ANALYSIS, DESIGN, TESTING AND PERFORMANCE OF FOUNDATIONS

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General Report – Session 1

INTRODUCTION

The planning, analysis, design, testing and retrofitting of foundations are a significant part of geotechnical engineering practice. This is reflected in the number of papers included in this session. Foundation analysis and design form the core of foundation engineering. Proper design requires appropriate site characterization. Sometimes, poor subsurface conditions at a site require ground improvement before foundations can be laid. Field-scale static load tests are sometimes performed to aid in or verify design. Not as reliable, but more economical, dynamic load tests are more often done with the same goals. In the event foundations fail these tests or, worse, later fail to perform adequately, corrective interventions or foundation retrofitting are needed. The papers of this session address all of these issues, and are organized in the present report along those lines.

Proper analysis and design require a reasonable basis on mechanics but should also be corroborated by the satisfactory performance of foundations designed using these analyses over an extended period of time. Well documented case histories help us in such corroboration. Experiments also allow us to compare predicted and measured foundation response; however, experiments are often restricted to model tests or to single-element tests (as in the load test of a pile). On the other hand, case histories sufficiently rich in details allow comparisons with simulations of entire foundation-structure systems, adding a measure of realism to the validation of analyses and design methods. Some of the papers included in this session presented exclusively case histories while others dealt with one or more aspects of foundation engineering that complemented the exposition of case histories.

Not every case history needs to be a complete account of successful or unsuccessful design and construction of a structure. Several papers dealt with case histories that we could consider unfinished, as in the case of structures that are apparently planned or under construction, and thus, much of the focus is placed on analyses and exploration of the likely response of the structure to design demands. The structures dealt with in these case histories include bridges, buildings and more unusual ones, such as liquid storage tanks. Most papers addressed pile foundations and the structures they support and their response under static or dynamic loads.

ANALYSIS AND DESIGN

In this section, we discuss those papers that deal with analyses (both static and seismic) and designs related to foundations. In the distant past, construction was done with virtually no analysis because the principles of mechanics and their application to structures were not in place. Structural analysis progressed quickly with the advent of Bessemer's steel manufacturing process and of reinforced concrete, which made the use of frames possible. These, in turn, made possible the construction of high-rises, which placed considerably more demand on foundations. Until recently, geotechnical analysis and design, with large emphasis on empiricism, lagged the progress that took place regarding analysis and design of the superstructure. However, advances in geotechnical analysis and in constitutive modeling of geotechnical materials have evolved into realistic representations of the various design problems that we face today. This continuous progress was facilitated by codes of practice that are not overly restrictive, but, by and large, the advances have not yet found their way into practice to the extent that they should.

In foundation design, we seek to design and construct foundations that will not reach limit states. The more commonly addressed ones, which by no means represent the totality of limit states that should be avoided in typical designs, are limit states related to excessive deflection — in which case, the foundation is essentially functional and in place but moves more than what the architectural finishing of a building or its superstructure or the superstructure's interface with its surroundings can tolerate — and limit states specific to the foundation element, such as plunging or crushing/rupture of the foundation element. In order to design against these limit states, we need analyses that allow us to estimate the limit states. Some of these analyses aim to determine the amount of movement of foundations and the internal stresses in the foundations (more commonly footings, piles, mats and piled mats); the results of these analyses are then used to verify if the related limit states are reached. Other analyses, particularly applicable to footings and piles, aim to determine the loads that would lead to collapse or plunging (extremely large displacements) of the foundations. These loads would be compared with working loads to make sure that adequate safety exists with respect to the foundations. This type of limit state is sometimes called "geotechnical failure" in the literature. The bearing capacity equation is an example of an analysis aiming to produce this type of result. The loads transmitted by the superstructure to the foundations are traditionally calculated by structural engineers and used by geotechnical engineers in their analyses.

Analyses of foundation deformation have, until recently, been elastic in nature. Such analyses are fundamentally inadequate, as the stress-strain response of soils is nonlinear from very small strains. The use of linear elastic analyses has always required engineers' best estimate of an overall representative elastic pair (such as Young's modulus and Poisson's ratio), an estimate that is very difficult to make even when the boundary conditions are simple. In contrast, traditional collapse analysis has relied on perfectly plastic response, according to which the soil would have a single value of strength (a single value of friction angle or cohesive intercept). These are also inadequate, for we know that soils can soften or harden upon shearing, and have a much more complex response than perfect plasticity. The major thrust in recent years has been to use more realistic models for soil mechanical behavior, which, in turn, requires more sophisticated analytical/numerical frameworks.

Another major thrust in recent research, which is gradually seeping into practice and is represented by some of the papers reviewed in this report, is not to produce foundation loads but rather to analyze the foundations and structure together. In fact, the superstructure, foundation and soil are components of a single system, with interfaces enabling the load transfer between the three components. One would expect that, over time, material modeling and numerical modeling techniques would develop sufficiently so that this type of analysis became the norm, rather than the exception, in how the foundations and the superstructure are designed.

The analyses described in the papers range from sophisticated, three-dimensional (3D) finite element (FE) analysis to simple analytical models. Often the traditional, simplified analytical tools (used in routine design) fall short of the design requirements of modern-day foundations, as indicated by Poulos and Bunce (2008) in their design of the world's tallest tower (the Burj Dubai Tower) and by Venkatesh et al. (2008) in their design of a barrage raft. Nevertheless, simple models often provide us with important insights into overall foundation response and help us perform quick, supplementary calculations (as illustrated by Barvashov et al. 2008).

A large number of papers included in this session deal with the design of pile foundations. Piles are routinely used as foundations for tall and heavy structures like high-rise buildings and bridges, as evidenced by the cases described by Poulos and Bunce (2008) and Yang et al. (2008). For important structures, extensive analysis is generally performed using input data estimated from laboratory and field-scale tests. Different design criteria and methods of analysis and design are used for piles depending on the soil type, loading conditions, function and importance of the structure, and code prescriptions.

Soil-structure (foundation-ground) interaction (SSI) analysis is an important tool used to obtain proper response of complex foundation problems. The foundation design for the Burj Dubai Tower (Figure 1), the world's tallest building, is a perfect example of the important role that SSI analysis can play in foundation design (Poulos and Bunce 2008). It is a comprehensive project on pile design that involved comprehensive site investigation, rigorous numerical analysis and pile load tests.

Figure 1. The Burj Dubai, world's tallest building.

Standard penetration tests (SPT), cone penetration tests (CPT), pressuremeter tests, cross-hole and tomography geophysical surveys and piezometer measurements were performed at the Burj Dubai site in addition to an array of laboratory tests that included routine tests as well as sophisticated triaxial, direct shear and resonant column tests. The ground conditions consist of a complex and highly variable horizontally stratified subsurface profile with medium dense to loose silty sand extending down to 2.2 m from the ground surface underlain by successions of very weak to weak sandstone interbedded with very weakly cemented sand, gypsiferous fine grained sandstone/siltstone, and weak to moderately weak conglomerate/calcisiltite. The groundwater levels are generally high at approximately 2.5 m below the ground surface.

The FE software ABAQUS was used to analyze the foundation response of the 160-story Burj Dubai Tower and Podium. The foundation consisted of a 3.7-m-thick raft supported on bored piles. The tower piles were 1.5 m in diameter and 47.45 m in length. The podium piles were 0.9 m in diameter and 30 m in length. The pile bases were socketed into weak rock; the shaft friction was assumed to be the main source of pile capacity. Altogether eight load cases were considered in the analysis, which included four wind-load cases and three seismic-load cases. The FE mesh consisted of a relatively fine mesh covering an area of 500 m $\times$ 500 m with 90 m depth and coarser far-field mesh covering an area of 1500 m $\times$ 1500 m with 300 m depth. The soil was modeled as a plastic material with a nonlinear-elastic stress-strain relationship within the yield surface (the amount of degradation of modulus was assessed using the constant normal stiffness and cyclic triaxial tests). The piles beneath the tower were modeled with beam elements connected to the soil strata by pile-soil interaction elements. The podium piles were also modeled as beam elements but were fully bonded to the soil strata. The loads from podium and tower were applied at concentrated points while the submerged weight of the raft was applied as a uniformly distributed load. The shear from the tower (due to wind) was applied as body forces to the tower raft elements. The superstructure shear walls were modeled as a series of beam elements; the moment of inertia was modified to account for the stiffening effect of the tower.

The ABAQUS analysis was validated by other analyses; a non-linear analysis was carried out using VDISP while linear analyses with small-strain modulus were performed using PIGLET and REPUTE (initial analysis using ABAQUS showed that the soil strains were within the small-strain range). Settlements obtained from ABAQUS and VDISP were modified to incorporate the effect of rigidity of pile cap so that comparisons with REPUTE and PIGLET could be performed. Supplementary S-Frame analysis, with soil modeled as linear springs connected to the raft and piles, was also performed. Load distributions in the piles were obtained using the above analyses. The FE analysis and the analyses by REPUTE and PIGLET indicated that the largest pile loads (of the order of 35 MN) were concentrated near the edges of the piled raft with minimum loads (of the order of 12-13 MN) occurring near the center, while the S-Frame analysis indicated the opposite. In reality, the authors expected the pile load distribution to be somewhere in between the two extremes.

Independent verification analyses were performed in addition to the above design analyses using data available from nearby Emirates projects. The commercially available finite difference (FD) program FLAC was used to perform axisymmetric analysis of the foundation system of the tower; the foundation plan was represented by a circle of equal area and the piles were represented by a solid block containing piles and soil. The software PIGS was also used to check the settlements obtained from FLAC. The results of the verification analyses matched reasonably well those of the design analyses.

Apart from the verification analysis, cyclic loading effects were studied using the computer program SCARP, which was supplemented by cyclic laboratory tests that included triaxial, direct shear and constant-normal-stiffness tests. It was found that a loss of capacity would be experienced when a cyclic load exceeded ± 10 MN. The maximum loss of capacity (due to the degradation of shaft friction) was of the order of 10-15%.

An overall stability check was also performed. A factor of safety of just less than 2 was assessed for vertical block movement excluding the base resistance of the block. The factor of safety against lateral block movement excluding passive resistance was greater than 2. The overturning factor of safety was approximately 5.

