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Design and Performance of a Piled Raft on Soft Clays

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SUMMARY

Soil conditions at the new Coronial Services Centre building site in South Melbourne are dominated by a 20m thick layer of compressible silty clay overlying sandy gravels. To achieve substantial cost savings, a pile stabilised raft was developed as the foundation system, rather than the more conventional piling system end-bearing in the gravels or siltstone. The system of six pile - stabilised rafts, each supporting a building which produced a different intensity of loading, was introduced to Australia for the first time on this project. Individual piled rafts were linked using structural articulation and movement joints, capable of tolerating differential settlements. Pile lengths under individual rafts were selected with the aim of ensuring that the development would settle relatively uniformly and thus that differential settlements between adjacent raft sections would be minimised. Elastic theory was used to design the piled raft system. This involved calculation of an equivalent Youngs modulus from settlements estimated for the design life of the structure. Settlement records of individual piled-rafts are being monitored during construction of the building.

Cast-in-situ enlarged base Frankipiles, founding at varying depths in the silty clays, were nominated as the preferred pile type. Very few piles had ever been constructed to derive support in this soft clay formation and a comprehensive load testing program, comprising static load testing of 6 piles as part of a preliminary program, was instigated. A further 17 piles were load tested during the course of the contract. A brief discussion of the design aspects of the piles and piled rafts is presented, followed by discussion of the predicted versus actual performance of the piles. Only limited data were available on the piled raft settlements at the time of writing.

1. INTRODUCTION

The South Melbourne area is underlain by complex geological conditions, compounded by a deep layer of highly compressible silty clay, known locally as the Coode Island Silt formation. The new Coronial Services Centre is a low-rise development comprising six buildings, ranging from one to three storeys. These buildings produce varying average foundation stresses, from 15 to 38 kPa. Such differences would produce large total and differential settlements if founded on conventional raft footings. One alternative for the foundation system for the development was to found the structures on deep, end-bearing piles. Whilst this solution effectively eliminated settlements, it was expensive and would result in the ground settling away from the structure due to the general settlement of the area. A design using shorter piles was pursued and the concept of a pile-stabilised raft evolved. This was seen to have a number of desirable features, including:

(i) A significant reduction in costs.

(ii) Since the use of the piles was to primarily control settlements, lower factors of safety could be used than for a "normal" pile foundation. Thus, the minimum number and size of piles could be adopted.

(iii) Piles of varying lengths could be used under different buildings to limit differential settlements between the buildings, to estimated values of less than 50mm over the 50 year design period.

2. SITE CONDITIONS

The site is located in the Yarra delta on a deep deposit of Coode Island Silt. The geology of the area has been described by Neilson (1976), while Walker and Morgan (1977) discuss some engineering properties of the Coode Island Silt and settlement performance of embankments and structures.

Subsurface Profile

Information on ground conditions and response at the site and surrounds was available from sampled boreholes, electric cone penetrometer tests, and observations of loaded and unloaded ground surface settlements.

Apart from the fill which showed marked variation, the subsurface profile appeared relatively uniform on the basis of bore information. It may be summarised as:

Depth	
0 - 2m	FILL (bricks, rubble, timber, etc.)
2 - 16m	SILTY CLAY (CH), Coode Island Silt. Mainly firm to stiff, highly compressible silty clay with shell fragments and occasional thin layers of fine sand.

16 - 22m CLAYEY SILT/SANDY SILT (MH) a member of the Coode Island Silt formation. Firm to stiff consistency and highly compressible. Frequent peaty material.

22 - 28m SANDY CLAY (GP)/CLAYEY SAND (SC)/ SANDY CLAY (CL) Moray Street Gravels. Very variable, inferred as lenses of predominantly sandy and gravelly material. Generally moderately to very dense.

28m SILTSTONE and minor interbedded sandstone. Silurian bedrock. Highly to moderately weathered

The groundwater occurred at about 1.5m depth

A typical profile from a cone penetrometer test is shown in Figure 1. This shows the Coode Island Silt (a clay) extending to a depth of around 16-18m, where it grades into a sandy silt overlying the Moray Street Gravels.

