Foundation Geometry and Load Testing Reduces Uncertainty for a Sports Stadium

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ABSTRACT: A special case study is presented to demonstrate the influence and evaluation of uncertainties involved towards foundation design in geotechnical engineering. Site heterogeneity, variability in geometry, mechanical properties of bedrock, and seismic risk are the main sources of uncertainties considered. The subject structure is a new covered sports arena to be constructed at the Asian part of Istanbul, Turkey. The site is an old rock quarry and subsequently has been utilized as a landfill basin to accommodate soils and trash from various excavations within the city. Main lithological units underlying the upper uncontrolled fill are limestone, sandstone and shale at varying locations and depths. Istanbul is seismically very active and the planned structure for the sports arena is a dome supported on widely-spaced exterior columns. The high seismicity of the region combined with the very heterogeneous and poor subsoil conditions, represent a big challenge for the decision process in design of foundations. Based on the various factors under consideration, it was decided to implement barrette foundations socketed in to the underlying bedrock to satisfy the specified vertical displacement based performance criteria of the structure. The shear resistance of the socket interface was estimated during design stage using various empirical equations utilizing compressive strength of the bedrock. It was concluded that variability in lithology and mechanical properties of bedrock at various locations and depths results in a large range of unit skin friction values estimated, using various relations proposed, bring further complication in the decision process in estimation of optimum socket length. O-Cell loading test procedure has been employed during design to verify and optimize the socket length.

INTRODUCTION

Uncertainty in foundation engineering is an important part of our profession. We have often faced various special conditions that bring the uncertainty in design and construction of foundations. Site heterogeneity, variability of soil properties, seismic risk, and ground water regime are some of the important factors that contribute to uncertainty. Obviously, uncertainty means risk; therefore, how to handle such situations directly dictates the level of risk and the consequent cost in foundation design. Consequently, uncertainty in foundation engineering brings the great challenge to the geotechnical engineering practice. In this paper, a special case study is presented to demonstrate various decision processes used to handle various uncertainties involved in foundation design and construction.

PROJECT

A well-known sports club, through their sponsor, has decided to construct a complex project that included a covered sports arena with a capacity of 18,000 people, a hotel, an office block and shopping mall located in the Asian part of city of Istanbul, Turkey. The site is located about 15 km north of North Anatolian Fault (NAF) line which is well recognized for past seismic events

(Figure 1). It is well known that a segment of NAF was responsible for 1999 Golcuk Earthquake of M_w = 7.2 that caused large number of human casualties and great financial loss. It is also known that a segment of NAF located beneath the Marmara sea will create a major earthquake of M_w > 7.0 in near future (a 67% probability within 30 years). Therefore, the earthquake resistant design of every structure, especially the ones that involve public safety such as covered sports arena in this case study are of critical importance to civil engineers.

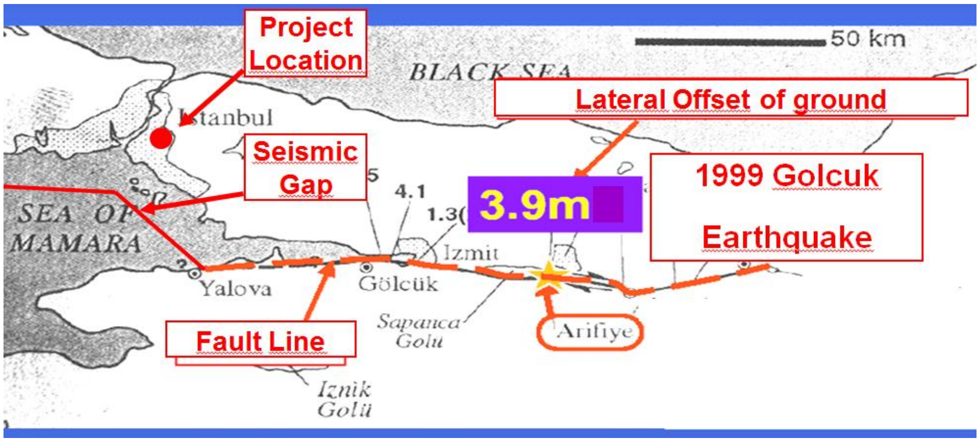

Figure 1. Location of NAF (URL-1).

