Journal of Civil Engineering and Architecture 19 (2025) 594-606
doi: 10.17265/1934-7359/2025.12.004

An Approach for Optimization of Drilled Shaft Design in Dubai

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Abstract: Drilled shafts are widely used in Dubai as the primary foundation option for high-rise towers and bridges. Hundreds of meters of drilled shafts are installed in Dubai daily. The piling industry has grown significantly in Dubai over the past two decades. Piles with large diameters up to 2.5 m and lengths reaching 100 m are common. Current practice for geotechnical axial load capacity estimation is generally conservative, with actual pile axial load capacities demonstrated by static and dynamic loading tests often substantially higher than theoretical design capacities in most cases. The main objectives of this study are to review the shortcomings of commonly used theoretical axial load estimation procedures, examine the factors and parameters controlling estimated axial capacity (input parameters), and suggest optimization of conservative design procedures through careful selection of key input parameters and calibration to static loading tests.

Keywords: Drilled Shaft, Rock Mass Factor, Rock Compressive Strength, Load Transfer.

1. Brief Background on the Geology of Dubai

The rock formations in the Dubai area primarily consist of mid to upper Tertiary sediments of the Berzman Formation, comprising an interbedded sequence of carbonate-rich arenaceous, argillaceous, and rudite/conglomeratic rocks. Weathered calcareous sandstones/calcarenites with thin interbeds of siltstone are typically encountered to depths of 18-25 m, underlain by dominantly argillaceous/calcisiltites with interbeds of polymictic conglomerates — featuring pebble to gravel-sized clasts of gabbros, chert, sandstones, and carbonate rocks. Some deep-seated thick evaporate-gypsum beds associated with clays and mudstones are present beyond depths of 80-100 m. These represent continental shelf deposits with a depositional environment similar to the present-day environment, characterized by frequent eustatic changes (sea level fluctuations). These frequent sea level changes, combined with harsh climatic conditions, have given rise to surficial sabkha deposits along coastlines since the Quaternary period.

2. Advantages of Drilled Shafts

The local geology in Dubai is characterized by extremely weak to weak sedimentary rocks (as per rock strength classification in BS 5930 [1]), including sandstone/siltstone/conglomerates/mudstone with compressive strengths generally ranging from 0.5-5.0 MPa. These materials are classified as intermediate geo-materials (IGMs) [2] or "Soft Rocks".

The use of drilled shafts in Dubai is attractive due to relative ease of drilling and advancing stable boreholes through the "soft" sedimentary rocks (IGMs) with time and cost effective drilling methods using augers/buckets. Bentonite muds and chemical polymers are typically used to support the drilled holes for piles of different diameters (as large as > 2 m) and as deep as 100 m. Temporary steel casing is advanced through the upper un-consolidated sandy soil. In many projects, were drilled hole through IGM is stable, water only is used as drilling fluid, as adopted in many projects. Drilled shafts in IGM resist the applied loads by skin friction as opposed to end bearing. Design

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maximum side friction resistance typically ranges from 200 to 600 kPa, depending on the local rock strength, mass parameters, and pile installation details.

3. Overview of Drilled Shaft Behavior Under Compressive Loading

A comprehensive description of axial load resistance behavior of drilled shafts in IGMs is provided in FHWA-RD-95-172: Load Transfer for Drilled Shafts in Intermediate Geomaterials [3], which presents a parametric finite element study assessing factors affecting drilled shaft performance and load transfer mechanisms. This study was calibrated using results from actual in-situ loading tests on instrumented piles. Key findings include identification of factors controlling the magnitude and mobilization of side friction, with IGM-shaft interface conditions and IGM compressive strength found to be of primary significance. A rough interface can mobilize up to four times the maximum side friction of a smooth interface, as illustrated in Fig. 1.

Side friction is highly influenced by initial horizontal effective pressure at the shaft-IGM interface, which relates to pile concrete fluidity (slump). Therefore, using uniaxial compressive strength to represent IGM strength underestimates actual capacity; triaxial tests (CIU) at confining/consolidation pressures representing in-situ conditions are recommended for more accurate strength estimation.

FHWA-NHI-10-016: Drilled Shafts Construction Procedures and LRFD Design Methods [4] presents typical design procedures and performance predictions based on actual behavior observed in static loading tests. Fig. 2 illustrates the load transfer behavior of a drilled shaft of length L and diameter B subjected to an axial compression load QT applied to the shaft butt (top). Fig. 2(b) shows the general relationship between axial resistance and downward displacement, while Fig. 2(c) illustrates load transfer along the shaft and end resistance mobilization.

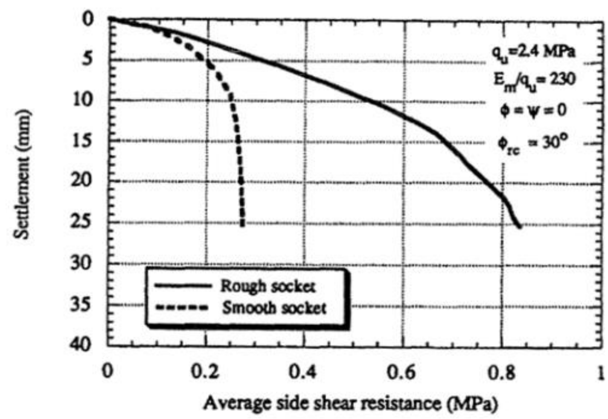
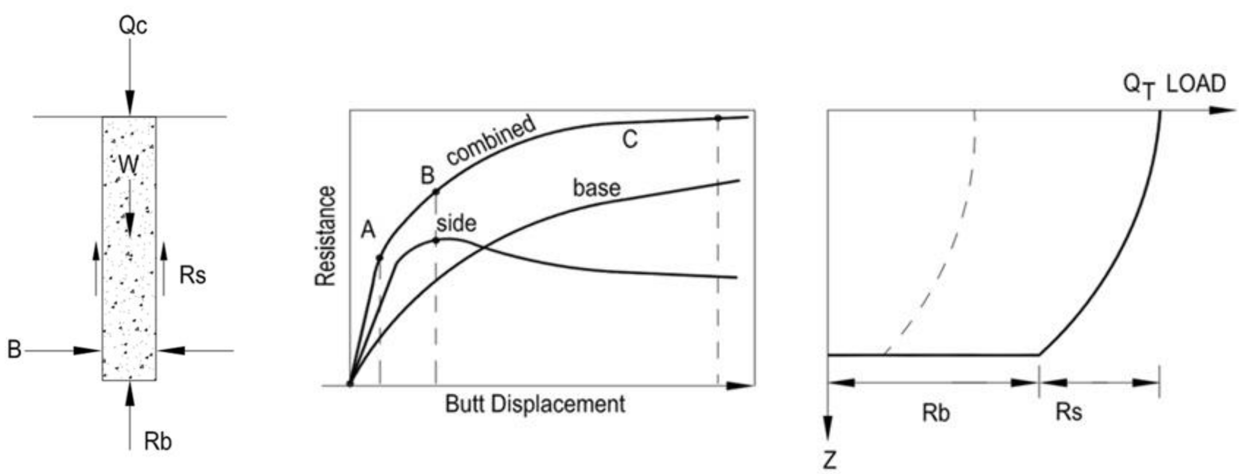


Fig. 1 Example of effect of interface roughness on load-settlement behavior infinite element analysis.



(a) (b) (c)

Fig. 2 Generalized load transfer behavior of drilled shaft in compression (FHWA-NHI-10-016).

Key behavioral aspects of drilled shafts include:

- Side and base resistances develop as functions of shaft displacement;
- Peak values for each resistance component occur at different displacements;
- Maximum side resistance occurs at relatively small displacements and is independent of shaft diameter;
- Maximum base resistance occurs at relatively large displacements and depends on shaft diameter and geomaterial type.

