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Geotechnical Parameter Assessment for Tall Building Foundation Design

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Abstract

This paper discusses the design parameters that are required for the design of high-rise building foundations, and suggests that the method of assessment for these parameters should be consistent with the level of complexity involved for various stages in the design process. Requirements for effective ground investigation are discussed, together with relevant in-situ and laboratory test techniques for deriving the necessary strength and stiffness parameters. Some empirical correlations are also presented to assist in the early stages of design, and to act as a check for parameters that are deduced from more detailed testing. Pile load testing is then discussed and a method of interpreting bi-directional tests to obtain pile design parameters is outlined. Examples of the application of the assessment process are described, including high-rise projects in Dubai and Saudi Arabia

Keywords: Case histories; foundations; geotechnical parameters; laboratory testing; in-situ testing; pile load tests

1. Introduction

"Super-tall" buildings in excess of 300 m in height are presenting new challenges to engineers, particularly in relation to structural and geotechnical design. Many of the traditional design methods cannot be applied with any confidence since they require extrapolation well beyond the realms of prior experience, and accordingly, structural and geotechnical designers are utilizing more sophisticated methods of analysis and design. In particular, geotechnical engineers involved in the design of foundations for super-tall buildings are leaving behind empirical methods and increasingly employing state-of-the art methods.

Foundation and building behaviour is highly interactive, with movements of the foundation resulting from the building loads, which in turn influence the behaviour of the building. Foundation behaviour is mainly governed by the prevailing ground conditions, the foundation type, plus the magnitude and distribution of the building loads. The foundation design of high-rise buildings should therefore be considered as a performance based soil-structure interaction (SSI) issue and not limited to traditional empirically based design methods, such as a bearing capacity approach with an applied factor of safety.

The type of foundation system for a high-rise building is dependent on the main design elements (building loads, ground conditions and required building performance) as well as other important factors like local construction conditions, cost and project program requirements. Often the subsurface conditions at high-rise building sites are far from ideal. Geotechnical uncertainty is the one of the greatest risks in the foundation design and construction process. Establishing an accurate knowledge of the ground conditions is essential in the development of economical foundation systems which perform to expectations.

This paper summarises the process of characterising the ground conditions and quantifying the relevant geotechnical parameters required for foundation design. The various stages of geotechnical assessment are discussed, together with the techniques available for estimating the geotechnical parameters. Two case histories will be described briefly to illustrate the application of these principles to practice.

2. Ground Information for Parameter Assessment

In the assessment of geotechnical parameters for foundation design, it is first necessary to review the geology of the site and identify any geological features that may influence the design and performance of the foundations. A desk study is usually the first step, followed by site visits to observe the topography and any rock or soil exposures. Local experience, coupled with a detailed site investigation program, is then required. The site investigation is likely to include a comprehensive borehole drilling and *in-situ* testing program together with a suite of laboratory tests to characterize strength and stiffness properties of the subsurface conditions. Based on the findings of the site investigation, the geotechnical model and asso-

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ciated design parameters are developed, which are then used in the foundation design process.

Increasingly, geophysical methods are being used to supplement data from conventional borehole drilling. Such methods, which include downhole and cross-hole techniques, have a number of major benefits, including:

1. They provide a means of identifying the stratigraphy between boreholes;
2. The can identify localized anomalies in the ground profile (e.g., cavities, sinkholes or localised pockets of softer or harder material);
3. They can identify bedrock levels;
4. They provide quantitative measurements for the shear wave and compression wave velocities. This information can be used to estimate the *in-situ* values of soil stiffness at small strains, and hence to provide a basis for quantifying the deformation properties of the soil strata.

The site investigation works are desirably supplemented with a program of instrumented vertical and lateral load testing of prototype piles (e.g., bi-directional load cell (Osterberg Cell) tests) to allow calibration of the foundation design parameters and hence to better predict the foundation performance under loading. Completing the load tests on prototype piles prior to final design can provide conformation of performance (i.e., pile construction, pile performance, ground behaviour and properties) or else may provide data for modifying the design prior to construction.

3. Stages of Design

There are commonly three broad stages in foundation design:

1. A preliminary design, which provides an initial basis for the development of foundation concepts and costing.
2. A detailed design stage, in which the selected foundation concept is analysed and progressive refinements are made to the layout and details of the foundation system. This stage is desirably undertaken collaboratively with the structural designer, as the structure and the foundation are an interactive system.
3. A final design phase, in which both the analysis and the parameters employed in the analysis are finalized.

It should be noted that the parameters used for each stage may change as knowledge of the ground conditions increases, and the results of *in-situ* and laboratory testing become available. The parameters for the final design stage should desirably incorporate the results of foundation load tests.

4. Design Parameters

Many contemporary foundation systems incorporate both piles and a raft, and in such cases the following parameters require assessment:

1. The ultimate skin friction for the various strata along the pile.
2. The ultimate end bearing resistance for the founding stratum.
3. The ultimate lateral pile-soil pressure for the various strata along the pile.
4. The ultimate bearing capacity of the raft.
5. The stiffness of the soil strata supporting the piles, in the vertical direction.
6. The stiffness of the soil strata supporting the piles, in the horizontal direction.
7. The stiffness of the soil strata supporting the raft.

It should be noted that the soil stiffness values are not unique values but will vary, depending on whether long-term values are required (for long-term settlement estimates) or short-term values are required (for dynamic response to wind and seismic forces). For dynamic response of the structure-foundation system, an estimate of the internal damping of the soil is also required, as it may provide the main source of foundation damping.

Moreover, the soil stiffness values will generally vary with applied stress or strain level, and will tend to decrease as the stress and strain levels increase.

5. Empirical Relationships for Preliminary Design

For piles in soil, initial assessments for preliminary design are often based on the results of simple *in-situ* tests such as the Standard Penetration Test (SPT) and the Static Cone Penetration Test (CPT). For piles in rock, the correlating factor is usually the unconfined compressive strength (UCS). Some common correlations are summarized below.

