Rock socket capacity of bored piles in weathered granites from internal jack tests

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ABSTRACT: Large diameter bored piles were installed for the new viaducts A0293 and B0293 constructed at Keyridge Realignment of National Route 3, Section 2 from Key Ridge to Hammarsdale, South Africa. Two (2) full-scale internal jack pile load tests were undertaken to assess rock socket behaviour for a socket diameter of 1.35m. The tests were conducted in variable strength and fractured Archaean Granite bedrock with an RQD of 40% or less and intact UCS strengths ranging from 50 to 150 MPa. The rock strengths were classified by SANRAL Geotechnical Guidelines as rock strength varying from R3 to R5. Methods of rock socket capacity evaluation were developed mostly for sedimentary rocks. The method of Horvath and Kenney (1979), appears to produce excellent correlations when compared to actual rock socket capacity data obtained from instrumented internal jack pile load tests. This paper reports on the findings of the rock socket friction capacity for large diameter bored piles based on two instrumented internal jack tests and compared with predicted rock shaft resistance based on literature.

Symbol	Description
RQD	Rock Quality Designation (%)
UCS	Unconfined Compressive Strength (MPa)
τ	Rock socket friction capacity (kPa)
D	Diameter of the bored pile (m)
L	Socket length of the pile in bedrock (m)
Q	Applied load during jack test (kN)
σ	Stress in the rock socket (MPa)
fs	Predicted shaft resistance (kPa)
Ep	Modulus of elasticity of pile (MPa)
Er	Modulus of elasticity of rock (MPa)
ϕ	Friction angle of rock socket interface (°)

1 INTRODUCTION

The N3 corridor from Durban to Gauteng in South Africa is currently undergoing a significant upgrade aimed at enhancing transportation capacity across the region. This project targets explicitly Section 2 of the N3, extending from the M13 Interchange (No. 35 – Hillcrest/Durban) to the bottom of Key Ridge at the Hammersdale Interchange (No. 43 Hammersdale/Inchanga). The proposed upgrade involves the construction of a dual carriageway that will feature a total of five lanes: two slow lanes, two fast lanes, and a dedicated truck lane in each direction, thereby facilitating improved traffic flow and accommodating the growing demands of the transport network. The project is located within the Outer West municipal area of the eThekwini Metropolitan region, specifically between the coordinates 29°46'16.89" S, 30°41'31.21"E (Cliffdale), and 29°45'51.14"S, 30°40'48.20" E (Drummond) in Durban, KwaZulu-Natal, South Africa. Additional structures, including viaducts, are indicated in Figure 1 extracted from Terratest (PTY) LTD (Dralle 2020).

