Analysis of Load Capacity of Deep Test Piles in Shanghai Alluvium

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Summary Static and dynamic load testing were performed on two identical open-ended steel pipe piles installed to depths of 80 metres in Shanghai, PRC. The 914 mm dia. piles were each fully strain gauged and driven through soft Quaternary alluvium to found on dense alluvial sands. Load testing established the capacity of both piles. The work formed part of the pre-construction investigations for the 88-storey Jin Mao building, located in the Pudong District of the city. Pile driving was monitored using a Pile Driving Analyser. Static load testing of the two piles was in accordance with ASTM D 1143 with cyclic application of load to a maximum of 18000 kN. The paper provides results of geotechnical site investigations; descriptions of pile installation; and pile load-settlement behaviour and shaft load distribution. Comparisons are made between measured load capacity and estimates using several published pile design methods.

1. INTRODUCTION

This paper describes the installation and loading tests of 914 mm dia., open-ended, steel pipe piles driven to depths of about 80 m for the Jin Mao Building in Shanghai, China. The primary purpose of the pile loading tests was to establish the engineering load capacity, settlement behaviour and constructability time/cost data for deep, high capacity foundation piling to be utilised to support the mat foundation of the 88-storey tower.

The building is located in the Pudong Finance and Trade Development Area and will comprise an 88-storey multi-use, office/hotel tower with adjacent 6-7 storey commercial and retail buildings. The building was completed in August 1998. Figure 1 shows the location of the site.

The entire Jin Mao building site is underlain with three levels of basement. The foundation system for the tower consists of an approximately 65 x 65 m reinforced concrete mat foundation supported on deep, high capacity piling. Construction details are described by Tuchman, 1998. The low-rise areas are also supported by a lesser capacity piling system and inter-connected pile caps.

2. SITE GROUND CONDITIONS

The city of Shanghai lies in the Yangtze River Delta, where soils comprise Quaternary alluvial sediments to depths of 150 - 400 m (Anon, 1992).

The Jin Mao building site is located in a former residential area with densely placed houses and narrow roads. The site is relatively flat with ground levels from about 3.6 to 4 m above Mean Sea Level (MSL). The site is about 0.5 km west of the Huang-pu River.

Previous geotechnical investigations of the site show that sub-surface materials consist of Quaternary sediments to depths of at least 140 m below ground level. Groundwater lies at a depth of about 0.5 m at the site.

The soils beneath the site have been divided into ten layers, with Layers 7 and 9 further divided into two sub-layers. These layer descriptions are used consistently in the Shanghai area (Fang, 1986; Gao *et al*, 1986). Table 1 summarises the typical soil profile at the Jin Mao site.

Layer 7 is commonly used as a pile founding layer in the Shanghai area. Layer 9 was considered as a possible pile founding layer for the Jin Mao building, in order to achieve acceptable tower foundation settlements.

The two test piles for this program, designated "ST-1" and "ST-2", were installed in Layer 9-2.

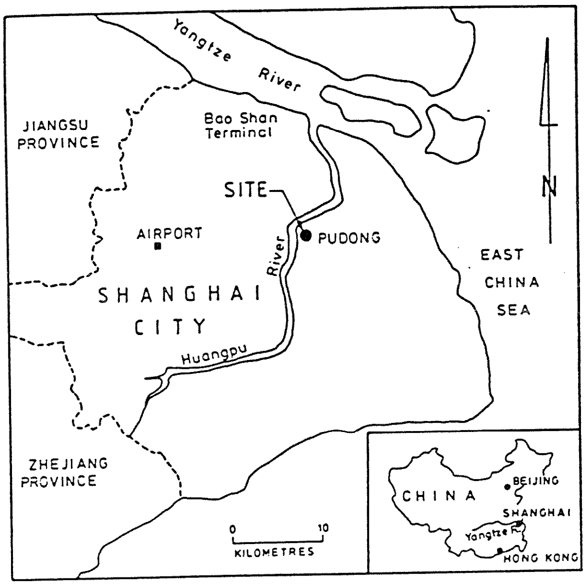

Figure 1. Site locality plan.