The liquefaction verification was done using the Japanese Road Association method and the method proposed by Seed et al. (1984). The analysis indicated that the marine deposits and sand down to 3.5 m below ground surface may potentially liquefy although the water table is at a depth of 2.5 m below the ground surface. Intermittent layers within the sandstone layer between –9.8 m and –14.25 m may also liquefy. However, it was found that liquefaction would have minimal effect on the foundations and that no reduction of soil strength parameters was required.

The extensive analysis was supplemented with pile load tests. Static load tests were done on seven trial piles and eight work piles during the construction stage. Dynamic load tests were also performed on 5% of the work piles. The maximum settlement observed under working load conditions from the static pile load tests was approximately 0.5% of the pile diameter. The factor of safety against geotechnical failure was found to be in excess of 3.

The importance of analysis is again highlighted in the paper by Yang et al. (2008), in which they described the seismic design of pile foundations for three bridges, namely the Golden Ears Bridge (GEB), the Canada Line Fraser River North Arm Transit Bridge (CFB) and the Roger Pierlet Bridge West (RPB). For seismic SSI, the ground response (acceleration) data is generally obtained from site response analysis. The acceleration data is then used, along with other inputs, to obtain the foundation response.

The GEB is a 5-span continuous 968-m long hybrid extradosed cable-stay bridge. It comprises three equal main spans that are 242 m long with two side spans 121 m long. Three types of sediments were present at the bridge site: Fraser river sediments (over-bank silty to silt clay loam overlying sandy to silt loam), Sumans drift sediments (described as raised proglacial deltaic gravel and sand) and Capilano sediments (consisting of marine and glaciomarine stony to stoneless silt loam to clay loam with minor sand and silt). Organic soils, including peat, are also present. The subsurface profile at the main river crossing consists of loose to medium dense sands down to 35 m on the south bank and typically down to 20 m within the river channel resting on normally- to lightly-overconsolidated clays and silts extending to large depths. The thickness of the near surface sands decrease towards the navigation channel and the north side of the river. The four main tower foundations consist of 12 drilled shafts divided into two subgroups with each subgroup supporting each tower leg (Figure 2). The drilled shafts were cased with a diameter of 2.5 m in the upper sand portion; the lower parts of the shafts were uncased with a diameter of 2.4 m extending to a maximum depth of 90 m. Ground densification was performed at the site using vibro-flotation and vibro-replacement to reduce the potential for liquefaction.

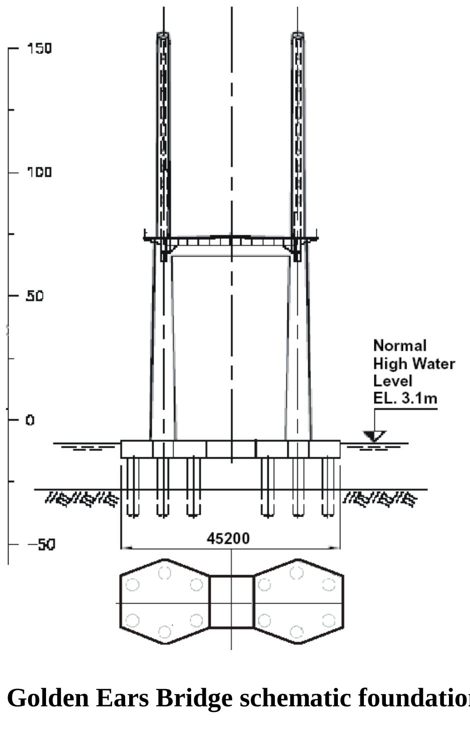


Figure 2. Golden Ears Bridge schematic foundation layout.

The CFB is an extradosed precast concrete segmental box girder bridge, 562 m long with a 180-m long extradosed main span, 139-m long side spans and 52-m long transition spans (

Figure 3). The subsurface consists of loose to dense silty sand and clayey silt over hard or very dense glacial till-like soils. Open-ended steel pipe piles with 2 m diameter were used for the main piers while pipe piles with 0.914 m diameter were used for the remaining piers. The piles were driven into the till to a depth of about 10 m. Some of the piles were battered to better resist horizontal impact.

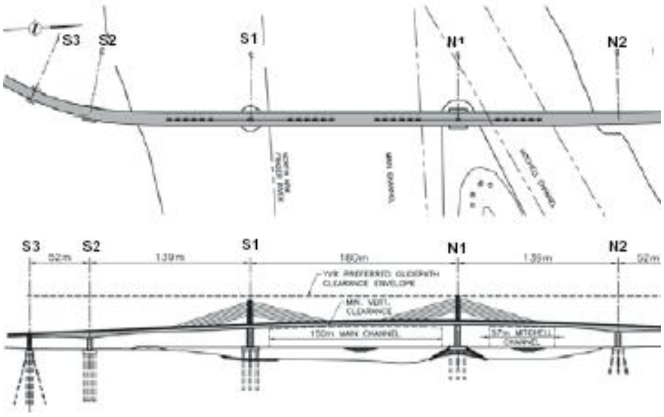


Figure 3. Plan and elevation of Canada Line North Arm Bridge.

The RPB is a five-span, 103-m long structure with three 22-m center spans and two 18.5-m side spans (Figure 4). The subsurface at the site consists of normally consolidated sensitive clays of 40-50 m depth underlain by Pleistocene glacial till comprising of very stiff silt and clay or dense to very dense sand with occasional gravel and cobbles. The foundations consist of four open ended steel pipe piles with 0.61 m diameter at each pier driven to a minimum embedment of 2.4 m into the till.

The project seismic design criteria and performance criteria required three design earthquakes with 475-year return period, 1000-year return period and 4750-year return period for the GEB. Elastic performance and immediate access, repairable damage and limited access and no collapse with possible loss of service were the performance criteria set against the above seismic events, respectively. For the CFB, a 475-year event with repairable damage and a 100-year event with no significant damage and elastic performance were used as the design criteria. A 475-year event with no collapse and with peak firm-ground acceleration of 0.24g and a peak firm-ground horizontal velocity of 0.22 m/sec were prescribed for the RPB.

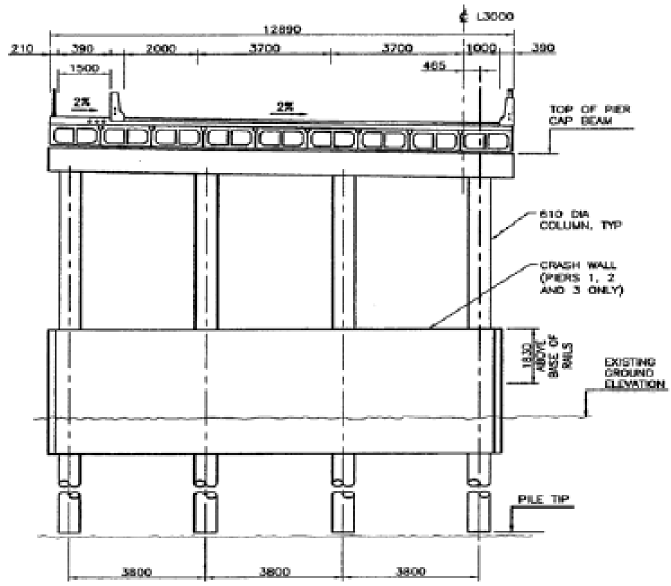


Figure 4. Roger Pierlet Bridge typical pier foundation.

Site specific response analysis was carried out at the GEB site with an assumed 150 m depth as the top of elastic half space to determine the seismic design input motions. The nonlinear site response analysis was done using FLAC 3D with hysteretic constitutive model UBCHYST for the 2475-year design earthquake. For the 475- and 1000-year design earthquakes, FLAC (nonlinear) and SHAKE91 (equivalent linear) were used for analysis. It was observed that ground motion amplified for periods greater than 0.6 seconds and de-amplified for periods less than 0.3 seconds for all the three design earthquakes. The site response analysis for the CFB site was carried out using the equivalent linear SHAKE91 analysis. The seismic motions were amplified for periods less than 0.6 seconds on the north side to as much as twice the values on the south side of the bridge. For longer periods, the amplification was larger on the south side. For the RPB site, no site response analysis was performed; acceleration values for 475-year design event were obtained from the Canadian bridge design code. For the GEB, CFB and RPB sites, the dominant longitudinal and transverse modes were > 5 seconds and > 2.5 seconds, ~1.8 seconds and ~1.3 seconds, and > 1 second and > 0.5 seconds, respectively.

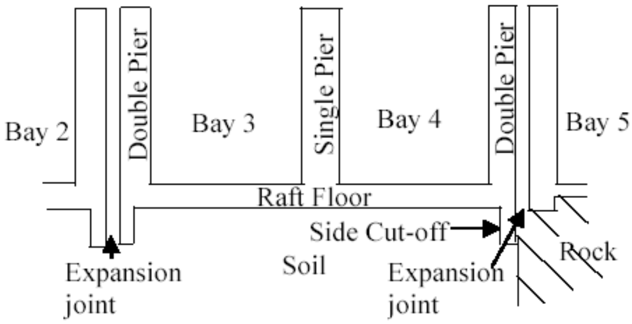
The seismic loads on the piles originate partly from the motion of the superstructure and partly from the differential lateral ground displacement during shaking. The inertial forces due to the superstructure act at the pile head while the kinematic ground-movement force acts along the pile shaft. Often, the loss of soil support due to liquefaction has to be taken into account. Yang et al. (2008) considered all these factors in the SSI. They reported analysis for three particular cases: piles in a group with multiple rows, piles in a single row and single piles.

In order to obtain the pile response against inertial forces, ground stiffness was calculated by obtaining the nonlinear load-displacement response, which includes the effect of the pile cap. The soil stiffness affects the inertial forces calculated from the global bridge dynamic seismic analysis, and often iterations are necessary to reach a final solution. Soil stiffness was calculated for both pre- and post-liquefaction soil conditions (stiffness after liquefaction remaining the object of present-day research) in the case of sands and also for non-liquefied soil if adequate ground densification is done. The software GROUP 3D V6.0 was used to calculate the load displacement and moment-rotation curves for the GEB pile groups. An equivalent linear soil spring approach (with iterations to determine the correct equivalent linear modulus) was adopted to perform modal spectrum analysis of the pile groups. For the CFB pile groups, a similar approach was adopted. For the piles in a single row below RPB, the bridge structural seismic analysis modeled the bridge superstructure, substructure and the piles (using p-y curves). The software LPILE was used to develop the p-y curves. The modal spectral analysis was subsequently performed following an iterative procedure. For the analysis of the single piles (drilled shafts) of the GEB, each drilled shaft was modeled by two parallel vertical fictitious members directly below the column in the 3D global bridge model so that the effect of coupling between horizontal deflection and rotation at the head could be captured. An iterative technique was adopted so that the forces obtained from the 3D global bridge model matched the inputs of LPILE that produced the nonlinear pile deflection and rotation.

