The Design and Performance of Cast In Situ Piles in Extensively Jointed Silurian Mudstone

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SUMMARY Reasonable methods exist for the design of piles socketed into rock which is massive or which has only a few tight joints, however, difficulties arise when the rock is extensively jointed. Such difficulties were experienced during the construction of Melbourne's Eastern Freeway, which required twin bridges to be constructed over the Yarra River in an area known as the "Studley Park Fault Zone", where the Silurian sedimentary rocks have undergone severe folding, faulting and fracturing. The joints in the area were found to be slickensided, generally with thin clay coatings and with joint frequencies of 10 to 100 joints/m.

In order to confirm the practicability of economically constructing sockets in the faulted rock and in order to develop a design method, two test sockets were constructed. An end-bearing test was carried out in one and a side-resistance test was carried out in the other. During the subsequent bridge construction, eight of the service piles were proof loaded to reduce the expected high first-loading settlements. The work was supported by detailed geological logging of N size core and the rock sockets, triaxial tests on intact samples of rock, and pressuremeter tests. The results of the pile tests and the pressuremeter tests have been related usefully to joint frequency.

1 NOTATION

- D pile diameter
- f_b base resistance
- f_{be} base resistance according to linear elastic analysis
- f_{b1} base resistance at $\rho/D = 1\%$
- f_s side resistance
- f_{se} side resistance according to linear elastic analysis
- f_{su} peak side resistance
- J_f joint frequency in joints/m
- q_a unconfined compressive strength
- α side resistance reduction factor $= \dfrac{f_{su}}{q_a}$
- ρ pile settlement.

2 INTRODUCTION

The construction of Melbourne's Eastern Freeway required twin bridges to be constructed over the Yarra River in an area known locally as the "Studley Park Fault Zone", where the Silurian sedimentary rocks have undergone severe folding, faulting and fracturing. The joints in the area were found to be slickensided, generally with thin clay coatings and with joint frequencies of 10 to 100 joints/m.

Initial investigations indicated that it was desirable from economic and fixity considerations to found those bridge piers which were adjacent to the river on piles socketed into the mudstone. The design of piles socketed into rock has usually been based on allowable side and base resistance stresses which have been determined from a consideration of data pertaining to relatively intact rock. The Eastern Freeway piles were designed in 1975 and at that time rational methods of designing piles in extensively jointed rock did not exist, and it was therefore necessary to carry out the work described in this paper. In order to confirm the practicability of economically constructing sockets in the faulted rock and in order to provide a basis for designing the service piles, two test piles were constructed and loaded to failure.

This paper describes the test pile work and the analysis made of the properties of the jointed rock mass as a case history, and relates the results of the pile tests to the pile design method proposed by Williams *et al.* (1980).

3 SITE INVESTIGATION

The Eastern Freeway site was located east of Melbourne as shown in Figure 1, in an area where the Silurian mudstone was overlain by up to 3 m of Recent Alluvium. The mudstone was typical of that existing around Melbourne in that it consisted predominantly of siltstone with minor sandstone and a negligible amount of claystone. The bedding thickness varied from 1 to 100 mm with occasional beds up to 300 mm thick. The bedding dipped generally at 50° to 70° towards the north-west. It was usually possible to identify three sets of approximately orthogonal joints, with the two dominant sets being parallel to the bedding and normal to the bedding. A more detailed description of the joints is included with the description of the rock sockets in Section 4.

The weathering of the mudstone has occurred under a reducing environment which has resulted in the colour of the claystone and siltstone being blue-grey and the sandstone being a paler grey. The mudstone is similar in this respect to that encountered by Parry (1970) for the King Street Bridge and by Parkin and Donald (1975) for the Johnson Street Bridge.

