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ABSTRACT

The influence of foundations on the design and behavior of tall buildings is explored by examining two built towers: Burj Khalifa, Trump International Hotel and Tower, and the partially built Plaza Rakyat, a 77 story tower in Malaysia. The paper reviews how foundation conditions were considered in the design of the buildings, and how the foundations were anticipated to influence the behavior of the towers.

INTRODUCTION

The influence of foundations on the design and behavior of tall buildings is explored by examining two built towers; the world's tallest structure, the Burj Khalifa; the second tallest tower in North America, the Trump International Hotel and Tower in Chicago; and the partially built Plaza Rakyat, a 77 story tower in Kuala Lumpur. Designed by Skidmore, Owings & Merrill LLP (SOM), each structure has a unique foundation which affects the design and behavior of the tower it supports. The paper reviews how the conditions of these foundations were considered in the design of the buildings. It also describes how the capacity and stiffness of the foundations directly influence the systems of their supported superstructures.

BURJ KHALIFA- DUBAI, UAE

At 828 meters, Burj Khalifa (Fig. 1) is the world's tallest building, eclipsing the height of its nearest peer by almost 40% (320 meters). Completed in 2010, it tops all three height categories defined by the Council on Tall Buildings and Urban Habitat. The 280,000 m^2 (3,000,000 ft^2) reinforced concrete multi-use tower primarily features a residential and office program, but also contains retail space and a Giorgio Armani Hotel.

Fig.1: Burj Khalifa Rendering

Structural System Summary

The Burj Khalifa is primarily a reinforced concrete building. The tower's structural system consists of reinforced concrete construction from foundation to Level 156; above Level 156 is predominantly the spire, consisting of a structural steel braced frame system. The tower's structural system is described as a "buttressed core", which consists of high performance concrete wall construction. Each of the wings buttresses the others via a six-sided central core or hexagonal hub. This central core provides the torsional resistance of the structure, similar to a closed pipe or axle. Corridor walls extend from the central core to near the end of each wing, terminating in thickened hammer head walls. These corridor and hammerhead walls behave similar to the webs and flanges of a beam, in order to resist the wind shears and moments. Perimeter columns and flat plate floor construction complete the system (Fig. 2). At the mechanical floors, 3-story outrigger walls link the perimeter columns to the interior wall system, allowing the perimeter columns to participate in the lateral load resistance of the structure; thereby utilizing all of the vertical concrete elements to support both gravity and lateral loads. The result is a tower that is extremely stiff both laterally and torsionally. It is also a very efficient structure, because the gravity load resisting system maximally resists lateral loads.
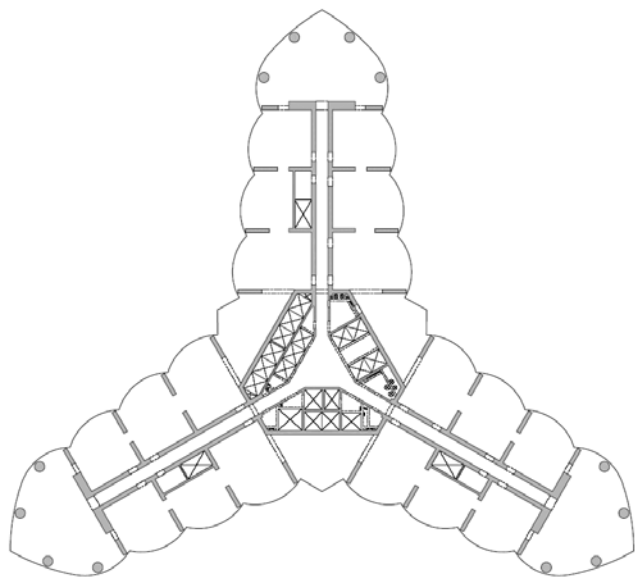

Fig. 2: Typical Floor Plan

As the building spirals in height, the wings set back to provide many different floor plates. The setbacks are organized with the tower's grid, thereby accomplishing the building stepping by aligning the columns above with walls below. In this manner, the system provides a smooth load path without any structural transfers. These setbacks also have the advantage of providing varying widths to the tower with each differing floor plate. By stepping and shaping the tower, engineers were able to "confuse the wind"; wind vortices are not able to organize because at each new tier the wind encounters a different building shape.