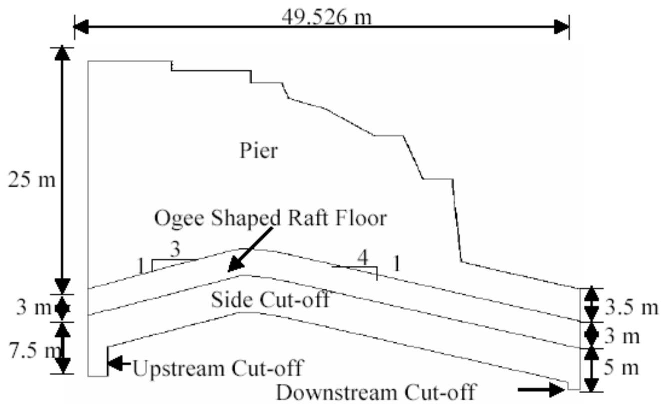
The kinematic interaction force analysis for GEB was performed using the FLAC 2D time history analysis, which uses the effective stress soil constitutive model UBCSAND to simulate soil liquefaction. The FLAC-analysis results indicated that, under the 2475-year event, the kinematic interaction forces between the liquefied soils and the piles occurred at the time the inertial forces were near their peak. Consequently, both effects were combined. For the CFB, piles adding the effects of inertial and kinematic forces were not necessary; the free field ground movement was assessed using an empirical lateral spread procedure and the SSI was performed using LATPILE. For the RPB, the FLAC dynamic analysis was performed to capture the effect of the kinematic forces.

The design of the ogee-shaped barrage raft floor (Figure 5) is another example in which involved numerical analysis may become necessary. Venkatesh et al. (2008) compared the analysis of a typical ogee-shaped raft floor (including piers of cut-off bays 3 and 4) with a simplified method proposed by Hetényi (1964), as recommended by the Indian code of practice, which assumes the soil as independent linear springs. The plan area of the raft is approximately 49.5 m × 30.5 m, and the height of the 3D structure (which includes the raft and the piers) varies from 35.5 m to 11.5 m. Such a complicated shape can be efficiently analyzed by 3D FE analysis, as was done by Venkatesh et al. (2008) using eight-noded, isoparametric brick elements with three translational degrees of freedom at each node for the cut-off pier, abutment wall and beam, and the underlying soil and rock. The barrage raft floor was modeled with four-noded plate bending elements with six degrees of freedom per node. The analysis was based on linear elasticity with the FE mesh extending to 50 m in both upstream and downstream sides, to up to 35 m on both sides of the raft and to 80 m in the vertical direction from the base of the raft (Figure 6). Venkatesh et al. (2008) found that deformations predicted by Hetényi's subgrade reaction method can be substantially lower than those obtained by FE analysis and that the bending moment from Hetényi's method was considerably different in terms of both magnitude and nature of variation.

Although interaction between the foundation and the ground were taken into account in the above papers, the effect of the flexibility (or stiffness) of the structure was not explicitly accounted for. Hora and Sharma (2008) presented an integrated soil-structure interaction analysis of a plane-frame structure by considering both the frame and the soil mass below in their 2D FE analysis. They modeled the soil as a nonlinear elastic material following a hyperbolic modulus-degradation law. The soil mass was discretized by eight-noded plane strain elements with two degrees of freedom per node; the finite elements were surrounded by six-noded infinite elements to capture the effects of the unbounded soil medium. The floor beams, columns and the foundation beam were assumed to be linear elastic, and were modeled using beam elements with additional degrees of freedom accounting for axial compression (or tension). Hora and Sharma (2008) compared their analysis with the traditional "non-interaction" analysis in which the frame columns were assumed to be fixed at the ground surface and with linear interaction analysis in which the soil was assumed to behave as a linear elastic material. They performed a parametric study and observed that the number of stories and the number of bays in a frame affect the deformation and stresses in the soil and that accounting for soil nonlinearity is important in the interaction analysis. Hora (2008) analyzed similar problems in a companion paper, in which the author explicitly considered the yielding of soil (in addition to hyperbolic modulus degradation) by using different plastic yield criteria in his FE analysis. He observed that the forces in the individual members of the frames obtained from the analysis were significantly different from those obtained from conventional frame analysis.



(a)



(b)

Figure 5 (a) Transverse section of barrage bays 3-4 and (b) longitudinal section of the barrage bay.

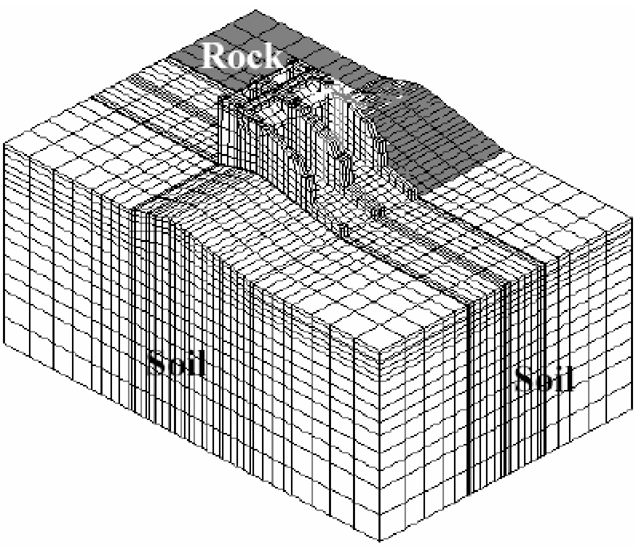


Figure 6. Finite element mesh for the analysis of barrage bays 3-4.

Nghiem and Chang (2008) reported on their investigation of soil-structure interaction in the case of a 33-story building constructed in Vietnam. As the building is to be founded on drilled shafts (bored piles), Nghiem and Chang (2008) first did a series of FE analysis of load tests on piles, calibrating the soil model as needed and testing mesh parameters for a good match with the results of the load tests. The drilled shafts for the Vietnam building will cross a thick soft soil layer (with blow counts less than 10) and bear on dense sand or gravel. The emphasis of their analyses was the seismic response of the building given certain input ground motions and various representations of the building foundations with various degrees of simplification (rigid base or flexible base modeled through one of linear springs, linear soil, nonlinear springs or nonlinear soil). The El Centro (1990) ground motion was used as input ground motion. Like Hora and Sharma (2008), Nghiem and Chang (2008) also found that the representation of the base of the structure, and in particular the distinction between rigid base and flexible base, to have important effect on the response of the structure. The analyses show that more realistic modeling of the foundations (including the foundation soil), accounting more fully for soil compliance and nonlinearity, produces less base shear. Analyses assuming rigid base are excessively conservative. The natural periods of the structure are likewise affected, with increasing natural periods resulting from consideration of flexible bases. Comparison of the analysis of the Vietnam building, which has a low-rise attachment to it, with an analysis for a hypothetical high-rise building with a geometry without any such complications suggest that details in the geometry may affect the prediction of building top deflection.

Numerical analyses such as done by Poulos and Bunce (2008) and Venkatesh et al. (2008) are often too expensive for routine projects. The profession greatly benefits when analytical models are developed that can capture SSI with accuracy comparable with that of sophisticated numerical methods yet produce foundation response in a fraction of a time. One such method, applicable to the settlement analysis of axially loaded piles in multi-layered soil, was developed by Seo et al. (2008). Pile design has traditionally relied on calculations of ultimate resistances reduced by factors of safety that would indirectly prevent settlement-based limit states. Analyses that can accurately calculate settlement for a given load will offer opportunities for more cost-effective design in the future. Seo et al. (2008) described an analysis of a single circular pile embedded vertically into a multilayered elastic soil deposit. The pile has a length L_p with a diameter B (= $2r_p$, where r_p is the pile radius) and is subjected to an axial load Q_t at the pile head. There are altogether N discrete soil layers, and the bottom (base) of the pile rests at the interface of the m^{th} and $(m+1)^{th}$ layer ($m < N$). H_i denotes the vertical distance from the ground surface to the bottom of any soil layer i; thus, the thickness of layer i is given by $H_i - H_{i\text{-}1}$ with $H_0 = 0$. All soil layers extend to infinity in the horizontal direction, and the bottom (N^{th}) layer extends to infinity downward in the vertical direction. The soil medium is assumed to be elastic and isotropic, homogeneous within each layer, with elastic properties described by Lame's constants λ_{si} and G_{si}. The pile is assumed to behave as an elastic column with Young's modulus E_p. The horizontal soil displacements in the soil mass due to the axial load Q_t are neglected in the analysis

The vertical displacement u_z at any point within the soil mass is assumed to be a fraction of the displacement of the pile at the same depth, with this fraction varying progressively from one at the pile location to zero at an infinite distance from the pile. Mathematically:

$$u_z(r, z) = \phi(r)w(z) \tag{1}$$

where $w(z)$ is the axial pile displacement function, and $\phi(r)$ is a dimensionless soil displacement decay function varying along r. From the displacement field of equation (1), strains are calculated and subsequently related to stresses using elasticity theory. The soil potential energy density is expressed in terms of the elastic constants and strains. The principle of minimum potential energy (according to which the first variation of the potential energy is equal to 0 at equilibrium) yields the governing differential equations:

$$\frac{d^2\phi}{dr^2} + \frac{1}{r}\frac{d\phi}{dr} - \left(\frac{\gamma_r}{r}\right)^2 \phi = 0 \tag{2}$$

for the soil and

$$-(E_i A_i + 2t_i)\frac{d^2 w_i}{dz^2} + k_i w_i = 0 \tag{3}$$

for the pile.

To facilitate the use of the analysis, a user-friendly spreadsheet program (ALPAXL) is available. This program relies on an iterative solution of the differential equations (2) and (3) and uses built-in functions of EXCEL. ALPAXL provides the results of the analysis, the deformed configuration of the pile-soil system and the load-settlement curve in seconds. It can be downloaded at http://cobweb.ecn.purdue.edu/mprezzi.