Site investigation drilling consisted of one N size bore drilled at each pile position. The cores were described according to the weathering classification of Neilson (1970) and particular attention was paid

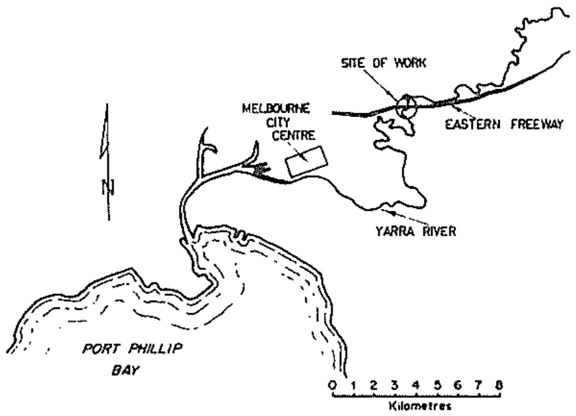

Figure 1 Locality Plan

to the condition and frequency of the joints. The frequency of the relatively blocky joint system was determined simply by counting the number of joints seen in the core for joint frequencies of less than 50 joints/m or by visually assessing the representative fragment size for greater joint frequencies. It was not considered worthwhile or practicable in this situation to endeavour to determine the frequencies of each joint set or to allow for the orientation of the bore with respect to the joint sets.

The intact fragments of rock were generally moderately or slightly weathered with a significant proportion being fresh fragments. The joint frequencies of commonly 10 to 100 joints/m indicated that the properties of the rock mass were likely to be very different from those of the intact fragments. It was therefore appropriate to carry out a series of Ménard pressuremeter tests to measure the strength and compressibility of the rock mass. Such tests have since been found particularly suited to the measurement of the in situ properties of the Silurian mudstone, e.g. Walker *et al.* (1975) and Walker (1978).

In order to provide the basis for a design method, the modulus of the rock mass as measured by the pressuremeter has been correlated with joint frequency as shown in Figure 2. The mass modulus is seen to decrease rapidly at relatively low joint frequencies in a manner similar to that observed by Deere *et al.* (1966), Manev and Avramova-Tacheva (1970) and Hobbs (1974). In a similar manner, the in situ shear strength from the pressuremeter results, calculated according to Gibson and Anderson(1961), has been correlated with joint frequency in Figure 3. The correlation shown in Figure 3 may be regarded as the lower bound of the shear strength-joint frequency correlation, because only pressuremeter tests which clearly indicated a limit pressure have been plotted. Other pressuremeter tests were made but did not indicate a limit pressure, however, such tests indicated a relatively high modulus which indicated a tighter joint system and a higher in situ strength.

The shape of the strength correlation is similar to that obtained for model rock by Lama (1974), and it is also similar to that of the modulus correlation, although the effect of joint frequency on strength does not appear to be as great as it is on modulus. The in situ modulus and strength have been combined in Figure 4 where it is seen that the modulus to strength ratio increases with modulus. The correlations in Figures 3 and 4 are reasonably well defined and they have been used to construct the correlation adopted between the mass modulus and joint frequency shown in Figure 2. An attempt was made to identify the dominant dip of the joints at each pressuremeter test position in an effort to make some allowance for any anisotropy, however, the blocky structure produced by the three joint sets made such an assessment very uncertain and the question of anisotropy was not pursued.

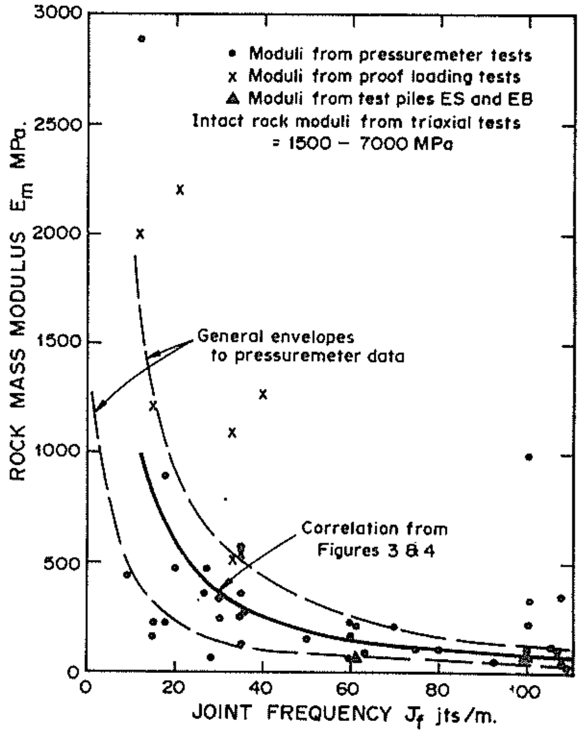

Figure 2 Correlation between joint frequency and the modulus of the rock mass.