4. Additional Design Considerations for Rock Sockets

When designing rock sockets, a key decision is whether to neglect either side or base resistance for strength limit state evaluation. Regarding base resistance, AASHTO [5] Article C.10.8.3.5.4a states: "Design based on side-wall shear alone should be considered for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large movements of the shaft would be required to mobilize resistance in end bearing" [5]. This philosophy provides designers with the option to neglect base resistance. Under most conditions, quality control and assurance costs are offset by economies achieved in socket design by including base resistance.

Reasons for neglecting side resistance of rock sockets include:

1) Potential strain-softening behavior at the sidewall interface;
2) Possible degradation of material at the borehole wall in argillaceous rocks;
3) Uncertainty regarding sidewall roughness.

Design for service limit states must account for differences in side and base resistance mobilization as functions of axial displacement. In Dubai, side friction provides the primary contribution to axial load resistance, with base resistance contribution typically limited and often ignored by consultants. To enhance end bearing mobilization, techniques such as "base grouting" can be employed [6], though these techniques have not yet been widely implemented in Dubai projects. Therefore, this discussion focuses primarily on optimization of side friction resistance.

5. Settlement Behaviour and Load Transfer Prediction

Simple empirical correlations exist for drilled piles that provide accurate estimates of load-settlement behavior.

5.1 L1-L2 Rule: Drilled Shaft Load-Settlement Behavior (Chen and Kulhawy, 2002).

A database of high-quality static loading tests was used to establish the well-known L1-L2 rule [4], which describes typical load-settlement and load transfer behavior of drilled shafts, as shown in Fig. 3. The L1-L2 normalized curve presents typical behavior of drilled shafts under axial load, showing the transition from linear to non-linear response and defining failure load (at 3-5% D settlement for piles socketed in IGM). The L1-L2 criteria were found applicable to static loading test results in Dubai and can be used to evaluate test results and as a simple design tool.

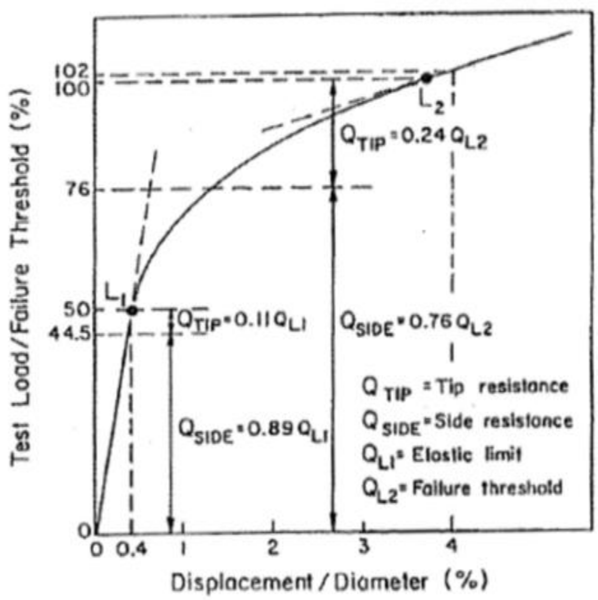


Fig. 3 Average normalized load-displacement curve that forms the basis of load test interpretation for compression (Chen and Kulhawy, 2002).

6. Local Drilled Pile Design Practice

Drilled shaft performance in Dubai has been validated through numerous static loading tests on instrumented piles. In most cases, test results showed actual capacity substantially exceeding theoretical “nominal” design axial pile capacity. A recent study by Prof. Rolf Katzenbach (May 16, 2017) presented in Dubai, “Identification of soil and rock parameters by scientific interpretation of field measurements,” demonstrated through back-calculation of static pile loading tests that only approximately 50% of pile length was actually necessary to resist design loads.

Fig. 4 shows results of multiple static pile loading tests conducted at a major site in Dubai, plotted on the normalized L1-L2 graph, indicating actual capacity significantly higher than nominal design theoretical capacity. Loading tests were conducted on working piles up to 150% of nominal working load Qw. Nominal design is based on a global safety factor of 2.5, so Qw = 0.4 Qf. By magnifying the L1-L2 simulation curve, data fit with a 1.25 times magnified curve, indicating actual pile design working load exceeds 1.25 times the adopted Qw. This reveals important opportunities for design refinement and potential cost savings.

However, static loading tests conducted during construction on working piles are difficult to use for design improvement. In some projects, preliminary test piles during design and pre-construction stages have effectively improved theoretical design, with side friction resistance increases exceeding 50% achieved in several projects based on O-Cell pile loading test results during design stage. Fig. 5 presents a typical example of 5 O-Cell tests conducted at a major project site, showing maximum unit skin friction (fmax) versus theoretically estimated fmax from site investigation results.

Results of O-Cell tests for intervals far from the O-Cell zone tend to underestimate friction resistance due to limited development. Detailed descriptions of O-Cell tests and utilization of t-z curves for load test simulation and pile performance analysis are provided in Fellenius (2014): “Basics of Foundation Design” [7] and Unipile software [8]. Fig. 6 presents typical t-z curves from one of these O-Cell tests, showing limited unit friction mobilization away from the cell zone.

Therefore, optimization of theoretical axial load estimation procedures is crucial to bridge the gap between conservative theoretical pile capacity estimates and actual pile behavior, achieving significant savings. The use of preliminary test piles is strongly encouraged as they provide reliable calibration of theoretically assessed side friction and pile behavior.

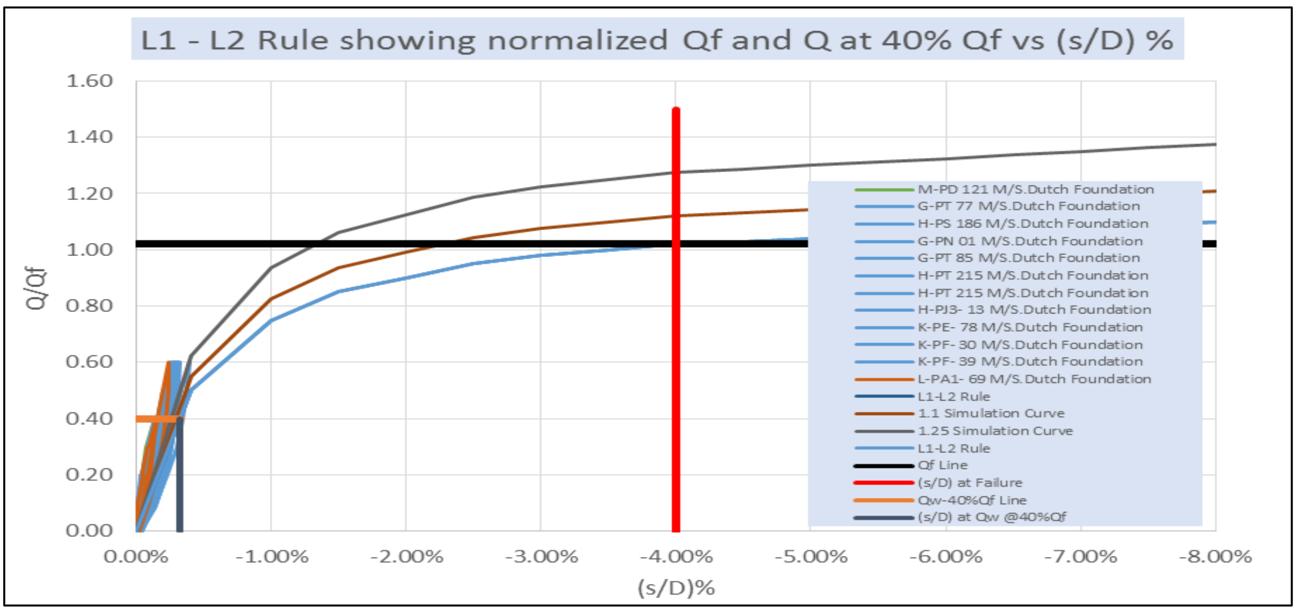


Fig. 4 Normalized pile load tests results plotted on L1-L2 graph.

