Friction to unconfined compression strength correlation for evaluating capacity of weak rock foundation sockets

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Abstract

Historically, deep foundations in weak rock have been designed as friction elements using frictional resistance (f_s) calculated from the unconfined compressive strength (UCS) of rock. Most of the published correlations of f_s to UCS were developed based on load tests on low-capacity piles in specific geological conditions, using UCS values not necessarily representative over the test depth. There is a large variation in foundation design depths calculated using these correlations. This paper presents a correlation between f_s and UCS of weak rock, developed using data from 44 bidirectional load tests from high-capacity deep foundations in weak rocks. The dataset used in this study, is one of the largest used for weak rocks, with high test loads in the range of 100 to 320MN and the depth of foundations mostly in the range of 20 to 87m below ground level. Bi-directional load test data from La Maison tower site is then simulated in Plaxis, and ultimate skin friction developed is compared against the skin friction calculated using the new corelation. The actual ground profile and foundation layout of La Maison tower is then modelled in Plaxis with the required foundation depth derived using the recommended corelation, to check serviceability limits. The resulting maximum settlements are found to be well within the acceptable limits. The correlation factor of 0.5 between f_s and UCS is thus recommended for estimating rock socket friction for design of deep foundations in weak carbonate rock formations.

1 Introduction

Deep foundations in weak rock formations are typically designed as rock sockets and the design is a function of the loading magnitude, geometry, elastic properties, and the side resistance of the socket. Until the early 1960's there was little research and few acceptable design methods for rock socketed piles (Tomlinson and Woodward, 2007). However, in the next decade several investigations of the design of rock socketed bored piles resulted in various improved design methods which considered the principles of applied mechanics and the rock socket properties, for example, Pells et al. (1980), Kulhawy and Phoon (1993). Many of these design methods predicted the bearing capacity of piles socketed in rock from unconfined compression strength (UCS) of intact rock.

Kulhawy et al (2005) reviewed these methods to predict rock socket friction (f_s) and highlighted a relative lack of sophistication in earlier empirical methods and also commented that in the load test-based methods, the UCS of rock used in developing these correlations was not the average UCS over test depth and might not represent the values at the test locations. Furthermore, many of these methods were developed mainly for small diameter shallow piles with low capacities in specific geological conditions (Williams and Pells, 1999, Alrifai, 2007, Ibrahim et al., 2009, Latapie et al., 2018, Manoj et al., 2020). There is also a variation in the correlation coefficient α of 0.15 to 0.80 between the lower and upper bound relationships.

Based on a detailed literature survey and comparison of published case studies, mostly focused on the tall towers in the United Arab Emirates (UAE) where more than 20% of world's tall towers are being built, the methods used to estimate ultimate friction (f_s) for high-capacity deep foundations in the weak carbonate rocks, are assessed to be conservative, probably due to a lack of geology - specific methods for the region. (Manoj et al. 2020, Latapie et al. 2018, Alrifai 2007, Ibrahim et al. 2009).

This paper presents the results of a study based on 44 high- capacity load cell tests on barrettes and piles supporting tall towers in weak rock, along with UCS within the test depth at these locations, which are back-analysed to develop a relationship between f_s and UCS of rock. The actual friction measured closest to the load cell locations are considered conservatively as ultimate values, while developing these correlations. The rock sockets are actually supported by the rock mass and not the intact rock and therefore using the frictional resistances measured from Osterberg cell test is considered to take in to account the effect of discontinuities in rock. Based on the study of high-capacity load tests on large diameter piles and barrettes, a new correlation to estimate ultimate skin friction from UCS in weak rock sockets is recommended in this paper. The correlation was then tested via a load test simulation by finite element modelling of the load test done at the La Maison tower location. Resulting ultimate friction from the simulation, is then compared, and confirmed with the calculated skin friction from the new correlation.

The foundation design of the supertall La Maison tower (Manoj et al. 2020) is then repeated with barrette and pile lengths calculated using rock socket friction calculated using the new correlation. The small-strain stiffness modulus E_d obtained from down hole seismic tests is used to model ground stiffness behaviour in the model, as recommended by Poulos (2017). Model results are presented which show that when pile and barrette lengths are revised using the new correlation, group settlements are found to be within acceptable limits. The new correlation is recommended for use to estimate rock socket friction in weak carbonate rock formations and a design chart is proposed on this basis.

2 Design Of Friction Piles And Barrette Foundations In Weak Rock

2.1 Basic Design Approach

Rock socketed piles typically transfer applied load to the supporting ground in side shear, end bearing or by a combination of both. The initial transfer of shaft load through shear stresses on the interface is largely an elastic process and the socket roughness will also play a role in the shear load transfer of rock socketed piles (Poulos and Davids, 2005, Poulos, 2010, Katzenbach and Choudhury, 2013). The typical load settlement behavior from a pile load test is shown in Fig. 1.

The ultimate capacity, Q, of axially loaded pile can thus be expressed as the sum of base capacity Q_b, and the shaft capacity Q_s. Thus:

$Q = Q_b + Q_s$ {1}

where $Q_b = A_b f_b$ and $Q_s = A_s f_s$

and where A_b is area of the pile base, f_b is unit ultimate end-bearing pressure, A_s is area of the pile shaft in rock socket and f_s is average unit ultimate frictional resistance of the rock socket.

As the pile is loaded in compression, the movement required to mobilize maximum shaft friction is typically only about 0.3–1% of the diameter of pile, whereas the base resistance of the pile needs a downward movement typically in the range of 10 to 20% of base diameter, for its full mobilization at point D in Fig. 1, when the pile will plunge downwards (Tomlinson and Woodward, 2007). Moreover, when the depth to diameter (D/B) ratio increases to the range of 8 or higher the mobilised toe resistance will be negligible and most of the load will be carried in shaft resistance (Rezazadeh and Eslami, 2017). The high-capacity piles and barrettes socketed in weak rocks, carry the applied load mainly by socket friction and only limited load will be transferred to the base, as evidenced by many reported load tests (Emrem et.al, 2008) including the ones considered in this study. Due to this fact, and also due to bottom cleaning issues as well as the risk of cavities, the piles, and barrettes in weak carbonate rocks for tall tower structures in the UAE are mostly designed as friction elements, ignoring any toe resistance.
From a practical design viewpoint, the ultimate capacity, Q, of an axially loaded pile is thus approximated as equal to the shaft capacity Q_s, i.e.

$Q = Q_s = A_s f_s$ {2}
The design approaches currently adopted for most of the tall tower designs in UAE are reviewed below.

2.2 Design correlations and current design practice in UAE

Piles and barrettes in the weak IGM (Intermediate Geo Material) and carbonate rocks in UAE are designed typically as friction piles and the end bearing strength is generally ignored in the design due to pile bottom cleaning issues and due to the risk of cavities. Some of the recently published and other commonly adopted correlations for the pile and barrette designs in the UAE are provided in Table 1.

Table 1
UCS to skin friction correlations based on available literature

Author	Year	Correlation based on UCS	Condition for which the correlation is developed
Manoj et al	2020	0.52 to 0.81$(UCS)^{0.5}$	Weak Carbonate rocks of UCS 0.6 to 3MPa based on load test data. Higher factor can be used if proved by load test
Latapie et al	2018	0.42 to 0.54$(UCS)^{0.5}$	Carbonate rocks in the middle east region
Charif, Najjar & Sadek	2010	$0.18(UCS)^{0.5}$	Carbonate rocks in the middle east region
O'Neill and Reese	1999	αφ(UCS)	Intermediate Geo Material
Zhang and Einstein	1998	$0.40(UCS)^{0.5}$	Smooth interface. Based on load test results from various locations take from literature
Zhang and Einstein	1998	$0.80(UCS)^{0.5}$	Rough interface. Based on test results - various locations - from literature
Reese and O'Neill	1988	0.15(UCS) when UCS < 1.9MPa $0.2(UCS)^{0.5}$ for UCS > 1.9MPa	-
Rowe and Armitage	1987	$0.45(UCS)^{0.5}$	Regular surface Interpreted from load tests
Carter and Kulhawy	1987	0.15(UCS)	-
Rowe and Armitage	1987	$0.60(UCS)^{0.5}$	Rough Surface Interpreted from load tests
Abbs and Needham	1985	0.375(UCS) UCS < 1MPa 0.375 + 0.1875(UCS-1) UCS = 1-3MPa 0.75 for UCS > 3MPa	Interpreted from load tests for Weak Carbonate Rocks
Horvath *et al.*	1983	(0.20 to 0.30) $(UCS)^{0.5}$	Interpreted from load tests for Shale and Mudstone. Factor of 0.2 to 0.3 for smooth to rough socket
Williams and	1981	αβ(UCS)	Sandstone, Shale and

Pells			Mudstone
Williams *et al.*	1980	αβ(UCS)	Melbourne Mudstone
Meigh and Wolski	1979	$0.22(UCS)^{0.6}$	
Horvath and Kenney	1979	$(0.20 \text{ to } 0.25)(UCS)^{0.5}$	Shale and Mudstone Factor of 0.2 to 0.25 for smooth to rough socket
Horvath	1978	$0.33(UCS)^{0.5}$	Based on load test data
Rosenberg and Journeaux	1976	$0.375(UCS)^{0.515}$	Interpreted from load tests for weathered and soft rocks

A review of design of tall towers in UAE reveals that large diameter piles or barrettes of most of these towers are designed using the following equation for ultimate skin friction by Horvath et al. (1983), which has actually been developed using load test data from short piles in shale and mudstone:

$f_s = (0.20 \text{ to } 0.30)\ (UCS)^{0.5}$ {3}

In order to demonstrate the variability in these existing design methods and the existing data gap, a design has been performed using the commonly used equations selected from those presented in Table 1, using the subsurface profile from La Maison tower in Business Bay in Dubai (Manoj et al. 2020). The results, presented in Fig. 2 show that for barrette of 1.2 x 2.8m size, the required design depth to generate 40 MN design capacity will vary between 10m to 62m below cut off level following these various design methods, demonstrating a remarkable variation in the estimated design requirements.