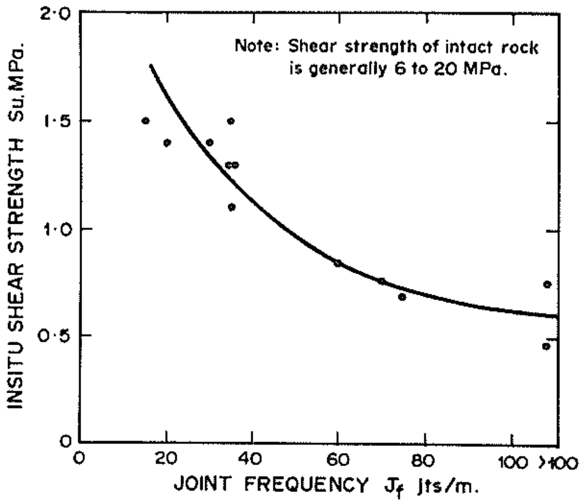

Figure 3 Correlation between joint frequency and shear strength of the rock mass.

Moisture content samples were taken from the cores approximately every 300 mm and samples were selected where possible for triaxial testing to assist in estimating the strength of intact rock on the basis of moisture content. Triaxial tests were made on samples under unconsolidated undrained conditions with a confining pressure of 0.7 MPa, thus following the procedure established by Parry (1970) and continued by Parkin and Donald (1975).

The results of the triaxial tests have been plotted against moisture content in Figure 5, where the correlation found by Parkin and Donald (1975) is

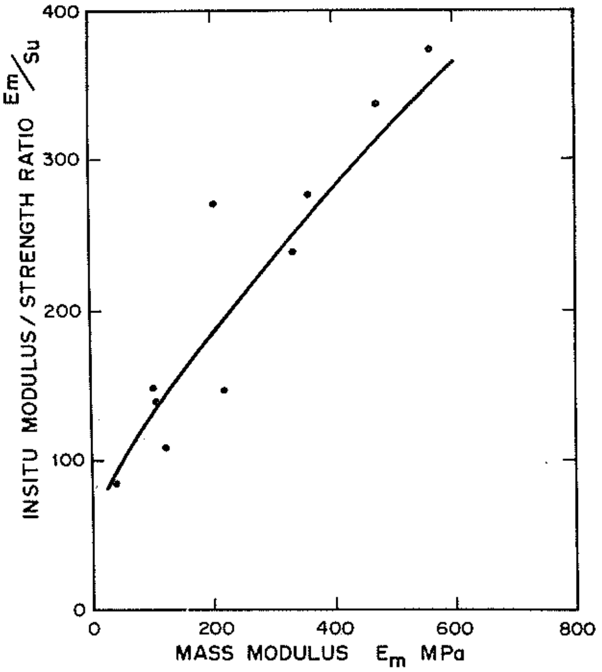

Figure 4 Relation between the modulus of the rock mass and the modulus/strength ratio

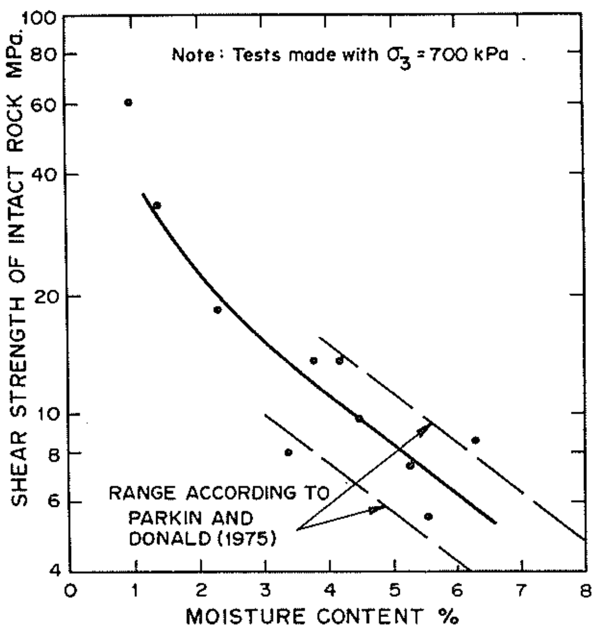

Figure 5 Correlation between triaxial shear strength and moisture content

also shown for comparison. The triaxial shear strength of the intact rock is generally in the range of 6 to 20 MPa, which is much greater than the shear strength of the highly jointed rock mass indicated in Figure 3.

