

A Retrospection on Foundation Design of Taipei 101

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SYNOPSIS : Large diameter bored pile was selected as the foundation type for Taipei 101. The pile construction method and specific construction procedures were determined based on the results of trial installation and pile load tests. The baseline for foundation design was established using the friction versus depth characteristics of each ground layer obtained from the pile load tests. As the ground profile and depth to the top of rock formation varied significantly on this site, the pile length, bearing capacity and settlement for single pile were analyzed using the information interpreted from adjacent boreholes. The post grouting at pile tip was mandatory for pile construction. Nevertheless, it was treated as a measure reducing the influence of construction uncertainties and providing extra safety for the foundation system.

Keywords : Taipei 101, bored pile, full casing method, reverse circulation method, pile load test, Taipei Fault.

1. Introduction

Taipei International Finance Center, also known as Taipei 101, is located in Xinyi District, Taipei, Taiwan, and currently the world's tallest completed skyscraper. The main tower of Taipei 101, soaring more than 508 meters above street-level, contains 101 floors above ground and five underground levels. A six-story L-shaped podium building is linked to the main tower on its north and east sides. Planning for the Taipei International Finance Center began in 1996. The construction started in 1999 and completed in 2004. Figure 1 shows the general plan and elevation of Taipei 101.

Taipei International Finance Center was the first ever build-operating-transfer (BOT) project in Taipei. In 1997, the Taipei Finance Center (TFC) Corp. received a 70-year concession of the site from the Taipei Municipal Government. The TFC Corp as the project owner utilized the strategy hiring consultants to provide project management, designs, contract documents and construction management and contractors to implement detailed design and construction. As commissioned by the TFC, the partnership between the manager of project management Tuner Steiner International, the architect C.Y. Lee and Partners, the structural engineer Thornton Tomasetti and the Geotechnical consultants Sino Geotechnology Inc was formed. The contractor of the civil works was KTRT, a joint venture between Kumagai, Taiwan Kumagai, RSEA, and Ta-Yo-Wei Engineering Corps. Moh and Associates, Inc. (MAA) was commissioned by the project owner as a special consultant for supplemental geotechnical investigations, design optimization for foundation system and deep excavation. This paper is aimed at providing a review on some major design considerations of Taipei 101.

As shown in Figure 1, the Taipei 101 complex, including the main tower and the podium building, is approximately 158m x 160m, within which the footprint of the main tower is 87m x 98m. The excavation depths for the main tower and podium building are 21.8m and 21.65m, respectively. The foundation mat and basement structures for the main tower and podium building are integrated with each other while the above ground structures are separated by expansion joints at grade. The structure of the main tower consists of 16 Super Steel Reinforced Concrete columns, 4 on each side.

The loads transferred through each of the mega-columns to the foundation are greater than 100MN while the total loads applied on the foundation are greater than 4,000MN. Based on the consideration of geological characteristics of the site, experience and workmanship of local contractor, equipment availability, construction cost and demanding schedule, large diameter cast-in-place bored pile was selected as the foundation type (Chen, 2000). Retrospections of the decision making for foundation type selection, pile load tests, construction method selection, foundation design, and construction procedures modification for this project are presented in this paper.