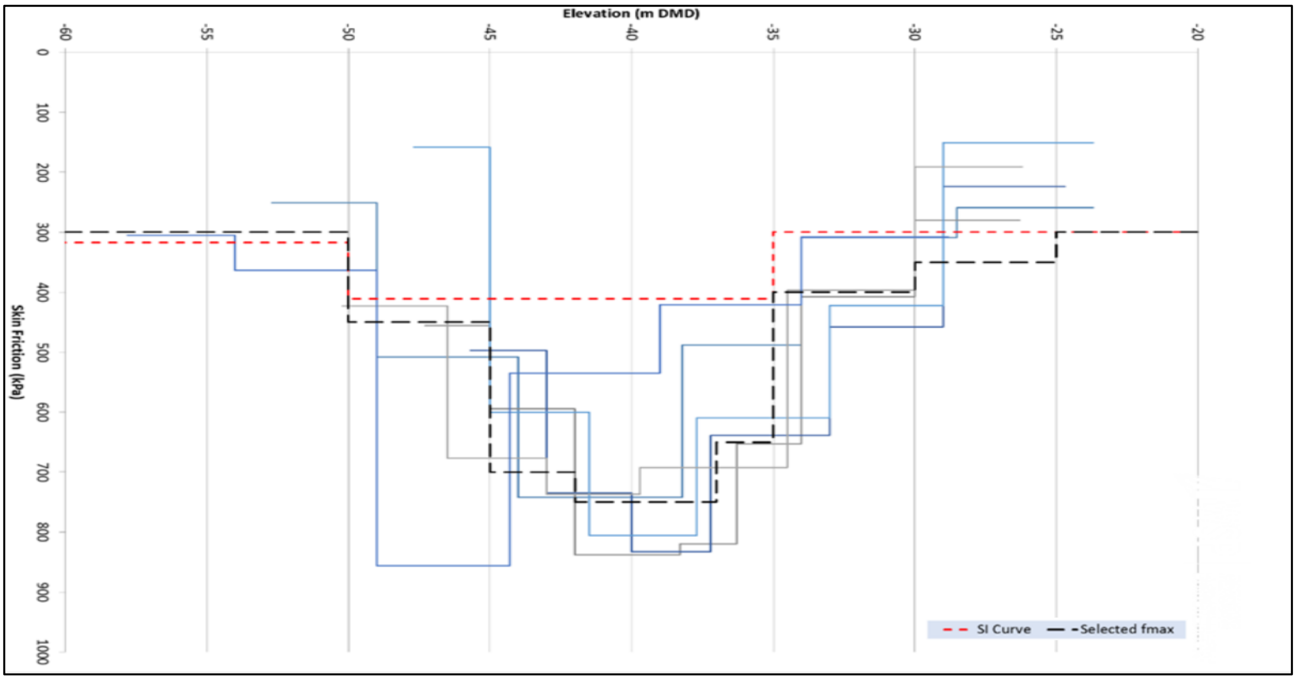


Fig. 5 O-Cell Tests on PTP piles showing theoretical line (Red) and actual measured average unit friction.

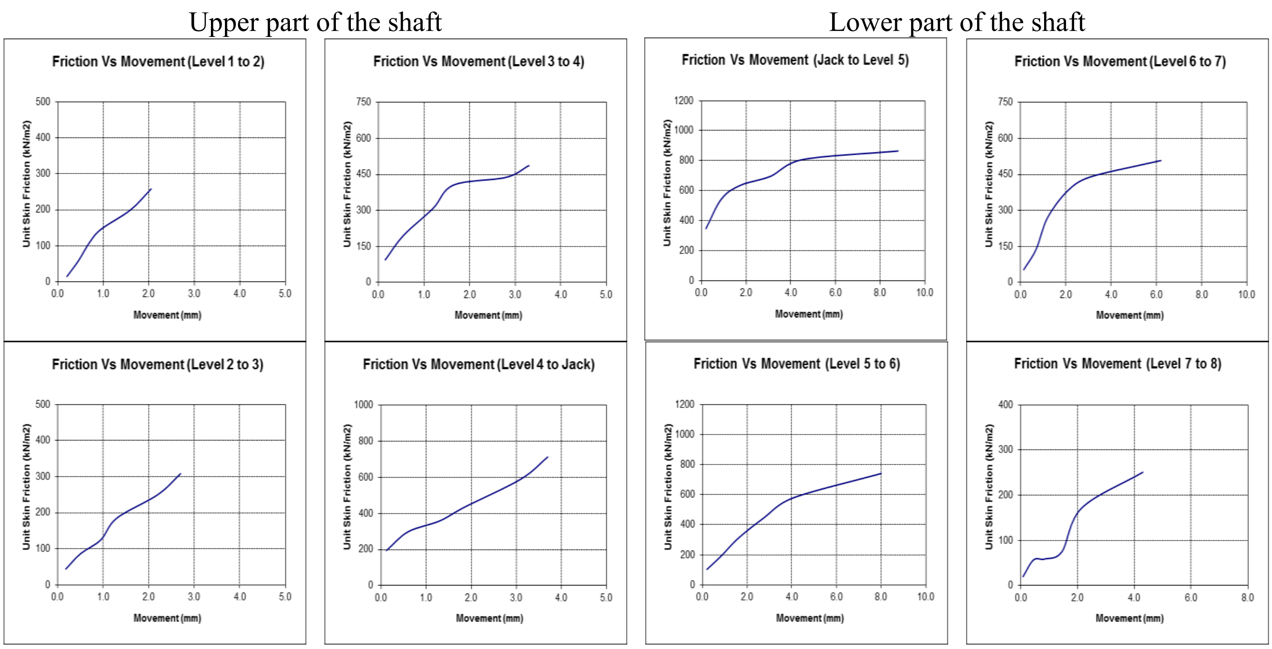
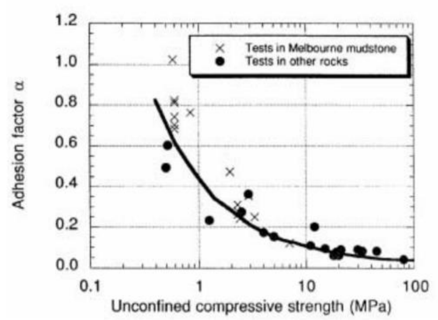


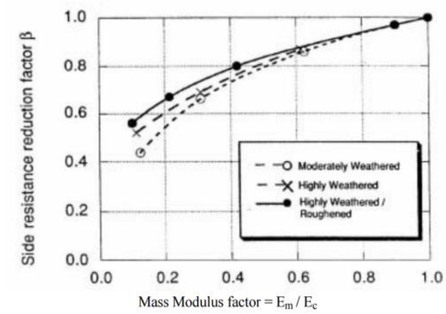
Fig. 6 Typical t-z curves for typical test pile (TP 8A – 1200 m diameter).

7. Theoretical Assessment of Maximum Side Friction in Local Practice

In local practice, maximum side friction (fmax) is typically estimated using empirical equations such as Horvath and Kenney [9], though the Williams and Pells procedure [10] is more commonly applied. The Williams and Pells method for skin friction evaluation is described in references [10, 11], with the relationship between rock compressive strength in sockets and side-wall shear resistance (adhesion factor) presented in Fig. 7.

