On Design and Construction of Pile Group Foundation of Taipei 101

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ABSTRACT: The evenly distributed cast-in-place reinforced concrete group piles with socketed length into soft bedrock of 15 to 33 m were designed as the foundation for Taipei 101. The high-rise building is extremely sensitive to the foundation settlement. Besides, the bearing behavior of a cast-in-place bored pile is largely determined by the way it was installed. Accordingly, the design of pile group foundation for Taipei 101 was based on a series of full scale pile trial installation as well as comprehensive instrumented pile load tests with compressive and pull-out load up to 40 MN and 22 MN respectively. The characteristic *t-z* curve for each subsurface stratum was evaluated and used to predict the pile load-settlement behavior for the specific soil stratification of each pile located, thus each pile length was determined according to the anticipated loads during service. Besides, the pile group effects, including bearing capacity reduction and settlement increase, were considered in the foundation design. The creep behaviors for piles embedded into bedrock were also analyzed by using the measured results of pile load tests. The superstructures, basement, mat, piles and retaining diaphragm walls were modeled into one integral system for the structural design of foundation, thus the estimation of foundation behavior under various load combinations were conducted using the above mentioned model with the sub-grade reaction under foundation mat. Based on the investigation of trial installations, the construction specification was proposed for the installation of reverse circulation piles. For piles under the main tower, the measures of bottom cleaning and post-grouting were employed to improve the pile bottom sediments and increase end bearing capacity. Both the conventional static and STATNAMIC dynamic loading tests were employed to verify the bearing capacities and behaviors of production piles. Results of the proof load tests met the design requirements well as compared with the simulation using pile ultimate load test results.

Keyword: Pile load test, Performance-based design, Pile Group, Creep behavior of pile, Pile bottom cleaning and post-grouting

1. INTRODUCTION

The Taipei International Financial Center, which is called the Taipei 101, is consisted of a 101-storey main tower and a 6-storey podium both with an integrated 5-storey basement excavated to depth about 22 m (Figure 1). Layout for the building and foundation is shown in Figure 2. Total area of the project site is about 160 m × 160 m and the main tower is in dimension of 87 m × 98 m. The main tower was erected up to the 91st floor at the height of 391 m, and the tower top was built to the 101st floor at the height of 438 m and roof at the height of 448 m. In addition, the 60 m pinnacle was added to the spire at the height of 508 m.

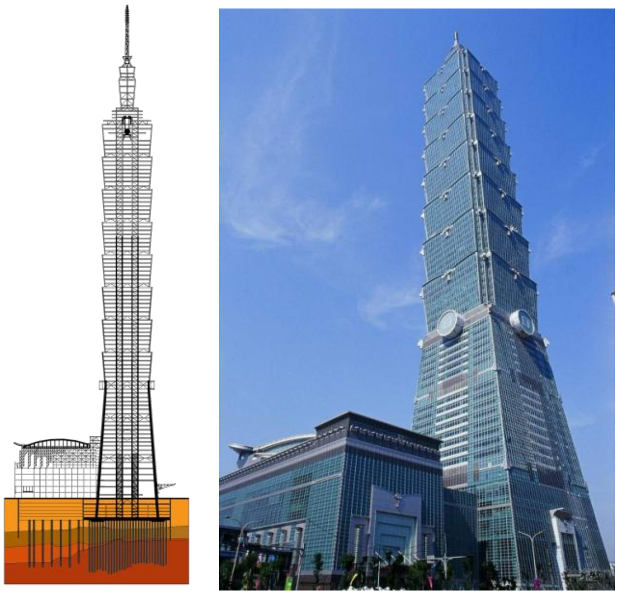


Figure 1 Scheme profile and photo of Taipei 101

Total vertical load of the 101-storey main tower was estimated to be over 4,000 MN. Accordingly, a solid foundation slab with thickness of 3.0 m to 4.7 m was designed to support the structure load and transmit the load to 380 piles with a diameter of 1.5 m and even center-to-center distance of 3.96 m beneath the foundation slab. Those piles had embedment lengths from 40 to 60 m below the foundation level and were socketed into the bedrock by 15 to 33 m. For the 6-storey podium, the design load was controlled by uplifting force caused by the groundwater pressure, thus 167 piles with a diameter of 2.0 m, even spacing of 9 m and socket lengths of 5 to 28 m were designed.

The reverse circulation method was used for the installation of production piles of Taipei 101 after reviewing the construction efficiency, cost consideration and available capacities obtained from the pile ultimate load tests.

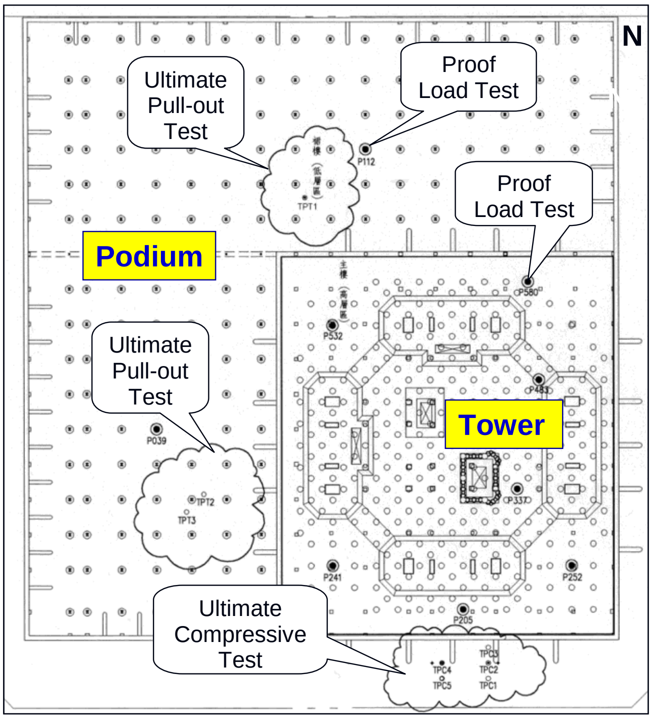


Figure 2 Layout of foundation plan and pile load test of Taipei 101

2. SITE CONDITIONS

2.1 Regional Geological Conditions

The project site of Taipei 101 is located at southeast boundary of Taipei Basin in the north of Taiwan. Geological map of the Taipei

Basin is shown in Figure 3. It is seen that the Taipei (thrust) Fault passes through the site vicinity.

Based on the review of existing literatures, earthquake records, GPS monitoring results, and tectonic force field studies, as well as the comprehensive investigation including subsurface exploration, geophysical testing, along with numerous fossil, mineral identification tests, and C14 dating results, it is concluded that the Taipei Fault, having a fractured zone in width about 10m, is located at least 200m away from the southeast corner of the site. The Fault has not being active for at least 45,000 years and due to the relaxation of the thrust tectonic force field, it is unlikely that the Fault will become active again in the geologically near future. Therefore, the Taipei Fault may be considered non-active in engineering practice.

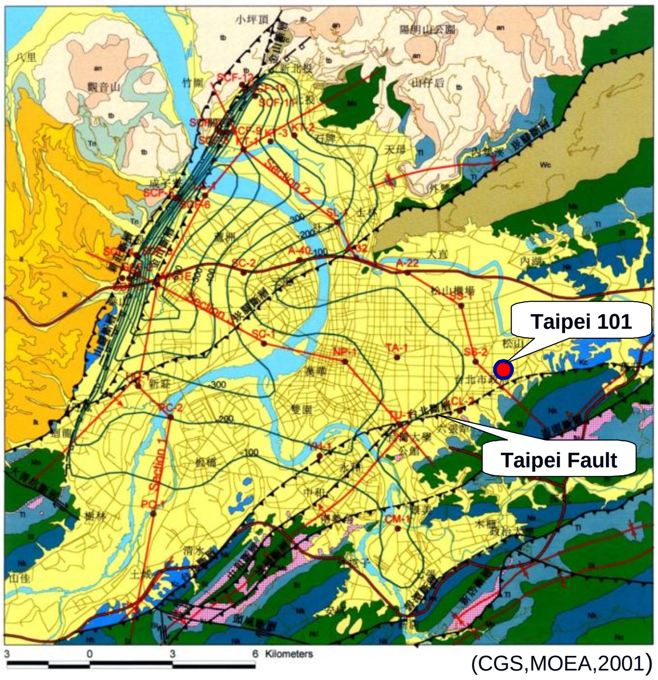


Figure 3 Geological map of Taipei Basin and site location

2.2 Subsurface Strata Distribution and Characteristics

During the feasibility evaluation, preliminary design, detailed design and pile construction phases, 151 borehole explorations, sampling together with various in-situ testis and related laboratory tests were conducted.

Figure 4 shows the typical subsurface profile for the project site. The subsurface strata of the site are mainly consisted of lacustrine alluvial soft to stiff silty clay, colluviums and the Pliocene Kueichulin Formation bedrock. The bedrock is distributed in depths of 40 to 65 m below the existing ground level and considered as the competent bearing stratum. Contour of the bedrock surface elevation is shown in Figure 5 and indicates that the bedrock elevation is shallower in the main tower area.

The Kueichulin Formation bedrock is the youngest rock formation found in the Taipei Basin. Results of investigation indicate that the bedrock within a depth of 10 m is poorly cemented with lower strength; and the cementation and strength of bedrock increase below the depth of 10 m. Because the contents and thickness distribution of colluviums vary significantly, the colluviums are classified into two sub-layers, CV1 and CV2, according to the contents, grain size and strength evaluated by the SPT-*N* values. The silty clay with an average thickness about 30 m distributes rather uniformly in the site and the upper 20 m of the silty clay layer is considered as very soft to soft.

The simplified subsurface strata and representative engineering parameters for the project site are summarized in Table 1.

