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PROJECT BRAVO

REPORT ON GEOTECHNICAL INVESTIGATIONS UNDERTAKEN AT THE POWER STATION SITE

VOLUME 1

TEXT AND APPENDIXES A - D

1 – 6/07
March, 2008

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1. INTRODUCTION

In terms of Contract No. 4600008252 dated 3rd July 2007 Eskom Holdings Limited (represented by its Enterprises Division) appointed Partridge, Maud and Associates to undertake geotechnical investigations for the proposed new Project Bravo power station, situated near Balmoral in the Mpumalanga Province. The final investigations were to cover two areas: the site for the power station itself and the site for the ash dump. This report presents the results of investigations specific to the power block terraces.

1.1 Outline of the Scheme and Summary of the Brief

The proposed Project Bravo Power Station is to be a close replica of the Medupi Power Station. It will be fired by coal from an open pit mine to be established to the east of the site and straddling the existing R545 road. It will incorporate boiler and turbine house structures, chimneys approximately 180 m high, coal and ash silos, direct/indirect cooling system structures, conveyor systems, water reservoirs, a water treatment plant, office buildings, internal roads and ash disposal facilities. The latter may ultimately be of limited extent should the ash generated be used for backfilling of the open pit coal workings.

The overall development is, therefore, characterized by a diversity of structures with widely differing requirements in terms of foundation design and structural performance. Sources of suitable natural materials are required for earthworks (terraces and road prisms) and for use as fine and coarse concrete aggregate. An important requirement is that the materials on which the heavier structures are founded should be capable of withstanding pressures of up to about 1.0 MPa without settlements that would be prejudicial to the performance of these structures; and founding depths should, of course, be as shallow as possible.

The power station and ancillary structures are conventionally placed on one or more flat or gently sloping terraces created by cut and fill operations. In the present case the power island will occupy a flat terrace with a provisional elevation of 1502.8m, while the switchyard immediately to the north-west will be placed on a platform with a provisional elevation of 1500.2m (e-mail of 17/10/2007 from D. Short of Black and Veatch Corporation). Our present appointment by Eskom relates to these areas and the ash dump site only, and this report is limited specifically to factors relevant to the construction of the two terraces described above and general conditions for the founding of structures within the power block area.

The following specific tasks were listed in the Scope of Work for the investigations at the power station site:

- siting of exploratory boreholes, supervision of drilling and logging and sampling of the borehole cores
- test pitting and sampling of construction and foundation materials
- laboratory testing of core and soil samples
- water pressure tests and Goodman Jack tests in the boreholes
- detailed analysis of results and compilation of a final report including
    - All relevant factual test data, borehole logs and profiles

- Discussion and interpretation of the results of the drilling and materials testing programmes
- An assessment of available construction materials sources
- Proposed site preparation, earthworks and terrace construction philosophy
- Proposed use of road construction materials
- Recommended alternative foundation solutions, allowable bearing pressure, settlement magnitudes and practical considerations
- Seismic risk assessment
- Descriptions of likely subsurface conditions that will be encountered in excavations with respect to rock type, jointing, bedding planes, erodability, weatherability, durability, groundwater inflow and rock stability

Additional inputs were the execution of cone penetrometer (CPT) tests, and plate-bearing tests – carried out by means of cross-hole jacking within test pits.

2. GENERAL DESCRIPTION OF THE POWER STATION SITE AND ENVIRONS

The selection of the area on which to locate the Project Bravo power station was the culmination of a systematic process of environmental and geotechnical evaluation in which our firm was a participant. The site finally approved is that described in our report of 23rd April 2007 entitled "Project Bravo: Preliminary site selection based on geotechnical criteria", in which it was designated Site E. The preferred site put forward in that report, Site A, was adjudged by Eskom to be too close to the N4 freeway to be suitable for the final power station development.

The present site is located 20 km to the north-north-west of the existing Kendal Power Station and 8 km to the south-west of the village of Balmoral, on the Farm Hartbeesfontein 537 JR.

The area is one of rolling topography produced by dissection of the landscape by erosion along tributaries of the Holfonteinspruit and Klipspruit, both of which flow ultimately into the Wilge River. Local relief is of the order of 50 m. An elevated spur, whose surface lies above 1500 m over large areas, follows an approximately north-south orientation some 5 km to the east of the site. The R545 follows the crest of this spur. The coal source for the station is located in rocks of the Ecca Group beneath this spur.

On the site itself elevations range from about 1520 m in the south-east to about 1485 m along its north western boundary. With these variations in elevation, construction of the terraces for the power station will involve considerable earthworks.

A number of apparently perennial seep zones occur on the lower slopes near the site and most nearby valleys contain streams whose flow, although not large, appears to be sustained through most of the year. The groundwater hydrology of the area is the subject of a separate study carried out by Golder Associates Africa (Pty.) Ltd. Water rest levels in the boreholes drilled in the course of the geotechnical investigation ranged from about 5 m to 9 m below existing ground surface.

The site is occupied by grassland with occasional patches of low trees and shrubs. The climate of the area is sub-humid, with a mean annual rainfall at Balmoral of 665 mm, most of which occurs in summer. Temperatures are moderate, with a mean daily maximum of 24°C and a mean daily minimum of 7°C. Frost occurs on about 40 – 50 days per year.

3. GEOLOGICAL SETTING

3.1 Major Rock Types and their Weathering Properties.

The principal bedrock unit beneath the site is altered (baked) shale belonging to the Palaeoproterozoic Transvaal Supergroup. This rock crops out in the central portion of the site (see Plan 0.90/31). Because it is often highly fractured, this material is moderately susceptible to weathering, which gives rise to a mantle of gravelly residual soil, although clay-silts of low plasticity overlie it locally. Beneath this, less weathered material is usually present at fairly shallow depth, and founding of most major structures would be possible at no great depth below natural ground level on this rock type.

Intrusive into the shale is a diabase sill. The diabase is more susceptible to weathering than the shale and this locally gives rise to a surface layer of reddish, residual clay-silt, which may be several metres in thickness. This material is both compressible and of medium potential expansiveness, but it is not present everywhere above the diabase bedrock. Its distribution is shown on Plan 0.90/31. Beneath it, and present to surface in some areas, is a dense to very dense, yellowish sandy residual material. Both the reddish and yellowish soils residual from the diabase contain large, hard rock boulders, some of which may attain diameters of 2 metres. Much of the diabase coincides with that part of the terrace underlain by fill.

Over much of the higher-lying part of the site Permo-Carboniferous rocks of the Karoo Supergroup overlie the Palaeoproterozoic shale (see Plan 0.90/31). These include shale, siltstone, tillite and subordinate sandstone, and attain up to 40 m in aggregate thickness. These lithologies are variably susceptible to weathering, with pockets of residual soil extending to depths in excess of 30 m below natural ground level in places. A great deal (but not all) of this residual soil will be removed during terracing operations. Over most of the area these soils are clayey sands and sandy clays of low plasticity, but, in a few localities the residual shale and tillite plot within the medium potential expansiveness category. It is considered unlikely, however, that these soils will give rise to problems of heave (most are significantly compressible under moderate imposed laboratory pressures). What is likely to be of considerably greater significance is the presence of a boulder component in the tillite. The boulders are typically less than one metre in diameter, but can be larger in isolated cases. Since they are of various lithologies, many remain hard within the weathering mantle. Depth of founding for moderately loaded structures will be variable, but over a significant area near the centre of the site reasonably competent material occurs at or near the proposed terrace level.

3.1.1 Dwyka Group

Rocks of the Dwyka Group (Karoo Supergroup) in the study area are dominated by the Permo-Carboniferous (300 myr) tillite facies. This consists of polymictic clasts set in a poorly sorted, fine-grained matrix. In thin section the rock classifies as a wacke within which quartz grains and detrital rock fragments are common. More shaley facies containing occasional dropstones are relatively common, as are fluvioglacial gravels in a silty, or even sandy, matrix. The shale classifies in thin section as a claystone in which quartz and clay minerals predominate; where more silty it classifies as a sandy mudstone. In the fresh state the tillite is a hard rock, the shale softer. Both weather readily to residual soil. Dips are generally sub-horizontal. Different lithological facies occur both in vertical and horizontal succession and can be distinguished in section but not differentiated cartographically.

3.1.2 Diabase intrusions

The pre-Karoo country rocks in the area (Transvaal Supergroup) have been intruded, on and around the site, by sills of diabase which are related to the emplacement of the basic phase of the nearby Bushveld Igneous Complex at around 2100 myr. It is a greenish or bluish-grey hypabyssal rock, lithologically akin to the more coarsely crystalline grey norite of the Bushveld Complex. In thin section it classifies as a meta-dolerite in which plagioclase and pyroxene/amphibole are the dominant minerals; it has a relatively high biotite content. The diabase sills, which may be several tens of metres thick, are usually conformable but may take on weakly unconformable, basin-like forms.

3.1.3 Rayton Formation

The oldest rocks in the vicinity of the site are shales of the Rayton Formation. This is a unit of the Pretoria Group (Transvaal Supergroup) and has an age of about 2150 myr. In the fresh state it is a dark grey, thinly bedded, hard rock which has become indurated through the widespread intrusion into it of the diabase sills discussed in 3.1.2 above. In thin section it classifies as a claystone, in which the matrix is rich in clay minerals through which silt-sized quartz is sparsely distributed. Prevailing dips in the shale in the vicinity of the site are about 15° to the south-west, but range from 10° to 25°.

