A Review on the Performance of Bored Pile Foundations through Instrumented Pile Load Tests in Qatar

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Abstract

The level of infrastructure development Qatar has accomplished over the past 10 years has not been reached by any other country and this pace is expected to continue to achieve Qatar's 2030 Vision. High-rise buildings and bridges are mostly constructed on pile foundations which play an important role in overall cost of the project, but there is limited published literature available on the performance of pile foundations in Qatar. In this article, the performance of fully instrumented pile foundations embedded in rock formations of Qatar is reviewed, and discussion on observed behaviour is presented. This paper also uncovers the conservatism in pile design adopted by the piling industry. A total of more than 65 pile load test results were compiled to perform this study. Good practices are suggested that can be considered by the authorities and consultants in specifying a preliminary pile load test in order to reap its full potential. In projects, where the unit skin friction was validated through performance of pile load tests and later on pile design was optimized, 11% to 50% savings in piling cost was achieved contributing to sustainable projects.

Keywords: Preliminary pile load test; Pile performance monitoring; Rock skin friction; Qatar geology; Design optimization.

1 Introduction

Qatar has experienced rapid development of infrastructure and construction of high-rise buildings and bridges over the last 10 years. High-rise buildings and bridges are mostly constructed on pile foundations which contribute significantly to the overall cost of the project. Qatar is currently ranked as 19th tallest city in the world and second in the Middle East by numbers of completed buildings higher than 150m. Over 77 high-rise buildings (of heights over 100m) and several bridges have been built and many buildings are still under construction or are in the planning stages. This indicates the piling industry in Qatar has gathered a vast amount of information in recent years.

However, limited published literature is available on the performance of pile foundations, particularly in Qatar. The aim of this paper is to share interpretation of compiled instrumented pile load tests with the industry to help colleagues understand the behaviour of bored pile foundations embedded in rock formations of Qatar.

1.1 Available Database

Pile load test records and reports were gathered from some piling projects completed in Qatar. Details of these projects are not disclosed in order to ensure the confidentiality of data used to prepare this article. The compiled data comprised 65 pile load test results including 28 preliminary pile load tests, which were conducted using load cells. Among the available pile load test results, only those tests were considered for this study that fulfilled the criteria below:

- Preliminary and working pile load tests performed using bidirectional load cells.
- The pile load test reports are available including pile details, coordinates, strain, and measured skin friction values.
- Site-specific geotechnical factual reports with boreholes are available at or adjacent to the test pile location.

1.2 Bi-Directional Static Load Test (BDSLT)

Detailed description and operating principles of BDSLT are outside of the scope of this study. BDSLT is faster to perform, incurs less cost, reduces construction time and provides direct measurements of the load distribution and pile movement response. Through BDSLT, much higher test loads can be applied than with conventional pile load tests. The cost saving from using BDSLT ranges between 1/3 and 2/3 of conventional top load tests and increases further with any increase in applied load (Schmertmann et al., 1997).

A bi-directional cell is composed of a hydraulic piston assembly welded through steel plates on either side of this sacrificial cell assembly through steel reinforcement. When hydraulic pressure is applied, the piston or hydraulic cell extends, and the load is transferred to the pile. The pile side (or skin) and pile bottom take the load and generate resistance from the ground surrounding the pile and at the pile toe. The load cell applies the load both upward and downward in equal parts. The upper part of the test pile provides resistance to the upward load through the skin friction. The lower part of the pile (below the cell assembly) provides resistance to the downward load through either end bearing alone (if the cell is installed at the pile's toe) or through combined resistance provided by skin friction and end bearing (if the load cell is installed above the toe of the test pile). Telltale rods cased inside a pipe are attached to the top and bottom of hydraulic cell to measure the upward and downward movements against the reference frame at the surface. The movements are measured using displacement transducers.

2 Geological Conditions

At depths relative to civil engineering, the main geological formations, in stratigraphic succession, consist of upper Dammam formation (Simsima Limestone member), Lower Dammam formation (Dukhan Limestone and Midra Shale member) and Rus Formation. The geological formations are comprised of limestone, shale, siltstone, claystone, marl and gypsum. The Simsima Limestone member is present over 80% of the land surface of Qatar. In general, pile foundations have been formed in Simsima Limestone member with average thickness of 20m. **Fig. 1** illustrates variation of UCS[1] encountered in all the project sites with elevation. Simsima Limestone varies from weak to strong, while Rus Formation is generally extremely weak to moderately strong.