Seo et al. (2008) used the analysis to simulate the case study of Russo (2004). Russo (2004) presented a case history of micropiles used for underpinning a historical building in Naples, Italy. The test micropile was installed through a sandy soil to bear on a rigid layer. Due to the nature of the ground profile, linear elastic analysis with the input parameters provided by the authors of the case history proved to reproduce closely the results of the load test. Figure 7 shows both the measured and calculated load versus settlement curves. Figure 8 shows measured and calculated load-transfer curves for applied loads equal to 51, 253, and 542 kN. These figures show that there is very good agreement between the calculated and measured values, although the calculated values for the pile head settlement become smaller than the measured values for loads greater than about 400kN.

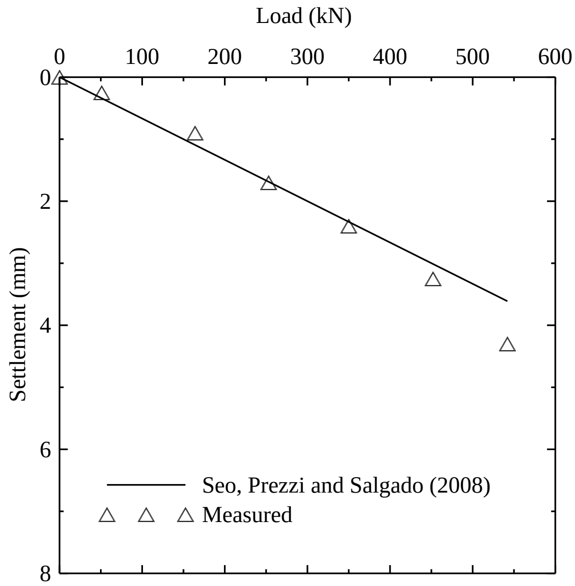


Figure 7. Load-Displacement curve at the pile head.

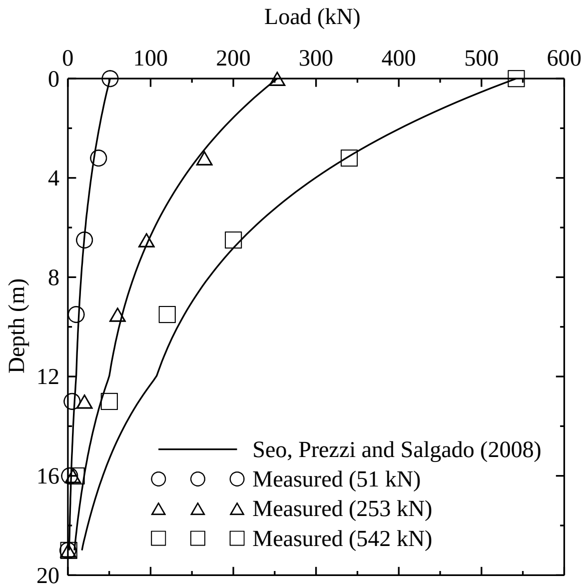


Figure 8 Load-transfer curves.

While Seo et al. (2008) showed that linear elasticity can produce useful results under very specific conditions (in the case they considered, foundations in very stiff ground), more sophisticated constitutive models are usually needed for realistic simulations of foundation problems. An analyst needs both a constitutive model that realistically reproduces the stress-strain response of soil in element tests of various types and numerical methods that can capture all the complexities of the boundary-value problems of soil mechanics. Many problems of interest in foundation engineering are large-displacement and large-strain problems. Traditional finite element analysis cannot handle the large distortions that a mesh undergoes in such problems. Techniques to overcome this shortcoming have been object of intense research in recent years. Boldyrev and Muyzemnek (2008) addressed this topic in their paper. They focused on the use of the Arbitrary Lagrangian-Eulerian technique in LS-DYNA and ANSYS combined with a modified Drucker-Prager model with explicit input of a dilatancy angle to solve the problem of indentation of a sandy soil by a rigid punch. They first determined the parameters for the constitutive model from laboratory tests and simulations of simple problems and then performed analyses of the rigid punch problem. The results are given in terms of the strain and density fields below the punch and of the vertical load-settlement curves for both centered and eccentric loads.

Not all projects call for extensive analysis. Barvashov et al. (2008) presented a few simple numerical and analytical solutions (geotoys), and advocated their use as tools for qualitative understanding of geotechnical problems. In order to understand the behavior of karstic terrains, they performed an axisymmetric FE analysis of a ground subjected to a uniformly distributed load and tracked the evolution of the plastic zones within the ground. They also analyzed a beam resting on a coupled Pasternak-Winkler foundation to show how the analysis can eradicate the artificial singularities that arise at the edges of footings resting on Pasternak foundations.

Dimitriu and Kooy (2008) examined the short- and long-term performance of large-diameter steel storage tanks on glacial tills. The diameters of these tanks ranged from 20 to 50 m, and their height ranged from 14 to 20 m. The motivation for their study was their involvement in the retrofitting and construction of new tanks. The study consisted of a historical record search, monitoring of pore pressure and settlement for the new cases and an analysis of data determined in this manner.

The older tanks were built on a relatively thin granular pad over the original ground surface. The granular pads did not have sufficient thickness, and drainage and freezing and thawing problems were apparent in some cases. The foundation soils were a fine-grained till with up to 55% clay-sized particles, up to 40% silt, and no more than 15% sand/fine gravel. In usual Canadian practice, these soils are referred to as "silty clay tills" or "glacio-lacustrine silty clays". The fine-grained components of these soils have low to medium plasticity and low activity. A typical profile consists of a top layer of actively weathered soil down to 1.5 m below grade, a desiccated crust layer down to 3.5-4.5 m, a so-called grey zone from the bottom of the desiccated crust to the bedrock. The grey zone is fully saturated and lightly overconsolidated, with OCR in the 1.3-3.5 range.

Data were available from tank surveys conducted on the older tanks from the late 1950s. These surveys contain records of long-term settlements of the tanks, measured along the rim of the tanks. Additionally, boring records were available. More complete data are available for the newer tanks, but these only provide short-term data. For the newer tanks, load tests were performed with the tanks filled up to various levels (with the possibility also of observations during unloading) and measurements made of settlements and pore pressures in the foundation soil during and after loading. These tests lasted up to 4 months.

The estimation of soil properties were complicated by a variety of factors, including the complexity of the profile, the large depth of influence and plan area of the large-diameter tanks, the fact that some increase of modulus with depth is expected (although uniform half-space assumptions were made in some settlement calculations), sources of settlement other than consolidation or elastic compression of the foundation soil (such as local shear of the sandy/gravelly pad) and the limited data from site investigations. Dimitriou and Kooy (2008) also expressed concern about possible effects of load oscillations and maintenance periods of these loads on the quality of settlement predictions (although, for design purposes, a conservative assumption regarding loads could be made, and this would be less of an issue). Nevertheless, some conclusions were drawn. For immediate settlement estimations, back-calculated elastic moduli E were observed to significantly exceed values estimated based on commonly assumed E/s_u (s_u is the undrained shear strength) ratios (1000-2000 according to USACE 1990). Differential settlements were as high as 200 mm, which, according to the tank industry, would cause a variety of problems for the tanks. However, Dimitriu and Kooy (2008) did not observe many problems, which may be indicative of the lack of knowledge regarding levels of tolerable settlement or may reflect the fact that continuous maintenance of the superstructure prevented these problems. This may offer a more general lesson. The foundation engineering industry still relies on relatively limited research on tolerable settlements done decades ago (notably, the works of Skempton and MacDonald 1956 and Burland and Wroth 1974), which do not even reflect some of the construction and architectural novelties in the building industry. If settlement estimations analysis become generally more accurate and geotechnical property estimation also improve, then there will be the need to refine the understanding of how much foundation movement modern structures of different types can tolerate.

Tsai and Zhang (2008) presented a case study that focused on the pile-load tests and the set-up effects on driven pile foundations for a 17-m-long bridge in Louisiana. The average mudline elevation of the site was −1.5 feet with the water table occurring roughly at the sea level. The average soil profile in the region consists of organic-reach, alluvial clays overlying a dense deltaic sand layer. The clay layer contains intermittent layers of silty sand. The overconsolidation ratio of the clay layer varied between 0.2 and 1.5; the average undrained shear strength normalized with respect to effective vertical stress was 0.17. The project was in an environmentally sensitive area consisting of marshes and wetlands. Minimal disturbance from the pile driving was permissible with no construction traffic allowed on the marsh area. The pile set-up study was important for the project in order to determine the proper construction sequence.

Nine piles instrumented with strain gages were load-tested, out of which six were prestressed precast square concrete piles with lengths varying between 130 m and 210 m, two were spin cast cylindrical piles with 54 inch diameter with a length of 160 m and one 195-m long open-ended steel pipe pile. The two 54-inch diameter piles were tested with a STATNAMIC^{TM} device and analyzed using the segmental unloading point (SUP) method (Mullins et al. 2002). The other piles were tested statically following ASTM D113. The pile capacities were predicted by the Tomlinson's α-method and the Nordlund method for cohesive and cohesionless soils, respectively. The difference in the predicted and measured capacities for these piles ranged from <5% to ~25%.

Dynamic testing with pile driver analysis (PDA) was performed at various time intervals from 2 to 5 days after initial driving. Tsai and Zhang (2008) inferred from the tests that the increase in pile capacity with time was due to the increase in side resistance values. Based on the tests, a set-up curve was developed for use at the site.
Koudelka (2008) compared the geotechnical design prescriptions of Eurocode 7-1 pertaining to bored piles with those of the Czech standard. According to Koudelka (2008), the Czech standard has been proven over a number of years of use in practice, which is in contrast with the Eurocode EC 7-1. According to EC 7-1, design can be based on any one of the following: 1. load tests that have been shown to be consistent with experience, 2. empirical or analytical methods that can be shown to be applicable, 3. dynamic load tests verified through representative static load tests, or 4. observation of a comparable foundation. Koudelka (2008) claims that the Czech standard has a more prescriptive approach, in which methods of calculation are specified in some detail.