This paper focuses on the foundation design of the sports arena, a dome structure about 100 m in diameter (Figures 2A and 2B). Some of the geotechnical uncertainties involved with the development include seismic risk, special site condition (heterogeneous trash fill in a former rock quarry), and subsurface site heterogeneity including bedrock surface topography, variability of lithology and mechanical properties of the bedrock.

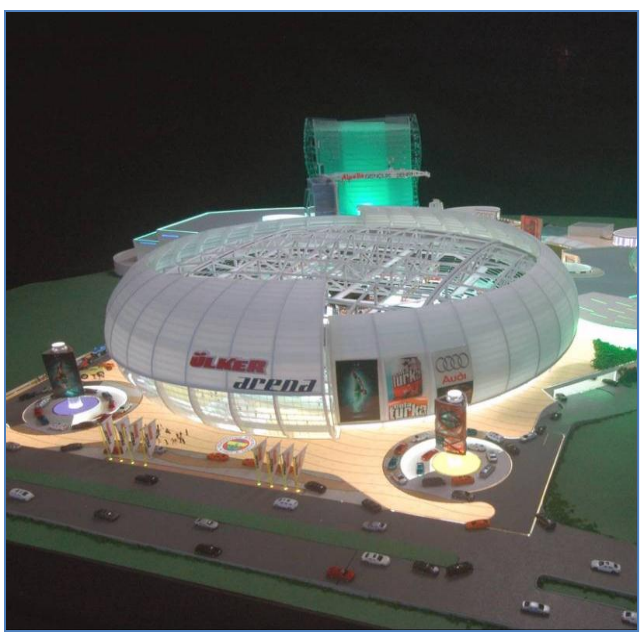

Figure 2A. Ulker Sports Arena (Development Design Group, 2009).

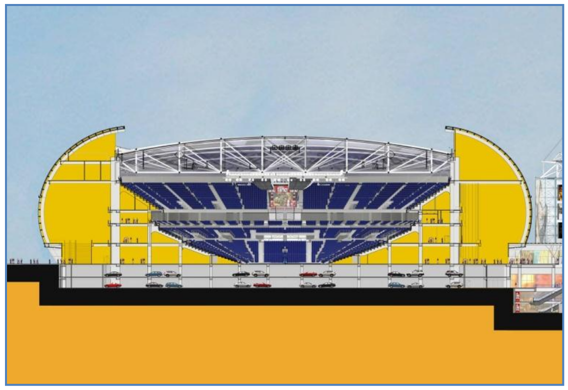

Figure 2B. Ulker Sports Arena -typical section (Development Design Group, 2009).

SUBSOIL CONDITIONS

The site is located at the junction of a main valley and a cross valley. Relatively shallow bedrock was quarried and the pit was later used as a landfill. The landfill consists of trash and the excavated surplus material generated at the various construction sites within the city. Consequently, the depth of bedrock from the existing ground surface is quite variable and more important unpredictable beneath the sports arena. In fact, the unpredictable variability of the bedrock surface topography represents a special source of uncertainty for the foundation design. Because the arena will have individual columns with high structural loads and the presence of trash in varying thickness beneath the structure, a deep foundation system socketed to bedrock will need to be considered. Consequently, to manage uncertainty involved with the bedrock surface topography, the elevation of bedrock at every column location must be determined at the design stage.

To address this uncertainty, a systematic subsoil investigation programme was implemented wherein one boring was advanced at each structural column-foundation location. The lengths of borings were specified to have a minimum depth of at least 10.0 m into the underlying bedrock. Consequently, maximum boring lengths of 65.0 m were realized at the bottom of the valley. The thickness of the uncontrolled fill located above the bedrock was quite variable as expected, reaching a value as high as 50.0 m. Bedrock surface under the foundation mat is illustrated in Figure 3.

