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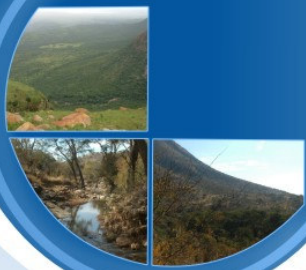

PROJECT LIMA SUPPLEMENTARY FEASIBILITY STUDY

Phase 2: Feasibility Study

DRAFT

Geotechnical Investigation

JANUARY 2007

BKS Palace Consortium

BKS

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PROJECT LIMA SUPPLEMENTARY FEASIBILITY STUDY

Phase 2: Feasibility Study

Report on Geotechnical Investigations

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EXECUTIVE SUMMARY

A. TOPOGRAPHY AND REGIONAL LOCATION

The high-lying Nebo Plateau to the west of the Steelpoort Valley comprises gently undulating terrain at elevations of around 1700m at the Upper Reservoir site. To the east the plateau ends at a steep escarpment trending northeast-southwest that is incised by steep-sided valleys flowing to the east and south-east, away from the escarpment into the Steelpoort Valley. The scarp face between the upper and lower reservoir is near-vertical and falls hundreds of metres to a steep debris slope that flattens eastwards to a pediment slope descending slowly towards the river in the valley floor. The total drop in elevation from escarpment crest to the river is approximately 700m.

B. METHODOLOGY OF FIELD INVESTIGATION

After the desktop study was completed and the preferred scheme chosen, the field investigation for the feasibility study was tailored to provide relevant design information and data on the major components of the scheme. The field investigation comprised the following elements:

- Drilling and logging of rotary core boreholes
- Borehole water acceptance (Lugeon) testing
- Hydrofracture and hydraulic jacking tests
- Wireline borehole surveys
- Groundwater level measurements
- Core orientation surveys
- Test pitting and trenching with both a TLB and large tracked excavator
- Sampling of rock, disturbed and undisturbed soil samples for subsequent laboratory testing
- Seismic and electrical resistivity traverses
- A high resolution aeromagnetic survey
- Laboratory testing on soil and rock samples

As the feasibility study progressed, the field investigation was continually modified and updated to ensure that relevant design information and data was obtained.

C. GEOLOGY – GENERAL

The rocks in the area fall within the Bushveld Igneous Complex and comprise felsic (granitic) rocks overlying the mafic (gabbroic) rocks. The high plateau is underlain by granitic rocks that are many hundreds of metres thick and which form the steep scarp slopes. Below the bottom of the scarp at the base of the felsic rocks is a leptite formation reported to be approximately 250m thick. This is in turn underlain by diorite beneath the pediment slope, grading into olivine-bearing diorite and gabbro beneath the valley floor. The diorites contain bands of anorthosite, magnetite and anorthosite/magnetite-rich diorite. From outcrop mapping the horizons reportedly dip at shallow angles towards the west, although this has not been confirmed by the limited drilling done to date. The rocks on the site are generally highly weathered at the surface, sometimes down to depths of tens of metres. The unweathered rocks are strong to extremely strong. In addition, the various rock types grade almost imperceptibly into one another with the boundaries not readily apparent.

All of the rocks discussed above have been intruded by dolerite/lamprophyre dykes, generally trending northeast (roughly parallel to the Steelpoort fault) and west of northwest (roughly perpendicular to the Steelpoort Fault).

There are numerous fracture/shear zones and faults in the area, many associated with the major Steelpoort fault that trends northeast-southwest and which controlled the formation of the river valley and adjacent escarpment. The Steelpoort fault lies to the north and west of the upper reservoir

Note on Rock Type Nomenclature

From an engineering perspective the felsic rocks **leptite, granophyre, melanogranophyre, granite and mixed granophyre/granite** are very similar. To simplify descriptions of the anticipated rock conditions during the feasibility study, these will sometimes be referred to as **granitic rock or granite**. From the same perspective, the mafic rocks **gabbro, magnetite gabbro, norite, anorthosite and troctolite** will generally be referred to as **gabbroic rock or gabbro**. **Diorite** is an intermediate igneous rock and sometimes contains bands of anorthosite and magnetite, both commonly associated with more mafic rocks such as gabbro.

D. GEOTECHNICAL INVESTIGATIONS

- **Upper Reservoir**

A rockfill embankment with upstream concrete membrane has been proposed for this site

Foundation conditions

Indications at present are that the south-eastern corner and the central part of the reservoir area are underlain by predominantly very hard granitic bedrock, either on surface or below a thin cover of boulders. The remaining areas are generally covered by topsoil overlying weathered granitic rock. Test pits dug in areas of outcrop all refused at depths between 2 and 4m, while in areas with topsoil, the pits could be excavated to the maximum reach of 5,5m of the excavator. The depth of refusal in the test pits is considered suitable founding material for the rockfill embankment. Excavations for the plinth will mainly be in highly to moderately weathered granite, founding for the plinth being in moderately to slightly weathered rock. This material will generally be easy to excavate and should be stable at slopes of 0,5H:1V above the ground water table. Groundwater depths vary from 10.2m to 18.2m with and average at 12.5m. Blasting will be required in some areas.

The seismic and electrical resistivity traverses, totalling a length of about 2000m were completed along sections of the dam wall centre line. However, due to re-sizing and re-alignment of the dam wall, these are uncontrolled as the initial boreholes are not aligned with the traverses. Initial results indicate that the bedrock would be groutable where the seismic velocity exceeds about 1800m/s. Excavation depths for the plinth would then vary between 4 and 10m with local sections of over 20m depth along the east side, and between 6 and 20m along the west and north sides. It is possible that the vertical depths indicated by the seismic profiling is distorted, and it is therefore essential that a few additional cored holes be drilled at selected positions in order to verify the results of the geophysical surveys (this will be confirmed during the extended feasibility investigations).

Construction materials

It is envisaged that all materials excavated for the plinth foundations will be placed in the downstream zone of the embankment as the excavation proceeds. This material will comprise of clayey sand with no rock fragments, soft rock gravel with occasional rocks, and rock fragments (core stones) with 10 to 20% of soil.

The rockfill will be obtained from rock quarried within the dam basin. The most promising area for quarrying is near the headrace intake and in the central area of the reservoir.

- **Lower Reservoir**

During the early phase of the investigations it was established that the foundation conditions along the present centre line are not suitable for a rockfill dam as originally proposed, and it was decided that an earth/rock embankment with clay core must be considered.

Foundation Conditions

Left flank

The left flank is underlain by a 0,5m to over 15m thick layer of colluviums which is underlain by a 8m to 20m thick layer of moderately to highly weathered diorite bedrock. Due to the variable permeability of the colluviums and weathered bedrock, a positive cut-off must be provided to a depth below which the rock can be sealed by means of conventional cement grouting. Based on the borehole results, the depth of the cut-off trench will average about 15m along the left flank with an expected maximum of 20m. The colluviums will be easy to excavate and should be stable at slopes of 0,5H:1V above the ground water table, but may become unstable below the water table. During October 2006, the ground water table was located close to the base of the colluviums. However, excavation of a cut-off trench to depths of up to 20m through colluviums, and 5 to 10m deep into weathered diorite that may contain corestones will not be easy. It may therefore be prudent to consider the construction of a jet-grouted cut-off wall along the deepest sections of the dam foundation.

The design of an embankment founded on the thick colluviums will have to take into account the consolidation characteristics and low shear strength of the founding material. The properties of the colluviums do not improve with depth, and stripping to an average depth of 1,5m is recommended.

Right flank

The right flank is underlain by a 4,5m to 19m thick layer of highly weathered diorite which is underlain by unweathered rock. At a depth of about 1m, the highly weathered diorite is a suitable founding medium for the shells of an embankment dam, however, the underlying material down to a depth of between 7,5m to 19m is highly permeable but may prove difficult to grout. This weathered zone will have to be sealed by means of a cut-off trench.

Where the trench is more than about 5m deep, it will have to be excavated with side slopes of about 0,5H:1V, and thereafter backfilled with clay. This material should be stable above the ground water table. Groundwater depth varies from 4.2m to 24.8m with and average at 12.5m. Curtain grouting can be performed either from the bottom of the trench before backfilling, or through the fill after backfilling.

The proposed spillway structure can be founded on slightly weathered diorite at a depth of about 11m, while a concrete-lined outlet chute along the ridge can be founded on moderately to highly weathered diorite at an estimated average depth of about 2m.

River section

The river section is underlain by a layer of alluvium above diorite bedrock with some outcrops. The thickness of the alluvium is variable and the depth of the cut-off trench is expected to be about 12m.

<u>Construction materials</u>

Material for embankment shells

Test pits in the dam basin show an extensive cover of colluvim. However, these test pits were dug by means of a TLB which could not penetrate the full depth of the colluvial cover, and the results are inconclusive. It is recommended that additional pits be dug by means of a large excavator.

The laboratory tests on colluvial materials on the left flank show the soils to classify as SC (clayey sands, sand-clay mixtures of low potential expansiveness with an average plasticity index of 12), CL (inorganic clays, silty and sandy clay of low to medium potential expansiveness and plasticity (average PI = 16.8) and MH (inorganic silt of medium to very high potential expansiveness and high plasticity (average PI = 44.8) according to the Unified Soil Classification System. Clay contents vary between 3 and 42 with an average of 19.75. These soils are suitable as general embankment fill, but are usually not sufficiently impermeable for use as core material, although there may be some suitable zones. The colluvium also contains varying percentages of gravel and boulders that may have to be removed during embankment construction. It is anticipated that about 500 000m^3 of colluviums could be obtained below FSL in the dam basin.

Material for clay core

Test pitting during the earlier feasibility stage investigations, showed at least 2m of colluvial soil in the area under irrigation downstream of the left flank. This material was considered suitable as core material, and should be further investigated. Other areas outside the FSL of the present dam were identified as potential borrow area but have not been investigated to date.

- **Intake / Surge Shaft**

No shaft drilling was carried out during this investigation phase, but it is recommended that the next drilling phase includes a 700m deep borehole be drilled at the proposed shaft position.

Borehole BH6 (Feasibility Report of November 2000) was drilled 200m south of the present proposed shaft position to a vertical depth of 201m and indicates that the rock condition should be good.

- **Pressure Tunnel**

Borehole PT 01 was drilled at an angle of 60° below horizontal to a depth of 400m measured on the incline, intersecting the pressure tunnel at a vertical depth of approximately 330m below the ground surface. The borehole is primarily in very strong rock diorite, with occasional bands of very strong to extremely strong rock magnetite or mixed (“zebra-striped”) magnetite-rich and anorthosite-rich diorite. A distinctive “zebra-striped” band approximately 20m thick was intersected where the borehole crosses the pressure tunnel; this is expected to have a very shallow dip and this very strong to extremely strong rock may therefore persist within the tunnel excavation profile over a fairly large distance. The diorite and magnetite-rich/anorthosite-rich sections are very competent rock with medium to wide spaced joints which are closed to slightly open. A number of very closely fractured zones do occur in the borehole over limited lengths, however, a fairly large very closely fractured zone was encountered in borehole PT 01 between approximately 235m and 250m. This is thought to be a near-vertical feature, probably intersecting the tunnel over a length of approximately 10m within the steel-lined section a short distance upstream of the bifurcations. This feature may well be associated with water inflows.