Wall and column concrete strengths range from C80 to C60 cube strength (11.6 ksi to 8.7 ksi cube strength), and utilize Portland cement, fly ash, and local aggregates. The C80 concrete has a maximum specified Young's Elastic Modulus of 43,800 N/mm^2 (6350 ksi) at 90 days. The actual concrete provided was equivalent to C100 concrete with an Elastic Modulus of 48,000 N/mm^2. Wall and column sizes were optimized using virtual work/LaGrange multiplier methods, yielding a very efficient structure. Wall thickness and column sizes were also fine-tuned to reduce the effects of creep and shrinkage on the tower. To reduce the effects of creep-induced differential column shortening between the perimeter columns and interior walls, the columns were sized so that the self-weight gravity stress on the perimeter columns was equal to the stress on the interior corridor walls. The outriggers at the five mechanical floors tie all the vertical load carrying elements together, further ensuring uniform gravity stress by essentially allowing the structure to redistribute gravity loads at five locations along the building's height, thereby reducing differential creep movements. With respect to concrete shrinkage, the perimeter columns and corridor walls were given matching thicknesses of 600 mm (24 in), which provided them with similar volume to surface ratios. This measure allows the columns and walls to generally shorten at the same rate due to concrete shrinkage.

Foundation Summary

The Tower foundation consists of a pile supported raft. Piles were utilized as settlement reducers. Ground conditions at the site generally range from medium dense to very loose silty sand overlying weak to moderately weak Calcarenite and very weak to weak calcareous sandstone imbedded with cemented sand. This in turn overlies gypsiferous sandstone, calcisiltite/conglomeritic calcisiltite, calcareous siltstone and a calcareous/conglomeritic strata. The ground water table is high, located approximately 2 m (6 ft) below the surface and is also very corrosive.

The geotechnical engineer of record is Hyder Consulting, Ltd. (U.K.), led by Grahame Bunce. Geotechnical peer reviews were conducted by AECOM/STS, led by Clyde Baker, and Prof. Harry Poulos of Coffey Geotechnics/The University of Sydney. A rigorous geotechnical investigation was conducted for the site by ACES (U.A.E.), and consisted of the following phases:

Phase 1: 23 Boreholes (three with pressure meter testing) with depths up to 90 m.
Phase 2: 3 Boreholes drilled with cross-hole geophysics.
Phase 3: 6 Boreholes (two with pressure meter testing) with depths up to 60 m.

Phase 4: 1 Borehole with cross-hole and down-hole geophysics; depth = 140.6 m

A 3D foundation settlement analysis was carried out by Hyder Consulting Ltd., based on the results of the geotechnical investigation and pile load test results. It was determined that the maximum settlement over time would be approximately 80 mm (3.15 in). This settlement would be a gradual curvature of the top of earth over the large site. The maximum measured settlement to this point (after completion of the concrete structure) is 46 mm (1.8 in) (Fig. 3). A hand calculation by Clyde Baker, based on a rigid block analysis, predicted a 50-60 mm settlement.

Fig. 3: Raft Monitoring Key Plan

The tower foundations were designed for two conditions: the tower raft taking 40% of the load (piles 60%), and the raft taking 10% of the load (piles 90%). The total load in the foundations (including the weight of the mat) divided by the area of the mat is 1465 kPa. The solid reinforced concrete tower raft is 3.7 meters (12.1 ft) thick and was poured utilizing 12,500 m^3 (442,000 ft^3) of C50 (7.25 ksi) cube strength self-consolidating concrete (SCC). The raft was constructed in four separate pours (three wings and the center core) (Figs. 4 and 5). Each raft pour occurred over at least a 24 hour period. Reinforcement was typically spaced at 300 mm (12 in) in the raft and arranged such that every 10th bar in each direction was omitted, resulting in a series of "pour enhancement strips" throughout the raft; the intersections of these strips created 600 mm x 600 mm (24 in x 24 in) openings at regular intervals, facilitating access and concrete placement. Due to the thickness of the tower raft, it was important to limit the peak and differential temperatures due to the heat of hydration, so as to accurately determine raft concrete mix design and placement methods. The C50 raft mix incorporated 40% fly ash and a water cement ratio of 0.34. Large scale test cubes of the concrete mix, 3.7 m on a side, were poured prior to the raft construction, in order to verify concrete placement procedures and monitor the concrete temperature performance.

