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Foundation Design and Performance of the World's Tallest Building, Petronas Towers

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ABSTRACT

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The analysis and design of foundations for the World's Tallest Building are described. The results of the exploration and in-situ testing program required to define the foundation support conditions are presented.

The Towers are supported on a mat foundation on barrettes in residual soil and weathered silt stone, sandstone formation overlying karstic limestone at depths ranging from 80 to more than 200 meters. The extensive grouting program required to fill major cavities in the limestone beneath the Tower mats is described.

The settlement analysis performed utilizing modulus information developed from the in-situ testing program is outlined. Barrette lengths were varied above the steeply sloping limestone bedrock in order to minimize the calculated differential settlement. Settlement measurements taken during construction indicate actual total and differential settlements less than predicted. The barrette strain gage and mat pressure cell instrumentation program is outlined and preliminary results to date presented.

KEYWORDS

Barrettes, mats, in-situ testing, settlement prediction, ground improvement, cavity filling, slump zone grouting, foundation instrumentation, foundation performance

INTRODUCTION

The newly constructed Petronas Towers in Kuala Lumpur Malaysia are the world's tallest buildings (451.9 meters (1482 feet) from street level to the top of the pinnacles), 10.9 meters taller than the 110 story Sears Tower in Chicago, Illinois (441 meters (1450 feet) from street grade to the flat-top of the building which has the highest roof top and highest inhabited space).

The Petronas Towers are also believed to have the world's deepest building foundations. The depth of foundations for tall buildings varies significantly with site geology; e.g. from 20 meters for the 110 story World Trade Center in New York, to 35 meters for the 110 story Sears Tower, 58 meters for the 100 story John Hancock building in Chicago, and 77 meters for the Sohio Corporate Headquarters Building in Cleveland, Ohio which was formerly believed to involve the world's deepest building foundations. Jin Mao tower currently under construction in Shanghai has friction pile foundations extending to 78 meters below grade. While all these foundations, except Jin Mao Tower, extend to solid bedrock, they are now far surpassed by the Petronas towers concrete barrette foundations which extend to a maximum depth of 130 meters below grade in soil and weathered rock, plus ground improvement cement grouting to depths up to 162 meters. Thus, measured from the bottom of the deepest foundations to the top of the building, Petronas Towers would measure either 582 meters (1909 ft.) or 614 meters (2014 ft.) depending upon whether the ground improvement was considered part of the foundation system.

Soil and Bedrock Conditions

A generalized soil and bedrock profile below the towers is shown in Figure 1. The geologic profile consists of 12 to 20 meters (39 to 66 feet) of medium dense, silty and clayey alluvial sand. The alluvium is underlain by a medium dense to extremely dense, sandy and gravelly silt and clay material which is a residual soil and weathered rock deposit known locally as the Kennyhill Formation. The bedrock below the Kennyhill is of Silurian age and consists mainly of calcitic and dolomitic limestone and marble. The rock surface is very irregular and has been weathered by solution activity creating numerous joints and cavities. As a result of the solution activity, isolated zones of the Kennyhill have eroded into the bedrock cavities creating soft or loose zones referred to as slump zones. The hard Kennyhill above arches over these slump zones so they do not feel the full weight of the overlying formation. For design purposes, the slump zones were defined as Kennyhill just above bedrock with Standard Penetration Resistance (N-values) less than 20 blows per 30 centimeters (20 blows per foot). By this definition, six slump zones were identified beneath the footprint of Tower 2, and none below Tower 1.

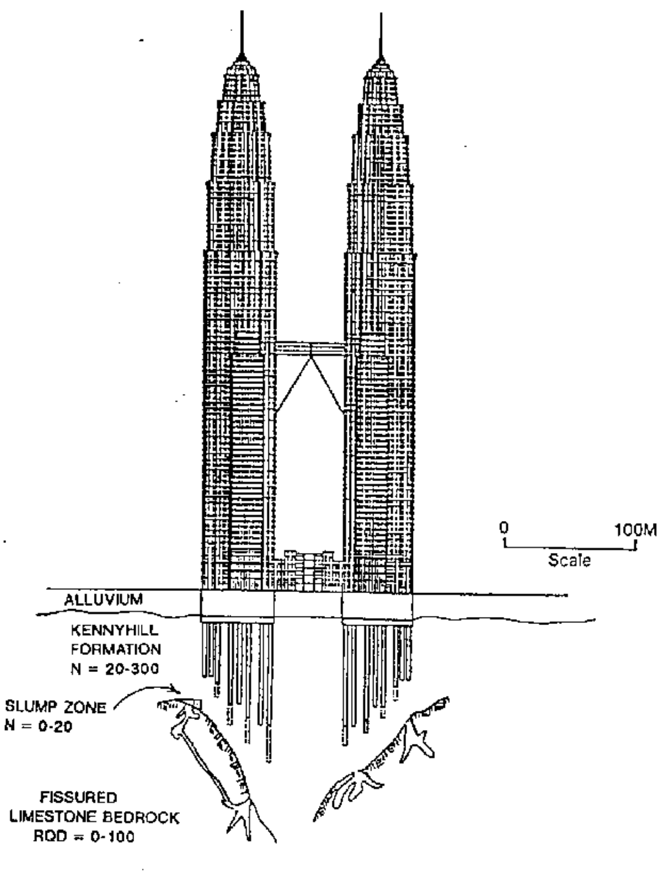


FIG. 1 TOWER FOUNDATION PROFILE

The rock surface dips steeply from northwest to southeast such that the tower bustles are situated over bedrock located 80 to 90 meters (260 to 295 feet) below street grade. The towers themselves are situated with rock at 100 to 180+ meters (330 to 590+ feet) below street grade. As shown in Figure 1, there is also a valley feature in the bedrock surface between the towers extending deeper than 200 meters. (658 feet)

Foundation Requirements

Due to the height, slenderness and structural interconnection of the towers, the developer and the designer aimed for predicted differential settlement as close to zero as practical (less than 1/2 inch , or 13 millimeters across the base of the towers).

With the anticipated geology and the goal of minimizing differential settlement, foundation alternatives studied included a "floating" raft, a system of bored piles socketed into limestone below any significant cavities, and a raft on friction piles located in the Kennyhill well above the limestone (grouting cavities and slump zones as necessary), with pile lengths varied to minimize differential settlement. The large size and great strength and stiffness requirements of a "floating" raft precluded its use. The great depth to bedrock made socketed bored piles impractical. Therefore, the friction pile scheme was used. During the preliminary design and soil exploration phase, it was found that the bedrock elevation at the initial tower locations varied so greatly that rock actually protruded into the proposed basement on one side of the tower. This made control of differential settlement impractical. The tower locations were then shifted approximately 60 meters to where the thickness of the Kennyhill formation was sufficient to support a raft on bored friction piles. There the required differential settlement limitation could be achieved by varying the length of piles or barrettes.

Exploration Program

The exploration program consisted of more than 200 borings and 200 probes on 8 meter centers in the mat areas to check for major cavities. In addition, 260 in-situ pressuremeter tests and 2 fully instrumented 3500 ton (31,000 kilonewton) pile load tests were performed to define the modulus properties of the supporting Kennyhill formation. The soil property summary and the pressuremeter test summary are shown in Tables 1 and 3 from reference 1. A representative Standard Penetration Resistance profile is shown in Figure 2. The load tests were of the Kentledge dead load reaction type with house high blocks of concrete providing the reaction. To assure maximum side friction test pile TP-1 was post grouted through machette type "skin grout" tubes attached to the rebar cage. The concrete was tremie placed (by pumping) through the bentonite slurry used to maintain open shafts, and the post grouting took place while the concrete was still "green" (within 24 hours of placement). Based on the results of the 2 instrumented load tests, which showed the much higher side friction developed on the post grouted pile (see Figure 3) the decision was made to post grout all pile or barrette foundations. The maximum side friction developed during the test was about 300 kPa.