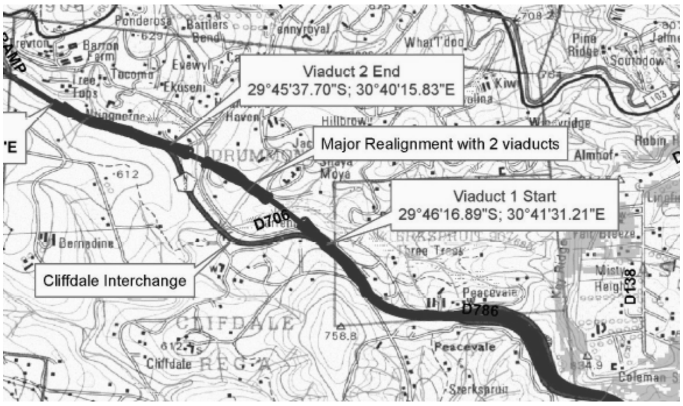


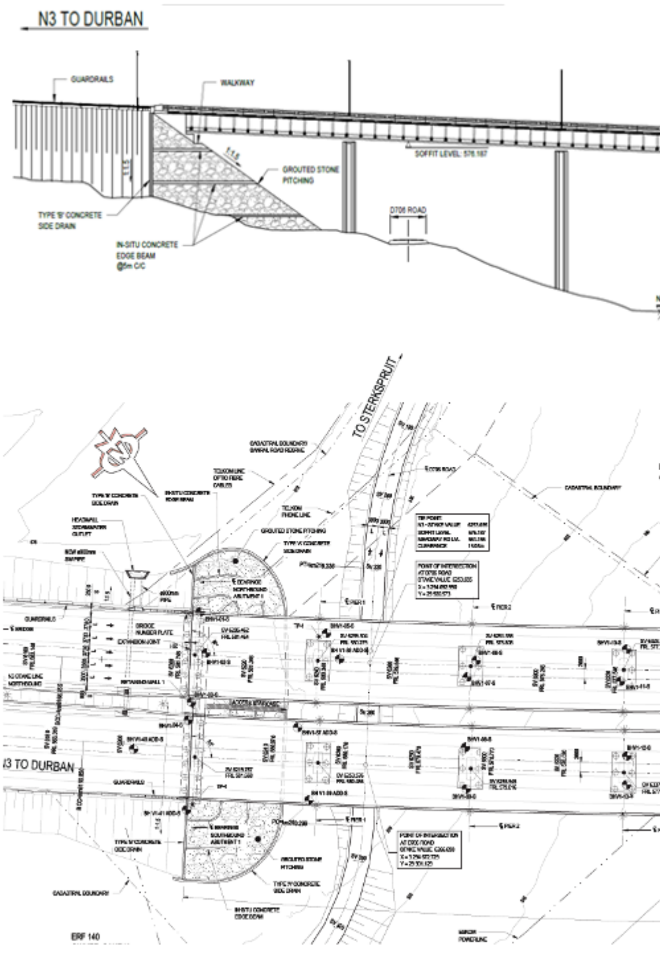
Figure 1. Locality of the Proposed N3 Key Ridge Upgrade Project on National Route 3

This eastern region of the N3 corridor is characterized by a wet climate and exhibits a Weinert value N-value of less than 5, indicating that chemical degradation is the predominant process affecting the area. This classification is determined by the weathering patterns observed in the natural road materials present in the region. These weathering patterns are an important factor to consider when assessing the geotechnical properties of the underlying materials, particularly as they influence the stability and performance of the proposed pile foundations for the project (McDuling et al. 2008).

As part of this study, static load tests were conducted to assess the performance of installed piles, particularly those used for bridge abutments. The tests involved top-loading the piles with a steel beam, incorporating reaction piles to monitor settlement and load-bearing capacity. The results indicated minimal settlements, recorded as less than 0.5% of the pile diameter. However, this paper specifically focuses on isolating and evaluating the rock socket friction capacity of the piles. For the internally jacked piles, the load was applied by incorporating a hydraulic jack into the shaft of the pile. When activated, the jack applied bi-directional forces to the pile shaft above and below it, similar to the O-cell test developed by Professor Osterberg. The high-capacity (10MN to 20MN) bored piles, with a rock socket diameter of 1.35m and an upper pile shaft diameter of 1.5m, were installed using oscillators. These oscillators rotated through 20 degrees to advance the casing down the pile shaft, while an auger was used periodically to remove spoil from the casings. The high-performance Bauer Rigs (BG 28 and BG 36) were employed to facilitate the installation process, ensuring the efficiency and precision of the piling operation (Dralle 2020).

The construction of the viaduct involves passing through various types of rock, including Archean granites, which are found in the Natal Metamorphic Province. This paper focuses specifically on deep bored piles that range in depth from 18 to 37 meters, which are socketed into the Archean granites common to the region. These high-capacity bored piles are the primary support for Viaduct B0293. Figure 2 attached illustrates the general arrangement and elevations of the viaduct.

This paper aims to demonstrate the applicability of available design relationships that address the shaft resistance of decomposed rock, specifically focusing on determining the maximum rock shaft resistance for piles socketed into Archean granite formations. These formations, as investigated from borehole data to a depth of 39 meters below the surface, present a unique set of challenges. In this context, a database comprising of seven load tests of rock-socketed shafts has been compiled to validate existing design relationships and to identify trends in the behavior of the rock's unconfined compressive strength (UCS). The UCS values ranged from 8.9 MPa to 105.5 MPa for TP1 and 12.5 MPa to 80.8 MPa for TP5. The variability in UCS across the pile diameter underscores the challenges in accurately predicting rock behavior and designing for optimal rock socketing. Given that rock quality may be lower than anticipated, it is crucial to establish an effective relationship for rock shaft resistance to prevent excessive rock socketing. The shaft resistance of bored piles socketed into granite rock has not been extensively studied, which complicates efforts to determine the potential skin friction for piles constructed in fractured igneous rock formations can carry (Dralle, 2020).