3. TEST PILE DETAILS

After considering the merits of installing various types of piles, it was decided that the pre-construction pile testing program would assess open-ended, steel tube piles, 914.4 mm in diameter (external) x 20 mm wall thickness.

The testing program also aimed at confirming the capability of the piles to be driven into the deeper, 9-2 soil stratum located approximately 75-80 m below existing grade. At such a depth, the piles were expected to exhibit a minimum design (or service) load of 7360 kN with a factor of safety of at least 2 on pile load capacity.

The design toe levels of the two test piles and twelve reaction piles (RP) are shown in Table 2.

Table 2. Pile design toe levels.

Pile Ref No.	Toe Depth below surface (m)	Toe Elevation (m MSL)
ST-1	79.00	-75.00
ST-2	79.10	-75.10
RP-1 to RP-9	79.00	-75.00
RP-10 to RP-12	55.00	-51.00

All reaction piles were installed at a minimum centre-to-centre spacing from the test pile of 5500 mm (about 6 pile diameters).

Figure 2 shows installation levels of the two test piles.

For piles to be installed to an elevation of -75 m MSL, the Contractor elected to drive each pile using a Delmag D-100 diesel pile driving hammer in three segments to an elevation of -59 m MSL, which approximately corresponds to the top of Layer 8. A BSP HA-30 hydraulic hammer was then used to drive each pile to its design toe level of -75 m MSL, which is approximately 10 m into Layer 9.

Table 1. Typical soil profile of Jin Mao building site.

Layer No.	Depth Range (m)	Soil Description	SPT "N" (blows per 300mm)*	Pressuremeter Elastic Modulus (reload) E. (MPa)*	Pressuremeter In Situ Pressure P_o (kPa)*	Pressuremeter Limit Pressure P_l (kPa)*	CPT q_c (MPa)*
1	0-1	FILL, clay with gravel & coal waste	-	-	-	-	-
2	1-3	Silty clay, brown-yellow (CL)	2	10	50	200	0.59
3	3-7	Very soft silty clay with thin silty sand layers, grey, (CL)	2	15	75	300	0.66
4	7-17	Very soft clay, grey (CH)	<1	15	120	450	0.45
5	17-26	Silty clay, grey, with organic matter (CL)	5	20	150	800	0.84
6	26-29	Clay, dark green (CL)	20	25	200	1100	1.83
7-1	29-36	Sandy silt (ML)	35	50	275	1300-2100	11.66
7-2	36-63	Sand, fine, grey-yellow (SP)	>50	85	400	1300-3300	23.26
8	63-71	Sandy silt within layers of fine sand & clay (ML)	>50	85	500	2900	14.46
9-1	71-76	Sandy silt, grey (ML)	49	90	650	3000	12.6
9-2	76-124	Sand, fine (SP)	>50	125	650-1100	3300-4800	21.55
10	124-to >137	Silty clay, grey (CL)	>50	75	1250	4800	-

* : typical result of *in situ* test for each soil layer, without any depth correction.

The Delmag D-100 pile driving hammer was described by the Contractor as having a driving ram of 20 tonnes mass. The BSP HA-30 hydraulic hammer has an overall mass of 56 tonnes, a 30 tonne driving ram and has a rated peak driving energy of 30 tonne-metres. Pump *et al* (1998) provide further details of pile installation.

4. PILE DYNAMIC TEST LOADING

4.1 Monitoring of Pile Driving

Monitoring of pile driving was carried out for a number of the piles during periods of expected heavy driving. The Contractor's internal Scientific Research Institute used a Pile Driving Analyser (PDA), designed and manufactured by Pile Dynamics, Inc. of USA, to measure at the pile head strain and acceleration during driving. Data were interpreted by classical wave equation methods (Pei & Wang, 1986). Results of this analysis provided instantaneous measurements (in real time at each hammer blow) of the following:

- maximum pile driving impact forces and driving stresses; and
- estimates of the static load capacity of the pile at the time of driving (these should be considered as approximate only and do not allow for pile set-up).