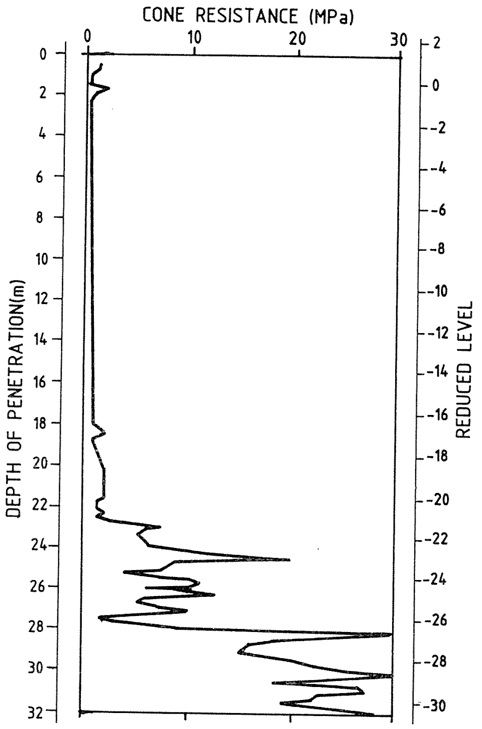

Figure 1: Cone Resistance of Coode Island Silt

Laboratory Test Results

Classification tests were used to establish the extent of the Coode Island Silt in boreholes, distinguished by having high plasticity and low sand content compared with the underlying silt.

Undrained shear strengths of the Coode Island Silt measured on samples recovered by a hydraulic piston sampler ranged from 32 kPa to 66 kPa, with no systematic variation with depth. This contrasted with the interpretation from the q_c versus depth plot from the cone penetrometer which suggested a linear variation of shear strength with depth. The latter values were used in design.

Consolidation tests were performed to establish mainly the reconsolidation pressure and the creep behaviour. Values of the creep coefficient were found to range from 0.7 to 1.6 percent per log cycle of time, over a range of samples and representative stress increments. The test results showed an apparent preconsolidation pressure which was greater than the calculated overburden stress, therefore it was expected that the soil response would follow the reloading rather than the virgin loading curve.

Nearby Settlement Observations

Level observations were provided by the Road Construction Authority from the adjoining street over the period 1981-1985. These indicated a settlement rate of around 10mm/year, a figure which has been reported from other observations in the vicinity since at least 1960. It is noted that this creep rate corresponds for the known depth of Coode Island Silt and laboratory-measured creep rates to a time origin of initiation about 10 years ago. This is inconsistent with the knowledge that there were no initiating activities at around that time, and that the rate appears to have been unchanged for more than 20 years. The explanation of this paradox is not known.

The RCA monitored settlements of surcharged exit ramps for the West Gate Freeway from April 1980 to around November 1982. The results for an area close to the site were used to assess the compressibility and creep rate of the Coode Island Silt in-situ.

3. DESIGN OF SINGLE PILES

3.1 Specification Requirements

Cast-in-situ, enlarged base Frankipiles were nominated as the preferred pile type. All piles were to have a nominal shaft diameter of 450mm and were required to meet the following criteria:

(i) Three founding depths were proposed to meet the variable loadings, viz: 7m, 12m and 18m.

(ii) The nominal design loadings were 160 kN and 360 kN for the 7m and 18m long piles respectively.

(iii) All piles were to achieve an ultimate load of not less than 2.5 times the nominal working loads.

(iv) The deflection was not to exceed 30mm at the nominal design load.

(v) Base sizes were specified as a minimum of 750mm dia. for the 7m long piles; and a maximum of 750mm dia. for the 18m long piles.

3.2 Predicted Pile Load Capacity

Contrary to laboratory shear strength tests, cone penetration resistances showed an increase in strength with increasing depth. Cone resistances increased within the range 30Z to 50Z (kPa), with Z being the depth in metres. Cone data formed the basis of design.

Pile load capacity was assessed as follows:

ultimate base capacity Pbu = $q_c Ab$

where q_c = cone resistance below base
Ab = area of base

ultimate shaft capacity Psu = a CuAs

where Cu = average undrained shear strength along pile shaft (determined from $\frac{qc}{15}$)
a = conventional adhesion coefficient
As = area of shaft

The friction sleeve was not used to assess shaft friction of the pile because of the belief that unreliable results are often obtained by this method. (Tchepak, 1986). Also, the shear strength of the fill was conservatively assumed to be not less than that of the Coode Island Silt at the same depth.

The analyses indicated an ultimate pile capacity of about 300 kN and 850 kN for the 7m and 18m long piles respectively, for piles constructed to the specified pile geometries.

This generally confirmed preliminary assessments of the geotechnical reports. Accordingly, the specified factor of safety on nominal working loads could not be guaranteed by the contractor for the "conforming" configurations. In particular, a 950mm dia. base was recommended for the 7m long piles to achieve a factor of safety approaching 2.5 and this solution was offered with the tender submission.

