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National Bank House Melbourne, Foundation Design and Performance

D.J. HENKEL
Director, Ove Arup & Partners, Sydney
A.B. PHILLIPS
Chief Geotechnical Engineer, Ove Arup & Partners, Sydney
and
E. GULDAGER-NIELSEN
Associate Director, Ove Arup & Partners, Sydney

SUMMARY. National Bank House, Melbourne has its main structural load distributed between a raft under the central core and four heavily loaded external columns supported by large diameter caissons. The raft is founded on weathered basalt 5 m below ground level and the caissons terminate as rock sockets in Silurian mudstone at more than 26 m depth. The paper outlines the methods used for analysis and design of the foundations, and compares predicted with measured settlements. Conclusions were that the in situ elastic moduli of weathered basalt and mudstone in Melbourne are much higher than can be deduced from small scale laboratory and field tests; skin friction acting on bored caissons in weathered basalt and mudstone may be calculated using a friction factor α of 0.3 to 0.4; and that the King Street Bridge relationship between moisture content and elastic modulus may be used with modern socket design methods to yield a reliable estimate of settlement.

NOTATION

(Note: $\bar{\ }$ over any symbol indicates mean value)

A = shaft cross-sectional area
Cu = undrained triaxial shear strength
Cu_b = Cu beneath socket base
Cu_s = Cu along socket side
D = shaft diameter
E' = drained (long term) elastic modulus
E'_b = E' beneath socket base
E'_s = E' along socket side
Ec = elastic modulus of concrete
P = load on caisson
P_n = load applied to the nth section of shaft
P_{sn} = total skin friction on nth section of shaft
S = socket load
S_{all} = allowable socket working load
S_{ult} = ultimate socket load
w = moisture content
x_n = length of nth section of shaft
α = shaft adhesion factor
δ = settlement of caisson shaft
ρ_n = elastic shortening of the nth section of caisson shaft

1. INTRODUCTION

The design and construction of the foundations for National Bank House at 500 Bourke Street, Melbourne, in 1974 presented a significant challenge. The method used for design of the rock sockets was based on the King Street Bridge relationship between strength and moisture content (Parry 1970).

Although advances have since been made in design methods for rock sockets, for example by Pells and Turner (1978) and Williams et al (1980), it was considered useful to present the design method actually used together with the measured settlements.

Performance data of this type are not common and they provide an opportunity to check other design methods.

2. STRUCTURAL CONCEPT

National Bank House was conceived as a square sectioned, 41 storey tower, with 5 basement parking levels immediately adjacent to it on the Little Bourke Street side. The general layout is shown on Fig. 1. The unique structural concept involved highly loaded external columns at each corner of the tower and a central core to carry the remaining load. A raft foundation was initially conceived beneath both core and columns, but non-uniform settlements were predicted due to varying strata thicknesses. To overcome this it was decided to found the core on a raft, but support columns on deep caissons. Differential settlements were clearly of importance with this mixed foundation arrangement and much effort went into trying to predict them accurately.

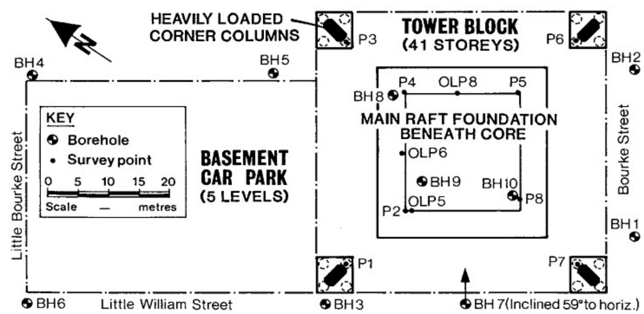

Fig.1 Site plan showing boreholes and survey points

3. GEOLOGY

3.1 General

The bedrock in central Melbourne comprises a great thickness of folded Silurian mudstones, the upper portions of which were weathered before they were covered, in Tertiary times, by extensive basalt flows. The Tertiary basalt has in turn weathered, ultimately forming a firm to stiff residual clay.

