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The Design of Foundations for the World’s Tallest Buildings

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THE DESIGN OF FOUNDATIONS FOR THE WORLD'S TALLEST BUILDINGS

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ABSTRACT

The talk presents the essential requirements for the design of foundations for the world's tallest buildings from a geotechnical perspective, discusses briefly the basic foundation types and several key principles to remember, including the need for close structural engineer and geotechnical engineer cooperation. The special in-situ testing and load testing techniques commonly used are also presented. International case histories where performance has been monitored are used to illustrate some of the basic points as well as to compare prediction with performance. As an additional feature, the experience of gradually increasing allowable bearing pressures in a given geology over a sufficient time span to observe performance is also presented using Chicago high-rise experience.

ESSENTIAL REQUIREMENTS FOR FOUNDATION DESIGN

An essential requirement for cost effective foundation design is good communication between the structural engineer and the geotechnical engineer. In the writers' experience, the best results occur when the structural engineer and the geotechnical engineer work as a team, have mutual confidence in each other's competence and experience and develop the exploration program together.

An adequate subsurface exploration program will have sufficient borings for general stratigraphy using routine boring and sampling procedures, and will also have selected borings for undisturbed sampling for triaxial and consolidation testing. In addition, special testing like in-situ pressuremeter tests, cone penetration tests, dilatometer tests, and geophysical testing for shear wave velocity should be performed.

Foundation analysis will include settlement prediction and bearing capacity analysis using simple and approximate methods for obtaining quick order of magnitude values and then fine tuning with more complex methods involving finite element programs where the size and complexity of the project warrants the additional analysis.

The design of the tallest buildings today involves instrumented load test programs, since in many cases, loads are sufficiently high to require design values above local code standards. The load test program is preferably done as part of the design analysis in advance of construction, but in some cases the load test program is done as the first part of construction to confirm assumed design values. Four types of load tests are available: conventional tests with a load frame, the Osterberg load cell which is used in bored piles or drilled shafts deep enough so that shaft resistance can be balanced against end bearing to test for maximum friction and end bearing in a single test, and the third type of test which is a dynamic test wherein a dynamic force is applied by a falling weight with the blow cushioned and effects monitored using procedures similar to the pile dynamic analyzer. In the fourth type the force is applied by explosive gas pressure and effects monitored in a process called the Statnamic test. While both of the case histories presented later in this paper use conventional load test frames, many of the current tall buildings that are going into construction have used the Osterberg cell test because of its higher capacity potential, lower cost and convenience.

Finally, an essential requirement is appropriate construction observation and settlement monitoring. This requires experienced observers during excavation to see that the foundations are installed as designed and that the design assumptions are felt to be valid. Strain gauge and pressure cell instrumentation of foundation elements are required to confirm how the load is actually distributed along or beneath the foundation element or shared between elements.

FOUNDATION TYPES FOR VERY TALL BUILDINGS

The foundation type used depends on the site geology. Where rock is shallow, mats or footings on rock can be used. Where a dense stratum is overlain by soft deposits, piles or drilled shafts bearing in the dense stratum can be used. The deepest driven piles the writers are aware of are in the Jin Mao project in Shanghai which went approximately eighty meters to a dense granular bearing strata. The tallest buildings in Chicago are supported on rock socketed caissons which have been extended through soft deposits to rock. Long friction piles are used where normally consolidated sediments are extensive such as New Orleans or Las Vegas.

The fourth foundation type is a combination of a mat supported on piles, drilled shafts or barrettes (rectangular piles constructed with a slurry wall excavator) where the load is carried partially by the piles and partially by the mat. Examples of this type are the Petronas Towers in Kuala Lampur and the 101 Financial Center in Taipei, which are two of the current world's tallest buildings. The Burj Khalifa, the tallest building in the world just recently completed also utilizes this combination of a mat on piles.

Some principles to remember:

1. There is no geotechnical limit to friction piles. Friction piles can always be made long enough that structural capacity governs, provided the friction deposit is deep enough and the soil and/or rock is drillable.
2. For a mat on friction piles in similar material, the load will be shared between mat and piles based on relative modulus and area based on calculated compression of piles and soil including significant stressed zone below the piles.
3. Where the ground alone is strong enough to support the building with mat only but settlement is the issue, the purpose of the piles is primarily to reduce the settlement, i.e. stiffen the ground. The longer the piles, the less the settlement as more of the stress bulb is in the "stiffened" ground.

CASE HISTORIES

Petronas Towers, Kuala Lumpur in Malaysia

The first case history for this paper is the Petronas Towers, Kuala Lumpur, Malaysia, which until recently, were the world's tallest buildings, 10.9 meters taller than the 110 story Willis Tower in Chicago, Illinois.

The Petronas Towers are also believed to have the world's deepest building foundations. The Petronas Towers barrette foundations extend to a maximum depth of 130 meters below grade in soil and weathered rock; plus ground improvement cement grouting was performed to depths up to 162 meters. Thus, measured from the bottom of the deepest foundations to the top of the building, Petronas Towers would measure either 582 meters (1909 feet) or 614 meters (2014 feet) depending upon whether the ground improvement was considered part of the foundation system.

Soil and Bedrock Conditions. A generalized soil and bedrock profile below the towers is shown in Fig. 1. The geologic profile consists of 12 to 20 meters (39 to 66 feet) of medium dense, silty and clayey alluvial sand. The alluvium is underlain by a medium dense to extremely dense, sandy and gravelly silt and clay material which is a residual soil and weathered rock deposit known locally as the Kenny Hill Formation. The bedrock below the Kenny Hill is of Silurian age and consists mainly of calcitic and dolomitic limestone and marble. The rock surface is very irregular and has been weathered by solution activity creating numerous joints and cavities. As a result of the solution activity, isolated zones of the Kenny Hill have eroded into the bedrock cavities creating soft or loose zones referred to as slump zones. The hard Kenny Hill arches over these slump zones so they do not feel the full weight of the overlying formation.

