Foundation design and construction challenges at the Rea Vaya Sandton BRT cable-stayed bridge

INTRODUCTION
The M1 cable-stayed bridge, currently under construction near the Marlboro Drive off-ramp in Sandton, forms part of the Rea Vaya Bus Rapid Transport (BRT) network, one of the largest projects ever undertaken by the City of Johannesburg. The bridge, which is built in partnership with the Johannesburg Development Agency (JDA), will provide vehicular and pedestrian access from Sandton, across the M1, towards Alexandra. Michael Pavlakis & Associates were appointed to carry out the geotechnical investigation and the design of the pile foundations for the bridge.

THE BRIDGE
The 271 m long structure extends from Katherine Street in Sandton, across the M1 highway and on to Lees Street in

Figure 1: Longitudinal section of the bridge with underlying geology
The cable-stayed section comprises a main span of 83 m crossing the M1, and a 39 m long back span provided with a central tension pier. Most remaining spans vary in length from 25–30 m. A 50 m high slightly inclined concrete pylon, comprising dual columns each having a base length of 6 m, carries the cable-stayed section. Mechanically stabilised earth walls form the transition from the abutments to the roads on either side of the bridge. Two ramp bridges provide pedestrian access from either side of the highway to the 3 m wide pedestrian section of the bridge deck, which has a total width of 13.35 m over the freeway.

**GEOTECHNICAL INVESTIGATION**

A two-stage geotechnical investigation was carried out, with the second stage related to a more extensive study of the pylon and back span piers – as the original bridge design envisaged the construction of a ‘normal’ bridge. Further drilling work was carried out at the early stages of construction. The exploratory work included:

- The drilling of a total of 15 rotary core diamond drill boreholes down to depths varying from 15–35 m, as part of a geotechnical investigation that included other bridges along the route.
- Two 800 mm auger holes drilled to ‘refusal’ of a relatively weak Soilmec auger rig. Refusal near the pylon occurred at 15.1 m.
- Standard penetration tests (SPTs), uniaxial compressive strength tests with strain measurements and point load index tests on rock core.
- Laboratory index, consolidation and shear box tests on undisturbed and remoulded soil samples, and chemical analysis of groundwater samples.

**Figure 2: Typical pressuremeter test curve**

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BH.18
Depth = 8.5 m
Em = 100.1 MPa
P*L = 7.2 MPa
Em/P*L = 13.9
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**Construction of the cable-stayed main span between the pylon and Pier 4**
A series of Menard pressuremeter tests (PMT), carried out mostly at 1 m depth intervals within the pylon and back span section of the bridge for a more accurate appreciation of the engineering behaviour of the various formations encountered. The tests were carried out using the latest pressuremeter equipment.

Typical results of a pressuremeter test carried out within very dense residual granite, tending towards very soft rock, at the depth of 8.5 m, are shown in Figure 2. The pressuremeter modulus $E_m$ is related to the stiffness of the soil or rock tested while the net limit pressure $P^*$ is a parameter that can be used to compute shear strength and bearing capacity.

**GROUND CONDITIONS**

The simplified geology along the bridge route is shown in an idealised geological section in Figure 1.

The site is situated within the 3 200 million-years-old Johannesburg granite-gneiss dome, part of the Basement Complex, at an average elevation of 1 600 m AMSL, in a region of annual water surplus. It occupies part of a gentle rise that begins at Sandspruit to the west (where another bridge, part of the same project, is under construction) and reaches a maximum elevation near Louis Botha Avenue, from where it falls eastwards towards the Jukskei River.

The granites would thus be expected to be deeply weathered, and they are indeed so, with the residual soils, comprising orange-brown silty and gravelly sands (mostly ancient soils that probably formed on the African erosion surface over 100 million years ago), extending down to depths of 6–8 m. Typical strength parameters from shear box tests are listed in Table 1. Below these depths, the granite rock has undergone extensive differential weathering, which has resulted in large variations in the subsoil conditions along the bridge route. Thus, the west abutment and Pier 1, as well as Pier 6 and the east abutment, are underlain by competent granite bedrock of medium-hard rock or hard rock quality at the depths of 6–11 m, while within most of the remaining bridge route, including the cable-stayed section, severe weathering has taken place down to average depths of 23–25 m.