(Courtesy of BVI Consulting Engineers)

Figure 2. Elevation and general arrangement of the Viaduct.

2 GEOTECHNICAL CHARACTERISTICS OF THE STUDY AREA

2.1 *Geotechnical Context and Site Conditions*

The geotechnical conditions at the site are highly variable, dominated by fractured, weathered Archaean granite. Geotechnical investigations at various test sites involved drilling rotary core boreholes to depths reaching up to 39 meters, revealing significant variability in rock quality as shown in Figure 3.

At depths between 37.68 meters and 39 meters, the granite displayed high UCS values, confirming its potential as a load-bearing material. In contrast, the underlying strata exhibited significantly reduced UCS values, indicating weaker rock with increased fracture frequencies.

The Rock Quality Designation (RQD) at this depth improved to 62.67%, accompanied by a reduced fracture frequency of 9 fractures per meter, indicating the presence of more intact rock. However, below 39 meters, the Unconfined Compressive Strength (UCS) measured 14.2 MPa between 41.4 and 41.7 meters and decreased to 8.9 MPa between 42.2 and 42.6 meters for test point TP1. During this depth range, RQD values fell below 25%, and the fracture frequency increased to more than 20 fractures per meter. This variability in rock properties highlights the importance of thorough geotechnical evaluation and careful selection of pile termination depths to ensure structural stability (Dralle 2020).

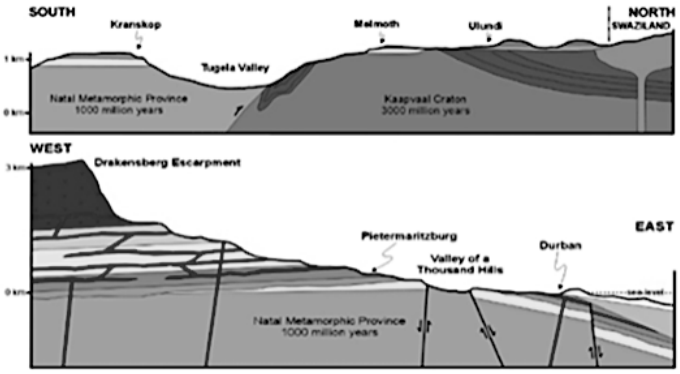


Figure 3. Geological cross-section from Kranskop to Swaziland (top) and from the Drakensberg Escarpment to Durban (bottom)

Two instrumented internal jack tests were conducted at test pile positions TP1 and TP5. The corresponding representative borehole profiles outlined in Table 1 summarize the rock mass socketing profiles for TP1 and TP5 as follows:

Table 1. Variability in rock properties underscores the need for precise geotechnical evaluation.

TP1			TP5		
Depth (m)	RDQ (%)	Fracture frequency	Depth (m)	RDQ (%)	Fracture frequency
39	62.67	9	28.36	44	12
40.50	63.33	15	29.66	84.3	6
42.00	46.67	>20	31.36	100	0
43.50	23.33	>20	32.36	100	0

2.2 *Natal Metamorphic and Structural Province*

The Mapumulo Metamorphic Suite rocks, characterized by biotite-hornblende gneiss, are found in a region south of the Umgeni River. The Natal Meta-morphic Province's Unnamed Rocks, massive, foliated granitic rocks, are found in a region near the Umgeni River. The easternmost fault zone is located at approximately km 3.70 to km 3.90, while the greatest fault zone is further west at km 6.50 to km 6.60. At 75° to the southeast, the primary group of discontinuities in the rock matrix is orientated steeply. (Clarke 2008).

2.3 *Geological Context (Archaean Granite Bedrock)*

Figure 4 below shows samples of weathered R3 and slightly weathered R4 Archean granite rock that was recovered from the pile bore.

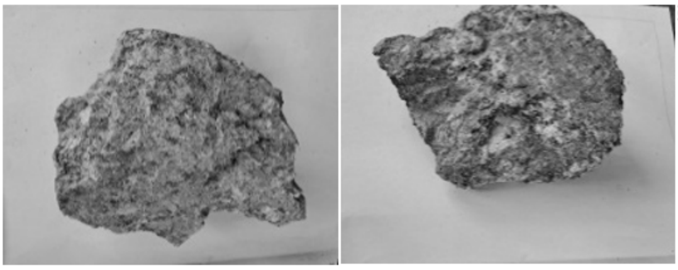


Figure 4. Variety of granitic rocks that are over two billion years old and originate from the Archaean

There was a close resemblance of Alluvial sands and sands residual from Granite bedrock South Africa's upper soils are characterized by a variety of granitic rocks, over two billion years old, originating from the Archaean and Vaalian Eras. Most weathering processes are chemical, with Weinert N values less than 5. Upper soils typically consist of Alluvial medium to coarse-grained sands with high SPT N values or dense quarzitic coarse to medium-dense gravelly sands (Dippenaar 2014).