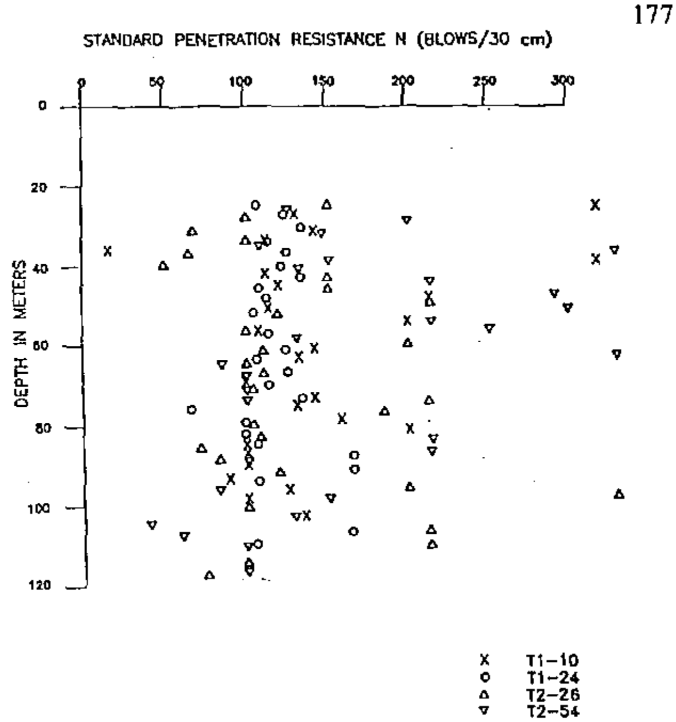


FIG. 3. Standard Penetration Resistance Profile

TABLE 1. Preliminary Geotechnical Design Parameters

Kenny Hill Formation	Zone A	Zone B
Residual Soils		
Elevation (depth below G.S.)	12±2 to 25±5	8±2 to 25±5
Bulk Density	19 kN/m³	19 kN/m³
Drained Conditions	$c' = 0,\ \phi = 30°$	$c' = 0,\ \phi = 32°$
Undrained Conditions	$c_u = 100$ kPa, $\phi = 0$	$c_u = 125$ kPa, $\phi = 0$
Deformation Modulus	75 MPa	85 MPa
Weathered Rock		
Elevation	25±5 to 30-90	25±5 to 100+
Bulk Density	20 kN/m³	20 kN/m³
Drained Conditions	$c' = 0,\ \phi' = 33°$	$c' = 0,\ \phi' = 34°$
Undrained Conditions	$c_u = 200$ kPa, $\phi = 0$	$c_u = 250$ kPa, $\phi = 0$
Deformation Modulus	100 MPa	125 MPa*

* Raised to 250 MPa after test pile program and pressuremeter test program.

FIG 2. (REF. 1)

TABLE 3. Pressuremeter Test Results

Boring	B14	B23	T1-10	T1-24	T1-54	T2-26	T2-54
E_d Min.	9.3 MPa	10 MPa	32 MPa	17.8 MPa	38.5 MPa	18.3 MPa	11.7 MPa
Max.	99	309	683	222	199.4	157	470
# of Tests	18	15	27	26	26	31	27
Avg.	37.6 MPa	133.9 MPa	67.9 MPa	109.8 MPa	101.8 MPa	64.1 MPa	149 MPa
E_d Min.	27.5	22.3	55	32	57.7	47.8	68.3
Max.	479	931	851	496	590.3	495	383.3
# of Tests	17	15	27	25	25	31	27
Avg.	186.9 MPa	391.8 MPa	176 MPa	226 MPa	223 MPa	190 MPa	535 MPa

Overall weighted
E_d Avg. = 94.3
E_d Avg. = 267

FROM REF. 1

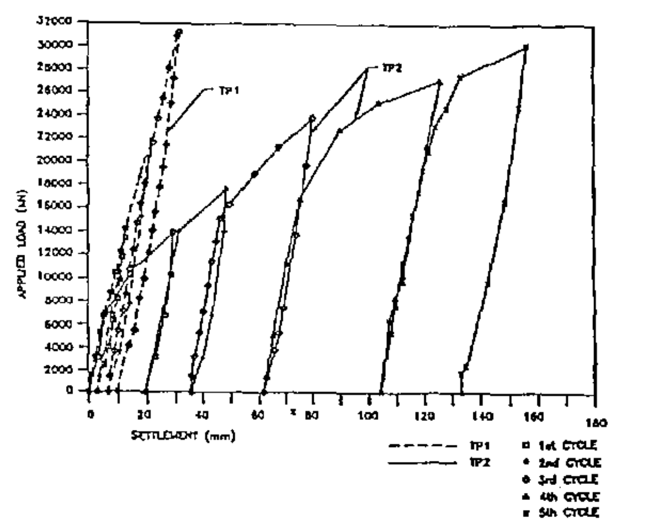


FIG. 3 RELATIONSHIP BETWEEN APPLIED LOAD WITH SETTLEMENT (TRANSDUCER) - TP1 AND TP2

Settlement Analysis and Assumptions

Extensive settlement analyses were performed utilizing the SAP 90 program and the Plaxis Axi-symetric program using soil modulus estimates based on back calculation from the test pile program and from averaging the rebound modulus slopes of the in-situ pressuremeter tests. Pile lengths were varied until calculated maximum differential settlement goals were achieved. Based on bearing capacity considerations only, barrette lengths of 33 meters would have been sufficient to support the design loads, but final pile lengths under the main towers varied from 40 meters to 105 meters based on settlement considerations. Figure 4 shows the predicted settlement and ground deformation for the final design case using Figures 15 and 16 from Reference 1.

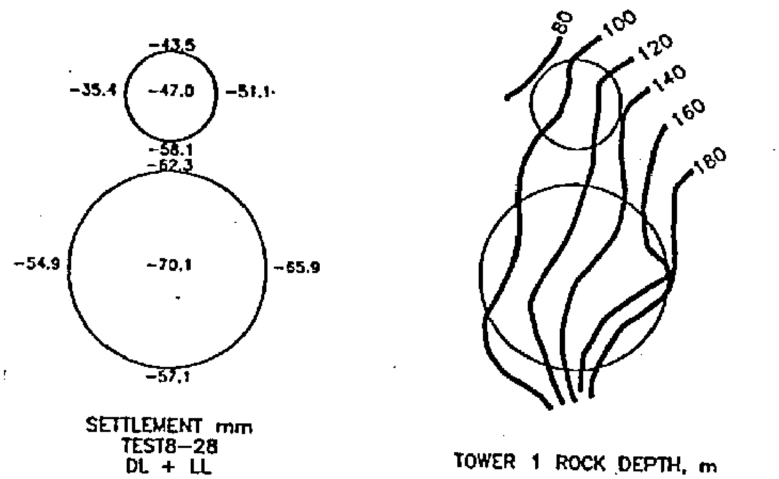


FIG. 15. Tower 1 Settlement Map and Rock Contour Plan

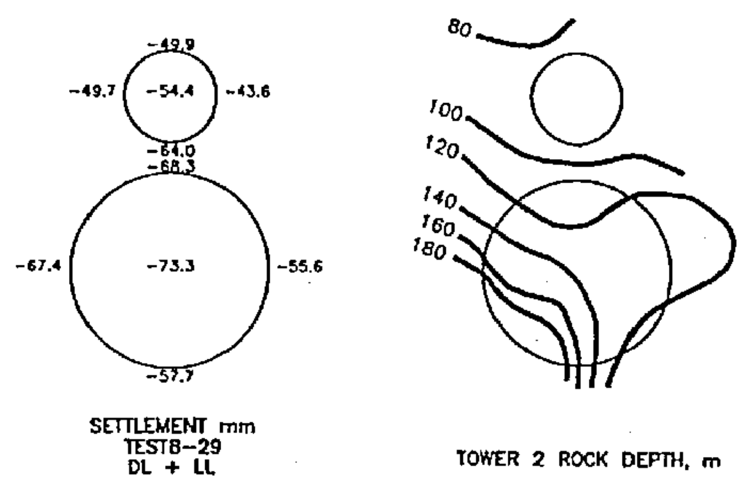


FIG. 16. Tower 2 Settlement Map and Rock Contour Plan

FIG 4. (REF. 1)

Details of both the soil property information obtained, design parameters developed and settlement analyses performed are given in Reference 1.

Required Ground Improvement

Since the boring and probing program uncovered a number of significant cavities in the limestone and slump zones at the limestone interface beneath the tower footprints, there was concern for potential unpredictable future settlement unless these zones were treated. The goal was to fill the voids in the limestone to make it relatively incompressible and to improve the slump zone areas so that they could be considered to act similar to the intact Kennyhill formation.

Ground Improvement Philosophy

The ground improvement program included fluid grouting of cavities in the limestone bedrock, and compaction grouting of the slump zone areas. Both grouting procedures were completed below the footprint of the tower/bustle areas, but not below the broader basement area.

