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From Supertall to Megatall: Analysis and Design of the Kingdom Tower Piled Raft

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As part of the booming development in the Kingdom of Saudi Arabia, Langan international was hired to assist in all phases of soil investigation and foundation design for the Kingdom Tower. The challenging geography of the site chosen for the tower presented a number of dilemnas that required innovative solutions, including in situ, laboratory, and full scale foundation testing and sophisticated computer modeling. Langan's investigations included geotechnical parameter selection and soil-structure modeling. The soil-foundation-structure-interaction analysis led to some surprising results that ultimately determined the final foundation design.

Structural System Description

The Kingdom Tower will be more than 1,000 meters tall and will have three wings with a center core. The tower structure will be made of reinforced concrete walls and floor slabs. The walls consist of three core walls that form a triangle at the center of the tower, wing walls that extend along the long axis of the wings, and short fin walls that are perpendicular to the wing walls. There are no individual columns or super columns in the entire building. The wall and slab structure combine to create a nearly infinitely stiff structural system from the center of the building to the extreme perimeter that created unique challenges in the design of the foundation.

The wall loads range from 50 megaNewtons (MN) for the small walls at the center of the tower to over 400 MN for the walls at the ends of each wing. The total gravity load (dead plus live) of the superstructure is about 8,800 MN (including the raft weight). The resulting average pressure on the subgrade below the raft is about 2.37 megaPascals (MPa). There are no service tension loads at the foundation level.

Left: Site location map (Jeddah, Kingdom of Saudi Arabia). Source: Langan Engineering & Environmental Services

The footprint of the tower foundation raft is approximately 3,720 square meters, with a center to edge of the wing distance of nearly 60 meters. The raft can be divided into four zones of roughly equal size: the three wings and the center core area. The gravity load takedowns resulted in a uniform loading on the raft for the four zones plus or minus 15 percent from the overall average area load on the foundations.

The foundation system for the Kingdom Tower is a system of 226 1.5-m-diameter and 44 1.8-m-diameter cast-in-place piles connected to a continuous concrete raft covering the entire pile field. The pile spacing is nominally 2.5 times pile diameter, creating a uniformly distributed pile filed. The raft has a thickness of 4.5 m at the center area and increases to 5 m at the ends of the wings. The pile depths range from 45 m at the wings to 105 m at the center of the tower. A three-dimensional view depicting the configuration of the foundation elements is provided.

Site Exploration and Subsurface Conditions

Due to the unprecedented height and weight of the building, and the fact that the project site is in an essentially undeveloped part of Jeddah north of the Corniche, an extensive and sophisticated site investigation program was developed. The site exploration consisted of seven borings, three borings to a depth of 120 meters, three borings to a nominal depth of 150 meters, and one boring at the center of the tower to a depth of 200 meters (the deepest boring related to building construction in the Kingdom of Saudi Arabia). The borings were accompanied by a suite of in-situ testing and laboratory testing, including pressuremeter tests, permeability tests, P-S suspension logging, uniaxial compression tests, and triaxial tests. A summary of the site exploration and laboratory testing program is provided, as seen in Table 1, and the boring location plan from the site investigation is presented.

Geologic Setting

The Kingdom of Saudi Arabia (KSA) is divided into two basic geological zones, the Arabian Shield and the Arabian Shelf (Pollastro et al 1997). About one-third of the KSA, including the project site, is underlain by the Arabian shield (Precambrian Basement Complex), which extends from the western coast for about 500 to 600 kilometers toward the east. The western coastal plain along the Red Sea is fairly narrow. Owing to favorable marine and climatic conditions, coral cultures grew along the coast during recent geological times giving rise to terraces of coralline limestone. Coral banks and reef limestone are often found either as outcrops on the shores or covered with sand and silt deposits. The upper layers of the coastal plain are mostly silty sandy deposits. With the retreat of the sea, the mode of deposition of sediments changed from subaqueous to subaerial and aeolian (wind) deposits. In the northern part of Jeddah where the site is located, the coastal plain is underlain by coralline limestone. In some areas of Jeddah, the coral is absent, but pockets of clay or silty sand are mixed with shells and deposits of salts and gypsum. The surficial deposits of sands and the coral limestone are part of the Quaternary (less than 2 million years ago) fluvial geologic unit. The Quaternary deposits are underlain by Tertiary (2 to 65 million years ago) basaltic rock.

