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Ingo Fox
Malcolm Pirnie, Inc., New York, NY

Mangtao Du
TAMS/Earth Tech, New York, NY

Stephen Buttling
Earth Tech, Bangkok, Thailand

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DEEP FOUNDATIONS FOR NEW INTERNATIONAL AIRPORT PASSENGER TERMINAL COMPLEX IN BANGKOK

Ingo Fox, P.E.
Senior Associate
Malcolm Pirnie, Inc.
New York, NY

Mangtao Du, P.E.
Senior Geotechnical Engineer
TAMS/Earth Tech
New York, NY

Stephen Buttling, Ph.D.
Project Manager
Earth Tech
Bangkok, Thailand

ABSTRACT

The foundation of the terminal in the Second Bangkok International Airport consists of over 25,000 bored piles (drilled shafts) and prestressed concrete cylinder driven piles installed in a complex stratified subsurface formation of soft and stiff clays interbedded by silty sands. A train tunnel to be located underneath the main terminal is a cut-and-cover tunnel supported on its sides by 1-meter thick reinforced concrete diaphragm walls and at the bottom a 2-meter thick basal slab. Particular technical challenges existed in the long-term settlement and potential downdrag caused by changes in effective overburden stresses as a result of excessive pumping of the deeper aquifers in Bangkok area. Long term performance of the terminal buildings would be threatened by potential differential settlements around heavily loaded Trellis roof pylons. To address these problems, innovative analytical procedures were developed and applied using advanced theories and numerical methods involving pile load transfer mechanisms, numerical integration of closed-form solutions to the Mindlin's Problems, Finite Element Analyses, as well as a field pile load test program including load-transfer measurements and bi-directional loading to validate the design. The pile load test program has been completed and demonstrated the validity of design parameters and assumptions. All of the 25,437 piles were installed in 12 months, 50% of the time available, and with progress peaking at about 4,000 piles a month.

The paper will describe a project overview, geology and subsurface conditions, design of foundation systems and the diaphragm wall, load tests, foundation construction, settlement measurements and evaluations, and will be concluded by discussing valuable experiences obtained as well as lessons learned.

INTRODUCTION AND PROJECT DESCRIPTION

The Second Bangkok International Airport (SBIA) contains the largest passenger terminal project being undertaken in the world. Planning and design of the airport facilities included the 0.5 million m^2 multilevel passenger terminal; landside terminal frontage; an 8,600-metre departure/arrival roadway system; two 2,500-car multilevel parking garages; and recirculation roadways within the airport complex area. A general plan of the terminal complex is shown in Figure 1.

The foundation of the terminal consists of over 25,000 bored piles (drilled shafts) and prestressed concrete cylinder driven piles (spun piles) installed in a complex stratified subsurface formation of soft and stiff clays interbedded by silty sands. A train tunnel and station to be supported by reinforced concrete diaphragm walls will be located directly underneath the main Terminal. Technical challenges existed in the long-term settlement and potential downdrag caused by changes in effective overburden stresses as a result of excessive pumping of the deeper aquifers in the Bangkok area. Long

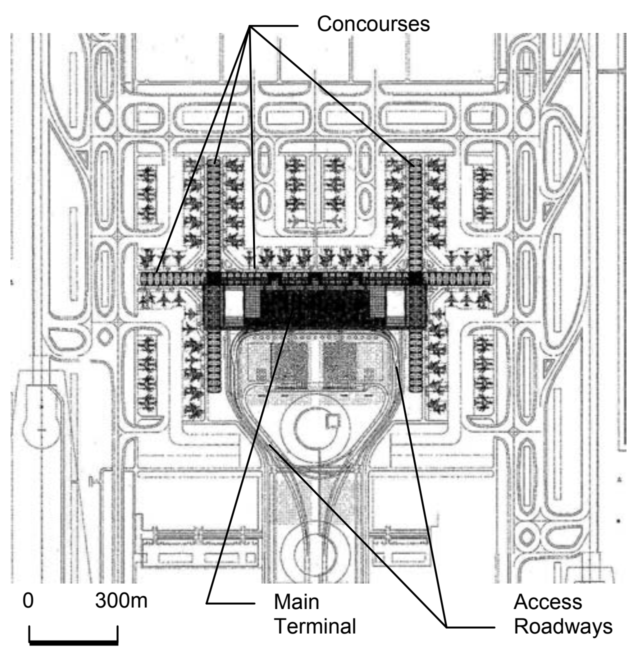

Fig. 1. General Terminal buildings plan

term performance of the terminal structures was also threatened by differential settlements around the heavily loaded Trellis Roof pylons, see Figure 2. Analytical procedures were developed using well established theories and numerical methods involving pile load-transfer mechanism, numerical integration of closed-form solutions to the Mindlin's Problems, and Finite Element Analyses. Based on these analyses, a foundation scheme was developed involving three different pile types, namely, the 600 mm diameter spun pile, the 600 mm diameter bored pile, and the 1000 mm diameter bored pile.