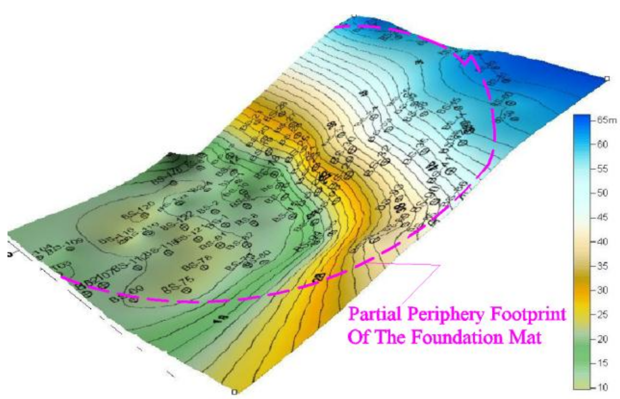

Figure 3. Bedrock Surface under the Foundation Mat

As expected, the landfill material contained all kinds of debris transported and dumped into the site and was quite loosely packed with various size void spaces. As a result, the uncontrolled fill is very susceptible to vertical displacements even without loading due to the probable partial wetting of the deposit during the life time of the structure. In addition, the hydraulic conductivity of the fill is quite high due to its loose state of placement which will represent a special problem during excavation of cast in-situ foundations to manage the loss of bentonite drilling slurry. As determined from the corings, the bedrock is very heterogeneous in both lithology and mechanical properties, which brings an additional uncertainty in foundation design. The main lithological units encountered were sandstone, siltstone, shale and limestone.

MECHANICAL PROPERTIES OF BEDROCK

Systematic core samples taken from the bedrock at various levels in each boring were subjected to uniaxial compression testing to determine the laboratory unconfined compressive strength. The uniaxial compressive strength, UCS-q_{ucs}, with depth is quiet variable as expected due to the in heterogeneity in lithology and the depth of the bedrock from the ground surface. Based on the laboratory results, there is a distinct increase in UCS with depth of bedrock at the site. The results were simplified for design as a function of depth (Table 1).

Table 1. UCS values according to depth of the top of bedrock to the surface.

d_b, m	UCS, MPa
0-15	2.5
15-25	5.0
25^+	7.5

The observed increase in UCS with bedrock depth is thought to be as a result of differences in the original and final bedrock surface topography because the site was an old rock quarry. Obviously, there are some values of UCS at certain column–foundation locations above and below the values resulting from the chosen mechanical model, imposing further uncertainty in design.

STRUCTURAL SYSTEM AND FOUNDATIONS

The domed superstructure contains 208 rectangular columns having a circular symmetry as seen in Figure 4. Obviously the outside column loads, having maximum value of 1250 tons, are much larger compared to the maximum 350 ton loads on the central columns. In other words, the distribution of vertical loads increases from the center to perimeter of the dome. A photograph of the nearly completed sports arena showing the arena's superstructure is presented in Figure 5.

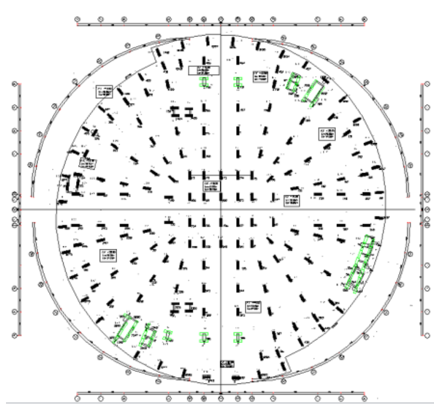

Figure 4. Layout of Columns and Foundations. (Balkar Engineering Group, 2009)

Figure 5. Covered Sports Arena (nearly completed, Jan., 2011).

The arena includes two parking levels below the ground surface. Column foundations were socketed into the underlying bedrock to a specified socket length based on the subsurface conditions encountered, the structural system, and the seismicity of the site. Cast-in-situ barrette foundations under each column were chosen for this purpose. Barrettes are rectangular-shaped load bearing elements which can resist large vertical and horizontal loads. The dimensions of the barrette foundations were 0.80 m by 2.80 m. Because a single barrette is located under each column, the total number of barrettes was also 208. Barrettes were connected with a structural reinforced concrete mat at the top against high lateral loads imposed by the structure (Figure 6).

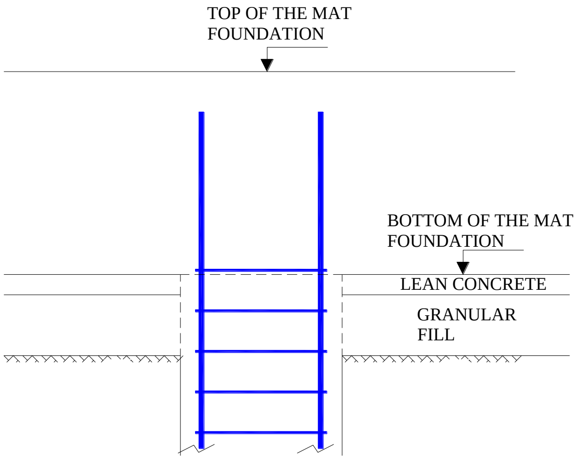

Figure 6. Typical Section for Barrette and Mat Foundation (ZETAS, 2009)

