

JOHANNESBURG DEVELOPMENT AGENCY

**NEWTOWN CULTURAL PRECINCT
SUPER-BASEMENT PROJECT
Ground Investigation Report**

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1.0 INTRODUCTION

Geotechnics Africa (Pty) Ltd. have been appointed by GAPP Architects and Urban Designers, on behalf of their Client, Johannesburg Development Agency, to undertake the ground investigation for the proposed Super-Basement Project within the Newtown Cultural Precinct.

This report describes the ground investigation carried out on the site, compares the findings of this investigation with data collected from surrounding sites and proposes a geological model for the site. The data is evaluated with regard to its geotechnical significance and makes general recommendations for the excavation for the proposed basement and the lateral support, as well as for dewatering, the design of the foundations and subgrade preparation.

2.0 THE SITE

The site of the proposed Super-Basement Project forms a portion of the site formerly occupied by the Johannesburg Power Station. The site investigated is bounded on the north by Jeppe Street, on the east by Bezuidenhout Street Extension, on the south by President Street and on the west by Goch Street.

The cooling towers of the former Power Station have been demolished but many of the ancillary buildings have been converted for other uses and the open space within the site has been landscaped.

The Mary Fitzgerald Square lies to the north of Jeppe Street and further to the north there is the Africana Museum, which occupies the former Johannesburg Market Hall.

The elevated section of the M1 North Motorway runs along Goch Street and the South African Breweries Centenary Building occupies another section of the former Power Station site to the east of Bezuidenhout Street Extension and immediately south of President Street.

There is an overall crossfall of approximately 6m on the site from the high point approximately parallel to the President Street boundary towards the Jeppe Street boundary on the north.

The area around the Market Buildings, now the Africana Museum, which includes the Mary Fitzgerald Square and the northern portion of the Power Station site, was originally known as the Brickfields and is marked as such on Tomkins Map (1890). Subsequently, the area was called Burgersdorp and early in the last century became known as Newtown.

The area originally consisted of marshy ground with plots for brick making. The area being populated mainly by poor people who earned their living from brick making and lived in shanties. This first informal settlement was surveyed and streets laid out in 1895, the brief to the surveyors being that the streets be established between the existing houses.

3.0 PROPOSED DEVELOPMENT

The proposed development will comprise a Super-Basement covering the total site with separate medium rise buildings above ground level with open landscaped concourse areas between.

4.0 THE INVESTIGATION

4.1 Field Investigation

The field investigation was constrained by the limitations of access to significant areas of the site as a result of the existing buildings, as well as by the existing services and the remnants of previous structures, which prevented drilling of holes in many other positions.

The fieldwork carried out consisted of the drilling of twenty-two large diameter (750mm) trial holes, which were drilled with a Williams LDH80 auger machine hired from ESOR (Pty) Ltd. between 25th and 27th March 2003.

The trial holes were inspected by an Engineering Geologist and the soil and rock strata exposed described using standard terminology (Ref. Jennings et al, 1973 and AEG Core Logging Committee, 1976). In situ crosshole jacking tests were carried out at selected positions within the trial holes and both representative disturbed and undisturbed soil samples taken from the sides of the trial holes for laboratory testing. In addition, two samples of groundwater were taken for chemical testing.

The detailed descriptions of the trial holes have been recorded on standard sheets, copies of which are attached to this report in Appendix A.

4.2 **Desk Study**

The Geological Map of Johannesburg at a scale of 1:5000 as compiled by J.H. de Beer in 1965 shows the majority of the Power Station site to be underlain by the Ventersdorp Andesitic Lava with a faulted contact near the southern boundary of the site with quartzite of the Government Reef Formation of the Witwatersrand Supergroup.

The data obtained from the trial holes drilled on the Power Station site showed this simple model to be incorrect. In addition to andesite and quartzite, both shales and a diabase dyke were found on the site. In order to assist with the interpretation of the data and propose a geological model which takes account of the additional data, a desk study was carried out to obtain information from the investigations undertaken on surrounding sites and place the data from the Power Station site in a regional setting.