Figure 2 justifies the need to develop a more efficient and optimized design method for high-capacity piles and barrettes, especially for the geological conditions of weathered weak rocks and IGM materials such as the carbonate rocks in the UAE.

3 Load Cell Tests

The bi-directional load cell test, such as the Osterberg cell (O-Cell) test (Osterberg, 1989) is now the method often adopted for static load testing for high-capacity piles and barrettes, which overcomes the limitation of the conventional top-loading test where the load capacity is generally limited to about 10 to 40MN. The load cell derives reaction acting in two directions, upward against side-shear and downward against both side shear and end-bearing resistance depending on where the load cell is placed. The test separates the resistance and displacement data for each component of the pile (England, 2003, England, 2010, Tan and Fellenius, 2012).

The uni-directional load for a given internal pressure is determined using the load cell's calibration coefficient. Typically, the load cell location is close to the base of the pile as shown in Fig. 3 and determination of side shear and end bearing resistance is straightforward. The loading is continued until either ultimate upward or downward capacity is reached, or until the maximum load cell stroke or load capacity is reached. Distribution of load throughout the foundation length is obtained by use of strain gages within the foundation. Analysis of the test results enables the design engineer to conveniently interpret the friction and end bearing strengths developed.

O-cell load test is usually performed in general compliance with ASTM D1143-07, standard test method for deep foundations under static axial compressive load using procedure A, quick test loading schedule. The load-movement curves for a pile or barrette section below and above the O cell assembly are plotted in real-time basis to facilitate interpretation of the pile or barrette performance during loading on site. The data collection can be tabulated live at every 30 or 60 seconds in a spread sheet format including pressures, loads, displacements, and strains in a transparent data collection process.

With design validation and value engineering from load cell test data, it is possible to achieve a significant improvement in the outcomes of the design methods, thereby achieving more efficiency and cost savings. There are several case studies where results of static load test on piles and barrettes in the UAE are presented (Ibrahim et al, 2009, Alrifai, 2007, Poulos and Davids, 2005). Value engineering and back analysis to revise the design lengths have rarely been attempted, other than in few cases for high-capacity load cell tests on barrettes and large diameter piles (Haberfield, 2013, Pereira et al, 2017).

The results from 44 O-cell tests of large diameter piles and barrettes, are used in this paper along with UCS results from the same depth zones, to back analyse and calibrate the data and arrive at an appropriate correlation between rock socket friction and UCS.

4 The Study Region And Data Set

4.1 The study region

The data set collected for this study are all from tall and supertall tower foundation locations. A large number of tall buildings constructed and proposed in recent decades have been in the Middle East region, which justifies the study region selection. The tall and super tall towers in Dubai are mostly clustered along Shaikh Zayed Road and within the Dubai Marina and Business Bay areas, from where the dataset shown in Fig. 4 has been collected.

Tables 2 and 3 provide details of the 44 load tests collected from 18 locations, of which 10 are on barrettes and the remaining tests are on large diameter piles.

Table 2
Details of study area and the test barrette details

Tower code	Site Location	Test Barrette Number	Barrette L X B (m)	Depth of Barrette m	Depth of O cell RL m (DMD)
HDS Tower-01	Business Bay, Dubai	B1	2.8x1.2	45.25	-48.3
		B2	2.8x1.2	45.25	-46.9
		B3	2.8x1.2	47	-45.4
OT-02	Ras Al Khor, Dubai	B4	2.8x1.2	50.2	-30.2, -43.2
		B5	2.8x1.2	80.5	-56.2, -71.2
		B6	2.8x1.2	87.4	-63.92, -83.4
ET-03	SZ Road, Dubai	B7	2.8x2	69.5	-56.2, -68.2
		B8	2.8x2	48	-35.2, -47.2
MO-04	SZ Road, Dubai	B9	2.8x0.8	8.15	-8.6
		B10	2.8x0.8	10	-8.6

Table 3
Details of study area and the test pile details

Tower code	**Site Location**	**Test Pile Number**	**Pile Diameter (m)**	**Depth of Pile (m)**	**Depth of O cell RL m (DMD)**
MD-05	Marsa, Dubai	1.	1.5	44.2	-37.02
SB-06	Jumeirah, Dubai	2.	1.5	24.92	-29.85
		3.	1.2	*15.4*	*-22.65*
IH-07	Dubai Marina, Dubai	4.	1.2	33	-25
		5.	1.2	41.7	-29
DG-08	Dubai Marina, Dubai	6.	1.5	27	-35.75
RT-09	Trade Centre First, Along SZ Road	7.	1.2	26.6	-32.13
		8.	1.2	21.36	-29.6
VT-10	Al Jadaf, Dubai	9.	1.5	42.25	-36.25
		10.	1.5	55	-40
PT-11	Dubai marina, Dubai	11.	1.5	55	-48
		12.	1.5	54.95	-44
		13.	1.5	54.5	-48
		14.	1.5	54.87	-44
DH-12	Dubai marina, Dubai	15.	0.9	32.34	-31.3
		16.	0.9	29	-30.3
PC 13	Dubai Waterfront, Jebel Ali, Dubai	17.	1.5	26	-25.5
		18.	1.2	22	-22.8
		19.	0.9	22	-22.5
		20.	1.2	22	-22.8
		21.	1.2	22	-19
		22.	0.9	24	-19
MA-14	Dubai Creek area, Dubai	23.	0.75	16.2	-17.10
		24.	1	21.75	-18.5
		25.	1	25.7	-20
IT-15	Dubai Marina, Dubai	26.	1.5	48.1	-31.5

Tower code	Site Location	Test Pile Number	Pile Diameter (m)	Depth of Pile (m)	Depth of O cell RL m (DMD)
		27.	1	30.85	-22
CT-16	Dubai Creek area, Dubai	28.	1.2	39	-27.5
		29.	1.2	39	-25.5
		30.	1.2	39	-25.5
BA-17	Business Bay, Dubai	31.	2	50	-39
		32.	1.8	55	-47.8
		33.	1.5	55	-38.4
VI-18	Business Bay, Dubai	34.	1.5	45	-30

4.2 Typical subsurface conditions in study area

The regional geology of Dubai is well explained in Macklin et al (2010), Purser, (1973) and Kirkham (1998). Locally, the study area typically consists of surficial loose to medium dense light brown to light grey/grey slightly gravelly, silty SAND which is mostly aeolian sand that extends to approximately 17m to 20m below natural ground level, with the presence of Sabkha at shallow depths. Sabkha is a general term used to refer to any salt flat, and the coastal Sabkha around Dubai are typically flat topographic areas of hyper-saline environments (Glennie 1996, Goodall, 1995). The underlying tertiary formation discharges brine into the Sabkha which forms an evaporated crust on the surface due to large evaporation losses when subjected to high temperature. This layer has no significance in the design of deep foundations of tall towers where the cut off level is below this layer, as shown in Fig. 5.

The underlying rock units are weak and mostly fit into the description of an Intermediate Geo-material (IGM), which is the material having compressibility and strength in between soil and hard rock as defined in AASHTO (2017). Very weak reddish-brown fine to medium grained Sandstone or Calcarenite and the Sandstone bed overlies weak, Conglomeratic Calcisiltite / Calcisiltite interbedded with Weak, Conglomerate which is followed by Siltstone, Claystone and Mudstone units in deeper layers.

Typical subsurface conditions from one of the tower sites (La Maison) in Business Bay, Dubai, are presented in Figs. 5 and 6a and 6b.

4.3 Selection of UCS values in the dataset

In order to obtain representative UCS values close to load test locations, the boreholes nearest to the load test location were given priority and the rock parameters and UCS results were taken from these locations, for the relevant depth zones. At a very few locations where UCS data were not available, Point Load Test (PLT) results were used with a UCS to PLT correlation of 4 to estimate the UCS values. This is based on typical correlations used across the UAE (Salah et al, 2014, and Elhakim, 2015). This was also verified using the data from the study region and PLT results were only used at very few locations where sufficient UCS data were not available. A statistical analysis was performed to arrive at representative UCS values, as described below. For each of the 44 pile load test results back analysed in this research, site characterisation and UCS values from the geotechnical investigation data and boreholes close to the test pile location were analysed. The test results and rock parameters from the study area are presented in the following sections.

A statistical analysis of the range of strength parameters within the study area is presented in Figs. 7a and 7b. Since most of the tall tower locations have several basements, the cut off levels of piles and barrettes are typically below the overburden soils. The piles and barrettes have been designed based on frictional resistance within the rock socket. The depth zone is selected accordingly. Both UCS and PLT test results from all the 18 locations have been collected and studied.