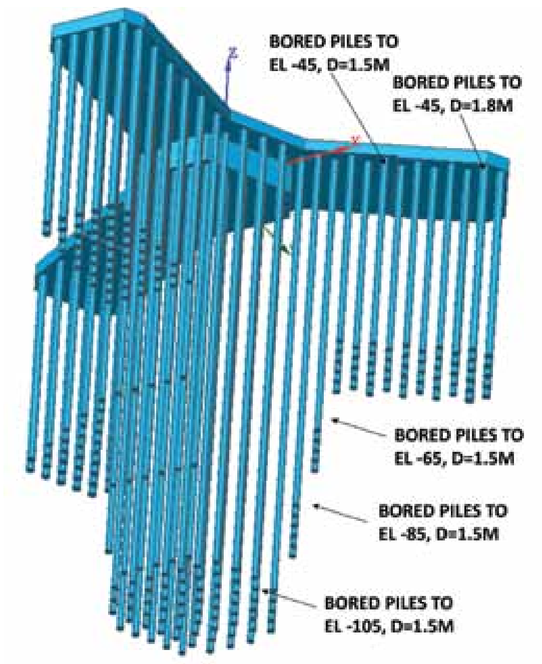
Subsurface Conditions

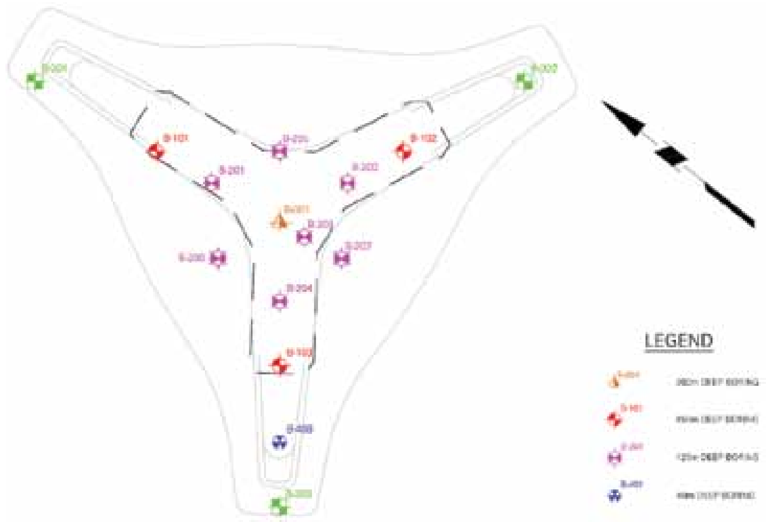
Subsurface conditions across the tower footprint generally consist of a thin layer of silty sand overlying coralline limestone rock, and then underlain by successive layers of gravel and weak conglomerate, and poorly consolidated sandstone with conglomerate and gravel inclusions to a depth around 90 to 110 meters. Beneath the poorly consolidated/decomposed sandstone is a lower gravel/conglomerate layer to about 120 meters. Beneath a depth of about 120 meters, more competent sandstone was encountered, to the maximum explored depth of 200 meters. The groundwater level at the site was measured to be about level with the Red Sea.

The coralline limestone encountered directly below the foundation level and to a depth of 45 meters is analogous in the US only to the geology of Miami and South Florida. The rock is relatively porous, having a specific gravity of about 1.8 (similar to medium dense

In-situ Testing	**Laboratory Testing**
1 boring to a depth of 200 meters	129 Water content of rock measurements
2 borings to a depth of 150 meters	129 Total density of rock
1 boring to a depth of 140 meters	72 Sieve and hydrometer (grain size) tests
3 borings to a depth of 120 meters	13 Atterberg Limits
7 groundwater observation wells	8 Point load tests of rock
95 pressuremeter tests, performed in 4 boreholes	129 Unconfined compressive strength tests of rock
2 P-S suspension logging tests	23 Instrumented unconfined compressive strength tests of rock
16 packer tests, 4 tests performed in each of 4 boreholes	8 Consolidated Drained (CD) Triaxial tests of rock
	3 Repetitive Triaxial tests of rock

Table 1: Summary of in-situ and laboratory testing. Source: Langan Engineering & Environmental Services





Left: Foundation configuration – 3D view. Source: Langan Engineering & Environmental Services

Bottom: Boring location plan. Source: Langan Engineering & Environmental Services

sand). The rock structure is comparatively strong, having an average uniaxial compressive strength of about 2.5 MPa.

A pronounced layer of the gravel and weak conglomerate was encountered about 50 meters below ground surface. The gravel was well graded, ranging from 25 to 100 mm in diameter and was rounded or sub-angular. The presence of the gravel and the potential for pile shaft instability and collapse when attempting to advance piles through the gravel layer greatly influenced the initial foundation design for the tower.

Table 2 summarizes the elevations range of the varying bedrock layers encountered during the subsurface investigation. The engineering characteristics of the various layers are discussed in a subsequence section.

Full Scale Pile Load Test Program

A total of six full scale pile load tests and one full-scale footing block test were performed as part of the design development of the tower. An example of the pile test is shown in Graph 1. The loads for the axial pile load tests were applied on circular piles and rectangular barrettes of varying lengths using Osterberg

Stratum	Approx. Range in Bottom Elevation (m)	Typical Bottom Elevation (m)	Range in Thickness (m)	Typical Thickness (m)
Surficial Sand	3 to 1.5	2.2	1.5 to 2	1.7
Coralline Limestone	-43 to -50	-48	49 to 51	50
Interbedded Siltstone	-40 to -48	-46	0.5 to 4	2.5
Upper Gravel / Conglomerate	-47 to -52	-52.5	3.0 to 10	5
Decomposed Sandstone	-90.0 to -105.0	-92	36 to 48	40
Lower Gravel	-95 to -110	-98	2.8 to 9	5.5
Sandstone	Encountered from about el -110 to			
	maximum explored depth of 200 m			

Table 2: Summary of encountered geological units. Source: Langan Engineering & Environmental Services