The following information was found to be available:

1. The profiles of the holes referred to on de Beer's map, which are contained in his thesis presented for the degree of MSc at the University of the Witwatersrand.

2. South African Breweries, Centenary Project Remainder of Lot 599 Newtown, Geotechnical report, December 1993, ARUP.
3. Geotechnical Investigation for the proposed Bezuidenhout Street Extension Roadworks between President and Jeppe Streets, Newtown for the Department of Planning of the Johannesburg City Council, November 1990, Schwartz Tromp and Associates.
4. Africana Museum, Site Investigation Report, September 1988, ARUP.

5.0 **SITE GEOLOGY**

5.1 **General**

The northern portion of the Power Station site, as well as Mary Fitzgerald Square and the Africana Museum, is underlain by the downfaulted graben of the Ventersdorp Group Andesitic Lava.

In three of the holes on the Power Station site, TH4, TH6 and TH7, a diabase dyke was exposed. This dyke runs in an east-west direction and stops abruptly on the eastern side of the Power Station where a quartzite was found in trial holes TH8 and TH11. In order to explain this, a fault is postulated and it has been inferred that the diabase was misinterpreted as being andesite in Hole 60 taken from de Beer's thesis.

The investigation carried out on the Centenary Site showed that the site was underlain by shales and quartzites of the Government Subgroup.

Trial hole TH5 drilled on the Power Station site exposed residual andesite, the results of this hole were compared with trial hole TH6 drilled for the Bezuidenhout Street investigation where it was considered to be a diabase. Although both the andesite and diabase weather to clayey silt residual soils, the diabase material found on the Power station site was characteristically speckled. Based on the inspection of TH5 and the re-

evaluation of the Bezuidenhout Street TH6, this is considered to be andesite. It is important to note that on the site of the Reserve Bank Building, immediately to the east of the Centenary Site, andesite lava was identified on the south of the site.

The residual andesite on the northern portion of the Power Station site is overlain by variable thicknesses of alluvium and characterised by a high water table. Trial hole TH4 on the south-west corner of the site is underlain at depth by the diabase dyke, but has alluvial soils to a depth of 3,8m and a water table at a depth of 4,7m.

It is postulated that TH4 lies on the channel of the Brickfields Spruit, which drains into the marshy former Brickfields area on the north of the Power Station site.

The transported soil in TH15 was observed to be leached implying that this hole is close to the marshy area of the original Brickfields.

Several of the holes in the central portion of the site refused on large concrete slabs, which were considered to be remnants of the former cooling towers. Regrettably, this meant that more precise data on the boundaries between the various material types could not be obtained.

The plan positions of the trial holes drilled on the site, as well as those drilled for the Centenary and Bezuidenhout Street investigations are shown on Figure 1, together with the postulated geological model.

Based on the postulated geological model, the site has been divided into a number of Geotechnical Areas, based on the underlying geology and the geotechnical conditions. These Geotechnical Areas are described briefly below:

Geotechnical Area A

This area, which occupies the majority of the site, is underlain by alluvium overlying residual andesite. It is lower lying and as a result has the water table closer to the existing ground level.

Geotechnical Area B

This area on the southern portion of the site is also underlain by andesite but there is no alluvium. As the existing ground level is at a higher elevation than in Geotechnical Area A, the groundwater table is deeper beneath the existing ground level.

Geotechnical Area C

This area comprises the diabase dyke, which separates Geotechnical Area A from Geotechnical Area B.

Geotechnical Area D

This area, which occurs on the south-eastern portion of the site, is underlain by quartzites and shales of the Government Subgroup.

It is extremely important to note that these Geotechnical Areas have been based on the postulated geological model, which has been derived from the data obtained from the current investigation as well as a re-evaluation of data from surrounding sites. The constraints imposed by the limited access on the site means that the data has been obtained at individual points and the boundaries between the various geological materials and consequently the Geotechnical Areas have been inferred.