[1] Unconfined compressive strength

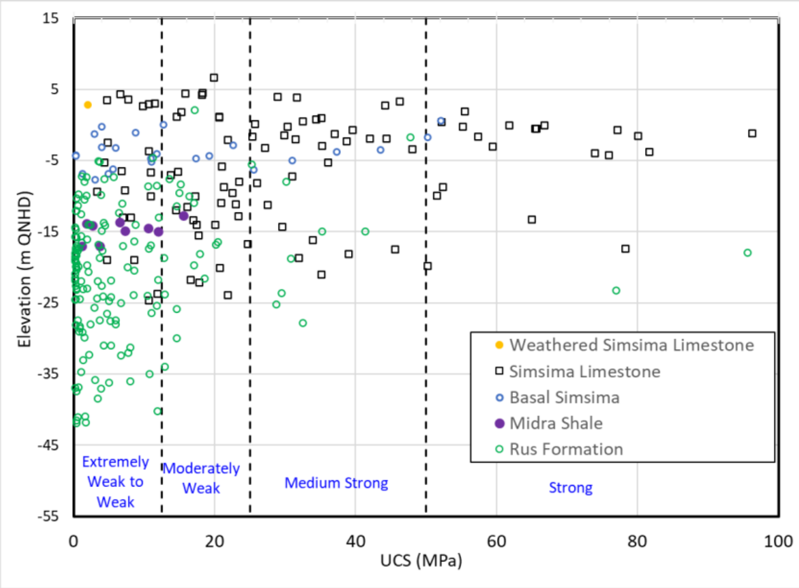

Fig. 1: Plot of UCS vs. elevation for the database

3 Empirical Correlations for Pile Design

The empirical correlations based on which piles are designed in Qatar relate the ultimate unit skin friction of pile with unconfined compressive strength (UCS) of rock considering correction factors for the rock discontinuities. In practice, the ultimate skin friction is estimated by relations provided by (Horvath et al., 1983) and (Williams and Pells, 1981).

$f_s = 0.3 \times (UCS)^{0.5}$	(1)	Horvath et. al (1983)
$f_s = \alpha\beta \times UCS$	(2)	Williams and Pells (1981)
$f_s = 0.40 \times (UCS)^{0.5}$	(3)	Zhang (1998)
$f_s = 0.375 \times (UCS)^{0.515}$	(4)	Rosenberg & Journeaux (1976)

Where,

f_s = Ultimate unit skin friction

UCS = Rock Unconfined Compressive Strength

α = Reduction factor with respect to UCS

β = Reduction factor with respect to rock discontinuities

4 Load Transfer Behaviour along the Trial Pile

Vibrating wire strain gauges are installed typically at predefined levels along the length of the trial piles to measure the strains. Load transfer (P) at each strain gauge level is calculated using the following equation. Subsequently, the mobilised skin friction is derived using the known pile stiffness. The computed skin friction at each level of strain gauge was obtained from the pile load test report.

$$P = \Delta_{SG} \times AE_p \qquad (5)$$

Where,

Δ_{SG} = Change in Strain Gauge reading

AE_p= Pile Axial Stiffness

It is well-known that the elasticity of concrete material does not remain constant over the range of stressing history during loading and unloading cycles but exhibits a strain-softening behaviour with increasing stress level. However, for this study, the pile stiffness used (AE) is based on the correlation of static elastic modulus (Ec) and cube compressive strength of the concrete (fc), as Ec = 4700 x $\sqrt{fc}$. The load carried by the pile is estimated by the difference between the loads calculated at any two levels of strain gauges. Consequently, the average unit skin friction is calculated between any two strain gauge levels by the change in calculated load divided by the circumferential area of pile between the two strain gauge levels.

Fig. **2** illustrates the load transfer behaviour along the test pile calculated from data collected by strain gauges installed along the length of the trial piles. The distance of each strain gauge above (d1) and below (d2) the cell level is normalized by pile length segment above (L1) and below (L2) the jack level. At every strain gauge, computed skin friction is available in the test report. It is obvious from the review of PTPs report data that the maximum load is transferred to the pile at the load cell level, thus, strain measurements close to the location of the load cell assembly provide the highest values of unit skin friction, irrespective of the ground conditions and the strength of rock. The values of measured skin friction gradually decrease as the distance to the load cell increases, as shown in Fig. **2**, and it is also observed that the transferred load to pile is reduced to about 90 to 97% at the level of strain gauges placed at 3/4th distance of pile length above and below the jack level.