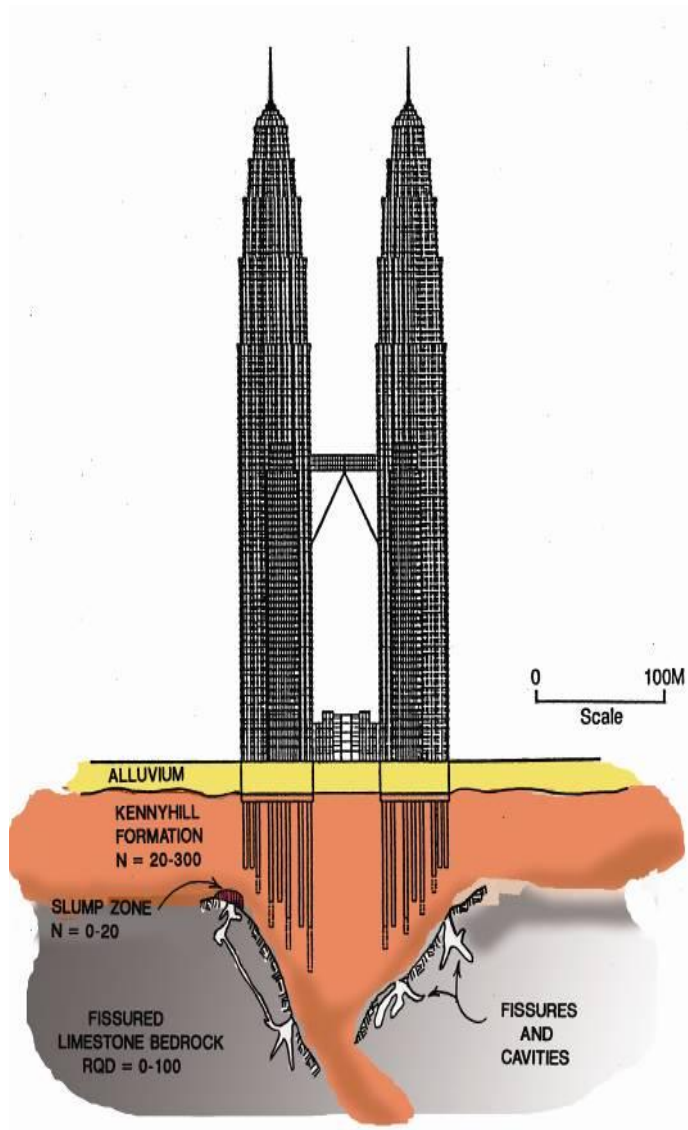


Fig. 1. Petronas Towers Foundation Profile

The rock surface dips steeply from northwest to southeast such that the tower bustles are situated over bedrock located 80 to 90 meters (260 to 295 feet) below street grade. The towers themselves are situated with rock at 100 to 180+ meters (330 to 590+ feet) below street grade. As shown in Fig. 1, there is also a valley feature in the bedrock surface between the towers extending deeper than 200 meters. (658 feet).

Foundation Requirements. Due to the height, slenderness and structural interconnection of the towers, the developer and the designer aimed for predicted differential settlement as close to zero as practical (less than 1/2 inch, or 13 millimeters across the base of each tower).

With the anticipated geology and the goal of minimizing differential settlement, foundation alternatives studied included a "floating" raft, a system of bored piles socketed into limestone below any significant cavities, and a raft on friction piles located in the Kenny Hill well above the limestone (grouting cavities and slump zones as necessary), with pile lengths varied to minimize differential settlement. The large size and great strength and stiffness requirements of a "floating" raft precluded its use. The great depth to bedrock made socketed bored piles impractical. Therefore, the friction pile scheme was used. During the preliminary design and soil exploration phase, it was found that bedrock elevation at the initial tower locations varied so greatly that rock actually protruded into the proposed basement on one side of the tower. This made control of differential settlement impractical. The tower locations were then shifted approximately 60 meters (196.9 m) to where the thickness of the Kenny Hill formation was sufficient to support a raft on bored friction piles. There the required differential settlement limitation could be achieved by varying the length of piles or barrettes.

Exploration Program. The exploration program consisted of more than 200 borings and 200 probes on 8 meter centers in the mat areas to check for major cavities. In addition, 260 in-situ pressuremeter tests and 2 fully instrumented 3500 ton (31,000 kilonewton) pile load tests were performed to define the modulus properties of the supporting Kenny Hill formation. The pressuremeter test summary is shown in Table 1.

Table 1. Pressuremeter Test Results

Boring	B14	B23	T1-10	T1-24	T1-54	T2-26	T2-54
Ed Min. Max. # of Tests Avg.	9.3 MPa 99 18 37.6 MPa	10 MPa 309 15 133.9 MPa	32 MPa 683 27 69.9 MPa	17.8 MPa 222 26 109.8 MPa	38.5 MPa 199.4 26 101.8 MPa	18.3 MPa 157 31 64.1 MPa	11.7 MPa 470 27 149 MPa
Er Min. Max. # of Tests Avg.	27.5 479 17 186.9 MPa	22.3 931 15 391.8 MPa	55 851 27 176 MPa	32 496 25 226 MPa	57.7 590.3 25 223 MPa	47.8 495 31 190 MPa	68.3 383.3 27 535 MPa
Overall weighted Ed Avg. = 94.3 MPa ER Avg. = 267 MPa							

A representative Standard Penetration Resistance profile is shown in Fig. 2.

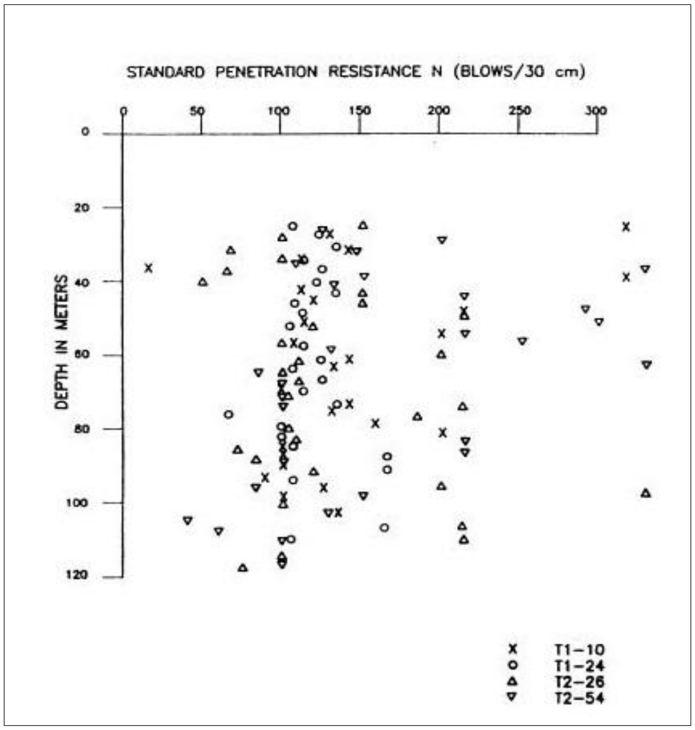


Fig. 2. Standard Penetration Resistance Profile

<u>Load Test Program.</u> The load tests were of the Kentledge dead load reaction type with house high blocks of concrete providing the reaction, as shown in Fig. 3. The results of the load tests are shown in Fig. 4. Both test piles were 70 meters long and constructed under bentonite slurry. One test pile was post grouted to break through any filter cake development. Further details are in Baker, et. al., 1998.

Fig. 3. 3,000 Ton Kentledge Load Test.