For design verification and construction quality assurance, a comprehensive pile load testing program was carried out at the beginning as part of the design effort and throughout the duration of pile construction. The load test program included measurements of pile load-transfer, an essential element in the design analyses. As construction of the superstructures progresses, foundation settlements are being monitored, providing a direct check to the design.

This paper examines design aspects, construction, testing, and monitoring for the deep foundations of this project.

Fig. 2. Model of the new Terminal showing Trellis Roof supported by pylons

GEOLOGY AND SUBSURFACE CONDITIONS

The new airport is located 25 km south east of Bangkok on the vast Chao Phraya Plain. Much is known about the subsurface conditions for this Plain since Bangkok is located thereon as is the large transportation network that radiates from Bangkok. Part of the elevated road system and a train line of that network pass by the SBIA site.

Strata

The soil formation consists typically of a multi-layered soil sequence of marine clays consolidated to different degrees of consistency, depending on depth, interbedded by medium dense to dense silty- and clayey-sands of variable thicknesses.

The site for the most part was stripped of the surface layers of topsoil and desiccated silt-clay soil in the course of constructing ponds for agriculture and pisciculture. The surface stratum essentially consists of soft to medium clay, 15 to 18 m in thickness that is highly compressible. This surface stratum is underlain by alternating harder clay and dense sand strata. Bedrock in the Bangkok area underlies the sedimentary deposits at a depth varying between 500 to 2000 m.

As a consequence, the foundation system of choice for the Terminal structures is bored and driven piles that transfer structural loads to the more competent strata beneath the soft surface stratum. Typically, pile foundations on the Chao Phraya Plain were designed to engage the upper 40 to 60 m of the sedimentary deposits.

The subsurface characterization at the site was obtained from 19 test borings performed in the design phase and additional 20 borings and 7 Cone Penetrometer Tests performed in the construction phase. The construction phase subsurface investigation was carried out in an attempt to better define the subsurface condition and to disclose any anomalies within the Terminal site that is over one square kilometer in area, the whole airport site being 32 square kilometers. Figure 3 shows the typical soil profile with a schematic foundation section.

Engineering properties of the soil strata pertinent in the bearing capacity and settlement analyses were obtained from field and laboratory testing, as well as the abundant local engineering experiences and available soil parameters obtained at sites with very similar subsurface conditions.

Piezometric Heads and Effective Stresses

Beneath the project site, there are several aquifers, which currently are being pumped to a depth of 100 to 200 m below the ground surface. The pumping has created piezometric drawdown conditions in the dewatered aquifers producing changes in effective stresses in the various soil layers. The change in effective stresses has been responsible for the ongoing overall subsidence of Bangkok and surrounding areas.

The current effective stress profile was calculated from piezometric measurements obtained from installed piezometers in the project site. The measurements confirmed the effect of deep well pumping. A marked reduction of the normal hydrostatic pore pressures exists in the lower portion of the soft clay layer and continuing into the first stiff clay layer, which overlies the first dense sand aquifer. This is shown in Figure 4. The calculated effective stresses from measured piezometric heads, agree well with effective stress measurements elsewhere in the Bangkok area (Thasnanipan et al., 1998.)