Monitoring using the PDA was conducted when each of the piles were being driven through Layers 7-2, 8 and 9-1. At these levels, the static load capacity was computed to be typically in the range 7360 kN to 9030 kN.

4.2 Redriving of Test Piles

Following the completion of driving of piles of ST-1 and ST-2, so-called "redriving" dynamic load tests were conducted using the PDA. In this case, the pile is subjected to typically 10 to 15 blows of the driving hammer, with the PDA data being gathered in the usual way. The test is conducted usually several weeks after pile driving, at which time it is assumed that soil pore pressures have dissipated and fine grained soils close to each pile have consolidated. During this period, the static load resistance of the pile will tend to gradually increase, which is often referred to as pile set-up.

Based on analysis using the CAPWAP computer program, the results of the redriving tests are summarised in Table 3.

No permanent penetration, as measured by the surveyor's level, was observed for either pile at the completion of the 15 blows from the hammer.

Table 3. Results of redriving tests using CAPWAP.

Test Pile No.	Pile Shaft Estimated Capacity (kN)	Pile Toe Estimated Capacity (kN)	Total Estimated Capacity (kN)
ST-1	12430	2690	15120
ST-2	13500	2720	16220

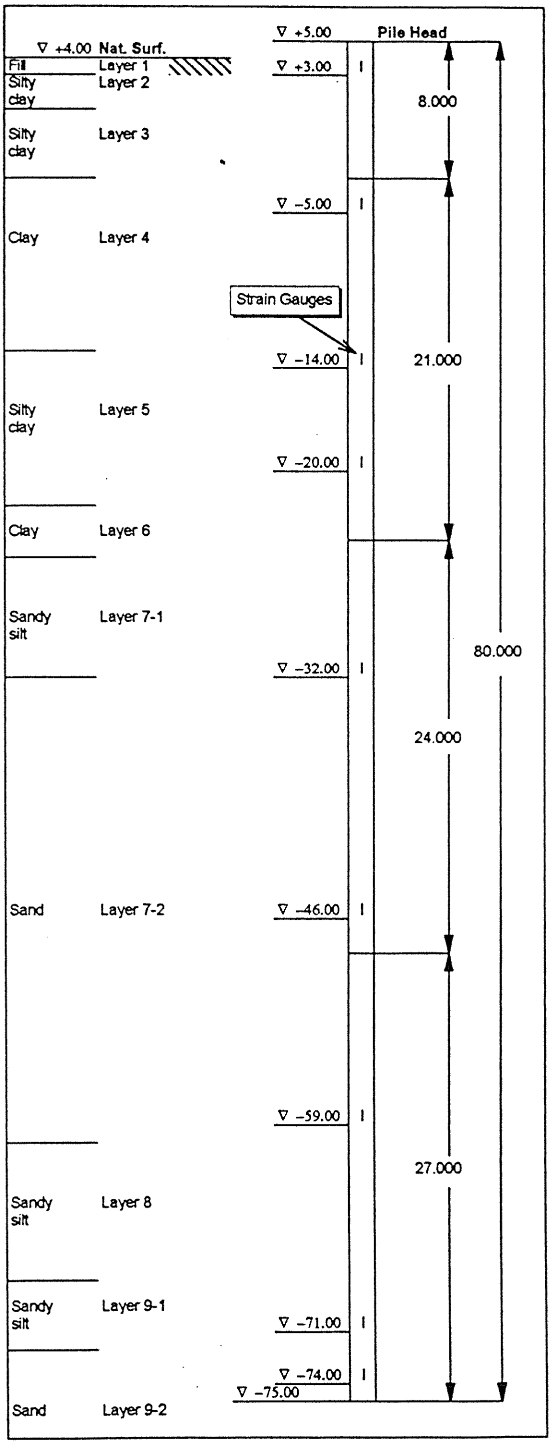

Figure 2. Test pile elevation.

The redriving test of pile ST-1 was conducted 11 days after completion of installation, whilst for pile ST-2 redriving was conducted 15 days after installation. Both piles were subjected to 15 blows from the hydraulic HA-30 hammer with the drop height set at its maximum of 1.2 m.