Five hydraulic jacking tests carried out in borehole PT 01 in the area of the pressure tunnel indicate an average minimum horizontal stress of 6.9 MPa (range 5.5 to 8.1) based on jacking, and the average maximum horizontal stress is 22.1 MPa (range 14.3 to 28.5) calculated from first breakdown pressures.

Surface topography suggests that a prominent depression trending roughly NNE that crosses a spur below the escarpment may represent a sheared or fractured zone that could extend down to the tunnel at about midway along its length. Such a feature may well be associated with water inflows.

- **Machine and Transformer Halls**

Two boreholes have been drilled in the vicinity of the machine hall, i.e. Borehole MH 01 immediately northwest of the machine hall to a depth of 350m and borehole SC 01 immediately southeast of the machine hall to a depth of 300m.

The distinctive “zebra-striped” band of mixed anorthosite-rich and magnetite-rich diorite approximately 20m thick occurs at similar elevations in these two boreholes, as well as in adjacent boreholes. The band is very strong to extremely strong rock and appears to occur in the upper levels of the machine and transformer halls. The band does not appear to form a discontinuity with the adjacent very strong rock diorite, i.e. no obvious plane of weakness appears to occur between the rock types. Measurements from wireline logging of boreholes indicate that there are three main joint sets in addition to the two near-vertical complementary joint sets present on outcrops throughout the site. These joint sets may lead to the formation of flat shallow wedges in the crown or slabs in the sidewalls of the caverns, which would have to be supported by means of rock-bolts.

Five hydraulic jacking tests carried out in borehole SC 01 in the area of the transformer hall indicate an average minimum horizontal stress of 7.0 MPa (range 5.6 to 8.4) based on jacking, and the average maximum horizontal stress is 24.2 MPa (range 16.8 to 32.0) calculated from first breakdown pressures.

- **Surge Chamber**

The borehole SC 01 was drilled in the area of the originally proposed surge chamber, which has subsequently been eliminated. The borehole is, however, well placed for the new transformer hall position and has been discussed under “Machine and Transformer Halls” above.

- **Tailrace, Access and Emergency Tunnels**

Boreholes TR 01 and TR02 were drilled on the previously proposed alignment of the tailrace tunnel and outfall portal respectively and were to provide an indication of likely tunnelling conditions as well as an indication of the founding for the outfall structure and likely portal and trench excavation slope requirements. Borehole TR 01 shows over 10m of colluvial deposits overlying highly weathered diorite to a vertical depth of 30m, with approximately 50m of hard rock cover to the tunnel, while borehole PT 02 shows shallow colluviums underlain by very closely jointed weathered to slightly weathered diorite rock down to approximately 25m depth, with improved rock conditions for about 20m above the tunnel at the portal. The portal and trench excavations will require shallow angle cut slopes or fairly robust support measures.

E. CONCLUSION

The feasibility stage geotechnical investigations have revealed the following:

- Poorer foundation conditions at both the upper and lower reservoir than originally anticipated
- In general, construction materials for the dam walls are all available within the dam basin. Clay material for the lower dam core is available in close proximity.
- Generally very good rock conditions for the underground works
- The various rock types grade almost imperceptibly into one another with the boundaries not readily apparent.
- Major faults/shear zones should not be discounted in the underground works in the area of the pressure tunnel, although some minor zones of highly fractured rock, which may well be associated with water inflows, should be expected.

- A distinctive “zebra-striped” band of very strong to extremely strong rock (mixed anorthosite-rich and magnetite-rich diorite) appears to occur in the upper levels of the machine and transformer halls. This band is expected to occur over a fairly extensive length in the pressure tunnel. The band does not appear to form a discontinuity with the adjacent very strong rock diorite, i.e. no obvious plane of weakness appears to occur between the rock types.
- Portal and trench excavations will require shallow angle cut slopes or fairly robust support measures.

Further investigations will be required in order to supply sufficient information and data for tender and detailed design to proceed. This would include, inter alia:

- determination of the principle in-situ stresses in the rock mass
- determination of the in-situ elastic rock modulus and Poisson’s ratio of the rock mass
- calibration of the seismic traverses at the upper reservoir by means of additional boreholes
- additional test pitting with a large tracked excavator to prove the availability of construction materials at the lower site
- a deep borehole at the position of the intake shaft to prove rock conditions along the length of the shaft

1. INTRODUCTION

The geotechnical investigations for the phase 2 feasibility study was conducted using available documentation, report and maps combined with the field work which included drilling of borehole and rock sampling, test pitting and sampling, geophysical surveys (seismic and resistivity surveys, wireline borehole surveys, hydrofracture, borehole water acceptance testing and core orientation surveys) as well as laboratory testing.

This draft report details the information available at the end of January 2007 and the evaluation and interpretation that could be carried out at that stage.

2. METHODOLOGY OF FIELD INVESTIGATION

After the desktop study was completed and the preferred scheme chosen, the field investigation for the feasibility study was tailored to provide relevant design information and data on the major components of the scheme. The field investigation comprised the following elements:

- Drilling and logging of rotary core boreholes
- Borehole water acceptance (Lugeon) testing
- Hydrofracture and hydraulic jacking tests
- Wireline borehole surveys
- Groundwater level measurements
- Core orientation surveys
- Test pitting and trenching with both a TLB and large tracked excavator
- Sampling of rock, disturbed and undisturbed soil specimens for subsequent laboratory testing
- Seismic and electrical resistivity traverses
- A high resolution aeromagnetic survey
- Laboratory testing on soil and rock samples

As the feasibility study progressed, the field investigation was continually modified and updated to ensure that relevant design information and data was obtained.

3. GENERAL DESCRIPTION OF THE SITE

3.1 Topography and Geology

The high-lying Nebo Plateau to the west of the Steelpoort Valley comprises gently undulating terrain at elevations of around 1700m at the Upper Reservoir site. To the east the plateau ends at a steep escarpment trending northeast-southwest and is incised by stream valleys flowing to the north and west, away from the escarpment, and by steep-sided fault-formed valleys flowing to the east and north over the escarpment into the Steelpoort Valley.

The scarp face is near-vertical and falls hundreds of metres to a steep debris slope that flattens eastwards to a pediment slope descending slowly towards the river in the valley floor, before rising rapidly eastwards again beyond the river. The total drop in elevation from escarpment crest to the river is approximately 700m.

There are numerous major fracture/shear zones and faults in the area, many associated with the major Steelpoort fault that trends northeast-southwest and which controlled the formation of the river valley and adjacent escarpment. The fault leaves the valley and crosses the escarpment north of the site and so lies to the north and west of the upper reservoir.

The rocks in the area fall within the Bushveld Igneous Complex and comprise felsic rocks of the Rashoop Granophyre Suite overlying the mafic rocks of the Upper and Main Zones of the Rustenburg Layered Suite. The high plateau is underlain by granophyre and melanogranophyre. These felsic rocks are several hundred metres thick and form the steep scarp slopes. Below the bottom of the scarp at the base of the felsic rocks is a leptite formation approximately 250m thick, dipping approximately 10 degrees westwards into the slope. This is in turn underlain by diorite beneath the pediment slope, grading into olivine-bearing diorite and gabbro beneath the valley floor. These mafic rocks underlying the leptite formation contain bands of anorthosite and magnetite, and all of the horizons dip at a shallow angle towards the west.

All of the rocks discussed above have been intruded by dolerite/lamprophyre dykes, generally trending northeast (roughly parallel to the Steelpoort fault) and west of northwest (roughly perpendicular to the Steelpoort Fault).

The proposed Scheme is entirely west of the Steelpoort River, having the upper reservoir on the plateau, the waterways descending to the power station caverns beneath the lower slopes and rising again towards the lower reservoir near the valley floor.

3.2 Major Rock Types and Condition

The rock strength terms used by the field geologists for the Feasibility Report (November 2000) have been converted to the ISO 2001 terms and **Table 1** below included to assist the reader.

Table 1. Rock strength descriptive terms

Term	Field Identification	UCS (MPa)
Extremely weak[1]	Indented by thumbnail	< 1
Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1 to 5
Weak	Can be peeled by pocket knife with difficulty, shallow indentations by firm blow with point of geological hammer	5 to 25
Medium strong	Cannot be scraped by or peeled with a pocket knife, specimen can be fractured with a single blow of geological hammer to fracture it	25 to 50
Strong	Specimen requires more than one blow of geological hammer to fracture it	50 to 100
Very strong	Specimen requires many blows of geological hammer to fracture it	100 to 250
Extremely strong	Specimen can only be chipped with geological hammer	> 250
[1] Some extremely weak rocks will behave as soils and should be described as soils according to ISO 14688-1		

Granophyre and Melanogranophyre.

The granophyre forms the top of the escarpment and upper scarp face and reportedly grades downward into melanogranophyre, which forms the lower scarp face. The unweathered granophyre is pinkish and pale grey, strong to very strong rock.

The melanogranophyre is reportedly similar in appearance but contains appreciable amounts of black hornblende.

The granophyre often outcrops on the plateau surface but is also completely to highly weathered in fracture zones with core stones to depths of over 20m.
Drill core data indicates that the sub-horizontal joints have a frequency of 2 – 3 per metre in borehole UR1, UR2 and UR5, the joint frequencies are in order of 6 – 8 per metre in borehole UR3 and UR4. Six major joint sets have been identified dipping at 70 – 90 degrees, 45 – 60 degrees and 0 – 10 degrees. They are usually less than 2mm apart, but also closer-spaced in places. Generally joints were partly open and filled with slightly clayey silty sand.

Leptite.

The Leptite layer does not outcrop anywhere in the area, but it occurs in the lower part of the escarpment and is covered by boulder talus.
It is reported to be reddish or grey coloured, fine grained quartz and feldspar rich rock. Horizons of micrographic felsite and microgranite occur, as well as irregular veins of fine grained granite and granodiorite, and patches of granophyre.
They are expected to be generally unweathered, strong to very strong rock with joint patterns and characteristics similar to the overlying granophyre and granite.

Diorite and Olivine Diorite.

Only the uppermost layer of these rocks occurring immediately below the leptite is diorite, and the rest of the very thick sequence is olivine diorite. The diorites and olivine diorites are indistinguishable without a microscope and they have the same engineering properties, so are referred to here simply as diorites.

The diorite is dark grey speckled pale grey or white, medium grained, strong to very strong rock. On the steeper slopes, immediately below the escarpment, the diorite occurs beneath a cover of boulder talus reported to be up to 20m thick. On the flatter pediment slopes lower down the diorite is generally covered by thick colluvial gravel or fine grained hillwash, ranging from 5 - 15m thick. Beneath the thick overburden the diorite is completely weathered or highly weathered to depths ranging from 10 - 25m. The highly weathered diorite is a yellowish to dark brown, medium to closely fractured, very weak to weak rock that excavates to a coarse, medium and fine gravely material.