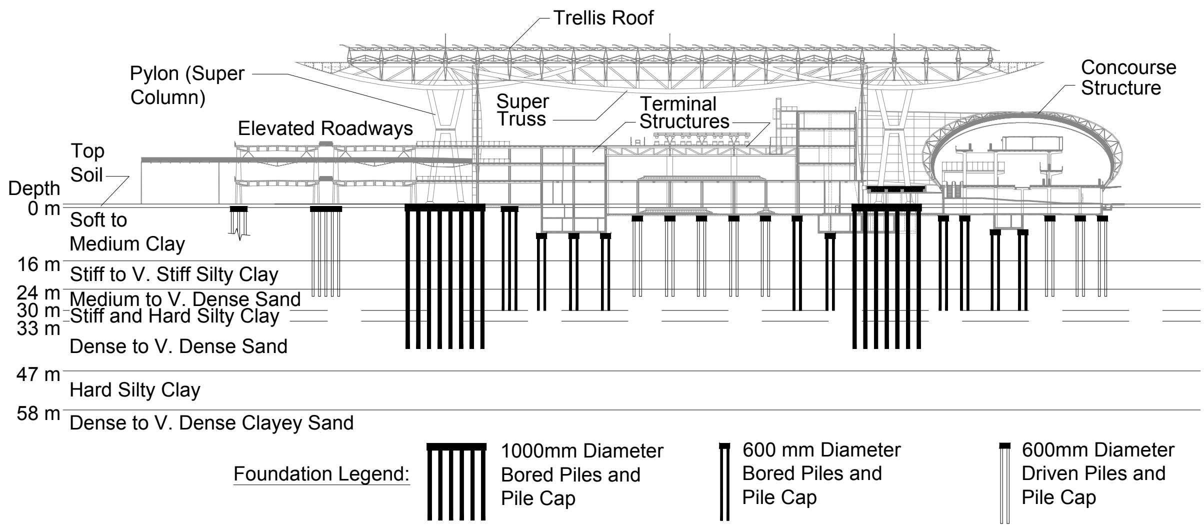

Fig. 3. Schematic foundation section and typical soil profile

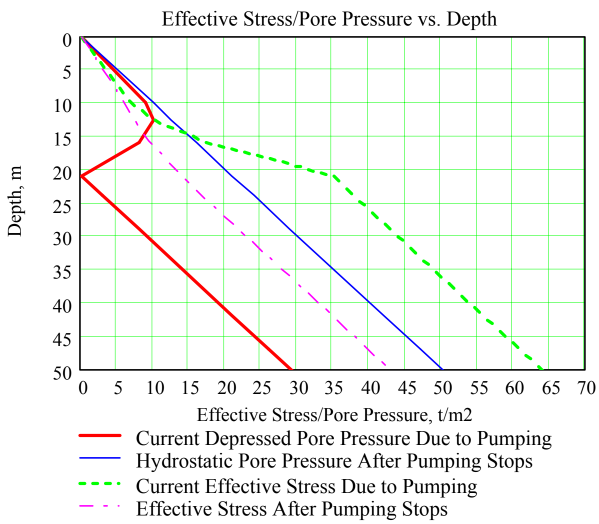

Fig. 4. Effective stress and piezometric conditions

As a result of deep well pumping, the current effective stresses are higher than the effective stresses corresponding to piezometric heads that are hydrostatic from the ground surface (Figure 4). This phenomenon has a significant effect on the pile design. As presented below, the governing pile bearing capacities were determined based on the hydrostatic piezometric heads, ignoring the effects of drawdown from deep well pumping. This may appear to be a conservative assumption, but may become the case in time as deep well pumping is either stopped or shifted from the area due to relocation of the industrial activities and new water supply from water treatment plants.

In fact, government policies have been directed towards reducing, and perhaps eventually eliminating, groundwater extraction in the Bangkok area. Measurements near the new airport site indicate significant rising in piezometric heads since 1997 in the aquifers 100 m to 200 m below grade, a reverse in the long-term trend of piezometric drawdown. As a result, the regional settlements have slowed down significantly: between 1981 and 1998 the cumulative settlement was about 150 mm, but it has essentially stopped since 1998.

DESIGN OF FOUNDATION SYSTEM

Conceptual planning and developments of SBIA started in the 1980's. The design work for the Terminal began in 1995 and was completed in 2000.

Apart from the local pile construction technology available in Bangkok, choice of pile type, diameter, and tip elevations revolved entirely around meeting the differential settlement criteria given by the project structural engineer and architect. The major concerns for the foundation design were (1) the long-term settlement and potential downdrag caused by changes in effective overburden stresses as a result of excessive pumping of the deeper aquifers and (2) potential differential settlements around the heavily loaded Trellis Roof pylons ("Super Columns", Figures 2 and 3), delivering up to 15,000 metric tons of vertical load.