5.1. Correlations with SPT

Typical of the correlations that the authors have employed are those based on the work of Decourt (1982, 1995) using the SPT:

Raft ultimate bearing capacity:

$$p_{ur} = K_1 .\ N_r \text{ kPa} \tag{1}$$

Pile ultimate shaft resistance:

$$f_s = a.\ [2.8\ N_s + 10] \text{ kPa} \tag{2}$$

Pile ultimate base resistance:

$$f_b = K_2\ .\ N_b \text{ kPa} \tag{3}$$

Soil Young's modulus below raft:

$$E_{sr} = 2N_r \text{ MPa} \tag{4}$$

Young's modulus along and below pile (vertical loading):

Table 1. Correlation factors K_1 and K_2 (after Decourt, 1995)

Soil Type	K_1 (Raft)	K_2 Displacement Piles	K_2 Non-Displacement Piles
Sand	90	325	165
Sandy silt	80	205	115
Clayey silt	80	165	100
Clay	65	100	80

Table 2. Typical Parameters for Small-strain Shear Modulus Correlations (after Hasancebi and Ulusay, 2007)

Soil Type	X	y
Sandy soils	90.8	0.32
Clayey Soils	97.9	0.27
All soils	90.0	0.31

$$E_s = 3N \text{ MPa} \tag{5}$$

where N_r = average SPT (N_{60}) value within the depth of one-half of the raft width
N_s = SPT value along pile shaft
N_b = average SPT value close to pile tip
K_1, K_2 = factors shown in Table 1
a = 1 for displacement piles in all soils & non-displacement piles in clays
a = 0.5-0.6 for non-displacement piles in granular soils.

Many correlations have been proposed to relate the small-strain shear modulus G_0 to the SPT-N value. These generally take the following form:

$$G_0 \approx X\,[N_{1(60)}]^y \text{ MPa} \tag{6}$$

where $N_{1(60)}$ = SPT value, corrected for overburden pressure and hammer energy
X and y are parameters that may depend on soil type.

Typical values of X and y are shown in Table 2.

5.2. Correlations with CPT

Two broad approaches have been adopted for utilising CPT data to predict pile capacity:

1. Correlations between cone resistance values and both ultimate shaft friction and ultimate base capacity (e.g., De Ruiter and Beringen 1979; Bustamante and Gianeselli 1982; Poulos 1989)
2. Correlations between sleeve friction and ultimate pile shaft friction (Schmertmann 1975, 1978). This approach is often considered to be less reliable than the above method because of the difficulties in accurately measuring sleeve friction.

A useful adaptation of the method of Bustamante and Gianeselli (1982) is summarised by Frank and Magnan (1995). The ultimate shaft friction f_s and base capacity f_b are given by the following expressions:

$$f_s = q_c / k_s \le f_{sl} \tag{7}$$

$$f_b = k_b \cdot q_c \tag{8}$$

where q_c = measured cone tip resistance
k_s = shaft factor
f_{sl} = limiting ultimate shaft friction
k_b = base factor.

Table 3 gives recommended values of k_s and f_{sl}, which depend on soil type and pile type. Values of k_b are given in Table 4. Here, the value of q_c used in Eq. (8) should be the average value within a distance of 1.5 base diameters above and below the base of the pile. Excessively large and low values are excluded from the average (Bustamante and Gianeselli, 1982).

For square or circular shallow footings and rafts, MELT (1993) suggests the following correlation for ultimate bearing capacity p_{ur}:

$$p_{ur} = a_1[1 + a_2.D/B]\ q_c + q_0 \tag{9}$$

where a_1, a_2 are parameters depending on soil type and condition

Table 3. Ultimate Shaft Friction Correlation Factors for CPT Tests (MELT, 1993)

Pile Type		Clay and Silt					Sand and Gravel			Chalk	
		Soft	Stiff		Hard		Loose	Med.	Dense	Soft	Weathered
Drilled	k_s	-	-	75[1]	-		200	200	200	125	80
	f_{sl} (kPa)	15	40	80	40	80	-	-	120	40	120
Drilled removed casing	k_s	-	100	100[2]	-	100[2]	250	250	300	125	100
	f_{sl} (kPa)	15	40	60	40	80	-	40	120	40	80
Steel driven closed-ended	k_s	-	120		150		300	300	300	(3)	
	f_{sl} (kPa)	15	40		80		-	-	120		
Driven concrete	k_s	-	75		-		150	150	150	(3)	
	f_{sl} (kPa)	15	80		80		-	-	120		

(1) trimmed and grooved at the end of drilling
(2) dry excavation, no rotation of casing
(3) in chalk, f_s can be very low for some types of piles; a specific study is needed.

Table 4. Base Capacity Factors for CPT (after MELT, 1993)

Soil Type			q_c (MPa)	k_b	k_b
Clay Silt	A	soft	<3	0.40	0.55
	B	stiff	3-6		
	C	hard (clay)	>6		
Sand Gravel	A	loose	<5	0.15	0.50
	B	medium	8-15		
	C	dense	>20		
Chalk	A	soft	<5	0.20	0.30
	B	weathered	>5	0.30	0.45

ND=non displacement pile; D=displacement pile

Table 5. Parameters A and B for Ultimate Bearing Capacity of Square Shallow Footings and Rafts (after MELT, 1993)

Soil Type	Condition	a_1	a_2
Clay, silt	All	0.32	0.35
Sand, gravel	Loose	0.14	0.4
	Medium	0.11	0.4
	Dense	0.08	0.4
Chalk	-	0.17	0.27

q_0 = overburden pressure at level of base
D = depth of embedment below surface
B = width of footing or raft.

Table 5 gives recommended values of a_1 and a_2.

The small strain shear modulus G_0 has also been correlated to the measured CPT value, for example, as suggested by Mayne et al. (2009) and Mayne and Rix (1993). The latter source gives the following relationship:

$$G_0 = 406(q_c)^{0.695} \;/\; e_0^{1.130} \text{ kPa} \tag{10}$$

where q_c = cone resistance, in kPa
e_0 = initial void ratio.