<table>
<thead>
<tr>
<th>Borehole number</th>
<th>Depth (m)</th>
<th>Density (kg/m$^3$)</th>
<th>UCS (MPa)</th>
<th>Secant modulus E (MPa) (at 50% UCS)</th>
<th>Strain at failure (%)</th>
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<tr>
<td>BH.18</td>
<td>8.7</td>
<td>2 250</td>
<td>0.1</td>
<td>0.3</td>
<td>0.13</td>
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<td></td>
<td>11.3</td>
<td>2 150</td>
<td>2.8</td>
<td>0.7</td>
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<td></td>
<td>14.9</td>
<td>2 400</td>
<td>7.4</td>
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<td></td>
<td>17.7</td>
<td>2 330</td>
<td>1.6</td>
<td>0.2</td>
<td>0.80</td>
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<tr>
<td></td>
<td>20.8</td>
<td>2 380</td>
<td>4.9</td>
<td>0.2</td>
<td>1.60</td>
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**Table 1: Shear box test results on undisturbed and remoulded samples of residual granite**

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<thead>
<tr>
<th>Borehole no and Auger hole no</th>
<th>Depth (m)</th>
<th>Dry density (kg/m$^3$)</th>
<th>Moisture content (%)</th>
<th>Cohesion C' (kPa)</th>
<th>Friction angle $\varphi'$ (degrees)</th>
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<td>Undisturbed samples</td>
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<tr>
<td>BH.17</td>
<td>3.05</td>
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<td>BH.19</td>
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<td>17.7</td>
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<td></td>
<td></td>
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<tr>
<td>AH.K1</td>
<td>4.0</td>
<td>1 355</td>
<td>12.6</td>
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<td>8.0</td>
<td>1 487</td>
<td>8.3</td>
<td>22</td>
<td>30</td>
</tr>
</tbody>
</table>

**Table 2: Typical results of uniaxial compressive strength tests on weathered granite rock core**

![Figure 3: Some pressuremeter test results showing variation of pressuremeter modulus with depth]
where the competent, slightly weathered or unweathered granite rock is found. Hard, coarse pegmatite bands are present in places, mostly in the pylon area. The results of uniaxial compression tests on weathered granite rock core, showing the significant variation of strength with depth in a single borehole, are shown in Table 2. The groundwater table is present mostly within the depth range of 3–8 m.

A narrow diabase dyke that intruded the granites, was identified during construction at the position of Pier 4, near the eastern edge of the M1. The pier is situated in an area that was covered by the old pedestrian bridge access ramp, and some 8–10 m east of the exploratory borehole drilled, with the edge of the dyke being only a few metres away from the nearest borehole. Trial holes and an additional rotary core borehole drilled at the centre of Pier 4 indicated that the diabase comprised a clayey-sandy silt containing large 300–1 500 mm very hard rounded boulders (weathering spheroids), extending down to the level of the hard diabase bedrock at a depth of 9 m. Pressuremeter tests were also carried out in the borehole to determine the engineering properties of the residual diabase.

**DESIGN CHALLENGES**

Although the decomposed granite was mostly dense within a few metres from the surface, in certain sections it was found to be less competent and medium dense down to depths of 3–5 m. The heavy imposed foundation loads, and bridge performance requirements, necessitated the use of pile foundations. The design challenges included the following:

- The proximity of the pylon to the edge of the highway, requiring the minimisation of the size of the foundation; associated excavation depths would also have to be minimised to avoid possible destabilisation and/or cracking, and possible disruption of the M1.
- The presence of the high groundwater table, making it difficult to manually clean the base of large diameter piles, despite the slow flow rates.
- The highly variable depths to competent bedrock, and the variable engineering characteristics of the residual granite.
- The presence of soft rock bands and occasional spheroids within the less competent very soft rock granite, which caused ‘refusal’ of the drilling rig at geotechnical investigation stage, at the depth of 15–18 m, above the proposed founding level of the pylon and some piers.
- The presence of hard rock pegmatite bands causing further difficulties, especially in the construction of rock sockets.
- The diabase intrusion, which affected the foundation of Pier 4 and some foundations of the eastern pedestrian ramp bridge; the residual diabase contained large hard boulders (spheroids) ‘floating’ within the relatively incompetent decomposed diabase, making it difficult to pile through to bedrock with usual piling equipment.