Analysis of Pile Load-Transfer

Not only is knowledge of the load-transfer mechanism essential in determining the pile capacity and the drag load in the pile, it also provides a basis for the settlement analysis for the pile foundation. The effective stress analysis, known as the β-method, was used since the load-transfer between a pile and the soil is essentially governed by the effective stress behavior.

Because the design of the foundations is controlled by settlement considerations it became even more relevant to understand the pile load-transfer into the subsurface. The approach of pile load-transfer used in the analysis was the one presented by Fellenius, 1996. The approach, as illustrated in Figure 5, involves the concept of a Neutral Plane.

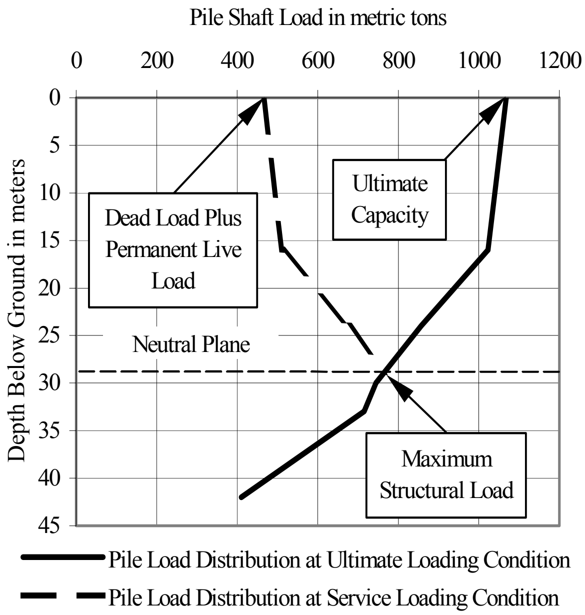

Fig. 5. Example of load transfer analysis

Under service conditions, loads from the structure will be applied to the pile head. Even if soil settlements are small, the soil will in most cases move downwards in relation to the pile and in the process transfer load to the pile by negative skin friction. This is inevitable due to the difference in stiffness between the soil and the piles. Therefore, every pile develops an equilibrium of forces between (1) the sum of permanent load (Dead Load plus "permanent" Live Load) applied to the pile head and drag load induced by negative skin friction in the upper part of the pile, and (2) the sum of positive shaft resistance and toe resistance in the lower part of the pile. The point of equilibrium, called the Neutral Plane, is the depth where the shear stress along the pile changes over from negative skin friction into positive shaft resistance. This is also where there is no relative displacement between the pile and the soil.

Design for the bearing capacity includes applying a Factor-of-Safety to the pile capacity in the soil/pile interface. The capacity is determined considering positive shaft resistance developed along the length of the pile plus full toe resistance, as would be the case in a load test where the pile is loaded to its ultimate bearing load. The loads to be considered in calculating the safety margin against a bearing capacity failure consist of permanent and temporary loads, but no drag load, because the drag load does not affect the bearing capacity of the pile. The pile cannot fail under the negative skin friction. Instead, all negative skin friction will have to reverse into positive skin friction before a pile can fail at its ultimate geotechnical bearing capacity. Downdrag is essentially a settlement issue, as well as a structural strength issue as described below, and was treated as such.

Design for structural strength includes consideration of the conditions at the pile head and at the Neutral Plane. At the pile head, the loads consist of permanent and temporary loads combined with bending at the pile head; and, at the Neutral Plane, the loads consist of permanent load and drag load, but no live load, and no bending in most cases.

Bearing Capacities and Maximum Structural Loads

As described in the subsection entitled "Piezometric Heads and Effective Stresses", the capacities were calculated for two effective stress conditions (Figure 4): (1) using existing piezometric heads which reflect deep-well pumping, and (2) using piezometric heads that would obtain in the future if pumping is stopped and piezometric levels rise to pre-pumping (hydrostatic) levels. The results of capacity calculations are given in Table 1. The effective stresses under (1) are higher than under (2), and so are, therefore, the load carrying capacities.

Based on the rationales presented above, the governing design criteria are the bearing capacities in the lower half of Table 1 and the maximum structural loads in the upper half of Table 1. The pile capacity calculations took into account the differences in load-transfer parameters between bored and driven piles regarding shaft friction and end bearing.