In both towers, cavities greater than approximately 0.5 meters (1.5 feet) in cumulative thickness above a depth of 160 meters were defined as requiring fluid grout treatment. The cavity filling was designed to limit loss of overburden or cavity collapse over the life of the structure. The goal of the slump zone grouting was to increase the effective modulus of the slump zone material to approximate that of typical Kennyhill. Since the slump zones and limestone cavities are located a significant depth below the tips of the piles (depths up to 160 meters or more), the ground improvement program provides a measure of security for the foundation system and mitigates the unknown features of the karstic bedrock. A depth limit of 160 meters was set as the level below which imposed stress increases (building loads minus basement excavation effects) were considered insignificant (less than 5% of the overburden stress).

Cavity Treatment

Within the footprint of Tower 1 and its attached bustle, bedrock cavities of approximately 0.7 to 9.8 meters (2 to 32 feet) in cumulative thickness were identified from the exploration program (seven locations in the tower, and eight in the bustle). Cavities of approximately 0.4 to 14.7 meters (1.3 to 48 feet) were identified in the Tower 2 area (five each in the tower and bustle). The cavity locations and bedrock depth contours are shown in Figure 5.

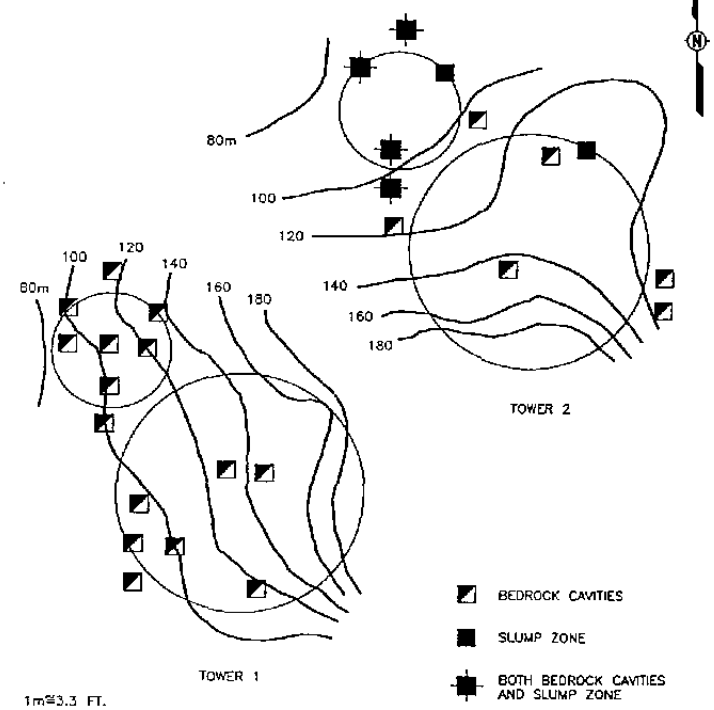


FIGURE 5. PLAN OF BEDROCK DEPTH CONTOURS AND GROUND IMPROVEMENT LOCATIONS
NTS

The standard fluid cement grout was a 0.6 to 1 water/cement ratio mix by weight, of ordinary Portland cement and water. A retarder was added to slow the set, given the significant depths of the cavities. The grout was prepared in 4 cubic meter (5.2 cubic yard) batches in an on-site batch plant erected for the work. Bulk cement was stored in silos and transferred by screw augers to weigh hoppers at the mixing vessels. The water was charged into a pair of cylindrical mixing vessels with conical bottom section, followed by the bulk cement and retarder. Large capacity, high shear pumps below the tanks mixed the grout by directing the flow around the top inside perimeter, which created a stirring action in the vessel. The cement was added via a funnel into the passing water flow.

After mixing, a 50mm (2 inch) inside diameter hose and supply/return manifold were set up near the injection hole. The grout was pumped from the plant to the hole using a Moyno helical pump, known for its steady operating pressure. During injection, a manifold pressure gage was observed. If refusal occurred, as evidenced by bentonite or grout return to the surface, or manifold pressure significantly above the equivalent frictional resistance of the hose circuit (typically 1 to 2 bars, or 15 to 30 psi), grouting was terminated. If there was no refusal after injecting a preset stage limit (typically 40 to 125 cubic meters, or 52 to 163 cubic yards), grouting was suspended for a few hours, the grout rods were flushed, and a subsequent stage was completed.

Slump Zone Grouting

In the tower areas, the design subsurface exploration was performed on an 8.5 meter by 8.5 meter (28 foot by 28 foot) grid. From that work, six areas in Tower 2 (and none in Tower 1) were identified as requiring compaction grouting in the 10 to 20 meters (33 to 66 feet) of softer Kennyhill materials just above the bedrock contact (Figure 6). At four locations in Tower 2, bedrock cavities coincided with the slump zone areas.

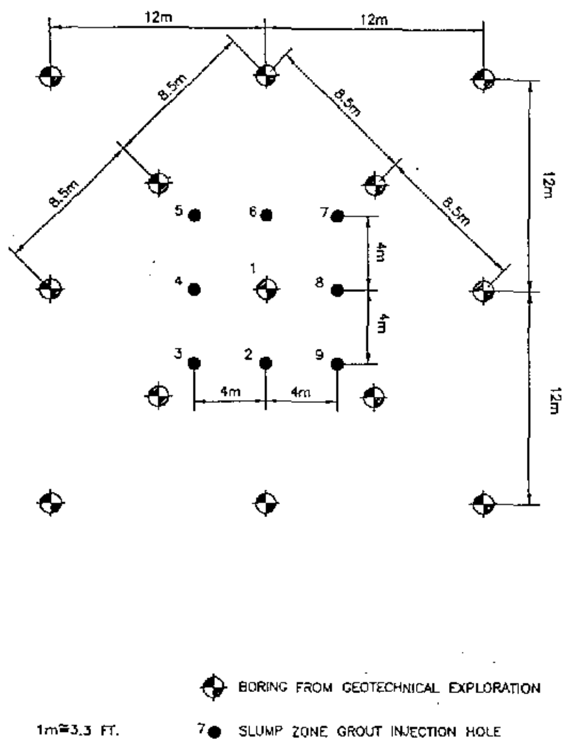


FIGURE 6. PLAN OF TYPICAL EXPLORATION BORING SPACING AND SLUMP ZONE GROUTING PATTERN.

The six slump zone areas were treated by drilling and grouting a pattern of eight injection holes on a 4 meter by 4 meter (13 foot by 13 foot) grid around the original exploration hole which was redrilled and grouted also (Figure 5). Thus, 6 x 9 or 54 injection holes were completed. Using rotary drilling procedures, the center hole was drilled first and any bedrock cavity treated with fluid grout. Then the slump zone at that location and the surrounding eight holes were treated with compaction grout in a random order. Typically, two slump zones on the Tower 2 pad were worked at any one time. The slump zone injection holes were drilled with 175 to 200 mm (7 to 8 inch) diameter roller bits and bentonite drilling fluid. The driller noted the relative ease of drilling, particularly in the previously defined slump zone above the bedrock contact. Within a single 4 meter (13 foot) square grouting pattern, the top of rock elevation varied by as little as 14 meters (46 feet) up to a maximum of 66 meters (216 feet). The zones above rock also had significant variation, with some holes indicating firm drilling all the way to rock.

After drilling, the drill rods were removed and a set of grouting rods were inserted. The bottom two sections were sleeved to increase their overall diameter and thus lessen the opportunity for the compaction grout to move up the borehole annulus during pumping. If that occurs, i.e. the grout takes the path of least resistance rather than moving outward and compressing the formation, there is little ground improvement and the rods can become bound in the hole.

The compaction grout was the equivalent of a stiff sand mortar, with a slump of 50 to 75 mm (2 to 3 inches) and made from fly ash cement, sand, water and various additives to decrease bleeding under pressure and improve pumpability. Two different suppliers were used during the slump zone grouting program, due to availability of properly graded sand, and timing of deliveries.

With the grout rods in place, a styrofoam separator was placed at the head of the grout string and the grout was pumped down the hole. A Schwing 750 concrete pump with Rockvalve was used to pump all the compaction grout. The pump was able to deliver a maximum of 60 bars (870 psi) pressure, but the maximum production grout pressure was kept at 40 bars (580 psi) at any given depth, unless a blockage needed to be cleared. With the grout rods just above the bedrock contact, the grout was pumped until a pressure of 40 bars (580 psi) was reached, or a volume of grout equivalent to about a 0.7 to 1 meter (2 to 3 foot) diameter column was injected. At that point, the grout string was raised 0.5 to 1 meter (1.5 to 3 feet) and the pumping continued. Occasionally, 2 to 4 meter (6 to 13 foot) depth increments were used when layered soft zones were noted during drilling. The grouting procedure continued until the top of the slump zone was reached and then the remainder of the injection hole was backfilled with mortar under nominal pressure, or in later holes with fluid grout from the grout plant.