3 CONSTRUCTION METHODOLOGY

3.1 *Description of Pile Installation Techniques Boreholes (drilled)-Piles -position*

Conventional drilling tools are limited by vertical thrust and proven to be uneconomical in rock strengths exceeding 100 MPa. However, in certain rocks with compressive strengths around 50 MPa (or more), alternative drilling techniques can be used to achieve more economical production rates (Larisch, 2012). On this contract, in most of the piling works, a rock coring bucket was employed to form the 1,4 m rock socket. Casings were advanced by oscillator action, and trailing casings were inserted whilst material in the casings was removed using an auger with a Kelly bar attachment, as shown in Figure 7.

Due to the varying conditions encountered during drilling, the planned pile depths were increased during installation. Contact core drilling was undertaken on every pile installed and was inspected by the geotechnical engineer and bridge engineers.

Figure 5. Bauer BG 36 Piling rig

Figure 6. Spoil removal from casing

3.2 *Conventional design of pile shafts in rock*

The design of piles that are socketed into rock is traditionally based on local knowledge gained from observing full-scale static load tests, as well as empirical factors related to the unconfined compressive strength of intact rock and conservative regulations set by cities or states (Seidel & Haberfield 1994). Several researchers have proposed empirical correlations between the uniaxial compressive strength of weak rock and the unit side resistance of socketed piles as measured in load tests. One widely used empirical model is described by the equation:

$$f_{su} = \propto q_u^{\beta} q_u \qquad (1)$$

Where f_{su} is the ultimate socket shaft resistance, $q_u$is the uniaxial compressive strength of the weaker material (in most cases rock), and alpha and beta are determined empirically from load tests shown in Table 2 below (Seidel & Haberfield 1994).

Table 2. Design parameters according to different design methods

Design Method	α Value	β Value
Horvath and Kenney	0.21	0.50
Carter and Kulhawy	0.20	0.50
Williams et al.	0.44	0.36
Rowe and Armitage	0.40	0.57
Rosenberg and Journeaux	0.34	0.51
Reynolds and Kaderabek	0.30	1.00
Gupton and Logan	0.20	1.00
Reese and O'Neill	0.15	1.00
Toh	0.25	1.00

Kulhawy et al. (1995) summarized design parameters proposed by researchers, highlighting their limitations in large databases. These relationships were developed for specific data sets, but their limitations became apparent when examining pile sockets at various sites and rock types (Kulhawy et al. 2005).

Rock socketed piles can be drilled using both analytical and empirical approaches. However, drilling large-diameter shafts in hard rock at depths exceeding about 10m can cause difficulties and affect rock properties, reducing bearing capacity and increasing settlement (Rezazadeh et al. 2017). Empirical methods estimate shaft friction using the unconfined compression strength of the rock. The capacity of rock socketed piles is primarily dependent on shear resistance developed at the interface between the pile shaft and surrounding rock. Current pile design methods account for rock strength, but the influence of rock type and construction procedures remains unexplored. Decisions are based on experience and test pile logs seen on the table below (Haberfield et al. 1996).

3.3 *Basic Literature Review of The Tests*

Using these theoretical equations ($q_s = b\sigma_c^{0.5}$*),* with b in the range (0,2–0,25). The method of Horvath and Kenny (1979) for ‘large’ diameter piles, was used presumably for diameters greater than 410 mm leading to $q_s = 0.2\sigma_c^{0.5}$ .Rowe and Armitage (1987) recommended a correlation between regular ‘clean’ sockets, and $q_s = 0.6\sigma_c^{0.5}$ for the initial estimation of side resistance in clean ‘rough’ sockets based on the results of many field load tests (Kodikara et al. 1992).