2.3 Groundwater Level and Pressure

The shallow perched ground water level is about 1.5 to 2.0 m below the existing ground level. The measured groundwater pressure of colluviums is lower than the hydrostatic pressure about 60 to 80 kPa due to the over pumping during 1960s to 1970s in Taipei Basin.

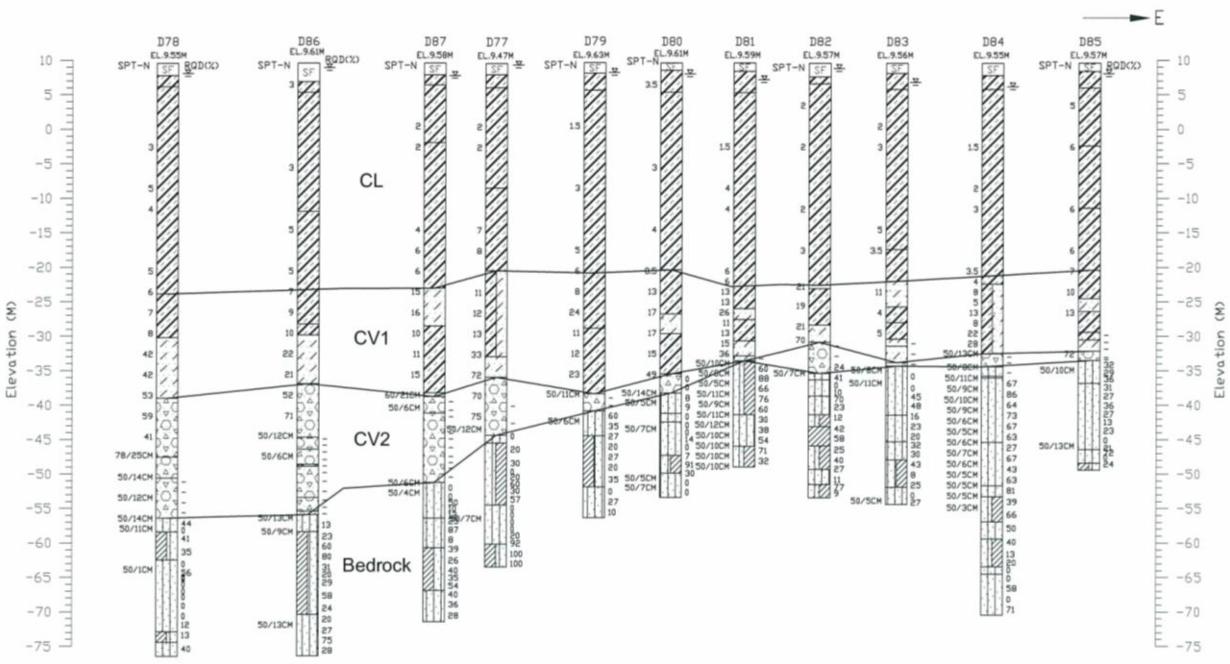


Figure 4 Typical subsurface profile at site for Taipei 101

Table 1 Simplified subsurface strata and representative engineering parameters

No.	Stratum Description		Stratum Bottom Below GL (m)	γ_t (kN/m^3)	W_n (%)	W_L (%)	I_P (%)	e	SPT-N Value	Shear Strength C/C' (kPa)	Shear Strength φ / φ' (deg.)	Cc	Cr	Su (kPa)	qu (kPa)	Es (MPa)	Vp (m/s)	Vs (m/s)
1	Backfill		1.2~3.2 (2.0)	17.2	32	-	-	1.10	1~12 (6)	-	-	-	-	-	-	-	600	176-200 (190)
2	Silty Clay (Alluvium / CL)	Upper	10.0~16.0 (12.0)	17.7	35	38	15	1.10	1~3 (2)	10 / 5	15 / 25	0.30	0.03	35	-	-	600-1550 (1150)	100-190 (140)
		Medium	18.0~24.0 (22.0)	17.7	39	43	19	1.20	2~6 (4)	10 / 5	17 / 28	0.40	0.04	40	-	-	1250-1550 (1480)	182-244 (200)
		Lower	30.5~33.9 (32.0)	17.7	37	43	20	1.10	4~21 (7)	20 / 10	18 / 30	0.40	0.04	60	-	-	1445-1538 (1490)	244-270 (260)
3	Silty sand & sandy silt (Collvuium / CV1)		32.3~41.8 (37.5)	19.1	23	-	-	0.70	9~59 (18)	- / 5	- / 32	-	-	-	-	-	1445-1610 (1510)	244-333 (270)
4	Silty clay or clay silt (Colluvium / CV1)		37.8~43.4 (42.0)	18.6	28	40	19	0.80	9~27 (15)	40 / 20	20 / 30	0.25	0.02	110	-	-	1538-1610 (1570)	244-333 (270)
5	Silty sand, gravel, silty clay & rock fragments (Colluvium / CV2)		41.6~69 (52.0)	19.4	22	-	-	0.70	15~>100 (33)	- / 0	- / 35	-	-	-	-	-	1538-1760 (1670)	238-485 (375)
6	Bedrock (SS/SH)	Poorly cemented	51.6~79 (62.0)	20.6	15	-	-	0.52	>50~100	- / 100	- / 40	-	-	-	20-135 (300)	250~450	1610-1760 (1700)	400-435 (420)
7		Fairly cemented	-	21.6	12	-	-	0.45	>50~100	- / 100	- / 45	-	-	-	30-7000 (1250)	500~550	1760-1940 (1900)	435-600 (485)

Note: 1. ( ) = Average value
2. C', φ' of bedrock = Peak shear strength; Cc, Cr = Compression index and Recompression index
3. Su, qu = Undrained shear strength and Unconfined compressive strength; Es = Elastic modulus; Vp, Vs = P-wave and S-wave Velocity

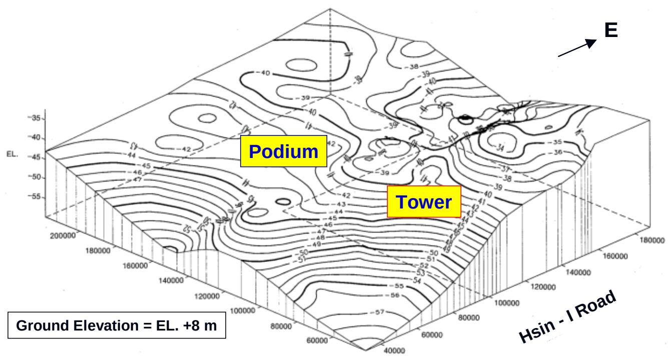


Figure 5 Contour of bedrock surface elevation

3. FOUNDATION TYPE EVALUATION

The structure for the main tower of Taipei 101 was designed in accordance with the concept of Mega-structure system, thus most of the building loads were concentrated on the area of eight mega columns located around the tower perimeter, and four service cores in the central area (Figure 6).

The preliminary structure analysis indicated that the average load of the main tower was estimated up to 500 kPa, and sustained load of over 100 MN was acted on each mega column. Accordingly, the deep foundation system providing high capacity with limit settlement should be considered by following the layout of Mega-structure system. After reviewing the subsurface strata characteristics of the project site and the structure loads, the large-diameter caissons, wall piles (or barrette piles), and group piles were studied as the foundation of main tower in the preliminary evaluation. Layout the preliminary foundation scheme is shown in Figure 7.

The bedrock, which was considered as the competent bearing stratum, distributed in depth of 40 to 65m and the bedrock within shallower depth about 10m was poorly cemented with low strength. Thus, the deep foundation should be socketed into the bedrock greater 10m at least to provide enough bearing capacity. It is seen that the caissons and wall piles could not be embedded into the bedrock to meet the design requirements in engineering practice. Besides, if any bearing capacity failure or excessive settlement occurred in any caisson, the fatal damage might happen to the main tower. Therefore, the evenly distributed group piles, which were integrated with solid foundation slab in thickness greater than 3m, were adapted to serve as the foundation of the Taipei 101 structure.

It was estimated that the piles should be socketed into the soft bedrock by at least 20 m. For the foundation design of Taipei 101 in 1998, considering the ability of construction industry in Taiwan, the techniques of reverse circulation pile (RCP) and all casing pile (ACP) were considered relatively mature as its quality and a proper installation procedure could be enforced. Therefore, a comprehensive program consisting of a series of pile trial installation and pile ultimate load tests was initiated in the preliminary foundation design stage.

