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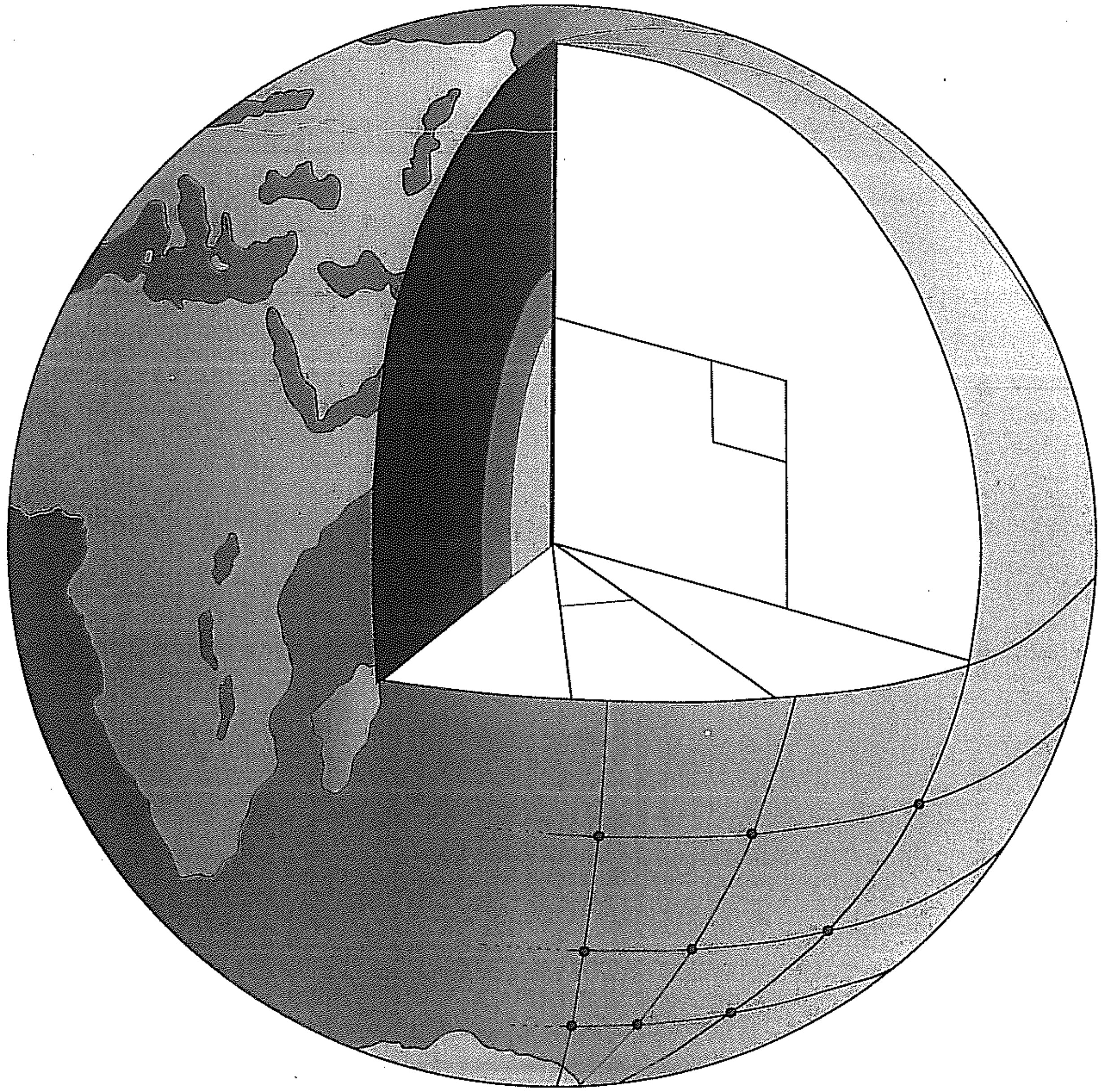

GEOTECHNICAL INVESTIGATION
of
RIVER SIDE VIEW EXT. 1 AND LAZY WATERS EXT.1.
SANDTON.

Client: Helderfontein Estate.
Report No. 02186 May 2002

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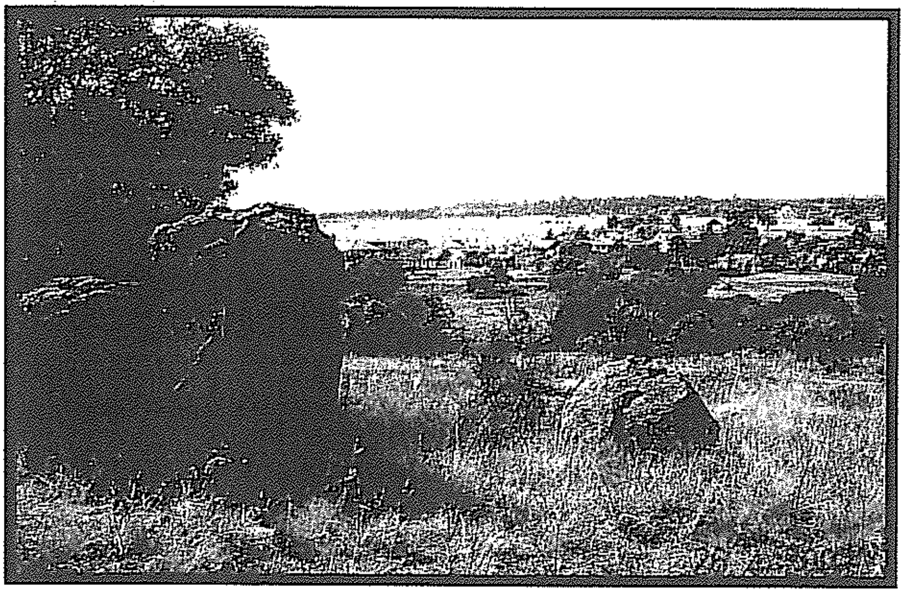