Immediately below the escarpment, fracture frequency ranges from 4 – 10 per metre and the spacing ranges from 5mm to 10700mm with an average ranging from 73mm to 2390mm. The joints are generally very tight to partly open, undulating and between 25% and 45% of the joints were cemented with calcite or chlorite. On the flatter pediment slopes fracture frequency ranges from 7 – 20 per metre and the spacing ranges from 5mm to 3800mm with an average ranging from 101mm to 739mm. The joints are generally very tight to partly open, rough to smooth planer and 90% of the joints were filled with clay/silty sand. RQDs (Rock Quality Designation) recorded for the boreholes' cores are relatively high.

Anorthosite

Anorthosites are often associated with gabbros and diorites comprising part of a layered sequence. Layers of varying thickness occur within the diorite, sometimes thinly interbanded with magnetite or diorite. Such a mixed layer 14m to 24m thick has been intersected in boreholes drilled in the underground waterways area.
The unweathered anorthosite is very pale grey to grey-green, strong to extremely strong rock. The joint pattern and characteristics are similar to the diorite in which it occurs.

Magnetite

Layers are generally around 1m thick and occur within the diorite, sometimes thinly interbanded with anorthosite or diorite. Such a mixed layer 18m thick was intersected in boreholes drilled in the machine hall area (see *Anorthosites* above).
A number of circular magnetite plugs up to 80m in diameter reportedly occur within the valley and are generally seen as surface boulders overlying the bedrock.
The magnetite is a black, unweathered, extremely strong rock with a metallic lustre.

Gabbro and Norite.

These generally strong to very strong rocks occur mostly to the east of the river and are only likely to be encountered in the site area.
It should be noted that the gabbros are very similar in appearance to the diorites in the area, as they differ only in the type of feldspar mineral they contain. Because the diorites grade almost imperceptibly into gabbro, which in turn grades into norite, the boundaries are not readily apparent in the field. Engineering properties of the unweathered rocks are fairly similar.

Dykes

Both dolerite and lamprophyre dykes occur in the area and, as the rock types and engineering characteristics are similar, the intrusions will be referred to as dolerite dykes. The dolerite is generally grey to dark grey, strong to very strong rock when unweathered, and the highly weathered dolerite is yellow-brown and dark brown very weak to weak rock. Dykes in the area can be expected to be weathered to residual soils as deep as 15m and highly weathered below this to around 20m.

Note on Rock Type Nomenclature

From an engineering perspective the felsic rocks **leptite, granophyre, melanogranophyre, granite and mixed granophyre/granite** are very similar. To simplify descriptions of the anticipated rock conditions during the feasibility study, these will sometimes be referred to as **granitic rock or granite**.
From the same perspective, the mafic rocks **gabbro, magnetite gabbro, norite, anorthosite and troctolite** will generally be referred to as **gabbroic rock or gabbro**.
Diorite is an intermediate igneous rock that grades imperceptibly into both **granodiorite** and into **gabbro**. In this report it is generally referred to as **diorite, olivine diorite** etc. However, the diorites in the area contain bands of anorthosite and magnetite, both commonly associated with more mafic rocks such as gabbro. For this reason it has sometimes been grouped with the gabbroic rocks, particularly in the Report on the Desktop Geotechnical Study (March 2006).

3.3 Seismicity

A more recent seismic risk investigation has been carried out by the Council for Geoscience for the De Hoop Dam. The results of this investigation will be included in the Phase 2 feasibility report.

4. GEOLOGICAL INVESTIGATIONS

4.1 Upper Reservoir

A rockfill embankment with upstream concrete membrane has been proposed for this site. The following geotechnical factors have been considered:

- Depth of stripping to provide suitable founding for the rockfill embankment.
- Depth of excavation to provide suitable founding conditions for the plinth, i.e rock that can be effectively grouted.
- Excavation characteristics of material from the core trench.
- Sources of material for rockfill embankment.

4.1.1 Foundation conditions

Five diamond drill holes have been completed. Vertical boreholes UR1 and UR2 were set out near outcrops, while inclined holes UR3, UR4 and UR5 were sited to intersect a suspected zone of deeper weathering. A shortcoming of the drilling is that no water pressure testing had been done in the upper weathered parts of the boreholes.

Thirteen test pits were dug with an excavator at various positions along the dam centre line.

Five seismic and five electrical resistivity traverses, totalling a length of about 2000m were completed along sections of the dam wall centre line. Unfortunately only one borehole could be used to calibrate the geophysical results. From the results of this borehole, it was concluded that the bedrock would be groutable where the seismic velocity exceeds about 1800m/s.

The geotechnical map prepared by Partridge, Maud & Associates (June 2000) shows the south-eastern corner and the central part of the reservoir area to be underlain by predominantly very hard granophyre bedrock, either on surface or below a thin cover of boulders. The remaining areas are shown to be covered by topsoil overlying weathered granophyre.

Test pits dug in these areas of outcrop, all refused at depths between 2 and 4m, while in the areas with topsoil, the pits could be excavated to the maximum reach of 5,5m of the excavator.

During field mapping three major joint sets were identified at the upper reservoir. Two of these joint sets are near vertical and the third joint set is near horizontal. The shear/fracture/fault zone across the spur has an estimated trend N 165 degree. Stereoplot of core orientation measurement identified six major joint sets, see **Appendix E.**

It is important to note that some small dolerite outcrop occur in the western linear depression.

The results of the core drilling and test pitting are summarised in **Table 2 and Table 3** respectively, and were plotted on the orthophoto layout plan, together with the seismic traverses on **Figure 1.** Soil profiles and core log description are given in **Appendix A and Appendix B** respectively whilst core photos are given in **Appendix C**. The results of the electrical resistivity survey correspond largely with the seismic results and have been used to assist with the assessment of groutability.

It should be noted that the layout of the reservoir has been changed from time to time as a result of new geological information and/or design requirements. The test positions may therefore not always correspond with the alignment of the plinth.

Also, considering the length of the centre line (over 3 300m), the results of 2000m of geophysical traversing, 5 boreholes and 13 shallow test pits that are not located exactly along the plinth alignment, cannot be considered fully representative of the site.

Borehole UR1 (vertical) intersected bedrock at 0,20m, and completely weathered zones to a depth of 6,30m. Unfortunately, water pressure testing was not done between the surface and depth of 12,50m. From the core log and photograph, it appears that the rock can be grouted below 6,40m

Borehole UR2 (vertical) intersected bedrock at 2,0m and completely weathered zones to a depth of 4,50m. Water pressure testing was only done below 8.70m. From the core log and photograph, it appears that the rock can be grouted below 4,80m.

Table 2. Summary of borehole information along the upper reservoir centre line.

BH No. (inclination°)	Vertical depth of stripping below rockfill (m)	Vertical depth to groutable rock for plinth foundation (m)	Vertical depth (m) to water level (21 October 2006)
UR 1 (90°)	0,20	6.40	10,20
UR 2 (90°)	2,00	4,80	11,70
UR 3 (50°)	1,50	12,00	18,20
UR 4 (50°)	11,0	14,00	10,95
UR 5 (70°)	0	4,70	11,27

Boreholes UR3 and UR4 are inclined at 50 degrees to intersect a zone where deeper weathering was suspected. Zones of completely weathered rock were encountered to vertical depths of about 10m in both holes, while highly weathered rock was encountered to 12 and 14m (vertical depths) respectively in UR3 and UR4. Minimum plinth founding depths are between 12 and 14m.

Borehole UR5 was drilled at 70 degrees to intersect lineaments. It encountered bedrock at surface and highly weathered zones to 8,50m vertical depth. The plinth can be founded at 5,0m depth.

The depth of refusal in the test pits is considered suitable founding material for the rockfill embankment. This depth varies between 1 and 3m in areas with rock outcrop, while elsewhere it is generally more than 5m and can locally be as deep as 11 m.

Excavations for the plinth will mainly be in moderately to highly weathered granite, comprising of small and large corestones in a matrix of soft or medium strong material, founding for the plinth being in moderately to slightly weathered rock. In many areas (e.g. near BH's UR3 and UR4) most of the material will be rippable, but in other areas, blasting will be required.

Excavation depths for the plinth (as derived from uncontrolled sections of seismic traversing and the widely spaced boreholes) vary between 4 and 10m with local sections of over 20m depth along the east side, and between 6 and 20m along the west and north sides.

Table 3. Summary of test pit information along upper reservoir centre line

TPU No.	Hillwash (m)	Total depth (m)	Comments
1A	0.15	1.0	Refused on corestones
2B	1.0	5.0	Maximum reach.
2	2.3?	5.3	Maximum reach.
3A	1.1	3.0	Refused on weathered granite
3B	0	2.0	Refused on corestones
4A	1.3	3.0	Refused on weathered granite

TPU No.	Hillwash (m)	Total depth (m)	Comments
4B	0.7	3.0	Refused on corestone
5	0	1,7	Refused on weathered granite
6	1.0	3.9	Refused on corestones
7A	0.5	3.0	Refused on corestones
7B	0.5	4.3	Maximum reach
8	0.2	5.0	Maximum reach
9	0.2	2.8	Refused on weathered granite
10	0.3	5.0	Maximum reach
11	1.1	2.9	Refused on corestones
12	0.3	5.0	Maximum reach
13	0.3	2.4	Refused on corestones

The above depths are much more than could be expected from surface observations and initial drilling. It is possible that the vertical depths indicated by the seismic profiling is distorted, and it is therefore essential that a few additional cored holes be drilled at selected positions in order to verify the results of the geophysical surveys.

The downstream slope of the plinth excavation will correspond with the slope of the concrete face on the embankment (1,5H:1V), while the upstream slope can vary from 0,5H:1V to 1:1, depending on the characteristics of the material. Materials above the groundwater level appear to be stable at these slopes. The groundwater level is situated between 10,2m and 18,2m depth with an average of 12.5m below ground surface.

Laboratory test results on soil sampled at the upper reservoir show that the majority of soil cover above the granitic rock is mainly made of clayey sand and was classified as SC with low potential expansiveness and an average plasticity index of 11. An isolated test pit (TPU8) shows a high potential expansiveness and plasticity index of 33, i.e. (PI = 33), this material was classified as MH (inorganic silt), the classification was made in accordance the Unified Soil Classification System. The Laboratory test results are summarised in **Table 4** and the full set is given in **Appendix D**

Table 4. Summary of laboratory test results on soil samples from the Upper reservoir.