LOAD TESTS ON FULL-SCALE AND LOBORATORY-SCALE FOUNDATIONS

Static Tests

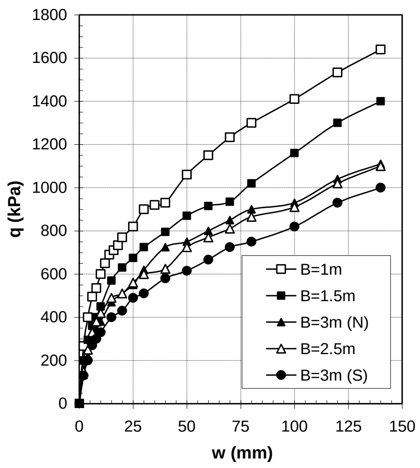
Geotechnical engineers often rely on a mix of theoretical and empirical design methods to estimate the ultimate capacity of a foundation and the likely settlement or lateral deflection of it under vertical and lateral design loads. A key feature of the process, which is critically based on the experience of the designer and the degree to which the soil has been characterized, is the selection of soil parameters. Because much of the decision-making process involves somewhat subjective decisions by the designer, it is common to measure the response of the element using either model or full-scale load tests. Ideally, all experiments should be performed on full-scale foundations in field conditions. This is rendered very difficult for two reasons: cost and the natural variability of soil. The natural variability of soil across the project sites might make the generalization of the test results difficult even in cases in which sufficient funds are available to perform full-scale experiments. Another feature of many tests on full-scale foundations is that, because of the high cost of installing the foundation and providing adequate reaction (for a load test), tests are rarely continued to adequate displacements. In the cases in which a proper instrumentation program has not been implemented, this can often result in misleading interpretation of the test results.

Of the contributions to this session, several papers describe load tests on foundations. These papers examine many of the issues currently of interest to foundation designers, namely the distribution of pile resistance between the shaft and base of piles; differences between compression and tension loading; the use of innovative instrumentation; the effect of time on bearing resistance; and the use of models to simulate full-scale behavior.

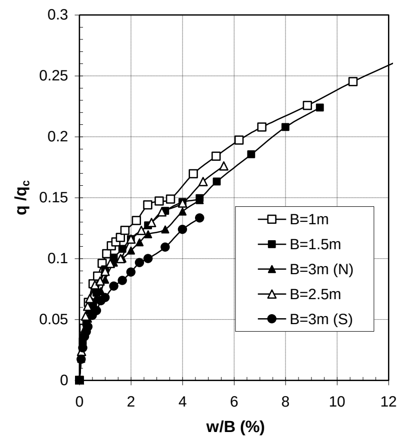
A well designed experimental program should have a reasonable (consistent) definition of "failure" or ultimate load, a detailed soil characterization plan through in situ and laboratory tests, and proper instrumentation with sufficient redundancy, calibrated and carefully interpreted. Scale effects that may exist even on prototype foundations must be recognized and minimized.

Many of the equations used to calculate the ultimate load capacity of foundations (based on either in situ test results or soil properties, as described by Salgado 2008) do not specify the amount of displacement required to mobilize the calculated ultimate resistance. This issue has been considered by many researchers (e.g., Lee and Salgado 2005 and Jardine et al. 2005) who have investigated the use of empirical equations linking in situ test data and bearing resistance mobilized at a specified strain level (usually 5% or 10% of the footing width B). The rationale for such definitions is clearly demonstrated by footing test data, presented by Briaud (2007), shown in Figure 9 in which the bearing pressure q is plotted as a function of footing settlement w.

The data shown in Figure 9 clearly suggest a strong scale effect, with the bearing pressure mobilized at a typical allowable settlement of 25 mm decreasing by approximately 60% when the footing width increases from 1 to 3 m. However, Briaud (2007) noted that, when the bearing pressure was normalized by *in situ* measurements (such as the Cone Penetration Test (CPT) end resistance q_c, the pressuremeter limit pressure or the Standard Penetration Test (SPT) N value) within the zone of influence of the footing, the data form a unique curve when plotted against normalized foundation settlement. The consequences of this finding are important when considering field and laboratory experiments and suggest that model footing or plate load tests can be used to investigate the response of full-scale foundations if differences between the soil strength in the zone of influence can be quantified using in-situ strength tests. Thus, a rational means of comparing the mobilized bearing resistance of footings of different size is to compare the resistance mobilized at a given normalized settlement (w/B).



(a)



(b)

Figure 9. Pressure-Settlement curves from full-scale footing tests on sand (after Briaud 2007).

Specifically with respect to piles, definition of the ultimate load as that corresponding to a settlement equal to 10% of the pile diameter has gained favor in recent years. Depending on the soil in which the pile is installed, its method of installation and the pile geometry, it is possible that a pile might reach a plunging load before the settlement reaches 10% of its diameter (as might happen for driven piles in clay). It is also possible that a pile will not reach that level of settlement even with a sturdy reaction system (consider the case of piles socketed into rock, for example). The degree of compressibility of the pile (dependent on its slenderness ratio and cross-sectional and material properties) will have an effect on the ratio of pile head to pile base settlement. For these reasons, use of a standard settlement-based criterion to define the ultimate load should be done with attention to the specific conditions of the project. Despite these factors, it is important for advancement of pile design methods and for improvement of pile designs that much more attention be paid in practice to understanding what ultimate or serviceability limit states we are designing against and that we define ultimate or tolerable loads accordingly.

The insensitivity of the normalized base resistance at relatively large strain levels (w/B $\geq$ 5%) to the foundation width has been incorporated into design approaches to estimate the bearing resistance of shallow footings (Briaud 2007), bored piles (Lee and Salgado 1999, De Cock et al. 2003) and displacement piles (Lehane et al. 2005). In contrast, significant scale effects exist when considering the mobilization of shaft resistance on piles. Because of the effects of dilation at the pile-soil interface, model piles are known to mobilize significantly larger shaft resistance than full-scale piles in similar ground conditions. Loukidis and Salgado (2008) showed, using finite element analyses of both full-scale and model tests, that the threshold for the existence of scale effects for drilled shafts (bored piles) is a pile-diameter-to-particle size ratio of the order of 0.01.

Load tests on shallow foundations can incorporate relatively sophisticated equipment, including pressure sensors to monitor the contact pressure, piezometers, inclinometers and movement plates to monitor pore pressure response and induced displacement in the zone of influence. Simple measurements of applied load and settlement can provide a significant amount of information and allow the construction of pressure-settlement plots (see Figure 9). In contrast, for the case of piles, some form of instrumentation is required to separate the components of shaft and base resistance that contribute to the overall resistance measured by a load cell during a load test.

The importance of careful calibration of instrumentation cannot be overstressed. Most instruments are sensitive to temperature variations, and allowance for differential temperatures from lab calibrations to field application should be considered. Instrumentation of concrete piles presents a number of challenges. Fellenius (2001) has clearly illustrated the need to quantify the nonlinear stiffness response of concrete in the interpretation of test data; however, the effect of creep should also be considered (Lehane et al. 2003). The consequence of ignoring the combined effect of nonlinear stiffness and creep are considered in Figure 10, which compares the load distribution inferred from a static load test on a 12-m-long, 762-mm-diameter CFA pile (Gavin et al. 2008). It is clear that the adoption of a constant concrete modulus results in misidentification of the true distribution of shaft resistance and overestimation of the base resistance in the pile test.

Further challenges to the accurate interpretation of strain gauges on concrete piles include residual strains arising during unloading, which affect the subsequent response to loading and concrete cracking during tension loading.

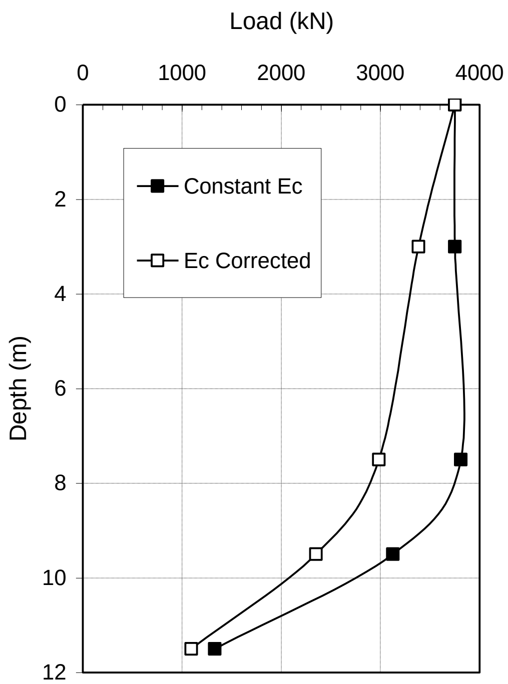


Figure 10. Effect of creep and strain corrections on load distribution in CFA pile (after Gavin et al. 2008).

Stuedlein et al. (2008) presented a case study involving compression and tension load tests performed on uninstrumented micropiles. The piles, which were formed with 140-mm diameter steel casings with a wall thickness of 13 mm, were drilled in to medium dense to very dense sand. The authors reported large differences between the load-displacement responses of the two piles, with the displacement at working load being five times greater on the tension pile than on the compression pile. A stiffness degradation model was used to assist the interpretation of the test results, and the authors found that load transferred along the cased section of the pile and end bearing resistance developed in the compression test significantly affected the load-displacement response of the piles. The authors concluded that instrumentation (e.g., strain gages) should be included in future load tests to allow an understanding of the load distribution in the piles. The likely dominant effect of interface dilation must be considered and would pose a significant challenge in generalizing the results of such tests to varying soil conditions.

Emrem et al. (2008) presented a detailed and useful case study on the use of an Osterberg cell (Figure 11) in a proof load test on a heavily loaded, 1500-mm-diameter, 47.6m-long drilled shaft (bored pile), cast for the foundations of the Princess Tower in Dubai, the tallest residential building in the world. The ground conditions at the site comprise approximately 10 m of dense to very dense sand overlying layers of very weak to moderately weak siltstone and sandstone. The Osterberg cell was placed at the mid-point of the piles and strain gauge arrays were placed along the pile length to determine the distribution of load in the pile. The authors presented an interesting comparison between predicted and measured shaft resistance. However, because the proof load was limited to 1.5 times the working load, the majority of load in the test was carried by shaft resistance developed by the rock in the vicinity of the Osterberg cell. It would be of interest to consider how the results of the load test were related to any other conventional load tests performed at the site (i.e., with the load applied at the pile head).