Grouting Difficulties

In the cavity grouting program, the most significant problem came as a result of interconnection of the previous exploration holes with the fissured and cavity prone bedrock, caused when those holes were backfilled with sand following boring completion. To have the best chance at intercepting the desired cavity, the grout injection holes were started on top of the exploration holes. It was not uncommon to lose the bentonite drilling fluid some distance above bedrock, causing a loss of return and flushing ability to remove drill cuttings. In those cases, fluid grouting was performed using a downstage method, with the first stage completed near the depth where the drilling fluid was lost, then subsequently drilling beyond that depth to reach the desired cavities.

A secondary problem in the cavity grouting program resulted from the delivery temperature of the bulk cement. There was so much demand in the city for bulk cement at the time of construction that the delivered material still had excess heat from its manufacturing process. When combined with the depth of grouting and the elevated ambient air temperatures, grout setting time tended to decrease. This was overcome by limiting stage duration and addition of mix retarder.

The most significant difficulties came during the compaction grouting program, which did not have the advantage of an on-site batch plant which was erected later in the foundation construction sequence. The compaction grout was supplied by two different redi-mix suppliers.

Compaction grout is, by definition, a stiff, low-slump material. In order to compact the formation it must be stiff so as to maintain a bulb at the tip of the grout string during injection. The low slump leaves little room for delays during placement, since anything that contributes to delay or intermittent stoppage in pumping can cause the grout to become stuck in the grout rods. The significant depths of placement on this project lessened the delay margin and required that the grout keep moving in the grout string. On more than one occasion, inner city traffic or truck availability caused a delay in delivery and a hole had to be stopped, the rods cleaned (if possible) and replaced.

Secondly, compaction grout must not bleed water excessively under pressure or it becomes stuck in the grout string. This happened frequently during the early part of the work, and led to switching grout suppliers to obtain a finer gradation of sand. Even with the use of hydraulic extraction rams, a crane and various additives in the mix, some rods were grouted closed before they could be extracted from the hole and cleaned.

Results

In the cavity grouting program, a total of 2300 cubic meters (3006 cubic yards) of fluid grout was placed in Tower 1, and 1100 cubic meters (1438 cubic yards) in Tower 2, including the checkhole volume. To appreciate the scale and significance of this volume of grouting relative to the potential for future settlement, the 2300 cubic meters of grout used beneath Tower 1 is equivalent to filling a void 1 meter thick under the entire 54 meter diameter tower mat. Quality control was accomplished by 1) proportioning of mix ingredients during batching; 2) preparation of fluid grout test cylinders; and 3) drilling of checkholes after grouting.

Compressive strength tests of grout cylinders averaged 21.6 N/sq. mm (3130 psi) for Tower 1, and 20.2 N/sq. mm (2930 psi) for Tower 2. These results exceeded the design requirement of 17 N/sq. mm (2470 psi).

The checkhole program included one to three borings adjacent to each injection hole. If the cumulative cavity thickness (as defined by the original geotechnical exploration) was less than 4 meters (13 feet), one checkhole was performed. Where the cumulative cavity thickness was more than 4 meters (13 feet), three checkholes were drilled in a triangular pattern around the injection hole. The checkholes were extended below the depth of the lowest documented cavity. Depending on the grout and/or remaining cavities observed in the checkhole rock cores, additional fluid grouting was performed through the checkhole drill casing, or, if the cavities were filled, the checkholes were grouted closed.

In most cases, backfilling or additional stage grouting in the checkholes was accompanied by bentonite or grout return to the surface, indicating they were filled. At locations such as T1-8 at the perimeter of the Tower 1 bustle, there was originally 9.8 meters (32 feet) of cumulative cavity thickness from 143 to 164 meters (470 to 540 feet) below grade. After several stages of grouting, completion of three checkholes (one with secondary grouting), it was decided to terminate that hole. The decision was based on the total volume injected at that location (545 cubic meters, or 595 cubic yards), the observation that the uppermost and largest cavity at 143 to 148 meters (470 to 485 feet) was almost entirely filled, and the significant depth from the tip of the future barrette pile at that location to the top of bedrock (80 meters or 260 feet).

In the slump zone grouting program, 900 cubic meters (980 cubic yards) of compaction grout was placed (not including the backfill volume). Quality control procedures included casting of 150 x 150 x 150 mm (6 x 6 x6 inch) mortar cubes for compressive strength testing at 7 and 28 days, and completion of two checkhole borings in each 9-hole injection pattern, with SPT tests performed at 3 meter (10 foot) intervals in the slump zones.

The compressive strength tests of mortar cubes generally exceeded the design requirement of 17 N/sq. mm (2500 psi).

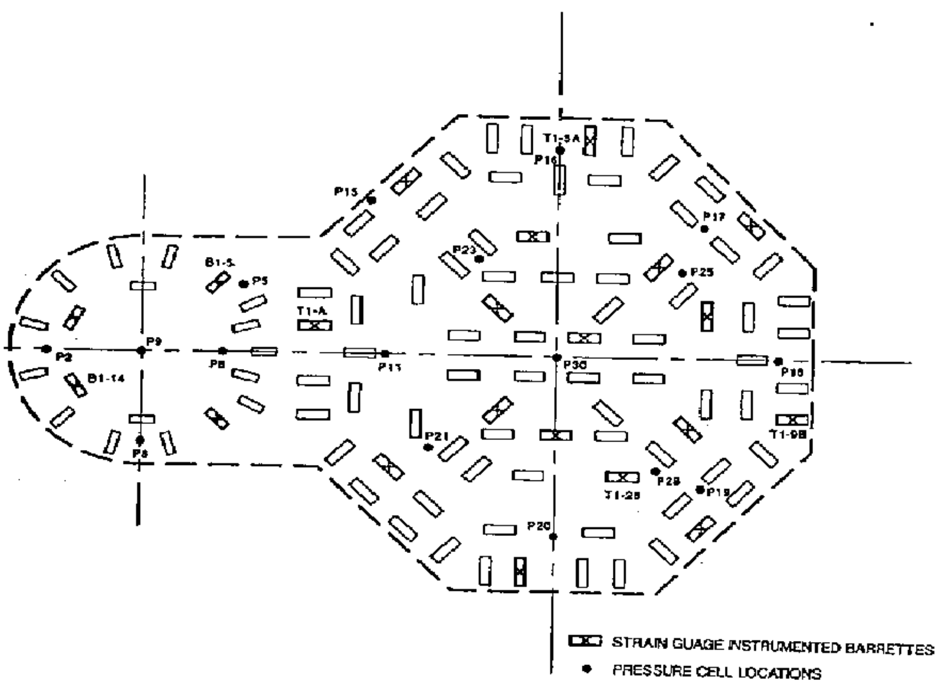
Perhaps as a result of the significant variation in bedrock elevation within each slump zone injection pattern, the checkholes provided only a semi-quantitative evaluation of the compaction grouting performance. As expected, where the drilling was firm, there was little grout take, and vice versa in the softer zones. It was not uncommon for two injection holes 4 meters (13 feet) apart (assuming parallel holes) to have significantly different grout take. In most cases, there was improvement in SPT values when the soft zone in a surrounding hole was similar to that identified in the original injection hole. Of course, all of the pre-grouting SPT data came from the center hole in each slump zone pattern, since that was the location of the original exploration hole. Where there was little improvement in SPT value, a significant equivalent volume of grout still was injected through that zone which helped increase the average theoretical equivalent modulus of the material.

Foundation Installation

Barrette Foundations 1.2 meter by 2.8 meters and 0.8 x 2.8 meters in section, for the towers and bustles respectively, were installed from an initial partial excavation level of -4 meters below grade to depths varying from 60 meters to 130 meters below grade utilizing slurry trench cutting machines of the hydro fraise type with counter rotating cutting wheels for the deeper foundations. The cuttings mixed with bentonite slurry were pumped to a desanding plant for regeneration and eventual recycling to the barrette shaft. By installing the barrettes prior to the major basement excavation to -25 meters, the barrettes acted to resist soil heave, destructuring and softening of the Kennyhill formation when the basement excavation was made. A 4.5 meter thick mat was constructed on top of the barrettes to transfer load from the core and perimeter columns to the barrettes.