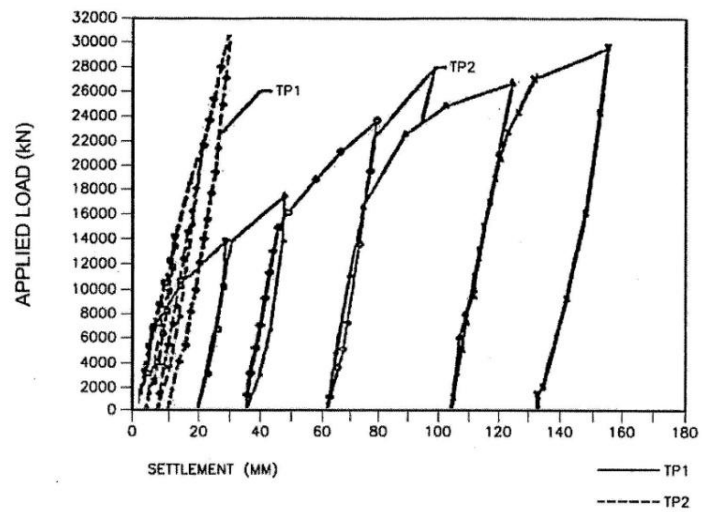
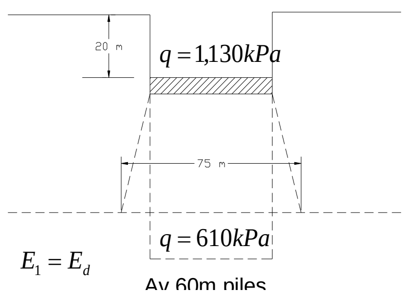


Fig. 4. Load Test Results for Post Grouted Test Pile (TP1) and Ungrouted Test Pile (TP2)

Settlement Analysis and Assumptions. Settlement analyses were performed using the equivalent footing method and simple hand calculations as shown on Fig. 5. Extensive settlement analyses were also performed utilizing the SAP 90 program and the Plaxis 3-D program using soil modulus estimates based on back calculation from the test pile program and from averaging the reload modulus slopes of the in-situ pressuremeter tests. Pile lengths were varied until calculated maximum differential settlement goals were achieved. Based on bearing capacity considerations only, barrette lengths of 33 meters would have been sufficient to support the design loads, but final pile lengths under the main towers varied from 40 meters to 105 meters based on settlement considerations.



$E_2 = E_d$

$E_{3,4,5} = 20 \times E_d$

$$E_B = \frac{3.2}{\frac{1}{E_1} + \frac{1}{0.85 \times E_2} + \frac{1}{E_{3,4,5}}}$$

$$E_B = \frac{3.2}{\frac{1}{94} + \frac{1}{0.85 \times 94} + \frac{1}{20 \times 94}}$$

$E_B = 135 MPa$

Pressuremeter Data

$E_{d_{Av}} = 94.3 Mpa$

$E^{+}{}_{Av} = 267 Mpa$

$\alpha = \frac{E_d}{E^{+}} = 0.35$, Use 0.4

Settlement Calculation – Menard Empirical Method

$$s_{Menard} = \frac{1.33}{3 \times E_B} q R_0 \left(\lambda_2 \frac{R}{R_0} \right)^{\alpha} + \frac{\alpha q \lambda_3 R}{4.5 E_1}$$

$\lambda_2, \lambda_3 = 1$ for a circle

$R_0 = 30cm$

$$s_{Menard} = \frac{1.33}{3 \times 135} \times 0.610 \times 30 \left(\frac{7{,}500}{30} \right)^{0.4} + \frac{0.4 \times 0.61 \times \frac{7{,}5}{2}}{4.5 \times 94}$$

$s_{Menard} = 0.55cm + 2.16cm = 27.1mm$

Settlement Calculation – Elastic Theory

$$s_{Elastic} = \frac{\mu_0 \mu_1 q B}{E}$$

$$s_{Elastic} = \frac{0.35 \times 0.92 \times 6{,}100 \times 75{,}000}{250{,}000} = 59mm$$

Elastic Compression of Shaft Down to Equivalent Footing Level

$$\Delta \ell = \frac{\sigma L}{E_{conc}}$$

$$\sigma = \frac{2{,}680{,}000 kN}{82 \times 1.2 \times 2.8} = 9{,}727 \frac{kN}{m^2}$$

$E_{conc} \cong 27{,}000{,}000 kPa$

$$\Delta \ell = \frac{9{,}727 \times 40{,}000}{27{,}000{,}000} = 14.4mm$$

Total Predicted Settlement

By Menard Empirical Method

$S = s_{Menard} + \Delta \ell$

$S = 27.1mm + 14.4mm = 41.5mm$

By Elastic Theory

$S = s_{Elastic} + \Delta \ell$

$S = 59mm + 14.4mm = 73.4mm$

Fig. 5. Settlement Analysis

Figure 6 shows the predicted settlement and ground deformation for the final design case from Baker, et. al. 1994. Max predicted differential was about 12 mm. Calculated average settlement from the equivalent footing method and average uniform conditions, ranged from 41 mm using the Menard rules to 73 mm based on elastic theory. This brackets the computer generated values using actual pile length and rock slope geometry;

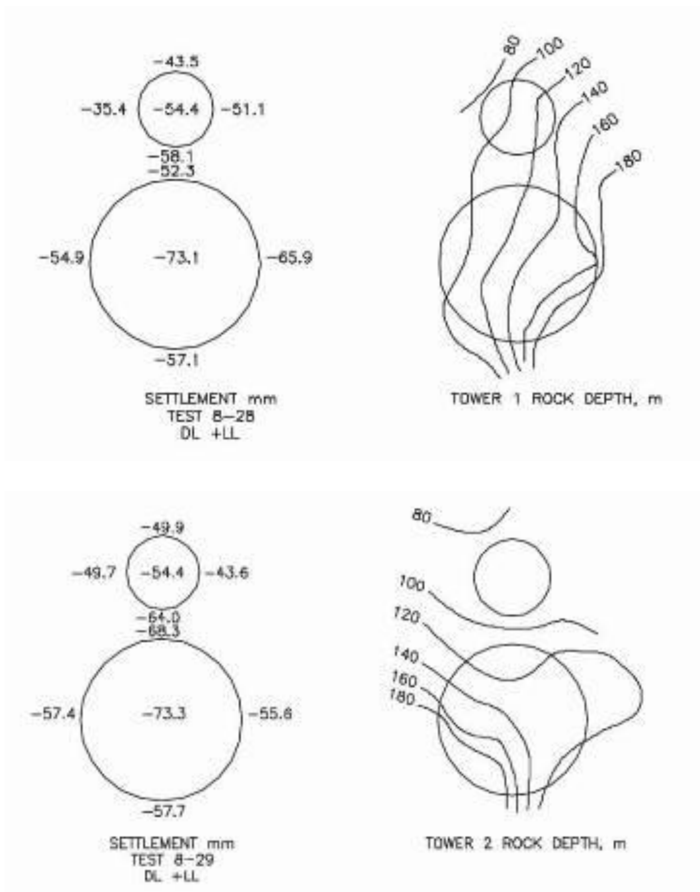


Fig. 6. Predicted Settlement Maps and Rock Contour Plan – Tower 1 (top) – Tower 2 (bottom)

Details of both the soil property information obtained, design parameters developed and settlement analyses performed are given in Baker, et. al., 1994.