4 TEST PILES

4.1 Pile Construction

The decision to construct and load to failure one end bearing only test pile and one side resistance only test pile was based on the need to:

(a) determine the feasibility of constructing stable sockets in the intensely fractured rock

(b) determine the allowable side resistance and base resistance components of pile capacity, and,

(c) assess the need to proof load the service piles in order to tighten the jointed rock and to thus reduce pile settlement during service loading. The advantages of cycling such proof loads were also to be considered.

A test site close to the service piles was selected where investigation drilling had shown that the rock was most intensively jointed and therefore likely to cause the greatest difficulty.

The sockets for the two test piles were drilled by a Caldwell 250B rig using a bucket auger, according to the depths shown in Figure 6. After the outer steel casings had been placed, the sockets were dewatered and inspected. An engineering geologist provided the following socket descriptions:

Side Resistance Socket: The exposed rock comprised predominantly blue-grey moderately weathered (Zone 3) mudstone. More highly weathered rock was brownish yellow. The mudstone was generally highly fractured and included frequent thin clay seams and

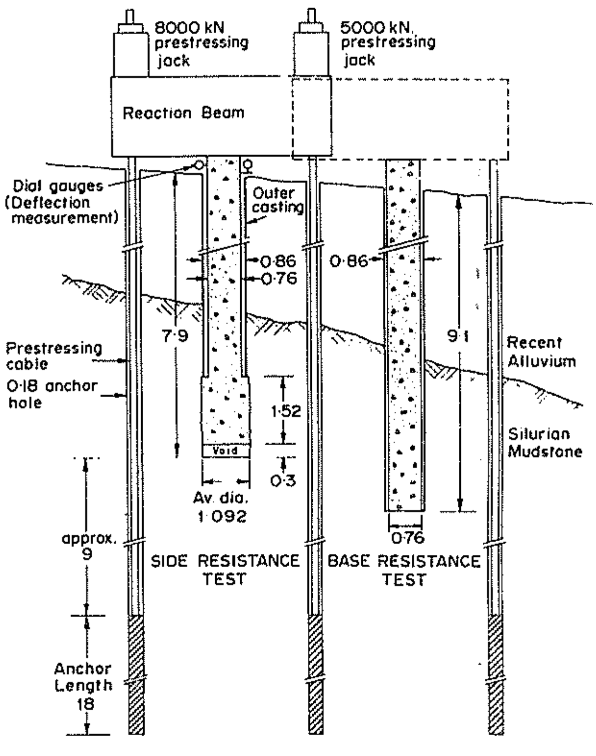

Figure 6 General arrangement of the test piles

thin irregular patches of more highly fractured and weathered rock. Three roughly orthogonal sets of joints resulted in a generally blocky structure in many parts. The two dominant joint sets were steeply dipping: one being parallel to the bedding at 50° to 70° and the other dipping at 70°. The third, less dominant joint set had a shallow dip and was often indistinct. The joints included thin clay films, or in the more fractured and lightly weathered areas, free clay. The joints were mostly planar and smooth, although some were slightly ridged and slickensided. The jointing appeared to be tight except for moderately tight areas of intense fracturing. The general joint frequency appeared to range from 40 to 120 joints/m with most of the socket exhibiting 40 to 80 joints/m. The appearance of the socket walls is sketched in Figure 7.

Base Resistance Socket: The rock mass was similar to that exposed for the side resistance test. The appearance of the socket base has been sketched in Figure 8.

The test piles were constructed and loaded according to the arrangement shown in Figure 6. The 9 m of free anchor length below the test piles was selected to minimize settlement interaction effects (Poulos and Mattes, 1975). Two precision spirit levels were attached to the reaction beam so that it could be maintained horizontal during simultaneous stressing of both rock anchors. Pile settlement was measured with dial gauges fixed to 5 m long insulated reference frames and with a precision level fixed to a rigid pedestal. The stability of the precision level was checked by sighting to a remote benchmark, and the stability of the reference frames was checked by the precision level. No movement of the level or of the reference frames was detected outside ±0.2 mm accuracy of the precision level. The pile settlements indicated by the dial gauges and by the precision level thus agreed to within ±0.2 mm.