3.3 Predicted Settlement Performance

It was recognised that the most reliable method of obtaining soil modulus values for pile design was to back analyse results from the proposed pre-production piling static load testing programme. However, a preliminary load-deflection prediction was made using a soil modulus profile E_s = 500 Z (kPa), with Z being the depth in metres. Because the distribution was assumed to vary linearly with depth, the average soil modulus acting over a pile length E_s was then taken as 250 Z (kPa). The theory developed by Poulos (1972), was used to construct load-settlement curves to failure. Predicted load-deflection curve for 7m Frankipiles showing the influence of base size, are shown in Fig. 2.

Predicted deflections for the 7m and 18m piles at working load are tabulated below.

Pile Length (m)	Working Load (kN)	Predicted Deflection (mm)
7	160	20
18	360	10

4. DESIGN APPROACH TO PILE-STABILISED RAFT

Basis of Design

A piled raft is one in which the piles are used to stiffen the load-settlement response and are not simply designed to carry a designated portion of the structure load. Methods of design using elastic analysis have been published by Davis and

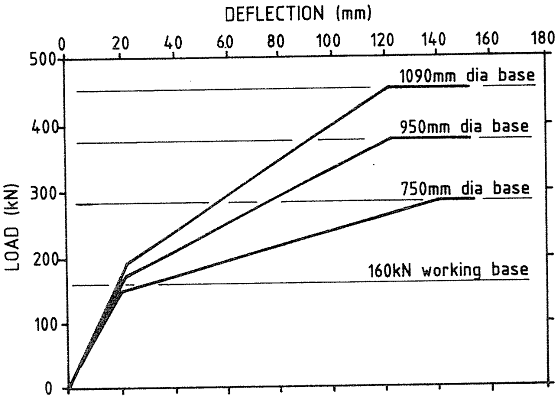

Figure 2 Predicted Load Deflection Curve
7 m Frankipile

Poulos (1972), Randolph (1983), etc. and examples of their use have been given by Burland et al (1977), Cooke et al (1981), etc. These case studies have been for stiff soils and creep has not been considered. A recent summary of the approach was published in 1985 by Carter et al.

For calculating the behaviour of piled rafts, the methods are based on elastic theory, i.e. both the pile and the soil are described by the elastic parameters, Youngs modulus and Poissons ratio. The approach used has been to accept elastic theory as the method of analysis and to calculate an equivalent Youngs modulus from the creep settlement predicted over the design life, taken as 50 years.

Building Load Data

Approximate area and loading details for the six separate buildings comprising the development as shown in Figure 3 are given in Table 1.

TABLE 1

BUILDING DIMENSION AND LOADING DATA

Building No.	Area (m^2)	Average Load (kN)	Long Term Stress (kPa)
1	406	15,230	38
2	726	10,670	15
3	676	19,620	29
4	864	26,200	30
5	940	14,450	15
6	140	2,870	21

Pile - Raft Stiffness

The calculation method was based on that published by Randolph in Carter et al (1985). The design charts are based on a uniform rectangular grid of column locations, with each column supported on the same number of piles. Therefore approximations to the building layouts were used comprising generally square rafts with regular column grids.

For the ground model, a stiff layer was assumed below a depth of 24m, i.e. including the Moray Street Gravels above the siltstone. While the gravels were expected to undergo some settlement, it was considered to be insignificant compared with that in the overlying Coode Island Silt.