<u>Required Ground Improvement, Foundation Installation and Instrumentation.</u> Since the boring and probing program uncovered a number of significant cavities in the limestone and slump zones at the limestone interface beneath the tower footprints, there was concern for potential unpredictable future settlement unless these zones were treated. The goal was to fill the voids in the limestone to make it relatively incompressible and to improve the slump zone areas so that they could be considered to act similar to the intact Kenny Hill formation. Details of the grouting program, foundation installation and instrumentation program are described in Baker, et. al, 1998.

The foundation installation and instrumentation programs are also described in Baker, et. al, 1994.

<u>Performance Evaluation,</u> Predicted maximum settlement for the completed towers was 70-73 mm, (2.8 inches) with maximum differential across the mat of 11 mm (0.5 inches). Based on settlement measurements taken during construction, it appears that both measured total and differential settlements of the towers were less than predicted, indicating that the goals of the deep ground improvement program were met.

The time settlement record through completion of Tower 1 and partial occupancy up to March 19, 1997 is shown in Fig. 7. The maximum reported average settlement for the core is about 35 millimeters with maximum reported differential settlement of 7 millimeters. This is approximately ½ of that predicted settlement which was based upon an assumed modulus for the Kenny Hill formation of 250 MPa. As depicted in Fig. 5, the predicted settlement following the Menard rules and equivalent footing method is only slightly more than that experienced through 1997 (41 mm vs. 35 mm).

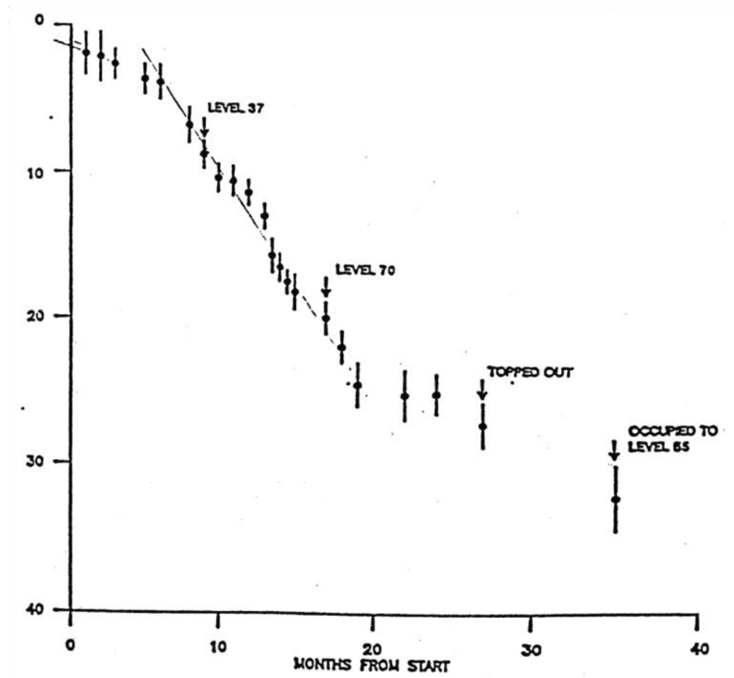


Fig. 7. Settlement of Petronas Tower No. 1 (from Baker, et. al., 1998)

It should be noted that part of the reported differential settlement is suspect since the major portion (about two-thirds) was reported immediately after pouring the concrete mat before significant additional load had been applied. Thus, the level of reading reliability may be only on the order of 2 to 3 millimeters.

From the less than anticipated differential settlement it appears as if the mat, barrettes and soil between the barrettes are acting as one massive block with the barrettes serving to knit the mass together.

In evaluating the foundation design and performance, the question needs to be asked as to why the settlement is only approximately one-half that predicted when extensive in-situ testing was performed including two full scale instrumented load tests and 260 in-situ pressuremeter tests.

In this connection it should be noted that correlation of prediction and performance would be improved if the prestressing effect of the barrette installation from the 4 meter level (with basement level at –20 meters) had been considered in making the prediction. Sixteen meters of soil excavation represents approximately 25% of the weight of the building. If this weight had been omitted, the predicted settlement would have been proportionately less.

Also, as a final observation, settlement predicted using the empirically determined Menard rules, as they were used by the authors in Chicago, and the simple equivalent footing method, comes very close to the observed settlement, particularly if allowance is made for some prestressing effect of the pre-excavation barrette installation.

<u>Burj Khalifa, Dubai, United Arab Emirates</u>

The second case history is Burj Khalifa, which is currently the world's tallest building at 163 stories. The senior writer was peer review consultant for the architect, Skidmore Owings and Merrill, and had the opportunity in that capacity to work with the geotechnical engineer of record, Hyder Consulting, a British engineering consulting firm. The geotechnical information is from Hyder 2003. The photographs are courtesy of the architect.

Figures 8 and 9 show the change in Dubai between 1990 and 2003.

Fig. 8. Dubai in 1990

Fig. 9. Dubai in 2003

<u>Subsurface Profile,</u> A comprehensive geotechnical investigation program was performed under the oversight of Hyder Consulting. A large number of both laboratory tests on soil samples and in-situ tests such as the in-situ pressuremeter test were performed. Based upon this investigation and testing program the profile shown in Fig. 10 was developed with the average drained modulus and average drained friction values indicated for each layer.