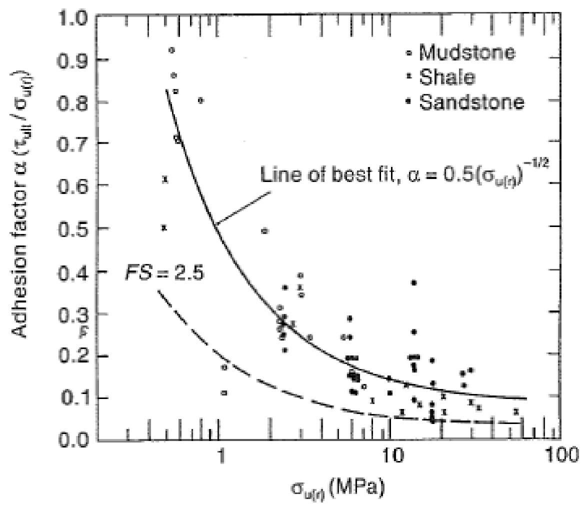


Adhesion factor α versus unconfined compressive strength (from Williams et al. 1980).



Side resistance reduction factor β versus mass modulus factor (from Williams et al. 1980).

A fitting curve is established as shown resulting in a simple equation as presented below:



Accordingly, fmax = α.β UCS, where α = 0.5 $(UCS)^{-0.5}$, therefore, fmax = 0.5 β $(UCS)^{0.5}$Mpa, in which UCS is the unconfined compression strength in MPa.

The reduction factor β represents rock mass discontinuities and Shaft-IGM interface roughness effects (e.g., use of bentonite vs chemical polymers, ..and other effects).

Fig. 7 Summary of Williams & Pells method for side friction of drilled shaft [10, 11].

Bentonite Effect: Using bentonite slurry during pile shaft advancement forms a bentonite cake, creating smooth interface conditions. Other drilling fluids like chemical polymers improve skin friction to some extent. While limited local data exists on friction degradation due to pile installation methods, FHWA-NHI-10-016 [4] presents study results (Brown 2002) comparing side friction of piles installed using different methods, including dry polymer, liquid polymer, CAD (cased ahead), CFA (continuous flight auger), and bentonite.

In local practice, fmax is typically estimated assuming clean sockets (using water or fluid polymer). For bentonite slurry, reductions of 20-30% are applied. The Williams and Pells procedure provides a non-linear (exponential) relationship between side friction adhesion factor and rock strength, which is more reliable than linear relationships as recommended in professional literature. According to Williams & Pells, two main parameters determine fmax range:

1. Selection of representative Compressive Strength for each depth interval, and

2. Selection of reduction factor β,

For clean socket, the reduction factor β depends on (J) rock mass factor or modular ratio, as shown J-β relation (Williams & Pells) presented above.

In local practice, J is estimated from RQD (Rock quality designation) using the relationship (Bieniawski, 1984).

For most of the sites, RQD is < 70% and hence a value of J < 0.2 is selected. Accordingly, the value of β < 0.65 is also selected.

Selection of design Compressive strength: The selection of compressive strength on the other hand is based on UCS (Uni-axial strength), which in the case of IGM provides a lower estimate of the rock strength. Further, large variations are typically obtained and therefore, the selection of design value for any depth interval becomes a difficult task. The graph presents typical UCS profile and selected design curve for a major project in Dubai.

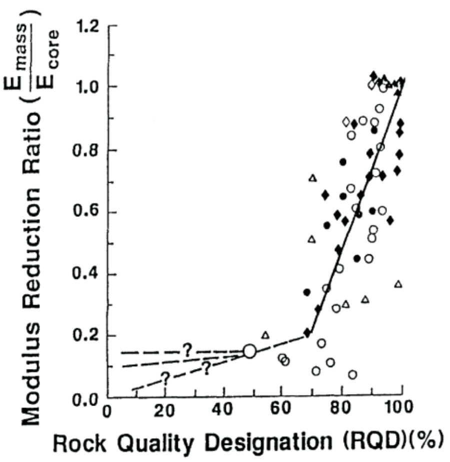


Fig. 8 Modulus Reduction Ratio as a Function of RQD (From Bieniawski, 1984).

The graph shows that the profile is divided into "homogeneous" intervals, the extremely high UCS results are excluded, and design value which is less than the mean or best fit line depending on data variation and number of test data ("BS EN 1997-2004, Geotechnical Design, Part 1 – General Rules" and "A procedure for determining the characteristic value of a geotechnical parameter by A. J. Bond, Geocentrix Ltd., Banstead Surrey, United Kingdom".

The procedures for selecting both β and design UCS look satisfactory, however, the obtained fmax estimates are much less than actual resistance illustrated by actual loading tests.

Where preliminary pile loading tests (PTP) are conducted, a load test correction factor can be introduced as follows: fmax – corrected = $C_{correction}$ x fmax (from theoretical estimate).

However, for the general case, where PTP tests are not planned or considered, then the following optimization design steps are suggested.

8. Optimization of Design Procedure

Selection of reduction factor β: The mass factor J is related to drilling parameters as fracture index or Fracture Frequency per meter in addition to RQD as shown in Table 1 (Tomlinson: Pile Design & Construction Practice [12]).

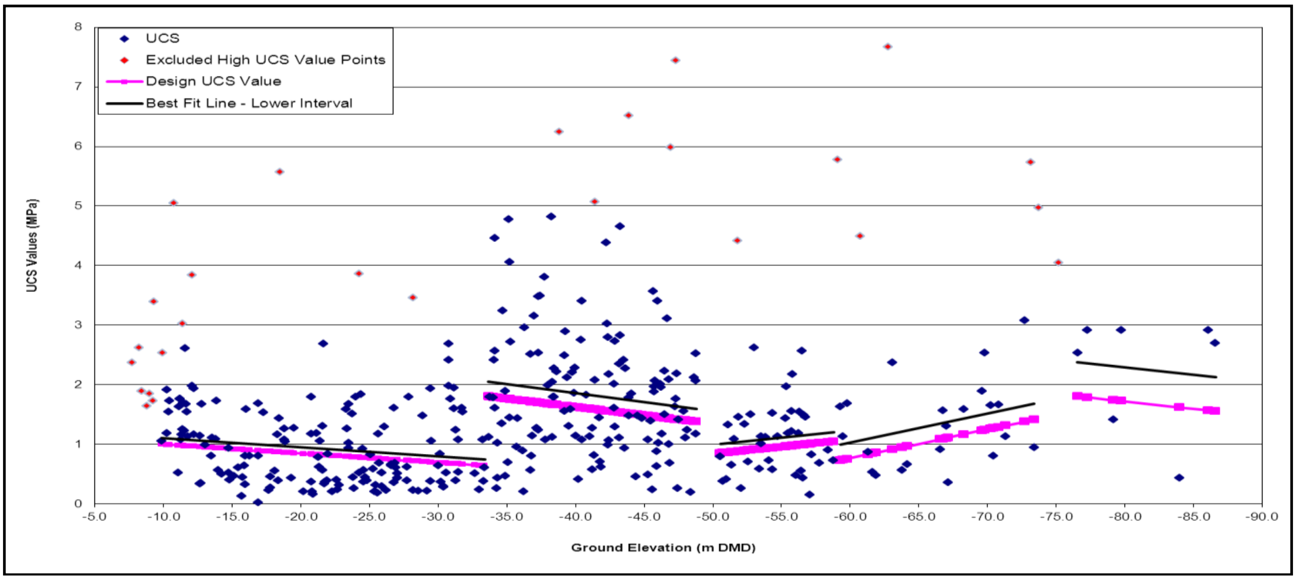


Fig. 9 Typical results of UCS tests profile showing large scatter of data, and showing selection of design UCS curve.

Table 1 Mass factor J as function of RQD and fracture frequency/m.