**THE PYLON**

The pylon foundation consists of a single pile cap 12.6 x 16.2 m in size, supported by a pile group comprising 20 bored (augered) cast-in-situ piles to resist the large imposed pylon loads. The 1 200 mm diameter bored piles were socketed a minimum of 2 m into the hard granite bedrock or very hard pegmatite at the depths of mostly 25–27 m.

All piles were drilled by a Casagrande B180 auger rig, while the sockets were constructed by a purpose-made core bucket using the same rig. Approximately one half of the piles intersected very hard pegmatites at depth, making it difficult to form the rock sockets, each of which often took more than a day to complete.

The large-diameter shafts, which carry serviceability limit state axial loads of 7.5 MN each, provide the necessary stiffness and robustness to limit pylon foundation settlement and tilt movements to acceptable levels. They also served to minimise the foundation size and to limit excavation depths. Even so, the edge of the pile cap is only some 5 m distant from the edge of the M1. The piles were provided with moderate rake of 1:8.

In carrying out pile group analyses, much reliance was placed on the results of the pressuremeter tests (PMTs), which provided a near-continuous record of the soil/rock engineering properties, from the surface to a depth of 33 m (information from the SPT tests was limited due to early ‘refusal’ at depths of 5–8 m, while, due to the presence of joints and fractures, much of the weathered rock core was not suitable for laboratory testing). These were also particularly useful in establishing the resistance of the piles to large applied horizontal loads, as the test models this condition directly.
An important issue was the cleaning of the base of the pile shafts to ensure adequate end bearing, which would serve to increase the factor of safety against overall bearing failure to the required value. Although groundwater seepage rates were very slow, the long duration of the pile-forming activities allowed the water to enter and fill the pile holes up to the groundwater table. Pumping of the water and manual cleaning of the pile holes, with personnel working inside a steel casing, was considered, but this was deemed to be difficult and unsafe at the depths under consideration. It was therefore decided to clean the base of the pile holes by means of airlifting. This worked well, due to the high groundwater table, as evidenced by the probing of the pile hole bottom after airlifting. All pylon large-diameter shafts were airlifted and concreted immediately afterwards by tremie. They were also provided with 50 mm diameter steel tubes to enable concrete quality checking by cross-hole sonic logging.

BACK SPAN AND INTERMEDIATE PIERS
The back span and intermediate piers are supported on 1 050 mm diameter bored piles founded at a depth of 20 m. They are required to resist a total uplift force of 18 MN (SLS), in addition to other substantial imposed loads. Tensile restraint is achieved by installing stressed rock anchors through 200 mm diameter steel sonic tubes installed within the piles. The anchor-fixed length was installed within the hard rock granite below the depth of 24 m. The anchors incorporated 32–40 mm diameter SAS950 threadbars, provided with double corrosion protection.

REMAINING BRIDGE PIERS AND PEDESTRIAN RAMPS
All the remaining piers, as well as the abutments, were supported on pre-drilled 610 mm diameter driven cast-in-situ (DCIS) piles. Pad footings were also possible, but these would have resulted in large and deep excavations in Katherine and Lees Streets, effectively necessitating the closure of those roads. Consideration was also given to using other pile types, including augered cast-in-situ and continuous flight auger (CFA) piles, but these were not practical or economical due to the following reasons:

- The variable subsoil conditions and difficulties in controlling the depth of founding, unless taken to ‘refusal’, which would have been uneconomical for certain areas
- The high groundwater table, the possibility of local pile hole sidewall collapse and the difficulty of cleaning the base of the auger piles
- The pressure of alternating hard and soft layers to large depths, and occasional boulders which could cause premature refusal of the proposed relatively small diameter piles in the case of both auger and CFA piles.

Each of the DCIS piles were founded at the fixed depth of 8 m below natural ground surface. They were founded within dense to very dense gravelly sand (residual granite), and occasionally granite rock, to carry a serviceability limit state axial load of 2.1 MN. The piles were provided with a 1:4 rake, which assists in stiffening the pile cap.