Due to the heterogeneous bedrock conditions and there was no redundancy in the barrette foundations, each barrette was designed based on the maximum column load of 1250 tons for conservatism. Barrettes offer various advantages compared to cast-in situ conventional circular piles for this specific project. First, it was possible to utilize one single barrette under each column to support a maximum service load of 1250 tons. Second, the foundations could be oriented to avoid torsional moment loading of the barrettes regardless of the direction of seismic shaking. Third, utilizing both the hydraulic grab and reversed circulation cutter technology used in barrette construction offered the confidence in obtaining desired socket lengths within the bedrock. Fourth, for the same section barrette offers a 35%[1] larger frictional surface compared to circular piles. This results in a 35% higher vertical capacity in case of a barrette compare to a circular pile having same cross sectional area. Fifth and last, tying all barrette foundations together with a concrete mat results in a much stiffer foundation system compared to isolated barrettes. Due to these advantages of barrettes over cast in-situ circular piles, their application in recent years have been greatly increased in various countries including Turkey, Durgunoglu et al., (2011), Kocak et al., (2012).

DESIGN OF BARRETTES

Considering the structural system, implications of performance criteria i.e., limiting vertical displacements under static and seismic loadings, subsoil conditions and high seismicity of the site, the barrettes were designed based on skin friction along the socket only. Low skin friction through the uncontrolled fill and possible low negative skin friction upon partial wetting of the fill both were neglected in estimating the vertical load capacity. The ultimate socket load through the bedrock, Q_{sult}, was estimated from:

$$Q_{sult} = A_s \cdot f_{sult} \tag{1}$$

where A_s = socket area which is given as $A_s = p \times L_s$ where p = perimeter of the barrette (7.2 m) and L_s is the socket length; f_{sult} is the ultimate unit skin friction developed along the socket. Based on the previous studies, f_{sult} could be estimated from:

$$f_{sult} = \alpha \cdot q_{UCS}^{\beta} \tag{2}$$

where α and β are correlation coefficients recommended by various authors and q_{UCS} is the average uniaxial compressive strength of the bedrock along the socket length. Therefore ultimate skin load, Q_{sult} will be equal to:

$$Q_{sult} = \alpha \cdot A_s \cdot q_{UCS}^{\beta} \tag{3}$$

Applying factor of safety, FS, the allowable is:

$$Q_{Ssafe} = \frac{Q_{sult}}{FS} = \frac{\alpha \cdot p \cdot L_s \cdot q_{UCS}^{\beta}}{FS} \tag{4}$$

Considering that the minimum socket length of barrettes are estimated based on the maximum service load, Q_{ser} (MPa), the socket length, L_s could be estimated from $Qs_{safe} = Q_{ser}$, i.e.

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[1] The cross section of each barrette is $A_b = 0.8 \times 2.8 = 2.4\ m^2$, whereas the equivalent diameter of a circular pile is $d_{eq} = 1.69$ m. The skin area of a barrette per meter $a_{sb} = 7.2\ m^2/m$ while the skin area of a circular pile is $a_{sp} = 5.3\ m^2/m$ yielding to ratio of $a_{sb}/a_{sp} = 1.35$.

$$L(m) \geq \frac{Q_{ser} \cdot FS}{\alpha \cdot p \cdot q_{UCS}^{\beta}} \tag{5}$$

In this relationship, p = 2(0.8 + 2.8) = 7.2 m, FS = 2, therefore:

$$L(m) \geq \frac{Q_{ser}}{3.6 \cdot \alpha \cdot q_{UCS}^{\beta}} \tag{6}$$

Table 2 summarizes different α and β values recommended by various authors. Estimated average values of f_{sult} (MPa) are 0.46, 0.69 and 0.87 for bedrock depth from ground surface ranges of 0-15 m, 15-25m and >25 m respectively for the site.