Selection of Pile Types

Pile type selection was based on both cost and technical factors. Costs per metre for the various pile types are given in Table 2. The estimated costs for 600-mm spun piles in tension took into account the extra cost for providing a suitable splice, the cost for cutoffs and the extra cost to excavate around the piles to achieve cutoff level. However, because of pile construction efficiency and to simplify construction of pile caps and below-ground structures supported thereon, bored piles were preferred and selected for all piles subject to tension. Matters of concern leading to this decision were reliability of splices in driven piles, cut-offs required for driven tension piles and for bored tension piles, and more time consuming and costly excavation to excavate to footing level for driven piles.

The cost data in Table 2 and capacity data in Table 1 indicate that the pile of choice for non-tension piles is the 600-mm driven spun pile, except in the areas listed below where technical reasons require the use of bored piles:

- 1,000-mm bored piles founding at El. -42 m were selected for pile caps supporting the Trellis Roof. Driven piles cannot be used because of space limitations to accommodate the required large number of 600-mm piles and the effect of a large number of piles on pile efficiency.

Table 1. Results of Pile Capacity Analyses

Pile Type and Diameter	Ultimate Load (ton)	Allowable Load (ton)	Drag Load (ton)	Maximum Structural Load (ton)	Depth of Max Structural Load (metre)
Effective Stress Condition (1), Current Condition with Pumping (Depressed Piezometric Heads)					
Bored, ϕ1000, Tip –42m	1,646	713	466	**1179**	32.5
Bored, ϕ600, Tip –30m	287	140	74	**214**	19.7
Driven, ϕ600, Tip –26m	425	176	125	**301**	22.6
Effective Stress Condition (2), After Pumping is Stopped (Hydrostatic Piezometric Heads)					
Bored, ϕ1000, Tip –42m	1,068	**466**	301	767	28.8
Bored, ϕ600, Tip –30m	216	**104**	56	160	18.3
Driven, ϕ600, Tip –26m	278	**121**	78	199	19.5

Table 2. Comparison Costs of Piling Types

Type of Pile	Estimated Direct Cost per Linear Metre, Baht
Bored Piles, Compression, 1,000 mm	3,950
Bored Piles, Compression, 600 mm	1,280
Bored Piles, Tension, 600 mm	1,400
Spun Piles, Compression, 600 mm	800
Spun Piles, Tension, 600 mm	1,400
Note: 1 US$ = 37 to 42 Thai Baht between 1998 and 2003	

- The 600-mm bored piles founding at El. -30 m were selected for two rows of Terminal building pile caps around any Trellis Roof pile cap. Non-displacement piles are required to minimize disturbance to bearing strata in the vicinity of Trellis Roof pile caps. Based on settlement analysis, presented below, the founding elevation of -30 m would provide the most favorable results in terms of differential settlements around the Trellis Roof pylons.
- The 600-mm bored piles were also selected under the service tunnels, where tension loads were expected and therefore bored piles preferred.

The various pile types are shown in the schematic foundation section of Figure 3.

Settlement Calculations

Settlement calculations performed for this project did not take into account the regional settlements due to deep-well pumping of sand aquifers.

Calculation of settlement for pile foundation pertains more to pile groups than to single piles. Settlement of the pile group is governed by the soil at and below the Neutral Plane. Settlement of the soil below the Neutral Plane is caused by the stress increase due to the permanent loads, i.e., Dead Load plus the "permanent" portion of Live Load applied on the pile group.

Settlement calculations first involved calculation of the induced stress increase in the soils using the close-form solution to the Mindlin's Problem No. 1 (Poulos and Davis, 1974), which provides solutions of the stress field caused by a vertical point load acting *inside* an elastic half-space. Loads from a pile group were modeled as a group of point loads acting on the Neutral Plane. The effect of each individual pile was then superimposed to each other. Subsequently, contributions from adjacent pile groups were superimposed to obtain the complete stress field. Using one-dimensional compression value, m_v, for each soil stratum, the settlement contours in the area of interest were then computed based on the stress field.

Computations of the stress field and settlement contours were performed using Mathcad™, a software product of Mathsoft Engineering & Education, Inc. of Cambridge, Massachusetts. A mathematical model was established in Mathcad™ to include the subsurface stratigraphy, loading conditions, and the geometry of adjacent pile groups. Numerical integrations and other computational features were then programmed within the model. An example of the computed settlement contours is shown in Figure 6.

Differential Settlements

Differential settlements were calculated between adjacent pile caps for (1) each area around a Trellis Roof pylon, (2) typical areas of the Terminal foundation outside the influence of Trellis Roof pylons, and (3) typical areas of the Concourse foundation outside the influence of Trellis Roof pylons.