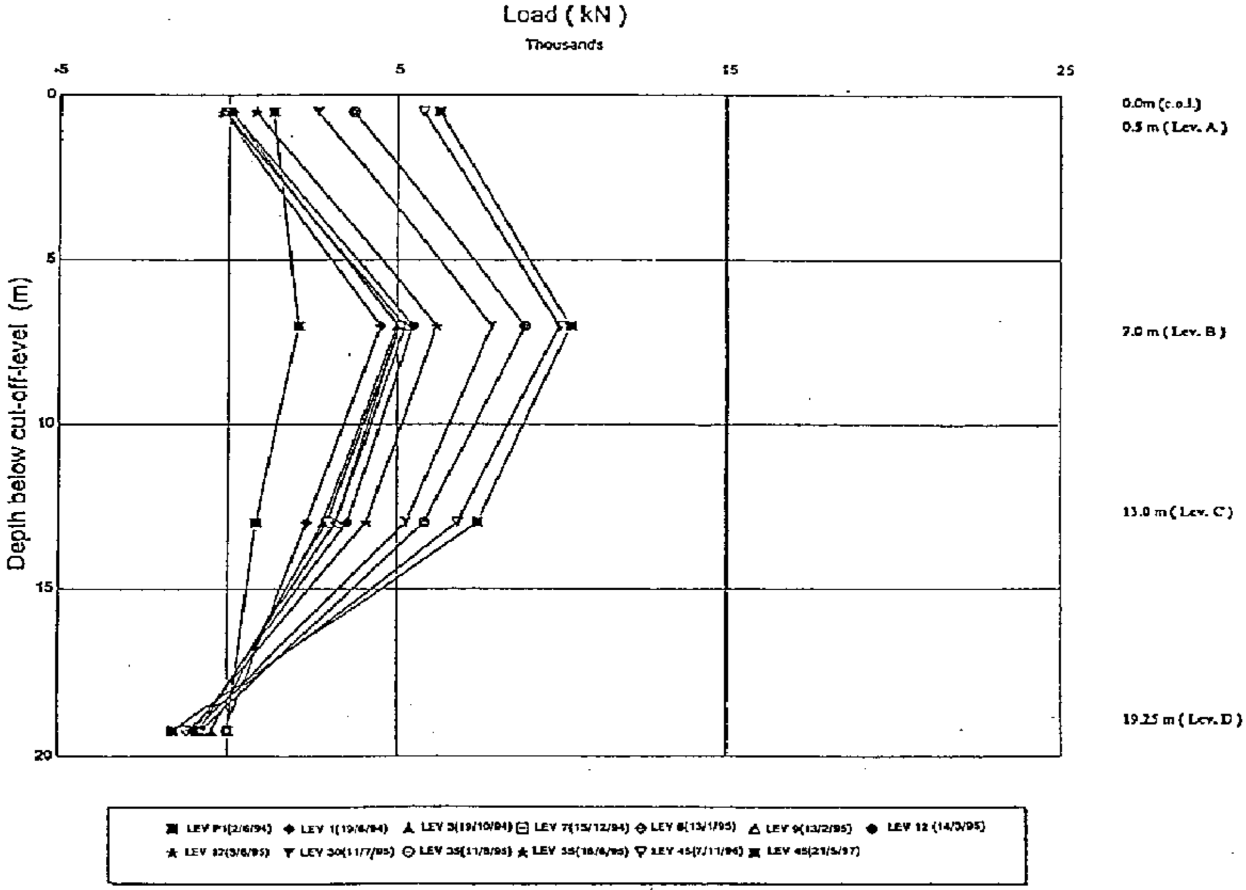
Instrumentation Program

The foundation instrumentation program for each tower consisted of instrumenting 21 representative barrettes and also installing 30 pressure cells beneath the mat at representative locations as noted in Figure 7. The Barrette instrumentation program consisted of two vibrating wire strain gages with thermistors and resistance wire strain gage at levels that varied in elevation from a minimum spacing of 7.5 meters to a maximum of 13 meters. Representative plots of calculated load versus depth as the tower was constructed are shown in Figures 8 to 13.