The UCS measurements across all 18 project sites where the 44 load tests are conducted, have been analysed statistically and average rock strength parameters are presented in Figs. 7a and 7b. The rock is weak to very weak with average UCS values in the range of 1 to 3 MPa. An average UCS profile from another similar study reported by Latapie & Lochaden (2016) and Latapie et al (2018) for the same area also supports the conclusion that the vast majority of UCS values are in a similar range, 1.5 to 3 MPa, to that shown in Figs. 7a and 7b.

5 Load Cell Test Data And Back Analysis

Only skin friction measurements from load cell test data have been used in this study. The load cell and corresponding ground parameter data have been collected from load test and foundation contractors as well as from the Dubai creative cluster authority for research purposes, and the project names are therefore confidential. All the locations are within the Dubai region around Dubai Marina to Bur Dubai, as shown in Fig. 4.

The load test data from all 44 tests conducted at 18 locations, and the geotechnical information collected at these locations, are presented in following sections along with results of back-analysis. Out of the 44 tests, 10 are load cell test data on barrettes, and the remaining 34 are for large diameter piles. Since the piles are designed as friction piles, the load cell during load test have been placed near the toe of the pile and measure predominantly the mobilized friction, which has been used in the analysis presented in this paper. At locations away from the load cell, the friction is only partly mobilized, and these data are excluded from the study. Strain gauges near the load cell only were considered in the study, together with UCS values at the corresponding depths.

The load distribution curves were plotted from the measured changes in strain gauge readings and estimated barrette properties based on cross-sectional area calculated from the average diameter from caliper report and modulus of elasticity.

Load transfer (P) at each strain gauge level was calculated as follows:

$P = \Delta \times AE_P$ {4}

where, Δ = change in strain gauge readings

AE_P = equivalent axial barrette stiffness

Modulus of Elasticity for a concrete cube (E_c) was calculated as $4700 \times \sqrt{f_{ck}}$, where f_{ck} is compressive strength of the concrete (Cobb 2009) based on the results of concrete cylinder compressive strength tests at 14 days conducted at the site. The difference between the loads at any two levels represents the shaft load carried by the portion of barrette between the two strain gauge levels. The average unit side shear between any two strain gauge levels of the barrette was calculated as the change in load divided by the surface area between the two strain gauge levels. The maximum mobilized unit side shears between the adjacent strain gauge levels were thus computed.

None of the load tests have been taken close to overall failure, and the friction close to ultimate values has only been fully mobilized close to the load cell locations and are included in the study. However, in the analysis conducted herein, the friction developed is conservatively taken as ultimate friction for the purpose of developing the correlation and therefore the developed correlation is considered to be conservative. The results of the analysis are presented below, with the results from load tests on barrettes being analysed first and presented in section 5.1. Since the data set for barrettes is relatively sparse, data from tests on large diameter piles are also included and results of analysis for both barrettes and piles are presented in section 5.2.

5.1 UCS to ultimate skin friction correlation for barrettes

Data from strain gauges closest to the load cell only were considered in the study and the representative values of UCS at each test depth zone corresponding to those respective strain gauges were collected. UCS values and the corresponding mobilized skin friction were plotted for all the ten sets of available barrette test results, and the results are presented in Fig. 8a.

Lower bound and upper bound relationship were obtained based on the data plotted. An average UCS-skin friction relationship for barrettes was derived as

$\mathbf{f_s = 0.6(UCS)^{0.5}}$ {5}

where,

f_s = maximum mobilised skin friction, MPa

UCS = Unconfined compressive strength of rock, MPa.

The above Eq. 5 is recommended for estimating the ultimate friction for barrettes in weak rock, when it is possible to verify the static capacity by testing preliminary test barrettes using bidirectional static loadcell tests (BDSLT). .

5.2 UCS to ultimate skin friction correlation for all barrettes and piles and regression analysis

UCS values and corresponding mobilized skin friction were plotted for all the 44 sets of test results available, for both barrettes and piles, and the results are presented in Fig. 8b.

For finding the relation between mobilised skin friction and UCS, a linear regression was used for the collected data. Linear regression is a frequently used and well-accepted technique to investigate the potential relationship between a variable of interest and one or more variables. For this study, an open-source tool 'RStudio' was used, which is based on programming language R, a software environment and graphics supported by the R Foundation for statistical computing.

The regression analysis shows a coefficient of 0.52 between mobilised friction and square root of UCS as the best fit with a minimal residual standard error 0.2924 and multiple R-squared of 0.8791. Hence it is suggested that the following average equation as shown in Fig. 8b can be adopted as the relationship between mobilised skin friction and UCS, for all deep foundation elements.

Mobilised ultimate skin friction $f_s = 0.52(UCS)^{0.5}$ **{6}**

Equation 6 is recommended for use to calculate ultimate skin friction developed when large diameter piles or barrettes are socketed into weak carbonate rock with UCS ranging from 0.6 to 3 MPa. It is recommended that the barrette and large diameter pile foundations for tall towers in carbonate rocks be designed using this relationship after verifying the capacity in preliminary tests. The test load for bidirectional load cell testing should then be estimated using ultimate friction predicted by this method and the design capacity should be verified for the site conditions. *5.3 Simulation of Barrette load test at La Maison Tower*

Finite element modelling to simulate the actual load test of the Barrette from the La Maison tower location was done by first modelling a single 1.2mx2.8m size barrette and 43.5m length, in Plaxis3D. The soil and barrette were modelled as volumetric elements in a Mohr Coulomb constitutive model. The boundary conditions of the soil profile at the vertical faces are defined in such a way that only vertical displacements are permitted. The bottom of the soil profile is restrained from movement. The load cell was modelled as a gap and restraints were provided to avoid movement in the X and Y directions for the gap. The vertical faces of the gap created to simulate O-cell were restrained from horizontal movement by utilizing surface displacement options. Interface elements were placed along the pile to model the interaction between the barrette and the adjoining soil. An initial stage was defined to initialise the stress in the model using K_o procedure. The volumetric barrette was then activated, and corresponding interface was also activated. Restraints for the horizontal direction for the O-cell modelled as gap was also activated at this stage before activating the loading as uniform pressure. Schematic diagram of the test simulation in the model is presented in Fig. 9 below.

The elevation considered in the model for O-cell and soil layers were as per actual conditions at the site. The design parameters used in the Mohr Coulomb constitutive model were adopted from geotechnical investigation data as presented in Table 4.

Table 4
Design parameters used in the numerical model for La Maison Tower

Strata Description	**Elevation (m DMD)**		**Thickness**	**Unit Weight**	**RQD**	**UCS**	**Φ’**	**c’**	**0.2E_d**
	Top	**Bottom**	**m**	**kN/m^3**	**%**	**MPa**	**Degree**	**MPa**	**MPa**
Gravelly sand[2]	2.88	-10.00	12.88	18	-	-	32	-	40
Calcarenite	-10.00	-14.20	4.20	20	95	2.20	39	0.20	185
Sandstone	-14.20	-23.00	8.80	20	95	0.75	41	0.07	300
Conglomerate 1	-23.00	-28.00	5.00	22	100	4.00	44	0.39	400
Conglomerate 2	-28.00	-31.00	3.00	22	100	2.50	44	0.24	400
Calcisiltite 1	-31.00	-47.00	16.0	20	70	1.17	37	0.08	640
Conglomerate 3	-47.00	-50.00	3.00	20	100	1.05	44	0.10	700
Calcisiltite 2	-50.00	-60.50	10.5	22	70	1.10	37	0.08	800
Conglomerate 4	-60.50	-63.80	3.30	20	100	1.15	43	0.11	900
Calcisiltite 3	-63.80	-90.00	26.2	22	70	1.30	37	0.08	960
Calcisiltite 4	-90.00	-100.0	10.0	20	70	1.50	37	0.10	960

Elastic modulus for concrete was taken 42.56 kN/mm2 based on the concrete cube test result from site. The concrete grade was C80 and cube test strength was 81 MPa. The generated soil profile, pile element and the soil interface, distributed load and generated mesh are shown in Fig. 10.

Loading was simulated by applying an equivalent surface pressure at the top and bottom of the load cell. Then with the calibrated model, a prescribed displacement of 200mm was applied to the calibrated O-cell model to check for the ultimate skin resistance. The prescribed settlement of 200mm was estimated as 10% of the equivalent barrette diameter, which is assumed to be the settlement at ultimate load (Tomlinson and Woodward, 2007). The interface friction generated in this model, which is the ultimate friction at the prescribed settlement, was then compared to the skin friction estimated using the proposed Eq. 6, as presented in Fig. 11.

It can be observed from the test simulation that the ultimate friction developed based on the back analysis using the criteria of 200mm (10% of equivalent pile diameter) is much higher than that estimated using the recommended correlation factor of 0.52 in Eq. 6. The correlation factor of 0.52 is therefore considered both reasonable and conservative for the purposes of barrette or pile design.

6 Foundation Design Of La Maison Using Eq. 6

In order to test the revised correlation and to check whether the revised design using Eq. 6 satisfies the serviceability limits, the La Maison tower design (Manoj et.al 2020) was revised by re-calculating the barrette length using the new correlation factor of 0.52 as presented in Eq. 6. The original design barrette length of 49m, calculated using a factor of 0.3, was revised to 35m using the factor 0.52 as in Eq. 6.