3.2 Site Investigation

Ten boreholes were sunk at the locations indicated on Fig. 1. Results were correlated with data made available from a large number of other developments in the vicinity to give a comprehensive picture of the local geology. The general succession on the site, beneath a shallow covering of fill, was as shown on Fig. 2. As expected from the geological history, the thickness of both the residual clay and

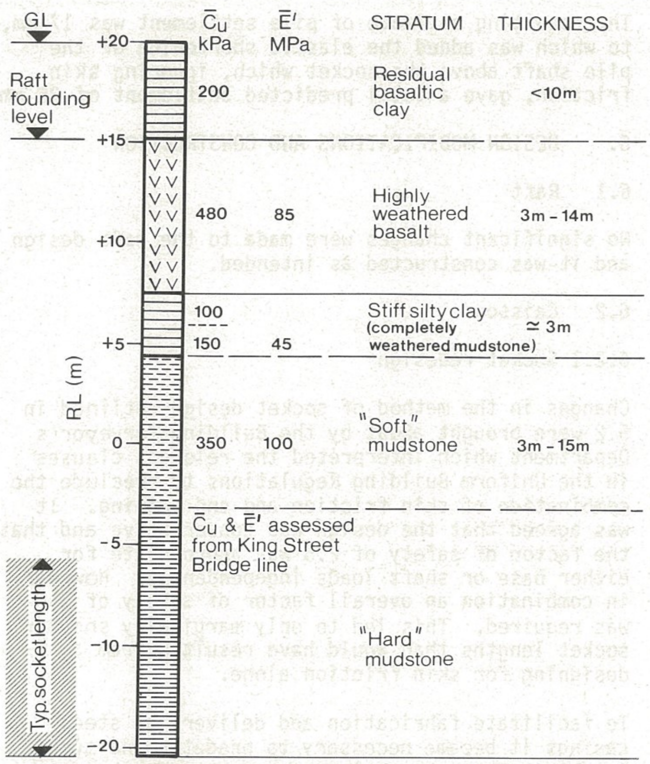

Fig.2 Typical ground profile and design parameters

the highly weathered basalt varied considerably.

4. ASSESSMENT OF DESIGN PARAMETERS

4.1 Introduction

Despite variability of the underlying strata, confidence was felt regarding the behaviour of the raft. Many buildings in Melbourne have been founded on the basalt without problems, so it was assumed that by adopting conventional design parameters an upper bound on raft settlements could be estimated. Less certainty existed at the time about the behaviour of rock sockets in the Silurian mudstones, and much time was spent establishing appropriate parameters. The typical ground profile and mean Cu and E' values assumed for design are indicated on Fig.2.

4.2 Basalt

4.2.1 Residual basaltic clay

The residual clay was of firm to stiff consistency with much of its original structure intact. Although the caissons passed through the stratum it was excavated beneath the raft and so plays only a minor part in overall foundation performance. From quick undrained triaxial tests a $\overline{Cu}$ = 200 kPa was adopted.

4.2.2 Highly weathered basalt

The basalt was highly weathered and highly jointed with a heterogeneous weathering profile. In places it was friable with frequent thin layers of soft clay sandwiched between harder bands. Sampling and testing proved difficult. Results from both this and the adjacent BHP site, gave typical undrained shear strengths for intact basalt in excess of 2 MPa. However, a single test on completely weathered basalt close to the interface with the underlying mudstone gave a shear strength of only 240 kPa. For design purposes $\overline{Cu}$ = 480 kPa was adopted as a reasonable, though probably conservative figure. Plate loading tests on the BHP site indicated an E' = 550 MPa for fractured basalt. Since the rock on the Bank House site appeared to be inferior to that on the BHP site this figure was modified by assuming that one third of the thickness comprised clay with an E' = 32 MPa. This led to a mean value of 85 MPa being adopted.

4.3 Silurian Mudstone

4.3.1 Comparison with King Street Bridge data

After much testing and examination of data from this and other sites, it was concluded that the King Street Bridge relationships for both Cu and E' versus w (Parry 1970) could be adopted for design in the Silurian Mudstone and even extrapolated to higher moisture contents than were commonly used at the time. The relationships can be conveniently represented in the form:

$$\text{Log Cu} = 4.345 - 0.126\,w \qquad (1)$$

$$\text{Log E'} = 7.206 - 0.175\,w \qquad (2)$$

where Cu and E' are in kPa and w is expressed as a percentage. Above w = 18% strengths were widely scattered making selection of design parameters extremely difficult.