The largest differential settlements occur in the vicinity of the Trellis Roof pylons. As illustrated in Figure 6, a heavily loaded pylon supporting the Trellis Roof results in large settlements and creates a "settlement dish" in its vicinity. As a result, the building columns located within this influenced area will experience differential settlements. Note that the terminal building pile cap typically consist of 4 to 8-600mm diameter piles, while the pile cap supporting a pylon typically consist of 25 or 35-1000mm diameter piles and is therefore much larger.

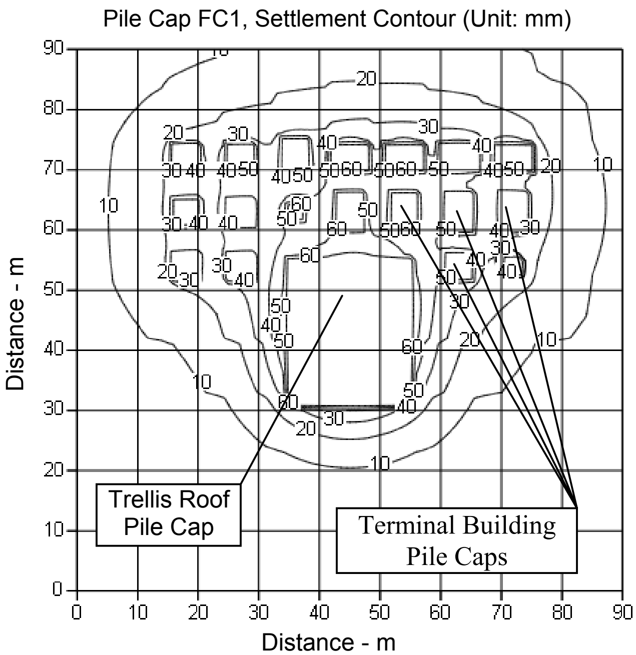

Fig. 6. Example settlement contours in vicinity of a Trellis Roof pylon

To some extent, the differential settlements depend on the sequence in which the various foundation elements are loaded. The analyses were based on the planned construction sequence, in which the Trellis Roof structure is constructed prior to construction of the surrounding Terminal building structures, as shown by the construction photos in Figure 9 below. In such a case, before the Terminal building foundations are loaded, the Trellis Roof foundation will already have undergone the part of settlement caused in the sand strata, but not in the clay strata. The differential settlements are therefore reduced.

The maximum tolerable differential settlements between structural elements were given by the structural engineer and architect. Essentially, differential settlements between adjacent building columns should not exceed 30 mm, corresponding to an equivalent angle of distortion of 1/300. This was particularly important for the glass curtain wall surrounding the Terminal building under the Trellis structure. Pile diameters and tip elevations were modified so as to obtain differential settlements that would not exceed the allowable limits. Table 3 presents the maximum calculated differential settlements between building columns in the areas surrounding various Trellis Roof pylons.

Table 3. Maximum Differential Settlements Near Trellis Roof Pylons

Location of the Super-Column Support	B-20	B-29	B-38	R-20	R-29	R-38
Settlement (mm)	15	30	30	20	15	20

For representative areas in the main Terminal and in the Concourses outside the influence of the Trellis Roof loads, the differential settlements were also computed. The resultant maximum differential settlements are approximately 15 mm for the Concourses and 10 mm for the Terminal, representing angles of distortion of 1/900 and 1/580, which are tolerable.

Total Construction and Post-Construction Settlements

The settlements in the sand strata occur during the course of construction, as the structural loads are applied, whereas most of the settlements in the clay strata occur after construction is completed. The computed maximum total construction phase and post-construction settlements are presented in Table 4. The somewhat smaller construction phase settlement in the Trellis Roof pylon areas result from the differences in the soil strata engaged by various pile types with different tip elevations.