Sample No.	Hole No.	Depth No. (m)	LL	PI	LS %	GM	%< 0.075 (mm)	PRA	US class	Heave
PL22	TPU1B	1,2	32	8	4,0	0,73	41	A- 4 (1)	SC	Low
PL23	TPU1B	3,5	32	6	3,0	0,92	40	A- 4 (1)	SC	Low
PL24	TPU2	1,0	36	14	6,5	0,82	45	A- 6 (3)	SC	L/M
PL25	TPU2	2,0	36	14	6,5	0,82	45	A- 6 (3)	SC	L/M
PL26	TPU2	0,3- 5,3	36	14	6,5	0,82	45	A- 6 (3)	SC	L/M
PL27	TPU4B	0,5- 3,0	35	12	6,0	0,77	46	A- 6 (3)	SC	L/M
PL28	TPU7B	0,5- 4,0	32	11	5,0	0,75	48	A- 6 (3)	SC	L/M
PL30	TPU8	1,4- 5,0	40	14	6,5	0,52	62	A- 6 (7)	ML	L/M
PL31	TPU8	2,8- 5,0	73	33	15,5	0,17	87	A- 7- 5 (20)	MH	H/VH
PL32	TPU9	0,2- 2,7	33	10	5,0	0,82	45	A- 4 (2)	SC	Low

LL = Liquid Limit
PI = Plasticity Index
LS =Linear Shrinkage
GM = Grading Modulus

US Class= Unified Soil Classification
C = Cohesion
VH = Very High
Med = Medium
H = High

4.1.2 Construction materials

It is envisaged that all materials excavated for the plinth foundations will be placed in the downstream zone of the embankment as the excavation proceeds. This material will comprise of clayey sand with no rock fragments, soft rock gravel with occasional rocks, and rock fragments (core stones) with 10 to 20% of soil.

Soil materials (clayey sand) excavated at the upper reservoir could be used as a sealing layer at the base of the reservoir or outer shell of the rock fill embankment.

Samples taken from the upper reservoir test pits were also tested for suitability for access road construction. The results are given in **Appendix D**. Despite the fine nature of the material, the compactability and strength will be adequate for subgrade, selected subgrade and subbase, and probably for basecourse if stabilised.

Rock fragments will be used for the construction of the rockfill embankment and balance of the rockfill will have to be obtained from rock quarried within the dam basin. The most promising area for quarrying is near the headrace intake.

4.2 Lower Reservoir

During the early stage of the investigations it was found that the foundation conditions along the present centre line are not suitable for a rockfill dam as originally proposed, and it was decided that an earth/rock embankment with clay core must be considered.

4.2.1 Foundation Investigations

The following geotechnical factors have been considered:

- Depth of stripping to provide suitable founding for shells of embankment dam.
- Depth of core trench to reach rock that can be grouted effectively.
- Excavation characteristics of material from the core trench.
- Sources of material for embankment shells.
- Sources of clay core material.

The results of seven cored boreholes (LR 1, LR 2, LR 3, LR 4, LR 7, LR 8 and LR 9) and three test pits (TPL 5, TPL 9 and TPL 10) were available for evaluation of founding conditions along the dam centre line. The results of the boreholes are summarised in **Table 5** and the longitudinal section across deeper boreholes is show in **Figure 2.**

A number of test pits were dug below FSL in the dam basin to determine the availability of construction materials for the embankment. Samples were taken for laboratory testing, test pit information are summarised in **Table 6** below.

Table 5. Summary of borehole information along the lower reservoir centre line.

BH No.	Thickness of transported material (m)	Depth of stripping (m)	Depth to groutable rock (m)	Depth (m) to water level (19 Oct. 2006)
LR 1	13,94	1,0	17,0	16.85
LR 2	5,50	1,0	9,30	5.43

BH No.	Thickness of transported material (m)	Depth of stripping (m)	Depth to groutable rock (m)	Depth (m) to water level (19 Oct. 2006)
LR 3	0,50	0,50	8,0	9,06
LR4	0,20	0,50	12,0	16,19
LR5	0,20	0,20	8,0	8.92
LR6*	0,60	2,0	16	21.40
LR 7	6,0	1,50	11,0	24,80
LR8	15,42	2,0	14	12,84
LR 9	4,5	1,0	8,0	4,56

* Hole inclined at 45 degrees.

Table 6. Summary of test pits at lower reservoir

Test Pit (TPL) No.	Machine used	Depth of hole (m)	Thickness of transported (m)
1	PC 300	6,25	6,25
2	PC 300	6,10	6,10
3	PC 300	5,50	5,40
4	PC 300	5,60	5,60
8	PC 300	4,65	0,65*
9	PC 300	5,70	3,60
10	PC 300	5,90	5,90
11	PC 300	5,00	5,00
15	Ford 550	1,60	1,60
16	Ford 550	2,50	2,50
21	Ford 550	2,10	2,10
22	Ford 550	2,70	2,70
23	Ford 550	1,70	1,70
24	Ford 550	2,00	2,00
26	Ford 550	1,90	1,70

Test Pit (TPL) No.	Machine used	Depth of hole (m)	Thickness of transported (m)
5	PC 300	4,50	0,20
6	PC 300	2,00	0,20
7	PC 300	1,50	0,20
12	Ford 550	1,50	0,20
13	Ford 550	1,30	0,50
14	Ford 550	1,50	0,20
17	Ford 550	1,50	1,50*
18	Ford 550	1,50	0,20
19	Ford 550	2,50	2,50*
20	Ford 550	1,70	0,20
25	Ford 550	2,30	1,00

* Results not in accordance with aeromagnetic interpretation.

From **Table 6**, it is evident that the Ford 550 TBL could not dig effectively in either the colluvial material or the residual diorite, and refused typically at about 2m depth.

The laboratory tests on colluvial and hillwash materials at the Lower Reservoir show that soils classified as SC (clayey sands, sand-clay mixtures of low potential expansiveness with an average plasticity index of 12), CL (inorganic clays, silty and sandy clay of low to medium potential expansiveness and plasticity, average PI = 16.8) and MH (inorganic silt of medium to very high potential expansiveness and high plasticity, average PI = 44.8) according to the Unified Soil Classification System. Clay contents vary between 3 and 42% with an average of 19.75%. The soil cover at the left flank is considerably thick (5 to 15m) compared to the soil cover at the right flank which is generally less than 0.5m. The Laboratory test results are summarised in **Table 7** and the full set is given in **Appendix D**

Table 7. Summary of Laboratory Test Results on Soil Samples from the Lower Reservoir.

Hole No.	Depth No. (m)	LL	PI	LS %	GM	%< 0.075 (mm)	PRA Class	US class	Heave	Coefficient of permeability (cm/s)	Friction angle (degrees)	C (kN/n)
TPL1	1,0	-	SP	1,0	1,04	22	A- 2- 4 (0)	SC	L			
TPL1	3,0	31	14	7,0	0,73	53	A- 6 (0)	CL	L			
TPL4	0,5	32	14	6,5	0,86	47	A- 6 (4)	SC	L/M			
TPL4	4,0	31	13	6,5	0,92	47	A- 6 (3)	SC	L			
TPL11	0,5	28	10	5,0	1,06	27	A- 2- 4 (0)	SC	L	1,184 x 10^{-6}		
TPL11	0,55	43	21	9,5	0,83	41	A- 7- 6 (4)	SC	M	6,652 x 10^{-9}	28,7	3,7
TPL11	2,7	127	77	18,0	0,36	75	A- 7- 5 (20)	MH	VH			
TPL11	2,75	115	69	21,5	0,35	70	A- 7- 5 (19)	MH	VH	7,089 x 10^{-9}	27,8	5,1
TPL4	0,55	30	13	6,5	0,62	53	A- 6 (5)	CL	L/M	4,320 x 10^{-4}	27,4	5,1
TPL4	4,1	30	15	7,0	1,05	43	A- 6 (3)	SC	L	2,252 x 10^{-4}	34,4	7,4
TPL25	0,3										33,2	0
TPL25	0,55	59	22	10,0	0,62	55	A- 7- 5 (11)	MH	M			
TPL25	1,6									2,026 x 10^{-8}		
TPL25	1,7	68	34	15,5	0,63	47	A- 7- 5 (10)	SC	H			
TPL26	0,3- 1,4											
TPL26	0,3	47	23	11	0,75	54	A- 7- 6 (10)	CL	M			
TPL26	1,5	67	34	15,0	0,31	76	A- 7- 5 (20)	MH	H/VH			
TPL4	1- 5									1,747 x 10^{-7}		
TPL19	0,5	24	4	2,0	0,81	41	A- 4 (1)	SP & SC	L			
TPL19	2,0	30	14	7,0	0,63	60	A- 6 (7)	CL	L			
TPL23	1,5	42	23	10,5	1,06	50	A- 7- 6 (8)	CL	M			

LL	= Liquid Limit	US Class	= Unified Soil Classification
PI	= Plasticity Index	GM	= Grading Modulus
LS	=Linear Shrinkage	C	= Cohesion
VH	= Very High	M	= Medium
H	= High	L	=Low

An aeromagnetic survey of the project area has been completed, and this showed a wide range of magnetic anomalies. Analysis of the aeromagnetic survey shows a number of prominent linear zones of lower intensity that could be indicative of deeper weathering. see **Figure 3**. The high resolution aeromagnetic survey has revealed some fracture/shear zones or faults within the footprint of the scheme; the full report is included in **Appendix F**. Eight joints set were identified during joints survey and six of these are similar to those found at the upper reservoir, see **Appendix E.**

4.2.2 Left flank

The left flank is underlain by a 0,5 to over 15m thick layer of colluvium, comprising of reddish brown silty sand with variable quantities of angular to sub-rounded gravel and boulders. The colluvium is underlain by diorite bedrock which is moderately to highly weathered to depths ranging between 8 and 20m.

The design of an embankment founded on the thick colluvium, will have to take into account the consolidation characteristics (void ratio >1) and low shear strength (N = 28 degrees and C = 5 kPa) of the founding material. The properties of the colluvium do not improve with depth, and stripping to an average depth of 1,5m is recommended.

Due to the variable permeability of the colluvium ($6x10^{-9}$ - $4x10^{-4}$ cm/s) and weathered bedrock (not tested), a positive cut-off must be provided to a depth below which the rock can be sealed by means of conventional cement grouting.

Based on the borehole results, the depth of the cut-off trench will vary between about 12m near the river, to a maximum of about 20m at Boreholes LR 8 on the left flank and LR6 on the right flank. The average depth for the cut-off along this flank will be about 15m. Below the cut-off, the rock mass has a moderate permeability (5 - 20 Lugeons), and could be sealed by means of cement grouting.

The layer of colluvium in the upper part of the cut-off trench varies in thickness between 0,5 and 15m, and will be easy to excavate. This material should be stable at slopes of 0,5H:1V above the ground water table, but may become unstable below the water table. During October 2006, the ground water table was located close to the base of the colluviums. The groundwater level is situated between 5,5m and 24,8m depth with an average at 15m below ground surface.

The zone of highly weathered diorite below the colluvium can be classified as rippable, but may be difficult to excavate in a narrow trench as shown by the partial refusal experienced by the Komatsu PC 300 excavator in TPL 5, 6 and 7. This material can stand vertically as indicated by TPL 5.

Excavation of a cut-off trench to depths of up to 20m (possible partly below the groundwater table) through colluvium, and 5 - 10m deep into weathered diorite (that may contain large corestones or layers of hard rock), will not be easy. Therefore, it is recommended that the following alternative options be considered:

a) Construction of a jet-grouted cut-off wall along the deepest sections.

b) Moving the dam centre line to a more favourable position.

The first option does not involve any excavation, but it will be expensive, and the effect of a rigid wall within relatively compressible material below the embankment must be considered.