Quartzite of the Rayton Formation occurs in a fairly narrow belt, extending in a roughly north-south direction, starting about 3 km to the north-east of the site and extending northwards across the N4 highway (Figure 3). It consists of a whitish or greenish very hard, fine grained quartzite which weathers locally to a silty residual soil where highly fractured or covered by transported overburden. In the fresh state it provides the only nearby source of crushed rock potentially suitable for use as road base, ballast, the capping layer of the terrace, as well as rockfill for placement as a foundation support. It was extensively exploited in the Spitzkop Quarry, immediately north of the N4, for use in the construction of that freeway. Its occurrence and potential use are considered further in Section 6.2 of this report.

3.2 **Structural Features**

A number of small-displacement faults are visible in rocks of the Rayton Formation. These were nowhere seen to continue through the covering of Karoo rocks where these overlap the older lithology, and it is therefore assumed that these features are generally of pre-Karoo age (i.e. older than about 300 myr). No displacements within the Karoo rocks could be detected on Landsat TM imagery of the area within a radius of 25 km of the site. The nearest large faults occur some 35 km to the north of the site, where they cause the juxtaposition of rocks of the Waterberg Group against Nebo Granite of the Bushveld Igneous Complex. These faults are similar to those present elsewhere around the periphery of the Bushveld Basin. They were discussed by A.L. du Toit (South African Geographical Journal 16, 3, 1933) and Partridge and Maud (The Cenozoic of Southern Africa, Oxford University Press, 3 – 18, 2000) who consider them to be related to subsidence within the Bushveld Basin during the Mio-Pliocene period (about 5 myr ago). It is therefore concluded that recently active "capable" faults, with the potential to generate significant present-day seismic vibrations, are not present in proximity to the site. Seismic risk in the area is considered further in Appendix G.

4. INVESTIGATIONS CARRIED OUT

The following work was carried out on the power station site and its terrace area (see Plan 0.90/31):

- 82 cored boreholes drilled by rotary methods. These were drilled in two phases. The 38 designated "bh" on the accompanying drawings were drilled prior to 27th August 2007. At that time it was discovered that a systematic error had occurred in Eskom's drawing office at the time when field co-ordinates were calculated from positions previously selected and marked on a layout plan of the power block jointly by ourselves and staff of Eskom. It was also discovered that, although these erroneously placed boreholes had been drilled within the power block area, none was in its intended position. A second phase of drilling was then begun; this comprised 44 boreholes, again sited jointly by ourselves and Eskom, beneath specific structures which had not had boreholes drilled beneath or near them during the first phase. These are designated "BH" on the accompanying drawings.
- 3 cored boreholes drilled within the Rayton Formation quartzite some 3 km north-east of the site to assess its potential suitability for exploitation as a source of crushed rock.
  All boreholes were logged in accordance with core logging convention reproduced in Appendix A, and the logs thus generated are included in that appendix.
- the execution of Standard Penetration Tests in soils penetrated by the boreholes. The results of these are presented on the borehole logs. At the same time Shelby tube samples were recovered from representative soils for laboratory testing.
- the execution of pump-in (single packer) water pressure tests in representative boreholes to determine the continuity of fracturing within the rock mass and the potential for consolidation grouting to improve its properties.
- the installation of standpipe piezometers in 26 of the boreholes in the power block to permit long-term monitoring of piezometric levels.
- Point-Load Strength tests carried out on representative samples of the borehole cores during logging. The results are included on the borehole logs.
- the execution of 34 Unconfined Compressive Strength tests in the laboratory of Rocklab, Pretoria. In the course of 27 of these the Young's Modulus of the specimen was measured and in 26 Poisson's Ratio was also determined. The UCS values are incorporated on the borehole logs, and the results are reproduced in full in a separate volume of laboratory test results.
- 15 super heavy dynamic probe tests in areas of moderately thick to thick soil cover. These were sited adjacent to boreholes BH2, BH9, BH11, BH13, BH21, BH28, BH31, BH32,

BH33, BH37, BH42, BH43, BH50, BH62 and BH64. The results are reported in Appendix B.

- 30 test pits excavated by 30 tonne Hyundai 305 LC7 excavator. Their positions are shown on Plan 0.90/31 with the prefix PB, and their profiles are recorded in Appendix C.
- laboratory tests carried out according to our instructions on soil and rock samples by the Civilab laboratory, Johannesburg. The results are reproduced in the form they were supplied by the laboratory in a separate report. The planned testing programme was as follows:

Foundation indicator -	42
Full CBR test, including moisture/ density relationship -	11
Consolidation (undisturbed sample) -	4
Consolidation (remoulded sample) -	9
Double oedometer test (undisturbed sample) -	6
CNSU triaxial test (undisturbed sample) -	9
CNSU triaxial test (remoulded sample) -	3
Shear box test (remoulded sample) -	4
Rapid collapse test @ 200 kPa -	2
Density only -	1
Soil chemistry -	11
Falling head permeability (remoulded sample) -	11
XRD analysis and petrographic description -	5

The consolidometer and triaxial tests on undisturbed samples (Shelby tube samples from the boreholes and block samples cut from the sides of the test pits) were commissioned for three main purposes:

- to provide parameters for the estimation of settlement beneath the filled section of the terrace
- to provide parameters for assessing the stability of cuts flanking the excavated section of the terrace and for determining the potential for slip-circle failures to occur below the toe of the fill
- to provide parameters for pile design in those areas were deep founding of structures will be required

The consolidometer, triaxial and shear box tests on remoulded samples from the test pits were commissioned to provide parameters for estimating settlements within the fill itself, potential additional settlements which might occur beneath structures founded on it and safe side-slope angles around its margins. For these tests disturbed soil samples were remoulded to 95% of maximum Modified AASHTO density. In the case of the triaxial tests the samples were compacted by static pressure in a split mould before being placed in the test cell; the consolidometer samples were consolidated in the test-ring, while the shear box samples were compacted in an oversize ring before being extruded into the box. In the case of one of the undisturbed triaxial samples (PB19 at 2.00m), the sample proved too small to be tested undisturbed and the material was remoulded to 100% of maximum Modified AASHTO density and tested in a shear box.

In the case of the laboratory falling head permeability measurements, remoulding was also to 95% of maximum Modified AASHTO density.

Summaries of the test results are provided in Tables 1, 2, 3, 4 and 6 which are reproduced in the body of this report.

5. RESULTS OF DRILLING, SAMPLING AND TESTING

5.1 General Nature of Soil and Rock Mass in Terrace Area

Two representative sections across the terrace are shown in Figures 1 and 2. These indicate that considerable local variations are present in the thickness of the residual soil and in depths where very soft rock (UCS 0,7 – 4,0 MPa) and soft rock (UCS 4,0 – 10,0 MPa) were first encountered in the boreholes. It should be noted, furthermore, that residual soil not infrequently recurs below these depths. The plan distribution of shallow very soft and soft rock below terrace level is shown on Plan 0.90/35.

Drawings 0.90/32 and 0.90/33 show interpolated elevations of the Dwyka/Rayton unconformity and isopachs of thickness of the Dwyka rocks respectively.

As can be seen from the geological Sections and from the borehole logs reproduced in Appendix A, the depth below original ground level at which material of "rock" consistency is first encountered in reasonable thickness and continuity ranges from 0.55m (in borehole bh10 in the Rayton shale) to 20.80m (in borehole bh11 in Dwyka tillite). The average depths for the different rock types are:

Dwyka – 9.59m (range 4.75 – 20.80m)
Rayton shale – 3.53m (range 0.55 – 7.55m)
Diabase – 4.56m (range 1.65 – 9.22m)

It should be noted, however, that the area underlain by diabase can be subdivided into an area in which clayey residual soils overlie the rock and one in which the residual soils are predominantly sandy (see Drawing 0.90/31). In the former the average is 5.12m, whereas in the latter it is only 2.70m. More detail on these variations is provided in relation to each soil type under 5.3 below.

Also apparent from the geological sections and borehole logs is that the residual soil overlying bedrock is thickest beneath the highest-lying north-eastern portion of the site where the terrace will be in cut. It is, in fact, only beneath the extreme eastern corner of the terrace that continuous rock can be projected to occur above the presently proposed terrace level (see log of bh1). This means that excavation down to terrace level will be largely in "soft" material, with only localized zones of rock present within the residual soil (which may range in consistency up to very stiff/very dense). It is our view that blasting, if required at all, will be restricted to fragmenting a few localized lenses of rock above terrace level which cannot be removed by heavy excavators, and occasional large boulders present within the residual tillite. The large-scale use of blasting should, we believe, be avoided as a preferred excavation tool for this project.

5.2 **Water Table**

An important consideration relevant to excavation for the terraces is that the Dwyka Group sedimentary rocks, which cover the higher-lying portions of the site, are near-horizontally bedded and display major vertical and lateral facies changes. Some of these are associated with localized perched aquifers, although the depths of significant water strikes in boreholes was, in most cases, well below terrace level. It should, however, be noted that, since the aquifers are confined, the water rest level is considerably higher than the strike level. The piezometric surface which this rest level defines is above the terrace surface over part of its area (Drawing 0.90/34). The occurrence of some surface wetness in this area is likely, particularly given the fact that the rest level measurements were taken at the driest time of 2007.