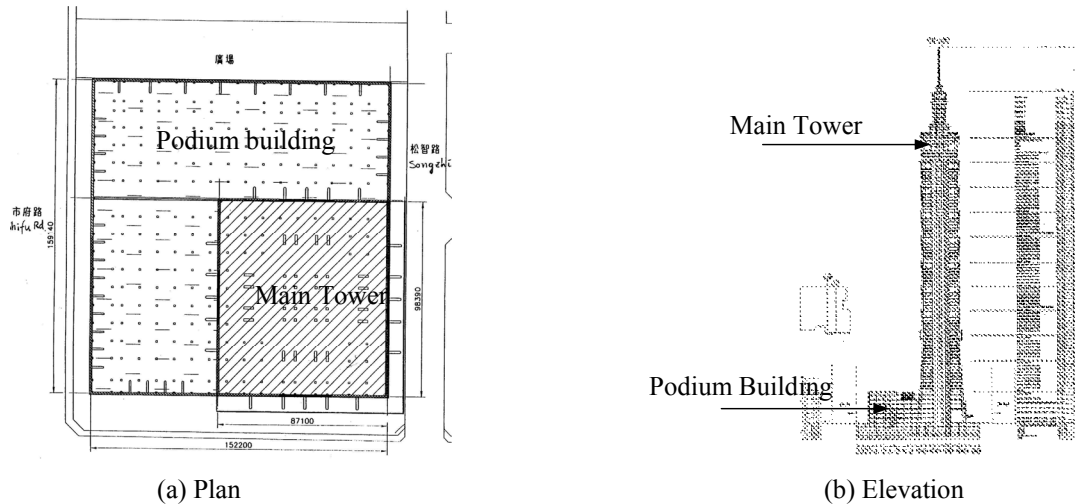


Figure 1. Plan and elevation profile of Taipei 101

2. Geological condition

Taipei 101 is located on southeast boundary of Taipei basin that was formed by the settlement of nappies between thrusts in the foothill range of north Taiwan during Pliocene and Pleistocene. The ground strata of the Basin are composed of Quaternary sedimentary deposits overlying Tertiary bedrock formation. As revealed in literatures, the Taipei Fault, known as a reverse fault, runs down the region close to the site of Taipei 101. However, the exact location and activity of this fault remained uncertain prior to planning and design of Taipei 101. According to investigations conducted by various organizations or researchers (e.g. TCRI, 1995; Sinotech, 1998; Lin and Lee, 1998), the fault runs through two possible lines as shown in Figure 2. One crosses along the Xinyi Road adjacent to the site on its south side and the other crosses the region near the site in northeast-southwest direction. The Taipei Fault was listed as a suspected active fault in the 1998 published geological zoning map (Central Geological Survey, Ministry of Economic Affairs, MOEA 1998). Both the exact location and activity of the Taipei Fault were critical if the Taipei 101 was to be built on this site. To ensure whether the site was appropriate for super highrise building, data of earlier investigations were reviewed and supplemental investigations including borehole explorations, foraminifera fossil analyses and radiocarbon dating were conducted (Chen and Huang, 2000). Results of these investigations showed the Taipei Fault runs down the region about 200m away from southeast corner of the site, and the Fault has not been active for a minimum of 45,000 year and thus can be considered non-activity (Chen, 2001).

To obtain necessary information for foundation type selection, foundation design and construction, soil investigations utilizing borehole exploration and field and laboratory tests were conducted during the planning and design phases. A total of 155 boreholes with a total length of 11,172m were drilled in five stages within which the locations of boreholes drilled after the second stage are shown in Figure 3. The general soil and rock profiles referred from Chen (2000) are shown in Figures 4 and 5, respectively. Based on the results of soil investigations, the ground consists of the strata: (1) the lacustrine soft to stiff silty clay with average thickness of 30m in which the top 20m is much softer than the bottom 10m, (2) the colluvium/alluvial soil with thickness varied from 19m to 28m, and (3) the Pliocene Kuechulin Formation which is an older sedimentary formation mainly composed of interlayered sandstone and shale. The Kuechulin Formation layer, with the depths varying from 42m to 69m, was taken to be the bedrock of this site (Chen 2001). As

located in the southeast boundary of Taipei Basin, the ground composition is different from those typically found in which the Sungshan Formation is composed of 6 interlayered silty sand and silty clay overlying the Chingmei Formation (Chin et al., 2007). Results of geological investigations showed the upper 10m bedrock is weak and poor cemented (MAA, 1998c). Simplified ground profile and engineering properties of the soils for preliminary foundation evaluation were given in Table 2 (MAA, 1998a). The groundwater table was at 1.5-2.0m below ground surface whereas at depths of the colluvium/alluvium layers the groundwater pressures were 60-80 kN/m² smaller than the hydrostatic pressures. This was a result of mass groundwater pumping between 1960s and mid-1970s. As the maximum groundwater drawdown was as great as 40m below grade at that time, the groundwater head has not been fully recovered since the prohibition of pumping in 1975.