4.2.3 Right flank

Four boreholes (LR 3, LR 4, LR 5 and LR 6) have been completed on the right flank. They encountered between 7,5m and 19m of highly weathered diorite, followed by unweathered rock. Test pit TPL 5 located along the same ridge showed at least 4,5m of highly weathered diorite.

At a depth of about 1m, the highly weathered diorite is a suitable founding medium for the shells of an embankment dam, but the zone between 1 and 7,5 - 19m) is highly permeable (about 30 Lugeons), and would be difficult to grout. This weathered zone will have to be sealed by means of a cut-off trench or jet grouting.

Where the trench is more that about 5m deep, it will have to be excavated with side slopes of about 0,5H:1V, and thereafter backfilled with clay. Curtain grouting can be performed either from the bottom of the trench before backfilling, or through the fill after backfilling.

According to the aeromagnetic map, the upper part of the right flank is in a Low intensity zone, indicative of deep weathering. This has been confirmed by deep weathering in Borehole LR6. Based on the results of the aeromagnetic map, consideration may be given to move the upper right flank centre line to an area of Medium intensity and possibly less weathering as shown on **Figure 3**.

The proposed spillway structure near TPL 5 can be found on slightly weathered diorite at a depth of about 11m, while a concrete-lined chute along the ridge can be founded on moderately to highly weathered diorite at an estimated average depth of about 2m. Groundwater level is situated between 8,92m and 24,8m depth with an average at 14.7m below ground level. It is anticipated that there will be no problem of slope instability due to ground water condition during excavation.

4.2.4 River section

The river section is underlain by a layer of alluvium above diorite bedrock. The thickness of alluvium is variable, but outcrops both upstream and downstream of the centre line, and the results of one borehole to the left of the river channel, indicate that it should generally be less than 5m. The depth of rock that is suitable for founding of a concrete spillway structure, varies between 5 and 12m, with an average of about 10m.

4.2.5 Construction materials

4.2.5.1 Material for embankment shells

Test pits in the dam basin show an extensive cover of colluvial material within the Low intensity areas on the aeromagnetic map. However, the test pits dug by means of the TLB could not penetrate the full depth of the colluviul cover, and the results are inconclusive. It is recommended that additional pits be dug by means of a large excavator.

The colluvial material is extremely variable in terms of grain size and varies from silty sand with occasional (5 - 10%) pebbles to coarse gravel and boulders (60 - 80%) in a matrix of silty sand. Boulders of up to 350mm in size have been encountered, and this may affect the method of material selection and embankment construction.

The laboratory tests on colluvial materials show that soils are classified as SC (clayey sands, sand-clay mixtures of low potential expansiveness with an average plasticity index of 12), CL (inorganic clays, silty and sandy clay of low to medium potential expansiveness and plasticity, average PI = 16.8) and MH (inorganic silt of medium to very high potential expansiveness and high plasticity, average PI = 44.8) according to the Unified Soil Classification System. Clay contents vary between 3 and 42 with an average of 19.75.

These soils are suitable as general embankment fill but are not sufficiently impermeable for use as core material. The colluvium also contains varying percentages of gravel and boulders that may have to be removed during embankment construction.

TPL 11 on the right bank gave good results at shallow (0.55m) depth, but deeper down, the soil contains too much clay.

It is anticipated that about 500 000m^3 of colluvium could be obtained below FSL in the dam basin.

4.2.5.2 Material for clay core

Test pitting during the earlier feasibility stage investigations, showed at least 2m of colluvial soil in the area under irrigation downstream of the left flank. This material was considered suitable as core material. Other areas outside the FSL of the present dam were identified as potential borrow area but had not been investigated.

4.3 Intake / Surge Shaft

No shaft drilling was carried out during this investigation phase, but it is recommended that the next drilling phase includes a 700m deep borehole be drilled at the proposed shaft position.

Borehole BH6 (Feasibility Report of November 2000) was drilled 200m south of the present proposed shaft position to a vertical depth of 201m and indicates that the rock condition should be good, comprising roughly 118m granophyre overlying melanogranophyre. The borehole did not extend into the underlying leptite and diorite horizons which are expected to be present over the remaining 450m of the shaft.

All of the rock is expected to be strong to very strong. Joints in the granitic rock are expected to be of the order of 500mm apart with open or partly open joints which are expected to become tight to very tight within a few metres depth. The gabbroic (diorite) rock is expected to be more jointed, with joints spaced between 150mm to 500mm and generally very tight to partly open. No dykes or faults are expected, although water bearing fracture zones trending northwest and northeast may be encountered below the water table, and water pressures up to 60 bars should be expected if this occurs.

Rock removed from the shaft should be eminently suitable for crushed fine and coarse concrete aggregates.

4.4 Pressure Tunnel

Borehole PT 01 was drilled at an angle of 60° below horizontal to a depth of 400m measured on the incline, intersecting the pressure tunnel at a vertical depth of approximately 330m below the ground surface. The borehole is primarily in very strong rock diorite, with occasional bands of very strong to extremely strong rock magnetitie or mixed magnetite and anorthosite. A distinctive “zebra-striped” band approximately 20m thick was intersected where the borehole crosses the pressure tunnel; this is expected to have a very shallow dip and may therefore persist within the tunnel excavation profile over a fairly large distance. The diorite and magnetite/anorthosite-rich sections are very competent rock and jointing is generally medium to widely spaced. Joints are expected to be closed to slightly open. A number of very closely fractured zones do occur in the borehole, however a fairly large very closely fractured zone was encountered in borehole PT 01 between approximately 235m and 250m. This is thought to be near-vertical, probably intersecting the tunnel over a length of approximately 10m within the steel-lined section a short distance upstream of the bifurcations. This feature may well be associated with water inflows.

Five hydraulic jacking tests carried out in borehole PT 01 in the area of the pressure tunnel indicate an average minimum horizontal stress of 6.9 MPa (range 5.5 to 8.1) based on jacking. The average maximum horizontal stress is 22.1MPa (range 14.3MPa to 28.6.1MPa) calculated from first breakdown pressures. This value decrease at the second breakdown pressure with an average of 9.3MPa (range 6.0MPa to 12.9MPa).

Surface topography suggests that a prominent depression trending roughly NNE that crosses a spur below the escarpment may represent a sheared or fractured zone that could extend down to the tunnel at about midway along its length. Such a feature may well be associated with water inflows.

4.5 Machine and Transformer Halls

Borehole MH 01 immediately northwest of the machine hall had been drilled vertically to a depth of 312m at the time of writing of this draft, and will continue to 350m. Borehole SC 01 immediately southeast of the machine hall was drilled to a depth of 300m.

The distinctive “zebra-striped” band of mixed anorthosite and magnetite approximately 20m thick (recorded in angled borehole PT 01 in Section 2.4 above) occurs at similar elevations in these two boreholes, as well as in angled borehole TR 01 approximately 450m to the southeast and in Feasibility Study borehole BH 1 approximately 380m to the northeast. The planar relationships between the elevation of intersection with the different boreholes is being analysed to determine the orientation of this marker band as an aid in identifying possible shear/fault zones by interpolation and extrapolation.

The band is very strong to extremely strong rock and appears to occur in the upper levels of the machine and transformer halls. The band does not appear to form a discontinuity with the adjacent very strong rock diorite, i.e. no obvious plane of weakness appears to occur between the rock types.

Five hydraulic jacking tests carried out in borehole SC 01 in the area of the transformer hall indicate an average minimum horizontal stress of 7.0MPa (range 5.6 to 8.4) based on jacking. The average maximum horizontal stress is 23.8MPa (range 16.8 to 31.1) calculated from first breakdown pressures, this value decrease at the second breakdown pressure with an average of 13.2MPa (range 7.0MPa to 20.9MPa).

4.6 Surge Chamber

The borehole SC 01 was drilled in the area of the originally proposed surge chamber, which has subsequently been eliminated. The borehole is, however, well placed for the new transformer hall position and has been discussed in Section 3.5 Machine and Transformer Halls.

4.7 Tailrace, Access and Emergency Tunnels

Boreholes TR 01 and TR02 were drilled on the previously proposed alignment of the tailrace tunnel and outfall portal respectively and were to provide an indication of likely tunnelling conditions and depth of overlying weathered rock, as well as an indication of the founding for the outfall structure and likely portal and trench excavation slope requirements.

Borehole TR 01 was drilled inclined 60° below horizontal to enable core joint orientation measurements to be carried out for stereographic analysis of potential shallow tunnel and rock portal failure modes and support requirements. The core joint measurements have been completed and the stereographic projections of the joint surveys have been carried out and the results are included in **Appendix D**.

Borehole TR 01 shows over 10m of colluvial deposits overlying highly weathered diorite to a vertical depth of 30m, with approximately 50m of hard rock cover to the tunnel, while borehole PT 02 shows shallow colluvium underlain by very closely jointed weathered to slightly weathered diorite rock down to approximately 25m depth, with improved rock conditions for about 20m above the tunnel at the portal. The portal and trench excavations will require shallow angle cut slopes or fairly robust support measures.

5 Geophysical Surveys and In Situ Testing

5.1 Airborne Aeromagnetic Surveys

Final Reports have been received for the airborne aeromagnetic surveys and the findings were summarized as follow:

- Granitic lithologies underlying the Nebo Plateau in the west are non-magnetic.
- Mafic rocks at the Upper Zone of the RLS are weakly to strongly magnetic.
- Identification of some fault and dykes on site.

The full report of airborne aeromagnetic surveys is included in **Appendix F** of this Report. Information from this work has been taken into consideration in the current preliminary design work, and will be further used during the Design Stage geotechnical investigations.

5.2 Seismic and Resistivity Surveys

Seismic and resistivity surveys have been very successful in assisting interpretation of the upper reservoir founding conditions, but since they were carried out on more recent proposed alignments than were used for the drilling, additional drilling will be essential to complete interpretation. The traverses at the upper reservoir are shown on the various geotechnical layout plans, and the graphic longitudinal sections of the seismic and resistivity surveys are presented on A4 figures in **Appendix G**.

Further seismic and resistivity surveys are planned for the lower reservoir Design Stage investigations to aid determination of founding, clay core and grouting depths beneath the proposed earth embankment. The full report of seismic and resistivity surveys are included in **Appendix G**.

5.3 Hydrofracture Testing and Hydraulic Jacking tests

Hydrofracture testing was carried out recently, near the end of the drilling of the deep boreholes for the underground waterways. The state of the stress has been measured using twelve tests in two boreholes to depths from near surface to 322m. The measurements were carried out from surface in the vicinity of the proposed pressure tunnels and powerhouse complex. The table below gives the projected stress values at a depth of 265 m

		Horizontal Stress versus		Stress at 265 m
		Lithostatic	Depth	
	Units	(MPa/MPa)	(MPa/100m)	(MPa)
1st cycle	$\sigma_{H\max}$	3.31	8.93	23.6
	$\sigma_{H\min}$	1.11	2.99	7.9
2nd cycle	$\sigma_{H\max}$	1.63	4.40	11.7
	$\sigma_{H\min}$	1.08	2.99	7.9
Jacking cycle	$\sigma_{H\max}$	1.37	3.70	9.8
	$\sigma_{H\min}$	1.10	2.97	7.9

The overall average maximum stresses to be considered in the design will be in the order of 22.8 MPa. The full report of hydrofracture testing is included in **Appendix H**.