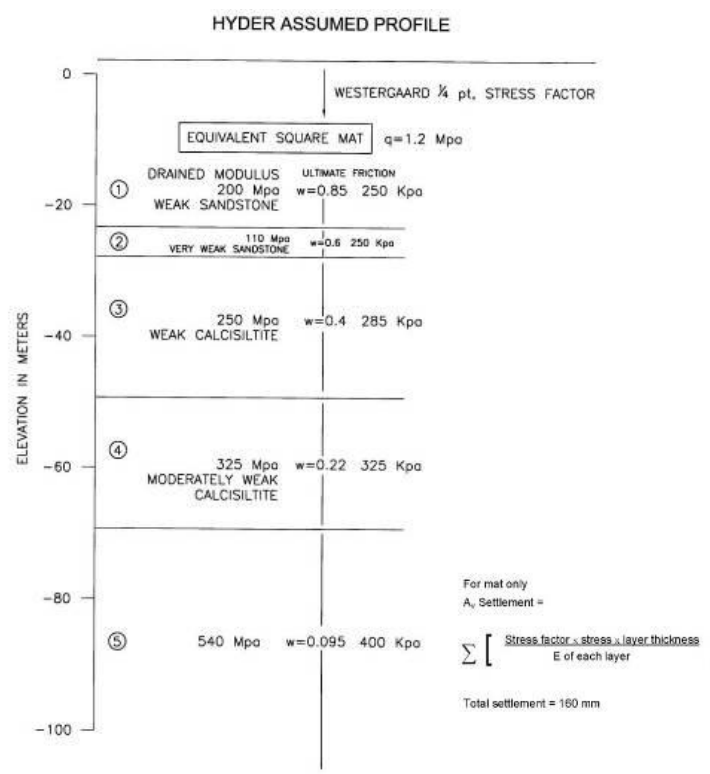


Fig. 10. Assumed Soil Profile at Burj Khalifa

Approximate Bearing Capacity and Settlement Analysis

Because the shape of the tower was such as to result in a smaller footprint the higher up the tower progressed, the total average building load was less than one might initially assume for a 163 story building. The foundation design concept for the structure was a mat on bored piles with the mat located at approximately -10 meter elevation with an average bearing pressure of 1.2 MPa. The supporting rock for the mat is classified as weak sandstone with a drained modulus of 200 MPa. The drained modulus values generally increase with depth and are assumed at 540 MPa below approximate elevation -70 meters. Since the typical unconfined compressive strength values in the weak sandstone are in the 1 to 2 MPa range, with many values much higher and only a few lower, and considering that the sandstone has a high friction angle, bearing capacity at a load of only 1.2 MPa should not be a concern. Thus, the primary question is one of settlement. In our role as peer review consultants, we performed a simplified settlement analysis early on utilizing the modulus values generated by the geotechnical engineer of record.

As noted in Fig. 11 and Fig. 12, we assumed a Westergard stress distribution because of the layered and cemented nature of the deposits and calculated the stress level at the center of each layer and summed up the total elastic compression at the quarter point of the mat. To simplify the geometry, we converted the three-winged mat into an equivalent square of 54 meters. The calculated settlement to a depth of twice the width of the foundation was 160 mm or more than 6 inches as shown in Fig. 11. Thus piles were required to reduce the settlement.

NO PILES

ASSUMPTIONS: FOUNDATION SYSTEM AND STRUCTURE SETTLE AS A RIGID UNIT

$$A_V \text{ SETTLEMENT} = \text{TO} \sum \left[\frac{\text{STRESS FACTOR} \times \text{STRESS} \times \text{THICKNESS OF EACH LAYER}}{\text{E OF EACH LAYER}} \right]$$

$$\text{TOTAL SETTLEMENT TO DEPTH} = 2B = 1.2\ M_{PA} \left[\frac{0.85x14m}{200} + \frac{0.6x4.5}{110} + \frac{0.4x21.5}{250} + \frac{0.22x20}{325} + \frac{0.095\ x38}{540} \right] =$$

$$= 0.160 \text{ m} = 160 \text{ mm}$$

(USING 1/4 POINT AS AVERAGE SETTLEMENT)

Fig. 11. Mat Settlement Analysis Without Piles

In fact, the normal procedure in Dubai for even moderate height buildings is to use a mat on piles rather than a mat only. To see what difference 45 meter piles would make in reducing the settlement, we performed another simplified analysis considering the mat on piles and the rock between the piles to act as a rigid block acting together under load with part of the load carried in perimeter shear around the perimeter of the block and the remaining load carried in bearing beneath the block.

This approach is a little different from the equivalent footing approach used in the simplified analysis for the Petronas Towers settlement in the first case history presented. To allow for creep effects, only two-thirds of the ultimate friction values were used in determining the load carried in perimeter shear. Assuming 45 meter long piles extending to elevation -55 meters resulted in approximately half the load being carried in perimeter shear and half in bearing. For calculating settlement from compression below the block, the equivalent footing area at the base of the block is then doubled to about 76 meters wide instead of 54 meters. This then significantly increases the 2B depth over which compression below is calculated. Figure 12 illustrates this calculation where the resulting compression in the 2B width below the block is 52 mm.

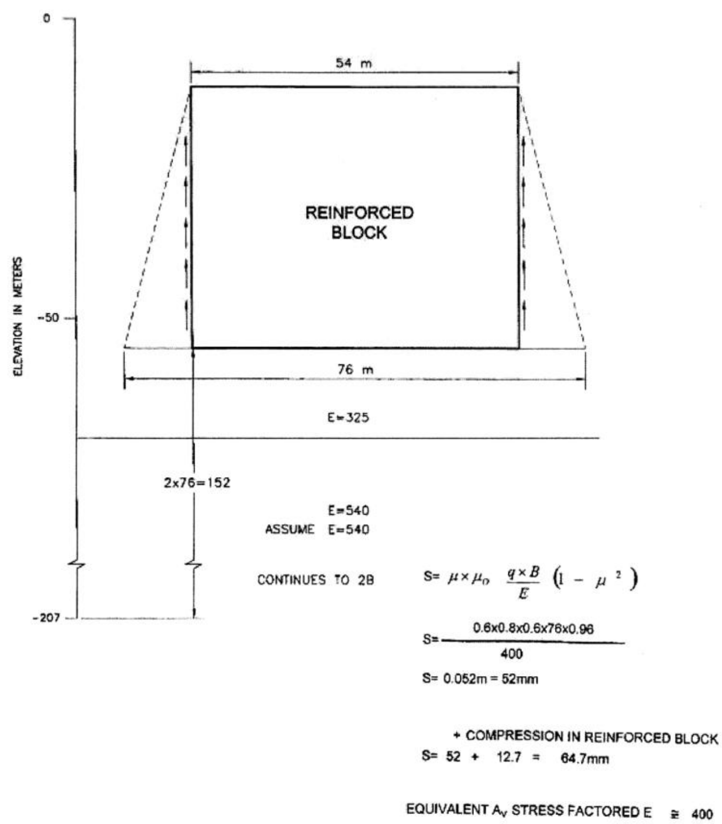


Fig. 12. Settlement Analysis of Mat on Piles

To this must be added the compression in the reinforced block which calculates out to be 12.7 mm based on the average stress and average modulus values in the reinforced block. Thus, the total predicted settlement for the mat on 45 meter long piles is approximately 65 mm, down from the 160 mm calculated without piles. These calculations are intended to be illustrative and approximate only, since the actual modulus properties of the deposits are strain dependent. The modulus values selected by Hyder and used here were based on locally empirically determined correlations using a relationship where Young's modulus equals 0.2 times the reload modulus determined in the pressuremeter test.

We note that this is a very low relationship compared to values determined elsewhere where Young's modulus is often taken as equal to the reload modulus (or sometimes even greater) such as at Petronas Towers where it was taken as equal. The explanation may be the relatively low density and high porosity of the weakly cemented sandstones and siltstones in Dubai.