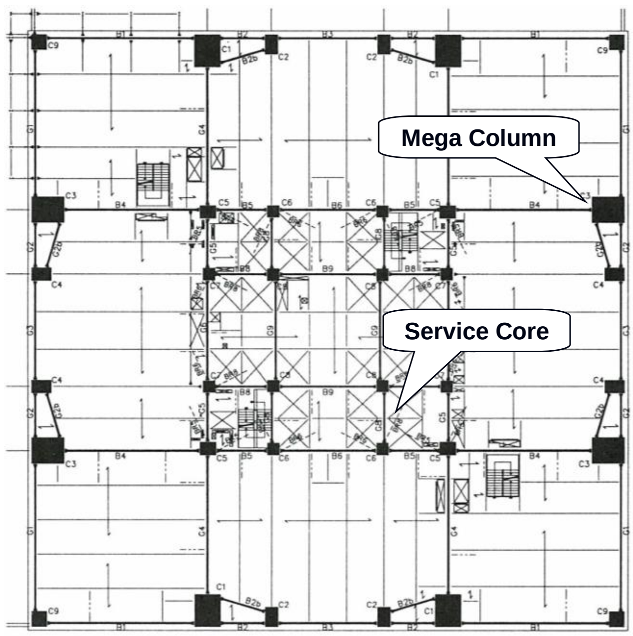
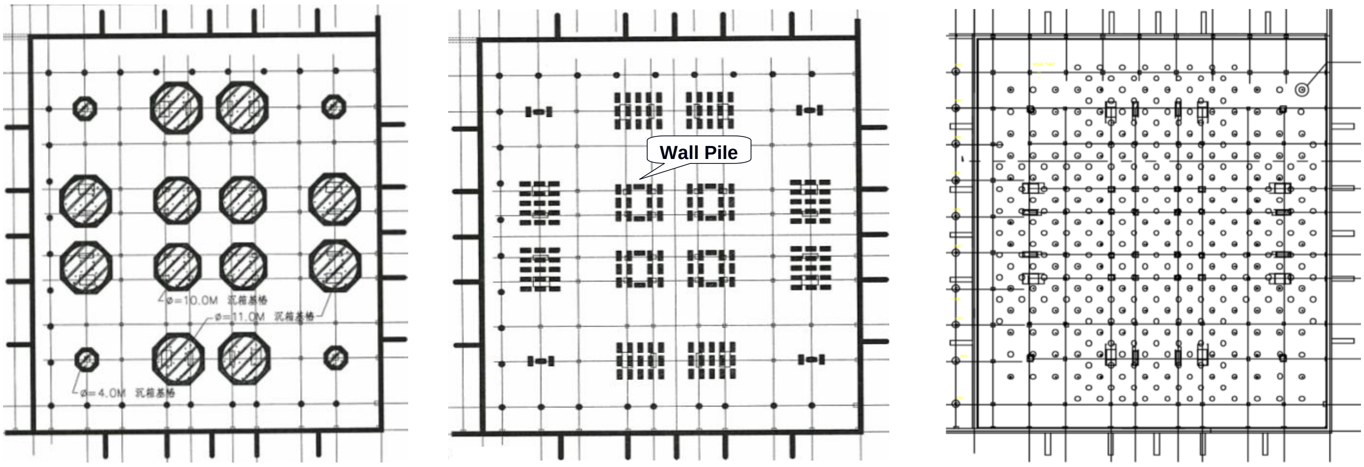


Figure 6 Typical frame plan of main tower



(a) Caissons in larger diameter (b) Wall piles (c) group piles

Figure 7 Layout of preliminary foundation scheme for main tower

4. PRELIMINARY PILE ULTIMATE LOAD TESTS

The comprehensive preliminary pile load test program was conducted not only to evaluate the ultimate bearing capacity and bearing behaviors of the cast-in-place bored pile, but also to investigate the pile construction feasibility such as the penetrating ability into bedrock up to 20 to 30 m and working rate, and to specify the construction methods and procedures.

4.1 Pile Ultimate Load Test Program and Arrangement

The preliminary pile ultimate load test program consisted of five compressive load tests and three pull-out load tests. The test location was selected considering the building layout together with the site specification and shown in Figure 2.

After reviewing the preliminary estimation of pile ultimate capacity, reaction apparatus available, shaft spacing for instruments implanted and limitation of pile slender ratio, and considering the practical application of load test results for the design of production piles, the diameter of 1.2m for test piles were determined. The test piles were socketed into the bedrock for lengths from 0.5 m to 20 m and fully instrumented. All the test piles were loaded by the reaction forces provided by anchor piles, which had various diameters of 1.2 m, 1.5 m, 2.0 m and 2.8 m and socketed lengths of 5 to 15 m. Accordingly the installation performances of piles with different diameters and socket lengths were investigated. The layouts of test piles and anchor piles for compressive and pull-out load tests are shown in Figures 8 and 9, respectively.

The test piles and anchor piles were constructed by reverse circulation method or all casing method so that the construction characteristics of different methods could be studied. Prior to the installation of test piles and anchor piles, borehole investigation was conducted at each test location to verify the stratification, thus the instrumentation, including rebar strain gauges and telltales, were installed in the key elevations. During pile testing, the applied load and displacement on the test pile head as well as the strain and displacement along pile shaft were measured by automatic data acquisition system simultaneously.

Basic information for the pile ultimate load tests is summarized in Table 2. During planning, the ultimate bearing capacities of test piles were estimated by the empirical static formulas together with the subsurface parameters shown as Table 1 and results of supplemental boreholes close to test piles; accordingly the planned test loads were estimated. Besides, after reviewing the records of test piles installation, additional test load about 5MN was increased for compressive load tests to set up the loading reaction system, thus sufficient test loads could be applied to obtain the ultimate bearing capacity of test piles.

Table 2 indicates the maximum applied test loads for compressive and pull-out tests are up to 40.6 MN and 22.0 MN respectively. It is found that most of the maximum applied test loads were greater than the planned test loads; and shown that the ultimate capacities calculated by the static formulas were underestimated. Besides, the larger pile capacities of RCP were expected, so the reverse circulation method was selected for the pile design and construction. Thus, the test results of RCP were further analyzed and described below.

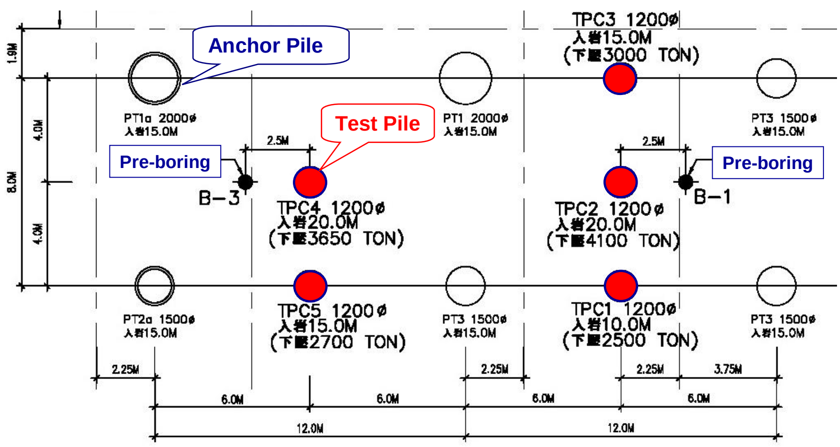


Figure 8 Layout of ultimate compressive load tests

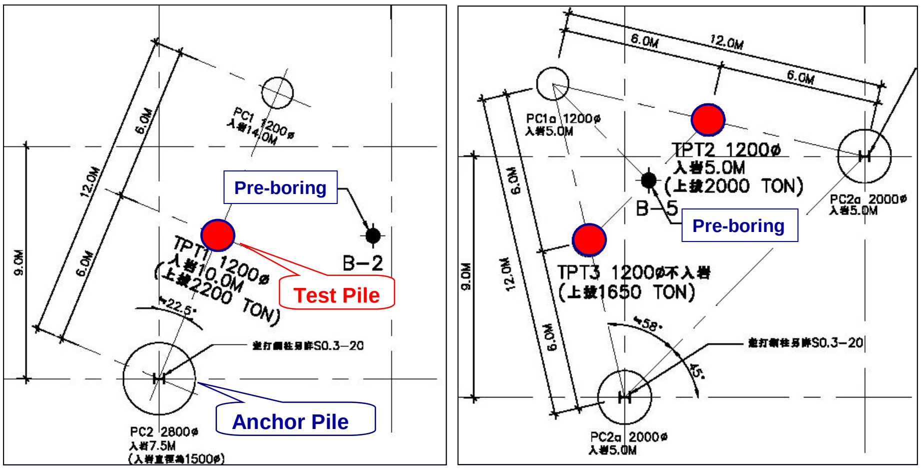


Figure 9 Layout of ultimate pull-out load tests

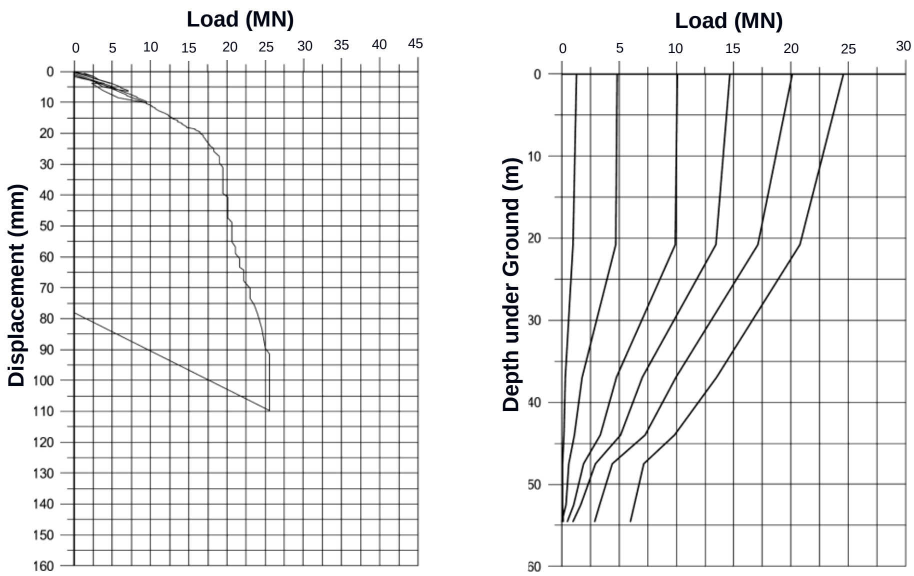
Table 2 Information for the pile ultimate load tests

Test Pile No.	Pile Type	Test Type	Pile Diameter (m)	Pile Length (m)	Socketed Length (m)	Allowable Design Load (MN)	Planned Test Load (MN)	Maximum Test Load (MN)
TPC1	RCP	Compressive	1.2	55.0	10.0	5.3	20.0	25.5
TPC2	RCP	Compressive	1.2	63.0	20.0	11.3	35.0	40.6
TPC3	RCP	Compressive	1.2	58.0	15.0	7.3	25.0	29.4
TPC4	ACP	Compressive	1.2	67.0	20.0	10.0	35.0	25.0
TPC5	ACP	Compressive	1.2	64.5	16.0	6.7	25.0	24.0
TPT1	RCP	Pull-out	1.2	61.3	10.3	7.0	20.0	22.0
TPT2	RCP	Pull-out	1.2	62.2	6.4	7.0	18.0	19.4
TPT3	RCP	Pull-out	1.2	58.0	0.5	5.0	13.0	15.0

Note: 1. RCP = Reverse Circulation Pile , ACP = All Casing Pile
2. The design load was estimated by static formula with safety factor of 3.