Fig. 4: Tower Raft Under Construction

Fig. 5: Individual Wing of Tower Raft

The Tower raft is supported by 194 bored cast-in-place piles. The piles are 1.5 meter (4.9 ft) in diameter and approximately 43 m (141 ft) long, with a capacity of 3,000 tonnes each (pile load tested to 6000 tonnes). The diameter and length of the piles represent the largest and longest piles conventionally available in the region. The 6000 tonne pile load test also represented the largest magnitude pile load test performed to date within the region (Fig. 6); 900 mm diameter piles were also tested. The C60 (8.7 ksi) (cube strength) SCC concrete was placed by the tremie method utilizing polymer slurry. The friction piles are supported in the naturally-cemented calcisiltite/conglomeritic formations, developing an estimated ultimate pile skin friction of 250 to 350 kPa (the actual ultimate pile skin friction capacity remains unknown, as no piles failed during the test program).
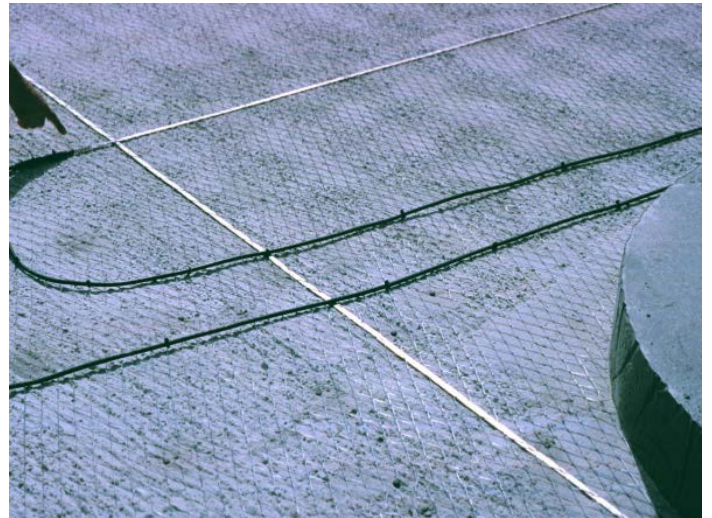
Fig. 6: Pile Load Test

Fig. 8: Cathodic Protection System

The high elevation corrosive ground water created a unique situation for the unprecedented scale of the project, as it contains approximately three times the sulfates and chlorides as sea water. As such, it was necessary to implement a rigorous program of anti-corrosion measures in order to ensure the long-term integrity of the tower's foundation system. Instituted measures include the implementation of specialized waterproofing systems, both over the entire raft and at the pile heads (Fig. 7), increased concrete cover to reinforcement, the addition of corrosion inhibitors to the concrete mix, application of stringent crack control raft design criteria, and the implementation of an impressed current cathodic protection system utilizing titanium mesh (Fig. 8). Additionally, a controlled permeability formwork liner was utilized for the tower raft. This results in higher strength/ lower permeable concrete cover to the rebar. The concrete mix for the piles was also enhanced, designed as a fully self consolidating concrete to limit the possibility of defects during construction.

TRUMP INTERNATIONAL HOTEL AND TOWER – CHICAGO, ILLINOIS, USA

Completed in 2009, Trump International Hotel & Tower, Chicago rises to a height of 353.8 m (1161 ft), (423.2 m [1388 ft] including the spire). The building is located on the north side of the Chicago River, between Wabash Avenue and Rush Street, at the site of the demolished Chicago Sun Times building (Fig. 9).

Fig. 7: Pile Head Waterproofing System

Fig. 9: Trump International Hotel and Tower, Chicago

Structural System Summary

Incorporated within the tower's 240,000 m² (2.6 million ft²) of floor space is 9300 m² (100,000 ft²) of retail space, parking for 1000 cars, 486 condo units, 339 hotel units, a health club, restaurant and ballroom. The stainless steel and glass tower rises from a landscaped plaza which includes a riverwalk that will link the upper pedestrian level with the lower level retail shops. The building features setbacks at Levels 16, 29, and 51 which correspond to the top elevations of prominent neighboring buildings, providing visual continuity with the building's surroundings. These buildings include the historic Wrigley Building on the east, Bertrand Goldberg's Marina City to the west, and Mies van der Rohe's IBM Building located directly across Wabash Avenue.