This means that skin friction values measured in the zone of the load cell may represent the expected design values since within this zone the load being applied by the cell begins to be transferred to surrounding rock. Consequently, rock in this zone carries a major part of the test load that mobilizes the maximum skin friction of rock. Hence, the remaining test load may not be sufficient to mobilize the rock's maximum skin friction along the entire length of test pile. Therefore, in the authors' opinion, geotechnical failure in rock formations of Qatar may only be reached, locally, at the load cell location.

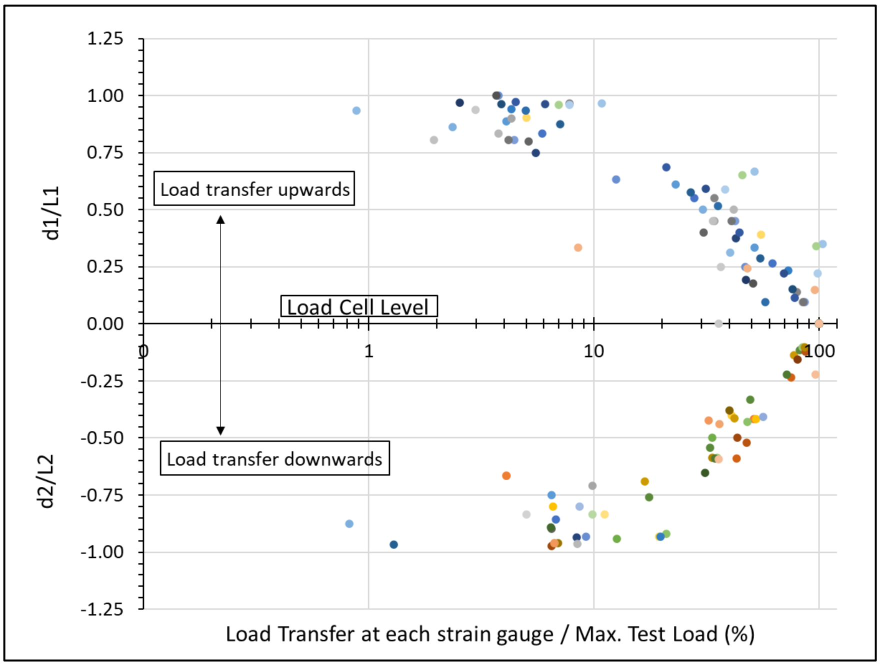

Fig. 2: Load distribution along the normalized PTPs length

5 Mobilized Skin Friction during Trial Pile Testing

Pile foundations in Qatar are usually designed in a way such that the geotechnical resistance is considered to be provided only by the skin friction and end-bearing is ignored. Hence, correct estimation of the unit skin friction becomes essential to get a cost-effective pile foundation system. **Fig. 3** presents ultimate skin friction values estimated by the empirical correlations commonly used in practice. This figure also shows mobilized skin friction achieved during the preliminary (trial) pile load tests. The unit skin friction measured within the load cell zone of the 27 selected PTP[2]s (as shown in **Fig. 2**) are the highest values measured during the tests. The rock UCS and RQD[3] values were taken from the boreholes drilled in the close vicinity of the each PTPs location.

The skin friction values near the cell assembly placed within the extremely weak to weak rock (as classified in accordance with BS EN ISO 14689) are considered to be fully mobilized exceeding the ultimate skin friction designed using empirical relationships. The approaches by Horvath et al (1983) and Williams and Pells (1981) may underestimate the ultimate skin friction that can be mobilized within extremely weak to weak rock (rock mass with UCS < 12.5 MPa[4]). However, empirical relation reported by Rosenberg & Journeaux (1976), originally developed for weathered and soft rocks, appears to be more suitable for pile design in extremely weak to weak rocks of Qatar.

On the other hand, the skin friction has not fully mobilized within the moderately weak to medium strong rock (UCS > 12.5 MPa). By looking at the pile deformations against the applied loads on PTPs, it implies that the skin friction is yet to reach its ultimate value. The data shows a clear and consistent trend of mobilized skin frictions for the rocks below and higher than the weak category.