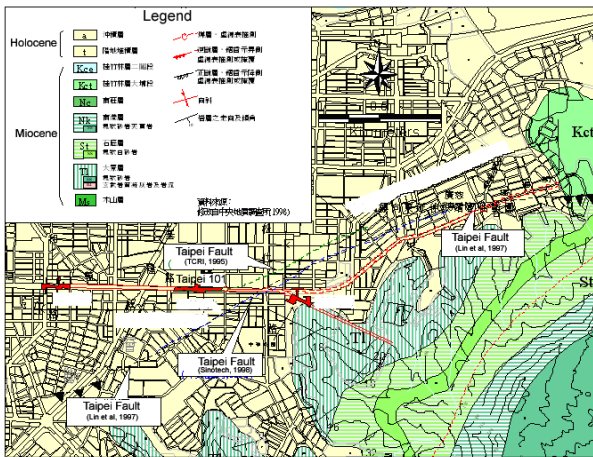


Figure 2. Possible locations of Taipei Fault

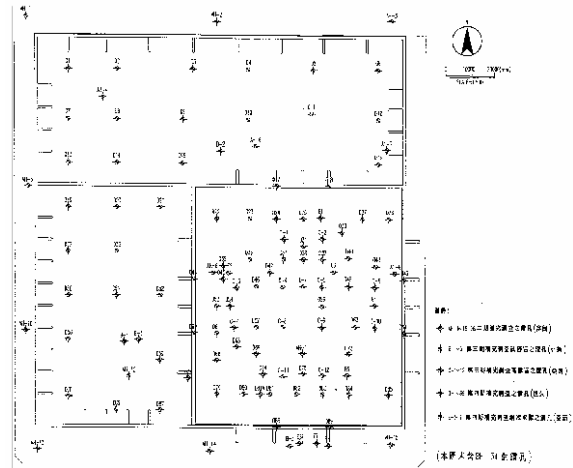


Figure 3. Layout of the boreholes

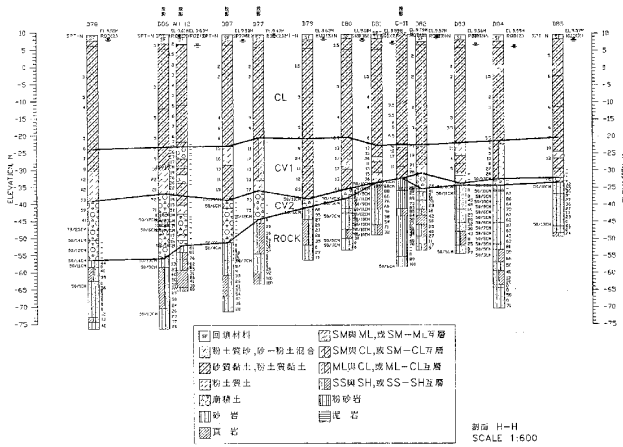


Figure 4. Ground profile

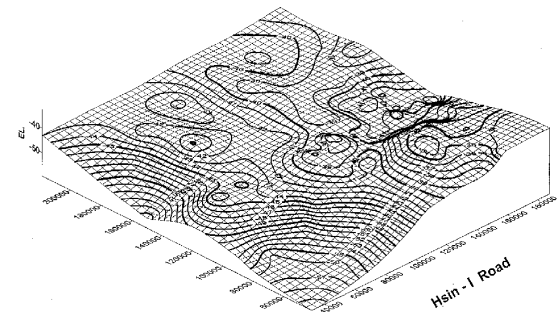


Figure 5. Contour of top of rock formation

Table 2. The simplified soil profile

Layer	Description	Depth (m)	Average Thickness (m)	γ_t (kN/m ³)	SPT-N (mean)	Effective cohesion c' (kN/m ²)	Effective friction angle ϕ' (degree)	Undrained shear strength s_u (kN/m ²)	
1	Backfill	1.2-3.2	2.0	17.5	1-12 (6)	-	-	-	
2	Silty Clay	Upper	10.0-16.0	10.0	18.0	1-3 (2)	5	25	35
3		Middle	18.0-24.0	10.0	18.0	2-6 (4)	5	28	40
4		Lower	30.5-33.9	10.0	18.0	4-21 (7)	10	30	60
5	Silty Sand/Sandy Silt	32.3-41.8	5.5	18.0	9-59 (18)	5	32	-	
6	Silty Clay/Clayey Silt	37.8-43.4	4.5	19.5	9-27 (15)	20	30	110	
7	Silty Sand, Gravel, Silty Clay, Rock	41.6-69.0	10.0		15- >100 (33)	0	35	-	
8	Rock	Poor cemented	51.6-79.0	10.0		>50 - 100	100	40	-
9		Well cemented	-	-	-	>50 - 100	100	45	-

3. Selection of foundation type

For supporting the huge loads transferred to the foundation through the structure components of the main tower, three types of foundations, large diameter caisson, barrette, and large diameter bored piles were evaluated (Xiao et al, 2001). The layouts of the preliminarily suggested foundation types for the main tower and podium building are shown in Figure 6. Comparisons made in the aspect of constructability (MAA, 1998a) are given in Table 3.

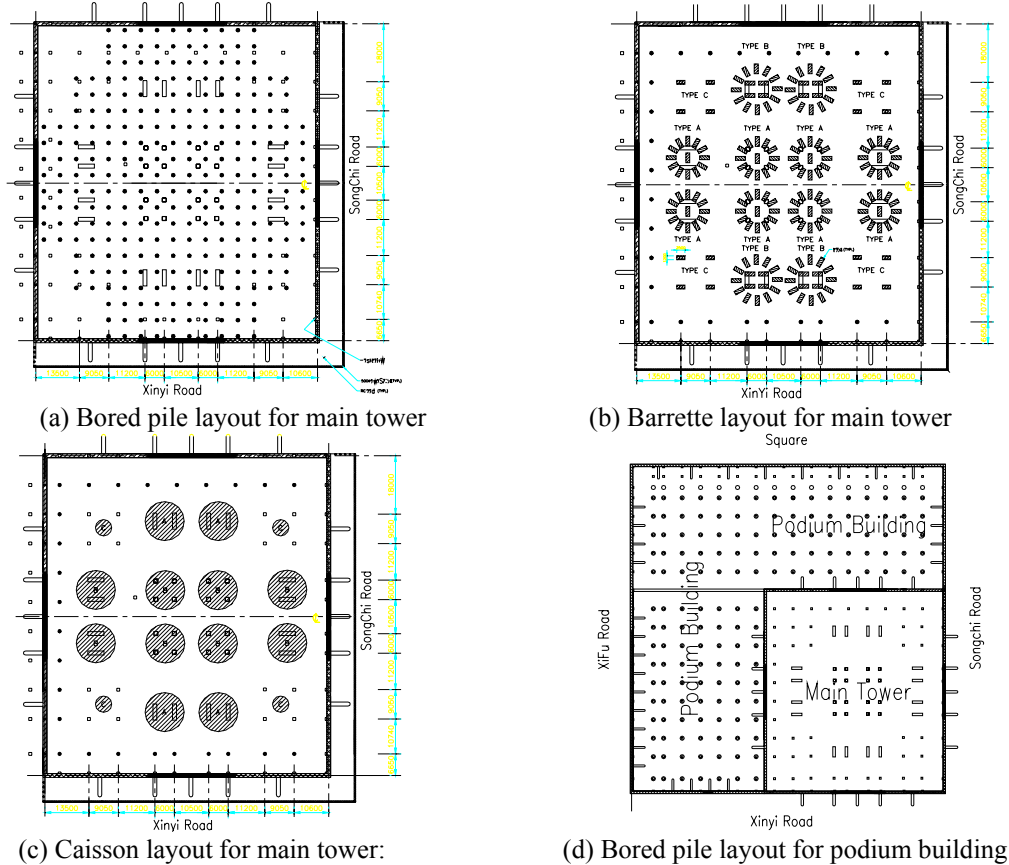


Figure 6. Preliminary suggested foundation type and layout

The caisson type foundation was soon precluded because of the hint of artesian or sub-artesian conditions and existence of the relatively soft bearing stratum at the site. As the huge loads transferred from the highrise structure to the foundation composed of a limited number of caissons bearing on relatively soft bearing stratum, the risk of the uneven settlements detrimental to the structure was unacceptable. Artesian or sub-artesian condition can result in groundwater ingress during construction. Although the risk associated with this can be reduced by ground improvement, it is not an economic measure considering the scale of the site.