5. STATIC TEST LOADING

5.1 Instrumentation of Piles

Test loading was conducted by means of six hydraulic jacks connected in parallel, each with a nominal capacity of 5000 kN, and reaction beams with a nominal design capacity of about 18000 kN.

Static load during all tests was measured by a calibrated, electrical pressure transducer attached to the hydraulic pressure line. Constant load was maintained at each load increment by an automatic pressure sensor connected to the hydraulic pumps.

Pile head settlement was measured using four linear voltage displacement transducers (LVDT) reading to a steel reference beam, which straddled about 5 m either side of the test pile.

Strain measurements along the sides of the piles were obtained using weldable vibrating wire strain gauges. The gauges and associated wiring were protected for their entire length with an angle section welded to the inside of the pile.

5.2 Testing Procedures

Piles ST-1 and ST-2 were test loaded firstly in accordance with the loading sequence in ASTM D 1143, "Standard Test Method for Piles under Static Axial Compressive Load", and subsequently by a less prolonged pile loading schedule traditionally used in Shanghai.

In accordance with the ASTM D 1143 pile test loading standard, the following procedure was adopted for both tests:

1. Load was applied in steps equal to 25% of the design load of 7360 kN. Loading was then cycled to design load, 1.5 times design load, and then 2 times design load, returning to zero load between each cycle;
2. Each load was maintained until the rate of settlement was not greater than 0.25 mm per hour or until 2 hours had elapsed;
3. Once the load had reached 2 times the design load, it was maintained for 24 hours and removed in decrements of 25% of the design load;
4. Loading in excess of 2 times the design load was planned to be in increments of 10% of the design load up to a nominal 90% of the yield stress of the pile section (equivalent to about 16580 kN).

Both piles were reloaded several days after the completion of the above test loading schedule using a commonly used Shanghai procedure, which involves the following:

1. Load was applied in steps equal to 20% of the design load, without cycling of load, up to a maximum test load of 16500 to 18000 kN;
2. Each load was maintained for one hour;
3. Unloading of the pile was in steps equal to 40% of the design load and each decrement was held for 20 minutes.

Both piles were test loaded as shown in Table 4.

Table 4. Pile test loading details.

Pile	**Test Type**	**Time from Installation (days)**	**Max. Applied Load (kN)**
ST-1	ASTM	23	16200
ST-1	Local	37	16500
ST-2	ASTM	35	16500
ST-2	Local	50	18000

5.3 Results for Pile ST-1

The load-settlement behaviour of pile ST-1 is shown in Figure 3, for both the initial (ASTM) loading procedure as well as the reloading procedure.

Linear, elastic deflection behaviour of the pile was evident up to a load of about 11500 kN, at which a yield point in the load-deflection plot was apparent. Non-linear, plastic behaviour was observed at greater load levels. After several cycles of load (including the re-loading test), final unloading of the pile showed a residual deflection of about 197 mm.

Distribution of applied load along the shaft of pile ST-1 is shown in Figure 4. A large proportion of the applied load was supported by shaft friction; at load capacity the toe resistance represents about 27% (ie 4190 kN) of the total resistance of the pile.

For load increments up to about 11000 kN, vertical creep movements of the pile head were small and possibly close to the resolution of the settlement measuring system. Above that load the rate of creep (in terms of mm of movement per log cycle of time), steadily increased to about 9 mm/log cycle.