4 PILE LOAD TEST

4.1 *Bi-directional*

Osterberg cells (O-Cells) are widely used for bidirectional load tests on high-capacity bored piles, particularly when high loads are applied. These hydraulic-driven devices apply load upwards against skin friction and downwards against end bearing individually or end bearing plus skin friction (Raja et al. 2021). The test load must be twice the design working load, with an ultimate pile capacity of 10500kN applied to the two sockets to minimize shaft friction and load applied to the rock socket (Raja et al. 2021).

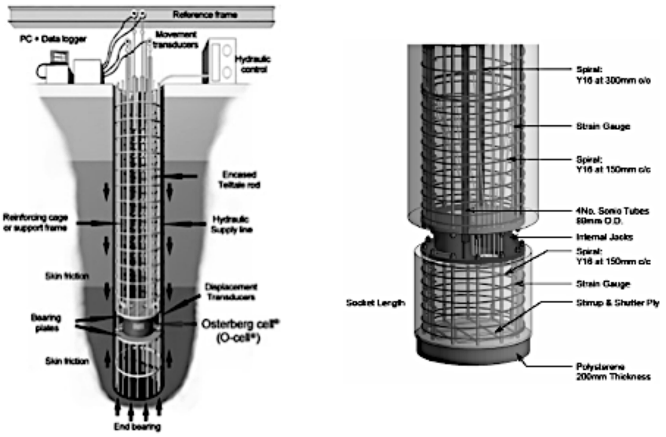


Figure 7. Typical O-Cell Test Setup on isometric Bottom Pile

4.2 *Typical Internal jack setup for TP1 and TP5*

The internal jack test setup involved placing a 200mm thick polystyrene "soft bottom" at the bottom of the reinforcing cage to test the side wall friction of the socket as shown in Figure 7 above. The test involved installing an internal jack bottom plate at a depth of 37.5m within a pile, with a socket and polystyrene below. Strain gauges and displacement transducers were used to measure movement data during the load test, corresponding R10 bars were placed 300mm up the reinforcing cage to protect strain gauge cables. The test procedure involved loading two 625 Mg hydraulic jacks applied 6125 kN into a pile, applying the load to a maximum of 100% of the working load (8500 kN). The load cycle was divided into four cycles, with the level of jack assembly determined by the consulting Engineers. Initial readings were recorded at 0 minutes, then at 10, 20, 30 minutes, and after 12 hours.

5 RESULTS AND ANALYSIS

5.1 *Interpretation Findings from Instrumented Internal Jack Tests*

The load-deflection curves analysed in the graphs below were derived from the studies conducted by Horvath & Kenny (1979) and Rowe & Armitage (1987), using Unconfined Compressive Strength (UCS) values on Table 2 below. The data collected during the load cycles have been plotted as X against Y and analysed according to Equations 1 and 2, $q_s = 0.2\sigma_c^{0.5}$ and $q_s = 0.6\sigma_c^{0.5}$ respectively.

Table 2. Summary of the UCS values used

	TP1	TP5
UCS(Mpa) values	80	12,5
	105	88,9

Measured load-deflection curves (Figs 8 and 9) indicated that TP1 demonstrated greater shaft resistance, resulting in minimal deflections under working loads.

5.2 *Rock Socket Friction Capacity Observations*

The results concentrate solely on the rock socket friction capacity, separating it from the contributions of shaft friction in the upper soil layers. This method guarantees an accurate evaluation of the frictional resistance at the pile-rock interface within the granite bedrock. It emphasizes the effects of Uniaxial Compressive Strength (UCS), Rock Quality Designation (RQD), and socket roughness on the mobilized capacity.