The author suggests that the test load should be increased/designed such that the pile shall be loaded until pile failure or until the jack capacity is reached to determine the ultimate skin friction and capacity of pile in Qatar. However, in projects where the unit skin friction was validated through performance of pile load tests and skin friction was found not fully mobilized, pile design was optimized and 11% to 50% savings in piling cost was achieved contributing to accomplishment of sustainable projects as reported by Cherian (2021) and for a project where the authors were involved.

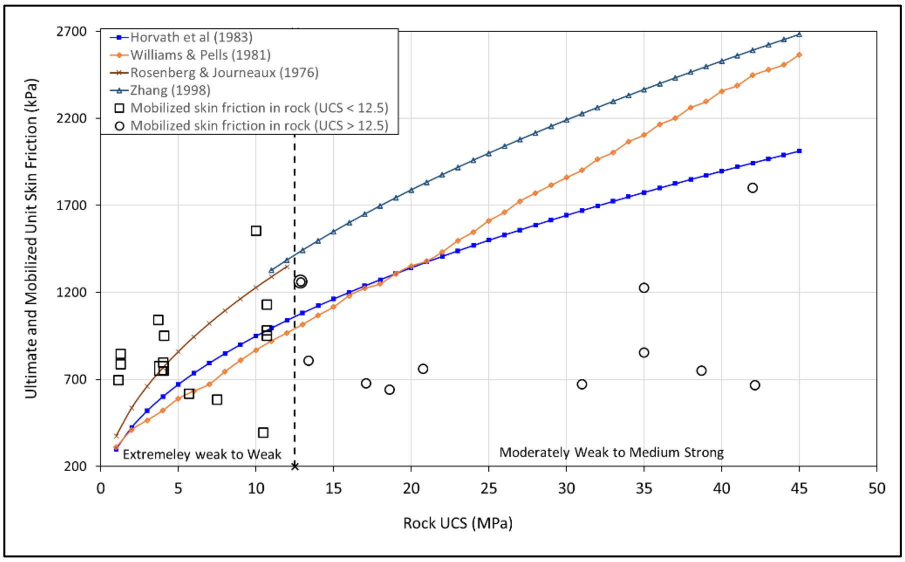

Fig. 3: Mobilized Skin Friction vs. Ultimate Skin Friction estimated by correlations.

[2] Preliminary Test Piles.
[3] Rock Quality Degeneration.
[4] MegaPascal.

6 Geotechnical Failure of Pile

In Qatar, almost all the pile load test reports conclude the results with a confirmation statement that "the test pile met the design requirements with regard to the design capacity, allowable settlement and hence concluded that the test result is acceptable", with no differentiation made for the failure criteria related to ultimate or working pile test whatsoever.

Geotechnical failure of a pile should be defined in terms of ultimate and serviceability limit states and these limit states should be considered when conducting load tests on trial piles and on working piles and failure criteria should be specified by the designer for each pile load test.

6.1 Ultimate Limit Failure Criteria for load tests

The target for the ultimate load testing on preliminary test pile (PTP) is to achieve the Ultimate Limit State (ULS) criterion. Table 1 provides the ULS criteria for the ultimate load test on trial piles (PTP) specified in the literature and codes.

Table 1: ULS failure criteria for bored piles in compression

Pile Type	Failure Criteria		
	Settlement	Residual settlement	Creep rate of head settlement
Preliminary Trial Pile	Settlement of the pile base diameter exceeds 10% of the pile diameter Or total axial movement exceeds 15 % of the pile diameter.	The residual settlement upon unloading from the ultimate load is 2.5% of the diameter or the load at which the pile head settlement = 0.15 + 0.1xDia (inches)	The load at which a sudden increase in the slope of the settlement-time curve occurs (plunging failure) – unlikely to occur in rock formations of Qatar
Reference	BS EN 1997-1, QCS 2014, ASTM D1143	DIN4026, Davisson (1972)	Bell and Robinson (2012)

Geotechnical failure for PTPs (using bidirectional load cells) may also be defined in terms of continued pile movement when the full elongation of cell stroke (typically around 200mm) occurs. The published guidelines and rule of thumbs for the pile failure criteria relate to the overall pile performance during a load test on preliminary pile (PTP) and working pile.