Table 2. Estimation of ultimate unit socket skin resistance

Reference Methods	Coefficients		$f_{sult,}$ MPa Depth Range		
	α	β	0-15m	15-25m	>25m
Horvath & Kenny (1979)	0.21	0.50	0.33	0.47	0.58
Carter & Kulhawy (1988)	0.20	0.50	0.32	0.45	0.55
Williams et al. (1980)	0.44	0.38	0.61	0.79	0.91
Rowe & Armitage (1984)	0.40	0.57	0.67	1.00	1.26
Rosenberg & Journeaux (1976)	0.34	0.51	0.54	0.77	0.95
Reese & O'Neill (1988)	0.15	1.00	0.38	0.75	1.13
Meigh & Wolski (1979)	0.22	0.60	0.36	0.58	0.74
$F_{sult-avg}$ (MPa)			0.46	0.69	0.87

Under these circumstances, if the average ultimate unit socket skin friction values are utilized to estimate minimum socket lengths proposed by various authors as in Table 2, the values of L_s = 7.5 m, 5.0 m and 4.0 m are determined based on the three depth ranges the bedrock surface from the ground surface at the column location. At most of the barrette locations, the depth of the bedrock is greater than 15 m. Therefore, using a single value of L_s = 5.0 m is considered to be appropriate and practical to be implemented during construction for design purposes. It should be further emphasized that the socket length of barrettes has a very slight influence on cost of foundations, considering that barrette length is directly influenced to the depth of bedrock from the base of foundation mat, and the thickness of the debris is much greater than the considered socket length. In foundation design, geotechnical engineers often consider cost vs. benefit relations in making decisions. Because the cost of the additional meter of socket length is so minor compare to the additional safety and practicality obtained by doing it, it was wise to be generous in deciding the socket length to be utilized in this specific case study. From these calculations and considerations, we conclude:

- Using a simple mechanical model for the variation of UCS vs. depth is not sufficient to remove further reliability concerns dictated by the estimation of unit skin friction along the socket. From Table 2 it can be seen that the estimated ultimate unit skin friction values are quite variable according to various α and β values recommended by various authors.

- In fact, in such a situation, minimum socket lengths chosen for design are determined based on the average values determined in Table 2, considering that the safe socket length will be verified by in-situ loading tests.
- It is clear that the variability of the bedrock mechanical properties, the UCS from one column to another and as well as with depth, brings a great range of skin friction values and consequently a reliability question in determining minimum socket lengths during design. It is also known from practice that the estimated socket lengths using the empirical relations proposed by various authors are quite conservative. In fact, this dilemma inspired Professor Osterberg to develop O-Cell testing which allowed utilization of very high vertical loads in in-situ testing of deep foundations including piles and barrettes, Osterberg (1998).

BD-SLT/O-CELL TESTING

The estimated vertical load capacity of a barrette constructed with 5.0 m socket length was tested using BD-SLT/O-CELL procedures to verify the design socket length and estimate the vertical displacement under the service load prior to the construction of deep foundations. The test barrette had 0.8 m x 2.8 m dimensions, same as the design, and an average length of 30.0 m. A maximum test load of 2500 tons, which is twice of the maximum column load of 1250 tons, was applied during the test to ensure a F.S. = 2.0 as utilized in the design. The test location had a bedrock depth of about 30.0 m from the surface to reflect a representative average fill thickness condition. Load cells were located in the middle of the socket length considering the very low resistance expected to be developed along the fill, and 2x700 tons load cells (hydraulic jacks) were used in testing. In addition, 60 strain-gauges at 10 different elevations were used to measure the skin friction distribution with depth along the surface of the barrette. The typical configuration of testing is presented in Figure 7.

The test was conducted according to ASTM D 1143-81 and loading up to 2500 tons was achieved in two consecutive steps. Load vs. vertical displacement relations were obtained by three direct measurements at top of the barrette, load cell top and load cell bottom as given in Figure 8.

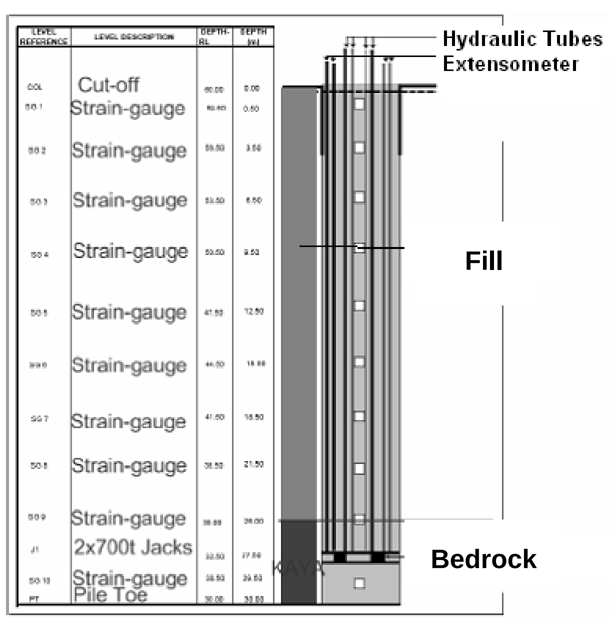

Figure 7. Configuration of Testing, ZETAS(2009).