5.4 Wireline Borehole Surveys

Wireline borehole surveys were carried out recently, together with the hydrofracture testing, in order to map hydrofracture orientations. The reports are still being prepared, but preliminary results have been included in **Figures 5** showing joint orientation measurements deep underground in the area of the power station complex. Azimuths of these results may need to be adjusted following further analysis, but are presented as a brief indication of some of the data that will be obtained from these surveys carried out over the full lengths of boreholes PT 01 and SC01. The full report of wireline borehole surveys is included in **Appendix I**.

6. Laboratory Testing

Soil sampling has been carried out for preliminary laboratory testing of materials for construction of the upper reservoirs and possible road construction materials from within the dam basins. Test results have been received and are included in **Appendix D**. Foundation indicator test, soil mortar, proctor and CBR testing have been carried out on these soil samples. The results show that the welling characteristic of these materials are negligible (<0.9%) with an average optimum moisture content of 12.9%. These materials have been classified as G7 to G9 materials. Properties of these materials are good enough to be used in the pavement layers such as subgrade, selected subgrade, subbase and probably for basecourse if stabilised.

Soil samples from the lower reservoir have been tested for foundation indicator, shear strength, consolidation characteristic as well as permeability testing, see **Appendix D**.
The laboratory tests on colluvial and hillwash from Lower Reservoir show that soils classified as SC (clayey sands, sand-clay mixtures of low potential expansiveness with an average plasticity index of 12), CL (inorganic clays, silty and sandy clay of low to medium potential expansiveness and plasticity, average PI = 16.8) and MH (inorganic silt of medium to very high potential expansiveness and high plasticity, average PI = 44.8) according to the Unified Soil Classification System. Clay contents vary between 3 and 42% with an average of 19.75%. The soil cover at the left flank is considerably thick (5 to 15m) compared to the soil cover at the right flank which is generally less than 0.5m. Friction angle and cohesion obtain from drained direct shear test varied from 27.4 to 34.4 degrees with an average of 30.24 degrees and 0 to 7.4 kN/m^2 with an average of 4.26 kN/m^2 respectively.

Permeability testing was conducted and the results were found to be highly variable depending on the type of material and ranging from 6.7 X10^{-9} to 4.3 X 10^{-4} cm/s.

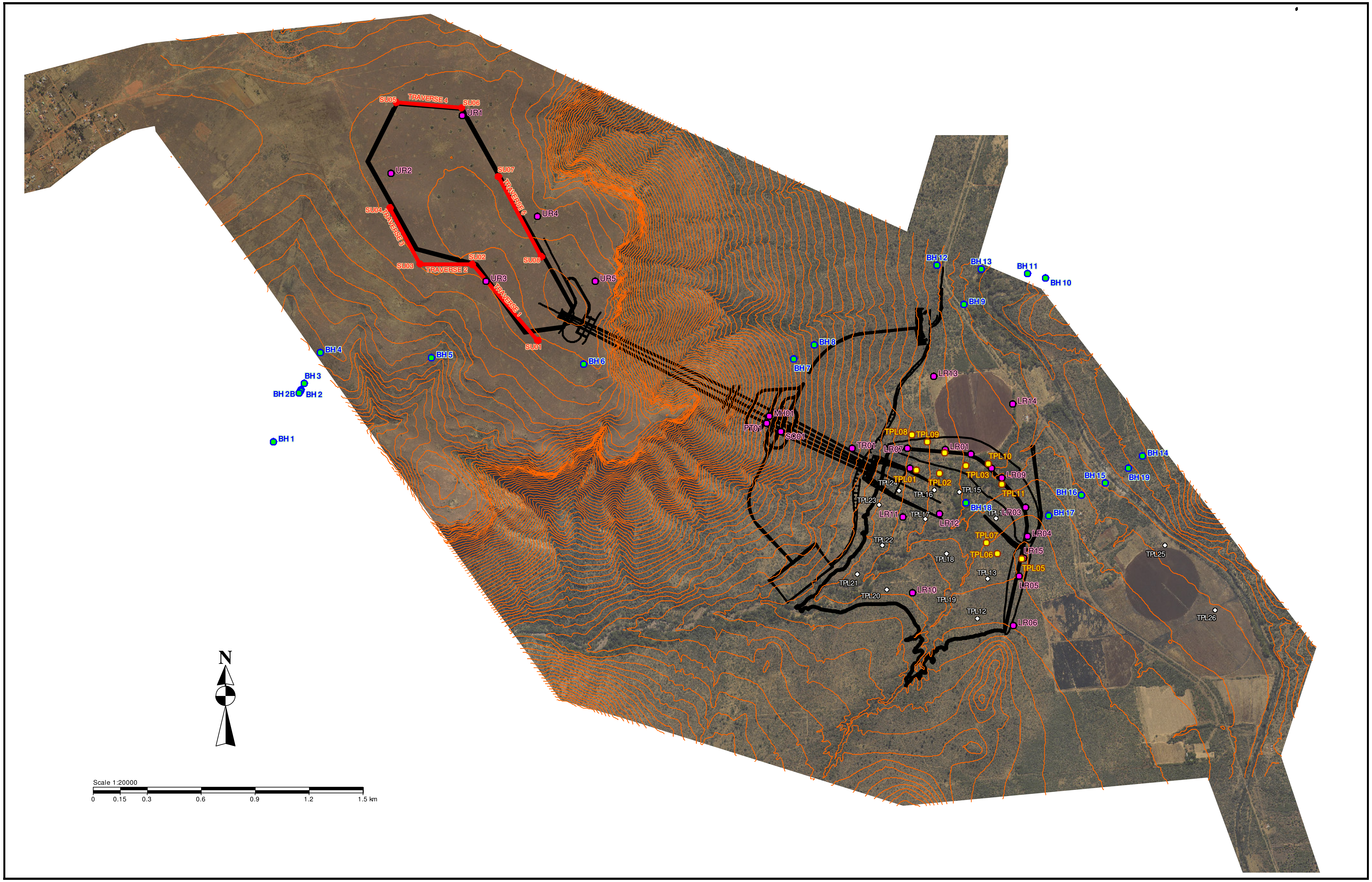
Conclusion

The feasibility stage geotechnical investigations have revealed the following:

- Poorer foundation conditions at both the upper and lower reservoir than originally anticipated.
- In general, construction materials for the dam walls are all available within the dam basin. Clay material for the lower dam core is available in close proximity.
- Generally very good rock conditions for the underground works.
- The various rock types grade almost imperceptibly into one another with the boundaries not readily apparent.
- Major faults/shear zones should not be discounted in the underground works in the area of the pressure tunnel, although some minor zones of highly fractured rock, which may well be associated with water inflows, should be expected.
- A distinctive "zebra-striped" band of very strong to extremely strong rock (mixed anorthosite-rich and magnetite-rich diorite) appears to occur in the upper levels of the machine and transformer halls. This band is expected to occur over a fairly extensive length in the pressure tunnel. The band does not appear to form a discontinuity with the adjacent very strong rock diorite, i.e. no obvious plane of weakness appears to occur between the rock types.
- Portal and trench excavations will require shallow angle cut slopes or fairly robust support measures.

Further investigations will be required in order to supply sufficient information and data for tender and detailed design to proceed. This would include, inter alia:

- determination of the principle in-situ stresses in the rock mass
- determination of the in-situ elastic rock modulus and Poisson's ratio of the rock mass
- calibration of the seismic traverses at the upper reservoir by means of additional boreholes



Legend

- Proposed Scheme
- Feasibility Study Boreholes
- Phase2 Boreholes
- Trial Pits
- Test Pits
- Seimic/ResistivityTravers