Load Test	Dimension (m)	Depth (m)	Drilling Fluid	Mobilized Test Load (MN)	Mobilized unit side shear (KPa)
Bored Pile Test 1	1.5m dia	45	Water	78	200 – 2,600
Bored Pile Test 2	1.5m dia	75	Mineral	68	100 – 1,600
Bored Pile Test 3	1.5m dia	45	Polymer	88	50 – 625
Bored Pile Test 4	1.5m dia	75	Polymer	66	200 – 700
Barrette Test 1	1.2m x 2.8m	45	Mineral	90	75 – 800
Barrette Test 2	1.2m x 2.8m	75	Mineral	67	50 – 1,100
Footing Block Test 1		1.7	N/A	8	3.3 MPa

Table 3: Summary of full scale test program. Source: Langan Engineering & Environmental Services

Material	Elevation, MSL (m)	Modulus of Deform (MPa)	Friction Angle	Cohesion (kPa)	Poisson's Ratio	USC (MPa)
Coralline Limestone and Siltstone Inclusions	5 to -40	500	24	170	0.35	1.5 to 2.5
Coralline Limestone, Siltstone Inclusions and Gravel/Conglomerate	-40 to -60	500 to 150 (1)	24	170	0.35	3
Gravel/Conglomerate and Decomposed Sandstone	-60 to -90	150	38	300	0.35	1.5
Decomposed Sandstone and Gravel/ Conglomerate	-90 to -110	150 to 500 (2)	24	--	0.35	3.2
Sandstone	-110 to -125	900 to 1200 (2)	--	--	0.35	2
Sandstone	-125 to 200	1200	--	--	0.3	2

Table 4: Design values (MPa). Source: Langan Engineering & Environmental Services

cells (O-Cell) by Fugro-LoadTest. A two-tier O-cell assembly was used to isolate finite segments of the pile or barrette. A equivalent top-down load of 70 MN was achieved for the piles and 110 MN for the barrettes. The test measurements included measuring the ultimate concrete-shaft skin friction and the load-settlement behavior of the pile tip. A summary of the full scale test results is provided, and representative unit skin friction plots for a pile drilled with natural slurry pile are presented in Tables 3 and 4 respectively.

The footing load test consisted of a 1.5 m square by 1.7 m high block of reinforced concrete bearing directly on the coralline limestone. The load was applied by hydraulic jack against a reaction beam. The beam relied on two concrete barrette elements. The settlement of the top of the concrete footing was measured with direct-read dial gauges. Borehole extensometers were also used to measure the deformation of the coralline limestone at finite depth intervals. The data were used to calculate allowable bearing capacity and the modulus of deformation of the coralline limestone and was extremely valuable in modeling the behavior of the pile-raft foundation system. The load-settlement plot for the footing load test is presented in Graph 2.

Geotechnical Design Parameters

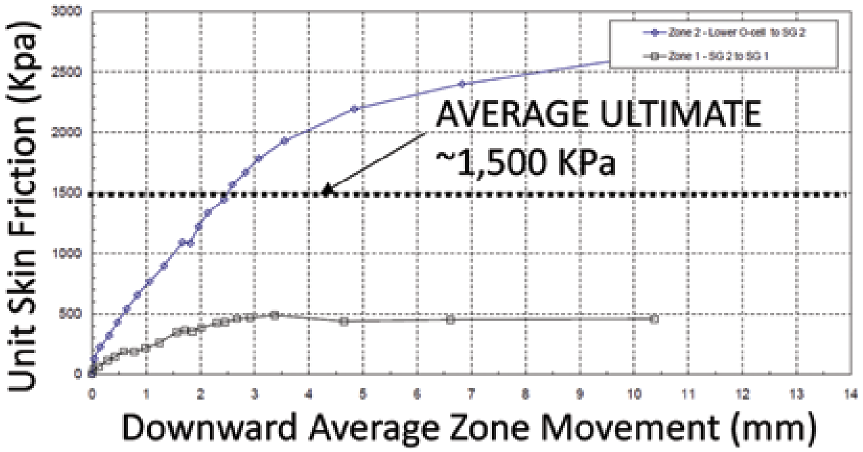
The geotechnical design parameters used as the basis for the conventional geotechnical capacity calculations and the finite element modeling were developed for six distinct layers. They were based on the in-situ testing, the laboratory testing, and the full-scale field load testing. Upper and lower bound limits were selected for each of these parameters and sensitivity checks were made with all of the parameters.