6.1 Design parameters and use of small-strain stiffness modulus from geophysics tests

The foundation design also must consider vertical displacements under serviceability limit state conditions, against the given critical load combinations (Katzenbach, 2016, Poulos and Badelow, 2016, Poeppel and Konstantinos, 2015). Several previous case studies presented load test results which indicated that the measured settlements were less than the predicted values. A good example is the Burj Khalifa, where the predicted settlements were in the range of 70mm to 80mm, whereas the actual measured settlements were around 45 to 55mm only (Russo et al, 2013). Manoj et al. (2020) demonstrated, using load test data from La Maison Tower, that the ground stiffness back analysed from load tests was 2.5 to 15 times higher than that indicated in typical site investigation reports. They also found that that the use of 0.2 times the small-strain stiffness modulus from geophysics tests (0.2 E_d) in settlement predictions, as recommended by Poulos (2017), gave results close to those from the load test results. The secant Young’s modulus used in the model was therefore estimated from down hole seismic test data as 0.2 times the E_d values. The Mohr Coulomb constitutive model was used and the design parameters used in the model were the same as that used in the single barrette model presented in Table 3.

6.2 Group modelling and settlement predictions

The total barrette group settlement and serviceability conditions were checked using a finite element model in PLAXIS 3D. Barrettes were modelled as beam elements in the group model and the raft was modelled via plate elements. Loads were applied as nodal reactions on top of the barrettes. A rigid interface R_{inter} of 1 was defined to consider the interaction between barrette and rock. The interface factor R_{inter} represents the interface between two different materials, to take into account the soil structure interaction and it is defined using the standard stiffness approach available in Plaxis.

The maximum vertical settlement at the core of the building was obtained as 40mm, as presented in Figs. 12a and 12b.

The maximum settlement from a group barrette FEM model with a barrette length of 35m, as estimated using Eq. 6, is well within the limits of 80 mm which is generally accepted as a permissible group settlement for tall towers (Poulos, 2009). This exercise demonstrates that the barrette or pile design with the revised Eq. 6 for calculating ultimate skin friction from UCS values as $f_s = 0.52(UCS)^{0.5}$, and using a modulus equal to $0.2E_d$, is validated for serviceability limits using the above numerical models.

It is noteworthy here that the final design of La Maison tower, actually approved for construction was based on calculations using ultimate skin friction from UCS values as $f_s = 0.4(UCS)^{0.5}$, and using a stiffness modulus equal to $0.2E_d$, which resulted in a design barrette length of 42m and group settlement prediction of 43mm, from finite element modelling (Manoj et.al.2020). The factor 0.4 is the lower bound value from the correlations presented in Fig. 8b.

A flow chart for design of piles and barrettes in weak carbonate rock deposits similar to those found in the Middle East region, is presented in Fig. 13.

7 Sustainability - Reduction In Carbon Footprint And Green House Gas Emissions

While tall buildings have become objects of pride for world cities and shape their skylines, they also often involve a higher unit area of embodied carbon footprint to build, compared to low rise buildings, and this then has an impact on global warming and climate change. Cement and steel, which are key ingredients in concrete, are the source of almost 8% and 11% respectively of carbon dioxide (CO_2), the largest contributor of greenhouse gas emissions of the world (Dhar et al. 2020). A more efficient design method for large diameter piles and barrettes in weak rock as recommended in this paper, will help reduce the huge volume of concrete consumption in foundation work, typically by about 30 to 35% as to a design using traditional methods. Thus, it will not only significantly reduce the foundation construction cost and time savings, but it will also reduce the carbon footprint of the tall tower. As an example, in case of the La Maison tower design, where preliminary design was performed using a correlation factor of 0.3, the revised design using a correlation factor of 0.52 resulted in an overall saving of 29% in the total length of barrettes.

8 Conclusions And Recommendations

Based on the analysis of 44 load test results from 18 tall tower locations in weak rock formations, correlations have been developed relating the ultimate skin friction f_s and UCS, for high-capacity large diameter piles and barrettes in weak rock. It is recommended that Eq. 6,

$\mathbf{f_s = 0.52(UCS)^{0.5}}$ be used for estimating ultimate skin friction in weak rock sockets where UCS is in the range of about 0.6 to 3.5 MPa.

While developing this correlation, only the data close to the load cell where maximum friction is developed, has been used. Even though the tests are not taken to failure, the friction values are assumed to be ultimate values while developing the correlation, and therefore the developed equation is considered to be conservative.

For major projects, preliminary pile load testing should be conducted by a bi-directional load cell to validate the load capacity initially estimated by above equation. For testing the working piles, techniques such as RIM cell technology or the O-Cell may be used. The load test results should be calibrated by back analysis to match the results obtained from the equation above.

For slightly higher values of UCS in the weak carbonate rocks, it is recommended that the same Eq. 6 may be used if the design will be verified by load cell testing. In the less desirable case in which there is no testing planned, the lower bound relationship of $\mathbf{f_s = 0.4(UCS)^{0.5}}$ may be used.

Based on a correlation and back analysis of ground stiffness parameters, it is recommended that for the piles or barrettes, a proper group analysis should be carried out to check settlements, using ground stiffness parameters at appropriate strain levels. Typically, a value of 0.2 times the small-strain modulus can be used within a linear elastic or elastic-plastic analysis for serviceability loadings. A flow chart for the design of piles and barrettes in weak rock, similar to the ones found in the Middle East region, is presented in Fig. 13. Use of the suggested approach should also assist in a reduction of the carbon footprint associated with foundation construction in such ground conditions.

Notation List

UCS	unconfined compressive strength
BDSLT	bidirectional static load testing
RIM	Reliability Improvement Method of bi-directional testing
RL	reduced level
E	modulus of elasticity
E_d	modulus of elasticity from dynamic tests calculated from V_s
V_s	shear wave velocity of a layer obtained from dynamic tests
f_s	the ultimate unit skin friction for the considered layer
f_b	ultimate unit end bearing for the considered layer
α	coefficient relating ultimate skin friction to UCS of rock