Table 4. Total Computed Construction and Post-Construction Settlements

	Settlements (mm)		
	Construction	Post-Construction	Total
Trellis Roof Pylons	20	90	110
Terminal Building outside influence of Trellis Roof Pylons	29	16	45
Concourses outside influence of Trellis Roof	36	19	55

TESTING, CONSTRUCTION, AND MONITORING

Pile Load Test Program

Pile load tests were carried out to confirm the foundation design, and were therefore based on the current effective stress conditions in the upper half of Table 1. Results of the load tests were used to verify pile bearing capacities and, more importantly, to investigate the load-transfer mechanism. The latter was important since the design largely hinged upon the differential settlement evaluation, which depends upon a good pile load-transfer analysis. Pile load-transfer was measured in load tests using vibrating wire strain gages installed along the pile shaft. In addition, two bi-directional load tests, similar to the Osterberg Cell load test, were also conducted. For better calibration of the test results, all load tests were performed on piles located near existing subsurface boring locations, where subsurface stratigraphy is readily defined. The pile load test program is summarized in Table 5.

Table 5. List of Pile Load Tests

Loading Type	Load Transfer Measurements	Number of Tests
1000mm Bored Piles (Drilled Shafts)		
Compression	Telltales Only	2
Compression	Vibrating Wire Strain Gages	1
Bi-directional	Vibrating Wire Strain Gages	2
Lateral	Inclinometer	1
600mm Bored Piles (Drilled Shafts)		
Compression	Telltales Only	3
Compression	Vibrating Wire Strain Gages	4
Lateral	Inclinometer	1
Uplift	Telltales Only	1
600mm Driven Piles (Concrete Cylinder "Spun" Piles)		
Compression	None	5
Lateral	None	1
Uplift	None	1

In compression tests, the load was applied through a group of hydraulic jacks acting against reaction piles. Pile head deflection was measured using dial gages. Figure 7 shows the set up of a typical compression load test with reaction piles and transfer beams connected by steel bars. To measure the load-transfer, 5 compression test piles were furnished with vibrating wire strain gages.

In the bi-directional test, the test load was applied through one or both of the two hydraulic jacks, one installed at the bottom of the pile and the other in the pile shaft at a designated elevation. The tip resistance and skin friction were measured separately. Vibrating wire strain gages were installed along the pile shaft to measure load transfer.

Tension (uplift) and lateral load tests were also performed for piles designed to resist tension or lateral loads. Lateral load tests were carried out on piles previously subjected to compression test so that 19 test piles were used in all.

Load Test Results

The test results provided good confirmations to the design assumptions with regard to pile load-transfer. A typical pile load-transfer plot as measured in a load test is shown in Figure 8.

Fig. 7. Typical load test setup for a 1000 mm diameter bored pile

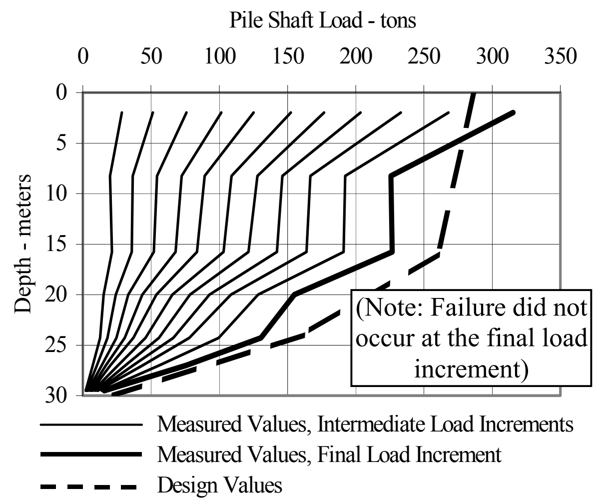

Fig. 8. Example of measured pile shaft load distribution

A few problems were encountered during some of the load tests with load-transfer measurements. Sometimes the pile shaft load calculated from the strain gage readings with assumed pile shaft stiffness did not match the actual applied load, in which case the pile shaft load had to be scaled so that a match would occur. In other cases, evidently malfunctioning strain gages resulted in unreasonable "kinks" in the shaft load distribution curves.

Pile Construction

The piling contract was awarded in advance of the main building contract on 12 September 2001. Pile testing started in October and construction started in November 2001. Since then, the piles and diaphragm walls for the Main Terminal Building and Concourses were completed by the end of July 2002 and all of the 20,000 m^2 of diaphragm walls and 25,437 piles, which included the elevated access roads, were installed within 12 months, 50% of the time available. The progress peaked at about 4,000 piles in a month. This significant achievement was made possible by a fleet of over 30 piling rigs supervised by a team of qualified construction engineers and inspectors. Several bored piling rigs were brought in from Singapore and resources from 4 subcontractors were used for the driven piling. Table 6 shows quantities of piles installed.