Settlement Prediction by Finite Element Analysis. The geotechnical engineer predicted settlements using a finite element program as shown in Fig. 13A and Fig. 13B with structural and foundation plan as noted in Fig. 14. The maximum values predicted are only slightly greater than the average value predicted using the simplified rigid block analysis.

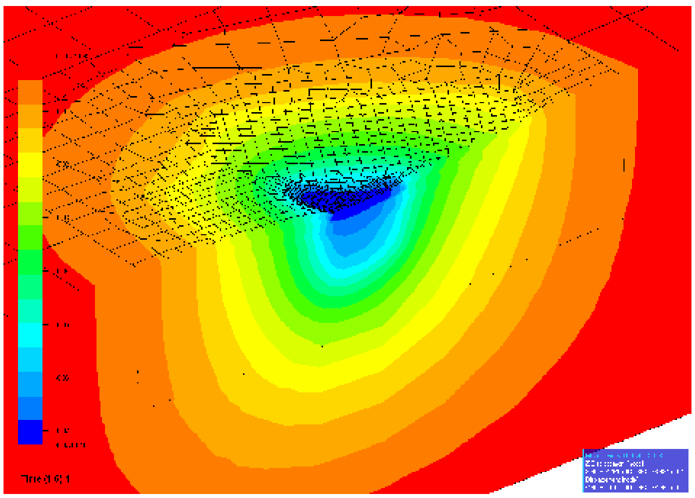


Fig. 13A. Predicted Vertical Displacement of Burj Khalifa through Wing A

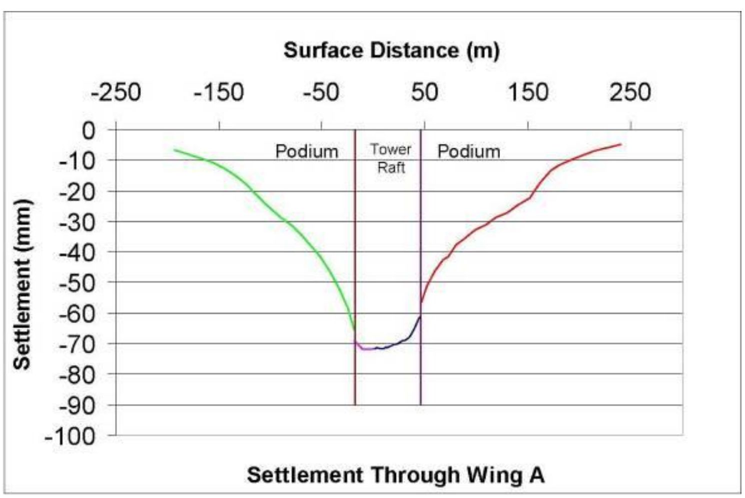


Fig 13B. Predicted Vertical Displacement at Tower Mat Foundation Level-Cross Section through Wing A and Podium

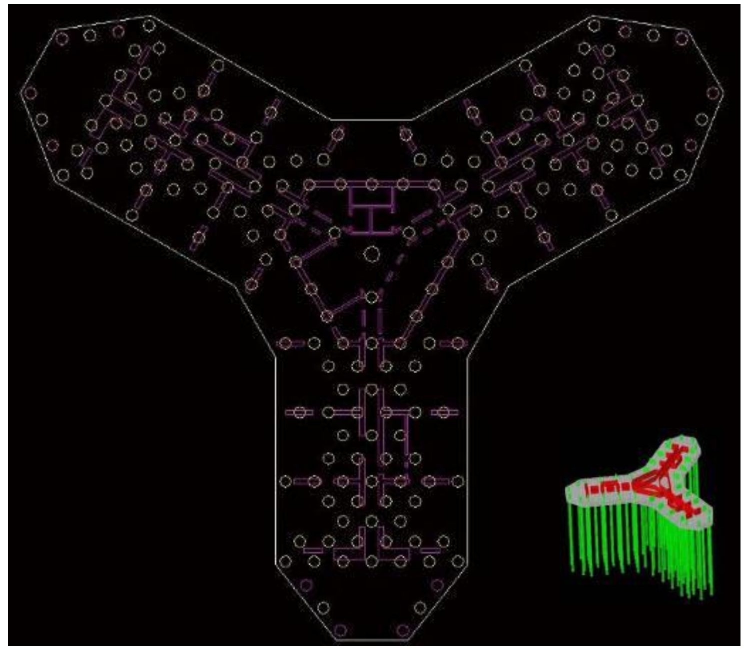


Fig. 14. Mat and Pile Foundation Plan

Burj Khalifa Construction. The following construction photo taken in March 2007 shows construction was approximately up to the 110th floor with approximately 70 percent of the dead load in place but with the façade still to come.

Fig. 15. Burj Khalifa in March 2007

Observed Settlements. Settlement during construction was monitored, and the observed settlement of Wing C in March 2007 is shown on Fig. 16. Observed settlement was in the range of 20 to 30 mm. We understand that settlement as of 2012 is about 45 mm – 50 mm with construction complete and the building occupied. This actual settlement has been about 15 percent below the prediction.

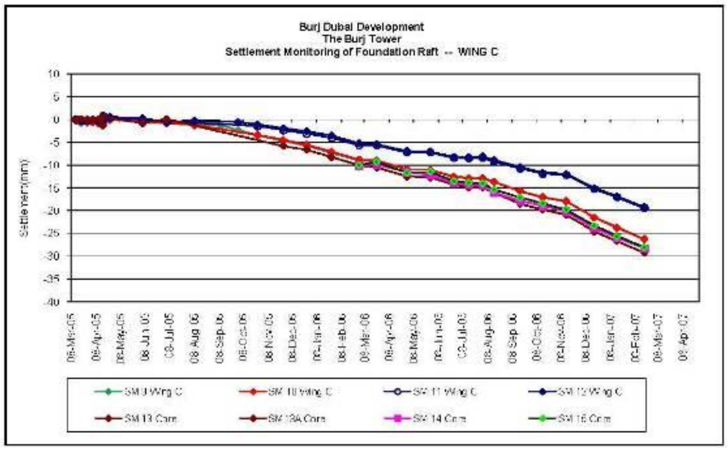


Fig. 16. Settlement Monitoring of the Burj Khalifa

One obvious conclusion that can be reached from the two case histories presented is that there is no universally accepted procedure for determining the correct input parameters for settlement analysis and that the locally determined procedures appear to be conservative, i.e., observed settlement is less than computed and predicted.

CHICAGO EXPERIENCE IN MAXIMIZING ALLOWABLE SOIL BEARING PRESSURES FOR HIGH RISE FOUNDATION DESIGN

The typical downtown Chicago soil profile is shown in Fig. 17 with the typical potential foundation types indicated on the profile.