Ali and Lee (2008) described an instrumented load test performed on a 500-mm diameter, 30-m long, jacked-in-place pile. The subsurface comprise a clay layer down to a depth of 17 m below ground level underlain by a sandy clay layer down to a depth of 24.5 m over sandy silt. The clay layers were soft, while SPT (N) values were approximately 50 in the silt. The authors described the use of an innovative deformation monitoring system for use in presstressed concrete piles. By using extensometers coupled with high precision, spring-loaded transducers to monitor settlement, the new system overcomes difficulties caused to conventional strain gauges due to the construction process involved in the formation of prestressed piles. The authors presented load-displacement curves inferred from measurements taken at various levels along the pile shaft. They noted that the robust system could provide important data during pile installation, which is critical to understanding the behavior of jacked piles which may develop very large residual stresses.

Pathak et al. (2008) discussed model tests on footings in a laboratory test tank. While most laboratory experiments use sand as a test medium, the authors used clayey sand compacted in the test tank at a moisture content of 12.5% to a density of

18.5 kN/m^3. The material was chosen to represent a c-ϕ soil with cohesion c = 4 kPa and friction angle ϕ = 34°. Nine footing tests were performed to investigate the effect of footings shape, size and aspect ratio (length L to width B) on the pressure-settlement relationship. The footings, which varied in size from 102 mm squares to rectangular footings measuring 152 mm × 508 mm, were loaded. The authors noted that the pressure-settlement response in all tests did not exhibit a defined peak, and that the pressure increased throughout the test. They defined the bearing resistance as the intersection between two tangents, one drawn to the initial linear portion of the footing-settlement curve and the other to the later stage of the test where settlements increased progressively. These were compared to the estimates of the bearing capacity using Vesic's and Terzaghi's versions of the bearing capacity equation. The authors concluded that footing capacity increases with the size of the footing and decreases with increasing aspect ration (L/B). It would be of interest to consider the data by comparing the bearing resistance mobilized at some normalized level of settlement (see Figure 9(b)), and also explore the contribution of varying the water content of the compacted soil on the bearing resistance in future tests.

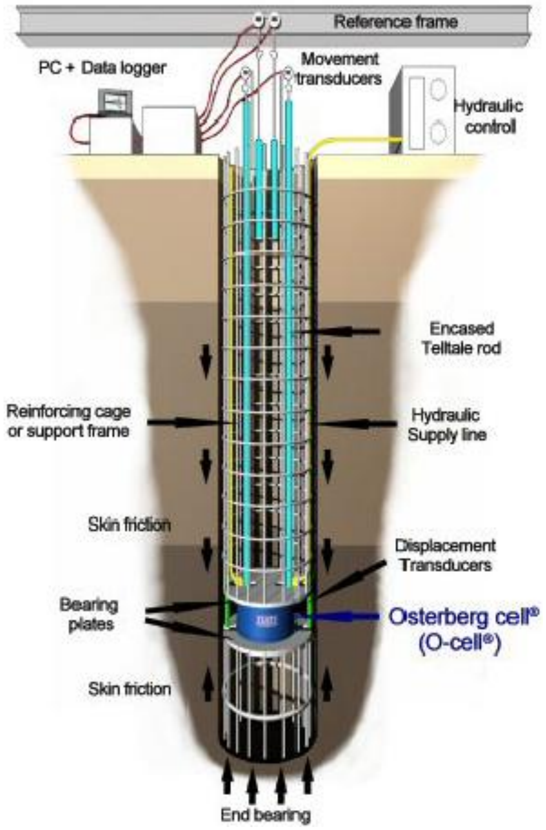


Figure 11. Typical Osterberg load cell set up.

Dynamic Tests

Static load tests are expensive and time-consuming. This has made it attractive to attempt to estimate what the static resistance of a pile is from how hard it is to drive the pile. The original attempts to accomplish this led to pile driving formulas, which are not reliable (even if still used today). A new technology to separate static from dynamic resistance during driving developed in the past 30 years. This technology and the underlying science, which we can broadly refer to as pile dynamics (Salgado 2008), has sprung largely from the works of Smith (1960), which developed the basic wave-equation analysis of pile driving, and Rausche (1970), who built the framework that allows linking the waves traveling in the pile to the force and velocity at the pile head due to a hammer blow and to the resisting forces that appear in the pile due to wave propagation.

Real-time estimates of pile resistance during driving are based on a simple theory, known as the Case method, to estimate ultimate capacity from measurements made during each blow. Analysis of the same data in the office using more elaborate theories (such as that in the CAPWAP program from GRL Engineers, Inc.) allows more accurate estimation of pile resistance and its distribution along the shaft of the pile as well as between base and shaft. The analyses, as currently used, are still somewhat crude, so dynamic tests cannot be expected to give comparable estimates of pile resistance as static tests; however, they can be performed at less cost and time and have the advantage of providing more timely feedback on the acceptability of the piling work.

The key conceptual/theoretical limitations of current pile driving analyses are the simplistic treatment of soil, which is represented by simple linear elastic, perfectly plastic springs at the pile-soil interface and dashpots with simple damping constants that are set empirically, without the separation of material and radiation damping. A continuum-based model with a more realistic soil constitutive model would greatly enhance the reliability of these theories, although it would be a formidable challenge to develop. One of the consequences of the relatively simple model for the pile shaft and base resistance is that confusion often arises in the interpretation of these tests regarding mobilization of resistance with induced displacement. When a pile is driven near the surface under easy driving conditions, the pile base resistance is that associated with very large base displacements or plunging. However, as refusal conditions approach and very small base displacements occur, the mobilized base resistance is only a fraction of the limit base resistance at plunging, although the shaft resistance would typically be fully or nearly fully mobilized along most of the shaft. If this is not recognized, comparisons, as we often see in the literature, of the estimated pile resistance from dynamic tests with resistances estimated from static load tests based on various criteria will, quite inappropriately, be made.

More recently, tests done at loading rates that are less than rates associated with pile driving were developed. These include the STATNAMIC™ test. Naghavi and Bazier (2008) presented a 3D FD analysis for simulating a STATNAMIC™ pile load test performed on a 3.75-m long pile, 0.1 m in diameter, driven to a depth of 2.8 m into a 6.1 m deep test pit with 5.5 m × 5.5 m plan area filled with dry uniform granular soil. The water table was below the depth of the pile base. The STATNAMIC™ device produced a single-pulse impact load; however, in order to extract more cycles of vibrations, a spring-mass oscillator was attached to the pile head. For analysis, the soil was assumed to follow a nonlinear (strain-dependent) modulus degradation model with Mohr Coulomb failure criterion. Rayleigh damping was used in the analysis so that shear modulus and damping could be varied as a function of soil shear strain. The authors performed a parametric study and found that the magnitude of the load and the pile slenderness ratio have significant effects on the response of piles when subjected to harmonic loading.

Over the last 10-20 years, pile dynamic tests, overwhelmingly done during initial driving, have been relied on to provide assurances that the piles as installed will develop the necessary resistances. Often, these tests are repeated some time after installation; the values obtained upon restrike of the piles can provide indications of load capacity variations with time, which, in turn, can provide an opportunity to modify designs based on possibly larger resistances. Interpretation of a dynamic test is much more involved than of a static test, relying on models of pile and soil interaction and wave propagation analysis that are still imperfect .

Ghazavi and Tavassoli (2008) presented a 3D FD analysis, using FLAC, of piles driven into the ground. The pile was assumed to be linear elastic while the soil was modeled as an elasto-plastic material obeying Mohr-Coulomb failure criterion. They verified the accuracy of their model by simulating a load test by Mabsout et al. (1994). A parametric study conducted by the authors indicated that pile penetration per unit time increased with increase in the taper angle.

Zand-Parsa and Zand-Parsa (2008) discussed how elasticity theory may be used to explain the increase in side resistance of driven piles with time by relating the release of elastic strain to the driving energy around the pile. They based their hypothesis on their experience in dealing with more than 2000 driven piles.

Akili (2008) investigated the possible causes behind the differences observed between the predicted and actual capacities (and between the associated factors of safety) of forty two driven pipe piles for offshore platforms in three adjacent fields, namely the Idd El Shargi, Bul Hanine and Maydan Mahzan fields, situated approximately 40 miles east of Doha in Qatar, which is at the southern shores of the Arabian Gulf. The author attributed the difference between the predicted and actual performance of the piles to the highly variable soil profile of the region (consisting of calcareous sands, silts and clays overlying diagenetic limestone interbedded with dolomites, marl, shale and hardened clays), insufficient site investigation operations and inadequate construction control.

The piles reported in the paper by Akili (2008) are 30-inch diameter, open-ended pipe piles with a minimum wall thickness of 1 inch. The piles were equipped with a 10-feet long driving shoe of 1.5 inch thickness. The design penetration depth ranged between 190 feet and 270 feet. Compressed air hammers Vulcan 020 and Vulcan 040 were used for the pile driving operations. Almost all the piles encountered refusal short of design penetrations. In fact, multiple refusals were encountered by several piles during the course of driving. The remedial measure available in the case of an early refusal was to drill the soil plug out (using a 26-inch diameter drill bit with a 30-inch underreamer). When the bit was 5 feet below the pile base, the underreamer was opened and a 30-inch diameter hole was reamed out for some depth below the pile base. As a result of the drilling operation, end bearing resistance could not be relied upon due to insufficient plug length. In several cases, when refusals were met, piles could be driven with ease after a short delay, which might have been due to relaxation of the stiff and overconsolidated clays and of the dense silty sands. In contrast, on several occasions, the piles could not be re-driven after a short delay in the driving operation; the author attributed this to clay setup.

The estimation of ultimate capacity for these piles were done in accordance with API RP 2A (1991) method. The shaft resistance was estimated using parameters appropriate for dense carbonate silty sands. The soil information was gathered from previously drilled borings in that area and from engineering evaluation of the installed records. Thus, the actual soil profiles at the particular pile installation sites were not available to be used in designs. Consequently, proper estimation of pile capacities and associated factors of safety were not possible; instead, estimated upper and lower bounds were calculated. Finally, Akili (2008) made a set of recommendations that may improve the pile design and installation processes in the region.

As discussed in the foregoing case study, estimating the capacity of driven piles is difficult because it is difficult to assess the amount of soil displacement and degree of disturbance caused by pile driving. Undoubtedly, the profession requires better analysis than what is available today. Chong (2008) presented an analytical method of predicting soil displacement around driven piles. Based on energy principles, the author derived equations that can be used to predict the heave and displacements in the soil around a pile caused by its driving.