According to the consultants of Moh and Associates, Inc (Jamiolkowski and Poulos, 1998), the barrette option was attractive from a technical viewpoint because it provided a high resistance to both vertical and horizontal loadings. If this option was adopted, it would be essential to excavate the first 30-35m with a Kelly grab, and then the remainder with a hydromill. The Kelly grab could be provided with an inclinometer sensor to assess verticality. The bored pile option was also viable, and the preference would be the larger 2.0m diameter piles because of the large length involved. Proper cleaning of the base would be essential, and would preferably involve the use of strictly specified procedures for the use of base mud cleaning. If considered desirable, the stiffness of the base of each bored pile can be improved by use of a special grout method. Both of the foundation types were suggested to have appropriate embedment into the bearing stratum at least 15 to 20 m.

Taking into account the demanding schedule of this BOT project, workmanship and experiences of local contractors and availability in construction equipment played critical roles in the decision making process. Large diameter bored piles were eventually selected as the foundation type for this project. (Chen, 2004).

Table 3. Comparison of bored piles, barrette and caisson

Foundation Type	Advantage	Disadvantage
Bored piles (2m or 1.5m diameter)	<ol style="list-style-type: none"> 1. Engineering economy can be achieved by using long piles 2. Pile length can be changed easily to compensate for change in soil/rock condition. 3. Suit for resisting events that produce extreme large lateral loads (e.g. earthquake and vessel impact loading) 4. Low vibration and noise during construction 5. High vertical precision (full casing method) 6. Local contractors with good experience and workmanship 7. Easy to obtain construction equipment from local supplier 	<ol style="list-style-type: none"> 1. Difficulty in cleaning tip sediments 2. Contractor's workmanship affects the quality and performance significantly. 3. Difficulty in excavation if ground is composed of cobbles or gravels (reverse circulation method)
Barrette (Typically 1.2m x 3m or 1.2m x 2.5m)	<ol style="list-style-type: none"> 1. Suit for resisting events that produce extreme large lateral loads (e.g. earthquake and vessel impact loading) 2. Low vibration and noise during construction 3. High vertical precision 4. Provide high horizontal and vertical resistance 	<ol style="list-style-type: none"> 1. Difficulty in cleaning base sediments 2. Insufficient local experience
Caisson (Typically 10.5m diameter)	<ol style="list-style-type: none"> 1. Easy to clean base sediments 2. Suit for resisting events that produce extreme large lateral loads (e.g. earthquake and vessel impact loading) 3. Provide high horizontal and vertical resistance 4. The bearing capacity provide by each of the caissons is equivalent to a great number of bored piles or barrette 5. The location of bearing stratum can be verified reliably by allowing crew to inspect soil/rock conditions visually. 	<ol style="list-style-type: none"> 1. Construction is easy to be affected by weather conditions 2. Difficulty in construction if excavation depth is large 3. Permeation grout/ ground improvement may be a necessary measure if artesian presents 4. Insufficient local experience 5. Difficulty in obtaining appropriate construction equipment from local supplier

4. Pile load tests

The foundation construction for Taipei 101 is unprecedented. The building is especially important and sensitive to settlement. To evaluate the construction feasibility, end bearing and friction resistance, and effectiveness of post grouting, trial installation and pile load test programs were developed. Results of the evaluations were utilized to select construction method, modify construction procedure, and served as a baseline for foundation design

4.1 Test configuration

As shown in Figure 7, a total of 20 piles including 5 for compression tests and 3 for pull out tests were installed. The diameter of these test piles was 1.2m. Among the compression test piles, three of which, TPC1 to TPC3 were installed by reverse circulation method with planned rock socket depths of 10m, 20m and 15m, respectively, and two of which, TPC4 and TPC5, were installed by full casing method with planned rock socket depths of 20m and 15m, respectively. The piles for pulls out tests, TPT1 to TPT3, were installed by reverse circulation method. The 1.2m, 2.0m and 2.8m diameter anchor piles were also installed by reverse circulation method. The construction of the test and anchor piles was also treated as trial installation. Table 4 shows the summarized information of these test piles (MAA, 1998b). The reference boring logs are shown in Figure 8.

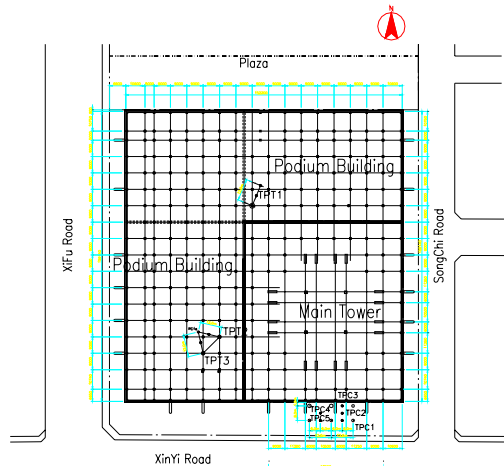


Figure 7. Layout of test

Table 4. Summary of the information of test piles

Pile no	Type of test	Construction method	Ground elevation (m)	Depth to rock head (m)	Rock Socket length (m)	Pile length (m)	Pile diameter (m)	Post grouting		Concrete strength (MN/m ²)	Max. Applied Load (MN)	Ref. Borelog
								perimeter	bottom			
TPC1	Compression	Reverse circulation method	9.47	42	13	55.5	1.2	Yes	Yes	49	27.0	B-1
TPC2			9.47	42	21	63.5	1.2	Yes	Yes	49	38.0	B-1
TPC3			9.47	42	16	58.5	1.2	No	No	49	33.0	B-1
TPC4		Full casing method	9.47	48.6	18.4	67.5	1.2	Yes	Yes	49	23.9	B-3
TPC5			9.47	48.6	15.9	65	1.2	Yes	Yes	49	27.0	B-3
TPT1	Pull-out	Reverse circulation method	9.44	51.2	11.1	62.3	1.2	Yes	No	28	29.8	B-2
TPT2			9.88	54.8	8.9	63.7	1.2	Yes	No	28	19.4	B-5
TPT3			9.81	54.8	4.7	59.5	1.2	No	No	28	16.5	B-5