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1. INTRODUCTION

1.1 Preamble

On the 12th March 2002, Geopractica were requested by Paul Carlisle of Carlisle and Associates to submit a proposal to undertake a geotechnical investigation on the Helderfontein Estate situated in Sandton.

The purpose of the investigation was to determine the subsoil conditions for the proclamation of a 160ha site to be known as Riverside View Extension 1 and Lazy Waters Extension 1 situated on Portions RE/5, RE/8, 11, 134/13, 156/14, RE/182, 186, 228, (a portion of 187) and 231 of the farm Zevenfontein 407-JR.

The proposal was accepted by Carlisle and Associates on behalf of VBGD Town Planners on the 4th April 2002.

1.2 Database

The extent of the site was indicated in the field on the 22nd April, by Mr. B. Heafield, the owner of the Helderfontein Estate.

The following information was supplied to Geopractica:

- An orthophoto of the site, dated 3 December 2001, drawing number 5806-108, entitled "Helderfontein Estate" at a scale of 1 : 2500 was provided to Geopractica by Mr. L. Druce of VBGD.
- A surveyed plan, dated 3 December 2001, drawing number 5806-107, entitled "Helderfontein Estate" at a scale of 1 : 2500 was provided to Geopractica by Mr. B. Alport of Kantey and Templer Consulting Engineers.

1.3 Objectives

It is proposed that the site is to be developed into a residential golf estate (Residential 1 and Residential 2), with Business and Special rights on limited portions .

The objectives of the investigation were therefore:

- to identify the soil and rock conditions below the site to a depth of 3.5 m or refusal.
- to recommend the most suitable foundation system and founding depth for the proposed structures.
- to classify the site.
- to comment on any perceived geotechnical problems which may affect either the design or construction of the project.

2. METHODOLOGY AND IMPLEMENTATION

2.1 Literature Review

2.1.1 Prior to the commencement of any field work, a literature review was conducted on general geological and photographic data obtained from various sources. Reference was also made to published as well as unpublished documents and maps. The project was planned using the "Guidelines for Urban Engineering Geological Investigations" as published by the South African Institute of Engineering Geologists in 1997.

2.1.2 The 1 : 250 000 geological map "East Rand" Sheet 2628 published by the Geological Survey of South Africa in 1986, was consulted as were the 1:50 000 topographic maps, sheets 2528 CC Verwoerdburg and 2628 AA, Johannesburg, published by the Chief Director of Survey and Mapping, in 1975 and 1983 respectively.

2.1.3 Aerial photographs at a scale of 1:17 000, flown in 1994 were obtained from International Aviation Services and were used extensively during the initial planning of the investigation. From the aerial photographs obtained, stereographic aerial photo interpretation was undertaken by means of a Wild Stereoscopic API unit. Geological boundaries, soil facet boundaries, as well as drainage lines and cultural features were identified within the areas to be investigated. This information, together with all data obtained during the literature survey was collated onto the previously mentioned orthophoto at a scale of 1:2500.

2.2 Field Work

2.2.1 On 24th April 2002, the fieldwork commenced. Test pit positions were placed on an approximate 200m by 200m grid to determine soil facet boundaries. The test pit positions were located on site by means of a Magellan 3000 GPS (Global Positioning System).

2.2.2 A CAT 428 TLB (Tractor Loader Backhoe) was used to excavate the test pits. Each test pit was profiled by an engineering geologist, employing the MCCSSO (Moisture, Colour, Consistency, Structure, Soil, and Origin) method, as proposed by Jennings, Brink and Williams (1973). A total of 36 test pits, at an average density of 2.25 test pits per 10ha were excavated to refusal or to a maximum depth of 2.5m.The test pits were sampled as deemed necessary, in order to characterise the mechanical properties of the soils, and the holes were subsequently backfilled upon completion of the field work. All the test pit profiles are presented in Appendix 2 of this report.

2.3 Office and Laboratory Work

2.3.1 Soil testing was carried out by Geopractica's soil laboratory. Summarised in Table 1. below are the types of laboratory tests carried out, the number of samples tested, and the engineering parameters determined. The laboratory test results are included in Appendix 3.

TABLE 1. Summary of Soil Tests

Laboratory Test	Engineering Parameters	No.
Foundation Indicator	Grading to 2 microns and Atterberg Limits	21
Collapse Potential Test	Determination of Collapse Potential and Consolidation Characteristics	8

2.3.2 Finally this report was prepared using the data derived from the literature survey, and primarily from the collation of all the data and information available from the field and laboratory investigations.