The model predicts a local maximum heave in the near vicinity of the pile with a reversal of displacement at distances closer to the pile shaft leading to a downdrag. The predicted soil displacements were in reasonable agreement with those obtained from field and calibration chamber tests.

Brusey and Yin (2008) wrote about setup effects they observed for tapered and pipe piles in connection with JFK airport development. The observations result from extensive experience associated with a variety of structures (various terminals, an air control tower, an office building, parking garages), all part of the JFK infrastructure. This experience included a considerable amount of testing of piles at the end of driving and at the beginning of restrike some time after driving. By considering load capacity gains estimated from such testing, they reduced the cost of foundation systems by about 20 million dollars over a period of many years.

The soil profile at the JFK location consists of 2.5 to 5.0 m of hydraulic fill, a layer of organic soil with brown peat and silty clay ranging in thickness from 0.6 to 3.4 m, and a glacial outwash (medium to fine sand with some silt) down to a depth of 11.6-14 m. The piles were typically driven to the sand layer. The groundwater table is at a depth of 2.4 m. Bedrock is very deep.

Although a certain amount of caution is called for in estimating setup from dynamic tests, Brusey and Yin (2008) were able to progressively take advantage of the data that showed some link between taper and setup, with the setup increasing with the length of tapered sections, reaching as much as 60% (load capacity gain) for the longer tapered sections they have used. Use of lagged instrumented static load tests in programs similar to that of Brusey and Yin (2008) would shed light on the rate of mobilization of resistance at various levels along the pile and help clarify the source of any gains in capacity or perhaps different rates of mobilization of capacity directly linked to taper.

Ghazavi and Ahmadi (2008) presented an interesting comparative field study of the load-displacement behavior of uniform and tapered precast concrete piles. The 12.5-m long piles, one 400 mm square and the other tapered from 570 mm at the top to 200 mm at the base (such that that concrete volumes were equal) were driven into soil described as soft saturated cohesive (with SPT N = 5). Static load tests were performed on 35 days and 289 days after installation. The stiffness response and ultimate load resistance determined using a range of interpolation procedures were compared for the two piles. The authors show that 35 days after installation, the initial stiffness response (up to a pile head displacement of 1 mm) of both piles was similar. The maximum test load of approximately 65 kN was achieved at a pile head displacement of 2.5 mm for the tapered pile and 5.2 mm for the uniform pile. The ultimate load defined using the extrapolation techniques was 30 kN for both piles. When the piles were re-tested 289 days after installation, the initial load-displacement response (up to 1 mm) was again similar. Thereafter, the tapered pile exhibited a stiffer response. Maximum test loads of 160 kN and 90 kN were obtained for the tapered and uniform pile, respectively, at a pile head displacement of 9 mm. The authors concluded that higher pore pressures generated during the installation of the tapered pile result in enhanced stiffness and strength following pore pressure dissipation. The variation in the pile diameter of the tapered pile with depth may result in accelerated dissipation of excess pore water pressures during the equalization period and therefore enhanced strength and stiffness. The small base diameter of the tapered pile may also result in much more rapid mobilization of base resistance on this pile. However, the benefit of this will be at least partly offset by the small base area. It would certainly be beneficial to include pore pressure sensors in future load tests and separate the base and shaft load to examine these effects.

DAMAGE AND REHABILITATION OF STRUCTURES AND FOUNDATIONS

Foundation design and construction aim to produce foundations that are free of defects and that will sustain structural loads without excessive deflection, thus allowing structures to perform safely and according to specifications. However, situations arise when foundations and the supported structures get damaged or fail to perform. When that happens, it is essential to understand what caused the problem and whether there are any imminent dangers, which then allows a proper solution to be devised.

Typically, foundations are damaged structurally during construction of the foundations (but, as Wu et al. (2008) demonstrated, it can happen after the foundations work and even after the entire project is completed). If structural defects in foundations are suspected, a variety of geophysical methods may be used to check the existence of and locate the defect. For piles, pile integrity testing (generation of a low-energy wave typically at the pile head and detection of its reflection also at the pile head) is commonly used. In the construction of shallow foundations, it is easier to inspect both the bearing soil and the construction of the structural element, so cases of defects are comparatively rare.

A defect-free foundation can still perform poorly if not sized properly or placed in unsuitable soil. Deciding the type, size and depth of placement of foundations constitutes the bulk of the work done by geotechnical engineers in the larger context of foundation design. The difficulties in getting the design correct get enhanced if the bearing soil behaves unusually.

Several cases are discussed in this section in which so-called problematic soils led to failures of foundations and structures.

When a foundation fails to perform, it is important to understand the reasons behind the failure. If foundation movement is the cause of failure but no imminent danger exists to the superstructure, then it may be desirable to implement a settlement control program before any intervention is pursued in order to understand whether foundation movement is ongoing and, if so, the rate at which it is happening. It is essential to assess what the effects of movement are in the superstructure; if there is structural damage, intervention may be urgent. In situations of this type, it is not uncommon for one or more foundation elements to be bearing on material much weaker than assumed in design (as illustrated by the case reported by Horpibulsuk et al. (2008)). Such cases typically require some type of underpinning, with additional foundation elements (typically driven piles or micropiles) being installed near the nonperforming foundation elements, followed by integration of the reinforcing piles with the existing foundation by structural means. Re-leveling is called for if settlements are large. Re-leveling is an operation that requires considerable care, requiring involvement of specialized engineers.

Horpibulsuk et al. (2008) presented the case of a student dormitory at Suranaree University of Technology in Thailand, which experienced excessive differential settlements. The building is a two-story, L-shaped, reinforced-concrete building supported by five types of footing. Column span was either 4 or 8 meters. Calculation of compressive stress induced on the footings by the authors yielded 120 kPa, which was the number used for footing design and was acceptable with respect to bearing capacity failure (although no details are given regarding bearing capacity calculation). These calculations also suggested that internal forces in structural members would not lead to structural failure of these members, which contrasted with the reality of extensive cracking of the structure. The cracking was considered to fall within the severe to very severe category of Burland et al. (1977). The implication of the discrepancy between the conclusions from the calculations and the observations of the damage to the building is that considerable differential settlement would have happened.

The soil profile at the site consists of an upper layer of silty sand with SPT N ranging from 12 to 20 underlain by a residual soil of claystone containing clay, silt and sand that has $N > 30$ and low compressibility. The upper layer thickness varies erratically from 0 to 3 meters, and this variation can take place over small distances. This is indeed observed at the site of the student dormitory, which was found to lay partly on the upper, weak layer, and partly on the stiff residual soil.

Underpinning using micropiles of diameters 10, 12.5, and 15 cm, with calculated structural capacities of 201, 259 and 374 kN, was used to address the problem. Horpibulsuk et al. (2008) tested 4 micropiles and developed a relationship between the standard penetration test blow count N and the undrained shear strength s_u of the soil by back-analysis of the pile load test data. The relationship was then used to predict the load-carrying capacity of the micropiles. Settlement of the underpinned micropiles was calculated by using the finite element method. In order to install the micro piles for underpinning, holes were made through the existing foundations and micro-piles were installed by pushing the piles using a hydraulic jack. High-strength steel rods and 'C' channels were installed to enhance bonding between the existing and the new foundations. Re-leveling of the building was also done.

Wu et al. (2008) presented an interesting case, in which a collision of a cargo ship with one of the piers of a bridge under construction over the Danube in Austria led to testing of the bored piles supporting that pier using pile integrity testing. The bridge is a cantilever bridge with a length of 460 m and five spans: three middle spans and two spans to the ramps. The construction procedure was such that two pilot piles were first installed, the caisson sunk, and the remaining piles then installed. During construction of one of the piers, a cargo ship collided with it and damaged both the pilot piles. Since the piles were below water level, a visual inspection of the damage to the piles was difficult, and pile integrity tests were used to assess the state of the piles. The top of the pile was about 9.5 meters below the river water level. The tests were carried out with the help of divers who cleaned the top surface of the piles, attached the sensors on the surface of the piles and hammered the pile surface. Tests were performed at three different times, after the damage, after repair of the pile heads, and after repairing all damage. The authors concluded that the results after repair of the pile heads indicated minor changes in the cross sections. However, after repair of the whole pile, the results indicated integrity of the piles. Repair was done by drilling holes in the broken piles and inserting high strength steel rods in the holes, then placing the casing and the reinforcement cage and pouring the concrete.

Punrattanasin and Gasaluck (2008) evaluated the characteristics of loess from Khon Kaen, Thailand, under various conditions. The loess consists of 65% sand, 30% silt and 5% clay and was classified as silty fine sand (SM) as per the Unified Soil Classification System. A three-story building founded on shallow foundations bearing on this loess settled severely when a water supply pipe located in front of the building broke. The authors conducted four 1-g physical model tests, two on untreated and two on treated soil recovered from the site, under natural and soaked conditions. Results from the tests conducted by the authors showed that increase in moisture content dramatically reduced the bearing capacity and increased settlements, even under small pressures, for the untreated loess. In contrast, foundations on the cement-treated loess samples experienced considerably smaller settlements and had much greater bearing capacity. A conclusion advanced by the authors was that addition of 5% of cement by weight of dry soil dramatically increased the bearing capacity of the loess under both natural and soaked conditions.

Osman and Salem (2008) described a case study about the damage caused to a 4-story reinforced concrete building (belonging to the Great Cairo Bus Station), 64 m × 14 m, built on expansive soils in the arid Katamia region in Egypt. Hundreds of cracks appeared in the brick walls and in some concrete elements (Figure 12). The expansion joints in the building underwent significant displacement; the adjacent conduits and service lines were damaged as well. Two boreholes were made and an open pit was dug for the purpose of investigation.

Figure 12. Rupture of beam and column near expansion joint of a reinforced concrete building in Egypt.

The subsurface at the building site consists of a 2-m-thick fill overlying a hard yellow silty-clay layer with traces of fine sand (−2.0 m to −3.5 m), a second hard yellowish silty-clay layer (−3.5 m to −6.0 m) and a very hard yellow-grey silty-clay layer with traces of gypsum. No water table was observed. A concrete layer was placed at a depth of 5.5 m below the ground surface over which the reinforced concrete strip footings of the building was laid. A replacement soil layer (mixed with water and oil) of thickness 0.75-1.0 m was laid below the concrete layer. The building was constructed at the lowest level of the site beside a ground water tank. No field tests were performed before design. Structural analysis and estimation of the stresses under the foundations were calculated using a 3D FE program SAP90, simulating both the before- and after-damage conditions. The thickness of the replacement soil was not sufficient to counterbalance the expansive soil beneath; the replacement layer absorbed water and oil from the bus-wash areas that got collected under the foundations. Water seeping from the adjacent ground water tank also added to the problem. All these resulted in a significant reduction in relative density of the soil under the foundations. Consequently, excessive differential settlement of the building occurred.