Contours of the water rest level recorded in the boreholes are presented on Drawing 0.90/34. Considerable variations in the piezometric level are evident. These may relate to the presence of local perched aquifers. As has been noted, the piezometric surface recorded in September 2007 lies above terrace level over a significant area in the south-east of the cut-section. The significance of this in relation to possible seepage at terrace level, as well as methods for minimising such inflows, are discussed in detail in the accompanying report on the hydrogeology beneath the terrace prepared by Golder Associates Africa.

5.3 **Soils in Terrace Area**

5.3.1 Hillwash

This transported soil generally occurs in proximity to the Rayton shale/diabase contact, where it ranges in thickness from 0.4 – 1.3m (average 0.7m) and comprises a red-brown, stiff, fissured sandy clay with occasional quartzite, tillite and diabase gravels, cobbles and boulders (up to 1.2m diameter). There is also a 0.3m thick horizon of transported, medium dense, intact silty sand in the extreme northern parts of the site (PB3 and PB4). The index properties of the clayey hillwash are as follows:

Clay content	29 – 30% (average 29%)
Silt content	32 – 36% (average 34%)
Sand content	27 – 32% (average 30%)
Gravel content	4 – 10% (average 7%)
Plasticity Index	15 – 19% (average 17%)
Liquid Limit	37 – 43% (average 40%)
Linear Shrinkage	7 – 8% (average 7.5%)

These soils classify as “CL” in terms of the Unified Soil Classification and “A-6” or “A-7-6” in terms of the HRB Classification. They are “low” or “low to medium” in potential expansiveness when plotted on a standard “activity” diagram. In the compacted state the one sample of clayey hillwash that was tested classified as G9 in terms of TRH14, with a maximum dry density of 1796kg/m^3 and optimum moisture content of 19.3%. The maximum swell was 1.0%.

A rapid collapse test on a sample of the hillwash showed it to have a relatively high void ratio of 0.957 and a collapse potential, when saturated at 200kPa, of 8.46%.

A falling head permeability test undertaken on a sample of the clayey hillwash, that had been remoulded to 95% of modified AASHTO density, gave an average value of 6.4×10^{-9} m/s.

Chemical tests indicate that, when saturated, the clayey hillwash soils will be corrosive towards metal. They have a negative Langelier Index, a Ryznar Index of more than 7.5. However, the Final Aggressiveness Index is only 666, with the result that these soils will be mildly aggressive only to concrete.

5.3.2 Pebble Marker

The pebble marker occurs as a surficial horizon over the major part of the site, with the exception of the lower-lying parts that are underlain by diabase. It ranges in thickness from 0.3 – 1.0m (average 0.65m), but tends to be thinner within the area that is underlain by Rayton shale. The pebble marker comprizes abundant sub-angular to sub-rounded, highly weathered to slightly weathered, soft rock to hard rock, quartzite gravels, cobbles and boulders (generally <0.5m in diameter), densely packed (generally clast supported) in a matrix of grey-brown, orange, yellow-brown or red-brown silty sand. The consistency varies from medium dense to dense. Of note is the fact that it is weakly ferruginized in places (e.g. PB7, PB25 and bh11). Owing to its limited thickness and variable grading, no laboratory tests were undertaken on this horizon. However, it will improve the quality of the underlying residual tillite and shales when mixed with these materials during earthworks operations.

5.3.3 Residual Dwyka Tillite

Residual Dwyka tillite generally underlies the pebble marker and is frequently interbedded with, or contains lenses of, residual Dwyka shale (and occasionally siltstone). The upper 0.7 – 1.4m (average 1.0m) is often ferruginized and consists of yellow-brown, red-brown and black, stiff to very stiff (or dense to very dense) ferruginized sandy silt or silty sand, with scattered ferruginous concretions and occasional gravel to boulder size clasts. Below this ferruginized horizon the Dwyka tillite occurs to depths ranging from 2.5 – 13.25m (average 5.85m). It is somewhat variable in composition but generally consists of a blotched yellow-brown, red-brown, orange-brown, pale grey and buff, stiff or very stiff (or occasionally dense to very dense) sandy silt (or occasionally silty sand) with occasional to scattered soft to hard rock gravel to boulder size clasts (generally <0.2m diameter, but occasionally up to 1.0m diameter). In places, it comprises a finer grained clayey silt or clay-silt (e.g. bh45).

The index properties of the Dwyka tillite, excluding the more clayey material in bh45, are as follows:

Clay content	5 - 25% (average 14%)
Silt content	20 - 41% (average 33%)
Sand content	33 - 63% (average 41%)
Gravel content	0 - 34% (average 11%)
Plasticity Index	S.P. – 17% (average 10%)
Liquid Limit	S.P. - 35% (average 25%)
Linear Shrinkage	1.0 – 5.5% (average 3.5%)

In terms of the Unified Soil Classification these soils fall into the “CL”, “SC” or “SM” categories and are “A-4” or “A-6” materials in accordance with the HRB Classification. They are “low” in potential expansiveness when plotted on a standard “activity” diagram.

The more clayey material in bh45 contains 68% clay and 24% silt, with a plasticity index of 35. It is an “A-7-6” material in terms of the HRB Classification and plots on the boundary between “low” and “very high” in potential expansiveness on a standard “activity” diagram.

In the compacted state the less clayey residual tillites have maximum dry densities ranging from 1953 – 1986kg/m^3 (average 1972kg/m^3) and optimum moisture contents varying from 9.3 – 13.4% (11.2%). Maximum swells range from 0.2 – 0.9% (average 0.4%) and the soils generally classify as G7 in terms of TRH14.

Falling head permeability tests undertaken on four samples of residual tillite, remoulded to 95% of their Modified AASHTO maximum dry densities, ranged from $1.8 \times 10^{-7} – 7.2 \times 10^{-9}$ m/s (average 5.5×10^{-8} m/s).

In the undisturbed state the residual tillite is usually overconsolidated, with M_v values ranging from 0.48 to 1.28.

Triaxial testing of five undisturbed samples of residual tillite gave the following results:

Total stress	C	7 – 40kPa (average 25kPa)
	φ	15 – 39° (average 21°)
Effective stress	C’	13 – 71kpa (average 25kPa)
	φ’	21 – 34° (average 30°)

Triaxial and shear box tests carried out on samples of residual tillite that had been remoulded to 95% of their Modified AASHTO maximum dry densities, gave the following results:

Total stress	C	17 – 124kPa
	φ	21 – 34°
Effective stress	C’	18kPa
	φ	32°

Chemical tests indicate that, when saturated, the residual tillite soils will be corrosive to metals and aggressive towards set concrete. They have negative Langelier Indices, Ryznar Indices in excess of 7.5 and Final Aggressiveness Indices generally in excess of 1000.

5.3.4 Residual Dwyka Shale and Siltstone

Horizons of residual Dwyka shale (and occasionally siltstone) are interbedded with the residual tillite and vary in thickness from 1.0 – 6.3m. There are also numerous lenses and horizons of residual shale that occur within the underlying shale and tillite bedrock to depths of up to 25m. These vary in thickness from a few hundred millimetres to several metres (e.g. 2.7m in bh21 at a depth of 24.8m). Like the tillites the shales and siltstones are variable in composition. In general, however, they are banded and blotched yellow-brown, orange-brown, maroon, pinkish-brown and brown and consist of stiff to very stiff laminated and jointed clay-silts or sandy clay-silts. There are occasional gravel size clasts in places (dropstone shale).

The index properties of the residual Dwyka shales and siltstones are as follows:

Clay content	19 - 66% (average 37%)
Silt content	24 - 78% (average 45%)
Sand content	0 - 41% (average 12%)
Gravel content	0 - 16% (average 6%)
Plasticity Index	10 - 36 (average 19)
Liquid Limit	17 - 59% (average 38%)
Linear Shrinkage	4.5 – 9% (average 6.0%)

In accordance with the Unified Soil Classification the soils are generally "CL" materials, while in terms of the HRB Classification they are "A-6" or "A-7-6" materials. On a standard "activity" diagram the shales and siltstones generally plot as "low" in potential expansiveness. However, the more clayey horizons are "medium" or "high" in potential expansiveness. Typical samples of the more clayey residual shale were subjected to X-ray diffraction analysis, which showed the fine matrix to be dominated by quartz and kaolinite, with small quantities of smectite and interstratified illite/smectite present (maximum 4 %). This confirms our view that these residual soils are unlikely to experience significant swell/shrinkage with changes in moisture content.

In the undisturbed state these materials have fairly high dry densities and low void ratios.

In the compacted state the two samples tested both classified as G10 in terms of the TRH14 with maximum swells of 2.0 – 2.3%. Maximum dry densities range from 1861 – 1904kg/m^3 and optimum moisture contents vary from 13.8 – 16.3%.

Falling head permeability tests undertaken on two samples of residual shale, that had been recompacted to 95% of their modified AASHTO maximum dry densities, gave results of 6.8×10^{-9} m/s and 1.4×10^{-8} m/s respectively.