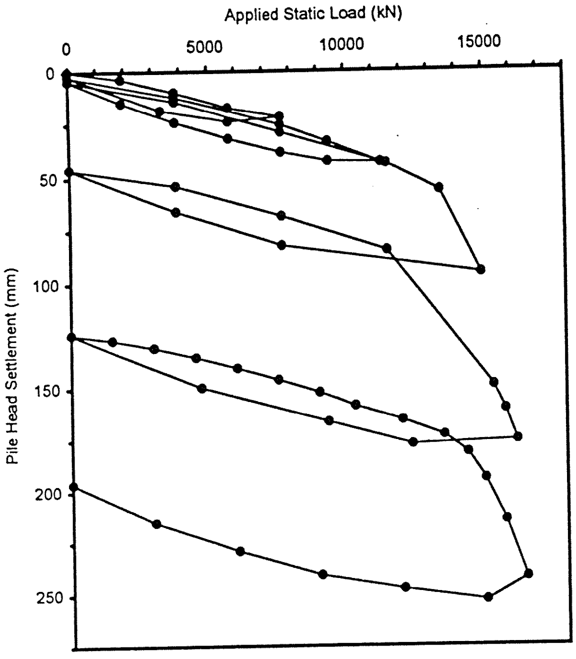

Figure 3. Pile load-settlement, Pile ST-1.

If the method of Davisson (1972) were to be adopted to assess the load capacity of the piles, a capacity of about 15300 kN is indicated for pile ST-1. The Davisson method takes into account the elastic compression of the pile section and the diameter of the pile.

With a factor of safety of 2 on load capacity, pile ST-1 has a service load of about 7650 kN. This compares with a design load of 7360 kN for the production piles.

5.4 Results For Pile ST-2

Pile ST-2 was also initially loaded in accordance with the ASTM procedure and subsequently reloaded using the local Shanghai procedure. The load-settlement results are shown in Figure 5.

Results showing distribution of applied load along the shaft of the pile are shown in Figure 6, which also shows that most of the total load applied at all load increments was the shaft resistance component. Toe resistance at load capacity was about 15% of the total resistance of the pile.

Similar to pile ST-1, this pile shows linear response up to a yield load of about 15000 kN, and a "Davisson" capacity of 16800 kN. With a Factor of Safety of 2, the service load is 8400 kN compared to 7650 kN for pile ST-1.

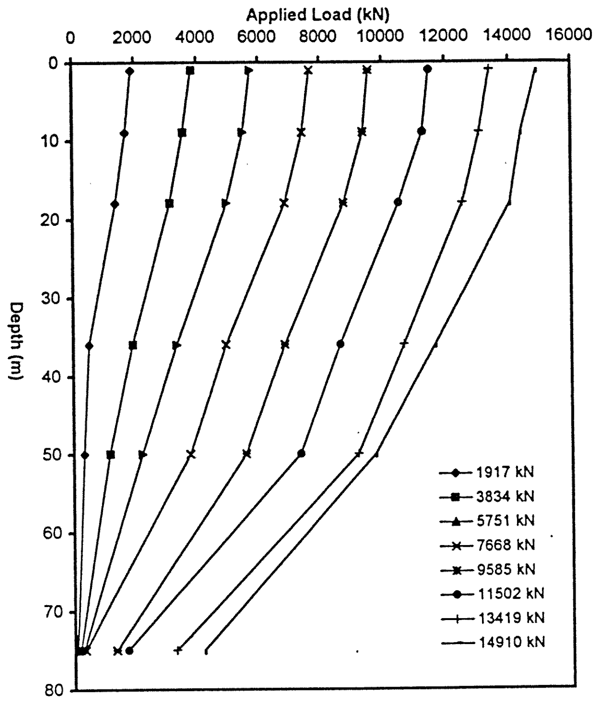

Figure 4 - Load Distribution, Pile ST-1

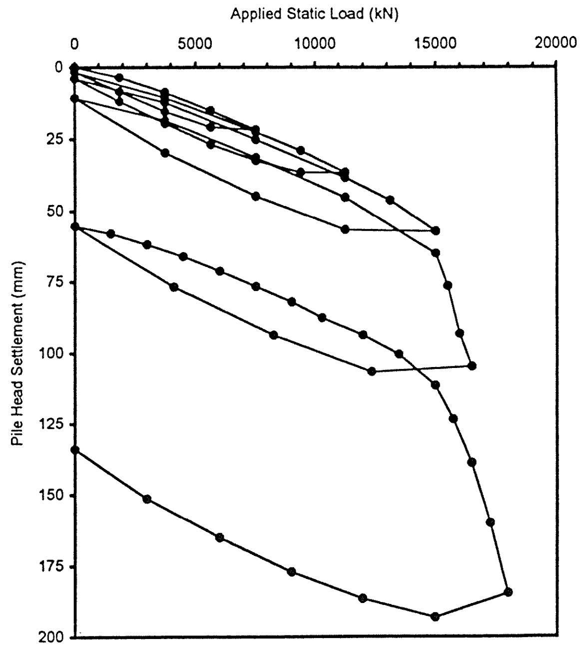

Figure 5. Pile load-settlement, Pile ST-2.