3. PHYSICAL ENVIRONMENT

3.1 Site Description

3.1.1 The site is situated some 5km north of the Fourways Mall in Sandton, and covers a surface area of approximately 160ha. The area investigated falls within a shallow valley formed by the Jukskei River which flows to the north and a tributary stream known as the Modderfontein Spruit, which flows towards the west. The western limits of the site are defined by the R511(William Nicol Drive), while the remaining boundaries are defined by adjacent privately owned agricultural holdings.

3.1.2 The area is largely undeveloped, with the Helderfontein Conference Centre occupying approximately 7.5ha on the southern portion of the site. The remainder of the property is currently used for low scale stock grazing. Numerous informal gravel roads and tracks, many of which have recently been blocked off the prevent unwanted traffic on the property, cross the area.

3.1.3 A proposed servitude for the planned provincial K 56 route passes through the property from west to east in the northern portion of the site.

3.1.4 The most prominent drainage feature through the area is the Jukskei River, which flows from south to north along the western side of the property. A smaller perennial stream, known as the Modderfontein Spruit flows from east to west through the northern portions of the site. This stream has been dammed at numerous places along its course, forming small ornamental features that have formed shallow wetland that attract prolific birdlife.

3.1.5 The general shape of the land is that of gentle sloping convex topography that slopes down towards the drainage features at a relatively shallow gradient of approximately 2% to 5%.

3.1.6 The higher lying portions of the property are vegetated by open natural grasslands, with occasional indigenous Wild Olive *(Olea europaea)* and Karee *(Rhus lancea)* trees. Generally associated with isolated outcrops of granite bedrock. The low lying portions of the property occupied by the Jukskei river and Modderfontein Spruit are well vegetated with established White Stinkwoods *(Celtis africana)* and River Bushwillow *(Combretum erythrpphyllum)* trees together with stands of exotic Eucalyptus trees. Alien invader species have largely been removed along the water courses and only isolated Black Wattle, Weeping Willow and Poplar trees remain.

3.2. Site Geology

3.2.1. From the available literature as well as the observations during the site investigation, it is evident that the site is underlain by granitic rocks of the Basement Complex, as exposed in the Johannesburg-Pretoria Dome.

3.2.2. Typically these Archaean intrusive igneous rocks are cross cut by dolerite dykes of various ages, and the edge of an northeasterly trending dyke was exposed in test pit C 6 along the southern edge of the site. As a result of deep and extensive chemical weathering, the dolerite dyke has been reduced to residual sandy and silty soil that has preserved the original mineral fabric.

3.2.3 Generally the weathering profile is largely controlled by two factors. Firstly, the tectonic forces that have operated on the area have created specific geological structures in the form of faults, joints and shear zones. These lineaments are exploited by weathering over geological time. Secondly the composition of the granite varies from granite gneiss, granodiorites, migmatites, porphyritic granites, and granites of different grain size. This variation in composition contributes, and largely controlled the weathering depth and patterns, resulting in deep residual soils forming adjacent to isolated rocky outcrops.

3.2.4 No significant geological faults are known to occur in the vicinity of the site, however several small extremely localised faults which are generally north south trending were noted in the bedrock exposed in the stream bed of the Modderfontein Spruit.

3.3. Hydrology

3.3.1 Ground water occurs in two ways in the granitic rocks of the Johannesburg-Pretoria Dome. Firstly, it is contained in the upper weathered profile as a primary aquifer, and secondly, ground water may also be found in fractures within the solid and unweathered granite below the weathered zone. These two aquifers are normally in hydraulic connection as there is no geological reason separating the two systems. Depending on the degree and depth of weathering as well as the amount of clay present in the weathered granite, the reported yield of boreholes drilled into the weathered granite can vary from less than 0,1 l/s to more than 7 l/s.

3.3.2 As a consequence of the irregular nature of the weathering profile, isolated "pockets" of deeper weathering may be present. Different "pockets" can be hydraulically well connected in places, whereas in other localities the lateral hydraulic link may only be very poor or may not exist at all. The nature of the hydraulic link will govern the sustainability in terms of yield of water pumped from production boreholes. Although the flow of ground water towards the borehole is governed by may factors, the long term sustainability of the boreholes depends on the hydraulic links between the different "pockets" of weathering.

3.3.3. Slow groundwater seepage was only recorded in test pit G3 at a depth of 2.1m. No ground water was recorded in any of the other test pits. A well, of unknown depth, is situated immediately north of test pit D 4, in which continues seepage is recorded and the water rest level is at surface, implying that the water is artesian. It is apparent that the aquifer is extremely localised as no seepage was recorded in any of the test pits excavated in the immediate vicinity.

3.3.4. The average annual rainfall in this area is approximately 750 mm, most of which occurs as heavy, isolated thunder showers between October and March. Storm water runoff from the site is primarily in the form of sheetwash, however some concentration will occur in small gullies that are orientated down the slopes of the site towards the Jukskei River and Modderfontein Stream.

3.4 Field Observations

3.4.1. A total of 36 test pits were excavated on an approximate 200m by 200m grid. The density of test pits is approximately 2.25 test pits per 10ha. Following the detailed aerial photographic interpretation of the area, it was noted that the soils distribution closely followed the underlying geology. Where adequate data was not acquired from test pits placed on the grid, additional test pits were excavated, at selected positions in between the existing grid.