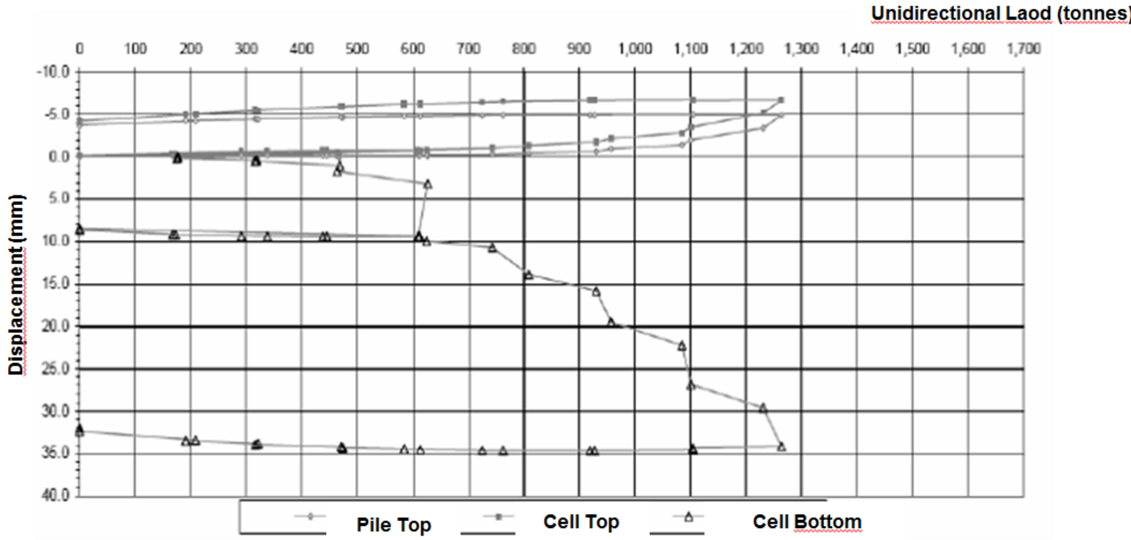

Figure 8. Load vs. Vertical Displacements, ZETAS(2009).

Using the procedure recommended by Osterberg, (1998), the real load-displacement relationship for the case of barrette loaded at the top is estimated in Figure 9.

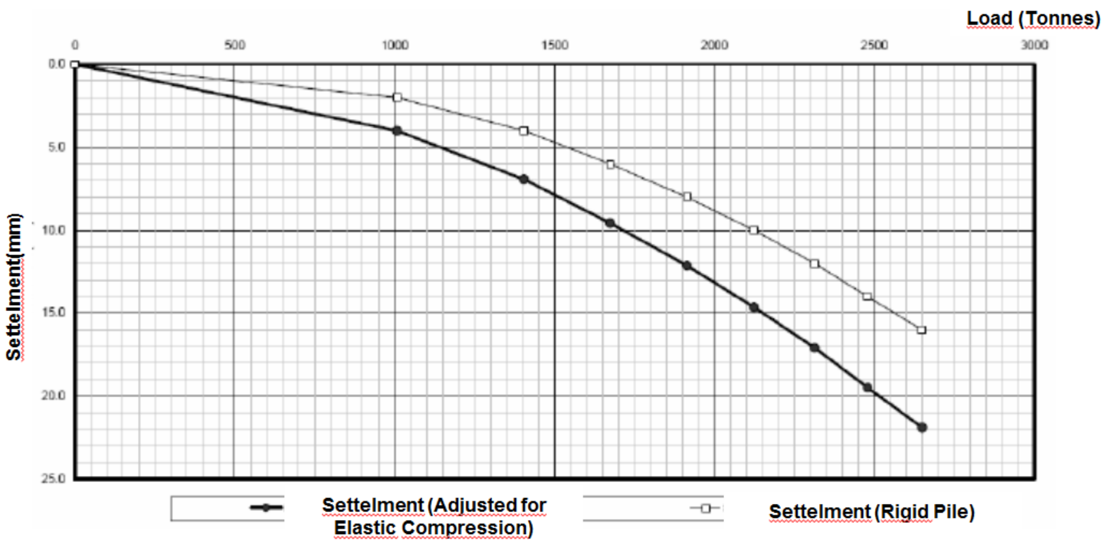

Figure 9. Load vs. Vertical Displacement, ZETAS(2009).