BKS Palace Consortium

CLIENT

Eskom

PROJECT

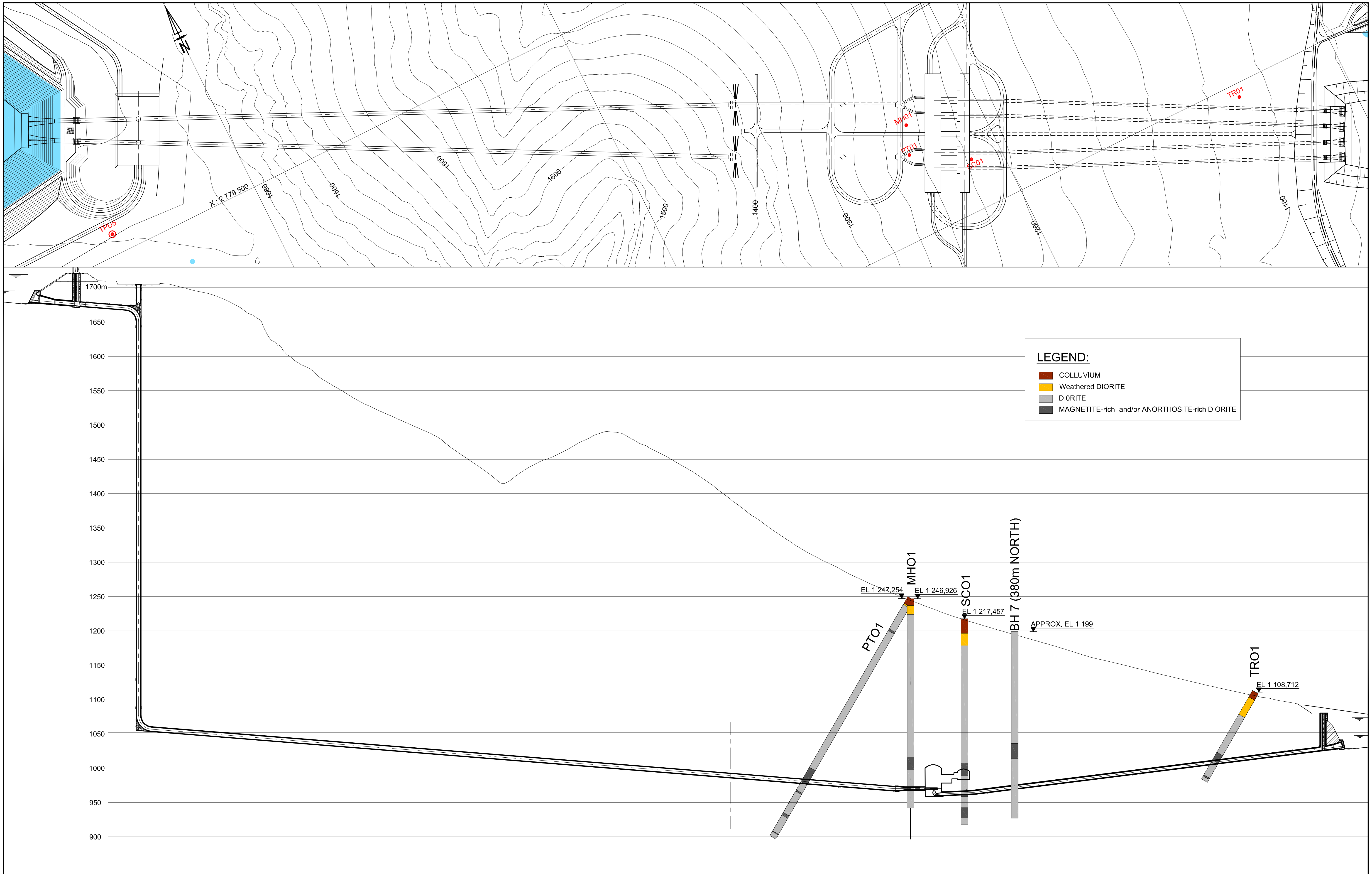
PROJECT LIMA
STEELPOORT PUMPED STORAGE SCHEME

DRAWING TITLE

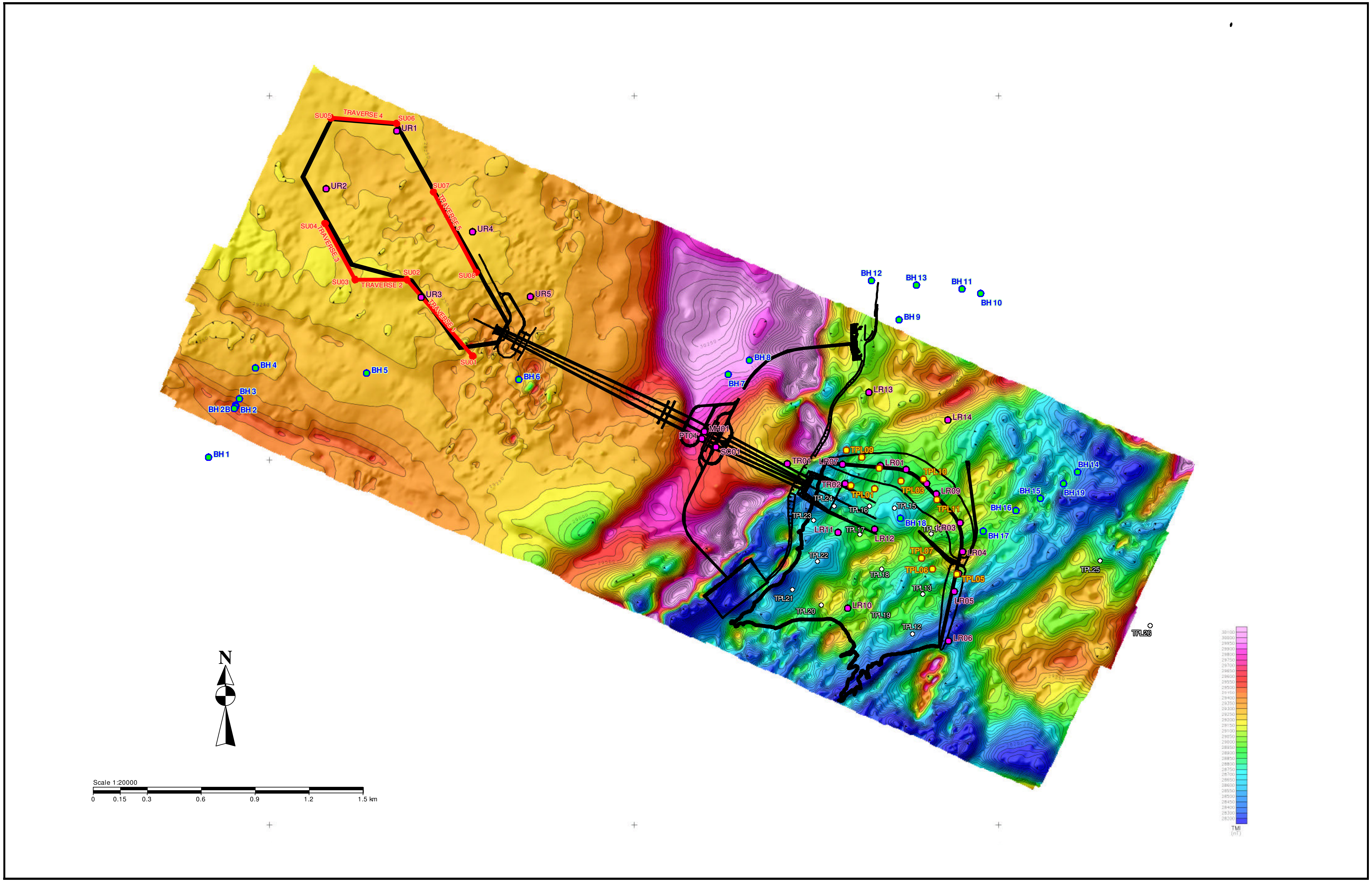
Orthophoto Layout Plan of Proposed Scheme, Boreholes, Trial Pits and Seismic Traverses

FIGURE 1

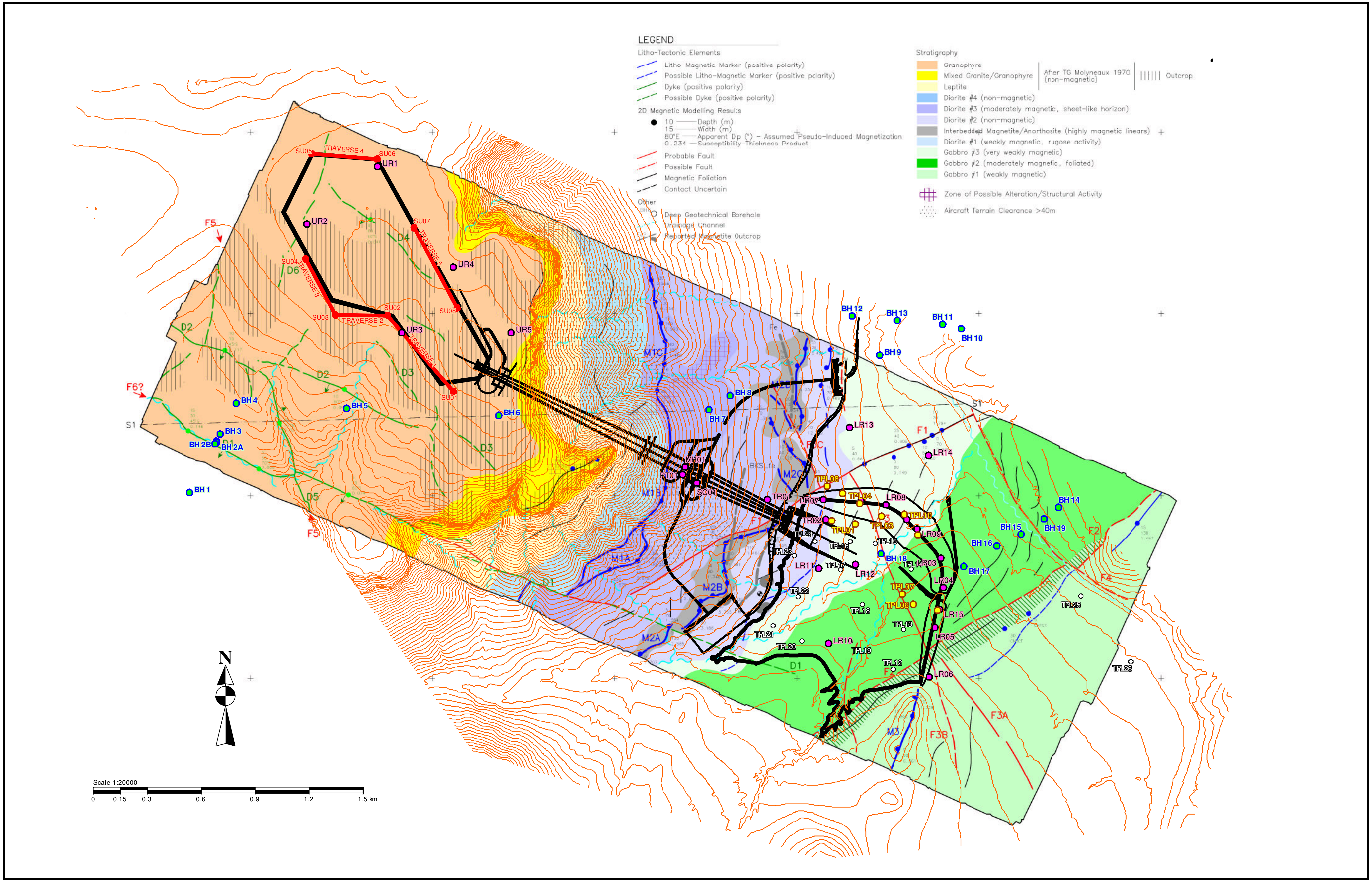
Date : December 2006
Map Ref. : P:/.../Lima_ALS_XY.map
System : WGS84/Lo29



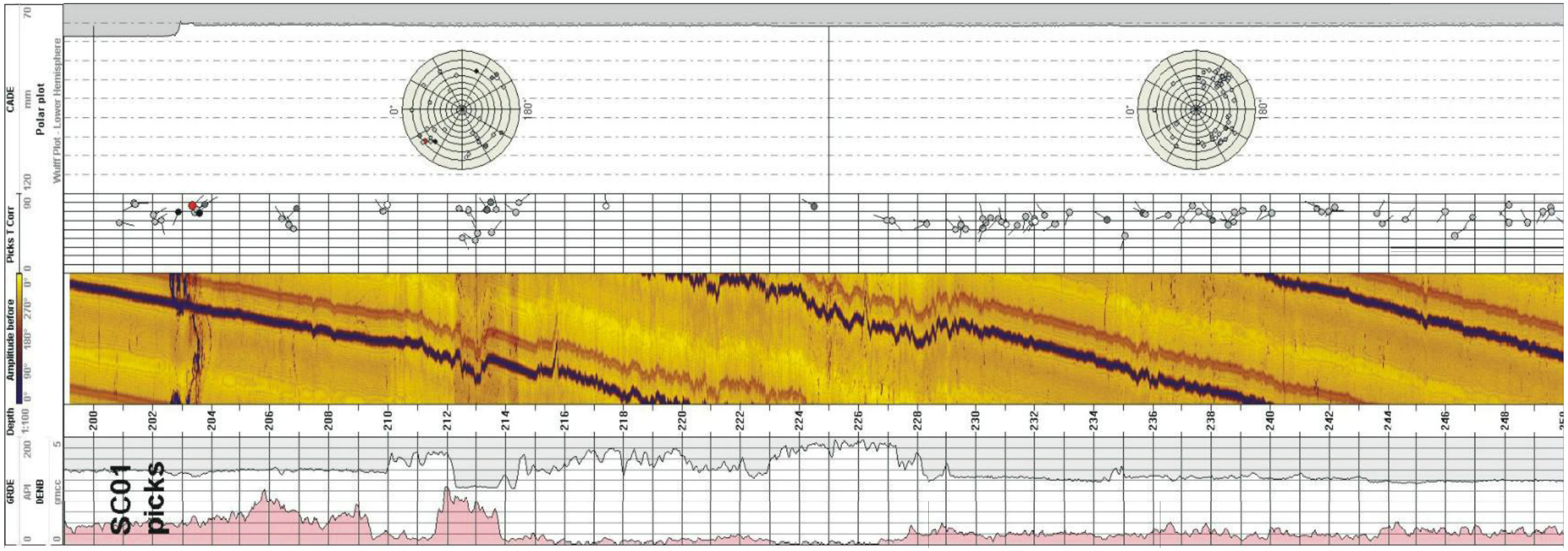
0 50 100 250 500m SCALE 1 : 2 500 (ORIGINAL A1 SIZE) SCALE 1 : 5 000 (REDUCED A3 SIZE)	BKS Palace Consortium	CLIENT Eskom	PROJECT PROJECT LIMA STEELPOORT PUMPED STORAGE SCHEME	DRAWING TITLE STEELPOORT PUMPED STORAGE SCHEME GEOLOGICAL LONGITUDINAL SECTION	FIGURE 2 SCALE AS SHOWN