TABLE 2. Summary of Trial Holes

TP No	Depth (m)	Material at EOH	TP No	Depth (m)	Material at EOH
A5	1.9	Very soft rock granite	E5	0.7	Very soft rock granite
A6	1.2	Very soft rock granite	E6	1.0	Very soft rock granite
A7	1.0	Very soft rock granite	F2	0.6	Very soft rock granite
B4		Very soft rock granite	F4	0.4	Very soft rock granite
B5		Very soft rock granite	F5	1.9	Very soft rock granite
B6	1.6	Very soft rock granite	F6	2.0	Hard dig, hole terminated in dense residual granite
B7	0.7	Very soft rock granite	G1	1.8	Very soft rock granite
C2	0.9	Very soft rock granite	G2	1.0	Very soft rock granite
C3	0.5	Very soft rock granite	G3	2.6	Hard dig, hole terminated in dense residual granite
C4	1.5	Very soft rock granite	G4	1.5	Very soft rock granite
C5	2.0	Hole collapsing in fill	G5	2.8	Hard dig, hole terminated in dense residual granite
D1	0.7	Very soft rock granite	H1	0.3	Hardpan Ferricrete
D2	0.8	Very soft rock granite	H2	1.4	Very soft rock granite
D7	1.2	Very soft rock granite	H3	0.8	Hardpan Ferricrete
E1	0.6	Very soft rock granite	H4	1.3	Hardpan Ferricrete
E2	0.8	Very soft rock granite	I2	2.0	Very soft rock granite
E3	1.7	Very soft rock granite	I3	1.6	Very soft rock granite
E4	0.5	Very soft rock granite	J2	0.7	Very soft rock granite

3.4.2. The area is generally blanketed by a thin horizon of loose silty sand and gravel, colluvium, which often overlies a pebble marker horizon which separates the transported material from the residual soils. The colluvial material is slightly ferruginised in places, however the soils tend to be very weakly cemented.

3.4.3. A ferruginised horizon occurs at a depth of between 0.4 to 1.2m in the eastern and northern portion of the site. This pedogenic horizon was described as well cemented nodular and honeycomb ferricrete, which becomes a hardpan ferricrete in isolated places.

3.4.4. The residual soils consist of medium dense to dense coarse sand, which contains abundant granite and quartz gravels. This horizon tends to become very dense with increasing depth and often grades into soft rock granite within 0.8 to 2.5m of the surface. Small localised and isolated outcrops of granite bedrock were noted through out the area. It is of interest to note that well developed transported and residual soil profiles are often recorded immediately adjacent to the bedrock. This is a typical characteristic of granitic environments, which often gives rise to localised tors (exposed bedrock often seen as heaps of boulders).

4. LABORATORY TESTING

4.1. Index Testing

For more accurate identification and classification purposes, twenty one samples were subjected to Particle Size Distribution and Atterberg Limits Tests. The tests were carried out on representative samples of the various soil horizons present within the site and the results are shown in Appendix 3, and are summarised in Table 3 below.

The results indicate that in general the soils in the area are of low plasticity, having a plasticity index of between non plastic to a high of 18. The potential expansiveness is therefore low, and no expansive soils appear to occur on the site.

TABLE 3. Summary of Indicator test results

TP No.	Depth (m)	Material	PI	PI (ws)	GM	MC (%)	Activity
A5	0.8-1.0	Clayey and silty sand. Rew. Res. Granite	15	8	0.96	14.1	low
A5	1.4-1.5	Silty coarse sand. Residual Granite	13	6	1.26	10.5	low
B6	1.2-1.4	Silty coarse sand. Rew. Res Granite	15	8	1.15	7.9	low
C2	0.7-0.9	Silty fine sand. Hillwash	10	7	0.72	4.91	low
C4	1.0-1.2	Silty coarse sand. Residual Granite	11	3	1.97	7.07	low
D1	0.5-0.7	Silty coarse sand and gravel. Residual Granite	14	8	1.24	5.73	low
D7	1.0-1.2	Clayey sand and gravel. Colluvium	18	8	1.40	8.36	low
E3	1.0-1.2	Silty coarse sand. Residual Granite	9	4	1.54	4.64	low
E6	0.6-1.0	Silty sand and ferricrete. Ferruginised Hillwash	11	5	1.44	6.39	low
F2	0.6-0.7	Silty coarse sand. Residual Granite	11	4	1.58	4.99	low
F6	1.0-1.2	Silty fine sand with gravels. Hillwash	7	4	1.00	5.22	low
G1	1.1-1.3	Silty coarse sand. Residual Granite	10	5	1.27	8.79	low
G3	1.4-1.6	Clayey silty sand. Gleyed Residual Granite	NP	NP	1.15	6.79	low
G4	1.0-1.2	Silty sand. Residual Granite	11	6	1.22	6.26	low
G5	0.6-0.8	Silty sand and ferricrete. Ferruginised Hillwash	NP	NP	2.1	6.8	low
G5	1.0-1.2	Clayey and silty sand. Rew. Res. Granite	10	6	0.9	13.8	low
G5	2.0-2.2	Silty sand. Residual Granite	11	5	1.2	11.2	low
H2	0.8-0.9	Silty sand. Hillwash	NP	NP	0.9	6.3	low
H4	0.8-1.1	Silty sand. Hillwash	NP	NP	1.19	1.2	low
I2	0.8-1.0	Clayey and silty sand. Rew. Res. Granite	12	8	0.8	14.3	low
I3	0.7-0.9	Silty sand and ferricrete nodules. Ferruginised Hillwash	NP	NP	1.6	6.6	low

4.2. **Collapse Potential Tests**

In order to establish the collapse potential of the various soil horizons, eight Collapse Potential Tests were carried out on representative block samples. The samples were placed under load in an oedometer, and soaked at an applied load of 200 kPa.