RQD (%)	Fracture frequency per meter	Mass factor j
0-25	15	0.2
25-50	15-8	0.2
50-75	8-5	0.2-0.5
75-90	5-1	0.5-0.8
90-100	1	0.8-1

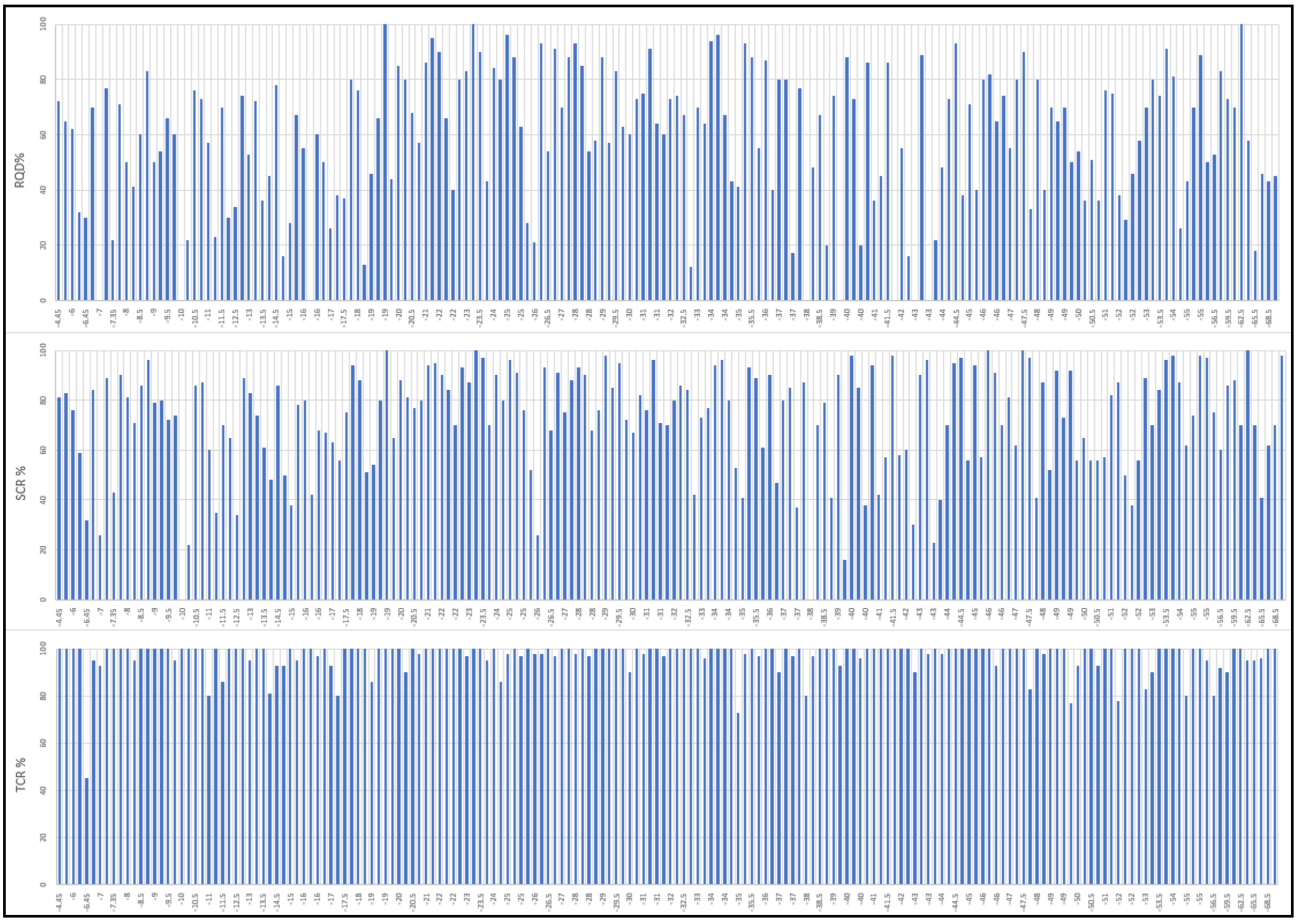


Fig. 10 Rock Core parameters — all BHs.

It is noticed that the above J – RQD relation is less conservative than the above Bieniawski, 1984 relationship. The following typical example shows results of high quality drilling results in a tower site in Dubai showing the different drilling parameters, and indicating RQD is <50-60%

Fracture Index vs Fracture Frequency: Fracture Index for all the cores (number of natural fractures/core run) — as measured for all BHs is shown in the Fig. 11.

Fig. 11 indicates a representative Fracture Index of <8, noting the core run is 1.5 m so the indicated results should be divided by 1.5 to five fracture frequency/m.

On the other hand J is defined as: E rock mass/E intact rock, and therefore it can be expressed as

J = {(Vs-site)/(Vs-Lab)}^2, as Vs-lab represents shear wave velocity of intact core, and Vs-site represents rock mass quality. The ratio of Emass/Ecore may also represented by Er (re-loading modulus) measured with in-situ pressure-meter tests (PMT) to CIU laboratory tests (E at 0.1% axial strain). PMTs and in-situ shear wave velocity measurements are routinely conducted in Dubai but are not explicitly used for pile capcity design. For these tests results to be used to estimate the mass factor (J), CIU laboratory tests (consolidated undrained) tri-axial tests on intact core specimens of IGM with local strain and bender element tests shall be conducted. A typical result of multi-stage TX (CIU) test on IGM material is shown below presenting the local strain modulus vs axial strain. The results are very useful to estimate the representative modulus of IGM at given strain level, as 0.1% typically used for geotechnical analysis at service load level.

Fig. 13 shows typical relationship obtained for the IGM modulus at 0.1 and 0.01% axial strain which represents small strain level.