Prior to 1969, foundation design bearing pressures were typically based upon unconfined compression tests performed on samples obtained either by 2 inch (50.8 mm) or 3 inch (76.2 mm) Shelby tubes and 2 inch (50.8 mm) OD split barrel samples obtained following ASTM specifications D 1587 and D 1586, respectively. The value was often increased by 1.25 based on "Terzaghi & Peck" (1948) for foundations on cohesive soil but with little confinement. The maximum allowable bearing pressure on good Chicago hardpan had increased gradually from 12 kips per square foot (ksf) (574.6 kPa) (the typical design value prior to the Depression and World War II) to a maximum of 30 ksf (1436 kPa) at the 65 story Lake Point Tower project built in 1965. Based upon the Skempton theory (1951) that the ultimate tip capacity for a deep foundation in clay (depth ≥ four times the bearing width) was nine (9) times the cohesion requiring a cohesion of 10 ksf (479 kPa) for a factor of safety of 3. The 30 ksf value was required if the bearing area was based on the largest caisson bell diameter that could be constructed with available equipment. Since unconfined compression tests sometimes failed to yield the necessary 20 ksf (958 kPa) unconfined compressive strength due to silt sand and gravel content in the hardpan, triaxial compression tests were necessary to confirm the design bearing pressure. While triaxial testing could be performed to demonstrate significant friction angles in the hardpan, theoretical bearing capacities at great depths became unrealistically high. In addition, the prediction of settlement appeared even less reliable.

The in-situ pressuremeter test offered distinct advantages in that it avoided the potential sample disturbance inherent in sampling and testing in the laboratory. It was seen as analogous to an in-the-ground load test, and in a very short time frame it was well correlated with building performance. Allowable bearing pressures on good hardpan increased from 30 ksf (1436 kPa) in the early seventies to 50 ksf (2390 kPa) in the late eighties.

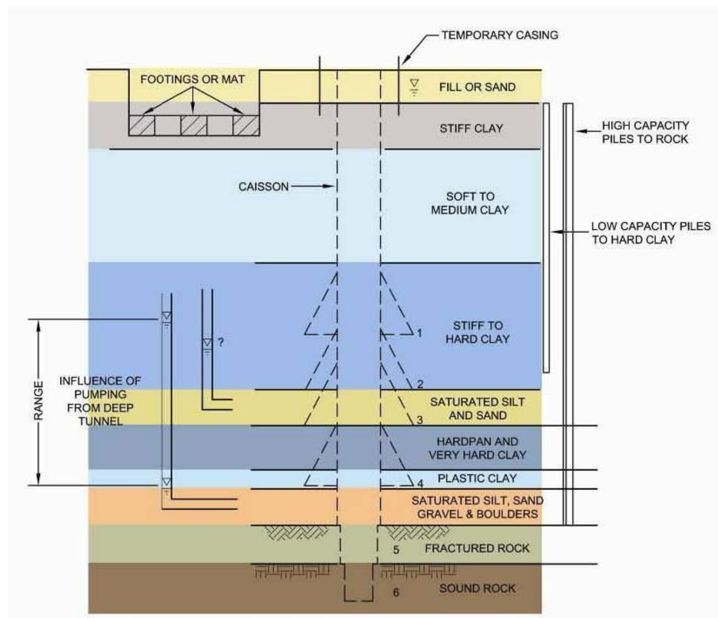


Fig. 17. Typical Soil Profile of Downtown Chicago

Determination of Pre-Consolidation Pressure

Early research by Lukas, et. al, 1976, indicated that the creep pressure determined during the performance of the in-situ pressuremeter test compared favorably to the preconsolidation pressure determined from well run consolidation tests. One of the difficulties of determining preconsolidation pressure from consolidation tests in glacial till is the difficulty of testing a sufficiently undisturbed sample to provide a sharp break on the void ratio versus pressure curve, thereby leaving considerable room for interpretation. The creep pressure from the pressuremeter tests appeared to be simpler and more reliably determined with consistency.

Settlement Theories Using Pressuremeter Test Data

The two most common approaches for predicting settlement using pressuremeter data in our experience are the Menard semi-empirical procedures described by Menard (1975) and Briaud (1992), and the elastic theory in which the pressuremeter is utilized to determine an equivalent Young's modulus. The question here is how best to determine the effective Young's modulus. Since the modulus undoubtedly varies somewhat with the stress and strain level (as well as Poisson's ratio), a theoretically correct approach would involve special tests at the stress/strain level anticipated in each soil strata below the bearing level. Details of both procedures are given in the references.

In both settlement prediction theories, it is assumed that the stress level is within the pseudo elastic range which in pressuremeter terminology means the total stresses must be below the creep pressure.

Caisson Load Tests and Correlations With Prediction From Pressuremeter Test Results

Performance of limited historic caisson load tests in Chicago compared with what might have been predicted using pressuremeter tests is presented in Baker and Pfingsten, 1998 with a tabular summary shown in Table 2 below.

Table 2. Correlation Between Full Scale Caisson Load Tests and Pressuremeter Tests in Chicago Hardpan

						Average Pressure-meter Modulus			Ultimate Capacity On:	
Test Location	Caisson Diameter (ft)	Caisson Elevation	Maximum Test Load Bearing Pressure (tsf)	Observed* Settlement of Base (in.)	Observed* Settlement @ ½ Max. Bearing Pressure (in.)	Ea (tsf)	Eb (tsf)	Pressure-meter** Settlement at ½ Max. Load Bearing Pressure (in.)	Pressure-meter (tsf)	9 x C (tsf)
Union Station 1	8.2	-60.0±	18.4	0.75	0.3	335	335	0.33		
Union Station 2	4.2	-60.0±	61.0	2.0	0.9	335	335	0.88	85.0	36
One Park Place	6.3	-67.4±	24.0	1.4	0.4	247	320	0.55	54.4	27
Univ. of Chicago	2.5	-38.0±	50.0	2.2	0.45	460	460	0.41	48.6	52

Conversion Key: 1 Ton Per Square Foot (tsf) = 95.8 kilopascals (kPa)
1 inch (IN) = 25.4 Millimeters (mm)

* First Load Only
** Based on Menard Rules and using α= 0.5

From this we can conclude that the settlement magnitude under a given load within the normal working load range can be reliably predicted on highly preconsolidated glacial till (Chicago hardpan) using appropriate in-situ pressuremeter test results and current pressuremeter theory.

Correlation With Building Performance. In the early use of the pressuremeter much confidence was gained when predicted settlement of the then tallest reinforced concrete building in the world (75 story Water Tower Place) matched closely the measured settlement after construction (2.0 inches vs. average of 1.94 inches with a range of 1.69-2.19 inches).