The severity of the problem typically associated with the material exhibiting a collapse potential, have been classified according to Jennings and Knight (1975) and are shown in table 4. below

TABLE 4. Severity of Collapse Potential (after Jennings and Knight 1975)

Collapse Potential (%)	Severity of Problem
0 - 1	No Problem
1 - 5	Moderate trouble
5 - 10	Trouble
10 - 20	Severe trouble
> 20	Very severe trouble

The results are shown in Appendix 3 of the this report, and summarised in table 5 below. The tests show that the transported hillwash, which blankets the site is potentially collapsible, with the degree of severity varying from 5% to 13%. The estimated total movement anticipate is between 20mm and 45 mm.

TABLE 5. Collapse Potential Test Results.

TP No	Sample Depth (m)	Description	Dry Density (kg/m^3)	Collapse Potential	Severity of Problem
A5	0.8-1.0	Clayey and silty sand. Rew. Res. Granite	1680	<1%	no problem
B6	1.2-1.4	Silty coarse sand. Rew. Res Granite	1310	4.6%	moderate
C2	0.7-09	Silty fine sand. Hillwash	1424	11.1%	severe
F6	1.0-1.2	Silty fine sand with gravels. Hillwash	1338	12.9%	severe
G5	1.0-1.2	Clayey and silty sand. Rew. Res. Granite	1648	<1%	no problem
H2	0.8-0.9	Silty sand. Hillwash	1808	<1%	no problem
H4	0.8-1.1	Silty sand. Hillwash	1683	5.1%	moderate
I2	0.8-1.0	Clayey and silty sand. Rew. Res. Granite	1643	<1%	no problem

5. INTERPRETIVE REPORT

5.1 Geotechnical Mapping of Site.

In order to map the geotechnical characteristics of the underlying soils, the geotechnical classification method proposed by Partridge et.al (1993) has been applied to this site. Table 6 shown below indicates the various geotechnical characteristics and the criteria used to evaluate the soils.

TABLE 6. Geotechnical Constraints Identified (after Partridge et.al. 1993)

	PARAMETER	CLASS 1 (Most favourable)	CLASS 2 (Intermediate)	CLASS 3 (Least favourable)
A	Collapsible soil	Surface collapsible horizon <750mm thick	**Collapsible horizon >750mm thick**	
B	Seepage	Water table permanently deeper than 1.5m below surface	Permanent or seasonal water table within 1.5m of surface	Swamp and marshes
C	Active Soil	<2.5mm differential movement expected	2.5-15mm differential movement	>15mm differential movement expected
D	Highly Compressible Soil	<2.5mm differential movement expected	**2.5 - 15 mm differential movement**	>15mm differential movement expected
E	Erodible soil	Low	Moderately dispersive soils, fissured clay, thick colluvium.	Highly dispersive soil, fissured clay, thick colluvium
F	Difficulty of excavation to 1.5m depth	<10% rock or hardpan pedocretes	**10-50% rock or hardpan pedocretes**	>50% rock or hardpan pedocretes
G	Undermined ground	depth of undermining is >100m in reasonably competent rock	Old undermined areas where stope closure has ceased	Where the depth of undermining is <100m
H	Instability in areas of soluble rock		Possibly unstable	Probably unstable
I	Steep slopes	Slope <6%	Slope 6-15%	Slope >15%
J	Areas of unstable natural slopes	Low risk	Intermediate risk	High risk
K	Areas subject to seismic activity	100 year max probability of <5 Mod Mercalii intensity	100 year max probability of 5-8 Mod Mercalii intensity	100 year maximum probability of >8 Mod Mercalii intensity
L	Areas subject to flooding		Areas above 1:50 year flood line but with slope <1%	**Areas below 1:50 year flood line**

The geotechnical properties of the soils which occur on the site have been mapped, and are shown on the geotechnical site plan included in Appendix 7. The site has been divided into five geotechnical zones, namely:

Zone 1	No limiting geotechnical criteria identified.
Zone 2A	Collapsible horizon greater than 750mm thick
Zone 2D	2.5 to 15mm differential consolidation settlement anticipated.
Zone 2F	10 to 50% rock or hardpan pedocrete within 1.5m of surface
Zone 3L	Areas falling below the 1 :50 year flood line

5.2 Design Solutions

5.2.1 *Geotechnical Zone 1 (No constraints identified)*

This zone represents an area of the site underlain by a thin surface horizon of transported soils that is less than 750mm thick. The underlying residual soils are expected to experience less than 2.5mm differential settlement.