5.3. Correlations with unconfined compressive strength

For piles in rock, it is common to correlate design parameters with the unconfined compressive strength, q_u, at least for preliminary purposes. Some of the available correlations are summarised in Table 6.

In employing such correlations, it should be recognised that, in the field, they may be influenced by geological features and structure that cannot be captured by a small and generally intact rock sample. Nevertheless, in the absence of other information, such correlations provide at least an indication of the order of magnitude.

More detailed correlations for rock mass modulus are provided by Hoek and Diederichs (2006), who relate the rock mass modulus to the Geological Strength Index, GSI, and a disturbance factor that reflects the geological structure.

5.4. Parameters for lateral pile response

The above correlations are for vertical loading on piles and rafts. For lateral response analyses of piles, the above correlations need to be modified, and as a first approximation, the following adjustments are suggested:

1. Young's modulus values for vertical loading should be reduced by multiplying by a factor of 0.7, to allow for the greater soil strain levels arising from lateral loading.
2. The ultimate lateral pile-soil pressure, p_y, can be approximately related to the ultimate end bearing f_b, as follows:

$$p_y = \eta \cdot f_b \tag{11}$$

where $\eta = 0.22\ (1+z/d) \le 1.0$
z = depth below ground surface
d = pile diameter or width.

6. Laboratory Testing

6.1. Triaxial and stress path testing

Conventional triaxial testing is of limited value for assessing design parameters for pile foundations, as the method of stress application does not reflect the way in which load transfer occurs from the piles to the surrounding soil. However, cyclic triaxial testing may be useful in providing an indication of the potential degradation effects on the stiffness/strength properties of the foundation ground material due to cyclic loading. For the Burj Khalifa project, cyclic triaxial test results indicated that a

Table 6. Correlations of design parameters for piles in rock

Parameter	Correlation	Remarks
Ultimate bearing capacity (raft)	$p_{ur} = a_0\, q_u$	a_0 can vary from about 0.1 for extremely poor quality rock to 24 for intact high-strength rock (Merifield et al, 2006). A value of 2 is likely to be reasonable and conservative in many cases.
Ultimate shaft friction, f_s	$f_s = a\,(q_u)^{b}$	a generally varies between 0.20 and 0.45 b in most correlations is 0.5
Ultimate end bearing, f_b	$f_b = a_1\,(q_u)^{b_1}$	a_1 generally varies between 3 and 5 b_1 in most correlations is 1.0, although Zhang and Einstein (1998) adopt b_1 = 0.5
Young's modulus for vertical loading, E_{sv}	$E_{sv} = a_2\,(q_u)^{b2}$	a_2 varies between about 100 and 500 for a wide range of rocks. b_2 is generally taken as 1.0

degree of degradation was possible in the mass ground strength/stiffness properties. However, under the anticipated applied loading, the foundations would be loaded to small strain levels and hence the potential degradation of strength and stiffness would be limited.

More sophisticated stress path testing can be useful in providing stiffness parameters over a range of stress appropriate to the foundation system. Such testing can be used to compare with values from other means of assessment.

6.2. Resonant column testing

The resonant column test is commonly used for laboratory measurement of the low-strain properties of soils. It subjects solid or hollow cylindrical specimens to torsional or axial loading by an electromagnetic loading system, usually harmonic loads for which frequency and amplitude can be controlled. It can be used to measure the small strain shear modulus and damping ratio of a soil or rock sample, and the variation in modulus and damping ratio with increasing shear strain level. Such data are valuable for carrying out dynamic response analyses of the foundation system.

6.3. Constant Normal Stiffness (CNS) testing

It is generally accepted by practitioners that there is no suitable laboratory test that can be reliably used to measure the ultimate shaft friction f_s. However, there has been a significant development over the past 20-25 years in direct shear testing of interfaces, with the development of the constant normal stiffness (CNS) test (Ooi and Carter, 1987; Lam and Johnston, 1982). The basic concept of this test is illustrated in Fig. 1, and involves the presence of a spring of appropriate stiffness against which the normal stress on the interface acts. This test provides a closer simulation of the conditions at a pile-soil interface than the conventional constant normal stress direct shear test. The normal stiffness K_n can be "tuned" to represent the restraint of the soil surrounding the pile, and is given by:

$$K_n = 4G_s / d \qquad (12)$$

where G_s = shear modulus of surrounding soil
d = pile diameter.

The effects of interface volume changes and dilatancy can be tracked in a CNS test, and the results are particularly enlightening when cyclic loading is applied, as they demonstrate that the cyclic degradation is due to the reduction in normal stress arising from the cyclic displacements applied to the interface.

7. In-situ Testing

7.1. Penetration testing

Conventional SPTs and CPTs are usually undertaken as a means of classifying and approximately quantifying the soil strata, and of facilitating estimation of geotechnical design parameters via correlations such as those mentioned above.

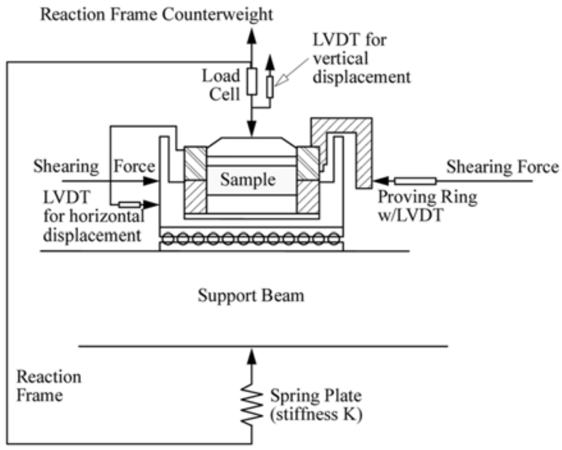


Figure 1. Constant Normal Stiffness Test Setup.

7.2. Pressuremeter testing

Pressuremeter testing can be used to estimate both strength and stiffness properties of the ground. The interpretation of test data is discussed by Briaud (1992) and Mair and Wood (1987). The stiffness values relevant to foundation design are generally the values derived from an unload/reload loop.