4.2 Side Resistance Test

The socket for the side resistance test pile was drilled to a nominal diameter of 915 mm, however, overbreak increased the average diameter to 1090 mm. The test pile was constructed with a 300 mm thick polystyrene foam compressible base as shown in Figure 6. A one-dimensional oedometer test was made on a sample of polystyrene to model its behaviour under the test pile and to indicate the load carried by the polystyrene base. The oedometer test indicated that the load carried by the polystyrene would not have exceeded 50 kN and the effect was therefore neglected.

The pile was loaded in increments of 500 kN by simultaneously stressing the two cable anchors. The load was measured according to a calibration between the load and pressure of the two stressing jacks. Each load increment was held for at least one hour, during which settlement and time readings were taken to provide the basis for the load-settlement curve shown in Figure 9 and the creep rate curve shown in Figure 10. The load-settlement curve indicates a "yield" point at a load of about 2000 kN (380 kPa) which corresponds to the beginning of a sharp increase in creep rate.

Although the creep rate increases sharply, the load-settlement curve shows a significant increase in side resistance as displacement increases and at a displacement of 175 mm the load was 4400 kN (840 kPa). The load-settlement curve gives no suggestion of an abrupt failure commonly observed for smooth sockets, and it has not developed a peak value which is approximately maintained at large displacements as commonly observed for rough sockets (Williams and Pells, 1979).

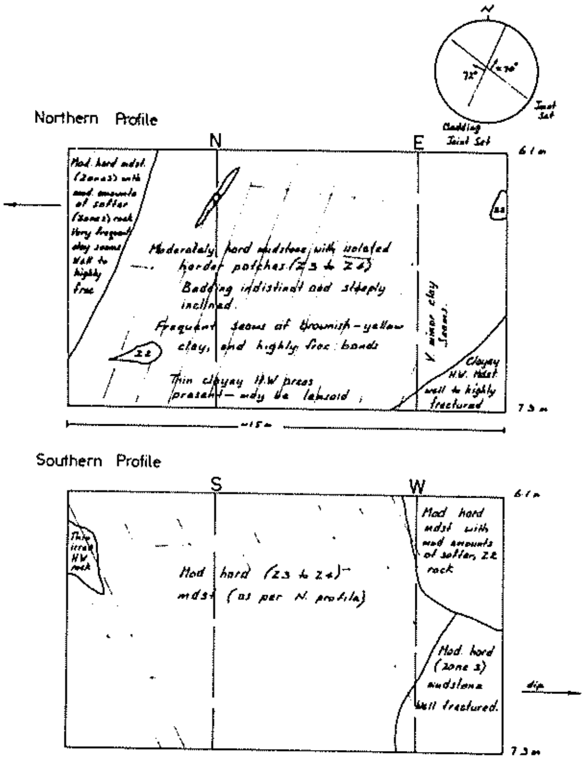

Figure 7 Geological structure of the rock exposed on the walls of the side resistance socket

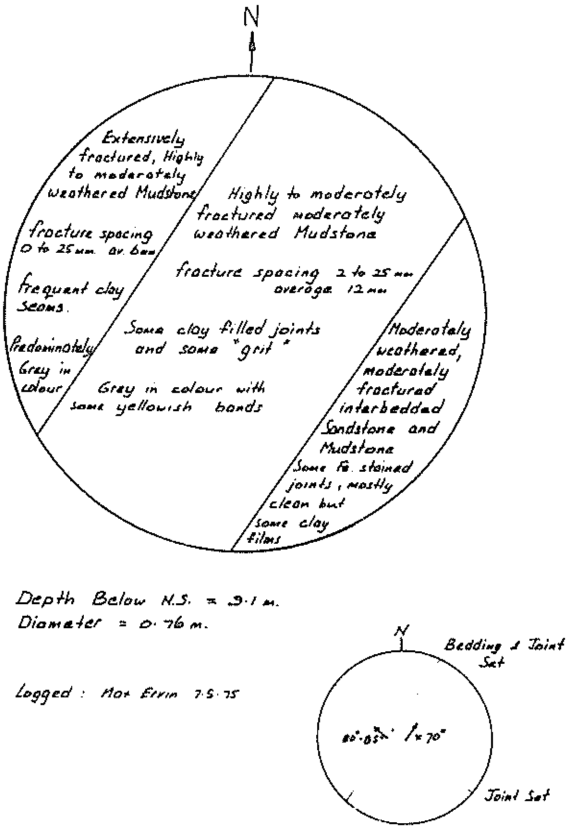

Figure 8 Geological structure of the rock exposed on the base of the end bearing socket