5.2 **Strata Encountered**

The strata encountered in the various Geotechnical Areas are described under their separate headings below:

Geotechnical Area A

The following trial holes, which exposed natural geological materials, were drilled in this area, TH1, TH2, TH3, TH10, TH14 and TH18. Based on the detailed profiles of these six trial holes, the following generalised profile has been compiled for Geotechnical Area A:

- Fill: Variable fill of between 0,9m and 2,8m was found to cover the area. It is probable that the deeper areas of fill represent replacement of material previously extracted for brick making. In this regard it is of interest that the groundwater table in TH2 lies within the fill horizon.
- Transported: In three of the holes there was a thin layer of sandy transported soil, which was, in places, leached and occurred to depths of between 1,5m and more than 3m.
- Alluvium: The alluvial soils were dark coloured and of soft to firm consistency, frequently containing rounded quartz pebbles. The alluvium occurred to depths of between 2,6m and 4,9m.
- Residual Andesite: The residual andesite was a clayey SILT derived from the weathering of the underlying andesitic lava, which had been reworked at the top. This horizon varied in consistency from soft to firm through very stiff and occurred to depths of between 6,5m and 16,4m below existing ground level.
- Andesite: The andesite bedrock was seen to be a highly weathered closely jointed very soft rock, which increased in consistency

with depth and in which the auger refused at depths of between 10,3m and 16,4m.

The groundwater table was close to the current ground surface in this area and was found at depths of between 1,6m and 6,5m below ground level.

Geotechnical Area B

The only hole on the site drilled in this area was TH5, the profile of which was found to be:

Fill: A surfacing of 300mm of bitumen macadam.

Transported: A 600mm thick horizon of soft to firm clayey SILT with some rounded quartz pebbles.

Residual Andesite: The residual andesite, which had been reworked to a soft to firm material in the upper 400mm was a clayey SILT, which varied in consistency from firm through very stiff and occurred to depths of more than 22,5m.

The groundwater table was at a depth of 11,1m below existing ground level.

Geotechnical Area C

This area is that underlain by the diabase dyke, which separates Geotechnical Area A from Geotechnical Area B. Three trial holes were drilled in this area, namely TH4, TH6 and TH7 and based on the details of these profiles; the following generalised profile has been compiled:

Fill: An horizon of variable fill varying between 0,7m and 2,7m in thickness.

- Transported: In TH4 only there was an horizon of medium dense clayey silty SAND, which occurred to a depth of 1,3m.
- Alluvium: A thin horizon of alluvium was found in TH4 and TH7 and was a soft to firm clayey SILT, which occurred to depths of between 3,1m and 3,8m.
- Residual Diabase: The residual diabase was also a clayey SILT, which varied in consistency from firm through very stiff, which occurred to depths of between 6,4m and 9,5m.
- Diabase: The diabase bedrock was seen to be a dark greenish-brown speckled greenish-grey and black or dark greenish-grey speckled white highly weathered closely jointed very soft rock, which increased in consistency with depth and within which the auger refused at depths of between 7,6m and 13,2m.

The groundwater table was found at depths of between 2,7m and 4,7m.

Geotechnical Area D

This area is underlain by sediments of the Government Subgroup comprising quartzite and shale. The following materials were found:

- Fill: A thin horizon of between 0,5m and 0,9m of variable fill.
- Transported Soil: In two of the holes, there was a leached transported soil comprising a silty or clayey SAND with some sub-rounded quartz gravel at the base. This horizon occurred to depths of between 1,7m and 2,8m.
- Alluvium: In TH15 only, there was a 200mm thick horizon of a soft clayey SILT of alluvial origin.

Residual Quartzite: In trial hole TH11, there was a thin 0,9m layer of reworked residual quartzite consisting of a medium dense slightly clayey silty SAND.

Quartzite: The quartzite bedrock was a very soft rock through soft rock to medium hard rock argillaceous quartzite in which the auger refused at depths of between 3,2m and 3,3m.

Residual Shale: In TH18, the residual shale was a very stiff laminated closely jointed clayey SILT, which occurred to a depth of 2,9m.