4.3.2 Stiff silty clay

Although derived from mudstone the thin silty clay layer had none of the characteristics of a rock. The King Street Bridge approach was therefore considered to be inappropriate. Values of Cu = 100 and 150 kPa and $\overline{E}'$ = 45 MPa were adopted, based on quick undrained triaxial test results.

4.3.3 'Soft' mudstone

Using the extrapolated Kings Street Bridge line $\overline{Cu}$ = 350 kPa and $\overline{E}'$ = 100 MPa were derived from the moisture contents.

4.3.4 'Hard' mudstone

The 'hard' mudstone was assumed to be incompressible as far as raft stresses were concerned, but for the design of caisson sockets Cu and E' were taken from the King Street Bridge line.

5. ENGINEERING DESIGN

5.1 Raft Foundation

Floor levels in the building required the raft foundation to be built on the basalt at a reduced level of +15.25 m (Fig. 2). In both the NW and NE corners of the tower the basalt was actually below this level so the overlying residual clay was excavated locally and replaced with mass concrete.

Using the parameters discussed in Section 4, raft settlement was assessed using a simple elastic method, assuming a 2:1 (vertical:horizontal) load spread with depth and a rigid base at the top of the 'hard' mudstone at RL-3 m. Estimated loads were:

Total dead load = 344 MN
Total live load = 54 MN

and estimated settlements for different possible loading situations were:

Dead load settlement = 55 mm
Additional live load settlement = 10 mm

Two pore pressure dissipation tests were carried out on samples of completely weathered mudstone, which was considered likely to be the most time dependent layer amongst the consolidating strata. Values of 4.6 and 18.6 $m^2\ yr^{-1}$ resulted, which yielded times for 90% consolidation of 100 to 250 days. It was therefore concluded that all settlements other than those due to the addition of live load would be largely complete during the construction period.

5.2 Caissons

Different arrangements of piles and caissons were considered for the four corner columns, and it was decided that four 1.83 m diameter caissons per column would be the most economic solution. These were designed as pure rock sockets, skin friction in the basalt and 'soft' mudstone being ignored.

The capacity of the rock socket in 'hard' mudstone was assessed by dividing the shaft into 1.5 m lengths and ascribing average moisture contents to each, appropriate values of Cu being selected from the King Street Bridge line. The ultimate skin friction was assumed to be equal to Cu, since no softening of the rock was expected to occur due to stress relief and any clay smear or broken material would be removed immediately prior to concreting with a high pressure air jet. Ultimate bearing capacity of the base was taken as being equal to 4.5 Cu rather than the theoretical figure of 9 Cu, to allow for the possible influence of joints and fissures.

The expression used for the ultimate bearing capacity of the socket, S_{ult}, was therefore:

$$S_{ult} = 4.5 \frac{\pi D^2}{4} Cu_b + \pi D L \overline{Cu}_s \qquad (3)$$

Using an overall factor of safety of 2.5, this gave an allowable design load of:

$$S_{all} = \frac{4.5}{4} \frac{\pi D^2}{2.5} Cu_b + \frac{\pi D L \overline{Cu}_s}{2.5} \qquad (4)$$

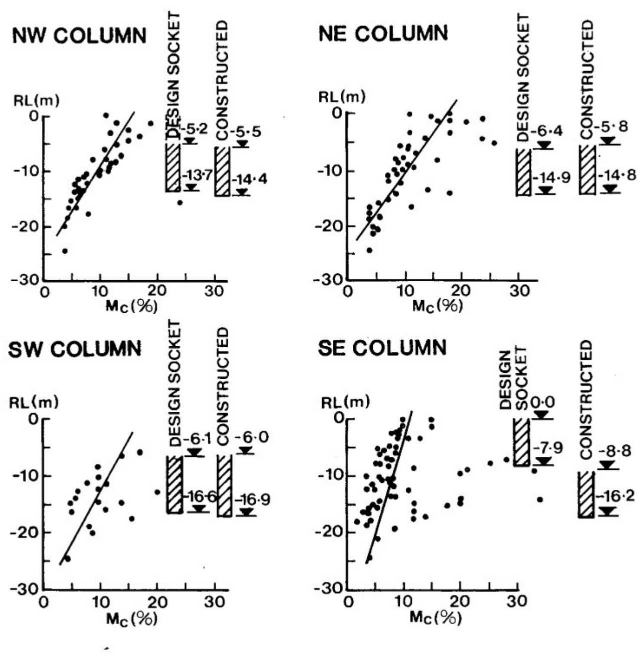

Fig. 3 Moisture contents over socket lengths

The maximum dead plus live load per caisson was 2.8 MN and the preliminary design length of each socket was estimated to be 8.5 m.