Modulus of Deformation

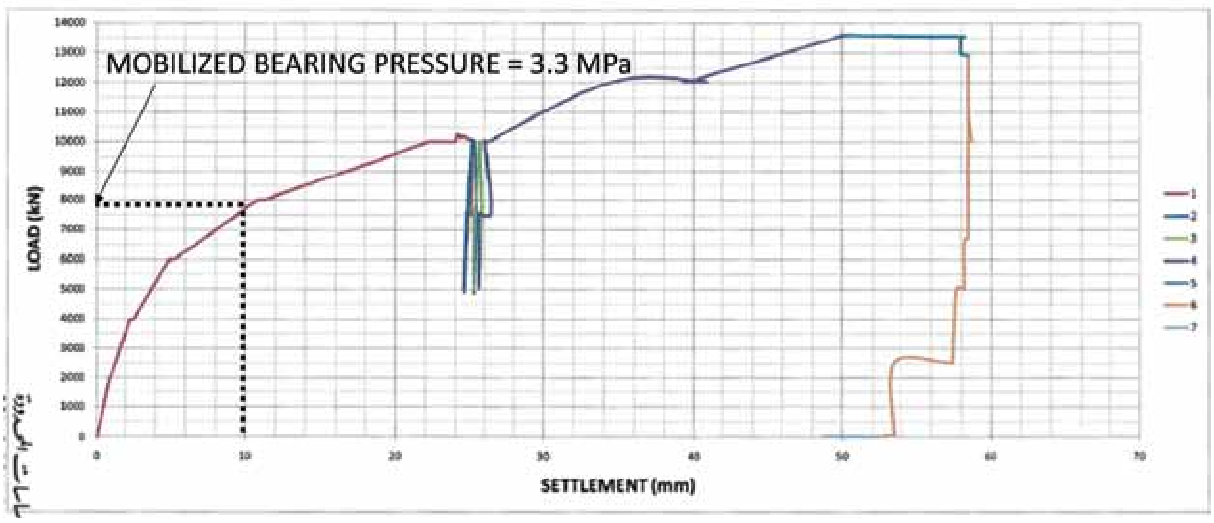
The design values of the modulus of deformation (Edef) were correlated using the results of the full-scale load tests, as well as results of the in-situ and laboratory tests, as seen in Graph 3. The results of the P-S suspension logging modulus were used to estimate the relative stiffness trends throughout the profile.

The continuous profile of Edef versus depth from various testing methods, as well as our "best estimate" profile used as the basis for

Opposite: Finite element model cross-section and material properties. Source: Langan Engineering & Environmental Services



Graph 1: Pile load test T1 (Natural Slurry) - Mobilized skin friction in coralline limestone. Source: Langan Engineering & Environmental Services



Graph 2: Footing load test - Load versus settlement plot. Source: Langan Engineering & Environmental Services

design is presented. The rock strata were grouped by engineering properties, rather than individual geological units. To account for the inherent variability in the rock parameters, we determined appropriate upper and lower bound limits to consider possible softer rock response ("softer profile") and stiffer response ("stiffer profile"). These softer and stiffer zones are based on the overall testing trends and ranges and were established to allow adequate sensitivity checks.

Foundation Evaluation – A Finite Element Analysis Approach

The vast majority of the foundation analyses were performed using finite element methods to model the piles, the concrete raft, and the complex soil-foundation structure-superstructure of the Kingdom Tower. Some early checks were made using more traditional hand calculations and proprietary spreadsheet calculations, but those results are not relevant to this paper. A three-dimensional finite-element analysis was performed to estimate the settlement response of the foundation raft, taking into account the foundation geometry, the engineering properties of the rock materials, and the loading conditions. The finite-element analysis was performed using the commercially available software package Midas GTS. The commercially available Plaxis 3D was used to check and validate the results. The following sections present a brief summary of our evaluation approach, our modeling assumptions, our initial runs, and the analysis results of the final foundation design. A section of the FEM model through the center of the raft and the strength and compressibility parameters used in the design are presented.

Modeling Assumptions

Our final finite element analysis was based on the following assumptions:

1. The foundation system consisted of a combined pile-raft foundation with 270 circular bored piles connected with a 4.5-meter-thick structural raft. The raft had an approximately 6-meter-deep depression at the center and was 5 meters thick at the edges.
2. The raft was modeled using solid tetrahedral volume elements, with