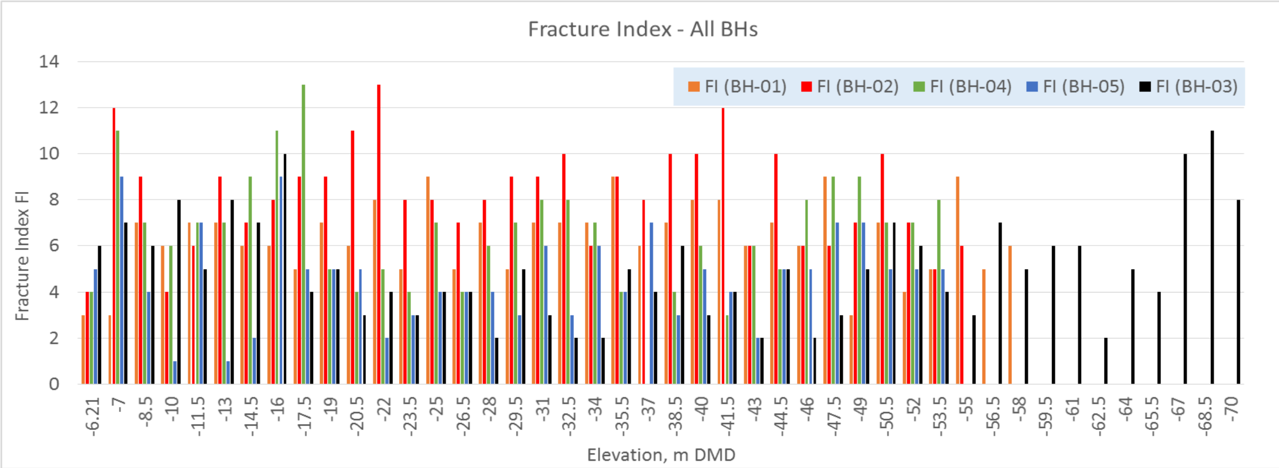
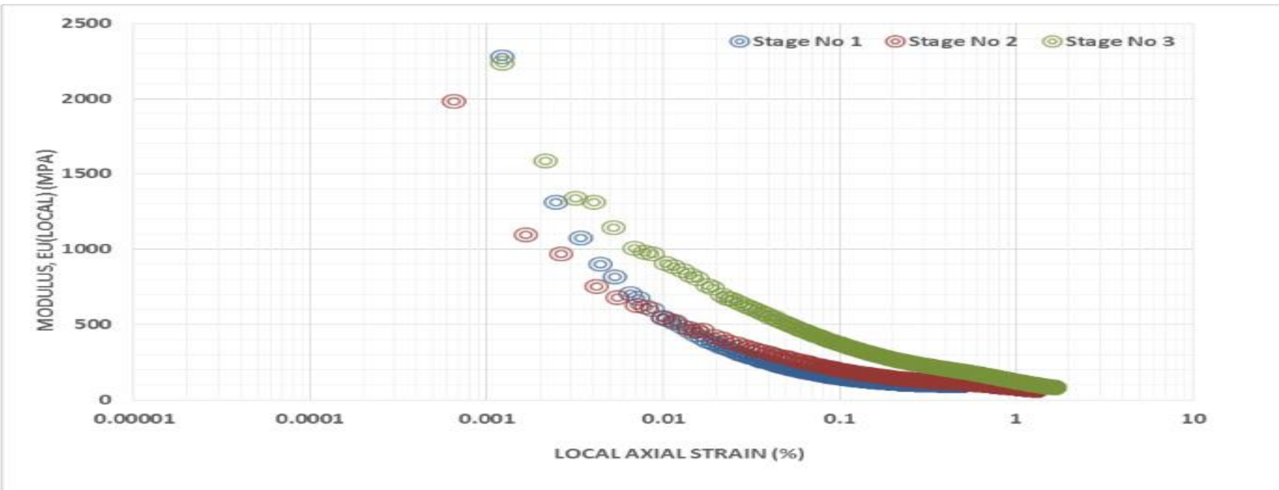
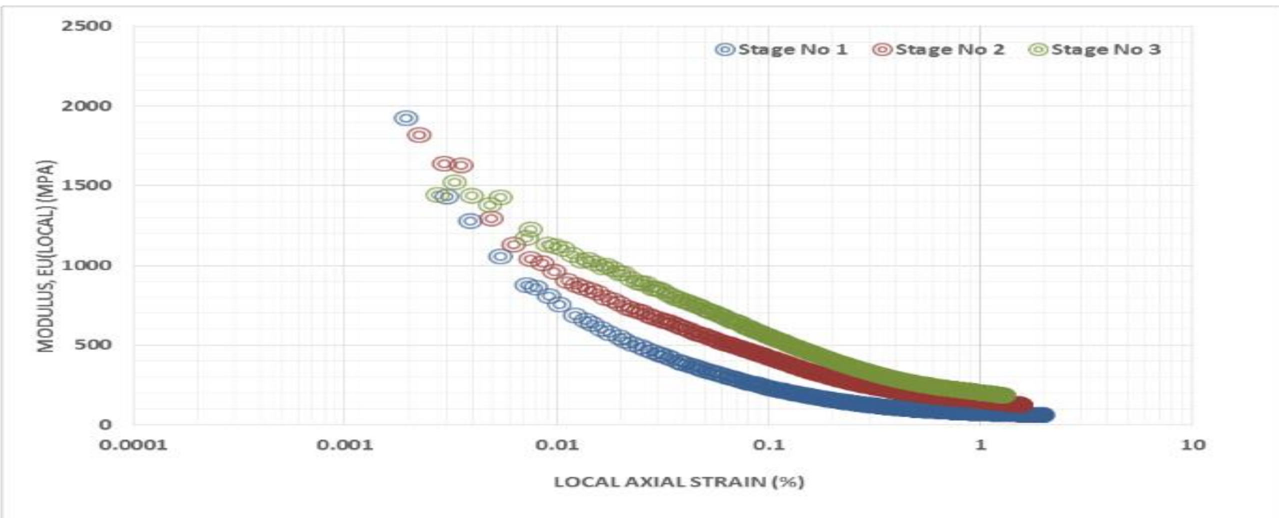


Fig. 11 Fracture index (fracture frequency) vs elevation – All Bhs.



Extremely Weak Sandstone (Consolidation pressure: 150, 300, 600 kpa)

Fig. 12.1 Modulus degradation obtained from instrumented CIU Tri-Axial Test (Local Strain).



Very Weak Calciciltite (Consolidation pressure: 400, 600, 800 kpa)

Fig. 12.2 Modulus degradation obtained from instrumented CIU Tri-Axial Test (Local Strain).

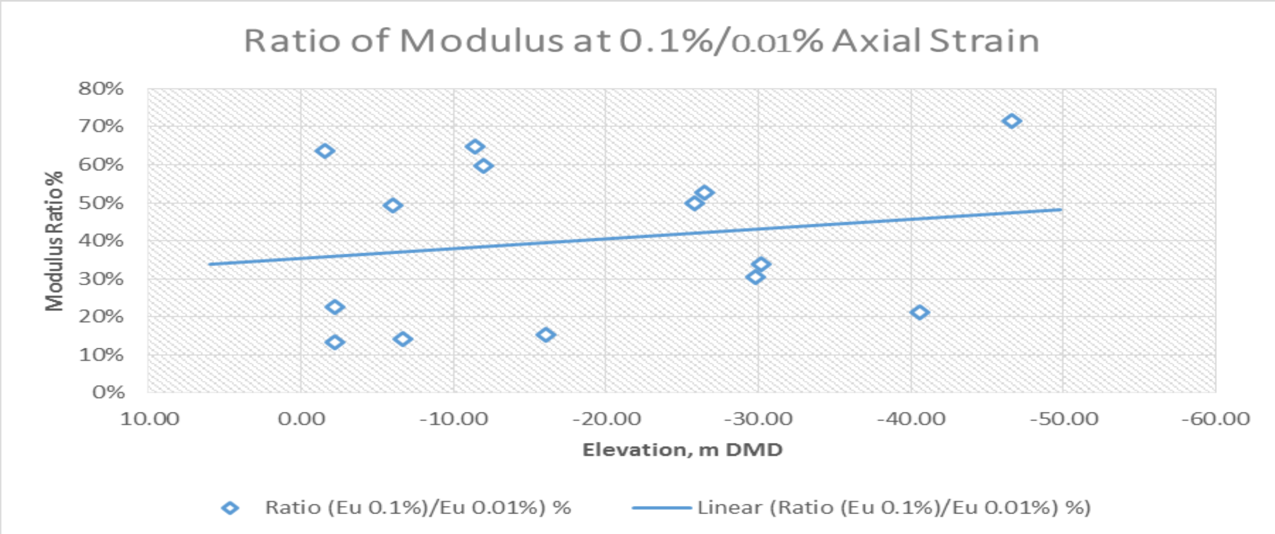


Fig. 13 Typical ratio of IGM modulus at 0.1to 0.01% Axial Strain obtained from Instrumented CIU Tri-axial tests.

Typical results from recently conducted investigation shows the correlation between PMT – Er and laboratory CIU modulus at 0.1% axial strain (local strain), which shows high ratio of about 1 between PMT – Er / Eu (local strain 0.1%), and indicating high mass factor J (Fig. 14).

The in-situ measured shear wave velocity is presented in the following typical profile recently measured in a prime site in Dubai (Fig. 15).

The laboratory measured Vs using bender element tests in CIU tests (after consolidation) ranged between 600 to 1000 m/s, giving J (Mass factor) = (Vs-site/Vs-lab)^2 of > 0.4 in general for the sites included in this study.

Note: it is obvious that the global or total modulus typically measured and reported in standard site investigations from UCS tests is misleading and represents a fraction of the actual IGM stiffness. Measurements of global or total strain are affected by local stress concentration conditions near the ends of the test sample, and therefore, local strain is generally advised to be measured.