Probable settlements were calculated using the method suggested by Mattes and Poulos (1969). Using $\overline{E}'_s$ = 450 MPa and $\overline{E}_b$ = 1000 MPa, settlements for individual sockets of 10 mm were calculated. This figure was modified:

i to allow for the group interaction of 4 piles, using the recommendations of Poulos (1968), which increased the estimate to 22 mm and

ii reduced by 25% to allow for the rapid increase of stiffness with depth.

The resulting estimate of pile settlement was 17 mm, to which was added the elastic shortening of the pile shaft above the socket which, ignoring skin friction, gave a total predicted settlement of 36 mm.

6. DESIGN MODIFICATIONS AND CONSTRUCTION

6.1 Raft

No significant changes were made to the raft design and it was constructed as intended.

6.2 Caissons

6.2.1 Socket redesign

Changes in the method of socket design outlined in 5.2 were brought about by the Building Surveyor's Department which interpreted the relevant clauses in the Uniform Building Regulations to preclude the combination of skin friction and end bearing. It was agreed that the design was conservative and that the factor of safety of 2.5 was appropriate for either base or shaft loads independently. However, in combination an overall factor of safety of 3.0 was required. This led to only marginally shorter socket lengths than would have resulted from designing for skin friction alone.

To facilitate fabrication and delivery of steel casings it became necessary to predetermine socket lengths. Three boreholes were sunk at each column location and the moisture contents of the recovered samples were determined (Fig. 3). The NW, NE and SW columns were virtually identical, but the SE column had distinctly lower moisture contents. Using the King Street Bridge relationship a founding level of -7.9 m was determined for the SE group compared with -16.6(SW), -13.7 (NW) and -14.9 (NE).

6.2.2 Construction

The SW, NW, NE groups were installed without major problems, although large corestones in the basalt tended to obstruct the auger's progress. Since the water table fluctuated around +13 m RL it was decided that shafts should be permanently cased through the completely weathered mudstone but the socket in 'hard' mudstone was left uncased.

In practice, it was found necessary to case from ground level to prevent shaft collapse. Casing removal proved difficult, so casings were left in the ground, the void between shaft and ground being pressure grouted before concreting.

Each socket was dewatered, cleaned and inspected in the dry. The rock was generally in good condition with only occasional thin layers which had weathered to a silty clay. Sockets were concreted under water forming a plug. Shafts were then pumped dry, reinforcement inserted and concreting completed in one continuous pour.

One of the caissons in the SE group intersected a dyke which was not located in any of the boreholes. For this reason the socket was extended downwards to -16.2 m RL where the dyke had given way to satisfactory mudstone and a sufficient socket length had been achieved. Because it was considered undesirable to found different caissons within one group at different levels, all sockets in this group were extended to the same level.

7. PERFORMANCE AND DISCUSSION

7.1 Settlement Measurements

During the construction period (Jan'75-Jun'77) and for 12 months thereafter, settlements were monitored using precise levelling methods on a number of survey points on the raft and each of the four main external columns. A deep bench mark was established in the basalt layer at some distance from the site. This was in turn checked against a remote bench mark to ensure that absolute settlements were being recorded. In May 1983 a final set of readings was taken. Full results are shown plotted on Figs. 4 and 5 and the locations of survey points are indicated on Fig. 1.

Some survey points could not be reached until October 1975. Where this occurred (Figs. 4 & 5) they were assumed to have an initial settlement equal to the average of the other points at that time. Random variations do occur indicating that, on occasions, errors in excess of 1 mm may have been made. However, on the whole a consistent downward movement is apparent at all points, despite the need to relocate some of them during construction.

7.2 Raft Performance

From Fig. 4 it is apparent that the raft settled uniformly with less than 2 mm differential. Average settlement at the end of construction was 15 mm compared with the estimate of 55 mm (Section 5.1).

Between the end of construction and 1983 a further settlement of 2 mm occurred. This figure is within the supposed accuracy of the levelling, but it is interesting to note that the design live load was 16% of the dead load and the increased settlement since the end of construction is 13%. It is also worth noting that, as expected, settlement occurred rapidly with no significant consolidation phase.