The Trump International Hotel and Tower is an all reinforced concrete building (Figs. 10 and 11) whose structural system allows for optimal floor to floor heights and utilizes flat plate gravity framing for the residential and hotel portions of the tower. Reinforced concrete promotes desirable damping behavior and high stiffness, allowing the structure to efficiently control the perception of motion during wind events.
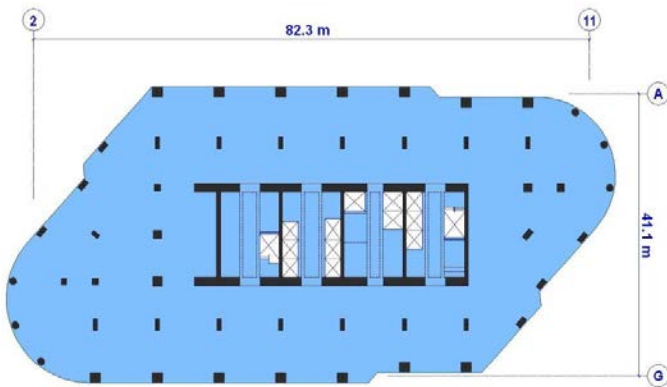
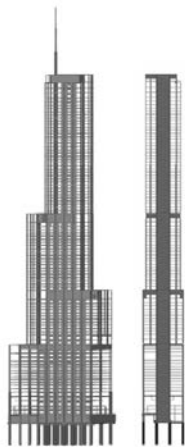

Fig. 10: Typical Residential Floor Plan

Fig. 11: Elevations: North/East

The lateral system consists of a core and outrigger system. Large outrigger elements at the mechanical levels tie the concrete core to perimeter columns, thereby engaging the perimeter columns in the building footprint. This increases the building's lateral stiffness, as well as its resistance to overturning wind moments. The core is located at the center of the building and consists of six I-shaped walls at the base, reducing to four I-shaped and one C-shaped wall after the first setback (Figure 10), with additional walls dropping off after each setback. The webs of these I- and C-sections are 460 mm (18 in) thick and 12.5 m (41 ft) long, and are oriented in the north-south direction. The flanges of the sections are 1.2 m (48 in) thick and range from 2.7 to 6.7m (9 ft to 22 ft) in length; they are oriented in the east-west direction. Flanges of adjacent core walls are connected by 1.2 m (48 in) wide by 0.8 m (30 in) deep reinforced concrete link beams.

The outrigger effect is most pronounced in the shorter direction of the building (north-south), as the width of the lateral system increases from 15 to 43 m (49 ft to 140 ft) when the perimeter building columns are engaged. The outriggers are large reinforced concrete wall-beams (up to 1.7 m [66 in] wide and 5.3 m [17'-6"] deep), which extend from the flanges of the core walls to the exterior columns at three of the tower's double-height mechanical floors (Levels 28-29, 50-51, and 90-91). These outrigger levels occur just below the building set-back levels. The outriggers also serve as transfer girders, because the columns are relocated at the façade setbacks. At the lowest building setback (Level 16) transfer girders allow for a column-free space at the ten parking levels. Perimeter belt walls at the roof and the three mechanical levels provide additional torsional stiffness and redundancy, while also serving to equalize column loads along the perimeter.

Typical residential floors are 230 mm (9 in) thick flat plates spanning to a maximum of 9.1 m (30 ft) without perimeter spandrel elements. This construction minimizes the structural depth of the floor, thereby allowing higher ceiling heights. Tower columns are typically 600 by 1200 mm (2 ft by 4 ft) rectangular sections at the top of the building and 1800 mm (6 ft) diameter circular sections at the base.

SOM specified a series of high performance concrete mixtures for the structure. Concrete cylinder strengths of 83 MPa (12,000 psi) at 90 days have been specified for all vertical column and wall elements up to Level 51. Local areas in the outrigger zones, however, require 110 MPa (16,000 psi) concrete at 90 days. 35 MPa (5000 psi) concrete was specified for the typical gravity framing. The benefits of the utilization of high strength concrete were twofold. Namely, the high strength concrete limited the size of the vertical load resisting elements, which in turn controlled the weight of the building and resulted in residential units with smaller vertical obstructions. Secondly, the inherent increase in the modulus of elasticity of high strength concrete allowed designers to utilize the overall stiffness of the structure to control building accelerations and occupant perception of wind events, without adding supplemental damping measures. Actual static

modulus of elasticity testing for the 83 MPa (12,000 psi) concrete yielded an average modulus of approximately 45.5 GPa (6600 ksi) at 90 days.

Foundation Summary

Trump International Hotel and Tower's subsurface conditions are typical for downtown Chicago properties and consist of upper layers of urban fill and soft to medium layers of silty clays, which are followed by extremely dense clay-like silt (referred to as hardpan). Below the hardpan were layers of very dense silty sand and very dense to extremely dense silt atop Dolomite Limestone Bedrock. Bedrock formations on the site were located approximately 34 m (110 ft) below the existing ground level. For these geotechnical conditions, buildings of moderate loads are typically founded on belled caisson in the hardpan strata, while heavily loaded high-rise structures utilize rock-socketed caisson construction.