Declarations

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Figures

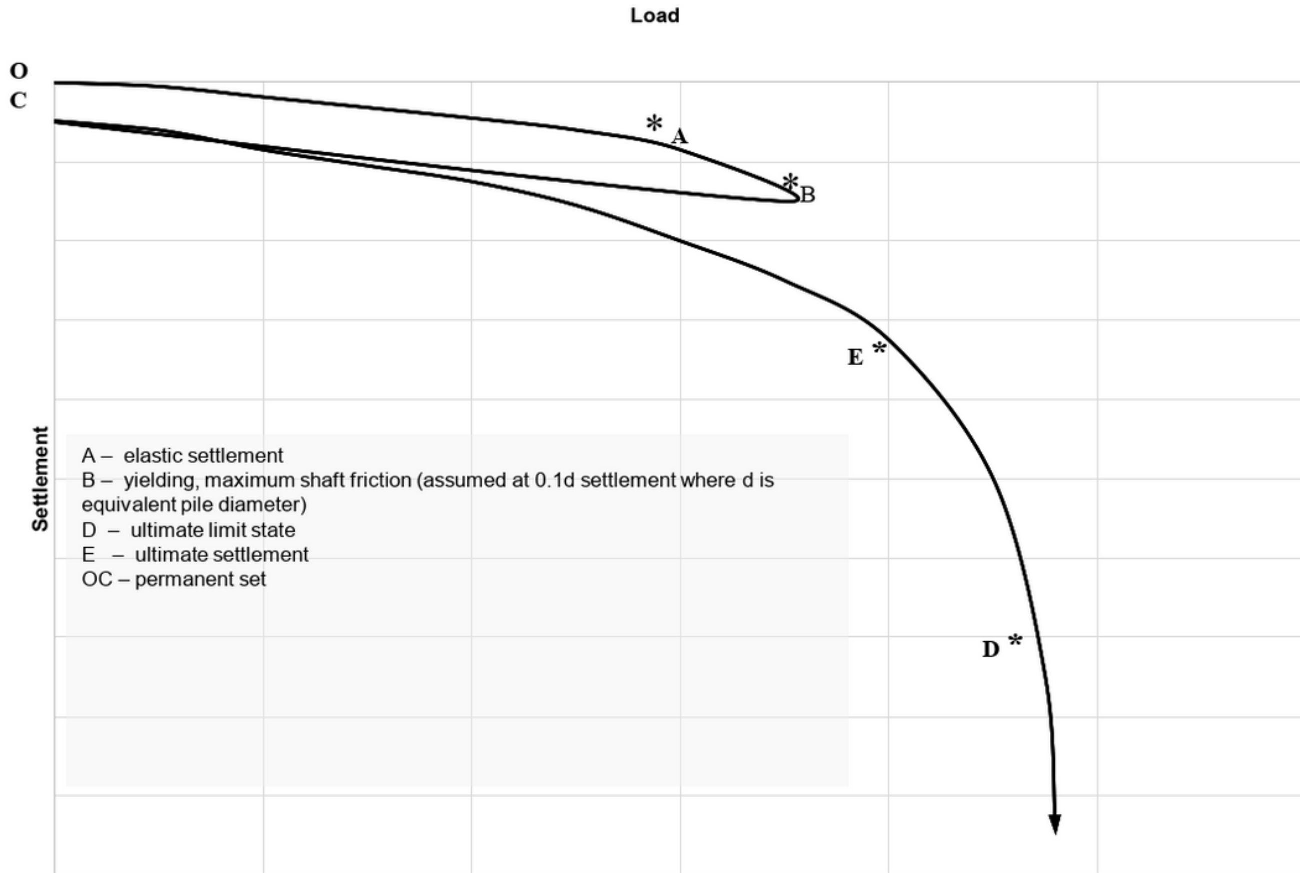
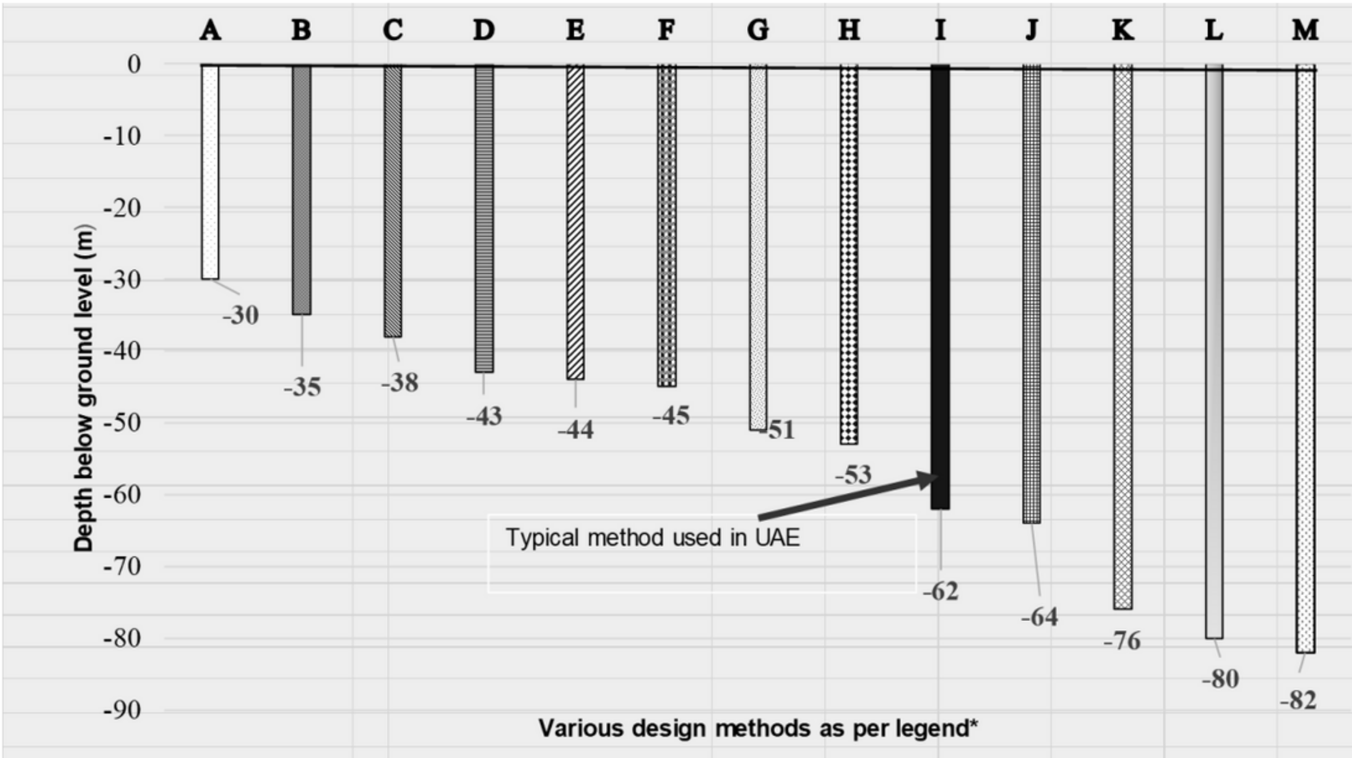


Fig 1. Typical load settlement curve for compressive load on pile

Figure 1

Typical load settlement curve for compressive load on pile



* Legend				I	Horvath and Kenney 1979
A	Zhang and Einstein (rough) 1998	E	Abbs and Needham 1985	J	Meigh and Wolski 1979
B	Rowe and Armitage Rough 1987	F	Rosenberg and Journeaux 1976	K	Carter and Kulhawy 1987
C	La Maison design	G	Horvath *1978*	L	Charif, Najjar & Sadek 2010
D	Zhang and Einstein (smooth) 1998	H	Horvath et al 1983	M	Reese and O'Neill 1988

Fig. 2. Variation in depth of barrette (1.2 x 2.8m) required to generate 40MN design capacity designed using various methods A to M (La Maison tower location)

Figure 2

Variation in depth of barrette (1.2 x 2.8m) required to generate 40MN design capacity designed using various design methods A to M (La Maison tower location)

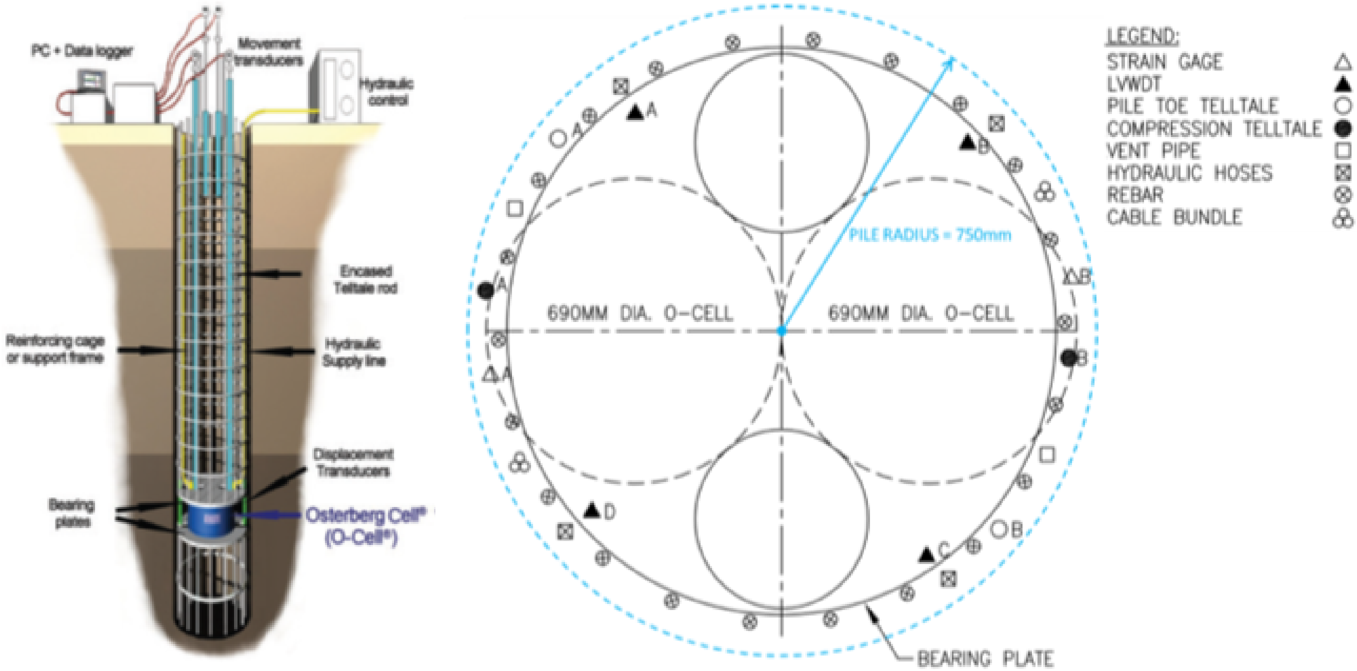
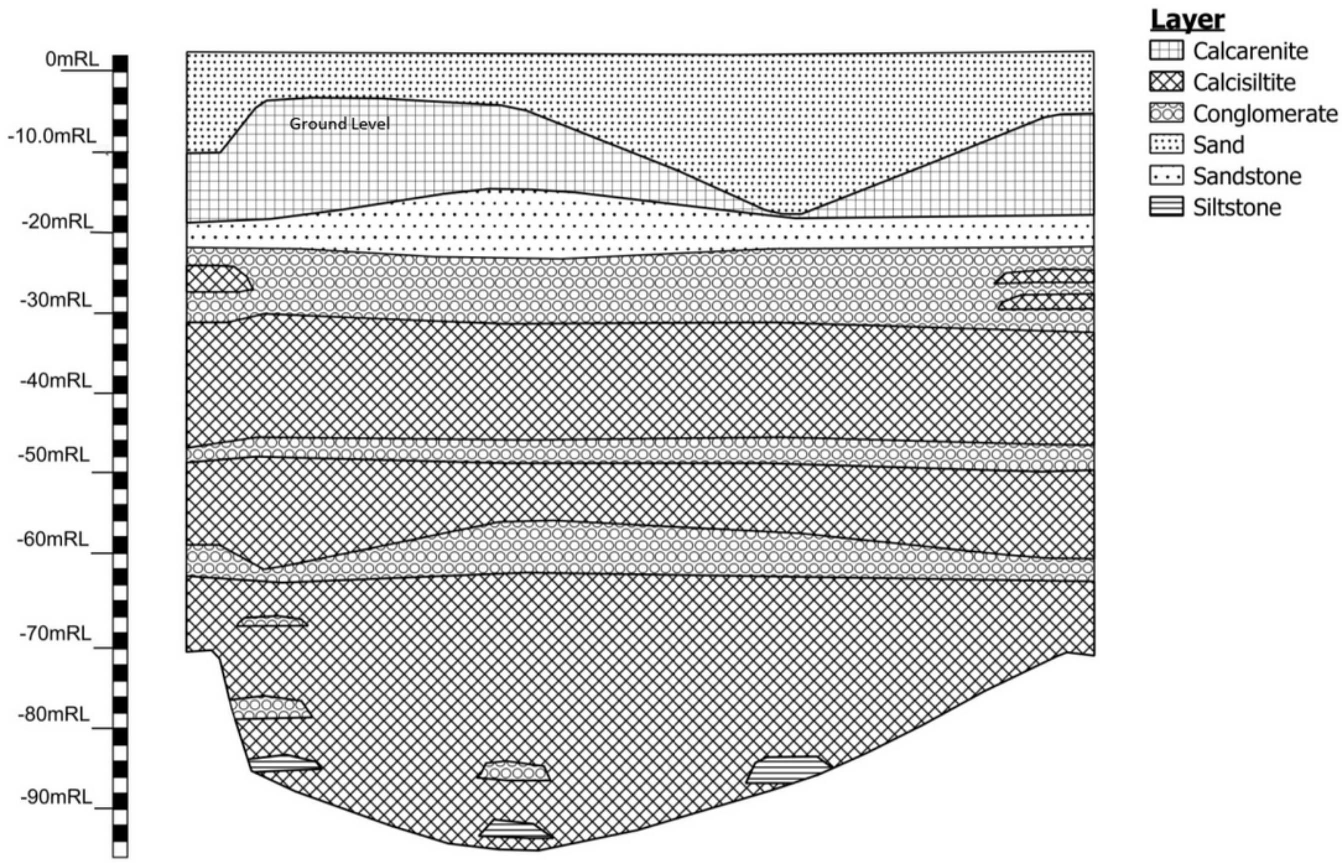