Padilla-Corona (2008) presented another case study of buildings distressed by expansive soils in the Cienega de Chapala region of Mexico. Expansive soils cover about 12% of Mexican land area. At the building site, the expansive clay layer extended from 1.2 m to 4.2 m below the ground surface. It was overlain by fill material and underlain by shale of medium compressibility and plasticity. The water table was not found within the top 10 m of the borehole depth. The buildings were mostly reinforced concrete structures (with some masonry walls), and some of them had drainage work. The foundations of these buildings, consisting mostly of one or two stories, were laid at 1.6-3.0 m below the ground surface. Significant damage was caused to these building by the heaving of the soil layers, particularly during the rainy seasons. After three years of operation, there were cracks on the floor, displacements and distortions of the construction joints in the ground floor, vertical cracks in the inner walls and deteriorations in the slabs. Fluctuations in the moisture content of the underlying soil (causing volumetric changes) due to improper construction of drainage system, rain water accumulation (due to improper leveling) and excessive irrigation practices in certain places were the causes of the damage. Based on the study, the author proposed some preventive and remedial recommendations.

Al-Hattamleh (2008) presented yet another case study on expansive soil in Irbid City, Jordan. Spread footings are widely used in that region, and often heaving occurs due to improper foundation design and inadequate site characterization. The soil profile in that region consists of a clay layer (with 65% clay content) with thickness varying from 1.5 m to >6 m. The groundwater is at great depth. Alternate seasons of summer and rain lead to alternate cycles of swelling and shrinking of the clay. The building in question is a one-storey residential building constructed on isolated footings connected by grade beams. The footings were over designed because originally a three-storey building was planned. Consequently, the stresses beneath the footings were less than the swelling pressure of the in situ soil. There were differential settlements in the building with measured heave equal to 250 mm at one point and measured settlement equal to 200 mm not far from it. The grade beams were improperly designed because of which they could not impart sufficient stiffness to prevent building deformation. The author performed laboratory tests to study the swelling behavior of the clay based on which he made predictions about the soil deformation.

Sajedi et al. (2008) presented a laboratory study on the gypsum reach soil of Mashhad, Iran. The soil undergoes large settlement due to high void ratio. At the same time, the soil swells in the presence of water due to high gypsum content. The buildings and roads in that area have undergone distress due to the presence of this soil; uneven floors, differential settlements, and cracks in the structures are common features of that area (Figure 13). In order to study the properties of the soil, a series of laboratory tests were performed in which the chemical and mechanical properties of the soil were determined. Based on the study, the authors suggested that alternate loading and unloading may be used as a potential method of ground improvement in that area.

Figure 13 Damage in floor due to swelling of gypsum soil.

Expansive soils are not the only cause of foundation heave. Meyer et al. (2008) presented a case study about evaluation and repair of a subterranean parking garage located in the east central area of Miami-Dade County in Florida that heaved due to the generation of excess pore pressure during the hurricane Irene in 1999. The authors discussed in detail the general geologic and hydrogeologic conditions of south Florida and site-specific subsurface and hydrogeologic information. The subsurface at the project site consists of 15-ft thick oolitic limestone of the Miami formation with standard SPT N values 15-35 underlain by a 12-ft thick sand layer with SPT N values ranging 5-10. Layers of sandstone and cemented sand of the Fort Thomson formation are present below the sand layer. The water table is near the ground surface and the subsurface formation has a high hydraulic conductivity. The garage requiring repair showed extensive cracking and bowing/heaving of the ground floor slab. The underside and exposed surface of the elevated concrete deck had extensive cracks. Accumulated water was observed throughout the lower level and flowing water was observed through cracks within the slab and at the interfaces of the slab to the footings. The foundation system of the structure consists of shallow column- and wall-footings supported on Miami limestone. The ground floor slab was constructed monolithically with the footings. The high uplift pressure due to flooding during the hurricane caused the ground floor slab to heave and crack and caused the outer footings to be raised resulting in the cracking of the elevated parking deck. Several repair and rehabilitation alternatives were considered. A water-proofed, hydrostatic-resistant slab with a tie-down system was selected as the preferred alternative. The authors discussed the traditional subterranean foundations and their repair measures used in the area and explained why they chose this particular remedial measure. They also provided a detailed exposition of the design, and construction of the geo-structure, and discussed the performance of the anchors, joint grouting and construction dewatering involved.

Graterol (2008) analyzed several foundation failures in plastic clays. The author defined an overload ratio R based on an empirical relationship between the undrained shear strength and yield stress of clay, and related the overload ratio with the foundation factor of safety F. Based on the relationship between factor of safety and overload ratio, the author identified different possible failure (or safe) regimes for foundations. The author then reviewed several foundation failures reported in the literature (including the Tower of Pisa) and showed how the F versus and R relationship can be used to corroborate the foundation response.

El Far and Davie (2008) presented a case study of settlement of tanks at a power plant site located on the Damietta branch of the river Nile. The presence of a highly plastic clay layer at a depth of about 10 m was the main cause of the settlement. The recorded settlement of two 30-m diameter oil tanks, after five months of monitoring, was 190-230 mm at the tank perimeter; the total predicted settlement was about 300 mm at the tank perimeter and was close 550 mm at the center. Interestingly, the settlement pattern of another 7-m diameter water tank, located close to the oil tanks, was quite different; within six days, the tank settled by 130 mm on one side and by only 20 mm on the other side. The water tank was only about half the height and less than one fourth the diameter of the oil tanks. The authors investigated this counter-intuitive phenomenon although no definite conclusion regarding its cause could be reached.

Barvashov et al. (2008) presented a case history of two newly-constructed buildings in Moscow that resulted in undesirable deformations of the adjacent pre-existing buildings. Moscow is undergoing a massive construction boom; tall and slim buildings on top of multi-level parking lots are being constructed in congested urban areas. However, such constructions affect the adjacent buildings adversely.

One of the buildings, a Γ-shaped, 20-storey residential building with approximately 20-m square plan dimension, was founded on a 1.2-m-thick raft. The raft was placed on 0.7-2.8 m thick fill overlying a layer of dense sand, and layers of sand and clay loams. The water table is at a depth of 10.6-11.8 m. The building was constructed at a distance of 1.5-2.5 m from two adjacent masonry buildings, one 5-storey and the other 8-storey high. The excavation for the building construction was supported by 18-m long precast piles with 0.75 m diameter. The soil beneath the foundations of the old buildings was grouted. After the excavation, the old buildings settled by 12-18 mm; after two years of construction, the maximum settlement observed in the old buildings was 24-34 mm.

The second 6-storey building, with an 8-m deep, two-level parking lot, was to be constructed adjacent to an existing historical building. There are four other existing historical buildings in the neighbourhood founded on alluvial sands. The water table is at 6.0-6.5 m below the ground surface. The construction operation began with strengthening of the subsoil and the footings of the existing buildings. Pilot holes with 377 mm diameter and 18 m length were drilled (for constructing cast sheet piles), which resulted in settlements in the adjacent building reaching up to 90.8 mm. At that time, the construction was stopped to assess the soil condition, and electro-dynamic cone penetration tests were performed. The tests indicated substantial loss in soil stiffness during borehole drilling.

Chirica (2008) presented another case study of an excavation in the historical center of Bucharest, Romania in which the construction of a new building affected the existing buildings. A new building was to be built at a site flanked by old historical buildings and high-traffic boulevards. Chirica (2008) discussed the adverse effects of the excavation on the pre-existing historical buildings, the difficulties and defects in construction, the in situ tests performed, and the monitoring of settlements. The author finally discussed the restoration of the historical Romanian church. The author emphasized the importance of field measurements and a proper coordination between structural and geotechnical engineers as keys to success for projects such as this.

Kolev (2008) presented an interesting case study involving karstic caverns in the southeastern part of Sofia, Bulgaria, discovered in 2006 during an excavation operation done for the construction of four identical seven-storey buildings. Many caverns were discovered in the region at that time, and geophysical (electro tomography) methods were used to map the entire region in details. The caverns are located in the Quaternary layers consisting of sand and clay. Interestingly, the locations of these caverns were originally determined twenty years back; however, the data was not published due to the political turmoil and instability in Bulgaria at that time. The identified caverns were excavated, cleaned and filled with gravels and pebbles with clay admixture compacted in layers of 300 mm thickness. A contact layer or an embankment was constructed over the filled caverns, which acted as a base layer on which the building foundations were laid. Caverns were also discovered underneath partially constructed buildings in which case, cement solution was injected in the caverns. However, the author considered this as an improper remedial measure because of the difference in strength and stiffness values of cement from those of sands and clays.

Stamatopoulos (2008) investigated more than thirty slope failures and related the level of ground movement to the degree of damage caused to buildings constructed on or near the crest of the slopes. He considered slope failures due to rainfall and earthquakes, and buildings that were founded only on shallow foundations. Such failures caused large deformations or even collapse of the buildings. Based on the study, the author defined a parameter relating the width of the foundations to the width of the soil mass below the foundations that settled. On the basis of this parameter, he related the amount of slope movement to the degree of building damage.

Redgers et al. (2008) made a case for estimating soil stiffness after ground improvement using the Continuous Seismic Wave (CSW) method and then using the data to make settlement predictions. The CSW method uses seismic surface waves (Rayleigh waves) to measure soil stiffness. The authors presented pre- and post-ground improvement soil stiffness, in terms of shear modulus G_{max}, at four sites. These four case histories were offered as evidence of the effectiveness of CSW as a method to estimate settlements of primarily slabs bearing on improved ground. Redgers et al. (2008) believe that the primary reason for better predictions using CSW is its ability to make better stiffness measurements in improved soils, which are more heterogeneous in nature, than traditional methods relying on data generated by penetrations tests such as SPT and CPT. A shortcoming of methods based on small-strain stiffness, however, is that they may not properly account for degradation of stiffness upon shearing, and thus may not calculate settlement under working load correctly.

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