It is therefore recommended that the structures built within Zone 1 are placed on strip footing placed on the residual soils at an average depth of 0.6m below natural ground level. The maximum allowable bearing pressures must not exceed 100kPa.

In the event of structures being placed on engineered terraces, it is strongly recommended that the footings are placed on residual soils that occur at depths of up to 0.8m B.G.L.

5.2.2 · *Geotechnical Zone 2A*
(Collapsible horizon greater than 750mm thick)

Collapse Potential tests have shown that the overlying transported soils within this zone can generally be considered to be moderately collapsible, with values ranging from 5% to 13% being recorded. Calculated total collapse settlements in the order of 20 to 45mm could be anticipated under applied loads of 100kPa. These values have been determined by assuming that 700mm wide strip footings will be placed at an average depth of 0.6m below current ground surface.

Structures constructed within the limits of Zone 2A, will require special foundation design and the proposed typical foundation solutions are shown in Table 7 below.

TABLE 7. Typical Foundation design in Geotechnical Zone 2A

Construction Type	Foundation Type And Building Procedures
Modified Normal	Reinforced strip footings Articulation joints in some internal and external walls Light reinforcement in masonry Good site drainage Foundation pressures not to exceed 80 kPa
Compaction of soils below footings	Remove in situ soils below foundations to a depth and width of 1.5 times the foundation width or to a competent horizon and replace with materials compacted to 93% Mod AASHTO density at -1% to +2% of OMC Normal construction with lightly reinforced strip footings
Deep strip foundations	Normal construction with sound site drainage Founding on a competent horizon below the problem horizon
Soil raft (further investigations will be required to determine design parameters)	Remove in situ materials to 1.0 m beyond perimeter of building to a depth of 1.5 times the widest foundation or to a competent horizon and replace with material compacted to 93% Mod AASHTO density at -1% to +2% of OMC Normal construction with lightly reinforced strip footings

5.2.3 *Geotechnical Zone 2D*
(2.5 - 15 mm differential consolidation settlement anticipated)

Representative undisturbed samples of the reworked residual granite were collected from the site and were subjected to Collapse Potential testing. From the results of these tests the anticipated consolidation settlements have been determined across the site. Within geotechnical Zone 2D it is anticipated that 2.5 to 15mm of differential settlement may be anticipated under applied loads of 100kPa. These values have been determined by assuming that 700mm wide strip footings will be placed at an average depth of 0.6m below current ground surface.

Potential founding solutions for the units to be constructed within Zone 2D are presented overleaf.

i. *Soil raft.* Remove in situ materials to 1.0m beyond perimeter of building (ie. the foot print of the structure) to a depth of 1.5 times the widest foundation, measured from the underside of the footings. Replace with the excavated residual granite or an imported G7 or better quality soil. The re-instated material must be compacted in 150mm thick layers to 93% Mod AASHTO density at -1% to +2% of OMC. Bearing capacity of the soil raft will be 120kPa. Foundations must be placed at a depth of 600mm below the top of the mattress and normal construction may proceed with brick force included between each course in the plinth wall for a minimum of 6 courses. The surface bed may be constructed directly on the soil raft.

ii. *Compaction of soils below individual footing.* Remove the in-situ soils below the foundations (both internal and external walls) to a depth of 1.5 times the foundation width or to a competent horizon measured from the underside of the footings. Replace with the excavated material compacted to 93% Mod AASHTO density at -1% to +2% of optimum moisture content, in layers not exceeding 150mm thick. Particular attention must be paid to the compaction at the edges of the trenches and at corners. Foundations must be placed at a depth of 600mm below the top of the mattress and construction may proceed with brick force included between each course in the plinth wall for a minimum of 6 courses.The maximum allowable bearing pressures must not exceed 100kPa.

For the surface bed the in-situ soils must be removed to a depth of 300mm, and replaced in 150mm thick layers with a selected G7 quality material, compacted to a minimum density of 93% of Mod AASHTO at -1 to +2% OMC.

iii. *Concrete raft.* A concrete raft designed by a competent structural engineer to tolerate the anticipated settlement.

iv. *Shallow piles.* Shallow augered piles or driven cast in-situ piles may be placed on the bedrock. The depth to refusal must be determined by means of Dynamic Probes (Super Heavy). The maximum allowable bearing capacity of the very soft rock granite is likely to be in the order of 500 kPa.

5.2.4 *Geotechnical Zone 2F* *(10 to 50% rock and pedocrete within 1.5m of the surface)*

Buildings placed completely on the very competent hardpan ferricrete or bedrock require no special precautions and standard strip footings may be used, and no foundation bearing capacity problems are likely to occur.

Should the footings straddle rock and soil, it is strongly recommended that the entire structure is placed on soil mattress. The rock should be removed to a minimum depth of 1.0m below the proposed founding level. A suitable G7 to G8 quality fill material should be placed in layers of 150mm, compacted to 93% Mod AASHTO at -1% to +2% OMC. The final two layers below the foundations should be compacted to 95% Mod AASHTO. The structures may be placed on conventional strip footings, applying a maximum load of 100kPa.