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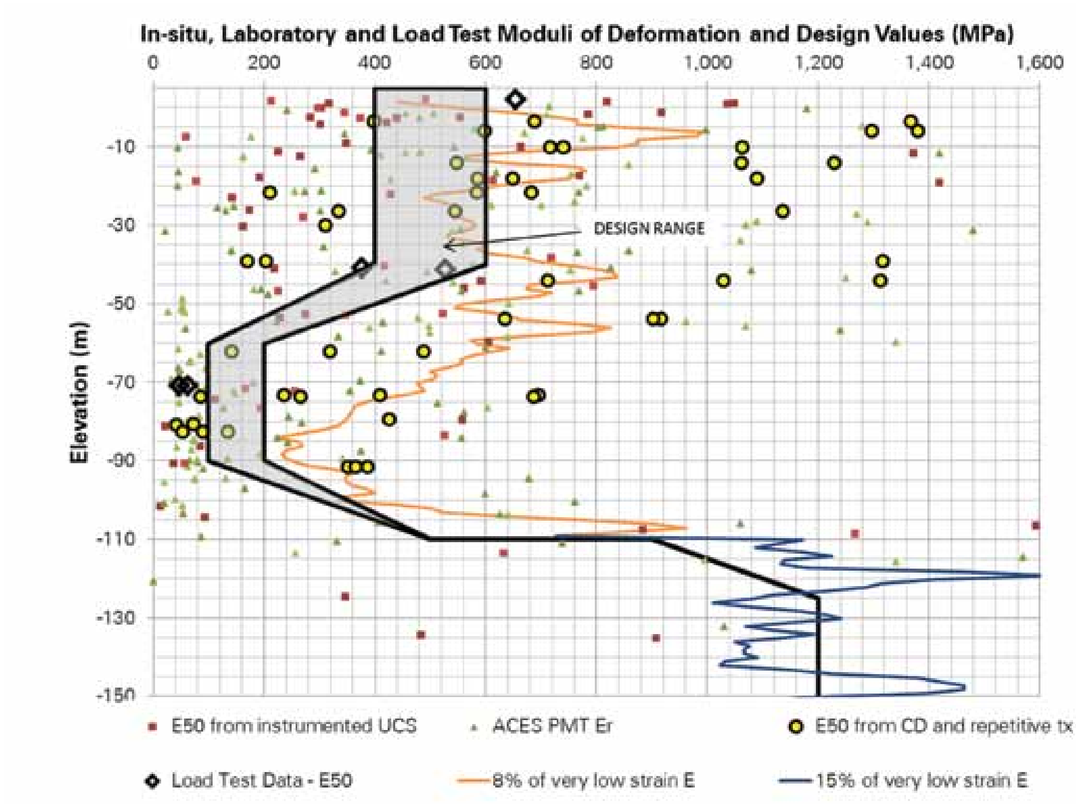
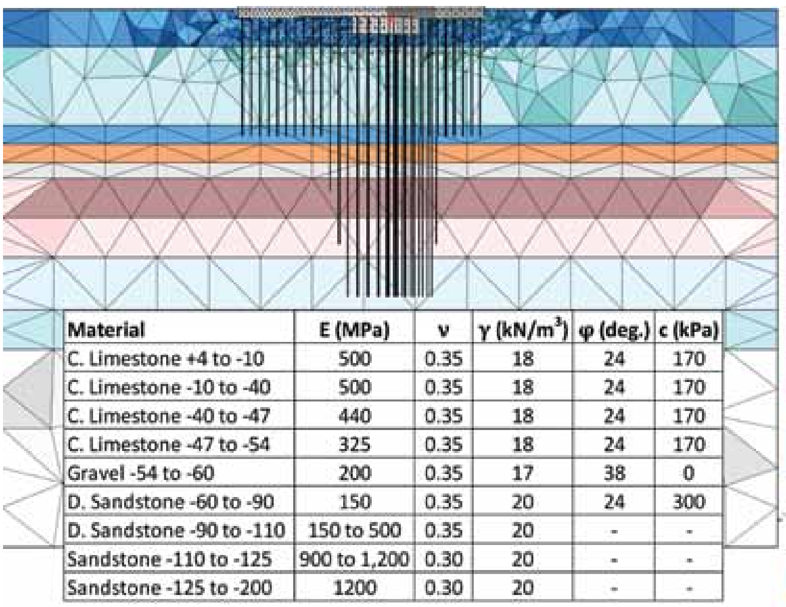
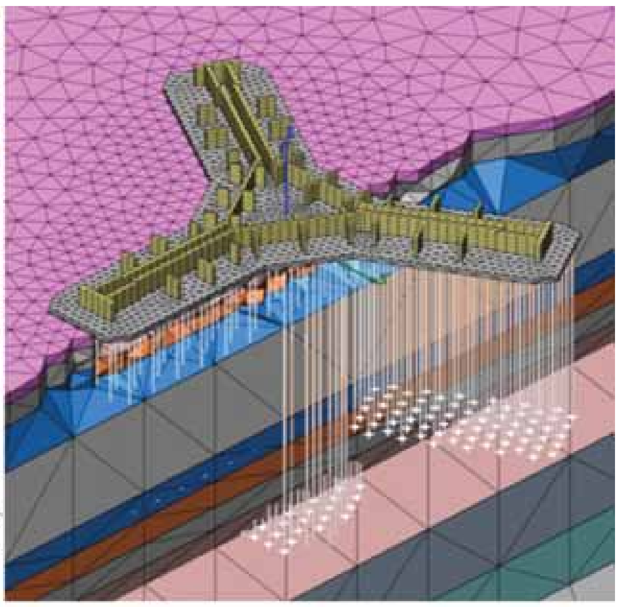


Graph 3: In-situ, laboratory, and load test moduli of deformation and design values (MPa). Source: Langan Engineering & Environmental Services