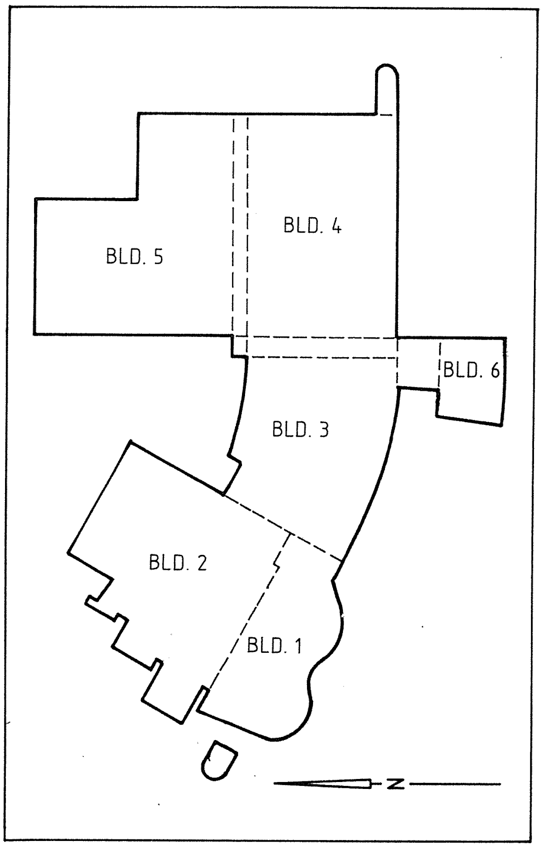

Figure 3 Building Layout

For a single pile group, the group stiffness, which is a function of pile size, spacing and soil modulus was calculated for the three pile lengths involved. This stiffness was then used to calculate a pile group global stiffness Kp which depends on the number and spacing of the groups as well as the depth of the compressible medium. The stiffness (Kc) of the raft alone was then calculated and combined with Kp to give an overall footing stiffness of Kf. Values used in preliminary analysis are summarised in Table 2 for different raft areas and numbers of piles beneath each column.

TABLE 2

FOOTING STIFFNESSES (kN/m)

Pile Length	30m x 30m 1-pile group	30m x 30m 3-pile group	16m x 24m 3-pile group	12m x 12m 1-pile group
7m	85,500	100,000	70,000	32,000
12m	95,000	115,000	75,000	45,000
18m	150,000	175,000	110,000	70,000

The percentage of load taken by the piles depends mainly on pile length and the charts indicated values of around 40%, 50% and 70% for piles of lengths 7, 12 and 18m respectively. The footing stiffnesses were used to give initial assessments of pile numbers and pile lengths in order to achieve approximately equal settlements of all buildings.

These designs were then reviewed by Dr. P.T. Brown of the University of Sydney to confirm the final pile lengths and groupings beneath each building. The estimated building settlements ranged from 90mm to 110mm over the 50 year design life, which gave predicted differential settlements within the required criteria. The final structural and architectural details were designed to accommodate differential settlements of 50mm between adjacent buildings.

5. FIELD PERFORMANCE

5.1 Single Piles

The results of the trial pile loading tests were back analysed using elastic methods (Poulos & Davis, 1968) to assess the pile stiffness K and average soil modulus Es Elastic methods were then used to proportion the ultimate load onto the shaft and base. This required the following assumptions to be made.

- The ultimate load of the pile was defined as the load corresponding to a deflection of approximately 45mm (i.e. 10% of the nominal pile shaft diameter).
- To assess soil modulus, values of load and deflection were selected either at values corresponding to working loads (provided that pile behaviour was not in an advanced "plastic phase"), or to load-deflection values appropriate to the initial elastic portion of the load deflection curve.

The results of these analyses are shown on Table 3. The average soil modulus values, plotted against the **base** depth of the pile are shown on Figure 4. The following observations may be made from these results.

(i) The average shaft friction on the 7m piles, at 20 to 30 kPa, was double that of the 18m piles (11 kPa only). This was possibly due to the effect of the fill playing a major role for the short piles.

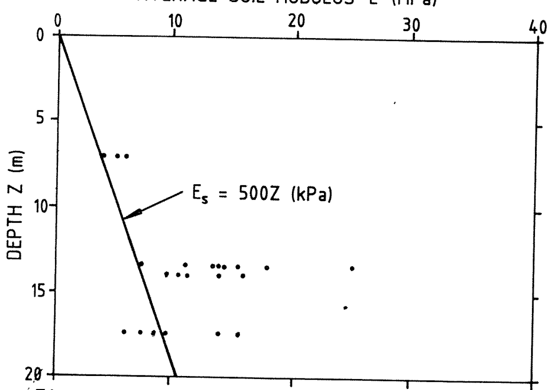

Figure 4 Back-calculated soil modulus

TABLE 3

BACK-ANALYSIS OF TRIAL PILES

Pile No.	Age At Test (Days)	Total Length (m)	Base Dia. (m)	Ultimate Load (kN)	Ultimate Resistance Shaft (kPa)	Ultimate Resistance Base (kPa)	Av. Soil Modulus (MPa)
23	7	7	0.95	440	22	355	4.0
40	13	7	1.02	440	21	317	5.3
62	11	7	1.02	500	33	269	5.7
79	7	18	0.75	370	10	637	16.0
81	13	18	0.75	500	11	1426	7.2
82	7	18	0.75	420	11	764	14.0
82 (Retest)	31	18	0.75	770	-	-	8.5
71	20	15	-	1000	11	818	15.9
44	18	15	-	900	11	515	11.3

(ii) Piles with base sizes of 750mm or less were not effective, resulting in low failure loads for the 18m long piles.

(iii) The bases for the 7m long and 12m long piles (constructed with base diameters of 950mm or greater), were most effective, supporting approximately 50% and 70% of the ultimate pile loads respectively.