Figure 3

Typical Cross section plan and elevation of O Cell in the pile bore

Figure 4

Location map showing tower locations in Dubai, UAE from where data is collected (Google Earth Image)



Notes
1) RL in Dubai Municipality Datum, DMD
2) Ground Level = 2.88mRL

Figure 4a. Typical subsurface profile at HDS Tower 01 (La Maison Tower) location

Figure 5

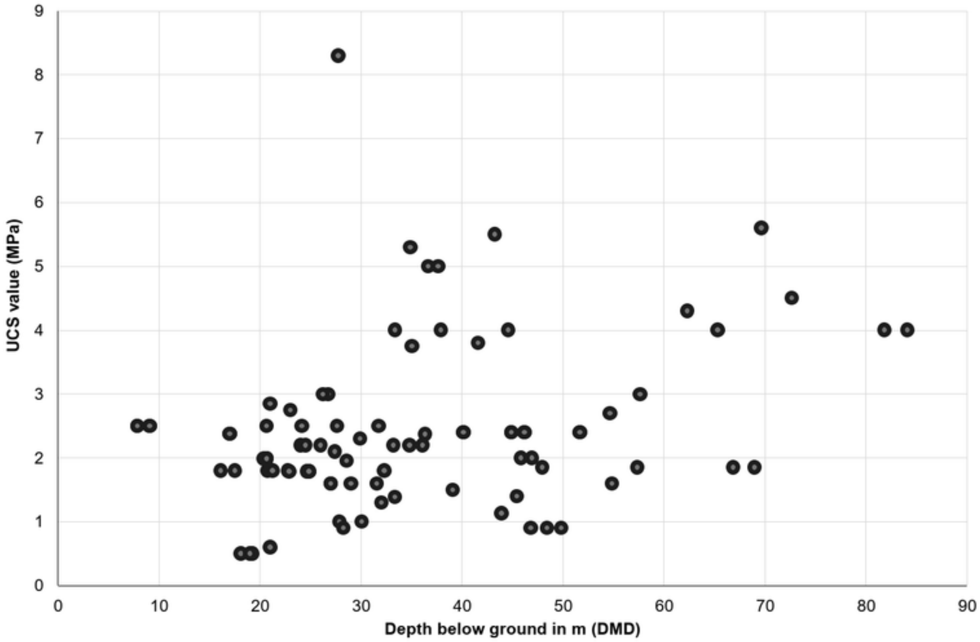
Typical subsurface profile at HDS Tower 01(La Maison Tower) location

6a 28 to 30m below ground level – Weak to medium strong Conglomerate near the top of barrette

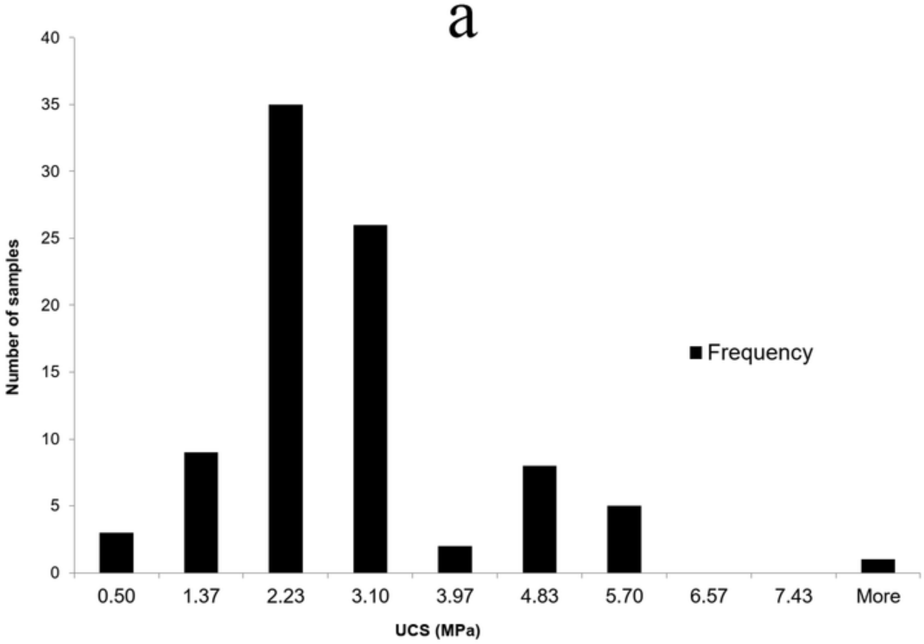
6b 50 to 55m below ground level – Very weak Calcisiltite near the zone where Load cell is placed

Figure 6

a and b Core logs showing the typical geology at significant depth zones at the O cell test location La Maison tower



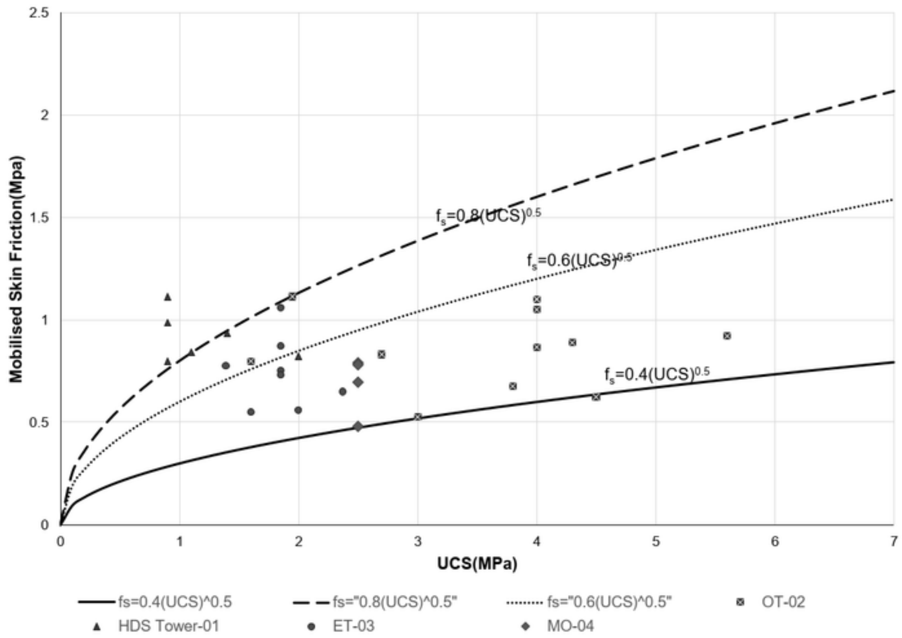
a



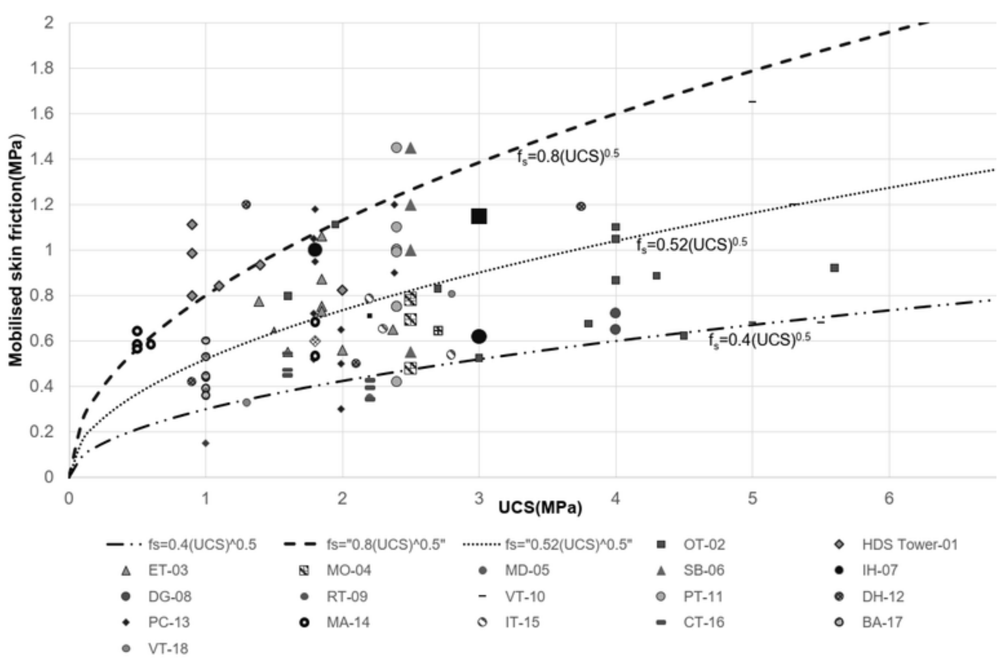
b

Figure 7

a. UCS Versus Depth within the study area b. Histogram showing UCS distribution within the study area



a



b

Figure 8

a UCS versus mobilised skin friction for barrettes b. UCS versus mobilised skin friction for barrettes and piles