5.2.1 *TP1 Summary for the Element Deflection*

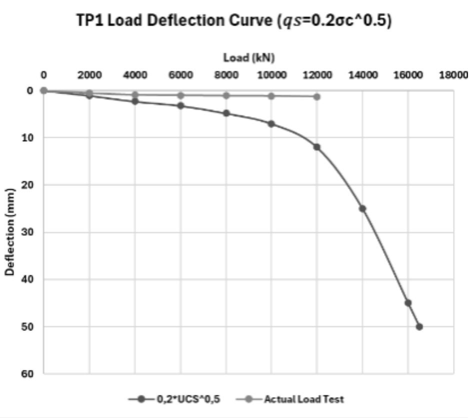


Figure 8. TP 1 - 0,6UCS ^0,5 Load deflection Curve

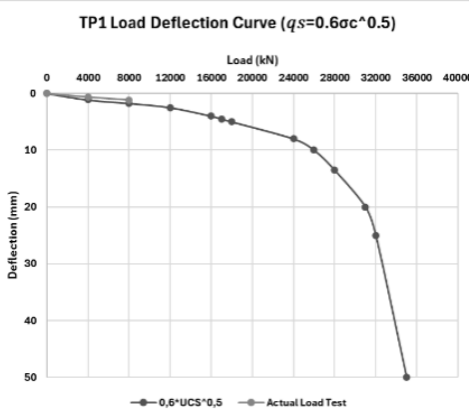


Figure 9. TP1 Summary of Element1 & 2 friction of 2054 kPa & 1788 kPa using 0.2(UCS)^0,5

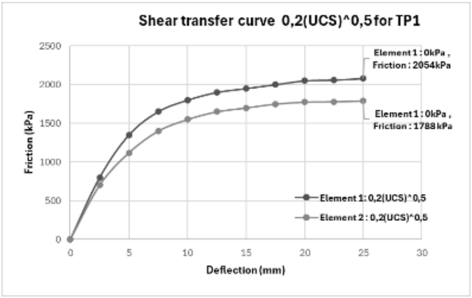


Figure 10. Summary of Element1 & 2 friction of 2054 kPa & 1788kPa using 0.2 (UCS)^0,5

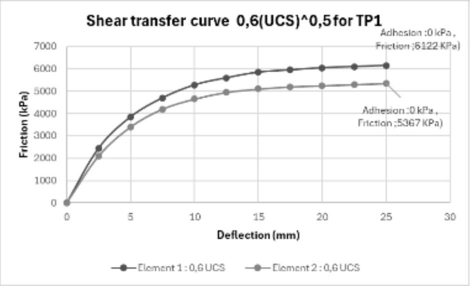


Figure 11. Summary of Element1 & 2 friction of 6122 kPa & 5367 kPa using 0.6 (UCS)^0,5

5.2.2 *TP5 Summary Results*

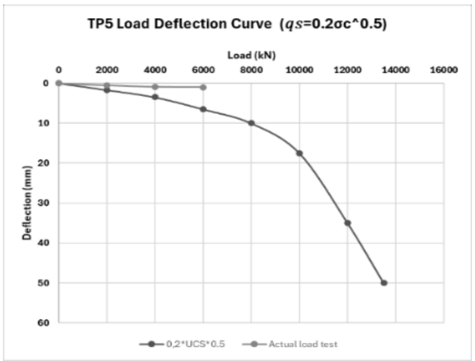


Figure 1*2.* TP5 0,6*UCS ^0,5 Load deflection Curve

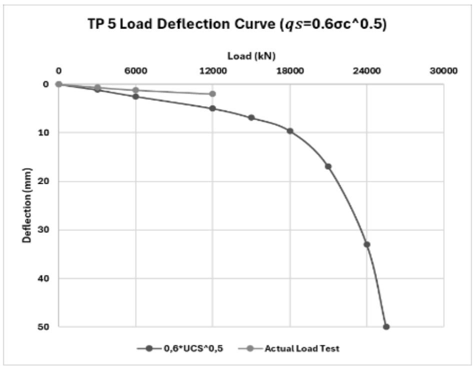


Figure 13. TP5 0,2*UCS ^0,5 Load deflection Curve

Load-deflection curves (Figures 12 and 13) above show higher deflections and reduced mobilized friction, correlating with weaker rock properties and higher fracture frequency.

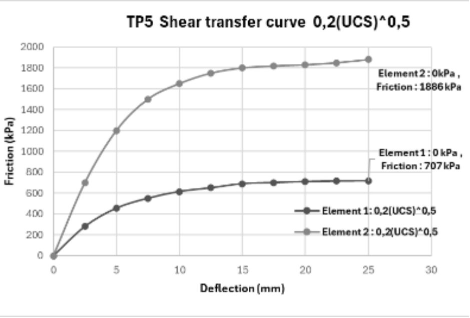


Figure 14. Summary of Element1 & 2 friction of 707 kPa & 1886 kPa using 0.2(UCS)^0,5