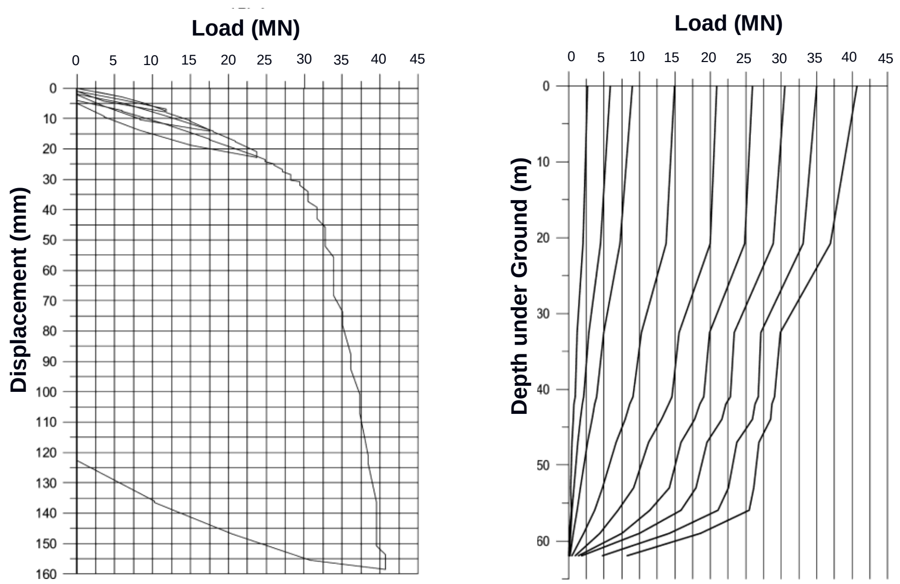
4.2 Interpretation of Pile Load Test Results

All the test piles were loaded to failure state to obtain the ultimate bearing capacities. Figures 10 to 12 show the load-displacement curves at pile head and the load-transfer curves along pile shaft under various loads for the compressive test piles of TPC1 and TPC2 and the pull-out test pile of TPT1.



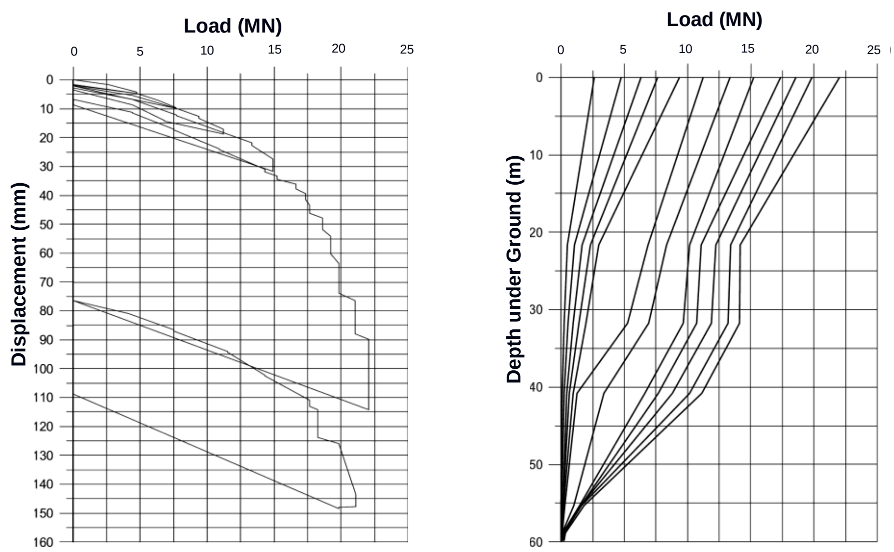
(a) Load-displacement curve (b) Load-transfer curves

Figure 10 Load test results of compressive test pile TPC1



(a) Load-displacement curve (b) Load-transfer curves

Figure 11 Load test results of compressive test pile TPC2



(a) Load-displacement curve (b) Load-transfer curves

Figure 12 Load test results of pull-out test pile TPT1

Based on the load-transfer curves of compressive load test pile of TPC2, the variation of the mobilized side friction along shaft surface for various strata versus the corresponding local pile shaft displacement (called *t-z* curve); and the reaction of end bearing versus pile tip displacement (called *q-w* curve) were evaluated as Figures 13 and 14, respectively.

Figure 14 indicates that significant tip displacements should be required to mobilize the end bearing of compressive test piles. It was suggested that the end bearing was neglected to estimate the pile capacities for safe side design; and the piles for the project site were designed as the friction piles and the end bearing was considered as an additional safety assurances only. However the evaluated *q-w* curves, shown in Figure 14, were still took into account to predict the load-settlement behaviors of piles with various dimensions during the performance-based design of pile foundation.

In addition to the evaluated *t-z* curves for various strata shown as Figure 13, the ultimate side friction resistances (t_{max}) along pile shaft for compressive test piles of TPC1 to TPC3 were also estimated according to the measured transfer loads variation along pile shaft and summarized in Figure 15.

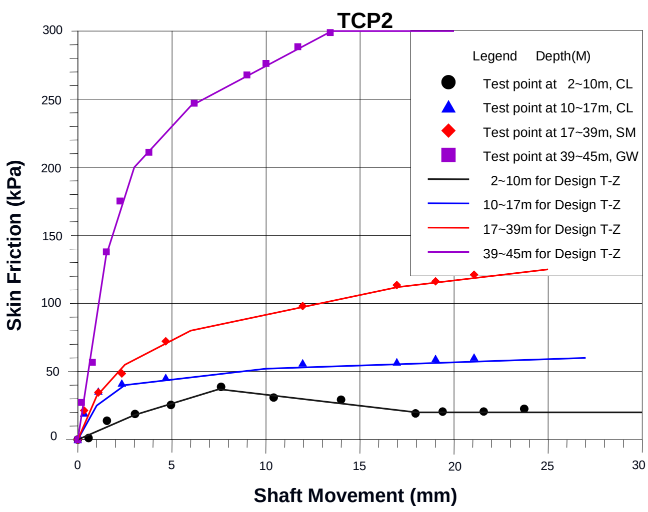


Figure 13 t-z curves for various strata of test pile TPC2

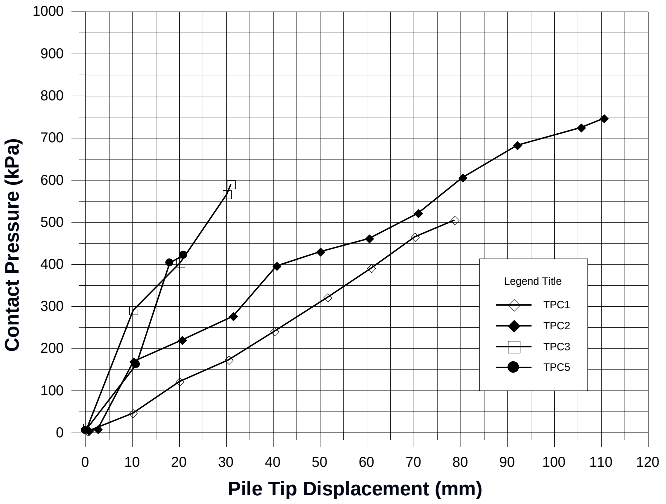


Figure 14 Pile tip contact pressure versus tip displacement

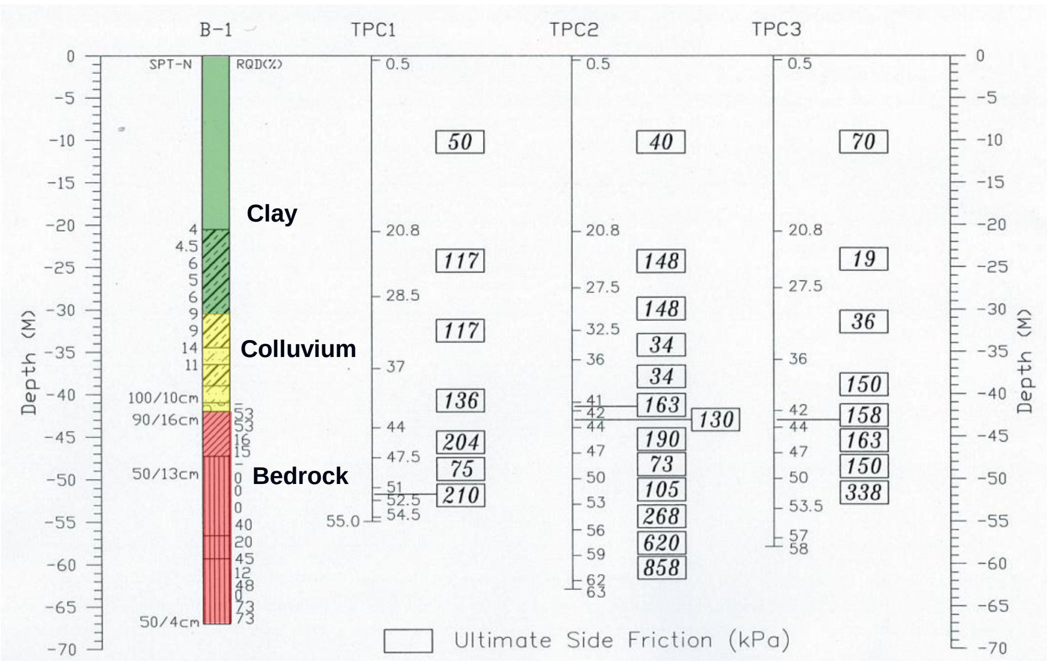


Figure 15 Estimated ultimate side friction for various strata of compressive load tests (RCP)

4.3 Simplified Strata and Ultimate Side Friction Resistance

Figure 15 indicates that the evaluated ultimate side friction resistances for piles socketed into the bedrock have great difference. It is found that the side frictions within socketed length could be correlated with the cementation degree of bedrock after examining the core samples. Besides, the contents and thickness of colluviums vary significantly within the project site. Thus for the pile foundation design, the average subsurface strata of the site were simplified into five categories as shown in Table 3; and the design ultimate side friction resistance for each stratum was also suggested in Table 3 but the end bearing is neglected. Accordingly, the ultimate pile capacities can be easily estimated by the conventional static formulas for the preliminary design phase.