5.5 Comparison of Results

Figure 7 shows a comparison of the distribution of unit shaft friction resistance for each of the two piles. Shaft friction for the piles are at comparable levels and steadily increase from about 15 kPa in the upper 10 m to 70-100 kPa in Layer 9-1/9-2.

It is relevant to also compare the influence of time following the completion of installation of each pile.

indicates that the additional 12 days allowed for set-up to occur in pile ST-2 (ie ST-1 tested initially at 23 days after installation and pile ST-2 after 35 days) may account for the 10% higher load resistance of the latter pile. This difference suggests that pile set-up over time may a possible feature of this site.

The pile toe resistance mobilised by each pile has also been evaluated. It would appear that the unit toe resistance at the pile load capacity was about 6380 kPa and 3860 kPa for piles ST-1 and ST-2 respectively. Assuming that the pile toe is fully plugged, however, toe load resistance would be expected to be well in excess of these values. The soil plug appears to have provided only a partial contribution to total load resistance

6. ACTUAL AND EXPECTED RESULTS

The following methods were adopted to compare the test pile load capacities:

- using the results of Standard Penetration Tests (SPT) according to the methods of Meyerhof (1976 and 1986);
- using the results of Static Cone Penetration Tests (CPT) according the method of Van der Veen & Boersma (1957); and
- by adoption of traditional methods of geotechnical design for layered soils as proposed by Meyerhof (1976) and Poulos & Davis (1980).

Comparisons are shown in Table 5.

Table 5. Estimated and measured pile load capacities.

Method	Base Load Capacity (kN)	Shaft Load Capacity (kN)	Total Load Capacity (kN)
Estimated by Standard Penetration Tests (SPT)	3300	5900	20270
Estimated by Static Friction-Cone Penetrometer	14100	NA	NA
Estimated by Traditional Design	3300-19000	7200	10500-26200
Measured Result - Pile ST-1	4190	11110	15300
Measured Result - Pile ST-2	2520	14280	16800

In the light of uncertainty expressed in the literature of the processes governing "critical" or "limiting" depth effects in deep piles in sand, a range is shown in Table 5 for pile base design capacities estimated by traditional design methods.

Critical depth approaches are based on an *in situ* effective vertical stress σ_v' being equal to overburden pressure to some critical depth z_c beyond which σ_v' remains constant. This idealised distribution implies that the average unit shaft capacity and the base load capacity become constant beyond a certain depth of penetration. This concept is challenged by Randolph (1993).