Triaxial testing of three undisturbed samples of residual shale gave the following results:

Total stress	C	33 – 253kPa (average 159kPa)
	φ	13 – 29° (average 20°)
Effective stress	C'	19 - 205kPa (average 122kPa)
	φ'	19 - 32° (average 27°)

Triaxial and shear box tests carried out on samples of residual shale, remoulded to 95% of their Modified AASHTO maximum dry densities, gave the following results:

Total stress	C	0 - 62kPa
	φ	15 – 20°
Effective stress	C'	4kPa
	φ'	30°

Chemical tests indicate that, when saturated, the residual shales and siltstones will be highly corrosive towards metal and highly aggressive to set concrete. They have negative Langelier Indices, Ryznar Indices in excess of 7.5 and Final Aggressiveness Indices in excess of 1000.

5.3.5 Clayey Residual Diabase

In proximity to the Rayton shale/diabase contact the upper 0.6 – 9.2m (average 2.3m) of residual diabase, where it underlies the hillwash, consists of red-brown (becoming orange-brown with depth) firm (becoming stiff with depth) sandy clay-silts/clayey silts with frequent ferruginous concretions in the top 0.5 – 1.0m. The index properties of these soils are as follows:

Clay content	13 - 32% (average 21%)
Silt content	35 - 60% (average 49%)
Sand content	13 - 39% (average 27%)
Gravel content	0 - 7% (average 2%)
Plasticity Index	11 - 21 (average 17)
Liquid Limit	40 - 53% (average 46%)
Linear Shrinkage	6.0 – 9.0% (average 8.0%)

These soils classify as either “CL” or “ML” in terms of the Unified Soil Classification and “A-7-5”, “A-6” or “A-7-6” in accordance with the HRB Classification. They plot as “low” or “medium” in potential expansiveness on a standard “activity” diagram.

They are characterized by low dry densities (1170 – 1395kg/m^3) and high void ratios (0.996 – 1.419), and are overconsolidated with M_v ranging from 0.86 – 2.59.

A rapid collapse test on a sample of the residual diabase showed it to have a collapse potential, when saturated at 200kPa, of 8.87%.

Compaction tests undertaken on a single sample of clayey residual diabase gave an optimum moisture content of 20.9%, a maximum dry density of 1680kg/m^3 and a maximum swell of 1.9%. In terms of TRH14 the clayey residual diabase classifies as a G10 material.

A falling head permeability test undertaken on a sample of the clayey residual diabase, after it had been recompacted to 95% of its modified AASHTO maximum dry density, gave an average value of 7.3×10^{-9} m/s.

Triaxial tests on three undisturbed samples of clayey residual diabase gave the following results:

Total stress	C	42 - 92kPa (average 45kPa)
	φ	6 - 17° (average 13°)
Effective stress	C’	16 – 37kPa (average 24kPa)
	φ’	19 - 33° (average 27°)

Chemical tests on a sample of the clayey residual diabase show that this material when saturated, is likely to be highly corrosive and aggressive to both metals and concrete. It has a high negative Langelier Index, Ryznar Index well in excess of 7.5 and Final Aggressiveness Index in excess of 1000.

5.3.6 Sandy Residual Diabase

At depth the clayey residual diabase, described in section 5.3.5 above, grades into a more sandy soil. This material, which ranges in thickness from 0.6 – 3.5m, consists of a yellow-brown speckled orange-brown and buff, dense or very dense, jointed silty sand or sandy silt. There are occasional hard rock weathering spheroids in places which may, in some cases, be in excess of 1.5m in diameter. The index properties of these soils are as follows:

Clay content	4%
Silt content	19%
Sand content	66 - 69%
Gravel content	8 - 11%
Plasticity Index	S.P. - 10
Liquid Limit	S.P. - 34
Linear Shrinkage	1.0 – 5.0

The sandy residual diabase is either an "SC" or "SM" material in accordance with the Unified Soil Classification and "A-2-4" or "A-2-6" in terms of the HRB Classification. It plots as "low" in potential expansiveness on a standard "activity" diagram.

A shear box test undertaken on a sample of sandy residual diabase, that had been remoulded to 100% of its in-situ density, gave peak strength values of C = 59kPa and $\varphi = 13°$. The angle of internal friction is considered to be too low for a material that is sandy in composition. This test result is, therefore, considered suspect.

5.3.7 Residual Rayton Shale

Over a small portion of the site Rayton shale bedrock is present either on surface or beneath a thin pebble marker horizon. Elsewhere the shale is weathered to residual soils that range in thickness from 0.2 – 4.0m (average 1.5m). The upper 0.2 – 1.0m frequently comprises angular, weathered shale gravels in a silty sand matrix. Beneath this, however, the residual soils generally consist of a yellow-brown or pinkish-brown, banded and blotched orange, red, grey and black, stiff or very stiff, laminated and jointed clay-silt or clayey silt.

The indicator properties of the silty residual shales are as follows:

Clay content	9 - 30%
Silt content	39 - 59%
Sand content	11 - 31%
Gravel content	0 - 22%
Plasticity Index	11 - 17
Liquid Limit	34 – 42%
Linear Shrinkage	4.5 – 7.0%

These soils classify as "CL" in terms of the Unified Soil Classification and "A-6" in terms of the HRB Classification. They plot as "low" in potential expansiveness on a standard "activity" diagram.

A falling head permeability test undertaken on a sample of the residual Rayton shale, after it was remoulded to 95% of its modified AASHTO maximum dry density, gave an average value of 3.8×10^{-8} m/s.

A shear box test undertaken on a remoulded sample of residual Rayton shale gave peak shear strength values of C = 0 kPa and $\varphi = 26^o$.

Chemical tests reveal that these materials, when saturated, will be highly corrosive and aggressive to metals and concrete. They have high negative Langelier Indices, Ryznar Indices in excess of 7.5 and Final Aggressiveness Indices of more than 1000.

5.4 Results of CPT Probes

A total of 15 CPT (super-heavy dynamic penetrometer) probes were driven adjacent to boreholes BH2, BH9, BH11, BH21, BH28, BH31, BH32, BH33, BH37, BH42, BH43, BH50, BH62 and BH64. The refusals at these are tabulated below and the full results are reproduced in Appendix B.

Borehole No.	Depth of Refusal (m)	Refusal Horizon
BH2	4.2	Very stiff residual tillite
BH9	8.7	Very soft rock diabase
BH11	4.8	Very soft rock Dwyka siltstone
BH13	1.2	Very soft rock Rayton shale
BH21	2.4	Very dense residual tillite
BH28	2.4	Very dense residual diabase
BH31	2.1	Very stiff residual Dwyka shale
BH32	2.7	Very stiff residual tillite
BH33	1.5	Very soft rock Rayton shale
BH37	1.8	Very stiff residual tillite
BH42	1.5	Very hard rock diabase
BH43	3.6	Very stiff residual tillite
BH50	2.7	Very stiff residual Dwyka siltstone
BH62	3.3	Stiff residual Rayton shale
BH64	2.7	Dense residual tillite *

* Refusal probably caused by boulder

Some individual records invite comment. Those from BH2 and BH11 reflect variations in consistency typical of residual soils overlying the Dwyka rocks, with consistencies ranging from stiff to very stiff down to the refusal at about 4 – 5m depth. BH9 in residual diabase had the deepest refusal at 8.7m; a peak in consistency in the very stiff range at 1.5m reflects desiccation/induration in the near-surface profile; this is followed by a significant thickness of material with a consistency of no more than firm.

6. CONSTRUCTION MATERIALS FOR THE TERRACE

6.1 **Use of On-Site Transported and Residual Soils**

The nature and properties of the soils occurring beneath the power island and switchyard terraces have been discussed in some detail in Section 5 above.

With few exceptions, samples of these soils classify with the CL category of the Unified Classification and are characterized by high silt contents, generally (but with a few exceptions) low Plasticity Indexes and, when compacted, by moderate Modified AASHTO maximum dry densities and relatively high Optimum Moisture Contents. While they will mostly excavate relatively easily from beneath the upper parts of the site, they are clearly not ideal materials with which to construct a major fill on its lower sections. However, we are of the view that, with homogenization by mixing, careful control of moisture content (insofar as weather conditions allow), and the use of high-energy compaction techniques accompanied by a rigorous method specification, a satisfactory result can be achieved. It must be emphasized, however, that, given the limitations of the local materials, the fill will not be capable of supporting more than lightly loaded structures without the occurrence of significant settlements. Special foundation treatment, such as the use of piles or the construction of engineered rockfill mattresses, will be required for the founding of most structures, except in those specific parts of the cut area where the excavated surface is underlain, at shallow to moderate depth, by rock of adequate consistency. This is discussed in Sections 9 and 10 of this report.

Three important issues must be raised in connection with the use of the <u>in situ</u> soils for construction of the fill:

i) Boulders with a wide range of sizes are present in the residuum derived from the weathering of the Dwyka bedrock. Many of these represent resistant rock types that have survived the formation of the deep weathering mantle that characterizes outcrops of Dwyka rocks in this area. Conventionally, the maximum size that should be incorporated within layers during the placement of a compacted fill is $^{2}/_{3}$ of the layer thickness. This is discussed further under 7.1 below.

   If 200mm layers are used in the course of placement using heavy vibratory rollers, boulders larger than 130mm would need to be bladed off the fill to spoil or for stockpiling to use in slope protection, prior to the compaction of each layer. Should impact compaction techniques be used, a greater layer thickness (up to 750mm) could be used; in this case we would, however, argue against the retention of boulders larger than about 250mm diameter as, if they protrude, they may damage the impact rolling equipment.

ii) When excavating the cut area the use of scrapers is advocated to ensure optimum mixing of the different materials present in the residual overburden. Scraper paths aligned from north-east to south-west are recommended, since the most prominent facies changes in the residual tillite appear to be orientated orthogonally to that direction.

iii) Because of the less-than-ideal characteristics of the available on-site materials, the compacted fill will be subject to settlement under light and moderate loads. We have assumed compaction of the fill to a uniform value of 95% of maximum Modified AASHTO density with <u>good control</u> to ensure that level of compaction in consistency achieved.