Note: Piles for compression test were installed 0.5m above ground level

Perimeter grouting was only conducted on socket length

Depth to rock was inferred based on the information provide from reference borehole log

Maximum applied load is measured from load cell

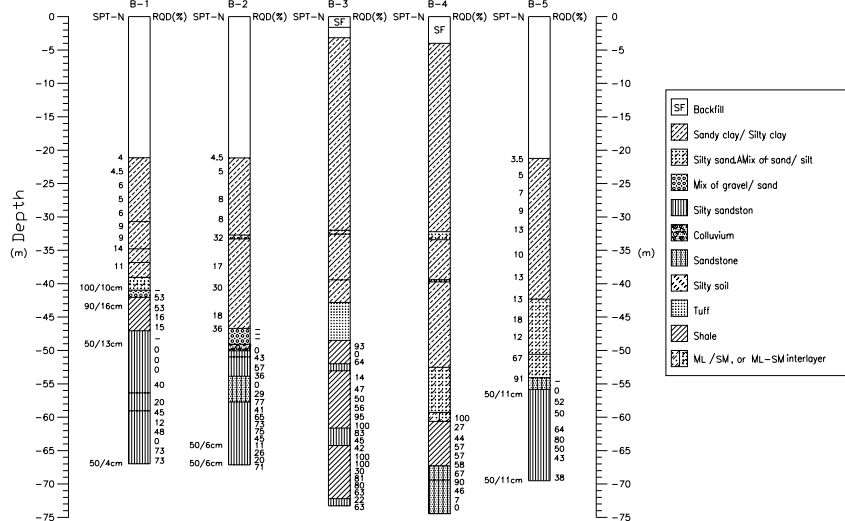


Figure 8. The boring log of B-1, B-2, B-3, B-4 and B-5

Post grouting was conducted for selected test piles at the tip or along rock socket to evaluate the effectiveness of the grouting methods. Because the top of the working pile was essentially located at the final excavation elevation rather than the ground surface, double casing as schematically illustrated in Figure 9 was utilized to minimize the friction along the shaft above final excavation elevation for load tests.

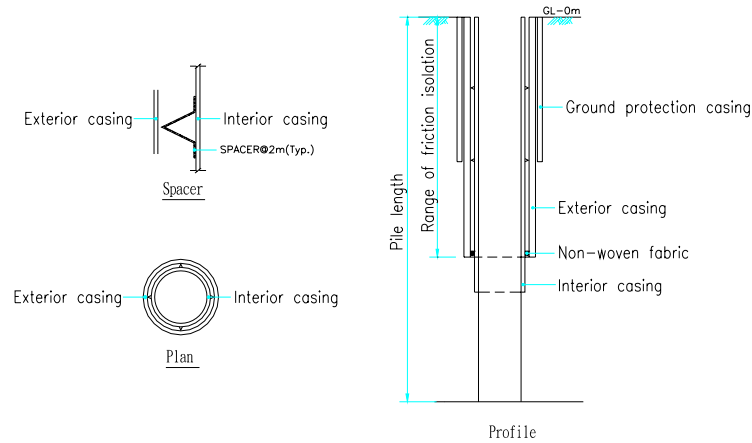


Figure 9. Schematic of double casing

Quality assurance was performed by ultrasonic inspection, cross-hole sonic logging test and coring. The ultrasonic inspection was conducted after the completion of pile drilling to examine the dimension along the depths of the drilled hole. The cross-hole sonic logging method was applied to evaluate the integrity of the test piles both before and after load tests. For each of the test piles, a 4-inch diameter Galvanized Iron Pipe (GIP) was installed to the depth close to pile tip before concrete pouring. After pile load tests, samples beneath the bottom of test piles were taken by coring for quality evaluation (MAA, 1998b).

4.2 Test results

Figure 10 shows the results of the pile load tests in terms of load versus displacement measured at pile head (Chen, 2000). Various techniques including Davisson's method (1973), Fuller and Hoy method (1970), Brinch Hansen method (1963), Terzaghi method (1942) were utilized to estimate the bearing capacity for the test piles. Results using Davisson's method (MAA, 1998b) are demonstrated in Table 6.