4.4 Design *t-z* Curve for Various Strata of Project Site

By summarizing the *t-z* curves obtained from all the pile compressive load tests together with the subsurface strata distribution and its ultimate side friction resistance (t_{max}) of each test pile, the normalized design *t-z* curves for the five simplified strata of the project site were evaluated and presented in Figure 16.

Table 3 Simplified subsurface strata and ultimate side friction

Stratum		Description	Ultimate side friction (kPa)
1	Clay(CL)	depth<30 m , silty clay, *N*<8	25
2	Colluvium (CV1)	>30m, clay & silty sand, *N* >8	80
3	Colluvium (CV2)	>30m, silty sand, gravel with rock fragments , *N* >30	150
4	Bedrock (Rock 1)	<10m below bedrock surface, poor cementation, *N* >50	150
5	Bedrock (Rock 2)	>10m below bedrock surface, fair cementation, *N* >100	500

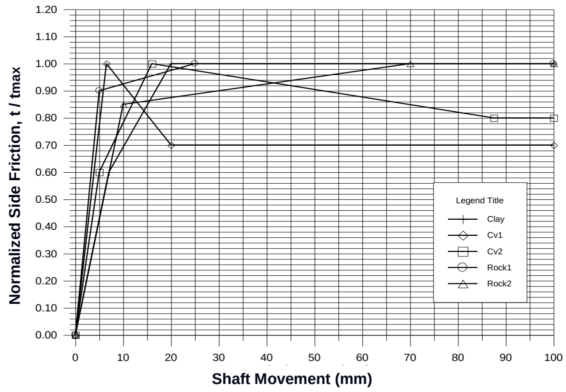
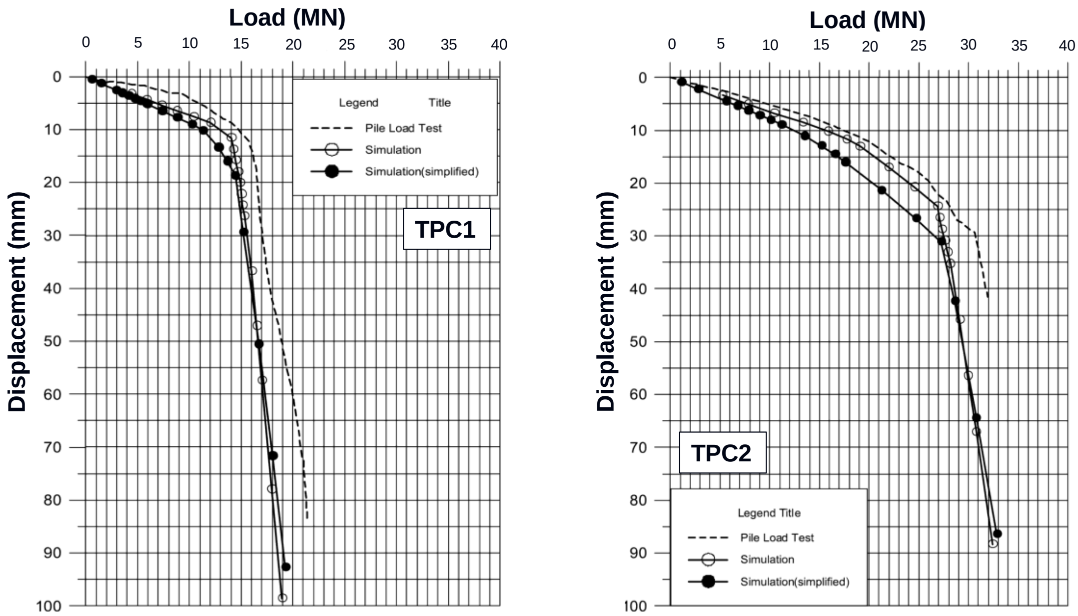


Figure 16 Normalized design *t-z* curves for various strata

To verify the safety for utilizing the normalized design *t-z* curves in the pile foundation design, the above design *t-z* curves were used to simulate the pile load test results and the comparison were summarized in Figure 17.

In the simulation, the reaction of end bearing versus pile tip displacement, i.e. *q-w* curve, of the bedrock was considered by using the conservative curve close to that evaluated from the load test of TPC2 (Figure 14). The side frictions versus shaft displacements were modified by the normalized design *t-z* curves both with the simplified subsurface strata of the project site shown in Table 3, and the individual subsurface strata for each pile test located, which was verified by the borehole investigation conducted prior to pile testing.



(a) Compressive load test of TPC1 (b) Compressive load test of TPC2

Figure 17 Comparison of pile load test results and simulations with *t-z* curves

The dashed lines in Figure 17 indicate the measured load-settlement curve of each test pile; the hollow-circle and the solid-circle lines are the simulation results with the normalized design *t-z* curves, which were modified by the individual subsurface strata of each pile test location and the simplified strata of project site respectively. Both the simulations are rather consistent with the pile test results.

It is found that settlements of the test piles under the allowable design load, which was estimated to be in the range of 8 to 14 MN with safety factor of 2, are much less than 10 mm that can be considered as conservative. As a result, the normalized design *t-z* curves were verified as representative for design purpose and used for the pile performance-based design of this project site.

5. PERFORMANCE-BASED DESIGN OF FOUNDATION

Performance of the important foundation system should be safe and predictable. Thus, the performance-based design was conducted for the foundation design of Taipei 101. Results of the comprehensive pile ultimate load tests and pile trial installation were used to analyze and predict the bearing behavior of piles. The team of structure and geotechnical engineers made it possible to model the superstructures, basement, mat, piles and retaining diaphragm walls into one integral system and estimate the foundation behavior under various design load combinations including the severe loading conditions such as earthquake and typhoon.

5.1 Prediction and Design of Single Pile

For the specific subsurface strata of each borehole conducted in the project site, the representative load-settlement curves for piles with various diameters and socket lengths were predicted as shown in Figure 18 by using the normalized design *t-z* curves and APILE2 program (ENSOFT, 1990), then the allowable bearing capacities with various pile dimensions were evaluated for each location of borehole. Figure 19 summarizes the estimated allowable bearing capacities of piles having a diameter of 1.5 m with various socket lengths for different boreholes conducted in the tower area. Consequently, the required pile length for each pile was determined according to the subsurface stratification at each pile location and the maximum anticipated load on the pile during service.

Because the pile ultimate load tests were conducted on the existing ground level, the maximum side friction (t_{max}) of colluviums, which consisted of the sandy soil and designated as CV2, was reduced by 20% off to consider the relaxation effect of overburden soil removal due to basement excavation. Accordingly, the load-settlement curves for production piles embedded below the foundation level were predicted by the modified design *t-z* curves with above reduction to consider the basement excavation effect; and the allowable pile capacities and the corresponding settlement could be estimated.

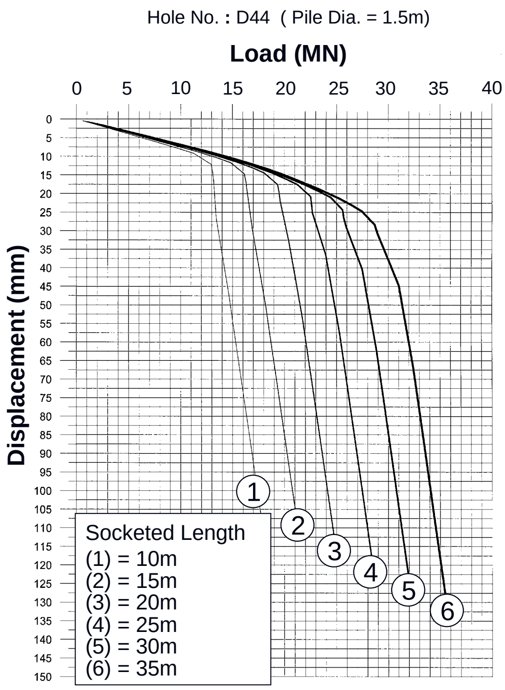


Figure 18 Predicted load-settlement curves with *t-z* curves

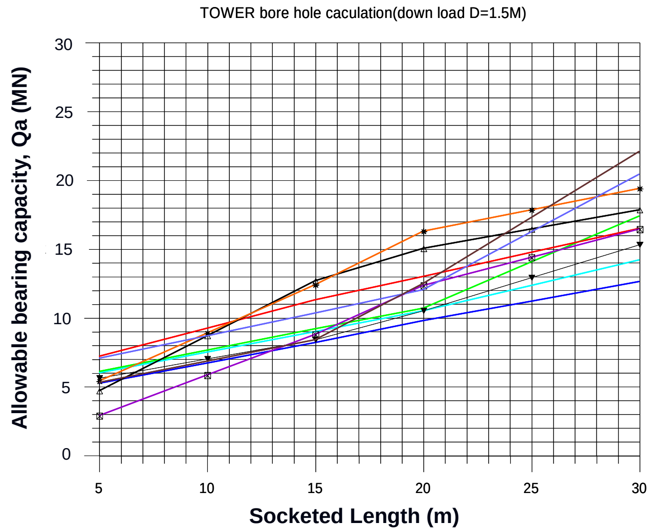


Figure 19 Estimated pile allowable capacities for main tower area

5.2 Evaluation of Pile Diameter

In the preliminary evaluation of main tower foundation, the pile diameter of 1.5 m and 2.0 m were suggested. After considering the restrict on distance between piles, in which the center-to-center distance between piles should be greater then 2.5 times of pile diameter (Taiwan Building Code, 2001), and locations of the king posts installed for the basement construction of the main tower, the pile diameter of 1.5 m was selected. Consequently, the group piles could be distributed more uniformly and the loads transferred along piles would be considered more effectively in the structure deign. Besides, the construction cost might be more economical.