6.2 Imported Rock Aggregate

Rock aggregate is available from a number of sources within a radius of about 30km of the site. Felsite of the Rooiberg Group is quarried and crushed in proximity to the town of Witbank and has long been used as a source of concrete aggregate, ballast and road aggregate. Quartzite of the Rayton Formation was quarried at the Spitzkop Quarry, some 5.5km to the north of the power station site and immediately adjoining the N4 Freeway (see Figure 3). This material was used extensively as an aggregate for roads in the area. The existing quarry is no longer operating and was evidently abandoned because local reserves of good quality rock had been well-nigh exhausted and because water seepage was inundating the deeper levels of the pit. A new quarry, exploiting the same material, has been opened to the south of the town of Bronkhorstspruit. Figure 3 shows the Rayton quartzite outcrop to the north of the power station site. Its southern end crosses land to be occupied by Eskom for the present project. Three boreholes (bh 68 – 70) were drilled in this area, two slightly off, and down dip of the outcrop. Their logs are reproduced in Appendix A. In the outcrop area usable rock is present almost from surface, but at even short distances beyond its edge considerable thicknesses of weathered overburden are present above material of usable quality. A quarry in this area would therefore have to be narrow and cover a considerable strike distance (and be taken to substantial depth) to produce significant volumes of material. A volume of up to 2 000 000m^3 could possibly be obtained here from a quarry 1000m long and 10m deep, but not all of this volume would probably be usable. This area is also fairly low-lying and problems may be encountered with shallow seepage.

A larger and more prominent outcrop of the Rayton quartzite is present to the north of the defunct Spitzkop Quarry, north of the R104 (Figure 3). Here very much larger volumes of rock would be assured and the water table is likely to be deeper (>20m on the evidence of the Spitzkop Quarry). It is recommended that, should Eskom consider it advantageous to exploit its own local source of rock aggregate for this project, the availability of this area for purchase should be further investigated.

A bulk sample of the crushed rock was taken from an existing stockpile in the Spitzkop Quarry for laboratory testing. The results are reproduced in the separate volume of test results prepared by Messrs. Civilab. Visually, the aggregate is of excellent shape. Losses during the Los Angeles abrasion test were very low (7.2% after 200 revolutions), and were similarly very low after 20 cycles of sodium sulphate soundness testing (0.6%). Water absorption was 0.15%, which is likewise very low. In short, indications are that this material will be entirely suitable for use in the capping layers of the terraces, as a rockfill mattress, as railway ballast and a crusher-run base for road construction. It is also entirely suitable for use as coarse concrete aggregate: reference to the volume of test results presented separately by Civilab shows it to have negligible alkali reactivity and a value of 200 kN in the 10 % Fines Aggregate Crushing Test (the minimum value set by SANS for concrete subject to surface abrasion is 110 kN).

6.3 Conclusions and Recommendations

It is concluded that, while the natural materials available on site can be excavated with relative ease and compacted in layers to create the terrace for the power station and switchyard, the nature of the materials will require that excellent control of moisture content and fill placement methods will be necessary to ensure optimum long-term performance of the fill and to keep settlements to a minimum. In order to accelerate the earthworks contract as far as possible and to achieve design densities in a consistent manner, we recommend the use of impact rolling technology for construction of the fill. The application of this technology is discussed further under 7 below and in Appendix D.

7. CONSTRUCTION OF THE TERRACE

As was indicated above, this report is restricted to the area of the power island and the switchyard. Eskom's design engineers, the Black and Veatch Corporation, have set the preliminary surface level of the power island terrace at 1502.8m, and that for the switchyard platform at 1500.2m. The engineering design of these facilities is the responsibility of Black and Veatch, and the proposals offered below are in no way intended to usurp that role nor to influence decisions which may depend on information or engineering requirements to which we have not been privy.

7.1 **Placement of Fill and Capping Layer**

Prior to the commencement of fill placement the surface should be thoroughly compacted by impact rolling after the completion of scrubbing and clearing of the root layer. The fill could be placed in relatively thin layers, using conventional heavy vibratory rollers for compaction. Time savings and better compaction of the relatively poor materials to be used in the fill could, however, almost certainly be achieved by using impact compaction techniques which would allow the use of thicker layers. This method of compaction also has the advantage that, because of the high energy inputs involved, the desired level of compaction can be achieved at lower moisture content (say 2% less than optimum). This would, in turn, result in a drier fill, which would help decrease the duration of time-dependent settlements. A method specification has been prepared at our request by José Gil of Compaction Technology (Pty) Ltd., who have been instrumental in the development and application of impact compaction technology at an international level. This is reproduced as Appendix D.

In order to facilitate moisture control during fill placement it is imperative that measures be taken to control any seepage that may occur from the cuts upslope of the terrace and from the terrace surface. Such measures are discussed in the accompanying report prepared by Golder Associates. We recommend, also, that a system of permanent under-drains be installed across the terraces in areas of both cut and fill to keep the terrace surface dry by collecting near-surface seepage and infiltrating rainwater. These should terminate in a major interceptor fin drain immediately above the Rayton/diabase contact as discussed in 7.3 below. The installation of a berm or cutoff drain around the periphery of the cut, to prevent water flows down the cut face, and the provision of a cutoff drain along the length of its toe to preclude the development of hydrostatic uplift-forces in that zone, are additional requirements that are discussed in sections 7.2 and 7.3.

Construction of the underdrains must be phased; they would have to be installed first beneath the fill area, and only beneath that part of the surface of the terrace which is in cut after excavations have been completed. In this latter area the drains must be positioned so as not to clash with the foundations of structures to be built on the terrace. The placement of the capping layer can be carried out only after the drains have been installed. Falls along the drains of around 1% should be adequate for their effective functioning.

We understand from Black and Veatch that there is unlikely to be time to integrate this system of underdrains with the permanent deep drains which are to carry clean and dirty water off the terrace. The shallow under-drainage system should be designed by a suitably experienced civil engineer in relation to the fill permeability values given in the volume of test results accompanying this report and the likely seepage rates determined by Golder Associates.

We assume that, as at Medupi, a 300mm capping layer will be used. Crusher run of Rayton quartzite would be entirely suitable for this purpose. Because of the thinness and vulnerability of this capping, we recommend that it be placed conventionally using heavy vibratory rollers.

7.2 Cut Slope Stability

The results from shear box and triaxial tests undertaken on undisturbed and remoulded samples of the regolith are summarized in Table 4. Using these results, stability analyses have been undertaken for a 15m deep cutting in the tillite and a 15 m thick fill overlying residual diabase. At the outset, however, we recommend these analyses be revisited once the terrace levels and positions of cuts have been finalized.

7.2.1 Cutting in the tillite

Based on the laboratory shear strength parameters, the residual tillite has been divided into an upper and lower horizon. The surficial horizon has been taken down to a depth of approximately 6m, before grading into more competent residual tillite with better shear strength parameters. For the sake of our analysis, the residual tillite has been taken down to a maximum depth of 10m, before grading into very soft to soft rock tillite.

Using the results summarized in Table 4 as input into SLOPBG[1] (*) – a computer modelling program using Bishop's Modified slope stability method – various cutting configurations have been evaluated, the results of which are present in Figures 4 and 5, and summarised in Table 5.

Cognisance must be taken of the following:

- The residuum is highly erodible. Should seepage from perched groundwater tables be allowed to daylight from the excavation face, the egressing groundwater will induce piping erosion in the regolith and sloughing of the excavation face, which in the long term will reduce the stability of the cutting locally. Thus, unless measures are implemented to intercept and divert the percolating groundwater, increased maintenance of the slopes will be required. Furthermore, the presence of a hydrostatic head near the cutting will reduce the overall stability (Figure 4).
- A berm or drain is required along the top of the cutting to prevent, as far as possible, surface flow down the face. (The drain must be well maintained). It is recommended also that slopes flatter than approximately 25° be vegetated as soon as possible after the earthworks.
- A cutoff drain must be installed along the toe of the cutting to ensure that hydrostatic uplift forces are not generated in this zone.
- Pipe drains should be provided at intervals along the cut face, as discussed in the report by Golder Associates.
-

[1] (*) SLOPBG – Bishop's Slope Stability Analysis, Ver. W2. Prokon Software

7.3 Stability and Settlement of the Fill

7.3.1 Stability of engineered fill overlying residual diabase

In assessing the stability of the proposed fill, a worst case scenario has been evaluated. To this end a 15m high engineered fill has been modelled overlying up to 6m of residual diabase, which in turn overlies diabase bedrock. The scenarios that have been evaluated include an engineered fill with two different slopes, overlying firm to stiff residual diabase. A further analysis has been undertaken with the provision of stone columns, installed using Dynamic Consolidation equipment, along the toe of the embankment to improve the shear strength of the residual diabase. The spacing of these columns should be decided by a specialist contractor (although for budgetary purposes two rows staggered with column spacing at 4m may be assumed).