INCREASING ALLOWABLE BEARING PRESSURE ON CHICAGO DOLOMITE

The Chicago code allows for a design end bearing pressure one foot into sound dolomite of 100 tsf and additional 20 tsf for each foot of additional penetration up to maximum of 200 tsf with no specific allowance for socket friction. In recent years we have managed to increase this allowable maximum up to 300 tsf by performing an Osterberg load cell test in which values for both end bearing and socket friction can be obtained by locating a load cell in the shaft at a location where it is calculated that the down pressure is balanced by the perimeter friction or rock socket bond resistance (Fig. 18).

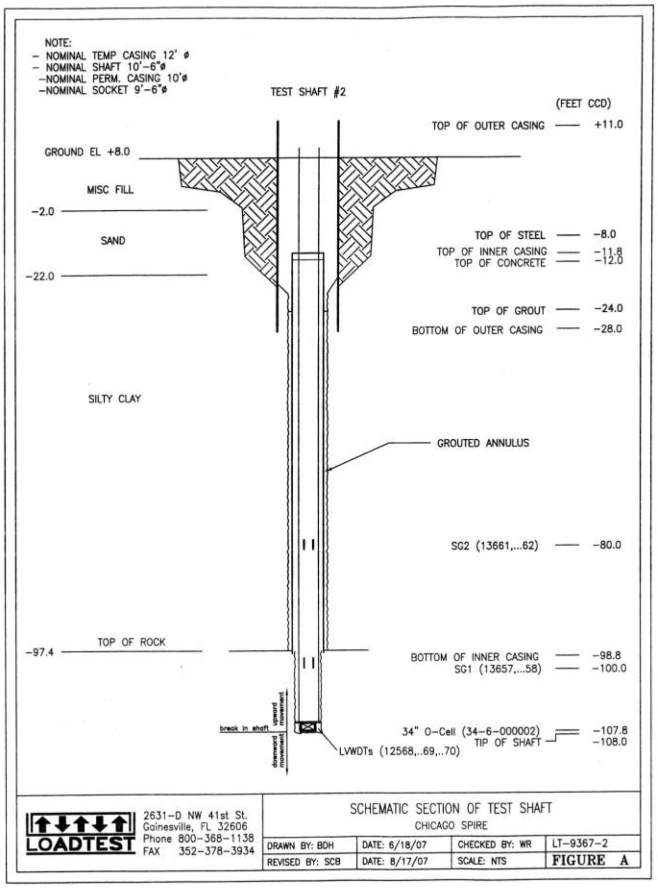


Fig. 18. Load Test Configuration at the Chicago Spire

Figures 19 and 20 illustrate the results from one of the load tests performed at the Chicago Spire which when finished (if ever built) is planned to be 2,000 feet tall. It is evident that even at pressures exceeding 600 tsf, the rock behavior is still almost linear elastic indicating that, at least in theory, much higher bearing pressures could be developed without failures.

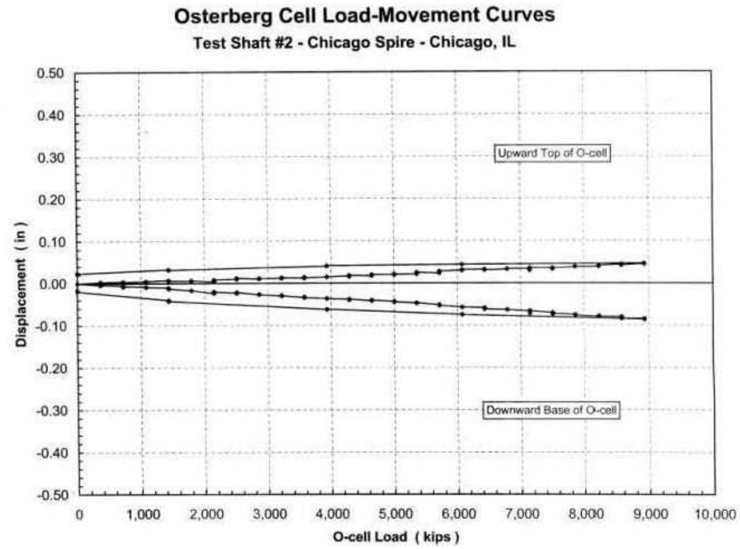


Fig. 19. Results of Rock Socket Load Test

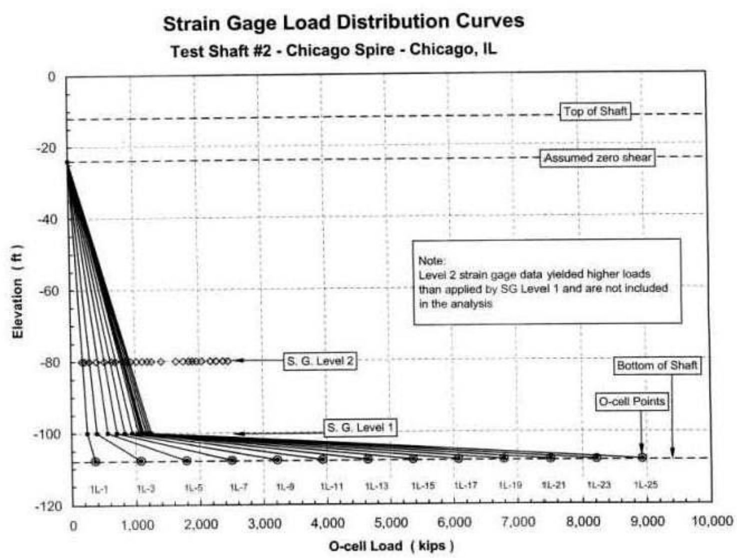


Fig. 20. Load Distribution in Test Shaft

It should be pointed out that the bearing pressure is being applied to only a portion of the caisson bearing area but is still believed to be sufficient and conservative to test the modulus and bearing capacity of the rock since the boring data indicates that rock gets better with depth. It is important to note that on high rise structures in Chicago supported on rock, measured settlement hardly exceeds the elastic compression in the caisson shafts further supporting the view that higher allowable bearing values are possible subject to maximum allowable stresses in the concrete.

In recent years 65 story plus buildings have been supported on the fractured bedrock surface to save the costs of socketing into sound dolomite and providing permanent steel casing. These caissons are constructed using polymer slurry and tremie concreting methods and have been designed with allowable bearing pressures in the range of 75 to 90 tsf based on in-situ testing using the Goodman Jack or high capacity pressuremeter, confirmed by Osterberg load tests. The One Museum Park project is the first high rise in the city to be supported on 90 tsf, top of rock caissons (Fig. 21).

Fig. 21. One Museum Park

Conclusion

Utilizing the in-situ pressuremeter test and the Menard empirical settlement calculations and bearing capacity analysis procedures, and by observing building performance over time, we have been successful in increasing allowable bearing pressures on good Chicago hardpan (very dense glacial till) from 12 ksf to 50 ksf on major Chicago high rises and with reliably predicted settlement. Utilizing the Osterberg load cell test we have been able to increase maximum allowable dolomite rock bearing pressures from 200 tsf to 300 tsf.

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