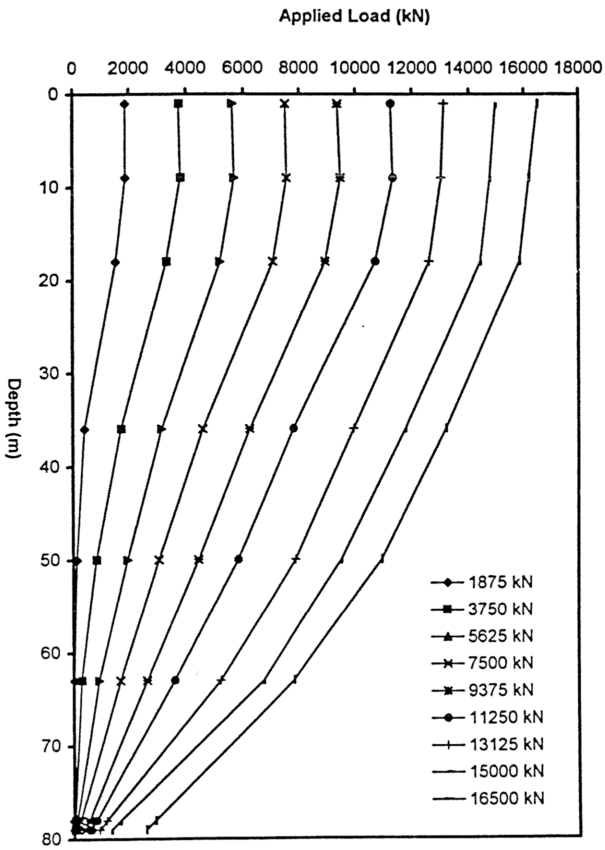

Figure 6 - Load Distribution, Pile ST-2

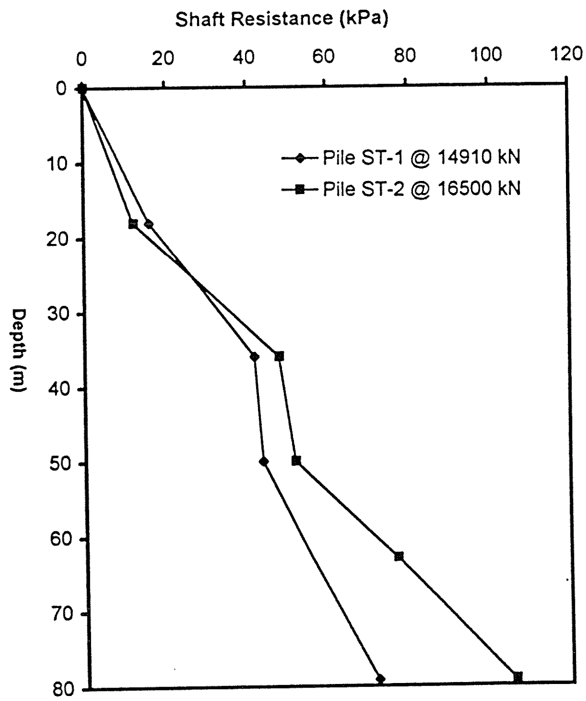

Figure 7 - Unit Shaft Resistance at Pile Capacity

Figure 7 suggests that critical depth effects for the two piles in the sand layers (ie beyond Layer 7-1, or below depths of about 36 m) are absent, or possibly masked by relative density increases at depth.

With respect to base capacity of the test piles, comparison with capacities expected on the basis of SPT and CPT suggests that only a portion of the available pile toe area may have contributed to base resistance.

Measured shaft load capacities are also significantly higher than would be otherwise expected, again possibly due to an apparent absence of critical depth effects.

The large discrepancies between expected and measured shaft and base capacities shown in Table 5 serve to highlight uncertainties associated with currently available design methods for deep piles in sands.

7. CONCLUSIONS

The test driving and load testing program for the Jin Mao building, involving two test piles and twelve reaction piles, confirmed the general suitability of deep, large diameter tube piles as the proposed piling system for the 88-storey tower. The work has shown that 914 mm diameter steel pipe piles can be readily driven to found in the dense sand, Shanghai Layer 9-2, at depths of about 80 m. This arrangement has been shown to yield a service load of at least 7600 kN per pile with a factor of safety of 2 on load capacity.

The test pile program has also shown that:

1. the two test piles exhibit similar load-settlement behaviour, although set-up may have brought about a slightly higher load capacity for pile ST-2, in that it had been installed 14 days earlier than pile ST-1;
2. static test loading using the ASTM procedure D 1143 provides pile load-settlement results that are similar to those obtained using the local Shanghai test loading schedule, the latter being considerably less time consuming;
3. the method of Davisson appears to represent a reasonable approach for determining load capacity from a pile load test at this site;
4. dynamic monitoring and CAPWAP analysis have provided a reliable indication of static load capacity of both piles;
5. a large proportion of the load resistance of each pile was mobilised by shaft resistance; complete soil plugging appears not to have occurred; and
6. a comparison of the results of the test loadings highlights the limitations and uncertainties associated with current approaches to design of piles in sand.

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