Shale: Beneath the residual shale, the shale bedrock was a dusty red highly weathered closely jointed very soft rock with refusal at 5,7m.

Groundwater was found in TH15 only at a depth of 3,9m.

6.0 FIELD AND LABORATORY TESTING

6.1 Field Testing

Crosshole jacking tests were done to determine the modulus of elasticity of selected strata in the trial holes. These tests consisted of bedding two 200mm diameter steel plates on opposite sides of the trial holes. The plates were then jacked apart with an hydraulic jack and the load on the plates and their separation recorded.

Based on the assumption that each of the plates moved equally a drained modulus of elasticity for the soil was calculated using the expression after Bycroft (1956).

$$E'_h = \frac{(7 - 8 \nu) (1 + \nu) P_{av} \cdot \pi a}{16 (1 - \nu) \rho_h}$$

Where P_{av} is the pressure on the plate
 ρ_h is the deflection of the plate
 a is the radius of the plate
 ν is the Poisson's ratio, taken as 0,2
 E'_h is the mass modulus in the horizontal direction

The results of these tests, which are summarised in the table below, were used to confirm the strength and deformation characteristics of the rock.

Table 1 - Summary of Crosshole Jacking Tests

Hole No.	Depth (m)	Material Type	Consistency	Stress Range (kPa)	E'h (MPa)
TH1	4,5	Reworked Residual Andesite	Soft to Firm	92 - 184	4,4
TH2	6,0	Residual Andesite	Firm	92 - 184	14,2
TH3	5,0	Reworked Residual Andesite	Soft	92 - 184	3,0
TH5	3,0	Residual Andesite	Firm	92 - 692	24,68
TH5	8,0	Residual Andesite	Soft to Firm	92 - 231	2,8

6.2 Laboratory Testing

Representative soil samples taken from the sides of the trial holes were submitted to Civilab (Pty) Ltd. for the following tests:

- Particle size distribution
- Natural moisture content and Atterberg limits
- Complete CBR tests

The results of these tests, which are summarised in Tables 2A and 2B, confirmed the field descriptions of the materials.

The results of the California Bearing Ratio tests showed that the surficial soils were very poor subgrades with the transported and leached transported soils classifying as

G10 materials in the TRH14 System and the reworked residual andesite classifying as worse than G10.

Two undisturbed samples were taken of the material through which the basement excavation would be dug for slow drained shear box testing. The results of these tests are summarised below and the detailed results of the individual tests are attached to this report as Appendix B.

Table 3 – Summary of Shear Box Tests

Hole No.	Depth (m)	Material Type	Consistency	C' kPa	ϕ°
TH1	4,5	Reworked Residual Andesite	Soft to Firm	25	24
TH5	8,0	Residual Andesite	Soft to Firm	18	30

Two samples of groundwater were taken from the water, which flowed into the trial holes. These samples were submitted to B.N. Kirk Inc. for chemical testing. The results of these chemical tests showed that the groundwater was mildly to fairly aggressive towards concrete, but very corrosive towards ferrous metals.

7.0 SITE EVALUATION AND RECOMMENDATIONS

7.1 Geology

The geological model for the site, which is shown on Figure 1, explains the data currently available but requires confirmation. Further exploratory trial holes and boreholes need to be drilled at selected positions on the site to confirm the material identifications and to enable better definition of the contacts.

7.2 Potential Expansiveness

The potential expansiveness of the materials where either the plasticity index or clay fraction of the whole samples were greater than 12 has been assessed.

Firstly, these were plotted on the Standard Activity and Plasticity Chart, see Figures 2A and 2B, and then they were assessed in accordance with the procedure recommended by Weston.

The more sandy transported soil was shown to be of low to medium potential expansiveness and the alluvium to be either of low or medium potential expansiveness. When assessed in accordance with Weston, these classifications were confirmed. We consider, therefore, that the alluvial soils should be treated as being of medium potential expansiveness.