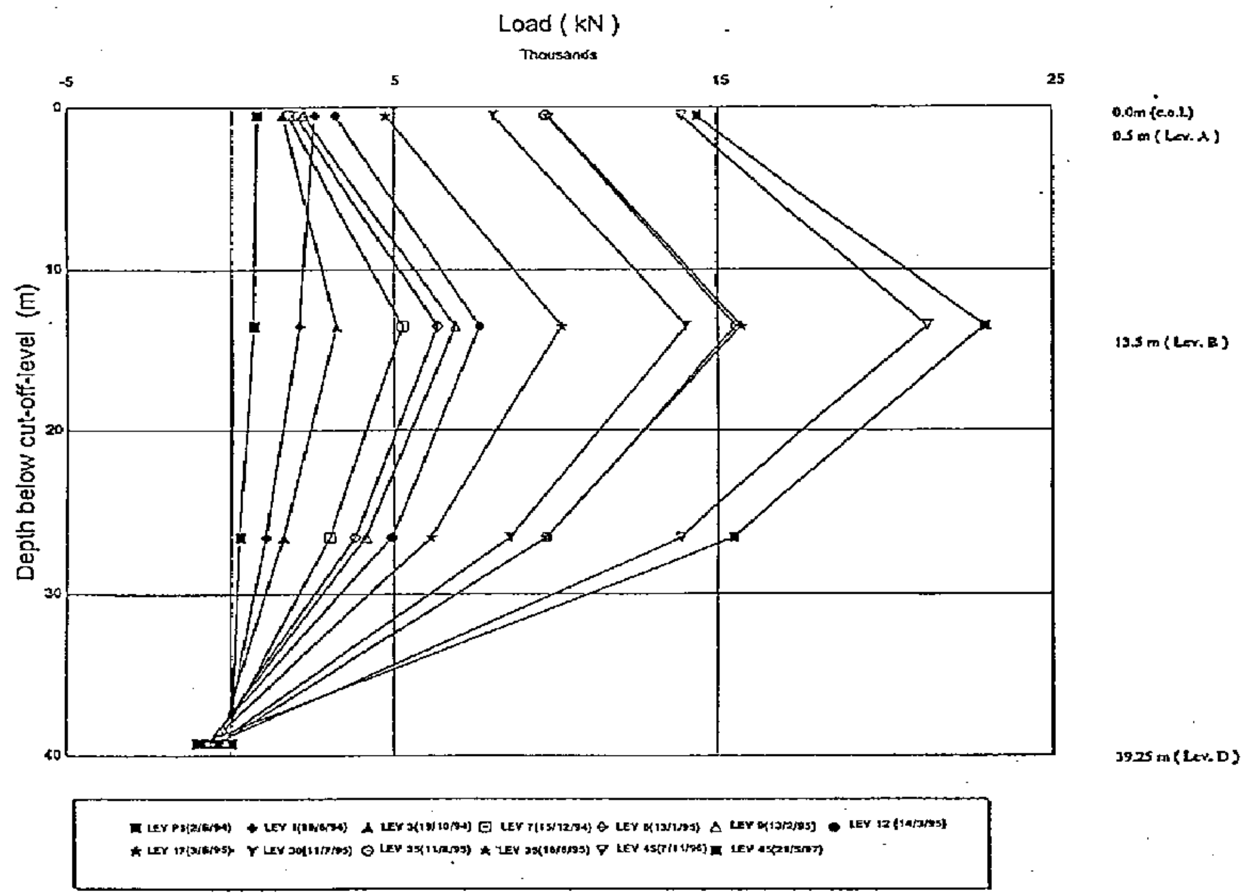
TOWER 1: LAYOUT PLAN OF INSTRUMENTED BARRETTES

FIG. 7



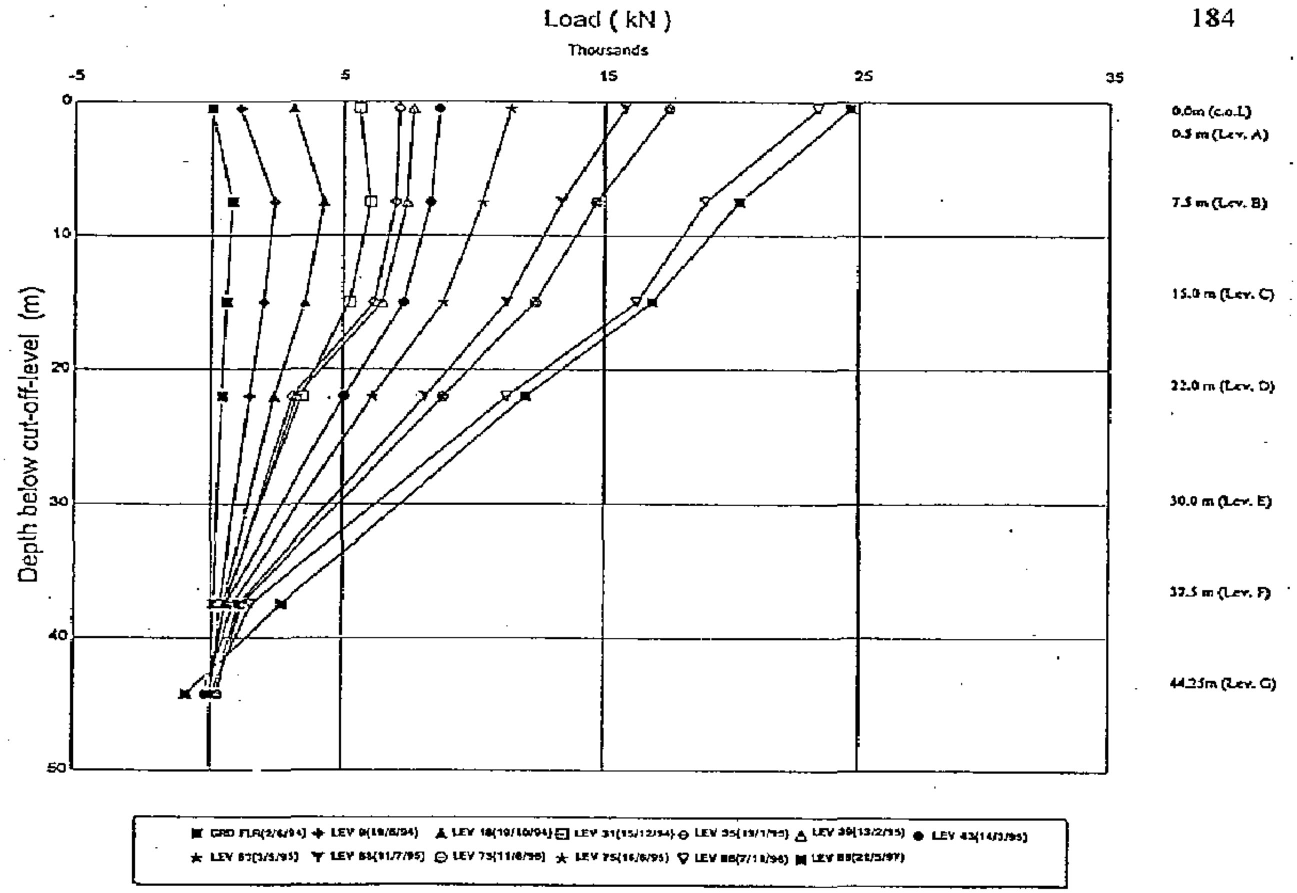
B1-14(SG): LOAD DISTRIBUTION CURVE
COMPUTED FROM SGs MEASUREMENT

FIG. 8.