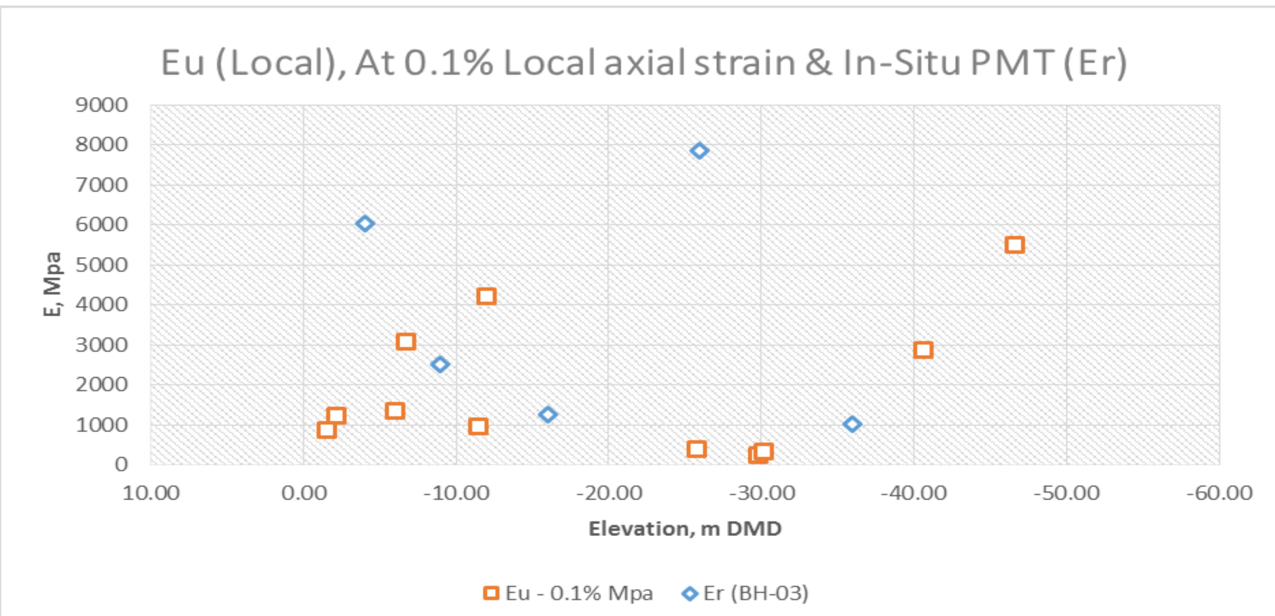


Fig. 14.1 Laboratory modulus at 0.1% axial strain versus in-situ pressure meter reloading modulus — site 1.

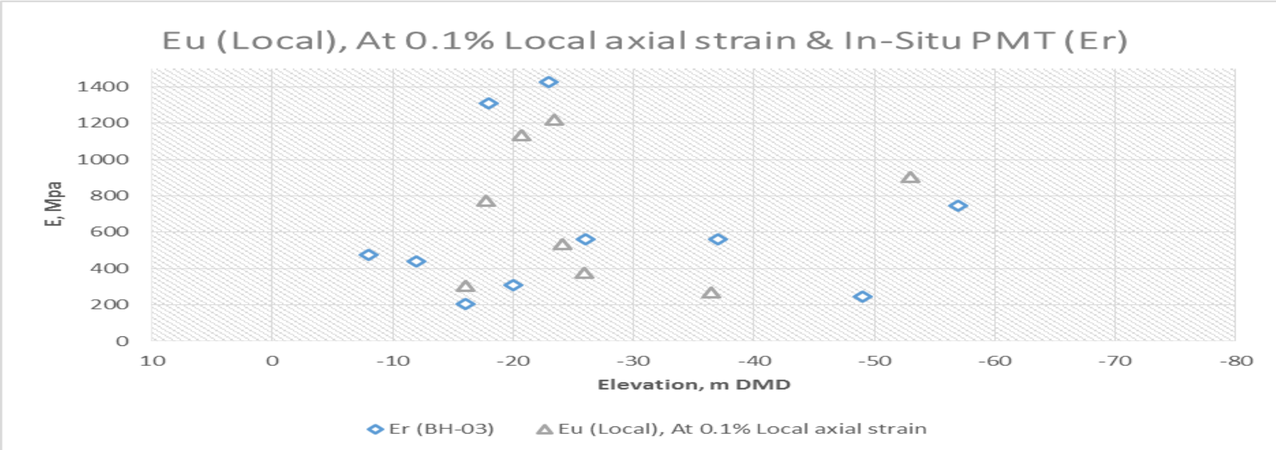


Fig. 14.2 Laboratory modulus at 0.1% axial strain versus in-situ pressure meter reloading modulus — site 2.

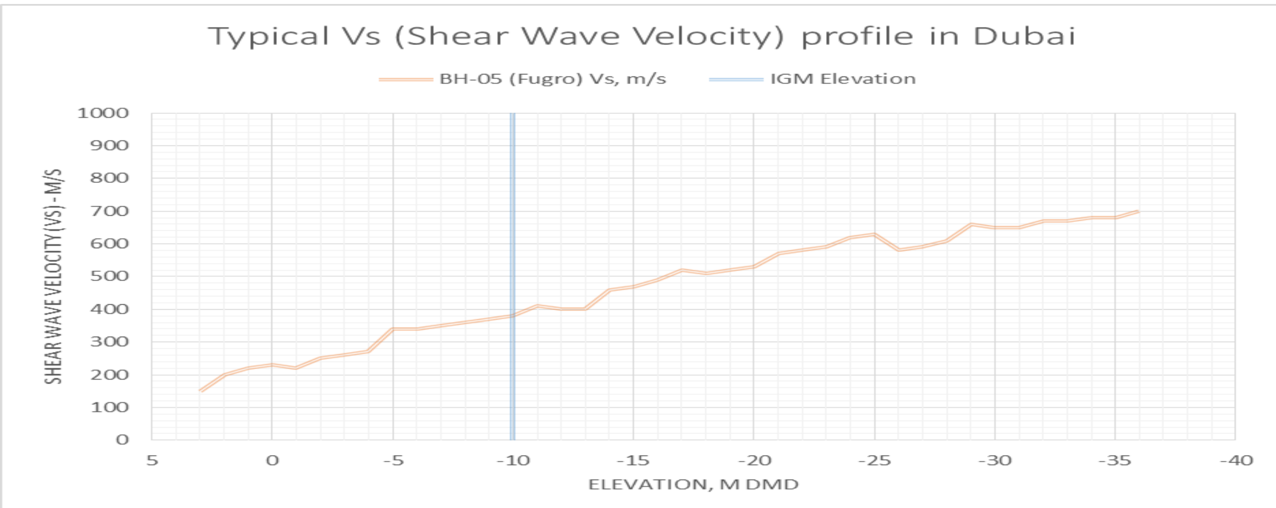


Fig. 15 Typical down hole seismic log of shear wave velocity profile in Dubai.

9. Compressive Strength of IGM

The uni-axial compressive strength is typically used to represent the strength of rocks. The compressive strength of Intermediate Geomaterials (IGMs), however, is highly affected by consolidation or confining pressure. High quality tests at GTC Lab in Dubai where recently conducted on many samples representing five main sites in Dubai showed a remarkable increase in compressive strength measured in CIU tests as consolidation pressure equal to in-situ overburden pressure (taking Ko = 1). The tests were conducted using computer controlled tri-axial testing equipment, on high quality samples with local strain and bender element tests. Photos of UCS and TX test machines are shown in Fig. 16. All the tests were directly conducted by senior laboratory testing specialist (Hardev Sidhu).