For this study, the authors have considered geotechnical failure as 10% of pile diameter. Fig. 4 shows measured settlement values (normalized by pile diameter) of 27 numbers of trial piles tested up to 300% of design verification load (DVL). It is evident from this figure that the majority of the piles have settled less than 2% of pile diameter with a couple of piles up to 4% mark except three tests (carried out for one particular project) where the maximum settlement of about 7% of diameter was achieved, indicating that none of the test piles reached the ultimate failure state.

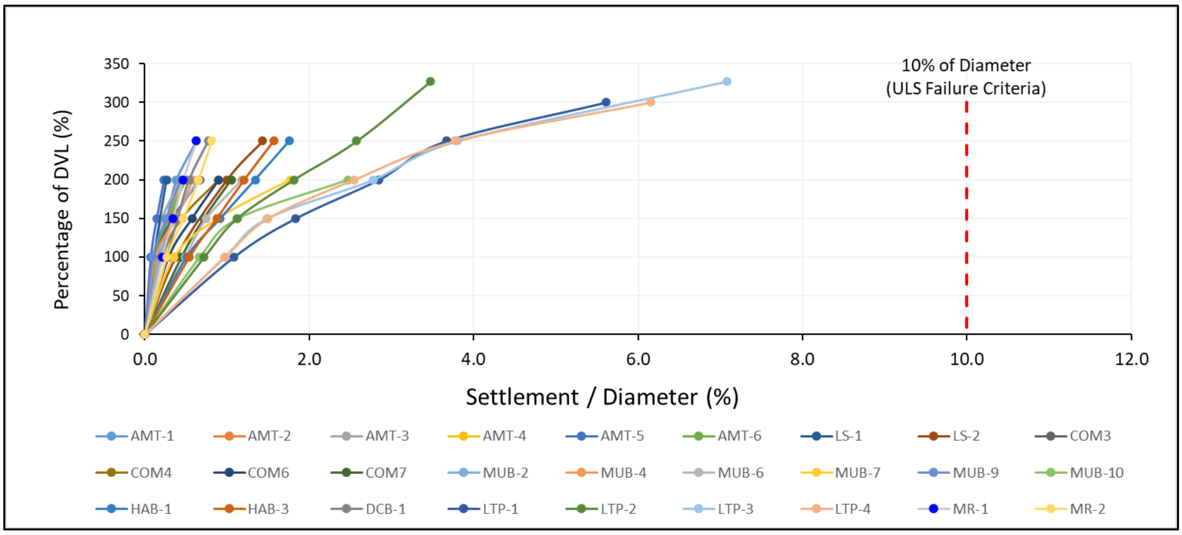

Fig. 4: Settlement of trial piles

Preliminary pile load tests are performed on trial piles, installed for test purposes only, before the design is finalised. As such, the pile design can only be optimized if skin friction has fully been mobilized. In case of a large diameter pile, it is often impracticable to perform a load test on a full-size trial pile such that the maximum skin friction could be mobilized, therefore, trial piles with smaller diameter may be considered for preliminary load tests in accordance with EN1997 and QCS 2014, provided that:

- the ratio of the trial pile diameter/working pile diameter is not less than 0.5
- the trial pile having smaller diameter is constructed using the same method as foundation working piles

6.2 Serviceability Limit Failure Criteria for load tests

Table 2 presents the suggested failure criteria to be considered for pile load tests on working piles. It is expected that the maximum accepted settlement suggested here does not exceed the tolerable movement limits of the structure. In any case, it is the responsibility of the engineer to state the performance specification in terms of maximum allowable pile settlement during load test at the design verification load (DVL).

Table 2: SLS failure criteria for bored piles in compression

Pile Type	Failure Criteria		
	Test Load	Max. Settlement	Residual Settlement
Working Pile	100% DVL	Exceeds 10 mm + elastic shortening; or 1% of Diameter; or Exceeds 7mm (second cycle)	The residual settlement upon the final release of the load exceeds 5mm
	150% DVL	14 mm + elastic shortening	
The maximum settlement values may be increased by 2.2% for every metre length of pile beyond 30 metres			
References: Federation of Piling Specialists (UK), Civil Design Criteria (Singapore)			

Bi-directional load tests on working piles are performed to measure the pile settlement against the design load. The settlements of a total of 44 numbers of preliminary and working piles at 100% and 150% of DVL are shown in **Fig. 5**. It is to be noted that the settlements presented in the figure are combinations of elastic shortening and pile vertical movements as given in the pile test reports. The details on adding elastic shortening to the pile vertical movements from the field tests are described by Loadtest (2002). It is clear that none of the piles has exceeded the serviceability limit state and even the combined settlement of pile at service load remains less then 10mm. The settlement criteria at 150% of DVL could not be verified due to unavailability of appropriate data to compute elastic shortening.