The reworked residual andesite and residual andesite soils plotted as being of between low and high to very high potential expansiveness. When assessed using the Weston method, these soils were shown to have swells at 1kPa of less than 1,6, which correlates with being of low potential expansiveness. We consider, therefore, that at their current high natural moisture contents, these soils will not be expansive.

7.3 Basement Excavation

7.3.1 General

The factors, which will influence the bulk excavation for the proposed Super-Basement, are described for the four Geotechnical Areas under their separate headings below. In addition, it is important to note that on large portions of the site, particularly near the centre, there are large obstructions resulting from uncleared remnants of the cooling towers and probably of other buildings, which were demolished prior to the landscaping of the site.

7.3.2 Geotechnical Area A

This Geotechnical Area is underlain by alluvium overlying residual andesite and andesite.

Bulk excavation in the alluvial and residual soils, which occur to depths of between 6,5m and 15,2m, will classify as 'soft' excavation in accordance with SABS 1200D.

Excavation of the very soft rock and soft rock highly weathered andesite lava will classify as 'intermediate' in accordance with SABS 1200D and andesite below auger refusal, which occurred at depths of between 10,3m and 16,4m, will require heavy ripping or possibly blasting and will classify as 'hard rock excavation'.

With regard to the classification of the excavability of these materials, we recommend that in the contract documents, two categories of excavation be used. These should be common and blasting with common being described in terms of its rippability with reference to the tined bucket width of a powerful backactor such as a Caterpillar 235.

It is important to note that the groundwater table in Geotechnical Area A is relatively close to the current ground surface, being at depths of between 1,6m and 6,5m below existing ground level. In addition, rapid inflow of groundwater into trial hole TH18 caused the sides of the trial hole to collapse and prevented further drilling of the trial hole. In this regard, particular attention should be paid to the recommendations for dewatering as given in Section 7.3 and lateral support in Section 7.4.

7.3.3 Geotechnical Area B

Bulk excavation in this area may be taken down to depths in excess of 22,5m in 'soft' excavation. It is important to note, however, that in the only trial hole drilled in this area, the water table was at a depth of 11,1m.

7.3.4 Geotechnical Area C

This Geotechnical Area is underlain by the diabase dyke. The transported soils and the residual soils derived from the weathering of the diabase dyke, which occur to depths of between 6,4m and 9,5m, will classify as 'soft' excavation.

The very soft rock and soft rock diabase, which was drilled by the auger, will classify as 'intermediate' and the diabase beneath auger refusal at depths of between 7,6m and 13,2m will require blasting and will classify as 'hard rock excavation'.

As for the excavation in Geotechnical Area A, we draw attention to the desirability of using two classes of excavation for the bulk earthworks contracts, namely common and blasting. We also would like to emphasis the high water table at depths of between 2,7m and 4,9m and the rapid inflow of groundwater into the trial holes.

7.3.5 Geotechnical Area D

This area is underlain by the Government Reef sediments comprising both quartzite and shale.

This Geotechnical Area is characterised by shallow bedrock at depths of between 0,5m and 3,9m and shallow refusal of the auger machine at between 3,2m and 5,7m. Excavations down to refusal depth should classify as 'common' and below this depth as 'blasting'.

The water table occurs at a depth of 3,9m as found in trial hole TH15.

7.4 Dewatering

The water table has been shown to be relatively shallow on this site, being closest to ground level on the lower lying northern portion of the site and deeper beneath the existing ground level on the higher lying southern portion of the site.

Where basement excavations are planned to go beneath the groundwater table, the actual position of this relative to the finished floor level of the basement needs to be

confirmed. In this regard, we recommend that piezometers be installed in the boreholes, which should be drilled to confirm the geological model. These piezometers should be monitored over a period of at least six months to provide confidence on the depth to groundwater. Furthermore, these levels should be related to a reduced level common to the site; this will require the survey of the plan positions and elevations of these holes.

Where basements are taken beneath the groundwater table, temporary dewatering will be required to enable the bulk excavation. In the permanent condition, the basement will either have to be tanked and designed to resist the hydraulic uplift or a permanent dewatering solution will be necessary.