From Figure 9, the vertical displacement under the service load is δ ~ 5.8 mm, and the vertical displacement under maximum test load is δ ~ 19.8 mm. The vertical displacement of barrette under service load was limited with a value of δ ~ 5.8 mm which is well below the vertical limiting displacement imposed by performance design criteria of the structure. The O-Cell vertical displacement measurement results also indicate that the selected socket length of 5.0 m was quite effective, i.e. not over designed because of the fact that the load-displacement relationship was not linear throughout the test, in other words, displacement under the maximum test load (twice the service load) was as much as 19.8/5.8 =3.4 times greater than the displacement measured under the service load.

O-Cell testing has also offered the measurement of unit skin resistance along the skin of the tested barrette. Variations of skin friction vs. depth at various loading steps are presented in Figure 10.

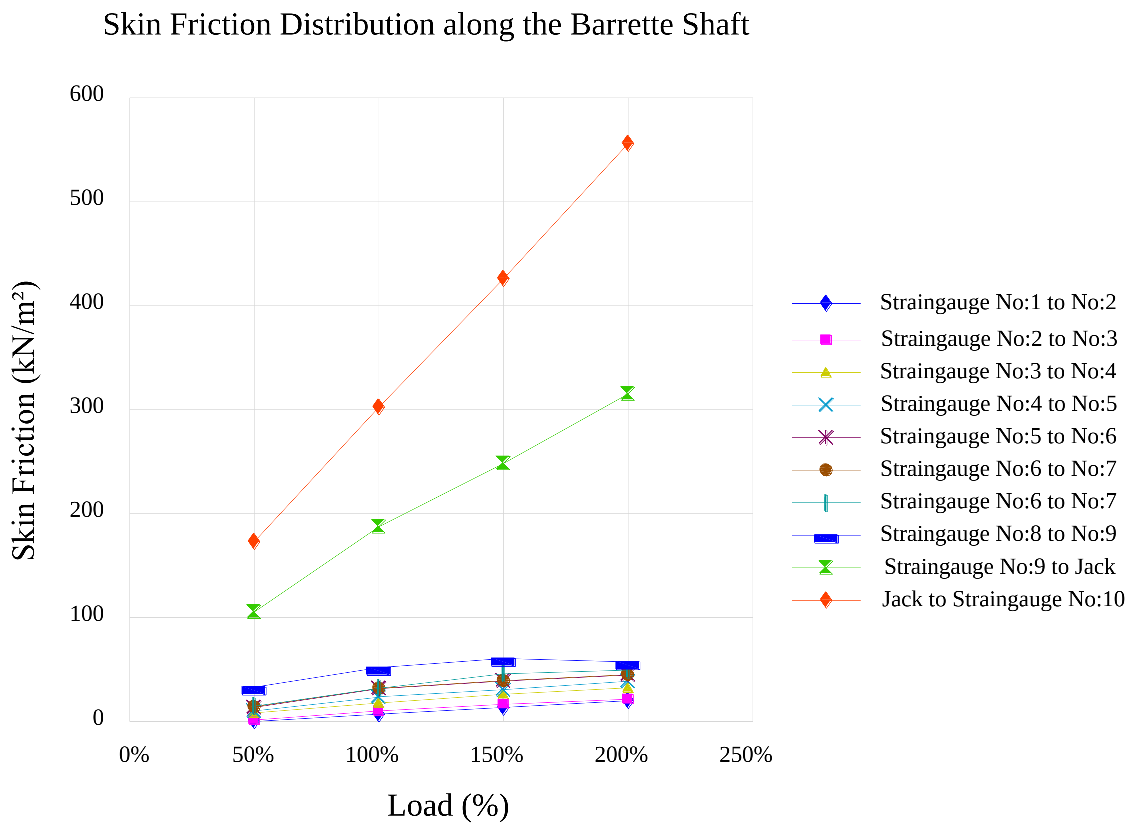

Figure 10. Unit skin friction vs. load (% of service load), ZETAS (2009).