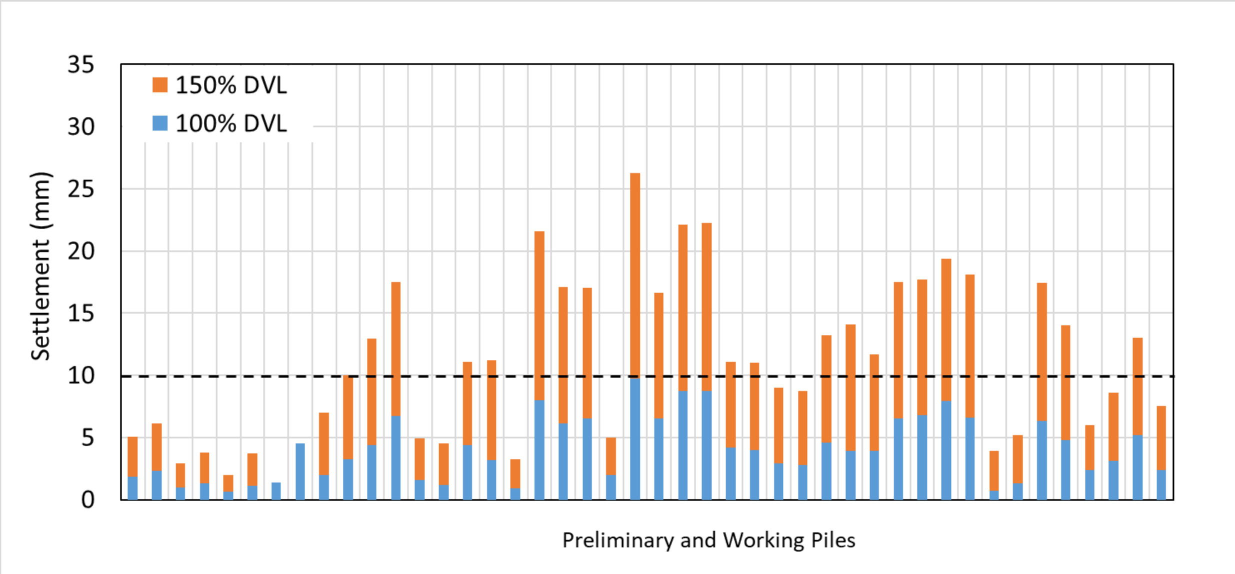

Fig. 5: Settlement of 44 test piles at 100% and 150% of DVL

7 Conclusions and Recommendations

This paper presents a review on the performance of pile foundations based on the typical geological conditions found in Qatar, aiming to provide an insight to engineers for optimizing the design of preliminary test piles. The following good practices are suggested for consideration by the authorities and consultants in specifying a preliminary pile load test in order to reap its full potential:

- The preliminary pile load test should be conducted on a trial pile with the aim of achieving ultimate failure criteria, and this shall not be considered as a means of verifying the proof or design load with some margin of safety. The ultimate bearing capacity of the piles may not be established if a pile load test ceases at a settlement less than 10% of the pile diameter. The results of trial pile test with ultimate failure will help in selecting more Optimized pile dimensions for the project, hence achieving savings in the overall project cost.
- If it is not practicable to perform preliminary pile load tests on a large-diameter pile with heavier loads, trial piles with smaller diameters may be constructed provided that the diameter of the trial pile is not less than 0.5 times the diameter of the working pile, and the trial pile with smaller diameter is constructed using the same method used for the foundation piles.
- For trial test pile designed as friction only pile, a "soft toe" or a compressible material of known thickness should be placed below the pile toe before construction and testing. This will ensure the loads are transferred to the shaft sides during the testing without any load being transferred

to the pile toe. The trial pile should be instrumented in such a way that the measurements could be used to derive the skin and end-bearing resistance separately.

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