5.2.5 *Roads and Terraces*

The results of the Foundation Indicator Tests have been used to classify the in-situ soils to determine the suitability of this material for the construction of terraces and pavement layers. The results of the tests are included in Appendix 3, and are summarised in table 8 overleaf. The soil classification varies from G5 to G10 material and may therefore be used in the construction of terraces as well as use in the in-situ sub-grade and selected layers of proposed pavements. Suitable materials for use in the sub-base and base course layers must be imported from a commercial source.

Terraces may be constructed using the transported and residual soils that occur on the site. The upper 100mm of the hillwash must be removed to stockpile for subsequent use as landscaping materials. The underlying soils should be ripped and re-compacted in-situ to a density of 90% Mod AASHTO. Fill materials obtained from the site should be placed in layers of 150mm and compacted to 93% Mod AASHTO at -1% to +2% OMC.

TABLE 8. Summary of Test Results for Road and Earthworks

Test Pit No	Depth (m)	Material	PI	LS	GM	CBR at 95%*	TRH 14 Class
A5	0.8-1.0	Clayey and silty sand. Rew. Res. Granite	15	7	0.96	13	G10
A5	1.4-1.5	Silty coarse sand. Residual Granite	13	6	1.26	18	G7
B6	1.2-1.4	Silty coarse sand. Rew. Res Granite	15	7	1.15	16	G10
C2	0.7-0.9	Silty fine sand. Hillwash	10	5	0.72	16	G7
C4	1.0-1.2	Silty coarse sand. Residual Granite	11	5	1.97	36	G6
D1	0.5-0.7	Silty coarse sand and gravel. Residual Granite	14	7	1.24	18	G7
D7	1.0-1.2	Clayey sand and gravel. Colluvium	18	9	1.40	13	G10
E3	1.0-1.2	Silty coarse sand. Residual Granite	9	4	1.54	27	G6
E6	0.6-1.0	Silty sand and ferricrete. Ferruginised Hillwash	11	5	1.44	25	G6
F2	0.6-0.7	Silty sand and gravel. Residual Granite	11	5	1.58	27	G6
F6	1.0-1.2	Silty fine sand with gravels. Hillwash	7	3	1.00	21	G7
G1	1.1-1.3	Silty coarse sand. Residual Granite	10	5	1.27	22	G7
G3	1.4-1.6	Clayey silty sand. Gleyed Residual Granite	NP	0	1.15	33	G6
G4	1.0-1.2	Silty sand. Residual Granite	11	5	1.22	21	G7
G5	0.6-0.8	Silty sand and ferricrete. Ferruginised Hillwash	NP	1	2.1	60	G5
G5	1.0-1.2	Clayey and silty sand. Rew. Res. Granite	10	5	0.9	19	G7
G5	2.0-2.2	Silty sand. Residual Granite	11	5	1.2	21	G7
H2	0.8-0.9	Silty sand. Hillwash	NP	1	0.9	30	G6
H4	0.8-1.1	Silty sand. Hillwash	NP	0	1.19	35	G6
I2	0.8-1.0	Clayey and silty sand. Rew. Res. Granite	12	6	0.8	14	G7
I3	0.7-0.9	Silty sand and ferricrete nodules. Ferruginised Hillwash	NP	1	1.6	46	G5

* Results are inferred from empirical test data, see Appendix 4

5.3. **Residential Site Classification**.

Based on the information obtained during the investigation, the site has been classified according to the Code of Practice of The Joint Structural Division of SAIEC and IstructE (1995) as shown below. Appendix 5 contains the complete Residential Site Class Designation.

i. Units constructed within **zone 1** are classified as a **C** site.
ii. Units constructed within **zone 2A** are classified as a **C2** site
iii. Units constructed within **zone 2D** are classified as a **S1** site.
iv. Units constructed within **zone 2F** are classified as a **R** site

5.4. **General**

5.4.1 *Excavation Classification*

In geotechnical zones 1, 2A and 2D it is expected that the excavation class will be "soft" according to SABS 1200 D: Earthworks, up to a depth of 1.5m. It must however be anticipated that boulders and remnant core stones may also be encountered throughout the site.

Within Zone 2F it is expected that excavations will be classified as intermediate, with the possibility that areas of hard excavation may be encountered within 1.5m of the surface. The routing of underground services will have to take cognisance of the presence of isolated outcrops of bedrock, and in many cases the only method of excavating through the material will be by means of blasting in these areas.

5.4.2 *Stormwater Management*

No ground water seepage was encountered on the site. It must however be anticipated that shallow ground water may occur in isolated areas throughout the site, particularly after periods of sustained rainfall. This is likely to be particularly prevalent within Zone 2F where seasonal perched water may be encountered within the shallow transported soils that occur above the bedrock or hardpan ferricrete.

5.4.3 *Trees*

It is recommended that all large root systems are removed and any cavities are properly back filled with suitable material compacted to 90% Mod AASHTO density at +2% to - 1% of optimum moisture content. This is particularly important when removing the large Eucalyptus trees that occur on the site.

Wherever possible it would be aesthetically and environmentally pleasing to preserve the many established indigenous trees and shrubs that occur, particularly in close association with the rock outcrops.

