, stability analyses, engineering and liner design of the proposed development which will need to be undertaken prior to- and feed into the detailed engineering design phase.

In view of the planned and currently ongoing mining operations for the sites and extent to which the sites are likely to be transformed between now and the onset of the detailed design of the ash dump, it is recommended that the extent to which the preferred site (surface area) will be potentially transformed, be ascertained and detail studies planned accordingly

Ramdayal.

Nithesh Ramdayal, PrSciNat
Engineering Geologist

Simon Owens-Collins, CPEng (Aust)
Senior Engineering Geologist, Associate

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APPENDIX H

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