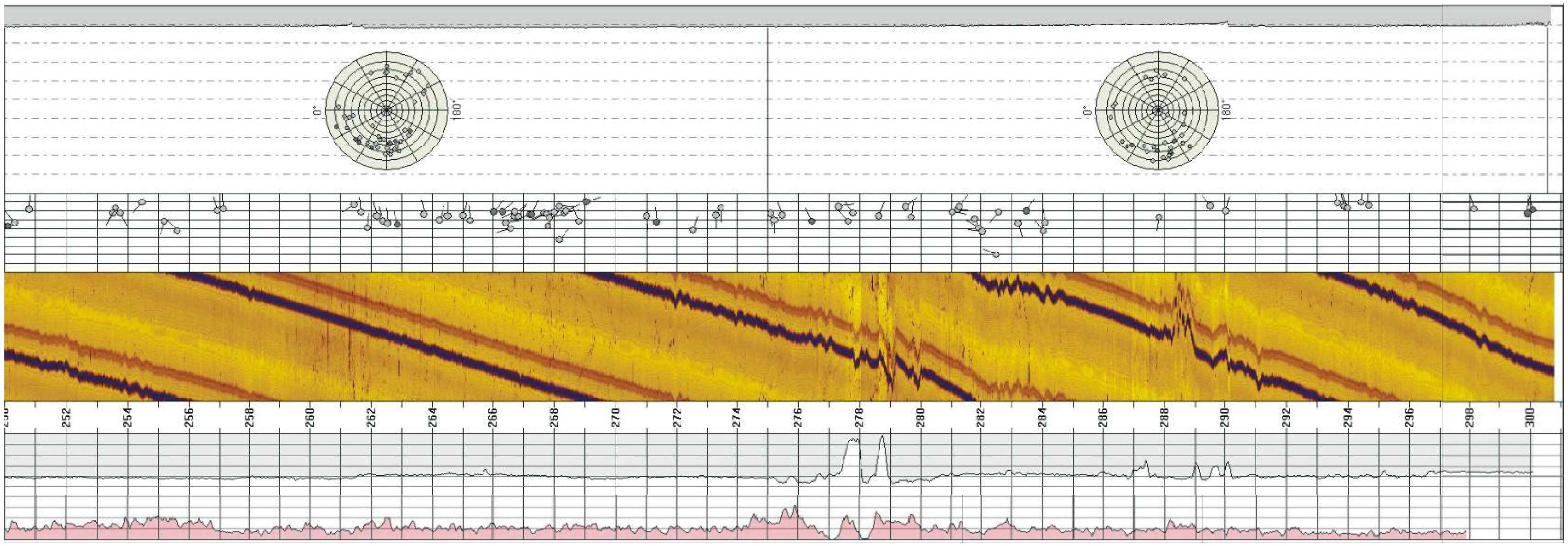
Legend Proposed Scheme Feasibility Study Boreholes Phase2 Boreholes Trial Pits Test Pits Seismic/Resistivity Traverse	**BKS Palace Consortium**	CLIENT Eskom	PROJECT **PROJECT LIMA** **STEELPOORT PUMPED STORAGE SCHEME**	DRAWING TITLE **AEROMAGNETIC SURVEY** **Total Magnetic Field Image** **Sunshaded from the SouthEast (135°T)**	**FIGURE 3** Date : December 2006 Map Ref. : P:/.../Lima_ALS_XY.map System : WGS84/Lo29 COPYRIGHT RESERVED



Legend		BKS Palace Consortium	CLIENT	PROJECT	DRAWING TITLE	FIGURE 4
Proposed Scheme	Test Pits		Eskom	**PROJECT LIMA**	**Helicopter-borne Gradiometer**	Date : December 2006
Feasibility Study Boreholes	Seismic/Resistivity Traverse			**STEELPOORT PUMPED STORAGE SCHEME**	**High-Resolution Aeromagnetic Survey**	Map Ref. : P:/.../Lima_ALS_XY.map
Phase2 Boreholes					**Geophysical Interpretation Map**	System : WGS84/Lo29
Trial Pits						

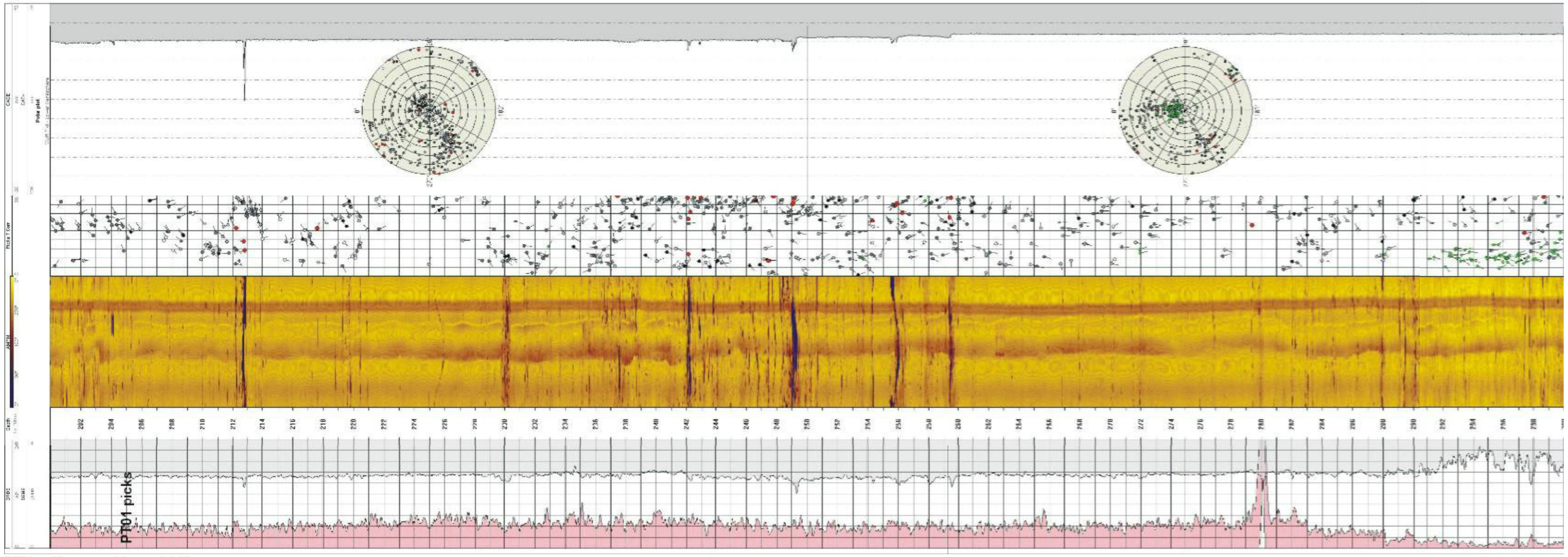


SC 01 (From 199 to 250)

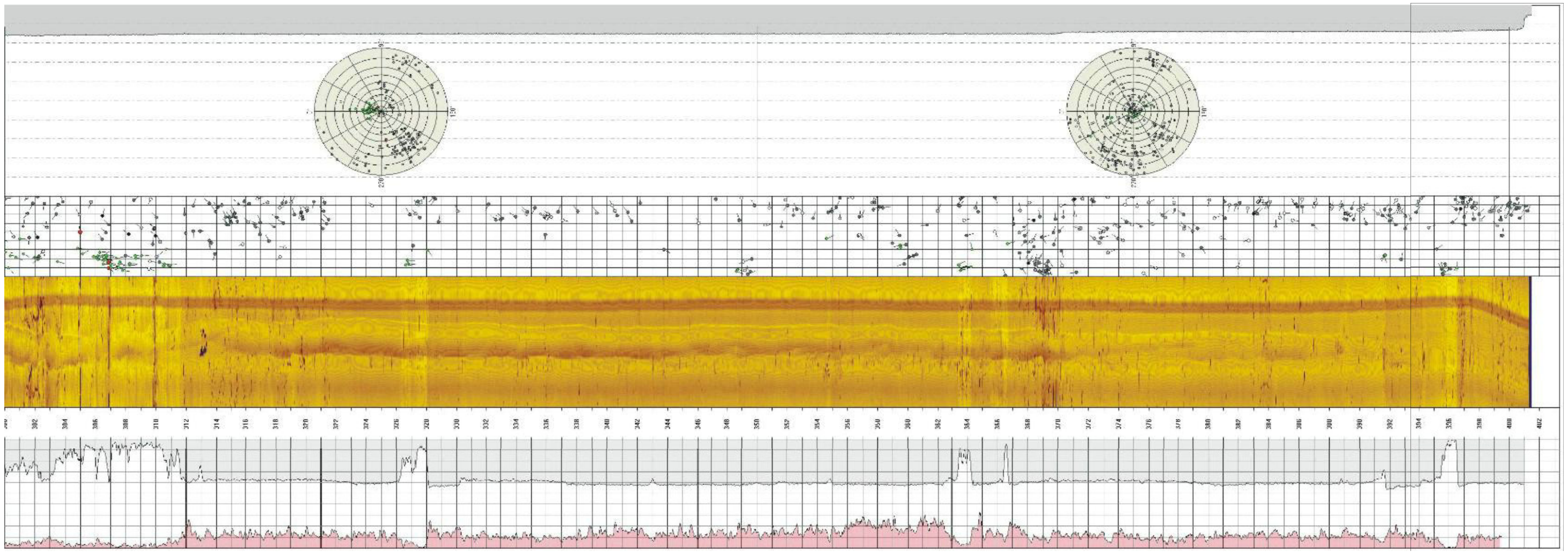


SC 01 (from 250 to 301)

BKS Palace Consortium	CLIENT	PROJECT	DRAWING TITLE	FIGURE 5A
	Eskom	**PROJECT LIMA** **STEELPOORT PUMPED STORAGE SCHEME**	**Wireline Borehole Survey Picture**	Date : January 2007 Map Ref. : P:/.../Lima_ALS_XY.map System : WGS84/Lo29



PT01 (from 199 to 300)



PT01 (from 300 to 401)

BKS Palace Consortium	CLIENT Eskom	PROJECT **PROJECT LIMA** **STEELPOORT PUMPED STORAGE SCHEME**	DRAWING TITLE **Wireline Borehole Survey Picture**	**FIGURE 5B** Date : January 2007 Map Ref. : P:/.../Lima_ALS_XY.map System : WGS84/Lo29 COPYRIGHT RESERVED