5.3 Considerations of Pile Group Effect

For the layout of group piles under the main tower, distance between pile centers is about 2.63 times the pile diameter of 1.5 m, thus the pile group effect was further evaluated. Because the piles were socketed into the bedrock about 15 to 33 m, only the side friction of strata, including the colluviums, above the bedrock was reduced with a coefficient(η) of 0.56 estimated by the Converse-Labarre equation (Moorhouse and Sheehan, 1968). By using normalized *t-z* curves together with the reduction coefficient of 0.56, the load-settlement curves were simulated and only 15% reduction in the total ultimate capacity of pile was estimated. As a safety factor of 2 was employed to design the allowable capacity, the 15% reduction should still provide the sufficient safety.

As all the piles were socketed into bedrock, the additional settlement induced by the group piles was estimated by considering only the elastic compression of bedrock. As shown in Figure 20, the stress of the loaded group piles was assumed to be distributed from the level of bedrock surface and 2/3 of the socketed length for considering the design normal load and maximum load conditions respectively, thus the overlapping stress distribution of adjacent piles in the pile group could be considered and the elastic compression of bedrock was calculated accordingly. A typical contour of bedrock compression under sustained load of group piles for the main tower area was shown as Figure 21.

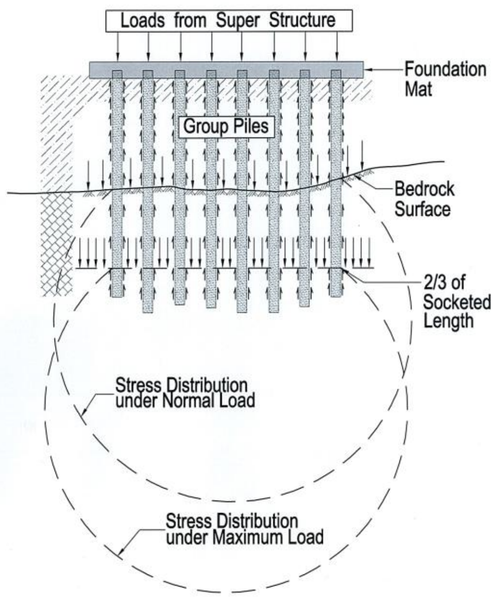
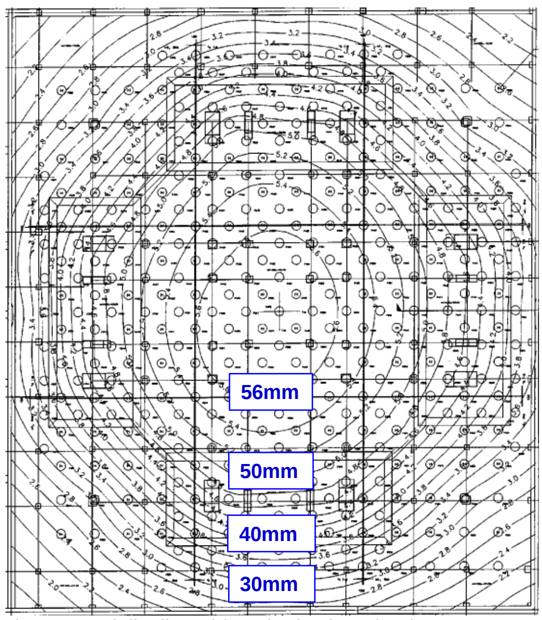


Figure 20 Stress distributions under group piles



(Note: Load distributed from bedrock surface)

Figure 21 Bedrock compression under group piles of tower area (under sustained load at low groundwater level)

5.4 Structure Design Model and Analysis Results

For the foundation design of Taipei 101, the structural analytical model took into consideration not only the pile-mat system, but also the 5-storey basements and the surrounding retaining diaphragm wall together with superstructure of the podium and the main tower up to 6th floor.

Analysis of the foundation behavior under various load combinations, including wind, earthquake loads, and variation of ground water levels, was conducted by using the above mentioned structure model together with the estimated distribution of sub-grade reactions, called K_v coefficient, under the foundation mat. The coefficient of sub-grade reactions was estimated by considering both the settlement of each pile under design load, which was obtained directly from the predicted load-settlement curves using characteristic *t-z* curves, and the compression of the bearing bedrock under load of group piles.

Based on the load distribution from preliminary structural analysis, the pile lengths were estimated and settlement of each pile as well as the bedrock compression due to load of group piles were calculated; hence the distribution contour was estimated accordingly. Then this distribution served as input parameter to the structural analytical model, and the revised load distribution on the foundation was obtained. This process was reiterated until a converged consistent result was obtained. The above procedures were repeated for all the critical possible load combinations.

For the final layout of pile foundation (Figure 2) under the long-term sustained load at low ground water level condition, the bedrock compression under load of group piles for the main tower area was shown as Figure 21; accordingly the corresponding distribution contour was evaluated and presented in Figure 22. As a result, the deformation of the foundation structure assembly and foundation settlement contour of the main tower, which calculated by the structure-foundation-subsoil interaction model, are shown as Figure 23.

The preliminary measurements indicated that the maximum foundation settlement at center of main tower was less than 20mm as the primary skeleton structures of tower were completed (Chen, 2004). Unfortunately, the instruments were damaged during further construction, thus there was no detail measured settlement available.

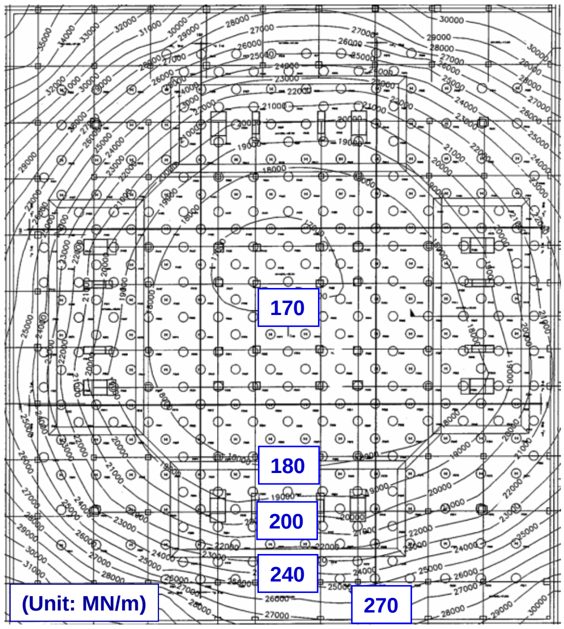
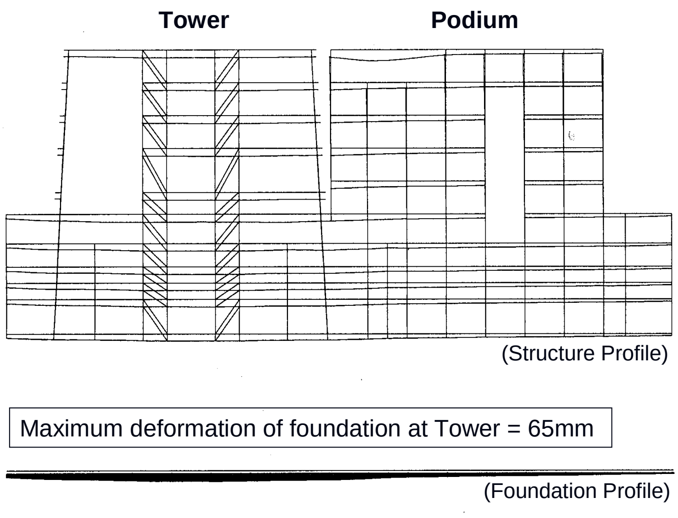
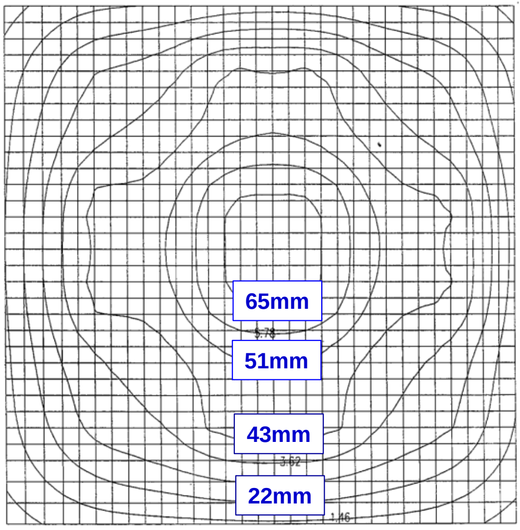


Figure 22 Contour of *Kv* distribution for tower area (under sustained load at low ground water level)



(a) Deformation of foundation structure assembly



(b) Foundation settlement contour of main tower

Figure 23 Foundation settlement calculated by structure-foundation-subsoil interaction (under sustained load at low groundwater level)

5.5 Evaluation of Creep Behavior for Pile on Bedrock

The bearing bedrock for piles of the project site was classified as soft rock. Creep behaviors of the bedrock under long-term sustained load acting on piles were further evaluated to consider the potential long-term pile settlement. The preliminary pile compressive load tests were conducted by adopting a maintained load procedure, thus the testing results were used to evaluate the time-dependent behaviors of piles under sustained load.