Typical results are shown as follows: the CIU compressive strength is the deviatoric stress at failure (Fig. 17). Linear trend line is added for each set of data for illustration only.

Fig. 16 Photos of UCS and CIU tri-axial computer controlled testing machines (GTC-LAB, 2017).

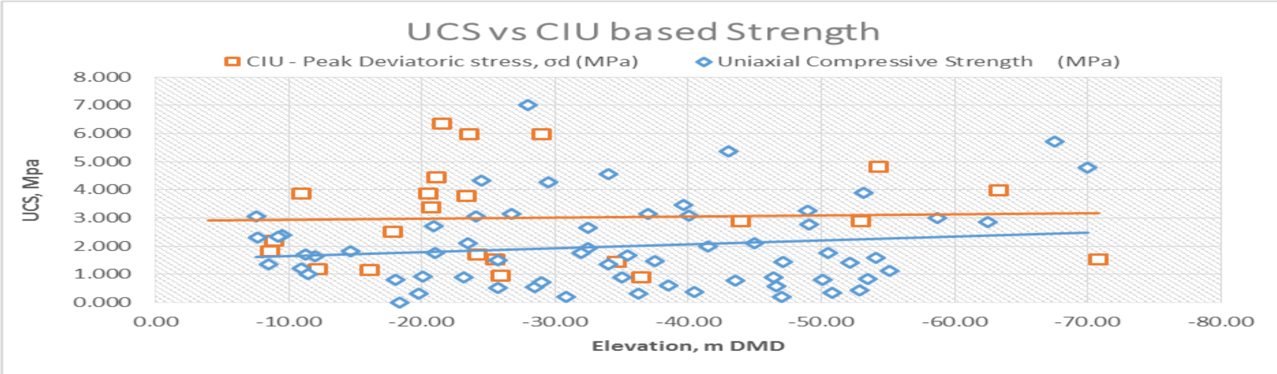


Fig. 17.1 **UCS vs CS (compressive strength) profile from CIU Triaxial Tests – Site 1.**

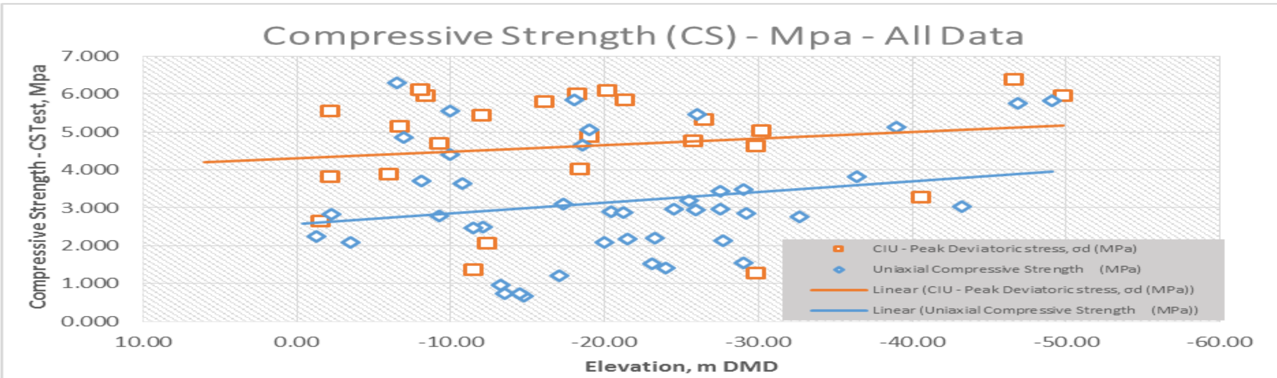


Fig. 17.2 **UCS vs CS (compressive strength) profile from CIU triaxial tests – Site 2.**

The compressive strength (CS) obtained from CIU tests is at least 50% greater than UCS tests for different rock types (Calcarenite, Sandstone, Conglomerates, Siltstone and Calcisitite).

The above strength profile represents the following typical site lithology (Fig. 18).

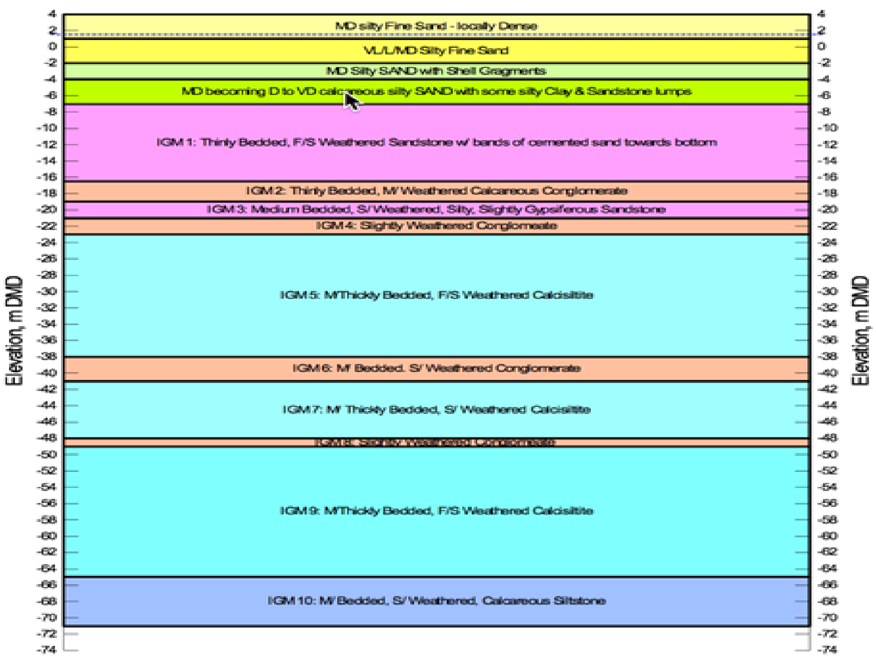


Fig. 18 **Lithology of typical site in Dubai.**

10. Conclusion

Williams &Pells equation provides non-linear relationship with fmax, and includes parameters related to IGM (Intermediate Geo-Materials) -Shaft Interface (β), and rock strength. By combining the effects of proper selection of the mass factor J, and the use of Tri Axial based compressive strength, the following improvement is obtained:

Nominal maximum unit friction (fmax) = 0.5 β √UCS

For typical β = 0.65 normally selected, then fmax = 0.5×0.65 √UCS = 0.325 √UCS

By proper selection of mass factor J, then β would range between 0.7 to 0.75, and by using triaxial compressive strength (CIU) test values rather than uniaxial/unconfined compressive strength (UCS) test values, with CIU shown to be 1.5 UCS in Dubai, then

fmax = 0.5×(0.7-0.75)×√(1.5×UCS), or fmax = (0.42-0.46)×√UCS

An improvement factor of 1.3 to 1.4 over nominal estimates of fmax. This level of improvement is substantial and is generally matching with results of static loading tests.

It must be however, pointed out that the use of this range of parameters requires that:

High quality drilling to be conducted such that realistic rock mass parameters are obtained.

In-situ PMTs and down-hole seismic tests to establish shear wave velocity and stiffness profiles representing the rock (IGM) mass.

Laboratory tri-axial (CIU) tests with consolidation pressure representing the overburden pressure, with local strain and few bender element tests.

Most of the above tests are typically conducted in site investigations for main projects in Dubai, however, the use of TX-CIU testing was used only in few projects.

The number of tests and locations, and estimate of representative strength and stiffness parameters for each homogeneous depth interval shall follow standard geotechnical evaluation methods and must be conducted by experienced geotechnical engineer, to be able to establish appropriate estimate of mass factor J and hence friction reduction factor β, in addition to selecting representative design compressive strength (CS) values for different layers.

The above procedure makes the application of Williams &Pells formula much more reliable and less conservative and approach results of actual static loading tests.

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