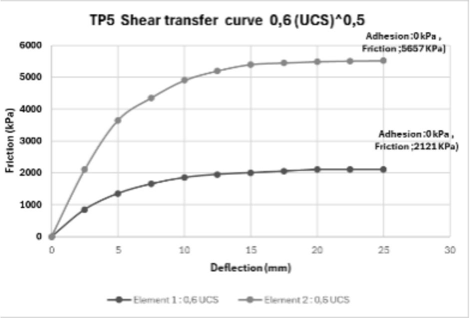


Figure 15. Summary of Element1 & 2 friction of 2121 kPa & 5657 kPa using 0.6(UCS)^0,5

The data readings of the test piles show the internal jack compared to the jack, top plate, and bottom plate for both the first and second load represented in graph form:

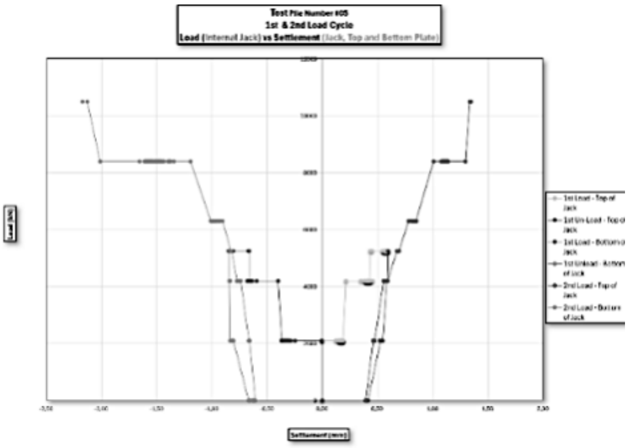


Figure 16. Typical load (internal jack) vs the settlements during jack, top and bottom plate

5.3 *Predicted vs. Measured Rock Shaft Resistance*

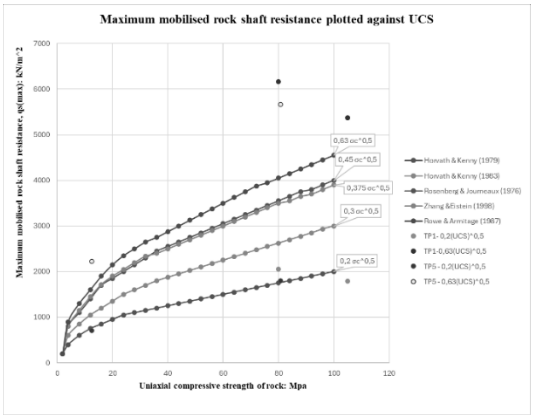


Figure 17. Maximum mobilized rock shaft resistance plotted against UCS

The maximum mobilized shaft resistance plotted against UCS (Fig. 17) confirms that TP1 consistently mobilized higher resistance due to its higher UCS and better RQD as compared to TP5's.

6 DISCUSSIONS

6.1 *Findings from Instrumented Internal Jack Tests*

The study examined the rock socket friction capacity of large-diameter bored piles in weathered Archean granite using full-scale instrumented internal jack tests. Results confirmed empirical relationships for predicting shaft resistance correlated with UCS values proposed by Horvath & Kenny (1979) and Rowe & Armitage (1987). The study highlighted the variability in rock properties across pile depth and the need for tailored geotechnical evaluations. The load tests showed higher friction capacities in TP1 due to higher UCS values and better RQD.

7 CONCLUSIONS

This research demonstrates that both empirical and analytical methods can reliably estimate rock socket capacity for large-diameter piles in Archean granite, provided local geological conditions are accounted for. The findings of this study emphasize that full mobilization of the rock socket shaft friction is assumed to occur at 0.5% of the pile diameter (1350mm). This is particularly significant when considering the interrelationships between unconfined compressive strength (UCS), rock quality designation (RQD), and construction methodologies concerning pile performance. The research further reveals that the end bearing capacity in both TP1 and TP5 is less significant than the shaft friction capacity. While the end bearing provided additional resistance, especially in **TP1**, the higher UCS and improved rock quality in this region allowed for better mobilization of end bearing forces under higher loads. However, it was the rock shaft friction, more than end bearing, that provided the predominant resistance.

The study validates the applicability of Horvath and Kenny's (1979) and Rowe & Armitage's (1987) models for granitic (igneous rock formations), emphasizing the importance of site-specific assessments. It highlights the sensitivity of pile capacity to rock quality, with enhanced socket roughness and UCS contributing to maximum friction capacity, as seen in TP1.

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