Using the results in the Table 4 as input into SLOPBG, the scenarios presented above have been modelled and the results are presented graphically in Figures 6a, 6b and 7, and summarized in Table 5.

Additional considerations to be borne in mind include:

- To prevent seepage into the regolith beneath the fill, it is recommended that a fin drain be installed along the upslope contact of the diabase sill with Rayton shale. The drain must be taken down to the bedrock diabase, and be designed to intercept and divert surficial percolating groundwater.
- The stability analysis makes no allowance for loads near the edge of the fill, and site specific analyses will be required once the layout of structures has been finalized; as a rule of thumb, structures should be positioned at least the height of the fill away from the embankment edge.
- A subsoil drain must be installed along the base of the embankment to prevent hydrostatic uplift forces being generated near the toe.

The consolidometer test results are summarized in Table 6.

7.3.2 Settlement of Fill

7.3.2.1 Settlement of underlying residual diabase

An assessment of the magnitude of the settlement of the residual diabase due to the stress imposed by a 20 m high fill has been carried out based on the following:

- The results of the consolidation and index property tests carried out on the residual diabase (PB18 at 4.2m and bh41 at 3.19 to 3.48m).
- The results of the SPT tests carried out in bh41.

The compressibility characteristics obtained from the above results may be summarized as follows:

Position	Depth (m)	Modulus of Compressibility (kPa)	Source
bh41	1.5	12250	SPT
bh41	3.2	18400	Consolidometer
bh41	3.5	16250	SPT
PB18	4.2	28000	Consolidometer
bh41	6.5	48600	SPT

Based on these results the following settlement predictions are considered to be relevant to the residual diabase due to the stress imposed by 20 m of fill:

Thickness of Residual Diabase (m)	Predicted Settlement (mm)
4	94
6	113
8	125

An assessment has also been carried out with regard to the feasibility of using dynamic compaction techniques to improve the compressibility characteristics of the residual diabase and in this way reduce the predicted settlement. The laboratory test results that have been used for this assessment are summarized in the Table below:

Position	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Moisture Content	Degree of Saturation
PB18	4.2	53	32	21	35.4%	92.6%
PB18	4.2				36.4%	88.3%
bh41	3.2	49	29	20	47.7%	96.4%

In terms of dynamic compaction the following conclusions can be reached from these test results:

- With the relatively high plasticity index and high degree of saturation there will be significant development of pore water pressure in these residual diabase soils during the dynamic compaction process. This will significantly affect the overall process and will probably require the installation of suitable instrumentation for the monitoring of pore water pressure to enable appropriate programming of the works to be carried out.
- Previous experience has shown that when the moisture content of the soils is higher than the plastic limit, as is the case with these soils, then the soils generally deform plastically under impact without densification.

Based on these conclusions, we do not believe that dynamic compaction will be effective in these residual diabase soils. If it is considered necessary to improve this horizon, then the best procedure would be to use dynamic replacement techniques rather than dynamic compaction. This essentially comprizes the installation of closely spaced stone columns into the overall residual diabase mass. These columns are installed using the same equipment that is used for the dynamic compaction process, as discussed under 7.3.1. above.

7.3.2.2 Settlement of the overall fill mass

The results of the consolidometer tests on selected compacted samples of the material that will be used for fill have been used to evaluate the compressibility characteristics of the fill. These compressibility characteristics have been used in a series of analyses to determine the magnitude of settlement that could occur within the fill under self imposed stresses. The modulus of compressibility values determined from the consolidometer tests are given in the Table Below:

Position	Depth (m)	Description	USCS Class.	Modulus of Compressibility (kPa)
PB1	1.0 to 5.0	Residual Dwyka tillite	SC	16700
PB4	1.25 to 4.5	Residual Dwyka tillite	SM	23500
PB7	1.5 to 4.1	Residual Dwyka shale	CL	8400
PB15	1.8 to 4.8	Residual Dwyka shale	CL	12000
PB23	1.4 to 4.3	Residual Dwyka tillite	CL	14250

Using these modulus of compressibility values an estimated settlement of 150 mm to 200 mm is arrived at for a 20 m high fill.

The results of the consolidometer tests, coupled with the results of permeability tests carried out on selected compacted samples, have been used to attempt to predict the time rate of settlement of the fill. As is typical for an analysis of this nature a wide range of times are obtained for consolidation to take place. These are mainly dependant on the interpretation of the laboratory test results and the assumptions made with regard to the dissipation of excess pore pressures that could develop within the mass of fill under self imposed stress conditions. Typical values obtained from these procedures vary from one week to six months for 90% consolidation to take place. As is usual for this type of assessment it is necessary to apply a significant degree of engineering judgement to the overall assessment of the time rate of settlement. Our general assessment is that 75% of the settlement will take place within 2 to 3 months after completion of the construction of the fill. A further 4 to 6 months will be required for settlement to be effectively complete.

It is well known that high fills such as these are prone to long term creep settlement. This is a phenomenon that cannot be predicted with any certainty using conventional testing and analytical procedures. The risk of long term creep settlement can however be reduced by specifying high compaction densities and the implementation of good quality control procedures to ensure that the specifications are met.

7.4 **Protection of the Cut and Fill Slopes**

The cut slopes and exposed sides of the fill will be highly susceptible to erosion because of the silty/clayey nature of the exposed materials. Appropriate permanent protection must therefore be provided immediately after the completion of the earthworks. A combination of geotextile covering and packed rock would be appropriate for this purpose. Boulders bladed off the terrace during fill placement, or rip-rap produced by crushing the Rayton quartzite could be used for this purpose. The addition of a soil layer to permit vegetation to establish on these slopes should also be considered. Detailed designs of this protection should be prepared by a suitably qualified civil engineer.

7.5 Deep Drains

Black and Veatch presently propose that these permanent conduits should carry three sets of pipes across the terrace. At the time of writing, we are still not in possession of any details on the positioning and depth of these drains. If they extend many metres below the terrace surface hard rock may be encountered over part of their length. Stability of the overlying softer rocks and residual soils has already been considered under 7.2 above. Other important factors will be the occurrence of seepage from the sides of the drains during excavation, and the control of pore water pressures after the drains have been lined. A separate analysis should be carried out once details of the drains are known. We draw attention to potential problems arising from damage to the rock mass should it be decided to excavate the drains using explosives.

8. ON-SITE MONITORING OF THE TERRACE DURING AND AFTER ITS CONSTRUCTION

Ongoing monitoring during the construction programme must be undertaken to ensure (a) that design specifications are met in respect of *in situ* compaction and the construction of the fill, and (b) that settlements experienced in the fill during construction accord with those predicted and are within the absolute limits that can be tolerated. Regular inspection of the cut-face above the terrace during the bulk earthworks contract will also be necessary in order to modify the drainage design in the light of local seepage patterns.

For the *in situ* compaction of all soils, and during impact compaction operations, levelling must be carried out to determine the extent of surface lowering. The measurement of *in situ* density in test pits excavated for the purpose, and the execution of Dropweight Cone Penetrometer and plate-bearing tests to confirm that the level of compaction achieved accords with design specifications, are other important requirements. The need for *in situ* testing throughout terrace construction is emphasized, since this will provide early warning of any need to modify compaction specifications or layer thicknesses.

The monitoring of flow from all permanent drains is important both during construction and, thereafter, during the life of the project, to ensure that the integrity of the drainage systems is being maintained and that blockages do not occur. In the event of such problems, facilities should be in place to institute remedial measures with the minimum of delay.

9. FOUNDATIONS

A variety of foundation types will be used to support the diverse range of structures making up the power station complex. A summary of foundation types, loadings and performance criteria for key structural elements, supplied to us by Eskom, is provided in Table 7. It is not the intention of this report – nor was it a requirement of Eskom's terms of reference for this project – that analyses sufficient to permit the design of individual foundations be carried out. Information necessary for this to be done was, in any case, not made available to us prior to the compilation of this report.

In the light of the results obtained in the course of the drilling, pitting and testing programmes presented in foregoing sections and in Appendixes E and F and the accompanying volume of test results, a broad set of foundation solutions was adopted as a framework within which to carry out further analyses. Plan 0.90/35 presents a general classification of the site in terms of depth to various classes of bedrock as a guide to the evaluation of these options.

- Solution 1 (some elements within the central part of the power island): Found on rock of "soft rock" consistency (or better) at shallow to intermediate depth below terrace level.
- Solution 2 (structures with heavy and moderate foundation pressures in areas where suitable founding is deeper than about 4 metres): Use piled foundations.
- Solution 3 (lightly loaded structures and structures not unduly sensitive to settlement): Increase thickness of gravel capping layer over terrace to form a load-bearing rockfill mattress.

Solutions 2 and 3 are not mutually exclusive as cost considerations and economies of scale or magnitude of potential settlement may, in some instances make piling a competitive option to founding on an engineered mattress, even in the case of less heavily loaded structures. The optimum solution must necessarily be determined in each case in the light of detailed design and performance criteria, which were not available to us at the time of writing. Tables 12 and 13 provide an assessment, based on present information, of the likely suitability of these different solutions for some key structures.