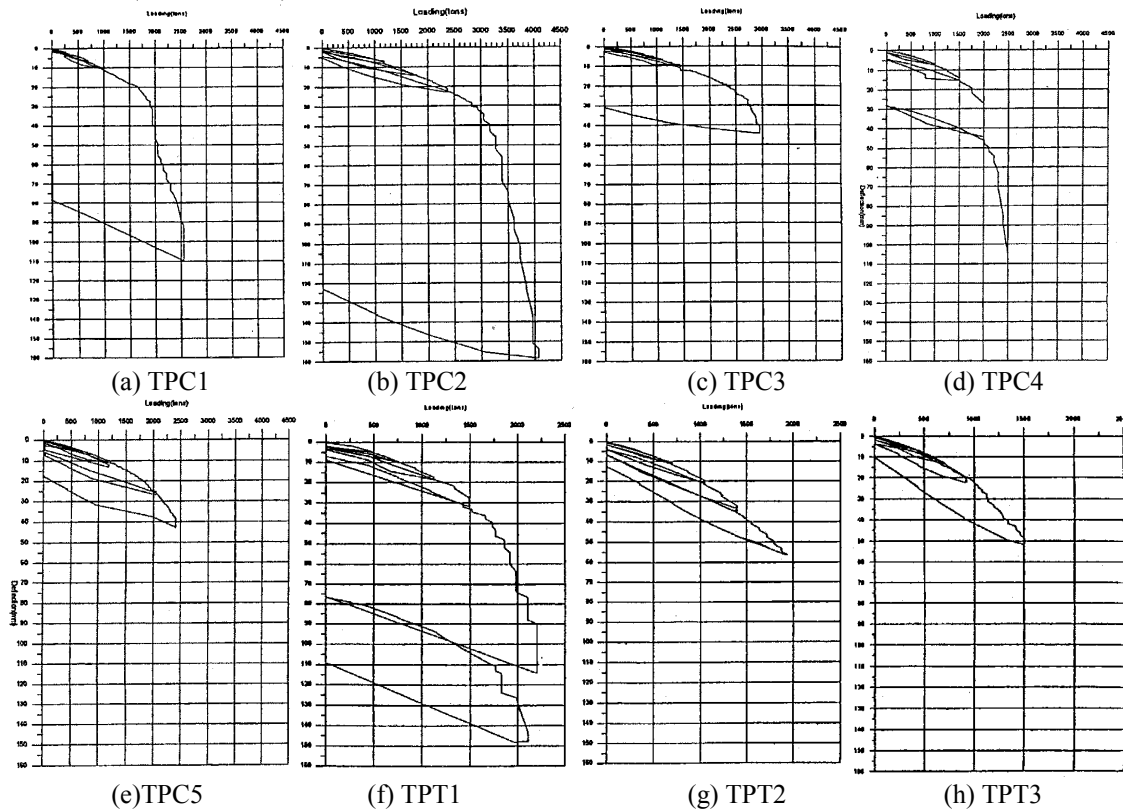


Figure 10. Load versus displacement curves

Further evaluation showed the friction resistance is generally 80-90% of the bearing capacity for piles under compression loads. Figure 11 shows the friction resistances along the piles interpreted from the slope of load versus depth relationships (MAA, 1998b). Results of the tests showed the frictions between the pile and surrounding soils were fully mobilized as the relative displacement reached 10mm. This is consistent with local experiences. However, it was also found the frictions between piles and the surrounding rocks kept increasing with increasing displacements (MAA, 1998b).

Table 6. Bearing capacities of the test piles

Pile No	Construction method	Pile Length (m)	Depth to rock head (m)	Post grouting		Max Applied Load (MN)	Davisson's method		
				Perimeter	Bottom		Bearing capacity (kN)	Friction (kN)	End bearing capacity (kN)
TPC1	Reverse Circulation	55	42	Yes	Yes	27.0	14750	13350	1400
TPC2	Reverse Circulation	63	42	Yes	Yes	38.0	26800	21500	5300
TPC3	Reverse Circulation	58	42	No	No	33.0	24300	18800	5500
TPC4	Full Casing	67	48.6	Yes	Yes	23.9	16000	8600	7400
TPC5	Full Casing	64.5	48.6	Yes	Yes	27.0	21500	17300	4200
TPT1	Reverse Circulation	62.3	51.2	Yes	No	20.0	11600	11600	
TPT2	Reverse Circulation	63.7	54.8	Yes	No	19.4	>13200	>13200	
TPT3	Reverse Circulation	59.5	54.8	Yes	No	16.5	16500	8500	