5.4.4 *Spoil and Waste*

An area on the western side of the Jukskei River, particularly within the 1: 100 year flood line has been used as an informal waste site. Extensive tipping of construction and other general household waste has occurred. This was exposed in TP C5, where the waste has been spread, levelled and covered, and it was noted that the waste is up to 2.0m thick. No construction must be permitted on the waste body.

Numerous heaps of waste have also been tipped randomly in the area between the river and the R 511, which must be removed to a more suitable waste disposal facility.

5.4.5 *Insecticides*

Extensive termite activity was noted on the site, particularly in the vicinity of TP H4 and TP G4. It is therefore recommended that a recognised pesticide/insecticide be used to combat the threat of ants/termites and rodents to the integrity of the foundations.

5.5 Construction Problems

- It must be anticipated that corestones and boulders may be encountered in the excavations.

5.6 Additional Investigations

No additional investigations are considered necessary for the assessment of the near surface soils.

6. CONSTRUCTION MONITORING

6.1 Excavation Inspection

It is recommended that all foundation excavations be inspected by a competent person prior to placing any concrete.

6.2 Control Testing

Regular checks on the quality and compaction of the backfill to the terraces should be made.

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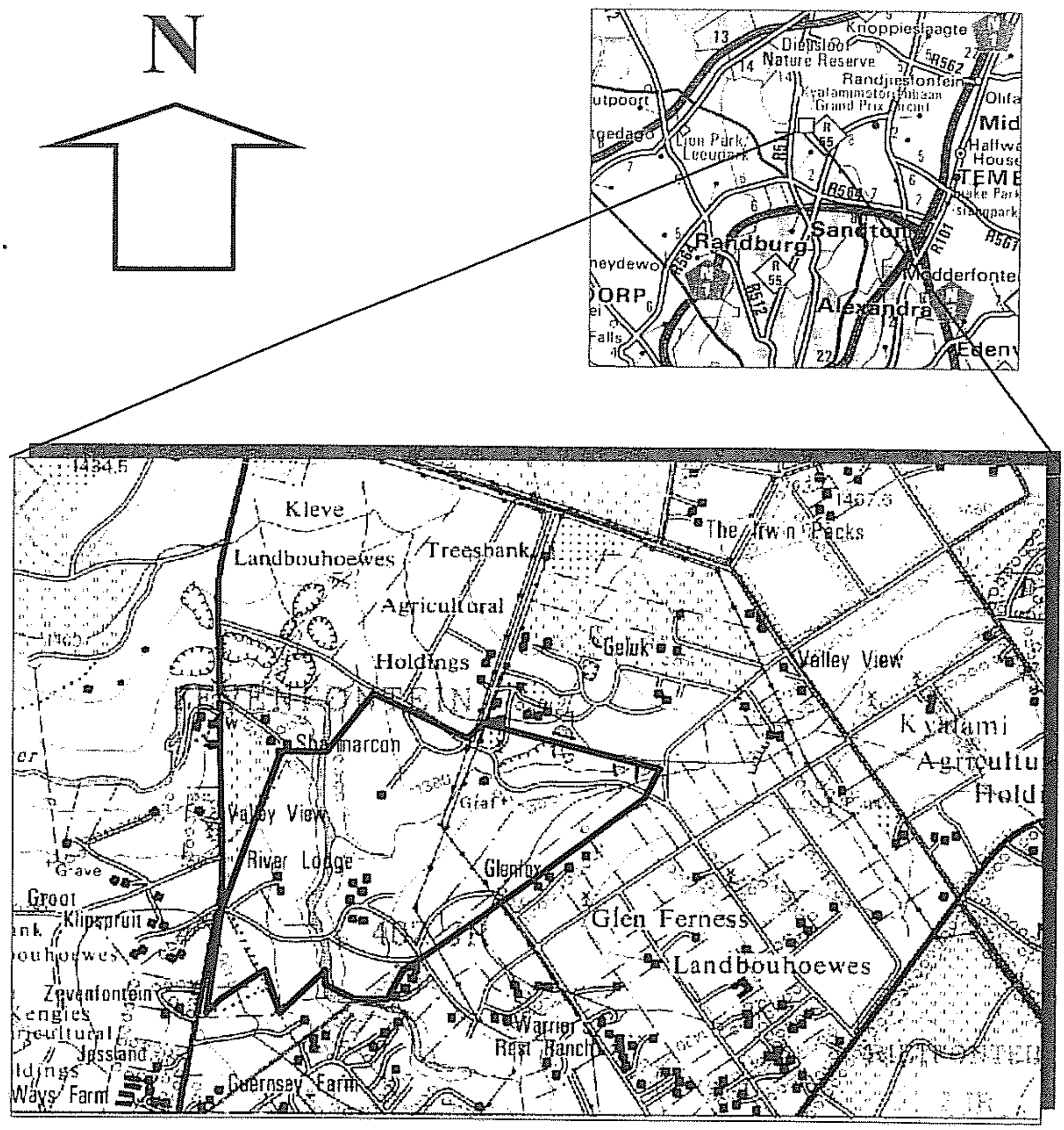
APPENDIX 1

LOCALITY PLAN



not drawn to scale

GEOPRACTICA	CARLISLE AND ASSOCIATES HELDERFONTEIN ESTATE, SANDTON. **Site Locality Plan.**	**Job No:** 02186 **Date:** May 2002 **Figure :** 1