Because of the tremendous weight of the tower, the structural design of Trump Tower was heavily influenced by its geotechnical conditions and made possible through a close collaboration of the structural design team at SOM and the geotechnical engineers at AECOM/STS Consultants, led by Clyde Baker. Based upon the recommendations of AECOM/STS, the design team selected hardpan supported belled caisson for the low rise and retail portions of the project (with allowable bearing pressures of 1,720 kPa [36 ksf]), while the tower was supported by rock-socketed reinforced concrete caissons, with permanent steel casings. When rock caisson construction is typically used in Chicago, the caisson is drilled to the top of rock and a permanent steel casing is then inserted and drilled into it, thereby attempting to seal the caisson shaft from groundwater inflow. The steel casing is then screwed into the rock, and sealed with grout. Once the casing is set, the bottom of the caisson is drilled or cored until the rock socket has reached its specified length. For Trump Tower, the specified minimum rock socket length was 1800 mm (6 ft). (It is important to note that the rock socket depth must be measured from the top of sound rock.) As the bedrock is typically overlain by a layer of weathered limestone and/or broken rock and gravel, weak zones may exist below the intended bearing elevations. Chicago Building Code (CBC) requires pre-construction rock probes to be drilled at each rock caisson location to determine the top of sound rock elevation for every caisson location.

The Chicago Building Code allows bearing pressures of up to 9,575 kPa (100 tsf) for caissons penetrating a minimum of 300 mm (1 ft) into the solid bedrock. For each additional 300 mm (1 ft) of rock socket depth the code allows an additional capacity of 1,915 kPa (20 tsf), up to a maximum of 19,150 kPa (200 tsf) for a 1800 mm (6 ft) rock socket depth. Although the geotechnical bearing capacities of the rock-socketed caisson allowed for the construction of a very tall, reinforced concrete structure, the code limited the maximum bearing pressures to 19,150 kPa (200 tsf). Current constructability constraints

resulted in a maximum caisson diameter of 3 m (10 ft), thereby limiting the total height of the building.

The highly loaded core of the building is supported by a 3 m (10 ft) thick reinforced concrete mat (Fig. 12) that transfers the enormous core load into twenty-four 3 m (10 ft) diameter rock caissons. The perimeter and interior columns are supported by 33 individual rock caissons up to 2.4 m (8 ft) in diameter, linked together by a series of caisson caps and grade beams. The capacity of the core is largely controlled by the number of maximum diameter (3 m [10 ft]) caissons that can be located within the footprint of the highly loaded core flanges. The location and density of core caissons was further complicated by the location of the shafts of the existing belled caissons that supported the printing presses of the demolished Chicago Sun-Times building, which formerly occupied the site.

Fig. 12: Mat Foundation Construction

In order to increase the bearing capacity of the caissons and allow a higher, heavier building, AECOM/STS proposed the use of the Osterberg Load Test. The Osterberg Load Test had been utilized on a limited basis at other sites in the City of Chicago, resulting in the permission of bearing pressures up to 22,000 kPa (230 tsf)-- 15% higher than the code allowed at 19,150 kPa (200 tsf). Based upon their geotechnical evaluation of the site conditions, AECOM/STS estimated a possible bearing capacity of 24,000 kPa (250 tsf), subject to performing a successful Osterberg Load Test. SOM and AECOM/STS specified that the caisson contractor perform an Osterberg Load test on the first production caisson to verify a minimum bearing capacity of 22,000 kPa (230 tsf).

The Osterberg Load Test (commonly known as the O-cell) was invented by Professor Jorj O. Osterberg of Northwestern University, and is utilized for testing deep foundation systems such as drilled shafts and piles. A production caisson is installed using a hydraulically driven, sacrificial loading jack located at the bottom of the caisson. The O-cell test requires no reaction frame, as the reaction is provided by the soil and rock working downward against end bearing and upward

against side-shear.

In April 2005, a successful O-cell test was performed on a production rock caisson. Based upon the results of the O-cell test, the City of Chicago granted a code variance for an allowable bearing pressure of 25,900 kPa (270 tsf), the highest allowable bearing pressure in the City of Chicago at that time. The increased bearing pressure, coupled with the utilization of 69 MPa (10,000 psi) compressive strength (cylinder) concrete in the caissons, allowed for the design of Trump Tower to reach a height of 92 stories above grade (100 framed levels when including mezzanines and basements). Because of the accessible dense rock formations which support Trump Tower, the foundation system is largely controlled by strength and constructability constraints and significant foundation settlement is not concern. Foundation settlement surveys performed during construction activities indicated total settlements in the order of only 6 mm (1/4"), as compared to the AECOM/STS predictions of 12 to 19 mm (1/2 to ¾").