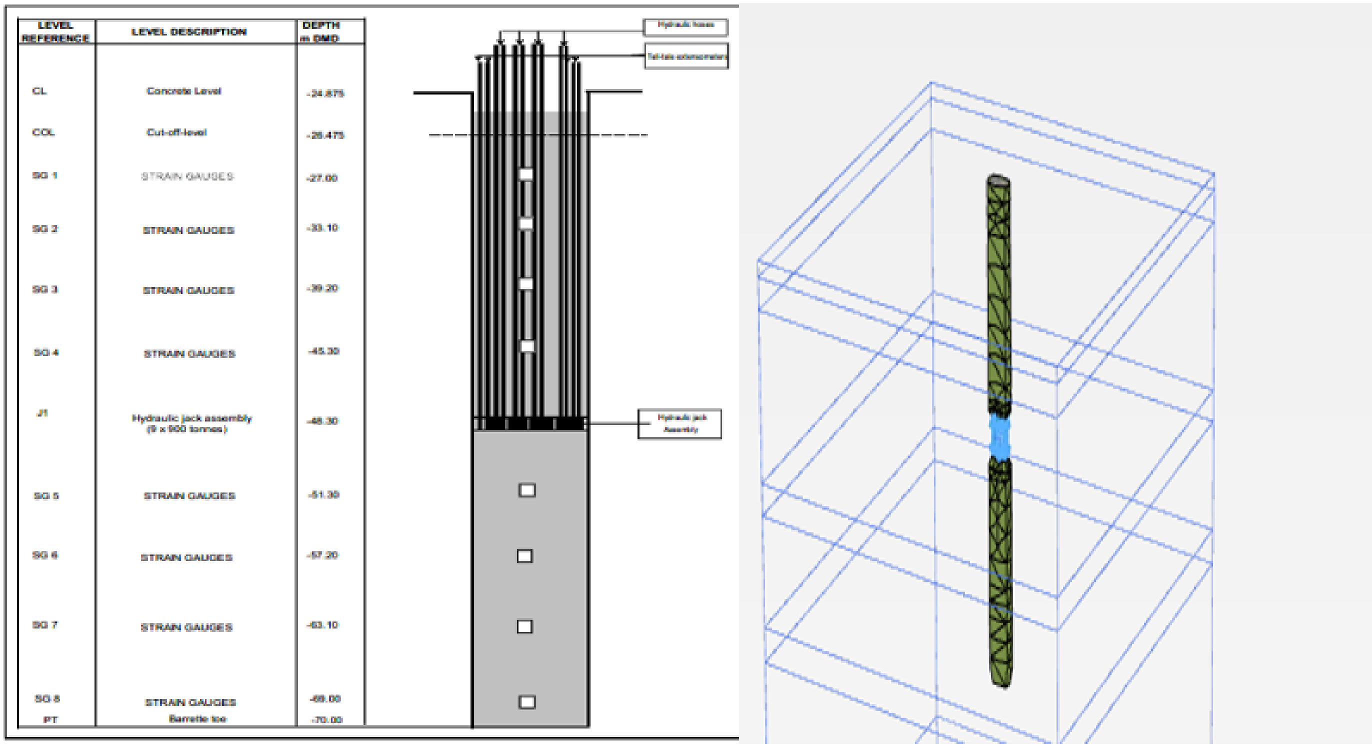


Figure 9

Schematic diagram of O cell test and O-cell Simulation in Plaxis 3D

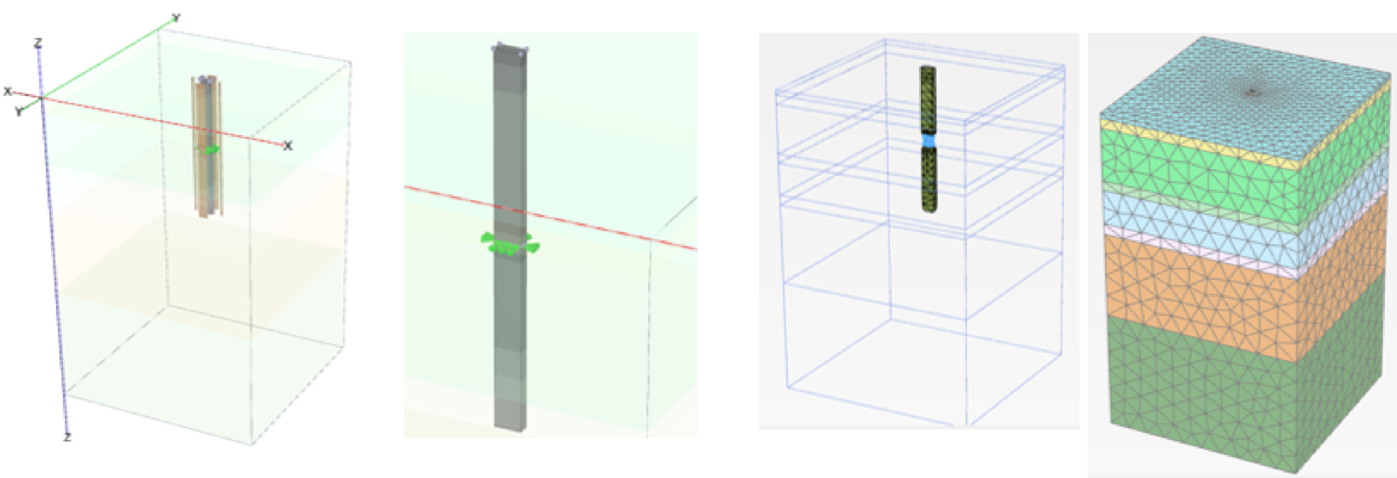


Figure 10

O-cell test simulation showing surface displacement restraint, distributed loads, and generated mesh

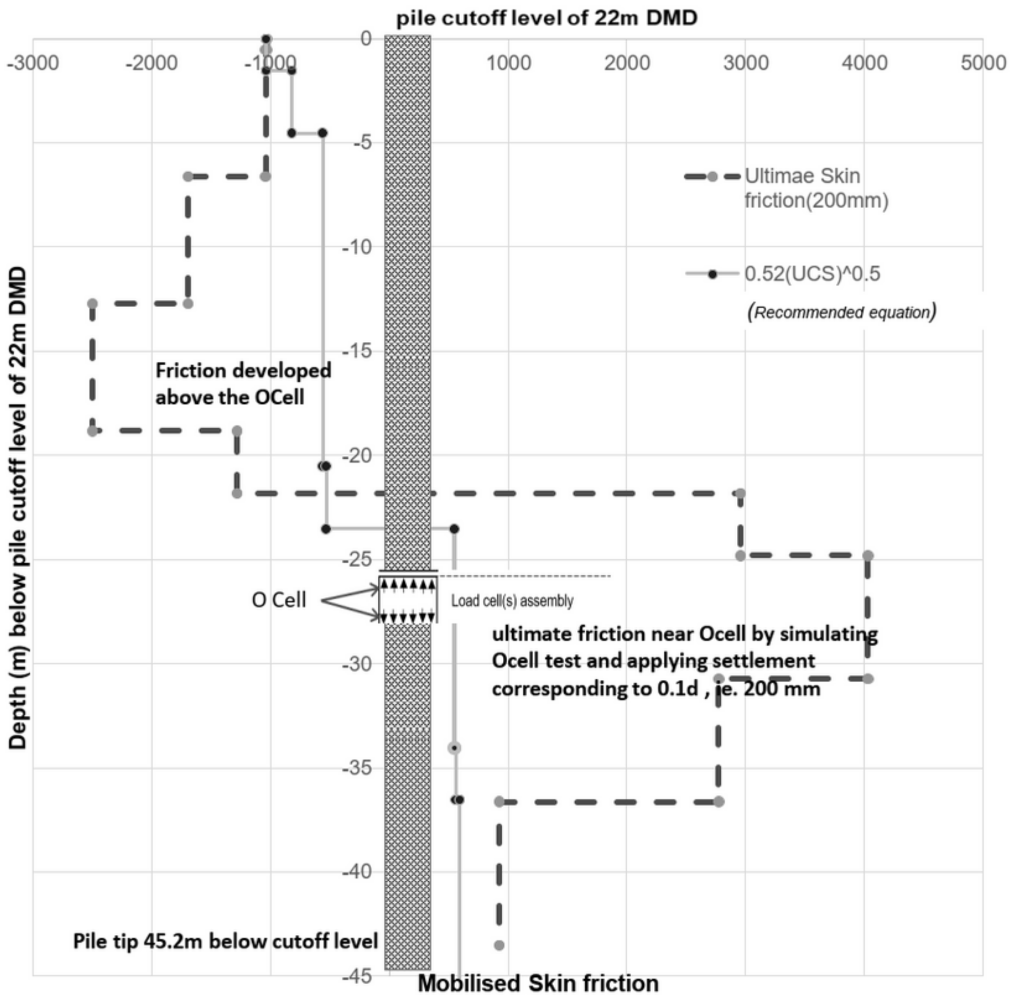


Fig. 7. Calibrated skin friction from load test compared with the predicted friction from recommended equation