The peak side resistance is commonly related to the unconfined strength of the intact rock, e.g. Rosenberg and Journeaux (1976), Hvorath (1978), Pells *et al.* (1978) and Williams and Pells (1979). The unconfined strength of the intact rock in this case was between 6.0 and 8.5 MPa for mudstone with a moisture content of 3 to 5%. The side resistance factor, α, is then given by:

$$\alpha = \frac{\text{peak side resistance, } f_{su}}{\text{unconfined compressive strength, } q_a}$$

$$= \frac{.840}{6.0 \text{ to } 8.5}$$

$$= 0.10 \text{ to } 0.14$$

The value of $\alpha = 0.10$ to 0.14 is towards the low side of the general scatter of results which form the basis for a relation between α and q_a suggested by Williams *et al.* (1980), however, it lies on the generally lower bound curve recommended as a design basis.

The results of the pile test were used to calculate the modulus of the rock mass according to the method suggested by Pells and Turner (1978). The first loading modulus indicated by the initial tangent to the load-settlement curve was 70 MPa. This modulus has been plotted at the average joint frequency of 60 joints/m on Figure 2, where it is seen to be below the adopted correlation although it is well within the general scatter of results. The slopes of the unloading and reloading parts of the load-settlement curve were similar and indicated a reloading modulus of 350 MPa, which is about six times the first loading modulus. The significant increase in modulus indicates a significant tightening of the joints during the initial loading and it therefore indicates the advantage of proof loading the service piles.

4.3 Base Resistance Test

The base resistance test was made on a 760 mm diameter pile as shown on Figure 6. The load was applied and the settlement was measured in the same manner as already described for the side resistance test.

The load-settlement curve shown in Figure 11 indicates a "yielding" at a load of about 1500 kN (3300 kPa), which is approximate to the increase in the rate of creep settlement shown in Figure 10. The load-settlement curve does not indicate a peak or residual capacity but shows a continuing increase of capacity with settlement, even at the maximum settlement of 150 mm. The result is similar in this respect to the base resistance tests carried out at depths of more than three pile diameters by Williams (1980), in which the base resistance continued to increase with settlement without exhibiting a well defined peak capacity. In the absence of a well defined peak capacity, Williams *et al.* (1980) have found it convenient to normalize the load-settlement curves from base resistance tests on the basis of f_{b1}, the resistance corresponding to a settlement ratio ρ/D of 1%. In the case of the test described, f_{b1} = 1760 kPa.

The initial tangent modulus of 80 MPa, calculated according to Pells and Turner (1978), has been plotted at 100 joints/m in Figure 2 where it is seen to correspond with the adopted correlation. The pile was subjected to six cycles of loading between 500 kN and 1500 kN which indicated a significant stiffening of the rock mass and a reloading modulus of 425 MPa. The increase in modulus of about 5 times was similar to the increase observed from the side resistance test.

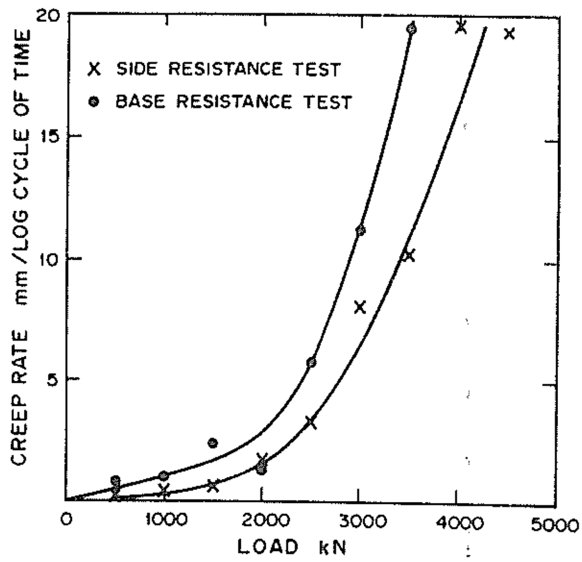

Figure 10 Creep rate at each load level for the side resistance test and base resistance test.