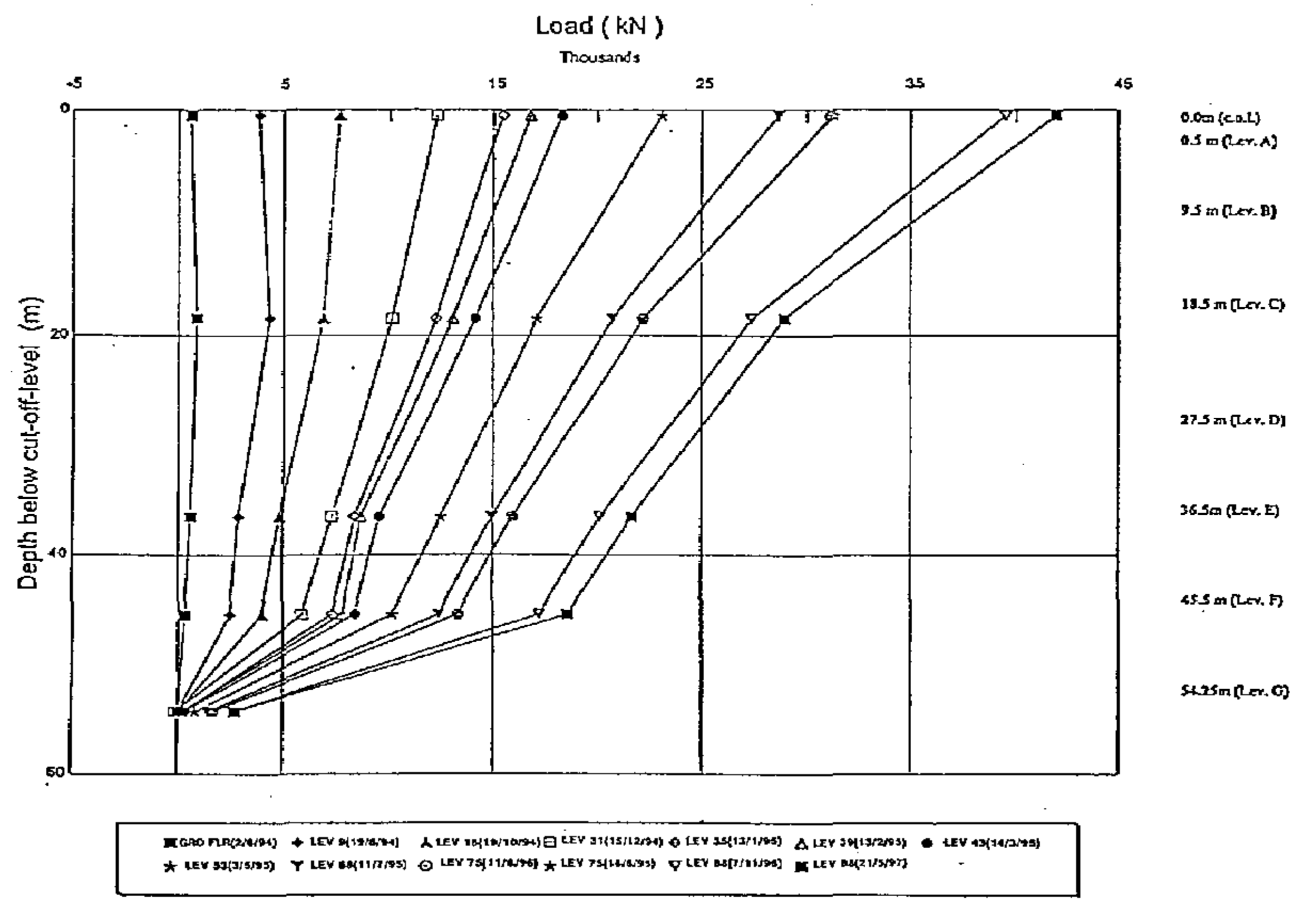
B1-5(SG): LOAD DISTRIBUTION CURVE
COMPUTED FROM SGs MEASUREMENT

FIG. 9



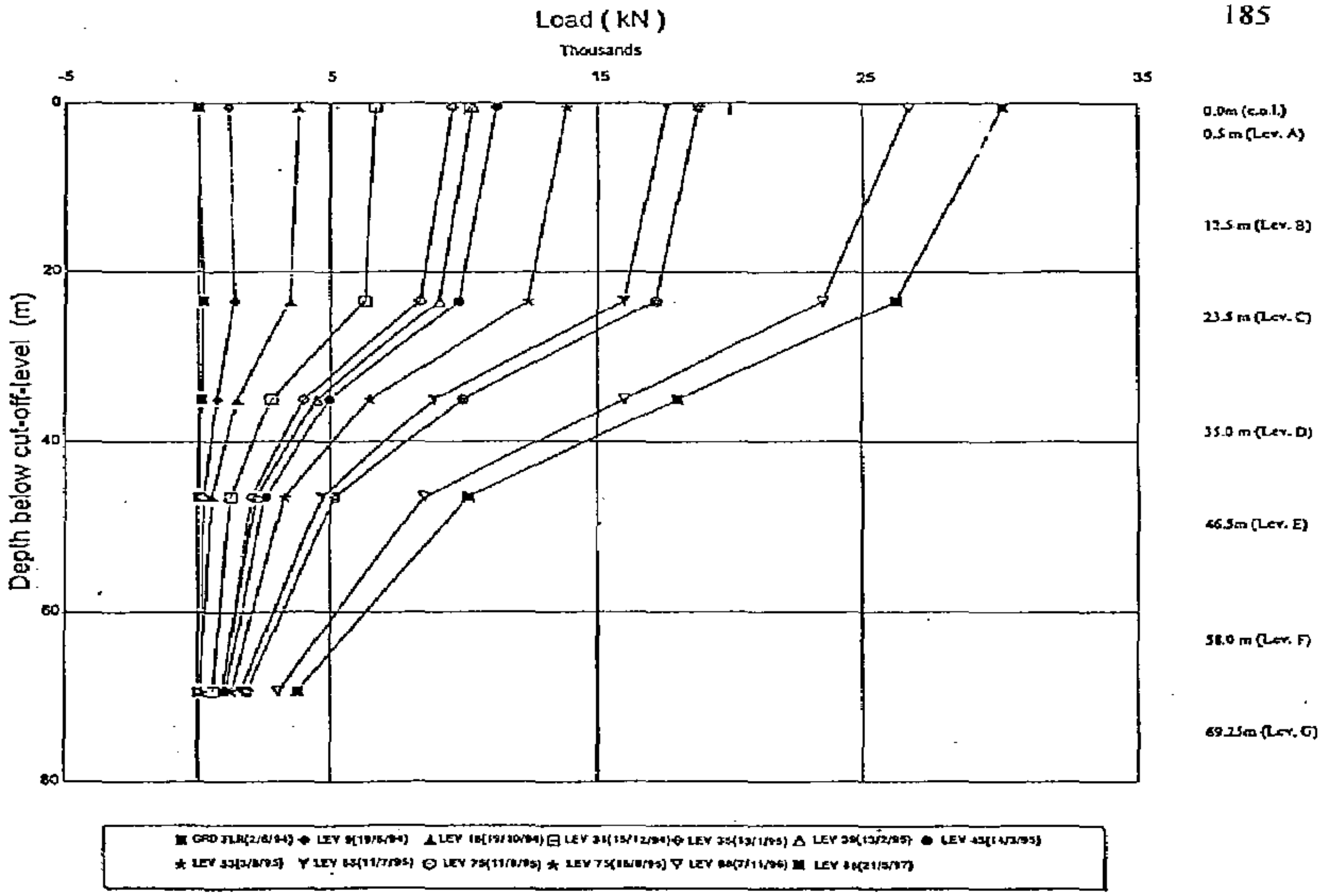
T1-1A(SG): LOAD DISTRIBUTION CURVE
COMPUTED FROM SGs MEASUREMENT

FIG. 10.



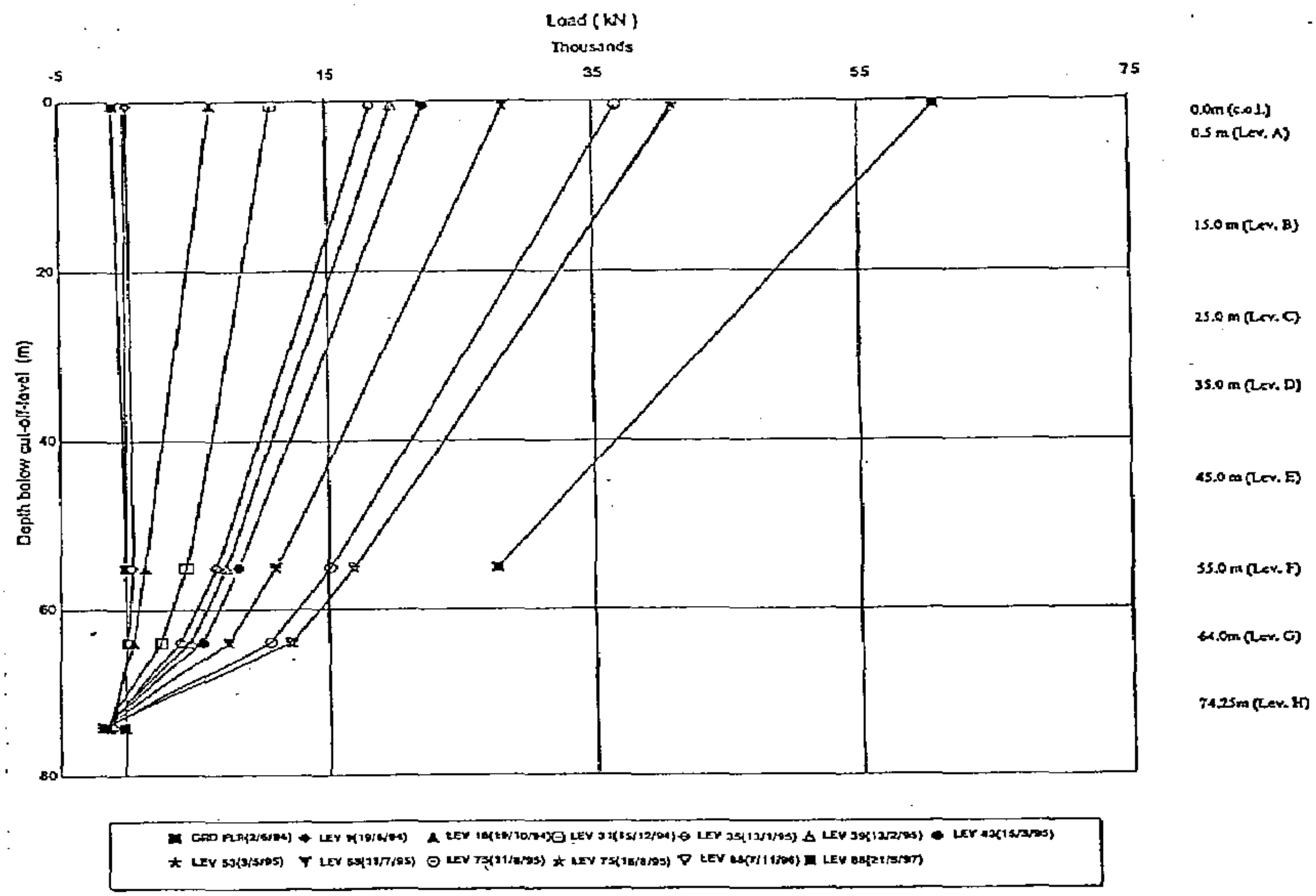
T1-2B(SG): LOAD DISTRIBUTION CURVE
COMPUTED FROM SGs MEASUREMENT

FIG. 11.



T1-9B(SG): LOAD DISTRIBUTION CURVE
COMPUTED FROM SGs MEASUREMENT

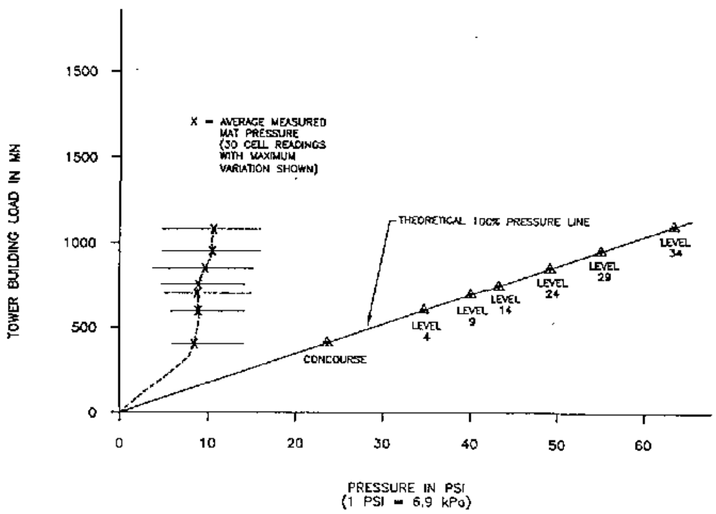
FIG. 12



T1-5A(SG): LOAD DISTRIBUTION CURVE
COMPUTED FROM SGs MEASUREMENT

FIG. 13

The 30 pressure cells placed at representative locations beneath the mat measured pressure in pounds per square inch. The average pressure recorded, along with the range in pressure, is shown versus building load in Figure 14 for Tower 1. Also shown on the figure for comparison purposes is the theoretical pressure based on the average building load over the mat foundation area.



RELATION OF AVERAGE MAT PRESSURE CELL READINGS AND BUILDING LOAD

FIG. 14

The load settlement history of Tower 1 during construction up to about 70 floors is shown on Figure 15. The time settlement record through completion of Tower 1 and partial occupancy up to March 19, 1997 is shown in Figures 16 and 17. The maximum reported average settlement for the core is about 35 millimeters with maximum reported differential settlement of 7.0 millimeters. This is approximately 1/2 of that predicted in Reference 1 where a maximum settlement of 72 millimeters and differential settlement of 12 millimeters was predicted based upon an assumed modulus for the Kennyhill formation of 250 MPA.