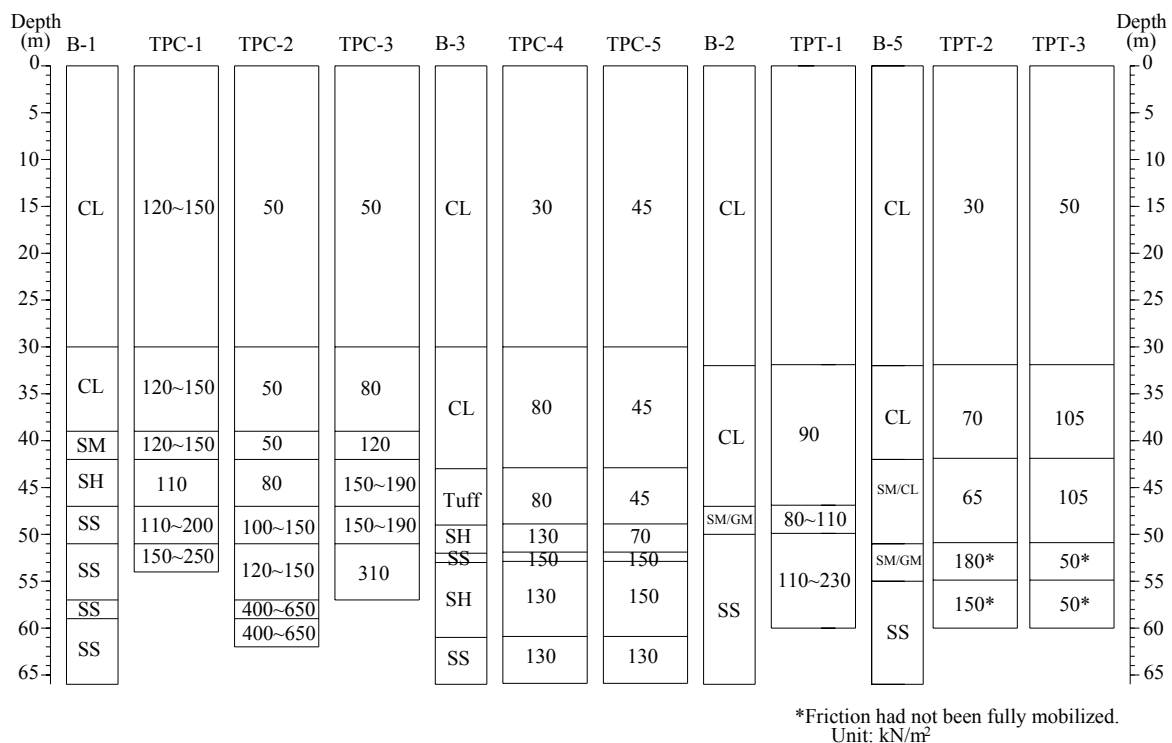


Figure 11. Frictional resistance along the test piles

4.3 Analysis of test results

As shown in Figure 10, the frictions of the piles TPC4 and TPC5 constructed by full casing method in rock sockets were apparently lower than those constructed by reversed circulation method. The reduction of friction resistance was a result the poor cemented mudstone or shale softened by the drilling fluids (MAA, 1998b). For the pile TPC3, as being

the only pile without post grouting in the compression load tests, the end bearing capacity and friction resistance were apparently no less than those of the others. As can be seen in Table 7, the end bearing capacity of pile TPC3 was almost fully mobilized at pile tip settlement of 20mm while those of the others required large settlement to mobilize (MAA, 1998b). This is possibly a result that this particular pile was constructed in a much shorter time period than the other test piles. This implies considerable bearing capacity can be generated if construction procedures are controlled appropriately (MAA, 1998a). In addition, results of the integrity evaluation showed poor quality concrete and base mud were identified close to pile tip for all of the test piles. Results of coring further verified this interpretation. Possible causes were evaluated and lead to modification in construction procedures for base mud cleaning and post grouting.

Table 7. Pile tip settlement at end bearing capacity

Pile no.	Construction method	Bearing stratum below pile tip	Reference Borehole	Pile length (m)	Pile tip settlement (mm)	End bearing capacity (kN)	Post grouting
TPC1	Reverse Circulation	Extremely weak to very weak silty sandstone	B-1	55	60 (20)	1400 (200)	Yes
TPC2	Reverse Circulation	Very weak to weak silty sandstone	B-1	63	110 (20)	5300 (1500)	Yes
TPC3	Reverse Circulation	Very weak sand stone	B-1	58	20 (20)	5500 (5500)	No
TPC4	Full Casing	Extremely weak to very weak shale	B-3	67	60 (20)	7400 (6700)	Yes
TPC5	Full Casing	Extremely weak to very weak shale	B-3	64.5	25 (20)	4200 (3800)	Yes

5. Modification for construction procedures

To avoid friction reduction as a result of the softening of poor cemented rocks, reverse circulation method was selected for pile construction. As the effectiveness of post grouting method was still uncertain, two additional piles with 1.5m in diameter and 83m in length and with average socket depth of 30m were installed for further evaluation. The construction and post grouting for these two bored piles were carefully controlled. The air-lift method was utilized for base mud cleaning. Nevertheless, interpretation of cored samples showed the thickness of sediments were about 7.5cm and 10.1cm that are marginal or unacceptable in practice. After a scrutiny review on base mud cleaning and post grouting methods, it was found the generally adopted U-shaped tube assembled on the bottom of reinforcing cage was not able to reach the cone-shaped bottom of excavation and resulted in the effectiveness. The construction procedures were accordingly modified (Lin et al, 2002).

The modified procedures for base mud cleaning and post grouting were divided into three steps: high-pressure flushing, low-pressure flushing and grouting. A high-pressure flushing with the pressures between 11,800kPa and 14,700kPa was conducted by lowering a drilling rod to the bottom of the pile via one of the four pre-installed grouting tube, and a high-pressure water jet was utilized to force sediments or weak material below the pile tip off the concrete surface. A low-pressure washing was then applied to force the solids of debris flow to the ground surface with returning water. After completion of the base sediment cleaning, the grout is injected through one of the four pre-installed grouting tubes connected with a perforated ring-shaped steel tube assembled with reinforcing cage at its tip. Only two of the grouting tubes were used each time while the others were closed. A packer was used to prevent grout from returning up. The grout was composed water and cement with the ratio of 1:1 with 28-day strength of 19,600kPa (Lin et al, 2002; Chen, 2000).

6. Foundation design

Figure 12 shows the designed layout of the bored piles. A total of three hundred and eighty 1.5m diameter bored piles with rock socket length between 15m and 30m and average center-to-center interval of 3.96m were utilized as the

foundation for the main tower. A total of one hundred and sixty six 2m diameter bored piles with rock socket length between 5m and 29m and average center-to-center interval of 9m was used as the foundation of podium building. The reverse circulation method was selected as the construction method.

As shown in Table 8, the ground profile was simplified to five layers provided with the friction resistances obtained from pile load tests as the baseline for foundation design (Ho and Koh, 2002). To reduce influence of construction uncertainties, the end bearing capacity was not included in the designed bearing capacity. Even though post grouting was specified for construction, it was treated as a measure providing extra safety for the foundation system (Chen, 2004).