Table 6. Quantities of Piles

	Quantity	Number of Rigs
600mm spun pile	15019	17
600mm bored pile	9647	12
1000mm bored pile	771	2
total number of piles:	*25437*	
Diaphragm wall	306 panels	2

The prestressed concrete cylinder piles (spun piles) were driven with hydraulic pile driving hammers with hammer weights ranging from 7 to 14 tonnes. The piles were poured in cylindrical molds and formed under centrifugal spinning (thus “spun pile”) before steam curing. They were manufactured in sections varying between 12 metres and 14 metres in length. Inner ends of the pile sections are fitted with 16 mm thick steel end plates, facilitating field splicing by welding.

Pile Dynamic Analyzer (PDA) tests were performed on selected piles at end of initial driving and at re-driving to monitor the hammer performance and confirm pile bearing capacity. The results were used to determine the driving criteria, resulting in a minimum rated energy of 200 metre-tonnes, which could be applied to each hammer and drop height combination. When the design cut-off elevation was below grade, piles were driven with a steel follower with an impedance approximately equal to that of the pile, so that the PDA tests still provided meaningful results. PDA test results were also used to show what adjustment had to be applied when the different followers were used.

Bored piles were constructed using the slurry method with bentonite or polymer as the support fluid. The bored pile rigs were equipped with augers and drilling buckets. Generally, pile bases bored under bentonite were cleaned using air-lift equipment and piles bored under polymer were cleaned with a cleaning bucket. On excavation for pile cap construction some piles were found to have weak concrete at the top and some had horizontal deviations greater than that specified. These were dealt with in the usual way by breaking down the upper part of the pile, modifying the pile cap or installing additional piles as appropriate. In all, about 0.8% of piles were found to be defective.

On completion of the piling, the pile cap and superstructure work was carried out under a separate contract. Early work included the formation of the Super Column foundations over groups of 25 or 35 one meter diameter bored piles. These were topped off with four concrete plinths each containing drain pipes and holding down bolts for the steel Super Columns. Figure 9 shows two photos of the erected Super Column and Super-Truss.

Fig. 9. Construction of typical Trellis Roof pylons (Super Columns) and Super Truss

Settlement Measurements and Evaluations

A settlement monitoring program was implemented to measure the construction and post-construction settlements. For reasons stated above, it is of importance to monitor the progress of settlements in the vicinity of the Trellis Roof pylons (“Super Columns”). Survey points were called for on each Trellis Roof pile cap, on the two rows of Terminal building pile caps next to every Trellis Roof pile cap, and at representative locations along the Concourses and along the Frontage Roads.

To date, with construction of the Trellis Roof structure underway, the maximum settlement measured at Super Column locations is 10 mm. This compares favorably with the computed construction-phase settlement of 20 mm as presented in Table 4, given that the loads at this stage on the Trellis Roof pile caps are estimated at about 60% of the final load.

CONCLUSIONS AND LESSONS LEARNED

Given the complex subsurface conditions at the site and the extremely large loads from the Trellis Roof columns, the foundation design initially appeared daunting. However, with careful evaluation of the subsurface soil and groundwater information and the use of soil mechanics principles, a solution was found to safely accommodate these loads and the task of controlling differential settlements between Terminal building and Trellis Roof columns with significant differences in vertical load. The design was based on the principle of effective stresses and well established pile bearing capacity formulae. Load tests during design and construction were used to verify and validate design assumptions.

A significant feature of the design is the recognition of the currently depressed groundwater pressures due to deep well pumping in the area. Most importantly, the design is governed by the future recovery of groundwater pressures back to their original hydrostatic state as a result of the government's effort in restricting groundwater extraction. The trend of groundwater recovery is evident based on recent piezometric measurements in the deep aquifers.

The design exploited differences in compressibility of various subsurface strata to limit the differential settlement between Terminal Building and Trellis Roof columns.

Pile type and pile installation techniques are well established in Bangkok and were successfully used at the new airport site. Pile installation progressed at a very rapid pace with very few construction problems. Good construction management and a keen contractor, familiar with the foundation system for the Terminal Buildings Complex shortened the construction time by half. Because of the relatively uniform subsurface stratification and good understanding of its properties, no variation orders were issued due to changed soil conditions.

Measured settlements to date are within the anticipated limits and long-term settlements will continue to be monitored and evaluated.

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