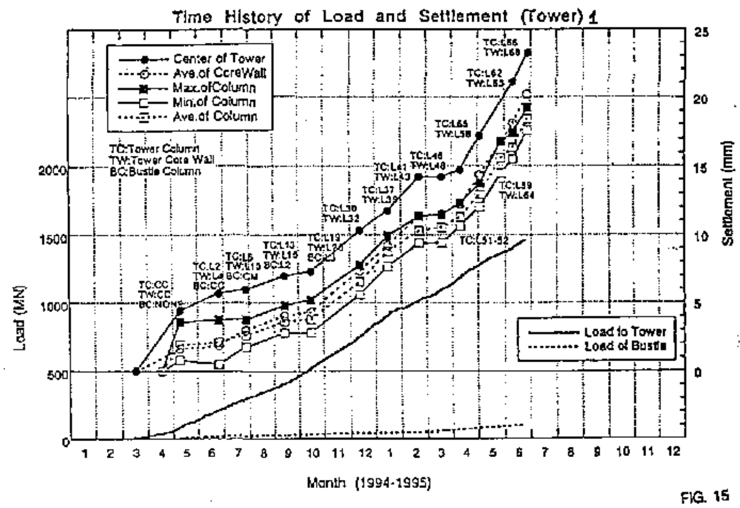


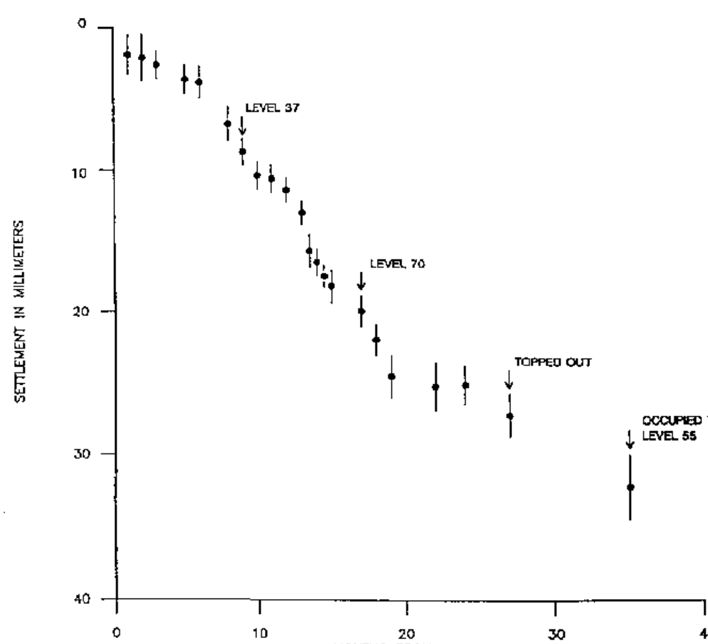
FIG. 15

Analysis of Instrumentation Data

The data available from the instrumentation program for both towers is immense and will be studied and reported on in due time by the local geotechnical engineers. The selective data presented here for Tower 1 provides a preliminary picture of the general results. A very high percentage of the installed instrumentation worked consistently throughout the construction. The data available on strain gage readings is complete through early Spring 1997 when the full load of the buildings was in place. The pressure cell information is only available through construction up to about the 35th level with less than half the full building load in place.

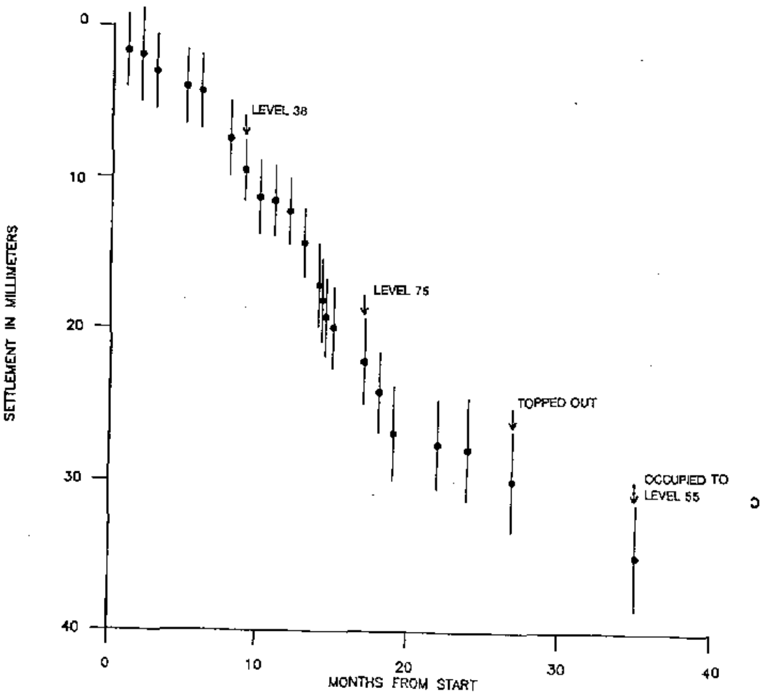
Performance Evaluation

Predicted maximum settlement for the completed towers was 73 mm, (2.8 inches) with maximum differential across the mat of 12 mm (0.5 inches). Based on settlement measurements taken during construction, it appears that both measured total and differential settlements of the towers are less than predicted, indicating that the goals of the deep ground improvement program were met.



SETTLEMENT OF TOWER 1 COLUMNS

FIG. 16



SETTLEMENT OF TOWER 1 CORE FIG. 17.

It should be noted that part of the reported differential settlement is suspect since the major portion (about 2/3rds) was reported immediately after pouring the concrete mat before significant additional load had been applied. Thus the level of reading reliability may be on the order of 2 to 3 millimeters.

The strain gage data from the barrettes indicate considerable variation in how the load is transferred down the barrettes indicating the variable nature of the Kenny Hill formation. In some of the barrettes, all the load is transferred in friction with no load reaching the tip or lowest strain gage level. However, in other barrettes significant load reaches the tips even when the barrettes are as long as 75 meters. Maximum developed friction along the piles varied from 73 kPa to 270 kPa. This is consistent with the test pile program where maximum developed friction reached over 300 kPa with no signs of slippage (Reference 1). It should be noted that even in those barrettes where some load is reaching the tip, the major portion of each new load is still being carried in friction.

With regard to the portion of the load carried by the mat, it appears to be in line with what would be expected by normal elastic theory. Initially, for small loads, the load is shared between mat and piles. With greater loads, a disproportionate amount appears to go into the piles This may be due to the fact that the upper portion of the Kennyhill just below the mat is typically softer than the deeper portions even though a uniform modulus was assumed in the design calculations.

In evaluating the foundation design and performance, the question needs to be asked as to why the settlement is only approximately 1/2 that predicted when extensive in-situ testing was performed including 2 full scale instrumented load tests and 260 in-situ pressuremeter tests. The writers offer no conclusions in this regard but suggest that the Kennyhill at the instrumented pile load test locations may be slightly weaker and more compressible than the Kennyhill at the actual tower locations. In addition, it may be that small scale in-situ pressuremeter tests can not accurately model large scale Kennyhill performance wherein the weathered rock structure varies drastically and the stiffness offered by the harder layers may not be adequately reflected by averaging the test results.

Acknowledgments

The ground improvement program was performed by Bauer Malaysia ,with Hayward Baker of Odenton, Maryland serving as a consultant and subcontractor for the compaction grouting. An international joint venture which included Bachy Soletanche installed the barrette pile foundations. The local geotechnical engineer and building engineer of record, Ranhill Bersekutu was responsible for the foundation design and assisted in overseeing the grouting program. Thornton Tomasetti of New York in association with Ranhill Bersekutu Sdn Bhd were responsible for structural design. The grouting program design and oversight, and peer review of the foundation design were completed by STS Consultants, Deerfield, Illinois. The barrette instrumentation sub contractor was Spectest SPN BHO. The project developer is Kuala Lumpur City Centre Berhad. The contractor for Tower 1 is a consortium comprising Hazama Corporation, J. A. Jones Construction Company, MMC Engineering Services Sdn Bhd, Ho Hup Construction Company Berhad and Mitsubishi Construction. The contractor for Tower 2 is a consortium comprising Samsung Engineering & Construction Co. Ltd., Kuk Dong Engineering & Construction Co. Ltd. and Syarikat Jasatera Sdn Bhd.

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