(iv) The unit base resistances varied from about 300 kPa at 7m, to about 800 kPa at 18m, which was consistent with cone penetration test values at the corresponding depth.

(v) The average soil modulus for the pile type in these soils could be expressed as 40 times the average cone resistance acting along the pile shaft, i.e., $E_s = 40\ q_c$.

(vi) The back analyses indicated that a lower bound profile of E_s with depth could be expressed as $E_s = 500\ Z$ (kPa). This is twice that assumed in design. (Refer Figure 4).

The "set up" effects with time, were most dramatic. One pile, No. 82, was tested three times during the course of the contract, the results being shown in Fig. 5.

The conclusions drawn from the study of piling prediction with actual performance is typical of many piling works, viz:

(1) The ultimate pile capacity could be reasonably accurately predicted.

(2) Single pile settlements estimates, without the benefit of a static pile loading test to determine deformation parameters, were generally in error.

5.2 Pile Raft Performance

The settlements of the various building segments have been monitored since pouring of the concrete floors. Generally four to six monitoring points were established on each slab. However many of the points were disturbed by finishing trades, and it became necessary to transfer to new points in August 1987.

The dates of the main concrete pours for the buildings reported were

Building 2 Nov 1986
Building 3 Sept-Oct 1986
Building 4 July-Aug-Sept 1986
Building 5 Aug 1986

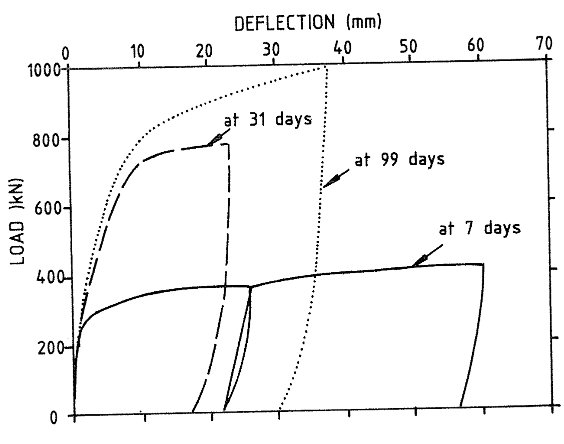

Figure 5 Results of Tests on Pile 82

The results of the monitoring are presented in Table 4 for the four buildings for which concrete pours had taken place in December 1986 and for which reasonable continuity of readings was available. The datum applied in taking these readings is a point on the kerb in an adjacent street. At the location of this datum, as for the surrounding area in the vicinity of these buildings, general area settlements of the order of 10mm per year are expected. The readings are therefore settlements relative to the general area, and not absolute values.

TABLE 4

BUILDING SETTLEMENTS

Building No.	Settlements (mm) Between Dates Indicated			
	Dec 86–Mar 87	Mar 87–Aug 87	Aug 87–Nov 87	Dec 86–Nov 87
2	1	1	15	17
3	6*	6	13	25
4	4*	-7*	12	9
5	5	-2*	11*	14

* Result from single point.

The results are affected by single erratic readings, which can occur when points are destroyed between levelling. Single readings are identified as such in Table 4. The negative readings (heave) are considered unlikely, and have distorted the total settlement recorded over the period of about a year. Further readings are needed to establish a clear trend, however the most recent set suggests relatively uniform settlement of the differently loaded buildings over the most recent monitoring period.

The estimates for building settlement resulted in a predicted upper limit in differential settlement between adjacent buildings of some 20mm in the 50 year design period. Based upon these predictions the project has been designed to accommodate, by way of articulated jointing segments, a differential movement between adjacent buildings of 50mm. Consideration of Table 4, although being early in the building life, would appear to indicate that the predicted differential settlements would not be exceeded in the life of the building.

The piled raft concept was developed using differing pile lengths to provide similar settlements for different loading intensities of the group of linked buildings. The results of monitoring settlements early in the building life suggest that the differential settlement criteria of 50mm over the design life should be achieved.

6. CONCLUSIONS

This paper sets out the background and the basis for the design of the piles and the piles raft. The site is a difficult one because of the existing area settlement, and the large component of creep settlement expected to arise from structural loadings. The pile criteria were difficult to establish by normal calculation and a pre-production test pile program was arranged to provide field parameters. The results of this program showed that pile capacities could be reasonably well predicted but deformations were overestimated.

The piled raft concept was developed using differing pile lengths to provide similar settlements for different loading intensities of the group of linked buildings. The results of monitoring settlements early in the building life suggest that the differential settlement criteria of 50 mm over the design life should be achieved.

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