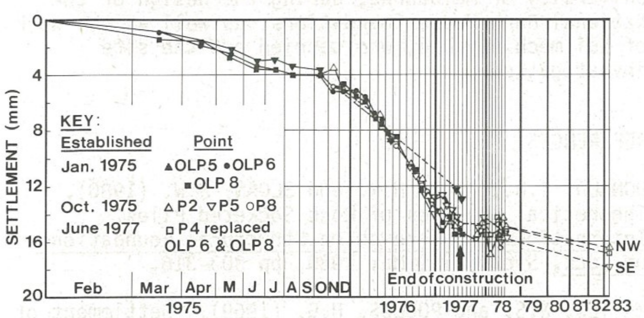

Fig. 4 Recorded raft settlements

The difference between the predicted and the measured settlements must be largely due to the conservative assumptions made at the time. However it is also associated with the difficulties of sampling and measuring the properties of rocks at various stages of weathering. The assumption that $\overline{E}'$ = 85 MPa in the weathered basalt could account for the major part of the over-estimate, but it is probable that the stiffness of the weathered clays and mudstones was also under-estimated.

The measurement of field behaviour is the only way that the time compressibility can be found, but the link between large scale field behaviour and laboratory and small scale tests remains tenuous.

7.3 Caisson Performance

7.3.1 Comparison with predictions

Fig. 5 indicates very similar performance of the caissons in the SW, NW and NE corners, with an average settlement of 12.3 mm. Settlement of the SE group was significantly higher at 17.2 mm and it

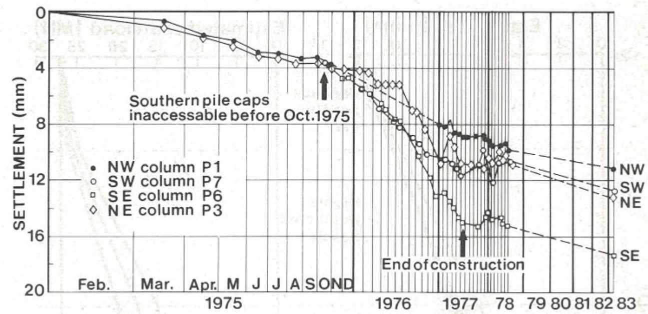

Fig. 5 Recorded caisson settlements

may be that this has in some way been influenced by the presence of the dyke.

The predicted settlement for a typical group was 36 mm, but the estimate contained several conservative assumptions. Of these the most significant must be the decision to ignore the effects of shaft adhesion through the upper layers, although grouting of the permanent casings would have ensured good adhesion with the ground.

7.3.2 Back analysis

Without knowing the settlement, or load, at points along the shaft, no true analysis can be made of caisson behaviour. However, using the general ground profile and design assumptions (Fig. 2) different possibilities have been examined.

The influence of skin friction above the socket has been assessed by dividing the shaft into 7 sections of approximately equal length. The maximum skin friction on each section has been taken as $\alpha \overline{Cu}$ where α is the normal factor used to compensate for softening during construction. Where settlement of a section is less than 5 mm, $\alpha \overline{Cu}$ has been reduced linearly to zero in proportion to the movement which which has taken place. The elastic shortening of the nth section has been calculated as:

$$\rho_n = \frac{(P_n - P_{sn})}{2} \frac{x_n}{AE_c} \qquad (5)$$

and the shaft load transmitted to the next section as $P_n - P_{sn}$.

Using a long term E_c = 20 GPa, the known caisson load P = 27 MN and the average recorded settlement δ = 12 mm, the apparent settlement and progressively reducing axial load in successive shaft sections has been estimated to the top of the uncased socket. Results are shown on Fig.6 for different α values.

Socket behaviour has been predicted using the method proposed by Donald et al (1980) for a two layer system. E' values were selected using the King Street Bridge line and the average moisture contents at the SW, NW and NE columns (Fig. 3) which resulted in E'_s = 250 MPa and E'_b = 750 MPa.

The resulting socket load deflection characteristics are shown on Fig. 7. By ensuring equal movement at the top of the socket and the base of the shaft, it is possible to assess a compatible distribution of load between the two. This is shown on Fig. 7 at the intersection of the 'socket response' and 'load delivered' lines. For the assumed King Street Bridge values it can be seen that an average value of α for the materials above the socket would be 0.37. To indicate the effect of the assumed E' values on the results an area has been shaded on Fig. 7 indicating the range of results which could be expected if E' was either twice, or one half, the assumed value.