7.3. Geophysical testing

Geophysical testing is becoming more widely used in geotechnical investigations, offering at least three major advantages:

1. Ground conditions between boreholes can be inferred.
2. Depths to bedrock or a firm bearing stratum can be estimated.
3. Shear wave velocities in the various layers within the ground profile can be measured, and tomographic images developed to portray both vertical and lateral inhomogeneity.
4. From the measured shear wave velocity (v_s), the small-strain shear modulus, G_0, can be obtained as follows:

$$G_0 = \rho \, v_s^2 \qquad (13)$$

where ρ = mass density of soil.

From G_0, allowance must be made for the effects of shear strain within the soil which will lead to a reduction in the secant modulus value that may be useful for routine design, as discussed below.

8. Derivation of Secant Values of Soil Modulus for Foundation Analysis

For application to routine design, allowance must be

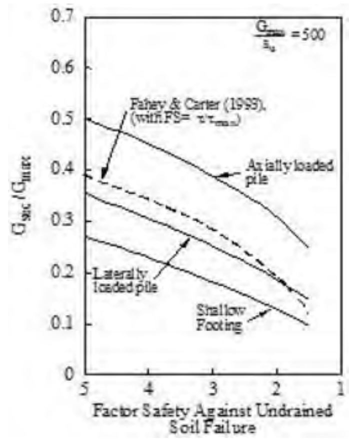


Figure 2. Typical example of ratio of secant shear modulus to small-strain value (Poulos et al., 2001).

made for the reduction in the shear modulus because of the relatively large strain levels that are relevant to foundations under normal serviceability conditions. As an example, Poulos et al. (2001) have suggested the reduction factors for clay soils shown in Fig. 2 for the case where G_0/s_u=500 (s_u=undrained shear strength). This figure indicates that:

1. The secant modulus for axial loading is about 30-40% of the small-strain value for normal factors of safety.
2. The secant modulus for lateral loading is smaller than that for axial loading, typically 30% for comparable factors of safety.

Haberfield (2013) has demonstrated that, when allowance is made for strain level effects, modulus values derived from geophysical tests can correlate well with those from pressuremeter tests. Fig. 3 reproduces such an example in which a reduction factor of 0.2 has been applied to the small-strain modulus values derived from cross-hole seismic test results. The modulus values so derived were found to be consistent with values obtained from subsequent pile load tests.

9. Pile Load Testing

9.1. Introduction

From the designer's viewpoint, pile load testing should ideally be able to satisfy the following requirements:

- provide information on all the above design issues
- be able to be undertaken on pre-production, prototype piles
- be able to be undertaken on any of the production piles without special preparation
- be relatively inexpensive
- provide reliable and unequivocal information which can be applied directly to the design process.

Some of the common methods of pile testing are summarised below, and then suggestions for the interpretation of the tests are offered.

9.2. Types of test

9.2.1. Static vertical load test

This test type is the most fundamental and involves the application of vertical load directly to the pile head, usually via a series of increments. Test procedures have been developed and specified by various codes, for example, ASTM D1143. The static load test is generally regarded

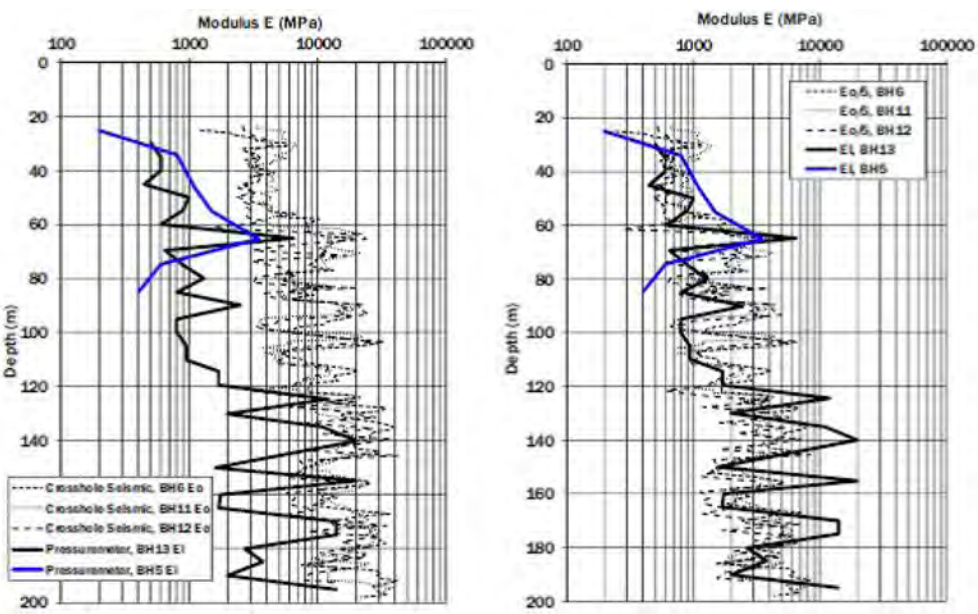


Figure 3. Comparison of modulus values from pressuremeter and cross-hole seismic tests (Haberfield, 2013).

as the definitive test and the one against which other types of test are compared. The test may take a variety of forms, depending on how the reaction for the applied pile loading is supplied. This is the type of test that the designer would like to carry out, as it best simulates the way in which a structural load is applied to the pile. Unfortunately, the ideal test cannot usually be achieved in practice, and the reaction system interacts with the test pile, thus creating some problems with the interpretation of the test data.

The usual basic information from such a test is the load-settlement relationship, from which the load capacity and pile head stiffness can be interpreted. However, such interpretation should be carried out with caution, as the measured pile settlement may be influenced by interaction between the test pile and the reaction system. Such interaction tends to lead to over-estimates of both capacity and stiffness, and therefore can lead to unconservative results, unless appropriate allowances are made for the effects of the interaction between the test pile and the reaction and/or settlement measuring system.