PLAZA RAKYAT- KUALA LUMPUR, MALAYSIA

Plaza Rakyat is a partially built 613,160 sm (6.6 million sf) mixed-use development and transportation center amidst the Jalan Pudu, Kuala Lumpur's busiest thoroughfare (Fig. 13). The project was intended to be anchored by a 77-story office tower; however, construction stalled during the 1997 Asian Financial Crisis and has not yet resumed. Located on a large triangular site which had previously been swampland, the project's geotechnical conditions and below-grade transportation infrastructure provided engineers with formidable foundation challenges. The 77-story office tower would have been, at time of completion, the tallest all-reinforced concrete building in the world and also one of the most slender with an overall aspect ratio of over 8 to 1. The structural engineering design for the tower continues a long tradition of systems development for tall buildings which attempts to refine and improve the efficiency and economy of the structure. A powerful new system, the belt wall / core interacting system, is introduced, which is applicable to very tall buildings in low to moderate wind climates and to buildings in the mid-height range in moderate to high wind climates. The system developed was based on improving the economy of the structure in response to local environmental and constructional conditions.

Structural System Summary

The structure for the 77-story Plaza Rakyat office tower is formulated based on the desire for economy through simplicity and repetition. The structure is framed entirely in reinforced concrete due primarily to the predominant use of the material over structural steel in Malaysia. A typical floor framing plan is shown in Figure 14. Gravity loads are collected on the exterior perimeter by large rectangular columns spaced 9.0 meters on center. For the lower half of the building, these columns have a

constant face dimension parallel to the plane of the exterior wall of 1200mm and vary in depth along the height of the building from 2700mm at the base to 1500mm at the Level 51 transfer. Column size transitions are made on the inside face of the column only in order to simplify formwork and detailing with respect to the exterior wall. The column sizes above Level 53 are reduced to 800mm square in order to provide less encumbrance to sightlines for the upper level office spaces.

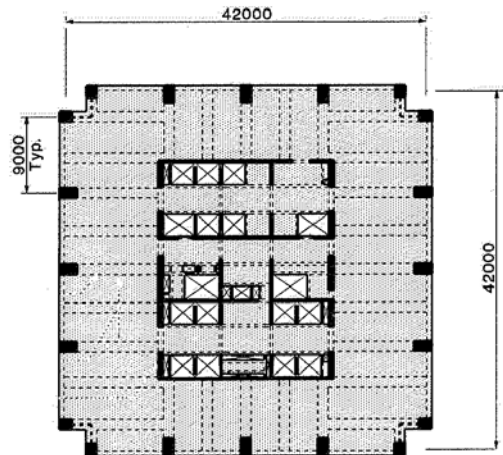*Fig. 13: Plaza Rakyat Office Tower, Kuala Lumpur, Malaysia*

Fig.14: Typical Floor Framing Plan

The concrete strengths utilized are quite low (C50 and C40 grades) for a building of this height in order to avoid the introduction of foreign concrete technologies. The design for the exterior columns was controlled primarily by considerations of strength under gravity loads alone. The remainder of the gravity load is supported by the rectangular core which is organized around the central elevator, stair and services areas. Vertical walls on the perimeter of the core vary in thickness from 850mm at the base to 450mm at the roof. Internal web walls in each direction, of constant 300mm thickness, frame the various elevator banks in the core. Besides designing the vertical load-carrying elements for strength under gravity loads, the design was developed from a standpoint of trying to equalize the working stresses of the core walls and exterior columns in order to reduce the effects of differential vertical shortening which can result in some out-of-levelness in the floors of very tall concrete buildings.

Between Levels 51 and 53, the building exterior steps inward approximately 3.0 meters. All exterior columns are transferred through two-story high reinforced concrete shear panels which avoid the necessity of deep transfer girders at Level 51. The transfer is accomplished through the strength and rigidity of the floor slabs in compression at Level 53 and tension at Level 51.