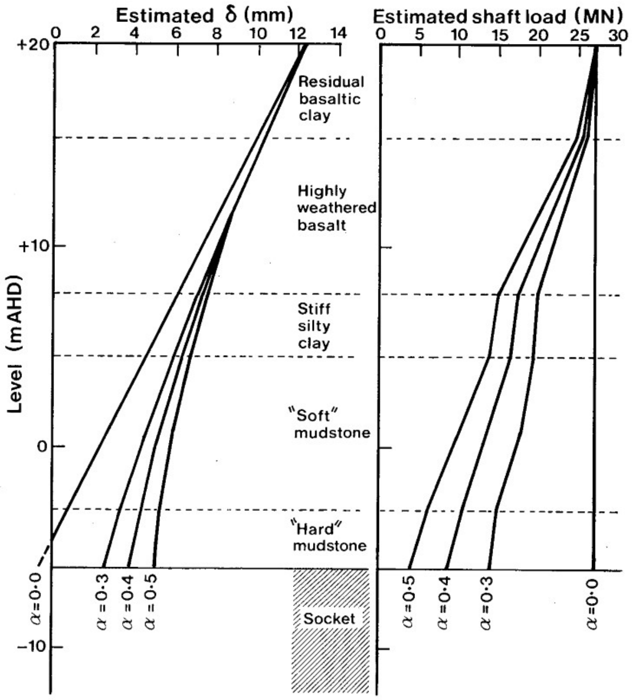

Fig. 6 Deduced shaft behaviour

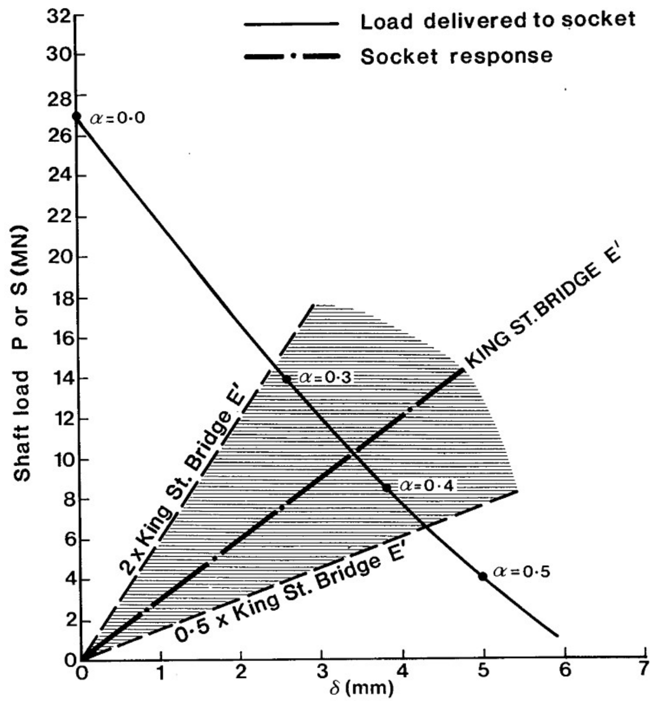

Fig. 7 Shaft socket interaction

This demonstrates that apparent α is not particularly sensitive to the E' values at the the base.

With the available information there can be no unique explanation of caisson behaviour. A high α could be considered, but this would require an extremely soft socket response to achieve the recorded settlements. Conversely a very stiff socket is possible, but very low mobilised α values would be necessary for compatible deflections. All the evidence therefore suggests that an α between 0.3 and 0.4 is appropriate for the assessment of skin friction in weathered basalt and mudstone in Melbourne under working conditions.

8. CONCLUSIONS

The back analysis of the settlement data from Bank House Melbourne cannot be used to give unique results. However, it strongly suggests that:

(i) The in situ E' values of highly weathered basalt and mudstone are likely to be much higher than those obtained from small scale laboratory or field tests.

(ii) Skin friction may, under working load, be calculated for cast in situ piles or caissons passing through the weathered basalt and mudstones in Melbourne using an α of 0.3 to 0.4.

(iii) The two layer approach to rock socket design suggested by Donald et al (1980) using the King Street Bridge E' values appears to yield reliable settlement values for large diameter, carefully constructed caissons.

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