9.2.2. Static lateral load test

There are several forms of lateral load test, but the most common and convenient is that which involves the jacking of one pile against one or more other piles; for example, ASTM Standard D3966 outlines a procedure for lateral load testing and test interpretation.

As with the static vertical load test, there are "side effects" if two piles are jacked against other piles. In particular, because the direction of loading of each pile is different, the interaction between the piles will tend to cause a reduced pile head deflection, and as a consequence, the measured lateral stiffness of the pile will be greater than the true value.

9.2.3. Dynamic load test

The principles of the dynamic load test are very well-established (Rausche et al., 1985; Goble, 1994). The test procedure is now accepted as routine, especially for quality control and design confirmation purposes. Despite its widespread use, the dynamic pile load test has a number of potential limitations, including the fact that the load-settlement behaviour estimated from the test is not unique, but is a best-fit estimate. Two measurements (strain and acceleration versus time) are taken, and from these, the complete distribution of resistance along the pile, as well as the load-settlement behaviour, are interpreted. Also, the load is applied far more rapidly than in most situations in practice, and hence time-dependent settlements are not developed during the test. Fortunately, under normal design load levels, the amount of time-dependency (from both consolidation and creep) is relatively small as most of the settlement arises from shear deformation at or near the pile-soil interface. Hence, the dynamic test may give a reasonable (if over-estimated) assessment of the pile head stiffness at the design load. However, it is expected to be increasingly inaccurate as the load level approaches the ultimate value.

For heavily loaded foundations such as those supporting tall buildings, dynamic load testing is generally not feasible as insufficient energy can be imparted to the pile to fully mobilise its capacity. The test may however provide a convenient means of obtaining the head stiffness of a single pile.

9.2.4. Bi-directional (Osterberg cell) test

This test was developed by Osterberg (1989) while a similar test has been developed in Japan (Fujioka and Yamada, 1994), and has been used increasingly over the past decade or so. A special cell is cast in or near the pile base, and pressure is applied. The base is jacked downwards while the shaft provides reaction and is jacked upwards. The test can continue until the element with the smaller capacity reaches its ultimate resistance. Using the Osterberg cell, load tests of up to 150 MN have been carried out. Despite its ability to provide "self-reaction", the Osterberg cell test (like all tests) has its limitations and shortcomings, including the following:

1. it is applicable primarily to bored piles
2. the cell must be pre-installed prior to the test
3. there is interaction between the base and the shaft, and each will tend to move less than the "real" movement so that the apparent shaft and base stiffness will tend to be larger than the real values.

9.2.5. Statnamic test

Statnamic testing was jointly developed in Canada and the Netherlands (Middendorp et al., 1992; Bermingham et al., 1989), and has also found considerable use and development in Japan. Comparative tests on piles subjected to conventional static testing and Statnamic testing have shown good agreement in load-settlement performance (Bermingham et al., 1994).

Statnamic testing appears to offer some advantages over other test types, including:

1. the test is quick and easily mobilized
2. high loading capacity is available
3. the loading is accurately centred and can be applied to both single piles and pile groups
4. the test does not require any pre-installation of the loading equipment
5. the test is quasi-static, and does not involve the development of potentially damaging compressive and tensile stresses in the test pile.

Inevitably, there are also some potential shortcomings, including:

1. certain assumptions need to be made in the interpretation of the test, especially in relation to the unloading of the pile
2. it cannot provide information on time-dependent settlements or movements. While this may not be of

great importance for single piles, it can be a major limitation when testing pile groups, especially if compressible layers underlie the pile tips.

9.3. Test interpretation

9.3.1. Ultimate axial capacity

For conventional static load testing, it is common for the test to be stopped prior to complete plunging failure being achieved. A vast number of suggestions have been made on how the ultimate axial load capacity can be estimated from such tests, some of which have been reviewed and assessed by Hwang et al. (2003). They can be classified into the following categories:

1. "Conspicuous turning point of the load-settlement curve". This is often a subjective assessment.
2. Settlement S of the pile head, including:
   a. S =10% of diameter typically (Terzaghi, 1943).
   b. Tangent flexibility of pile head, for example, Fuller & Hoy (1971).
3. Residual settlement (Sp) of pile head. Examples include Davisson (1972), who suggests that the ultimate capacity is the load at which the pile head settlement = 0.15 + 0.1d (inches), where d = pile diameter, in inches, and DIN4026 (Germany) in which the residual settlement upon unloading from the ultimate load is 2.5% of the diameter.
4. Creep rate of head settlement, where the ultimate capacity is taken as the load at which a sudden increase in the slope of the settlement-time curve occurs
5. Coordinate transformation of the load-settlement curve, with the procedure of Chin (1970) being typical. This involves plotting the ratio of settlement to load as a function of settlement, and defining the ultimate capacity from the slope of the straight line portion of this plot.
6. Employing a specified shape of load-settlement curve, such as that employed by Hirany and Kulhawy (1989).

Hwang et al. (2003) concluded that the approach attributed to Terzaghi (1943) was preferable to many of the other approaches.

The emergence of the bi-directional cell test has facilitated the interpretation of the ultimate load capacity, since a well-designed test will permit full (or almost full) mobilization of both the shaft and base resistances.

9.3.2. Ground modulus values

Interpretation of the pile load test to assess the pile and ground stiffness characteristics requires that account be taken of the site stratigraphy. For the model of ground behaviour assumed in the pile analysis, the relevant ground parameters need to be interpreted from the measured load-settlement behaviour. For example, if a load transfer (t-z) approach is adopted, the initial slope and subsequent shape of the load transfer curves must be assumed and then the parameters for the curves derived via a process of trial and error.

If an elastic-plastic soil model is assumed, then a distribution of Young's modulus and ultimate shaft friction with depth must be assumed and again, a trial and error process will generally be required to obtain a fit between the load-settlement behaviour from the theoretical model and the measured load settlement behaviour. More often than not, there will be no instrumentation along the pile so that there is no detailed load transfer information along the pile shaft. Thus, an assumption has to be made regarding the distribution of soil stiffness and strength with depth. This needs to be done in relation to the geotechnical profile in order to obtain reliable results.