The floor framing system for the tower was chosen based on value-engineering analysis between the architects, engineers, and the contractor to determine the system which would produce the least overall building cost - not necessarily the least structural cost. For this reason, a wide, shallow beam system was utilized with the beam width (1200mm) set to match the width of the exterior column, (an economical arrangement for the formwork system) and a beam depth of 500mm. See Figure 14. The beams are conventionally reinforced and are spaced 4.5m on center spanning between the exterior frame and the core wall. A continuous 600mm wide by 800mm deep spandrel beam connects the exterior columns on the building perimeter and also supports every other floor beam on the 4.5m module as well as the architectural curtain wall. While not as structurally efficient as a deeper beam section, the shallow beams serve to reduce the typical story height to 3.9m resulting in economies in the exterior curtain wall, elevatoring, interior partitioning, and vertical plumbing and mechanical riser costs. In addition, the wide beams shorten the effective one-way slab span transversely between the beams resulting in a typical office floor slab thickness of 120mm which also meets the applicable 1.5-hour fire resistance requirement between floors. On mechanical floors, due to heavier imposed loads, thicker slabs are specified with the corresponding beam depths adjusted such that the stem (portion of the beam below the slab soffit) remains unchanged. This allows for an extremely simple formwork system which may be repeatedly reused. Although the beam spans are significantly shorter above the Level 51 transfer zone,, the same beam profile is once again used as a logical extension of the system below.

Foundation Summary

The tower is supported below basement Level B6 on a pile supported 3.5m thick reinforced concrete mat foundation. All piles are 900mm diameter slurry piles varying in length from 22m on the exterior to 32m in the core area into the underlying Kenny Hill formation strata and are spaced at 2.7m on center. The Kenny Hill Formation is a sequence of weathered, clastic sedimentary rocks consisting of interbedded shales, mudstones, siltstones and sandstones which extend over a significant part of Kuala Lumpur City. While in Kuala Lumpur it is common to found significant high-rise buildings on limestone bedrock using bored piles or barrettes, for this project, it was decided to found the tower in the overlying Kenny Hill Formation using friction piles, provided settlement criteria were satisfied. Figure 15 shows the plan layout of the piles and mat foundation beneath the tower. Each pile has a working load capacity of 670 or 730 metric tons depending on their length in combined end bearing and shaft friction. Deeper piles were specified below the central core area based on the higher vertical loads in the center of the building in comparison with the exterior. The longer piles serve to reduce the "dishing" settlement effect due to the higher unit stress on the underlying soil directly below the core. The design criteria established a limit on long-term differential settlement between the core and the perimeter of 15mm. Overall settlements are predicted to be on the order of 50mm. With the assistance of Woodward-Clyde International, a piled mat foundation analysis was carried out based upon a finite element model utilizing soil springs to simulate the deformation characteristics of the underlying soil mass. The process involved adjusting the soil spring stiffnesses incrementally in order to simulate the settlement behavior predicted by the geotechnical settlement analysis. Once the overall and differential settlement profiles had essentially converged to the magnitudes and profiles predicted by the geotechnical engineer's soil settlement analysis, the mat foundation reinforcement was determined for the resulting shears and bending moments from the analysis model (Fig. 16).
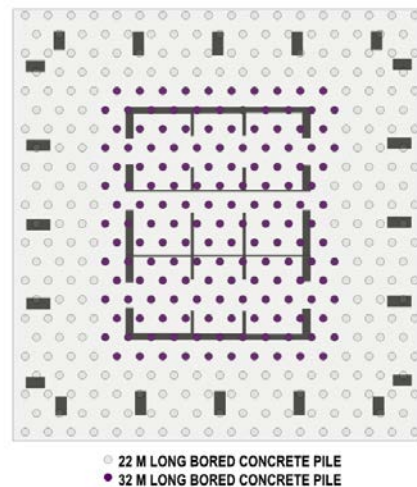

○ 22 M LONG BORED CONCRETE PILE
● 32 M LONG BORED CONCRETE PILE
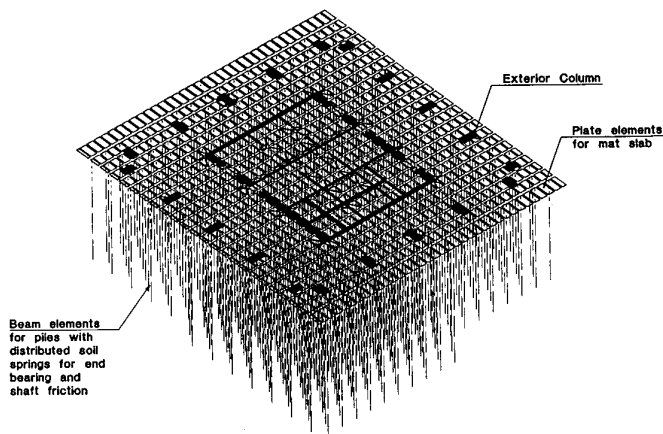
Fig. 15: Foundation Plan