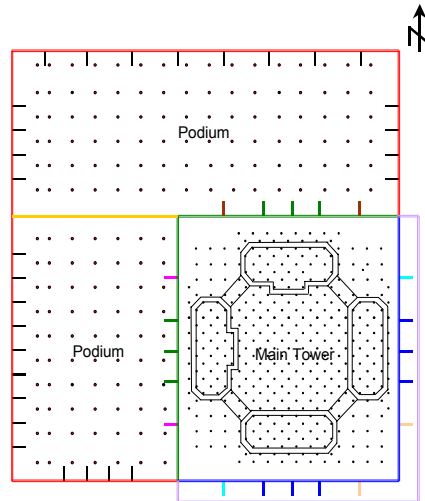


Figure 12. Design layout of bored piles

Table 8. Simplified ground profile and friction resistance

Ground Layer			Description	Friction Resistance (kN/m ²)
1	Clayey Soil	Clay	Silty clay at depth < 30m, N<8	25
2	Colluvium (I)	Cv1	Clay or silty sand at depth > 30m, N>8	80
3	Colluvium (II)	Cv2	Sandy gravel mixed with rock at depth > 30m, N>30	150
4	Rock Formation (I)	Rock 1	Poor cemented rock, N>50	150
5	Rock Formation (II)	Rock 2	Well cemented rock, N>100	500

As shown in Figures 4 and 5, the ground profiles and depths to top of rock varied significantly on this site. To obtain more reliable design results, the pile length was computed using adjacent borehole information. The bearing capacity was analyzed by inputting the t-z characteristics of each ground layer as shown in Figure 13 into the computer code APILE Plus 2 (Ensoft Inc., 1993). Table 9 demonstrates the results of computed bearing capacities for piles adjacent to boreholes B-1 and B-3 (Ho and Koh, 2002).

Table 9. Computed bearing capacity (piles adjacent to B-1 and B-3)

Rock Socket Length (m)	B-1	B-3
	Bearing Capacity (kPa)	
15	18,000	17,900
20	27,900	21,300
25	29,000	24,800

Settlement analyses were conducted utilizing the t-z characteristics of each ground layer established based on pile load test results. The pile group effect was taken into consideration. Results of analyses showed the maximum settlements at the central region of main tower were estimated to be 6-8cm. The maximum settlements occurred at the completion of the main tower structure were less than 2cm (Chen, 2004).

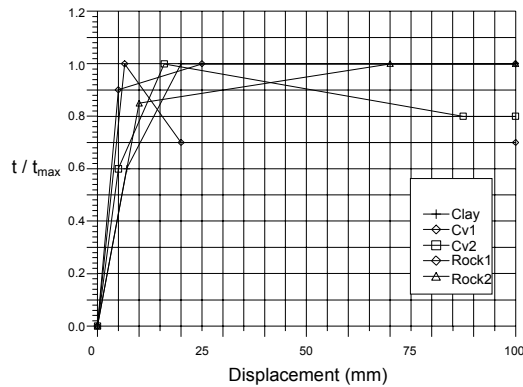
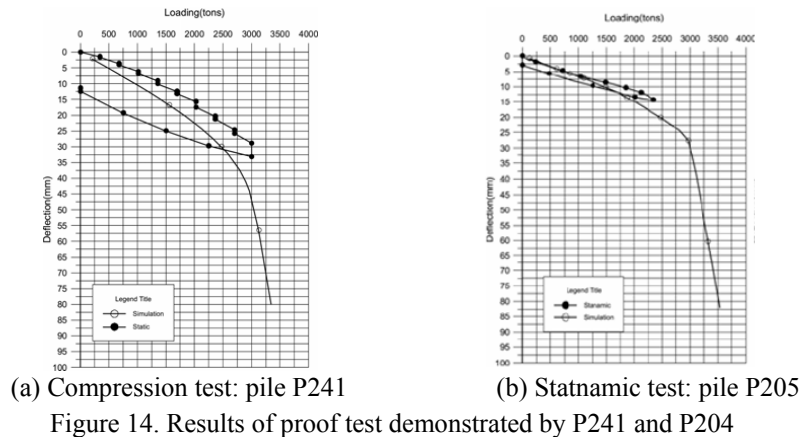


Figure 13. Normalized t-z curves

To verify if the design loads can be achieved in practice, seven compression tests and two pull out tests were performed as soon as part of the working piles were completed. In addition, two Statnamic tests were conducted at the final excavation elevation rather than the ground surface such that the behavior of piles under loading can be evaluated in a more realistic manner. Figure 14 demonstrates the results of the tests by two 1.5m diameter piles, P241 and P205 located in the main tower zone. The lengths of pile P241 and P205 are 48m and 43m below final excavation elevation with rock socket length of 24m and 20m, respectively. The design loads for these two piles are 12MN and 11MN, respectively. According to Ho (2007), results of these tests implied the working piles are able to provide sufficient bearing capacities.



(a) Compression test: pile P241 (b) Statnamic test: pile P205
Figure 14. Results of proof test demonstrated by P241 and P204

8. Summary and conclusions

For the particular rock formation on the site of Taipei 101, the friction resistance of bored piles constructed by full casing method along the rock interface were found generally smaller than those constructed by reverse circulation method. This was a result of the poor cemented rock softened by the drilling fluids. Thus, reverse circulation method was selected for pile construction. Base mud clearing and post grouting methods were modified to accommodate the inherent nature of the selected reverse circulation method. Results of trial installation and pile load tests indicated construction control is very important for achieving the expected performance. Because the ground profile and depth to top of the rock formation varied significantly on this site, the pile length, bearing capacity, and settlement of the piles were designed based on adjacent borehole information. The t-z characteristics of each ground layers obtained from pile load test results were utilized as the baseline for foundation design.

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