PIER 4: DIABASE INTRUSION
Pier 4, which is situated near the eastern edge of the M1, is underlain by a 20 m wide diabase dyke striking in a roughly north-south direction, perpendicular to the bridge centre line. The dyke contained an abundance of very large, hard, up to 1 500 mm spheroids/boulders which prevented the installation of the DCIS piles. The boulders were embedded within firm, mostly saturated clayey silt and extended to hard diabase bedrock at a depth of 8.5–9.0 m. A suitable piling rig to penetrate the boulders would be costly and would take months to organise. Consideration was also given to supporting the pier within the residual diabase, but this was rejected due to the random occurrence of the boulders, which could give rise to significant foundation tilt, depending on their configuration. Permissible foundation settlements would also be much smaller than values that would normally apply, as the adjacent pier was piled and would not settle significantly. Therefore, any settlement that the Pier 4 foundation would undergo, would be mostly differential.

An alternative was investigated, comprising excavation to bedrock and mass filling with weak concrete to a suitable level, where a normal concrete footing could be constructed. This would, however, require a steep excavation, as the centre of the pier was only 15 m from the edge of the highway. Strength data for analysing the excavation was obtained from the pressure-meter tests performed in a borehole drilled at the centre of the...
base, which indicated an undrained shear strength of 79 kPa, and 2 SPT tests (N-value = 16–17).

A drained analysis, based on estimated strength parameters, indicated that the excavation would be unstable if left standing for a significant period, bearing in mind the high water table at that position. An undrained analysis, however, based on the measured strength data, indicated a short-term factor of safety of 2.3 for a benched excavation cut at the steep angle of 70° below the depth of 2 m, and an average slope angle of about 60°, as shown in Figure 4. It was therefore decided to proceed with this solution, on condition that the excavation would be strictly and continuously monitored for stability.

During the final stages of excavation, a local collapse occurred, associated with the high moisture content and relative looseness of the trench backfill, linked to an old sewer pipeline. The sewer line was present at the edge of the excavation closest to the highway, and could not be diverted. The overall excavation, however, proved to be stable, with the undamaged sewer line visible at its northern edge. The entire operation was completed, and the weak concrete mass fill was placed, within a period of three days. The deeper section of the excavation (below 5 m) and the pouring of the mass concrete fill took two days, with the second day stretching to close to midnight, by which time the remaining excavation for the pier footing was much shallower and safe in the longer term.

PILE TESTING, SONIC LOGGING AND ANCHOR PULL-OUT TESTS

Taking into account the importance of the structure, a comprehensive programme of static pile load tests (maintained load tests) was implemented in order to confirm design parameters. This comprised the testing of two bored trial piles 900 mm and
1 200 mm in diameter, with the latter loaded to 15.7 MN (just over twice the working load), and of three DCIS trial piles along the route of the 271 m long bridge. The measured load-settlement behaviour of the piles proved to be close to that predicted, validating the design parameters.

Two working DCIS piles were also tested up to 150% of working load, and the results also proved to be satisfactory.

Sonic logging was implemented as part of the quality assurance process. The test proved reliable, picking up a fault in one of the pylon piles (highlighted by the contractor before testing) very
close to the suspected problem depth.

Three 40 mm diameter thread bars were grouted within the granite rock within the depth range of between 23–30 m and subsequently tested to their ultimate tensile strength. The results confirmed the high frictional bond developed between the non-shrink cement grout and the hard granite rock.

CONCLUSIONS
This article highlights some of the geotechnical challenges associated with building in the Basement granites in the Sandton area, and the importance of an adequate and fit-for-purpose geotechnical investigation. The principle that rock becomes less weathered and more competent with depth does not necessarily hold (at least within the normal founding depth range), nor the assumption that if the ground conditions are the same at two relatively close positions, these same conditions will apply between them. The pressuremeter testing work carried out, in combination with other routine SPT and laboratory tests, enabled a firm appreciation of the engineering behaviour of the soils and rocks underlying the site, leading to more economical and reliable foundation design. Experienced contractors, coupled with close supervision during construction, and synergy between all parties in working towards the common goal, serve to ensure quality of construction. An allowance for a comprehensive geotechnical investigation programme ensures safe and economical foundation design, and minimises construction difficulties and expensive delays.

ACKNOWLEDGEMENTS
The authors wish to thank the Johannesburg Development Agency for permission to publish this article, as well as the highly motivated design teams, consulting engineers Royal HaskoningDHV & Hatch Goba, and construction teams, WBHO & Terrastrata. The commitment of the entire project team to quality and safety has enabled the efficient resolution of the challenges presented by the ancient granitic environment in which the bridge is situated, at below budgeted cost, and assisted in providing a solid foundation to a structure that is destined to become an important landmark on Johannesburg’s skyline.

REFERENCES