At the maximum test load, the average unit skin friction mobilized in fill is $f_{smob\ (fill)}$ ~ 20 kPa, and the average unit skin friction mobilized in bedrock is f_{smob} ~ 300 kPa to 550 kPa.

It can be seen that the average unit skin friction along the lower half of the socket is about twice as much as it is along the upper half of the socket. That is quite reasonable considering that the site was an old rock quarry and bedrock along the lower half of the socket has experienced much less weathering and damage before placement of uncontrolled fill above the claystone bedrock. Further, the unit skin friction value of 550 kPa for the lower part of the socket under maximum test load is in good agreement with the estimated ultimate value using Carter and Kulhawy (1988) given in Table 2.

CONSTRUCTION OF BARRETTES

Mechanical and hydraulic grabs were utilized in excavation of the uncontrolled fill located at the top using bentonite slurry. Pre-grouting was used at some locations to prevent seepage of the slurry through the surface skin area of the each segment of barrette towards the loose uncontrolled fill during excavation. At lower elevations, cutters of Bauer BC-30 and MBC-30 were utilized with reverse circulation as well as using bentonite slurry to excavate the bedrock to achieve the required socket length of 5.0 m. The reinforcement cage was prepared according to its structural design and later lowered into each barrette excavation. The segment containing bentonite slurry and cage was concreted using classic methods used for tremie concrete. The specification related to bentonite slurry usage (TS EN 1538/2001) was strictly followed during construction of barrettes to ensure the concrete quality.

It is well known that one of the major problems faced during construction of circular cast in situ piles is the achievement of required socket lengths in bedrock using rotary pile drilling rigs contrary to barrettes utilized below the arena. The problem is even more pronounced with increasing diameter of the bored pile. Therefore, the geotechnical engineer must review almost every factor of uncertainty during design including constructability and the requirements and availability of special equipment and/or system if any, so that unexpected surprises are not faced during construction. It is a great dilemma to face with such conditions during construction which would result undesired delay in completion of the planned structure as a result of change in design and/or implement additional measures and unexpected increase in cost.

CONCLUSIONS

The major conclusions from this special case study that are pertinent to the choice of foundation system and the decision processes followed to deal with various uncertainties are:

- Placement of barrettes following circular symmetry in radial directions together with a foundation mat has resulted in an optimum configuration of foundations against rotational forces around the vertical axis subjected during earthquake. Therefore, a major uncertainty about how well the superstructure and the foundations will behave during a major seismic event was eliminated.
- No redundancy design by means of utilization of one barrette under each column, offered the possibility of optimum and cost effective design of foundations. Engineers must keep in mind that although no redundancy design is cost effective, strict application of technical specifications related to construction procedures and QC/QA is a must.
- Utilization of mechanical and hydraulic grabs in fills and cutters with reverse circulation in bedrock allows the possibility to construct the barrette foundations with optimum time, cost and effort in every subsoil condition.
- Furthermore, barrette foundations in terms of their constructability in most difficult subsoil conditions including bedrock and to originally designed elevations have offered great confidence and comfort in foundation design and construction for the sports arena.

From the above listed conclusions, it can be stated that the foundation system chosen for a specific project must consider all factors and implications of their uncertainties, and must be able to satisfy all pertinent concerns raised by the geotechnical engineer. Although estimation of vertical capacity of barrettes socketed in rock could be made during the design stage using uniaxial compressive strength of bedrock employing various empirical equations given in the literature, the estimated values seldom show the great range due to variability in mechanical properties of bedrock from one location to another and with depth. The number of empirical equations available for use is another source of uncertainty. To overcome limitations imposed by these uncertainties, implication of modern state-of-the art in-situ test procedures such as O-Cell testing, is the key approach for the verification and optimization of vertical load capacity of barrettes (and cast in-situ piles) even under very high loads. This verification leads to cost effective design and enhanced confidence in design safety.

Consequently, in this special case study a successful application of BD/SLT O-Cell test was used for verification testing of the barrette design (i.e. estimation of socket length and prediction of vertical displacement under service and maximum test loads). As a result, the O-Cell testing demonstrates that the foundations will experience limited vertical displacements under service loads, indicating a great comfort and confidence in the behavior of structure throughout its service life.

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