If instrumentation has been installed in the pile, and if proper account is taken of residual stresses in the interpretation of the results, then the value of Young's modulus of the ground, E_s, between each adjacent set of instrumentation can be interpreted by use of the following relationship developed by Randolph and Wroth (1978).

$$E_s = (\tau/w_s)d(1+\upsilon)\ln(2r_m/d) \quad (14)$$

where τ = local shear stress
w_s = local settlement
d = pile diameter
υ = ground Poisson's ratio
r_m = radius at which displacements become very small
τ/w_s = the slope of the derived load transfer (t-z) curve.

Randolph and Wroth (1978) give an expression for r_m and indicate that it is in the order of the length of the pile.

10. Example 1 - The Emirates Towers, Dubai

10.1. Ground investigation and site characterization

The detailed investigation involved the drilling of 23 boreholes, to a maximum depth of about 80 m. The deepest boreholes were located below the tower footprints, while boreholes below the low-rise areas tended to be considerably shallower. SPTs were carried out at nominal 1 m depths in the upper 6 m of each borehole, and then at 1.5 m intervals until an SPT value of 60 was achieved. SPT values generally ranged between 5 and 20 in the upper 4 m, increasing to 60 at depths of 8 to 10 m. Rotary coring was carried out thereafter. Core recoveries were typically 60-100% and rock quality designation (RQD) values were also between about 60 and 100%.

It was found that the stratigraphy was relatively uniform across the whole site, so that it was considered adequate to characterize the site with a single geotechnical model. The ground surface was typically at a level of +1 to +3 m DMD, while the groundwater level was relatively close to the surface, typically between 0 m DMD and -0.6 m

DMD (DMD = Dubai Municipality datum).

10.2. Parameter assessment and geotechnical model

10.2.1. In-situ and laboratory testing

Because of the relatively good ground conditions near the surface, it was assessed that a piled raft foundation system would be appropriate for each of the towers. The design of such a foundation system requires information on both the strength and stiffness of the ground. As a consequence, a comprehensive series of *in-situ* tests was carried out. In addition to SPTs and permeability tests, pressuremeter tests, vertical seismic shear wave testing, and site uniformity borehole seismic testing were carried out.

Conventional laboratory testing was undertaken, consisting of conventional testing, including classification tests, chemical tests, unconfined compressive tests, point load index tests, drained direct shear tests, and oedometer consolidation tests. In addition, a considerable amount of more advanced laboratory testing was undertaken, including stress path triaxial tests for settlement analysis of the deeper layers, constant normal stiffness direct shear tests for pile skin friction under both static and cyclic loading, resonant column testing for small-strain shear modulus and damping of the foundation materials, and undrained static and cyclic triaxial shear tests to assess the possible influence of cyclic loading on strength, and to investigate the variation of soil stiffness and damping with axial strain.

10.2.2. Test results

From the viewpoint of the foundation design, some of the relevant findings from the *in-situ* and laboratory testing were as follows:

- The site uniformity borehole seismic testing did not reveal any significant variations in seismic velocity, thus indicating that it was unlikely that major fracturing or voids were present in the areas tested.
- The cemented materials were generally very weak to weak, with UCS values ranging between about 0.2 MPa and 4 MPa, with most values lying within the range of 0.5 to 1.5 MPa.
- The average angle of internal friction of the near-surface soils was about 31 degrees.
- The oedometer data for compressibility were considered to be unreliable because of the compressibility of the apparatus being of a similar order to that of some of the samples.
- The cyclic triaxial tests indicated that the Unit 4 sand deposit had the potential to generate significant excess pore pressures under cyclic loading, and to accumulate permanent deformations under repeated one-way loading. It could therefore be susceptible to earthquake-induced settlements.
- The constant normal stiffness (CNS) shear tests (Lam and Johnston, 1982) indicated that cyclic loading had the potential to significantly reduce or degrade the skin friction after initial static failure, and that a cyclic stress of 50% of the initial static resistance could cause failure during cyclic loading, resulting in a very low post-cyclic residual strength.

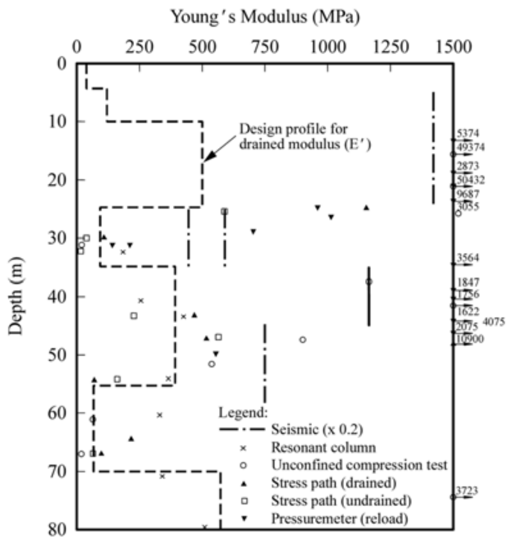


Figure 4. Summary of Young's modulus values.

Fig. 4 summarizes the values of Young's modulus obtained from the following tests:

- seismic data (reduced by a factor of 0.2 to account for a strain level appropriate to the overall behaviour of the pile foundation)
- resonant column tests (at a strain level of 0.1%)
- laboratory stress path tests, designed to simulate the initial and incremental stress state along and below the foundation system
- unconfined compression tests (at 50% of ultimate stress).