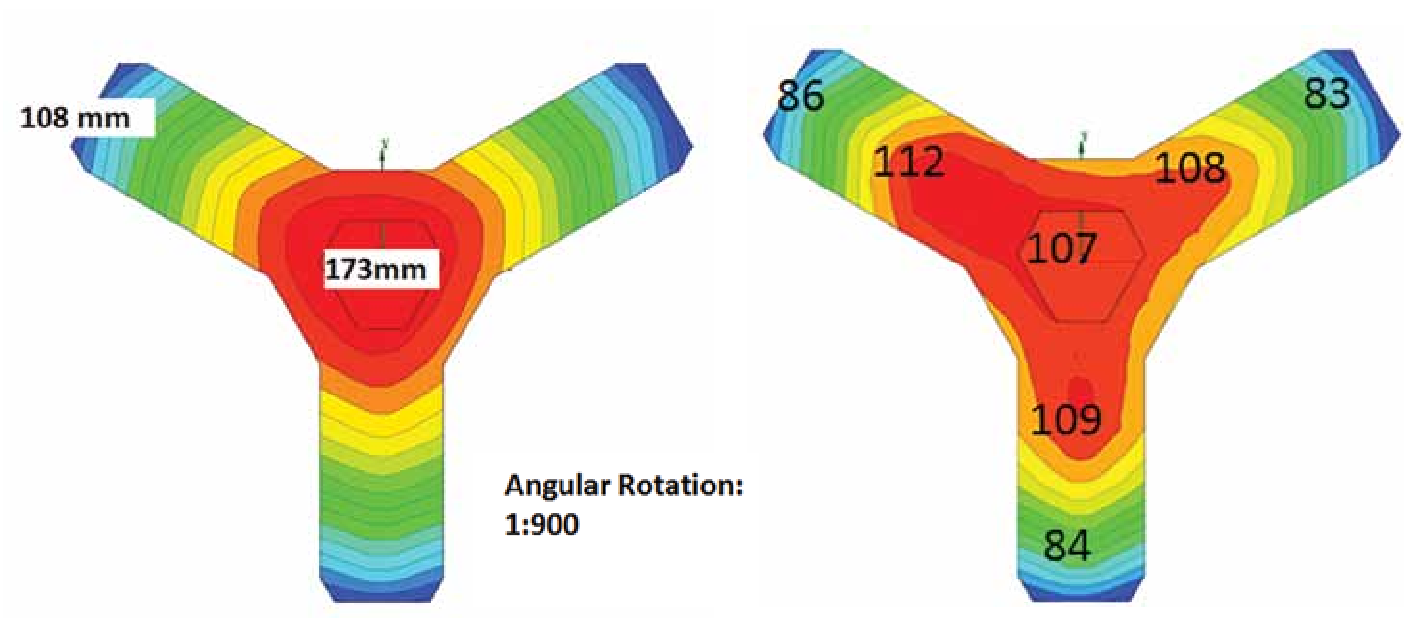


Material	E (MPa)	ν	γ (kN/m³)	φ (deg.)	c (kPa)
C. Limestone +4 to -10	500	0.35	18	24	170
C. Limestone -10 to -40	500	0.35	18	24	170
C. Limestone -40 to -47	440	0.35	18	24	170
C. Limestone -47 to -54	325	0.35	18	24	170
Gravel -54 to -60	200	0.35	17	38	0
D. Sandstone -60 to -90	150	0.35	20	24	300
D. Sandstone -90 to -110	150 to 500	0.35	20	-	-
Sandstone -110 to -125	900 to 1,200	0.30	20	-	-
Sandstone -125 to -200	1200	0.30	20	-	-



an equivalent elastic modulus of 36,700 MPa, and an elastic constitutive model. No other structural elements were incorporated into the Midas GTS model. The inherent stiffness of the concrete superstructure was captured in the structural ETABS model.

3. The Mohr-Coulomb soil model was used for the coralline limestone layer and the decomposed sandstone layer. The elastic model was used for the remaining layers. A non-linear soil-hardening model was also considered for the gravel/conglomerate layer but a review of the triaxial test stress-strain curves indicated a Mohr-Coulomb model was more appropriate.
4. All soil, rock, and gravel layers were modeled in three dimensions assuming horizontal stratification. We evaluated the variation of the subsurface parameters spatially and judged that a uniform horizontal stratification is appropriate.
5. The piles were modeled using the "embedded pile" option, which employs beam elements to model the structural characteristics of the piles and non-linear springs at the pile-rock interface. The beam elements took into account the pile geometry, deformation modulus, and Poisson's ratio. The pile/rock interface springs employed an ultimate shear stress at the rock interface, a pile-rock shear stiffness modulus, and very small tip bearing capacity and tip spring stiffness. We assumed the piles extending to elevation -45 will be drilled with natural slurry, and the remainder of the piles will be drilled with polymer slurry; accordingly we used the corresponding pile-rock interface properties.
6. The box-model boundaries were restrained in all three directions. The box-model dimensions were 300 meters by 300 meters in plan, and extended to a depth of 200 meters. The boundary conditions used did not influence the results under the raft.