The concept of creep coefficient, K_s, defined in the Germany code (DIN), was introduced to analyze the creep characteristics of pile under sustained loading. For each load increment during pile testing, the creep coefficient, K_s, was calculated by Equation (1).

$$K_s = \frac{d_2 - d_1}{\log t_2 - \log t_1} \tag{1}$$

where d_1 = settlement of load increment maintained duration of t_1
d_2 =settlement of load increment maintained duration of t_2

Figure 24 indicates the relationship between applied loads in pile testing and creep coefficient. According to the DIN code, the load related to the creep coefficient of 2 mm was defined as the ultimate creep load. The pile ultimate capacity evaluated by utilizing the preliminary load test result was considered as the yielding load and a safety factor of 2 was used to determine the allowable pile capacity. Accordingly the creep load ratio, which was defined as pile allowable capacity divided by the ultimate creep load, was calculated to be in a range of 0.45 to 0.56 for each test pile and summarized in Table 4.

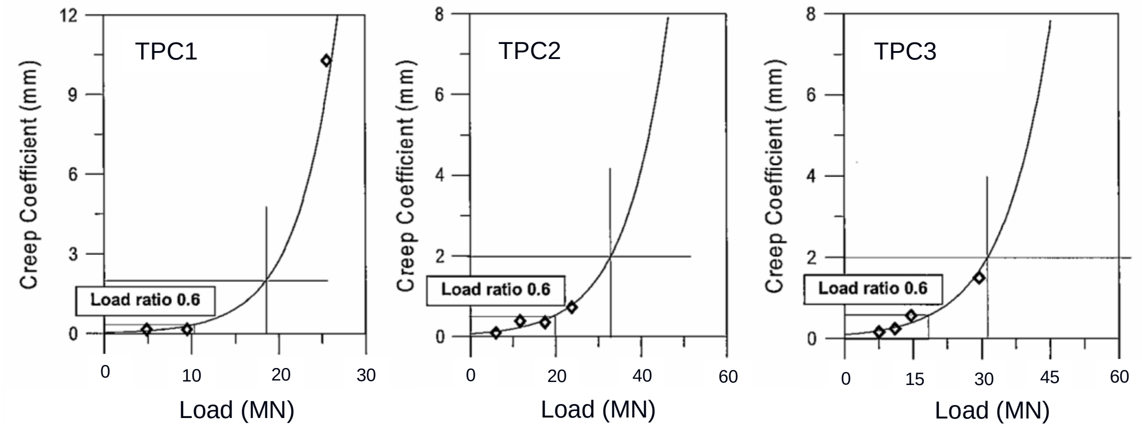


Figure 24 Creep coefficient and applied loads for compressive load test piles

Table 4 Creep load ratio for compressive test piles

Pile No.	Pile Length (m)	Socketed Length (m)	Pile Ultimate Capacity (MN)	Ultimate Creep Load (MN))	Pile Allowable Capacity (MN)	Creep Load Ratio
TPC 1	55	10	19.0	17.0	9.5	0.56
TPC 2	63	20	33.0	33.0	16.5	0.50
TPC 3	58	15	28.5	32.0	14.3	0.45

The average creep coefficient was evaluated to be about 0.5 mm/ (□log time duration) for the test piles of TPC1 to TPC3. As a result, the creep increment of about only 2.5 mm was estimated for the test piles under sustained load for 1 hr after testing to 10 yrs duration.

In fact, the specified bottom cleaning and post-grouting measures, which was illustrated in Figure 25, were conducted for the production piles, thus the pile capacities would be greater then those predicted by results of the ultimate load test. Besides, the earthquake, wind and some live loads were not included in considering the design sustained load. Accordingly, the long-term sustained loads acting on the production piles will be less then the design pile allowable capacities, thus the actual creep load ratio will be much less than the evaluated value shown in Table 4. Therefore the creep effect of the production piles embedded into the bedrock can be neglected for the main tower area.

6. CONSTRUCTION OF PILE FOUNDATION

6.1 Pile Trial Installation and Construction Specification

In addition to the verification of ultimate capacities of test piles, the pile construction characteristics were also investigated in the pile foundation design. A pile trial installation program consisted of over 20 cast-in-place concrete piles, which included the preliminary load test piles and anchor piles installed by reverse circulation or all casing method, with diameters ranging from 1.2 m to 2.8 m and socketed lengths from 0.5 m to 20 m into the bedrock.

Each pile trial installation was conducted under the full-time supervision by geotechnical engineers, especially for the verification of bearing strata, cleaning treatments of pile bottom sediments and drilling rates, thus improvements for the installation technique, tools and procedures were evaluated.

After reviewing the construction efficiency, cost consideration and ultimate capacities evaluated by the preliminary load tests, the reverse circulation method was adopted for the installation of production piles; and the corresponding improved construction specification was proposed. Prior to construction of production piles, two of the longest piles, called the sample pile, with total length up to 81 to 82 m and socketed length of 30 m were installed to verify the effectiveness of the proposed specification.

It is seen that the cleaning treatments of pile bottom sediments is a dominate factor for developing end bearing capacity and reducing settlement of reverse circulation piles. The procedures and requirements of sediments treatments, which specified in the pile installation contract, were briefly described as below.

For the piles installed in the main tower, two phases of treatments were carried out. The first one was conducted when the pile drilling reached design elevation; and the second one was performed after the completion of rebar cage placement and prior to concreting. For the podium area, there was a supplemental treatment for cleaning the sediments caused during planting of steel column for the top-down construction of basement.

The first treatment was carried out by pumping the bottom sediments directly using the pipe of reverse circulation apparatuses for an appropriate waiting period after pile drilling completed. The second and third treatments were conducted by the air-lift method which used the tremie pipe equipped with the air pump. During air lifting, the tremie pipe was lowered down to the pile bottom and the air pressure at the bottom of tremie pipe was controlled in range of 1,000 to 1,200 kPa at least, which was greater than the value of two times of groundwater pressure acting at pile bottom.

6.2 Measures for Improving Pile Tip Sediments and Capacity

According to the investigation on trial installations, the thick soft sediments around pile bottom were found by examining the strength of the samples, which were acquired by coring through the pre-embedded steel pipes along the pile shaft. Thus, the measures of pile bottom cleaning and post-grouting were conducted in production piles under the main tower to clean up the bottom sediments effectively and improve end bearing capacities. The safety for pile foundation of the main tower was increased accordingly.

The scheme of measures is shown in Figure 25. After the pile installation completed and sufficient strength of concrete acquired, the pile bottom sediments were flushed out with high pressure water through the pre-embedded pipes along the pile shaft; then the pile base was grouted with cement slurry and the a maintaining pressure about 5000 MPa. If the grouting pressure could not be maintained, the base grouting was controlled by the quantity of grouting (Koh et.al., 2004).

Results of the proof load testing on production piles indicated that the capacities and bearing behaviors were verified to meet the design requirements and introduced as below.

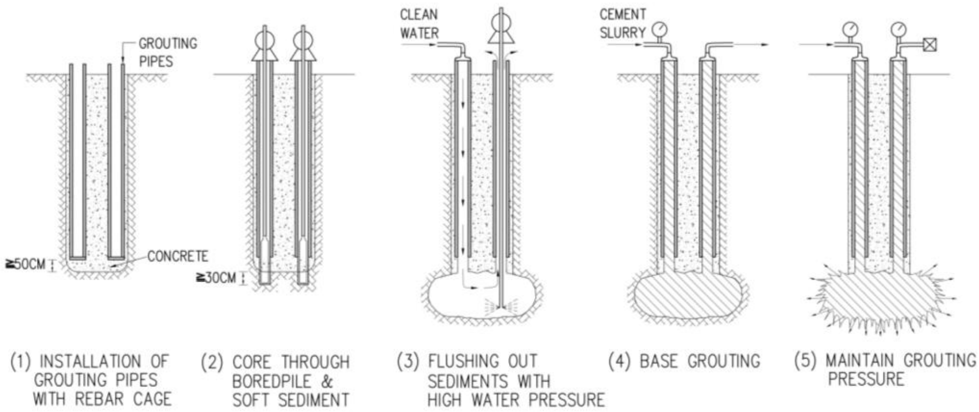


Figure 25 Scheme for pile bottom cleaning and post-grouting

7. PROOF LOAD TESTS OF PRODUCTION PILES

In order to verify that the design allowable pile capacity and sufficient safety factor at the allowable displacement can be reached by actual construction procedures specified according to the investigation of trial pile installation, nine sets of proof load tests were conducted on the production piles. Seven sets of compressive testing were conducted in the main tower and two sets of pull-out testing were performed in the podium (Figure 2). For the proof compressive tests, both the conventional static loading and STATNAMIC dynamic loading test methods were employed. In addition to testing on the existing ground level, the STATNAMIC tests were carried out also on the basement excavation level to evaluate the actual bearing behaviors of production piles with effective length below the basement level. The proof pull-out tests were conducted only by the static loads.

Prior to the proof load testing on the selected production piles, the compressive load versus settlement curves, which shown in Figure 26, were simulated by using the design *t-z* curves evaluated from the preliminary load test results. Accordingly the bearing behaviors for production piles with different effective embedded lengths were predicted.