Fig. 5 shows the ultimate static shear resistance derived

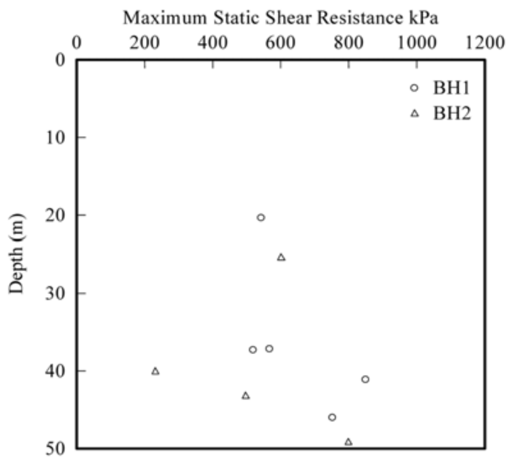
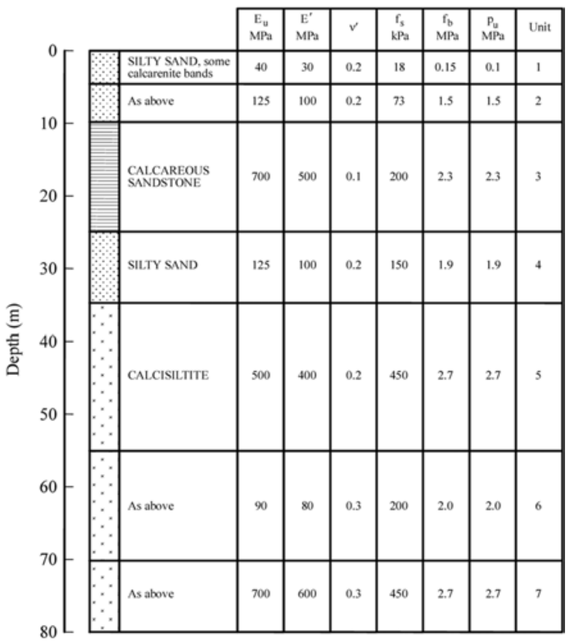


Figure 5. Ultimate skin friction values from CNS tests.



	E_u MPa	E' MPa	ν'	f_s kPa	f_b MPa	p_u MPa	Unit
SILTY SAND, some calcarenite bands	40	30	0.2	18	0.15	0.1	1
As above	125	100	0.2	73	1.5	1.5	2
CALCAREOUS SANDSTONE	700	500	0.1	200	2.3	2.3	3
SILTY SAND	125	100	0.2	150	1.9	1.9	4
CALCISILTITE	500	400	0.2	450	2.7	2.7	5
As above	90	80	0.3	200	2.0	2.0	6
As above	700	600	0.3	450	2.7	2.7	7

Figure 6. Geotechnical model adopted for design.

from the CNS test data, as a function of depth below the surface. With the exception of one sample, all tests showed a maximum shear resistance of at least 500 kPa. The measured values from the CNS tests were within and beyond the range of design values of static skin friction of piles in cemented soils suggested tentatively by Poulos (1988) of between 100 and 500 kPa, depending on the degree of cementation.

10.2.3. Geotechnical model

The key design parameters for the foundation system were the ultimate skin friction of the piles, the ultimate end bearing resistance of the piles, the ultimate bearing capacity of the raft, and the Young's modulus of the soils for both the raft and the pile behaviour under static loading. For the assessment of dynamic response under wind and seismic loading conditions, Young's modulus values for rapid loading conditions were also required, together with internal damping values for the various strata.

The geotechnical model for foundation design under static loading conditions was based on the relevant available *in-situ* and laboratory test data, and is shown in Fig. 6. The ultimate skin friction values were based largely on the CNS data, while the ultimate end bearing values for the piles were assessed on the basis of correlations with UCS data (Reese and O'Neill, 1988) and also previous experience with similar cemented carbonate deposits (Poulos, 1988). The values of Young's modulus were derived from the data summarized in Fig. 3. While inevitable scatter exists among the different values, there is a reasonably consistent general pattern of variation of modulus with depth. Considerable emphasis was placed on the laboratory stress path tests, which should have reflected realistic stress and strain levels within the various units. The values for the upper two units were obtained from correlations with SPT data.

The bearing capacity of the various layers for shallow foundation loading, p_u, was estimated from bearing capacity theory for the inferred friction angles, the tangent of which was reduced by a factor of 2/3 to allow for the effects of soil compressibility, as suggested by Poulos and Chua (1985).

10.3. Foundation analyses and performance

The behaviour of the test piles at the site was predicted on the basis of the assessed parameters, and found to agree reasonably well with the measured load-settlement behaviour. However, the analyses for the piled raft foundation systems were found to over-predict the settlements considerably, and this over-prediction was found to be attributable to an over-estimation of the pile interaction factors. The analysis of the test piles and the foundation systems for the Emirates Towers is described in detail by Poulos and Davids (2005).

11. Example 2 - Tower in Jeddah, KSA

11.1. Introduction

For a high-rise project in Jeddah Saudi Arabia, involving a tower over 390 m high, potentially karstic conditions were identified in some parts of the site. A piled raft foundation system was developed for this tower, as it was considered that such a system would allow the raft to redistribute load to other piles in the group if cavities caused a reduction in capacity or stiffness for some piles within the group. The means by which the geotechnical model and parameters were assessed is described below.

11.2. Geotechnical model

The quantitative data from which engineering properties could be estimated was relatively limited, and included the following:

- Unconfined compression tests (UCS)
- Shear wave velocity data
- Pressuremeter testing
- SPT data in the weaker strata.

Use was made of these data to assess the following engineering properties which were required for the settlement analysis, primarily, the Young's modulus of the ground deposits (long-term drained values), the ultimate distribution of pile shaft friction with depth and the ultimate pile end bearing capacity. The values adopted for the analyses are summarized in Table 7, and the procedures adopted to assess each of these parameters are described briefly below.

11.2.1. Long-term young's modulus

The assessment of this parameter is critical as it greatly

Table 7. Soil properties used for tower analysis

Depth at bottom of geo-unit (m)	Description of Geo-Unit	E_v (MPa)	f_s (MPa)	f_b (MPa)
20	Coralline Limestone (1)	450	0.2	2
50	Coralline Limestone (2)	600	0.2	9.8
70	Coralline Limestone (3)	1200	0.35	9.8
100	Coralline Limestone (4)	3000	0.4	9.8

*E_v = modulus of soil for vertical pile response; f_s = ultimate pile shaft skin friction; f_b = ultimate pile base load.