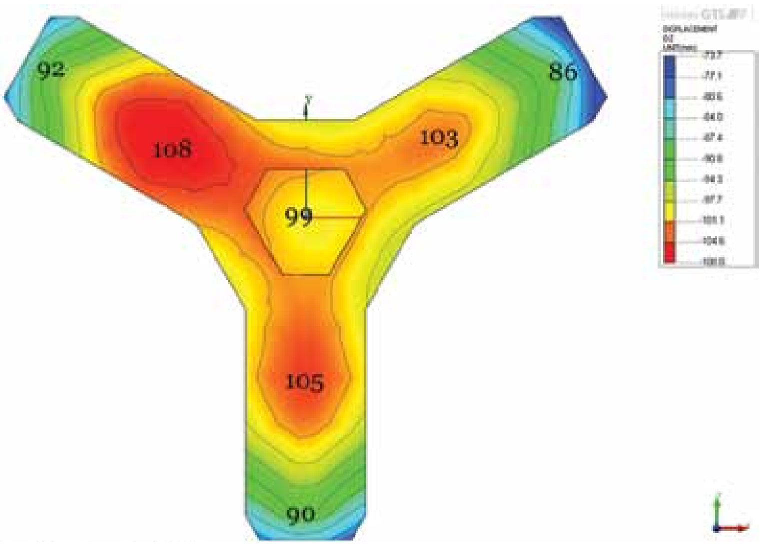


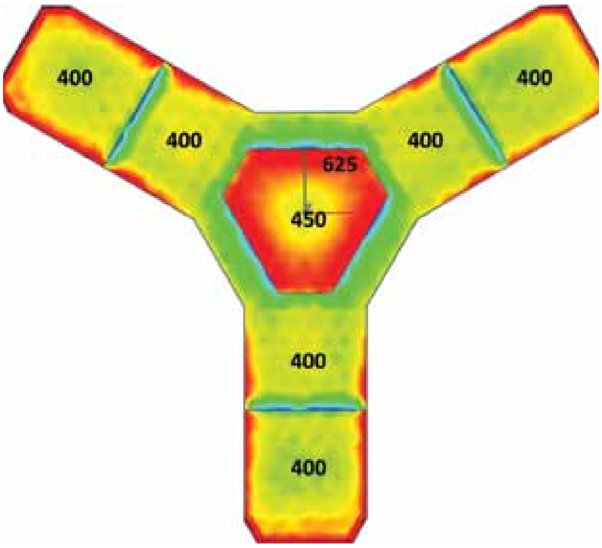
Opposite Left: Initial settlement prediction.
Source: Langan Engineering & Environmental Services

Opposite Right: Starting settlement estimate.
Source: Langan Engineering & Environmental Services

Left: Converged settlement estimate.
Source: Langan Engineering & Environmental Services

Right: Best estimate pile head axial loads (MN).
Source: Langan Engineering & Environmental Services





7. Total gravity loads (dead plus live) were about 8,800 MN (880,000 metric tones), including the weight of the raft. The loads were applied in the finite element model as pressure strips at the top of the raft. The pressures were calculated using the actual wall loads divided by the shear wall footprint area.
8. A simplified construction stage process consisting of three stages was followed for the modeling. In Stage 1, the model (soil/rock layers) was allowed to settle due to its own gravity weight, and then settlements were reset to zero. In Stage 2, the piles and raft foundation were installed. In Stage 3, the wall gravity service loading was applied.

Iterative Procedure between the Geotechnical and Structural Models

An iterative procedure was employed in coordination with Structural Engineer Thornton Tomasetti (TT) to achieve convergence between the results of the Midas GTS model and the ETABS structural model regarding the raft settlement magnitudes, the overall shape of the settlement contours, the pile loads, and the wall loads. The iterative procedure consisted of the following steps:

Step 1: Based on a first set of wall loads provided by TT, we ran Midas GTS and estimated raft and pile settlements. The software allows for calculations of individual pile head springs and area springs (modulus of subgrade reaction) for the raft bearing. For the Kingdom Tower, we had 270 piles springs and ten zones of area springs, three for each leg and one for the center core, were established.

Step 2: TT imported the pile and area springs in their model, and provided to Langan a new set of wall loads.

Step 3: With the revised wall loads we re-ran our model and provided TT new sets of individual pile springs and zones of average springs.

Steps 2 and 3 were repeated until the settlement contours between the geotechnical and the structural models differed by less than 5 mm, and the pile loads differed by less than 2 MN. This is less than 10 percent difference for each parameter.

Results

Initial Foundation Design

The pile lengths were developed by calculating the geotechnical capacity of the pile. The piles were considered frictional only with contribution of ending bearing ignored. A required pile capacity of 45 MN was determined by dividing the total tower dead plus live gravity load by the 270 piles. Although the raft would carry substantial load in bearing, the piles were proportioned assuming zero contribution from the raft. The depth of the coralline limestone was considered in determining the pile lengths, as was the presence of the rounded and sub-angular gravel directly below the limestone at a depth of about 50 meters from grade. The three factors led to an initial foundation design consisting of pile length of 45 meters from grade. The pile length provided adequate geotechnical capacity extended the full depth of the limestone, but stopped short of the potentially problematic gravel.

The analysis of the initial pile lengths revealed a common settlement profile with greater settlement at the center of the raft and less settlement at the edges of the three wings. The magnitude of settlement ranged from 173 mm at the center to 108 mm at the edges. The differential settlement from center to edge was 65 mm. The corresponding angular rotation from center to edge of raft is about 1 /900, a seemly very reasonable result. It must be noted, however, that the foundation model incorporated the foundation raft, only, without accounting for the nearly infinite stiffness of the superstructure. When the foundation-superstructure interaction was explored, the foundation settlement became predictably more uniform. However, the structure loads applied to the top of the foundation raft were distinctly non-uniform. The redistribution of loads is a predictable result of an infinitely stiff foundation on a relatively flexible foundation system. Recall that the initial load takedowns resulted in a nearly uniform loading across the entire raft. However three iterations with the structural model resulted in a three-fold increase in wall loads at the edges of the wings. In fact, the structural loads changed so greatly that the vast majority of the tower load was concentrated on the outer third of foundation wings. This created an unreasonable demand on both the superstructure walls and the on the foundation piles.