Figure 11

Calibrated ultimate skin friction from load test simulation compared with the predicted friction from recommended equation

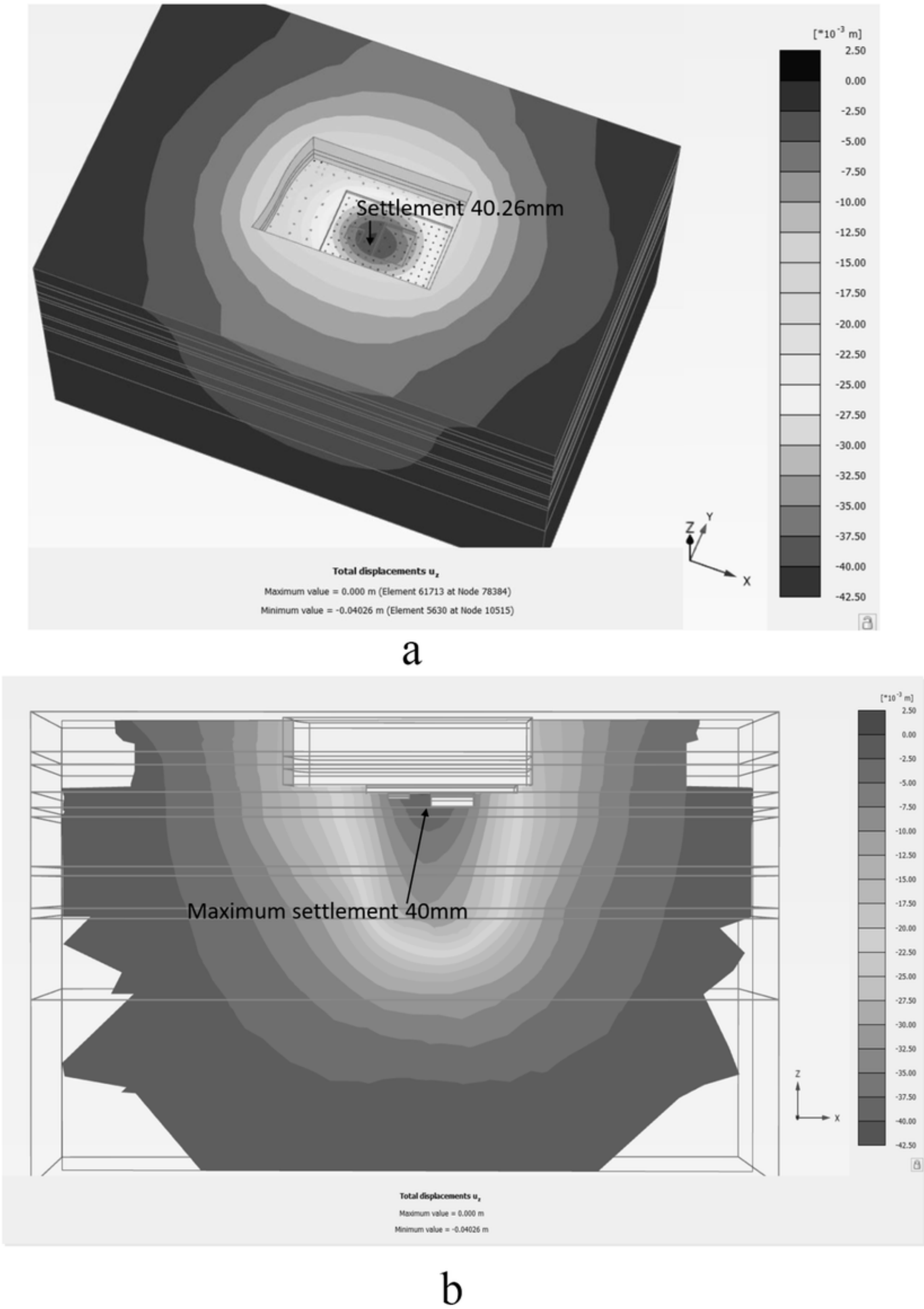


Figure 12

a. Settlement analysis using finite element Plaxis 3D Model for barrette group with 35m long barrettes, designed using a UCS to skin friction corelation factor of 0.52, at the La Maison tower location b. Cross section showing maximum settlement of barrette group with 35m long barrettes designed using a UCS to skin friction corelation factor of 0.52, at the La Maison tower location using Plaxis 3D

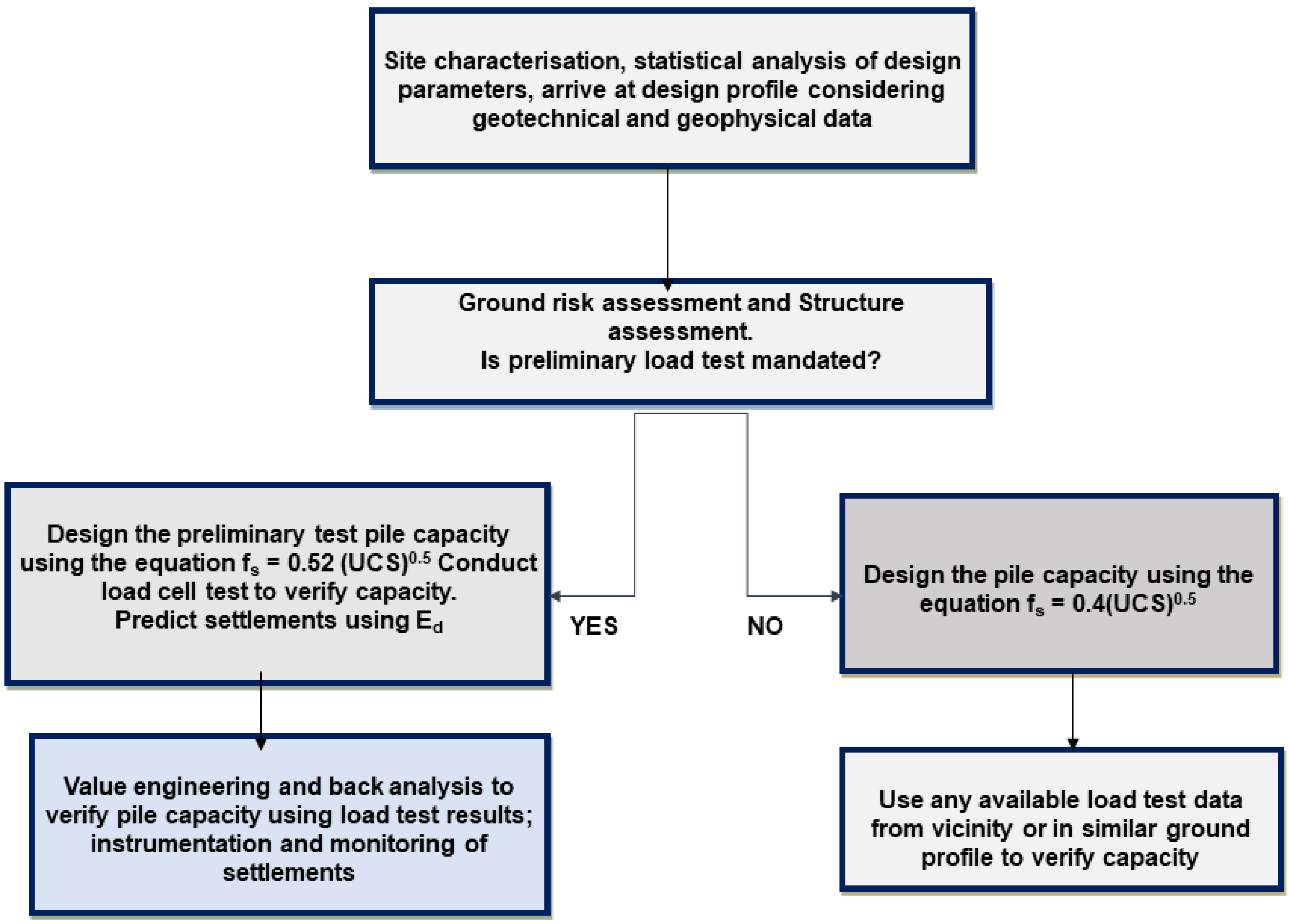


Figure 13

Design flow chart – Recommendation for design of barrettes and piles in weak rocks