In formulating the various generic solutions cognisance has been taken, also, of the results of water pressure tests carried out in 14 representative boreholes, as recorded in Appendix A. These tests were specified in order to determine whether grout injection might result in any significant improvement in rock mass properties beneath the site. Significant water takes (in excess of 1 Unit Lugeon) were recorded in only half of the boreholes; in the remainder most of the takes occurred in the zone of deep weathering which will be removed to form the terrace over a significant part of the site. For the most part, takes were very localized and associated with zones of fracturing/weathering of very limited vertical extent. We conclude, therefore, that no useful consolidating effect and increase in rock mass strength will be achieved by grout injection beneath foundations.

The different founding possibilities identified above will be discussed on a case-by-case basis for some key structures in the power station complex in the sections that follow. It should be noted that in Tables 12 – 14 and the associated text certain boreholes have been selected for analysis because they are considered representative of conditions in the vicinity of the structure concerned. The same boreholes have not necessarily been included in each table. Because of the problem referred to in 4 above not all of these boreholes are located directly beneath the structure under analysis.

9.1 **Founding on Rock**

Structures near the centre of the power island complex, including a large part of the turbine hall (including most of the Turbines and part of the Auxiliary Bay) are located in areas underlain, mainly at depths of 0 – 4 metres below terrace level, by rock of soft rock consistency. The area concerned is generally delineated, by interpolation between boreholes, on Plan 0.90/35. It must be pointed out that, in the case of the Turbines, further information, in the form of moduli determined by continuous surface wave analysis, can only be supplied once the terrace has been constructed, since the depth of penetration of Rayleigh waves is too small to allow this form of testing to be carried out meaningfully beneath the existing natural land surface at the present site. These tests will be carried out and reported upon once the terrace has been excavated to final level.

In analyzing the borehole and Goodman Jack records in the context of foundations placed directly on rock, account must be taken of the layered nature of the rock mass, especially in the case of the Karoo rocks in which softer horizons frequently underlie harder layers. This phenomenon is clearly evident from our inspection of the results presented in Appendixes A and E. As an aid to the prediction of ranges of likely settlement beneath foundations of all categories, values of Young's Modulus and Poisson's Ratio have been tabulated for rocks and residual soils of different types and consistency categories in Tables 8 – 11 (the strength ranges on which these categories are based are given in the Key to Descriptors which precedes the borehole logs in Appendix A).

Special mention must be made of that area of the site underlain by tillite and shale (and subordinate siltstone and sandstone) of the Dwyka Group. This area is in cut, resulting in the exposure of residual soils or tillite/shale rock at terrace level. A number of foundation options, depending on the structural loading and the depth to a suitable founding horizon for specific structures, can be considered for this part of the site. Complex and difficult geological/geotechnical conditions occur in this area in terms of the design and construction of piled foundations. These may be summarized as follows:

- The presence of boulders within the residual soil horizons (mainly the residual tillite)
- Highly variable founding conditions, with very soft rock or soft rock horizons being underlain by stiff or very stiff residual soils
- The presence of a high water table (essentially at terrace level over large areas)

9.1.1 Settlement analysis for selected structures when placed directly on rock or on rockfill foundations

Since the results from the laboratory and *in situ* tests are summarized in this report, they are not repeated here and the reader should refer to the summary tables as necessary.

To predict settlements for the more sensitive and/or heavily-loaded structures, the *in situ* and laboratory tests have been used as input into a commercially available software package – *Roclab* (Rocscience). The output from this program has facilitated the evaluation of the test results and provided interpolated rock mass moduli for each horizon in those boreholes used in our analysis (Hoek, E. and Diederichs, M.S. 2006. *Empirical Estimation of Rock Mass Modulus*, International Journal of Rock Mechanics and Mining Sciences, **43**, 203-213). Furthermore, to categorize the lithologies for use in a commercially available foundation settlement program, the materials have also been classified in terms of the Geological Strength Index (GSI) (Marinos, V., Marinos, P. and Hoek, E. 2005. *The Geological Strength Index: Applications and Limitations*. Bull. Eng. Geolgo. Environ. **64**, 55-65). The data thus generated have been used as input into Settle3D (Rocscience), a 3-dimensional program for the analysis of consolidation and settlement under foundations. The results of the analyses are presented in **Figures 8.1 – 11** and summarized in **Table 14**. To provide transparency in the calculations, the GSI and rock mass modulus allocated to each horizon have been included in the legend for each plot, e.g. "sr Dwyka shale (55/640)" = soft rock Dwyka shale with GSI of 55 and a bulk modulus of 640 MPa.

The following generalization has also been assumed in the settlement calculations, where applicable:

- The rock mass classification for horizons at depth will be no worse than the deepest horizon encountered in the relevant borehole.

10 . THE USE OF PILED FOUNDATIONS

In areas where sufficiently good rock is present at too great a depth to permit economic founding of heavy structures on rock using open excavations, and in the case of those less heavily loaded structures which will be vulnerable to likely settlements arising from the use of rockfill mattresses, piled foundations should be used.

The use of this option is discussed in the ensuing sections in relation to the options presented in 9 above. The sections are subdivided in terms of bedrock type.

10.1 **Dwyka tillite and shale**

10.1.1 Heavily loaded structures

In view of the complex and difficult conditions with regard to the design and construction of piled foundations it is recommended that, for heavily loaded structures, the first consideration should be to found these structures on the very soft rock/soft rock tillite/shale using conventional spread foundations or raft foundations. Only in areas where it is not practical or economical to excavate to very soft rock or soft rock should piled foundations be considered. A detailed cost comparison needs to be carried out to determine the cut off excavation depth from which a piled foundation will be preferable to a spread or raft foundation. It is considered likely that this depth will be between 3m and 4m. The depth will be dependent on the nature of the structure and the type of foundation. For example, for a heavily loaded raft foundation where the piled alternative would require a large group of high capacity piles with a deep pile cap the excavation, cut off depth would be greater than for a single spread foundation where the piled alternative is a single pile with a nominal pile cap.

Where it is necessary to adopt a piled foundation solution the most suitable pile type would be a large diameter augered or bored cast-in-situ piles. These piles are generally installed with diameters varying from 600mm to 1500mm. Working loads would typically vary from about 1500 kN (typical for a 600mm pile) to about 9000 kN (typical for a 1500mm diameter pile). As indicated above, complex conditions exist in this area in terms of the design and construction of piled foundations. The following general design and construction recommendations are considered relevant in this regard:

- A high capacity piling rig will be required (minimum torque of 15 ton metres). This will ensure that smaller boulders and lenses of rock up to about soft rock hardness can be penetrated without resorting to other specialist techniques (chiseling, coring etc). Specialist techniques will, in any event, be required when large boulders are encountered.

- Owing to the presence of a high water table either temporary casings will be required or the pile will have to be drilled under a head of drilling fluid (either bentonite or a polymer) to ensure stability of the pile excavation.

- In view of the variable founding conditions (very soft rock and soft rock underlain by residual soils) proof boreholes should be allowed for to confirm the founding horizon. This will vary from a selected number of boreholes for a group of piles to individual boreholes for highly loaded individual piles.

- The piles should be designed to carry vertical compressive loads in end bearing on soft rock (or better) tillite/shale encountered at refusal of a high capacity piling rig. Where the very soft rock or soft rock has been penetrated to a sufficient depth the piles could be designed with a combination of end bearing and side shear in a rock socket. For design purposes it should be assumed that refusal of a high capacity piling rig will be on soft rock tillite or shale. A typical allowable end bearing stress of 5 MPa would be applicable to the soft rock tillite/shale. Typical allowable working side shear values of 200 kPa and 350 kPa would be applicable to the very soft rock and soft rock tillite/shale respectively.

- Since the piles will be designed for end bearing on rock encountered at refusal of the piling rig suitable techniques must be adopted to ensure that the base of the pile is adequately cleaned of all drilling spoil. Typical acceptable techniques would air lifting and suction bailing. Under these conditions the use of a cleaning bucket is not considered acceptable.

- In instances where the design requires reasonable predictions of pile settlement then calculations should be based on elastic solutions using modulus values of 150 MPa and 400 MPa for the very soft rock and soft rock tillite/shale respectively.

- Normal South African design practice limits stress on the pile shaft to a maximum of 25% of the 28 day concrete cube strength. Nominal pile reinforcement is usually 0.8% of the pile cross sectional area.

10.1.2 Moderately loaded structures

Whenever possible moderately loaded structures should be designed with spread or raft foundations on the residual soils or very soft rock tillite/shale.

Where it is necessary to adopt a piled foundation solution then a driven displacement cast in situ (DDCIS) pile is considered the most suitable pile type. This pile type can be installed with diameters of 350mm, 410mm, 520mm and 600mm with typical working loads varying from 500 kN (350mm diameter) to 1800 kN (600mm diameter). The procedure for installing this pile essentially comprises the bottom driving of a temporary casing plugged with a layer of dry mix concrete within the casing. Once a set is reached the plug is expelled and a bulbous base is formed using dry mix concrete. The reinforcing cage is placed within the casing which is then filled with a high slump concrete. The rig then extracts the casing. The advantages of this pile are as follows:

- The plug and the subsequent bulbous base will ensure that no water can enter the shaft. The high water table conditions will therefore not present any problems.

- The nature of the installation techniques, in particular the forming of an enlarged base, are such that the pile load carrying capacity is not unfavorably influenced by the variable soil/rock conditions and the presence of boulders within stiff residual soil horizons. It will be quite acceptable to form the bulbous base on a boulder in stiff residual soil.

The general design recommendations given in Section 10.1.1 for allowable concrete stress and nominal reinforcement are also applicable to the design of the DDCIS piles.