Final Foundation Design

To mitigate the differential settlement between the center of the tower and the edge of the wings, the foundation piles at the center of the tower were lengthened significantly to "reinforce" the geo-materials with structural concrete. (This is somewhat analogous to reinforcing concrete elements with steel rebar). In the end, a center pile length of 105 meters was selected to maximize the depth of the reinforcement, but to stay at least 5 meters above the lower sandstone that is more competent and significantly stiffer. There was a concern that taking the piles to the competent rock would create a hard spot at the center of the raft. Two zones of intermediate length piles were created to transition from the 105-m-long center piles to the 45-m-long wing piles. The pile diameter was not changed for any of these studies.

Raft Settlements

Using the "best estimate" design parameters for the geo-materials, the first calculation of foundation settlement using the longer foundation piles resulted in a settlement range of 83 mm to 107 mm. Comparing the differential settlement of 24 mm to the 65mm from the uniform pile scheme shows that the longer piles successfully reinforced the ground below the center of the raft.

The final converged foundation settlement was about 109 mm at the center of Wing A, 100 mm at the center of the raft foundation, and decreasing to about 90 mm at the edges of the tower wings. The differential settlement from center to edge of raft is less than 20 mm, and the corresponding angular rotation was less than 1/2000 – an extraordinary result. Also note that the settlement profile changed very little from the start to finish of the analysis. The redistribution of the structural loads was equally modest, with maximum of about 50 percent increase in the outermost wing wall.

Bearing Pressures on Rock Subgrade

The maximum bearing pressure was about 625 kiloPascals (kPa) below the raft wings and at the center of the raft. This provides a margin of safety of at least 5 on the mobilized bearing capacity. This means that there is no potential for local crushing of the coralline limestone.

Pile Loads

The best estimate analysis shows pile head loads for the 1.5-meter-diameter piles varying from about 18 MN to about 29 MN; accordingly. For the 1.8-meter diameter piles, loads varied from about 24 MN to 38 MN. Note that although the gravity-load takedown predicted uniform pile loading, the final converged model yielded greater pile loads at the tips of the wings.

Load Sharing

The results of the finite element analysis were reviewed to determine the load sharing ratio between the piles and the rock subgrade. Our analysis showed that about 70 to 75 percent of the superstructure load is transferred to the piles and about 25 to 30 percent is taken by the rock in bearing below the raft subgrade.

Pile Springs and Rock Modulus of Subgrade Reaction

Pile springs and moduli of subgrade reaction were provided to TT to model the interaction between raft, piles, and the rock directly under the raft. Zones of uniform subgrade moduli were then provided to TT. The final model shows that the pile springs vary from about 190 MN/m to 430 MN/m, and the subgrade moduli zones vary from about 1,850 MN/m^3 to 35,330 MN/m^3.

Construction Progress

The foundation piles were constructed by Saudi Bauer over an approximate nine month period ending in November 2013. The pile shaft was advanced with a Bauer BG 40 rig bucket with teeth. Natural slurry (water mixed with soil cuttings) was sufficient to maintain an open shaft in the shallower piles in the coralline limestone. Polymer slurry was used when needed to stabilize the deeper shafts. The use of bentonite slurry was not permitted. The relatively poor performance of the test piles drilled under bentonite slurry, together with our past experience, suggests that the bentonite would have a significant negative effect on the load carrying capacity of the piles.

The technical specifications included rigorous quality control both during construction and post construction of the piles. A very nominal number of non-conformance reports were issued for the entire 270 pile field. The NCRs were all related to potential concrete defects – three were at a depth greater than 60 meters from grade and the remainder was shallow within 2 meters of cut-off. The deeper potential defects were below the

"A total of six full scale pile load tests and one full-scale footing block test were performed as part of the design development of the tower. The loads for the axial pile load tests were applied on circular piles and rectangular barrettes of varying lengths using Osterberg cells (O-Cell) by Fugro-LoadTest."

load carrying portion of the pile and deemed inconsequential to the performance of the foundation. The shallow potential defects were easily remediated.

There were no reported problems with pile construction through the continuous gravel layer at a depth of about 50 meters. The piles were constructed from the outside wings towards the center of the tower, so that the deepest piles were the last piles to be constructed. We postulate that the concreting of the shallower piles partially cemented the gravel zone before the deeper piles were constructed.

Analysis Summary

The Kingdom Tower Soil-Foundation-Structure-Interaction exercise reinforced some of the following conclusions:

1. The basis for raft settlements should be the geotechnical model, and the foundation flexibility assumed in the structural model should be calibrated to approximate the raft settlements of the geotechnical model.
2. The iterative process can provide insight to the redistribution of the column/wall loads, and drive the final foundation design decisions.
3. The geotechnical engineer does not have to attempt to model the stiffness of the superstructure by modeling the columns/shear walls above the raft level. The superstructure stiffness is captured through the column/wall loads redistribution.
4. Assuming two structures have the same foundation, the load redistribution phenomenon is more pronounced in the case of the structure with the "stiffer" structural system (for example a shear-wall system versus a column system).
5. The load redistribution necessitated "stiffening" the foundation by increasing the basic length of the piles near the center of the raft. The longer piles are not needed for increasing the foundation bearing capacity, they are needed to alleviate the increased outer wall stresses in the superstructure.
6. Construction of the piles through the continuous gravel layer at a depth of about 50 meters from grade was not problematic.

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