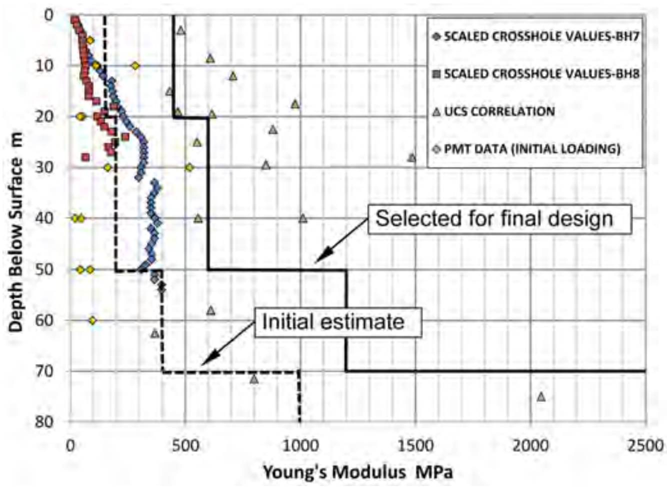


Figure 7. Young's modulus values derived from various sources.

influences the predicted settlement. Three different methods of assessment have been used:

- Modulus values from the pressuremeter (PMT) tests
- Values correlated to UCS via the correlation E_s = 100UCS, where E_s is long-term Young's modulus
- Values derived from the small-strain Young's modulus values obtained from shear wave velocity measurements, but scaled by a factor of 0.2 to allow for the effects of practical strain levels.

Fig. 7 compares the values obtained from each of these three approaches.

On the basis of these data, the following assumptions were originally made:

- From the surface to a depth of 20 m, an average long-term Young's modulus (for vertical loading), E_s, is 150 MPa
- From 20 m to 50 m, Es = 200 MPa
- From 50 m to 70 m, Es = 400 MPa
- Below 70 m, Es = 1000 MPa, which reflects the greater stiffness expected because of the smaller levels of strain within the ground at greater depths.

Subsequent to these initial assessments, a pile load test was undertaken using the Osterberg Cell technique. The pile head stiffness derived from this test was considerably larger than that implied by the initially-selected values of Young's modulus. Accordingly, the initially-selected values were multiplied by a factor of 3 for the final settlement prediction.

This is an example of a case where there was no detailed laboratory testing, so that the parameters derived from empirical correlations were adjusted on the basis of the pile load tests.

11.2.2. Ultimate pile shaft friction and end bearing

Use was made of correlations between the ultimate shaft friction, f_s, and end bearing, f_b, with UCS. For the reefoidal coral deposits, the following conservative relationship was used for our assessment.

$$f_s = 0.1(\mathrm{UCS})^{0.5} \text{ MPa} \tag{15}$$

where UCS = unconfined compressive strength (MPa)

The average ultimate shaft friction for the upper 50 m was thus taken to be 0.2 MPa (200 kPa). The subsequent pile load test revealed that this was a conservative estimate of shaft friction, as values of about 500 kPa were mobilized along some portions of the test pile, with an average value of about 310 kPa.

The following correlation suggested by Zhang & Einstein (1998) was employed for base ultimate capacity:

$$f_b = 4.8\ (\mathrm{UCS})^{0.5} \text{ MPa} \tag{16}$$

On this basis, for an average UCS of 4 MPa, f_b was 9.6

MPa. This value assumes that there are no cavities in the area of influence of the base of a pile.

11.2.3. Analysis of foundation system

The foundation analysis has been described by Poulos et al. (2013). The initial foundation analyses assumed that no significant cavities exist below the pile toes. If cavities were to be found during construction, then it would be necessary to re-assess the performance of the foundation system and make provision for grouting of the cavities if this was deemed to be necessary. Thus, subsequent to the foundation design, a further series of analyses was undertaken to investigate the possible effects of cavities on the settlements and also on the raft bending moments and pile loads. For these analyses, the commercially-available finite element program PLAXIS 3D was used.

Firstly, the effect of a single cavity at different locations along the centre line of the raft at different depths was examined. The cavity was introduced into the finite element mesh at various depths. Generally the locations of cavities beneath the foundation are not known, and only cavities found in specific boreholes can be precisely located. The effect of cavities at random locations and of random sizes was therefore gauged. To this end, a random number generator was used to select the location and size of the cavity. The centre of the cavity was constrained to lie within the footprint of the raft, and the depth was constrained to be within a 70 m depth. The number of randomly placed cavities was limited to 5 for each of the cases analysed.

It was found that the consequences of cavities, while not insignificant, may not be as serious as might be feared, because of the inherent redundancy of the piled raft foundation system. The maximum settlement increased from about 56 mm to 74 mm for the range of cases examined. The maximum bending moment in the raft was increased by only about 13%.

While the analyses undertaken were insufficient to enable a quantitative assessment of risk to be assessed, they did give a good appreciation of the sensitivity of the computed foundation response to the presence of random cavities. Clearly, using redundant foundation systems may not only reduce the risks associated with building towers on karstic limestone but also provide a much more economical foundation than using deep foundation piles in an attempt to carry foundation loads through the karstic zones.

12. Conclusions

This paper has set out a process for characterising the ground conditions and quantifying the relevant geotechnical parameters required for tall building foundation design. Three stages of parameter assessment have been described:

1. Preliminary assessment via empirical correlations. Such parameters can be used for preliminary design and checks on parameters derived by other means.
2. Detailed assessment via *in-situ* and laboratory tests. These will usually form the basis for detailed foundation design.
3. Parameters derived from pile load testing. These can be used to confirm the design assumptions and the adequacy of the construction techniques, and can also provide information for adjusting the foundation design before, or during, the construction process.

Two examples have been provided of how the combination of the three stages of parameter assessment can be used to characterise the ground conditions and provide the necessary information for the foundation design.

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