Fig. 16: Mat Foundation Analysis Model

A particular concern in thick mat foundation design is the potential for high internal temperatures during concrete curing. Many mitigation methods to control the concrete temperature in mass concrete pours have been proposed with varying results, They include: cooling of water and aggregates, chilled water pipes embedded into the concrete mat, insulation of the concrete after the pour and adding ice to the concrete mix design. The design of the piled mat of the Plaza Rakyat office tower utilized relatively low strength concrete in order to reduce the amount of cement required in the mix design, consequently reducing the heat of hydration during the concrete curing process. It also featured insulation of all exposed surfaces of the mat pour and used a tent constructed over the entire mat pour area to limit the thermal gain within the mat concrete. The design was predicated upon limiting the maximum temperature differential between any two areas in the mat at any time to 25 degrees Celsius. A thermodynamic heat transfer analysis was performed in order to predict the concrete temperatures through the thickness of the mat vs. time and the amount of insulation required to limit the temperatures to the prescribed values (Fig. 17).
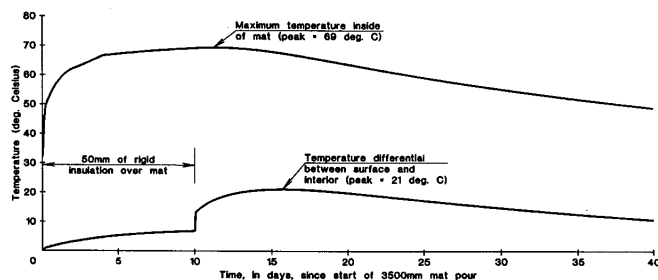

Fig. 17: Mat Foundation Concrete Temperature Analysis

To confirm the efficacy of the mix design and insulation methods, a 3.5 meter mat test cube was constructed and instrumented with thermocouples to measure the differential heat gain between the center and extreme surfaces of the mat prior to the actual mat construction. The temperatures measured for the test cube, as well as temperatures measured within the in-place tower mat were less than the prescribed limits set on overall and differential thermal gain. The tower mat was constructed in two separate continuous pours each of approximately 40 hours duration.

While construction of the Plaza Rakyat Office Tower was stopped in 1997 due to the Asian Financial Crisis, the tower piled-mat foundation and several levels of reinforced concrete below grade were completed prior to work being stopped (Fig. 18). Despite the project not being completed, there were many important lessons learned about deep foundation construction in the highly variable soils common in the Kuala Lumpur City area.

Fig. 18: Plaza Rakyat Office Tower Below-Grade Construction

CONCLUSION

For Burj Khalifa, Trump International Hotel and Tower and Plaza Rakyat, their foundation systems provide the interface between the man-made structure above and the natural geotechnical soil strata below grade. Understanding the relationship between the two is critical for the successful design of supertall building structures. In the case of the Burj Khalifa, the pile supported mat provided the required strength and settlement control which enabled the construction of the world's tallest building. For Trump Tower, the strength and constructability of the caissons were the controlling elements; bedrock strata provided outstanding settlement results and allowed for a very heavy reinforced concrete tower. For Plaza Rakyat Office Tower, the normally highly variable strata of the Kenny Hill Formation was determined to be consistent enough, over this project site, to enable the more economical drilled friction-type piles to be utilized, in lieu of bored piles to bedrock.

REFERENCES

Baker, W.F., Brown, C. and R. Sinn [2007]. "Structural Analysis and Design of the World's Tallest Reinforced Concrete Building." *International Conference on Multi-Purpose High-Rise Towers and Tall Buildings*.

Baker, W.F., Korista, D.S., Novak, L.C., Pawlikowski, J.J. and B.S. Young [2007]. "Creep & Shrinkage and the Design of Supertall Buildings- A Case Study: The Burj Dubai Tower", *ACI SP-246: Structural Implications of Shrinkage and Creep of Concrete.*

Baker, W.F., Korista, S., Sinn, R., Pennings, K. and D. Rankin [2006]. "Trump International Hotel and Tower." *Concrete International,* July 2006, pp. 28-32.

Baker W.F., Pawlikowski, J.J. and B.S. Young [2007]. "The Challenges in Designing the World's Tallest Structure: The Burj Dubai Tower." *ASCE/SEI Structures Congress.*

"Burj Dubai Development, Dubai, UAE: Geotechnical Assessment Report." [2003]. Hyder Construction, Ltd., UK.

"Prairie Sets SCC Record in the Heart of the City." [2005]. *Concrete Products,* pp. 6.

"Subsurface Exploration and Geotechincal Engineering Analysis and Report for the Trump International Hotel & Tower, 401 N. Wabash Avenue, Chicago, IL" [2004]. STS Consultants, Ltd.