Figure 27 shows the calibration results of STATNAMIC tests with the static load tests on the same production piles. As the plastic settlement induced by the sustained static load was neglected, the STATNAMIC test results were rather consistent with those of static load tests. Thus, the STATNAMIC test method was verified to be used for proofing the design capacity of production piles, especially when testing on the foundation level was requested

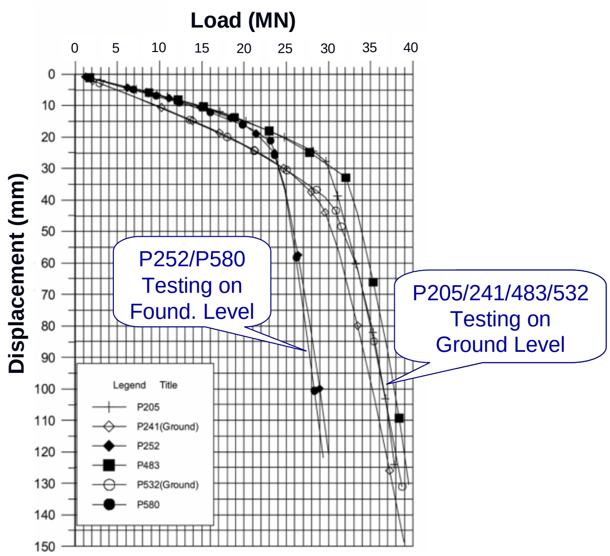
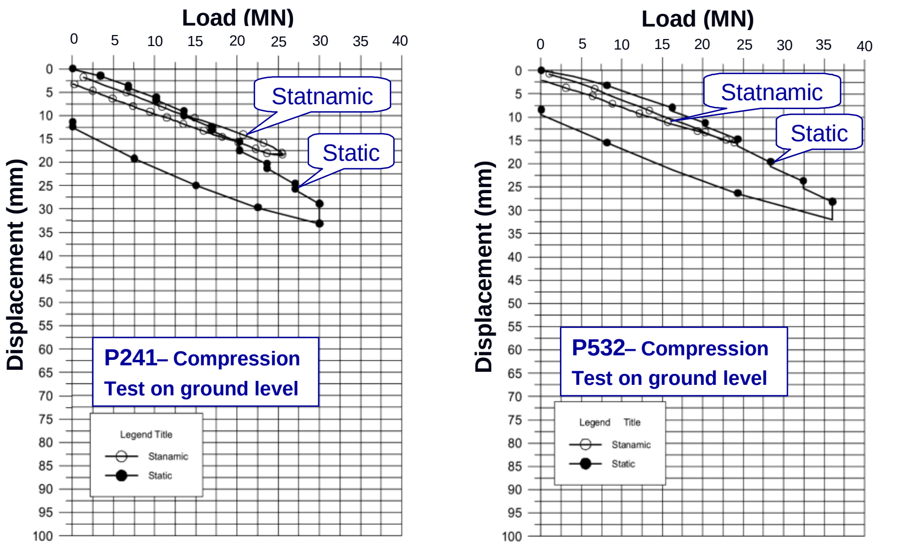


Figure 26 Simulation for bearing behaviors of proof load test piles

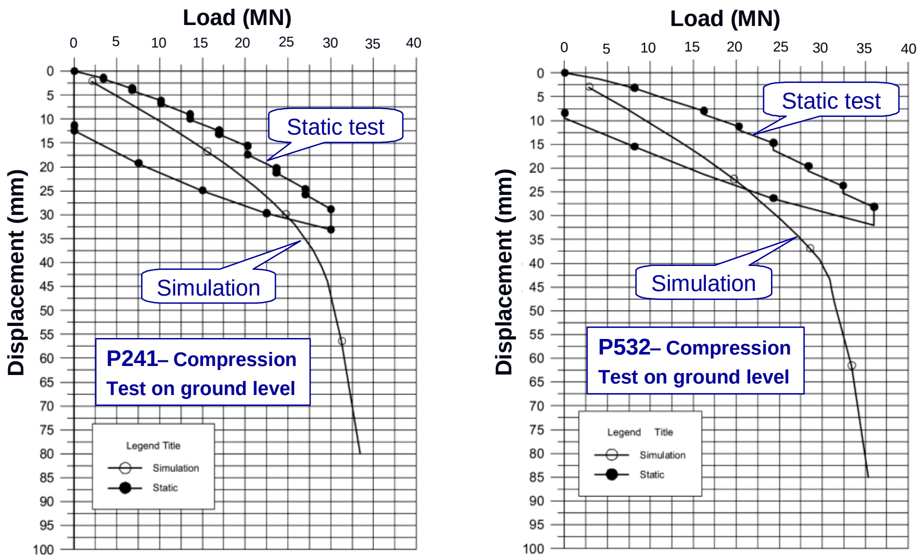


(a) Testing on ground level for pile P241 (b) Testing on ground level for pile P532

Figure 27 Calibration for statnamic load test with static load test

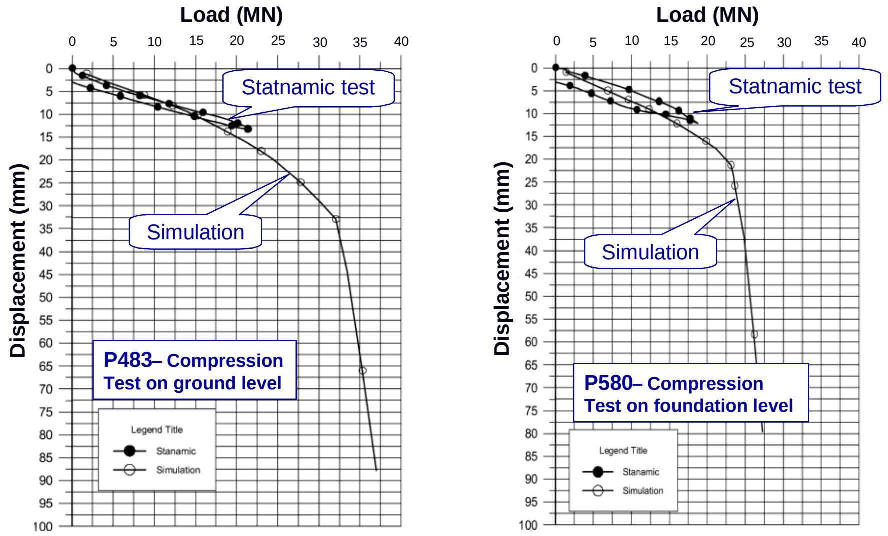
For proof compressive tests in the main tower, the corresponding load-settlement curves, which were simulated by using the characteristic *t-z* curves, were compared with those obtained from the static as well as STATNAMIC tests and shown in Figure 28 and 29 respectively. The comparisons indicate that the greater bearing capacities under smaller settlements for tested production piles are expected than those of simulation. As a result, the capacities of production piles installed in the main tower were proofed to satisfy the design requirements. It is also found that the results of STATANMIC tests on the ground level and foundation level, which were shown in Figure 29, are very close to the simulation results. Besides, Figure 30 indicates that results of the proof pull-out testing on production piles in the podium will also provide sufficient uplift capacities under limit displacement.

Thus, it is concluded that the production piles, which were designed and installed in accordance with the preliminary load test results and specified construction specification, should provide sufficient capacities and required bearing behaviors for the foundation design of Taipei 101.



(a) Comparison for pile P241 (b) Comparison for pile P532

Figure 28 Comparison of simulation and static proof load test



(a) Comparison for pile P483 (test on ground level) (b) Comparison for pile P580 (test on foundation level)

Figure 29 Comparison of simulation and statnamic proof load test

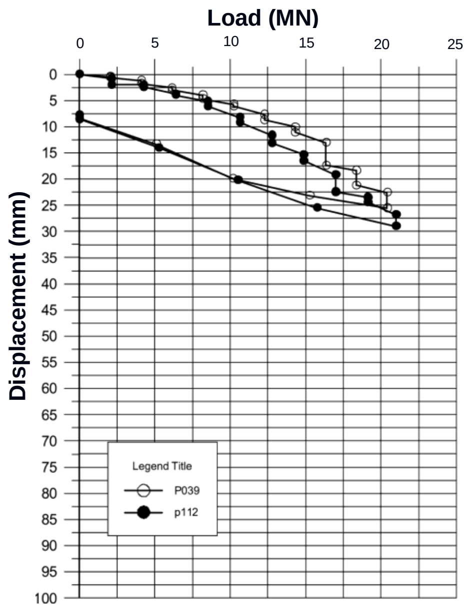


Figure 30 Results of pull-out testing on production piles

8. CONCLUSIONS

Verification and design for the pile foundation of Taipei 101 indicates that the comprehensive preliminary pile ultimate load tests and trial installation together with the improvement specified in the construction specifications are essential. The team-up of structure and geotechnical engineers make it possible to model the superstructure, basement, foundation and retaining diaphragm walls into an integrated system and to estimate the foundation behavior under various design load combinations. Thus, performance-based design for the foundation of high-rise buildings was carried out. Based on the results of pile ultimate load test, evaluation and design for the pile group foundation of Taipei 101, the following conclusions were summarized.

(1) Settlements of the test piles under the allowable design load with safety factor of 2 were much less than 10mm and the normalized design t-z curves were used for the pile performance-based design concept.

(2) The end bearing was neglected to estimate the pile capacities due to safety consideration. However, the evaluated *q-w* curves were still took into account to predicate the load-settlement behavior of piles for the performance-based design of pile foundation.

(3) The maximum side friction evaluated in loading tests was reduced by 20% off to consider the effect of overburden soil removal during basement excavation, because the pile ultimate load tests were conducted on the existing ground level. Thus, the corresponding design *t-z* curves were modified for the pile foundation design.

(4) Pile group effect was considered that the side friction of strata above bedrock was reduced with coefficient of 0.56. Only a 15% reduction in the total ultimate capacity was estimated with the modified *t-z* curves.

(5) The creep behavior for pile on bedrock was also evaluated that increment about only 2.5mm was estimated for test piles under load sustained for 1hr after testing to 10 yrs duration. However, the creep effect of the production piles embedded in the bedrock can be neglected for the main tower area.

(6) The measures of bottom cleaning and post grouting were suggested to be conducted in the production piles, thus the end bearing behaviors were improved.

9. ACKNOWLEDGEMENTS

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