In order to design the dewatering of the basement in the variable ground conditions, we recommend that pump tests be carried out within each of the four Geotechnical Areas.

These pump tests need to be designed to take account of the proposed bulk excavation level and the finished floor level of the basement.

7.5 **Lateral Support**

The upper portions of the excavation will be in fill, which will be highly variable. In the fill an assumption of $C' = 0$ and $\phi' = 30$ may be made. Because of the heterogeneous nature of the fill, which may contain large unnatural obstructions, some variable overbreak may be expected and allowance should be made for making this overbreak good and supplying some form of temporary shuttering on vertical faces.

The transported soils, including the alluvial soils, are currently very moist or wet and are of low strength. In the sandy transported and alluvial soils, an assumption of $C' = 0$ and $\phi' = 28$ may be made. The clayey alluvium may be assumed to have a $C' = 1\text{kPa}$ and $\phi' = 21^\circ$.

The results of the slow drained shear box tests on the reworked residual andesite and residual andesite show that the following parameters may be used for design purposes.

Material	Design Parameters	
	C' kPa	ϕ'
Reworked Residual Andesite	8	24°
Residual Andesite	4	30°

The design parameters for the reworked residual diabase and residual diabase may be assumed to be the same as those of the andesite soils.

The investigations carried out for the Centenary Site showed that the following parameters may be assumed for the Government Reef Sediments:

Material	Design Parameters	
	C' kPa	ϕ'
Shale	2,5	30°
Quartzite	1,5	38°

Stabilisation of the sides of the excavation may be achieved by either cutting the sides to batters of 30 degrees or by supplying an active lateral support by means of either earth anchors or soil nails. Where the basement excavation is less than 10m, we consider that a soil nail solution, together with a secondary support, which may be achieved by guniting, will be preferable but for excavations to depths of more than 10m, earth anchors will most probably be required.

The final choice of lateral support system for the sides of the excavation will depend on the depth of the excavation, the requirements for working space and a cost comparison between cutting to batters and an active nail or earth anchor solution.

7.6 Foundations

The type of foundation most suitable will depend on the depth of the basement excavation, as well as the structural loads.

The majority of the residual materials in Geotechnical Areas A, B and C are of low strength and highly compressible. In these areas, spread foundations should preferably be taken down to the very soft rock andesite or diabase on which contact stresses of up to 1000kPa may be used. Where the depths to very soft rock are considered to be uneconomical for spread foundations, then a piled foundation solution should be considered.

In view of the high water table and the potentially collapsible nature of the materials, we consider that a pre-bored driven cast in situ pile would be the most suitable pile type.

In Geotechnical Areas D, the Government Reef Sediments will provide good founding strata at shallow depths. Initial design should assume contact stresses of 1000kPa, although for large bases, this may be increased at depths below auger refusal to stresses of up to 2000kPa.

7.7 Subgrade Preparation

The results of the CBR tests showed that the transported soils and reworked residual andesite soils were very poor subgrades. The reworked residual andesite soils and residual andesite soils, as well as the soils derived from the diabase are known from other sites to be very poor subgrades.

Subgrade preparation in these soils should be:

1. Cut 300mm of the exposed soils to spoil.

2. Rip the top 150mm of the exposed subgrade to a depth of 150mm, mix with 2% lime and 2% cement and compact to 93% of the Proctor maximum dry density at optimum moisture content to optimum moisture content +2%.
3. Place an imported 150mm layer of lower selected subgrade to be a G8 material compacted to 90% of the modified AASHTO maximum dry density at optimum moisture content + or – 1%.
4. Place an imported 150mm layer of upper selected subgrade to be a G7 material compacted to 93% of the modified AASHTO maximum dry density at optimum moisture content + or – 1%.

The subgrade in the Government Reef Sediments is expected to be much better than over the rest of the site and should comprise the ripping and recompaction of the exposed sediments to 90% of the modified AASHTO maximum dry density with one selected subgrade layer of a G7 material, which may be either imported or possibly may be obtained from the residual quartzite.

8.0 REFERENCES

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