Augered or bored cast in situ piles will also be suitable for moderately loaded structures. The recommendations for design and construction given in Section 10.1.1 will then also be applicable.

10.2 Rayton shale

The major part of this area is in cut or shallow fill. The fills increase in height towards the south east. In this area shale rock is overlain by a relatively thin layer of transported and residual soils. In areas of cut or shallow fill shale rock will be at or very close to surface. It is therefore apparent that in this area of the site piled foundations will only be appropriate in a few areas of deep fill and/or deep residual soils.

In general terms, the soil and rock profile in this area results in relatively straightforward conditions in terms of the design and construction of piled foundations. Drilling will be carried out through fill, transported and residual soils onto the shale. Drilling through the fill should not present any problems since the fill specification requires that all large boulders in the fill be removed during the fill placement. Very soft rock shale will be penetrated by high capacity piling rigs and drilling will continue until soft or moderately hard shale is encountered. In certain areas moderately hard shale will be encountered immediately below the fill or transported/residual soils. A further advantage with regard to the use of piled foundations is that the water table is below rock head level.

It is anticipated however that the majority of the heavily and moderately loaded structures in this area will be founded on the shale rock (heavily loaded and moderately loaded) or residual shale (moderately loaded) using conventional spread or raft foundations. For certain structures the depth to rock and/or residual soil may be such that it will be more practical and economical to adopt a piled foundation solution. As for the area underlain by Dwyka tillite and shale a detailed cost comparison needs to be carried out to determine the cut off excavation depth from which a piled foundation will be preferable to a spread or raft foundation. In view of the straightforward piling conditions in this area this depth will probably be shallower than the area underlain by Dwyka tillite and shale.

Where it is necessary to adopt a piled foundation solution the most suitable pile type would be an augered or bored cast in situ pile. These piles are generally installed with diameters varying from 600mm to 1500mm. Working loads would typically vary from about 1500 kN (typical for a 600mm pile) to about 9000 kN (typical for a 1500mm diameter pile).

The following general recommendations are applicable to the design and installation of large diameter auger or bored piled foundations in this area:

- A high capacity piling rig will be required (minimum torque of 15 ton metres). This will ensure that the very soft rock shale can be penetrated and that some degree of penetration is achieved into the soft rock and moderately hard rock shale. Piles should be drilled to refusal of a high capacity piling rig.

- Piles should be designed in end bearing only unless it can be established that a sufficient rock socket has been achieve to develop side shear. For design purposes it should be assumed that end bearing is on soft rock shale. A typical allowable end bearing stress of 7.5 MPa would be applicable to the soft rock shale.

- In instances where the design requires reasonable predictions of pile settlement then calculations should be based on elastic solutions using modulus values of 400 MPa for the soft rock shale.

- Since the piles will be designed for end bearing on rock encountered at refusal of the piling rig suitable techniques must be adopted to ensure that the base of the pile is adequately cleaned of all drilling spoil. Hand cleaning would be the preferred method. The use of cleaning buckets is not considered to be acceptable.

- In certain areas abrupt refusal of the piling rig could occur immediately below the fill and transported soils on moderately hard rock (or better) shale. This will not present a problem for vertical compressive loads but may be problematic with short piles where lateral soil resistance is required to resist high moments or shear forces applied at the pile head. Under these circumstances specialized techniques (coring and chiseling) will have to be adopted to form a socket into the shale to resist the moments and/or the shear forces applied at the pile head. A similar problem will occur if piles are required to resist tension loads. Specialized techniques (coring and chiseling) could also be used to form a socket to resist tension loads. Alternatively, the piles could be cast with a selected number of sleeves within the shaft and tension dowels installed through the sleeves into the underlying rock using rotary percussion drilling techniques.

- In areas where the piles are drilled through fill it will be necessary to make a design allowance for additional vertical compressive load due to negative skin friction associated with the long term consolidation of the fill. The effective shear strength parameters given in sections 5.3.3 to 5.3.7 of this report should be used to calculate the value of negative skin friction that needs to be taken into account.

- The general design recommendations given in Section 1.1 for allowable concrete stress and nominal reinforcement are also applicable to the design of these piles.

10.3 Diabase

In this area fill of between about 5m and 12m will be placed to form the terrace. Piled foundations will be required for all structures placed in this area. Piling through the fill should not present any problems since the fill specification requires that all large boulders in the fill be removed during the fill placement. The main problem in terms of pile installation is the presence of hard rock core-stones within the soil profile, specifically within the residual diabase, underlying the fill. These core stones generally seem to be less than 600mm in diameter, although in isolated instances core stones of up to 2m in diameter have been observed. An advantage for the installation of piled foundations in this area is that the water table is below rock head level.

10.3.1 Heavily loaded structures

For heavily loaded structures the most suitable pile type would be large diameter augered or bored cast in situ piles. These piles are generally installed with diameters varying from 600mm to 1500mm. Working loads would typically vary from about 1500 kN (typical for a 600mm pile) to about 9000 kN (typical for a 1500mm diameter pile). The following general recommendations are applicable to the design and installation of large diameter auger or bored piled foundations in this area.

- A high capacity piling rig will be required (minimum torque of 15 ton meters). This will ensure that smaller core-stones can be penetrated without resorting to other specialist techniques (chiseling, coring etc). Specialist techniques will in any event be required when large core-stones are encountered. Temporary casings will probably be required in instances where chiseling is carried out.

- In this area there is generally an abrupt transition from the residual soils to hard rock diabase. Refusal of the piling rig will occur on this hard rock diabase. It will probably not be practical to penetrate the hard rock to any significant depth, even using specialist techniques (coring and chiseling). Piles should therefore be drilled to refusal of a high capacity piling rig and designed to carry vertical compressive loads in end bearing. High allowable end bearing stresses (greater than 10 MPa) would be applicable to the hard rock diabase and in practical terms the sizing of piles will probably be controlled by the allowable concrete stress rather than by the allowable end bearing.

- In instances where the design requires reasonable predictions of pile settlement then calculations should be based on elastic solutions using a modulus value of 1000 MPa for the hard rock diabase.

- Since the piles will be designed for end bearing on rock encountered at refusal of the piling rig suitable techniques must be adopted to ensure that the base of the pile is adequately cleaned of all drilling spoil. Hand cleaning would be the preferred method. The use of cleaning buckets is not considered to be acceptable.

- As indicated above it is anticipated that over the major part of this area abrupt refusal of the piling rig will occur immediately below the fill and transported soils on hard rock diabase. This may be problematic with short piles where lateral soil resistance is required to resist high moments or shear forces applied at the pile head. A similar problem will occur if piles are required to resist tension loads. The nature of the hard rock diabase is such that even with specialized techniques (coring and chiseling) it may not be practical to form a socket to resist moments and lateral forces or tension forces. Under these circumstances the piles should be cast with a selected number of sleeves within the shaft and shear and/or tension dowels installed through the sleeves into the underlying rock using rotary percussion drilling techniques.

- The piles will be drilled through fill of variable thickness. It will therefore be necessary to make a design allowance for additional vertical compressive load due to negative skin friction associated with the long term consolidation of the fill. The effective shear strength parameters given in sections 5.3.3 to 5.3.7 of this report should be used to calculate the values of negative skin friction that need to be allowed for.

- The general design recommendations given in Section 10.1.1 for allowable concrete stress and nominal reinforcement are also applicable to the design of these piles.

10.3.2 Moderately loaded structures

For moderately loaded structures consideration should be given to the use of driven displacement cast-in-situ (DDCIS) piles as described in Section 10.1.2. The advantage of this pile type is that it will not be necessary to penetrate the core-stones within the residual soils. It will be quite acceptable to form the bulbous base on core-stones which are generally present in dense or very dense residual diabase. A disadvantage of this pile type is that in areas of deep fill the negative skin friction allowance could be a large percentage of the total allowable vertical compressive load. As a result there could be a limited vertical load capacity available to carry imposed structural loads.

Augered or bored cast in situ piles will also be suitable for moderately loaded structures. The recommendations for design and construction given in Section 10.3.1 will then also be applicable.

10.4 Quality Assurance

It is essential that a rigorous quality assurance programme be implemented on site during the installation of piled foundations. It is normal to back up such a programme with pile load tests and pile integrity tests.

With large diameter auger piles founded on rock and DDCIS piles the application of a rigorous quality assurance programme during pile installation will almost always ensure that design requirement will be achieved in terms of load transfer to the residual soils or rock. In view of this it is recommended that full scale pile load tests on trial or working piles be kept to the minimum acceptable number for a project of this type.

A more important requirement is to ensure that a suitable testing programme is in place to carry out control checks on the following aspects:

- The interface between the pile concrete and the underlying rock to confirm that end bearing has not been compromised by inadequate cleaning of the pile base. This requires that tubes be cast into the shaft of selected piles (or all high capacity auger or bore piles) to a level of about 1m above the base of the pile. Rotary core drilling is then carried out to check the concrete/rock interface.

- A detailed testing programme needs to be put in place to check the integrity of the concrete in the pile shafts. Sonic coring testing techniques are recommended for the large diameter piles. This requires that a minimum of three tubes be cast into the pile to carry out the testing. One of these tubes can also be used to carry out rotary core drilling to check the concrete/rock interface as described above. Sonic impact tests are recommended for smaller diameter piles.