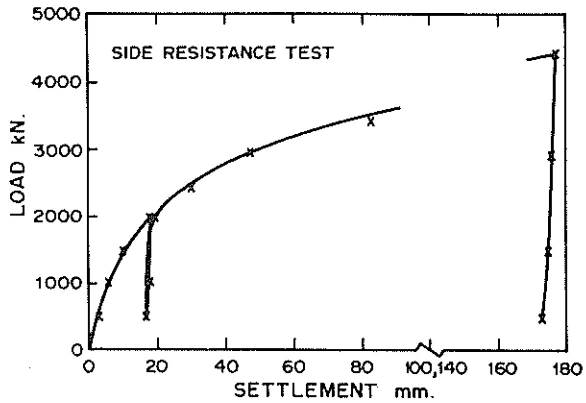

Figure 9 Load-settlement curve for the side resistance test

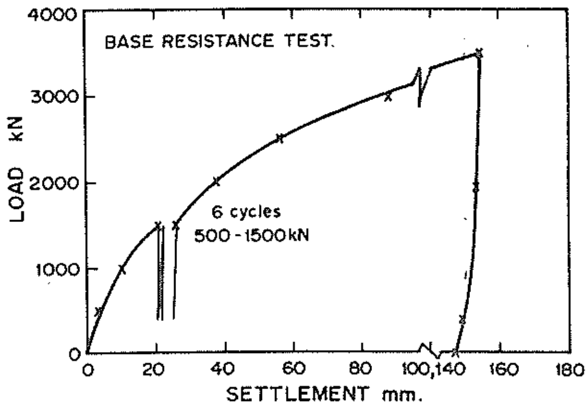

Figure 11 Load-settlement curve for the end bearing test

5 CONSTRUCTION AND PROOF LOADING OF SERVICE PILES

The test pile programme indicated that it was practicable to proceed with the installation of the proposed rock socketed piles, and that proof loading was necessary for the eight piles socketed into the most fractured rock to ensure that unacceptably large settlements did not occur under service conditions. The piles were proof loaded to 1.5 times their design load.

The sockets were constructed using rotary drilling techniques under a positive head of water, and the piles were cast under water with a tremie pipe. A single cable anchor was installed through the centre of the piles to be proof loaded and proof loading was then carried out using two prestressing jacks mounted in series.

The results obtained from the proof loading tests are presented in Table 1 where the rock mass moduli which have been calculated from the pile tests are seen to be higher than the moduli suggested by the pressuremeter tests. The reasons for the difference in moduli are not certain, however, the following three effects may have contributed to the difference. Firstly, lateral loosening of the rock round the site investigation bores may have reduced the essentially horizontal modulus measured by the pressuremeter, whereas similar loosening in the case of a rock socketed pile may not have seriously affected the essentially vertical modulus measured by the pile tests. Secondly, the jointing of the

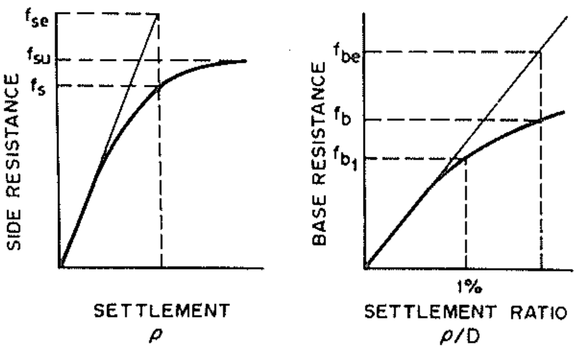

Figure 12 Definition of terms used to normalize the load-settlement curves

TABLE 1

SUMMARY OF PROOF LOADING TEST RESULTS

Pile No.	Average Joint Frequency J_f	Maximum Settlement mm	Estimated Rock Mass Moduli MPa	Measured Rock Mass Moduli: 1st Loading MPa	Measured Rock Mass Moduli: Re-loading MPa
EB	100	-	55	80	425
ES	60	-	105	70	354
E1	21	1.5	470	2400	6860
E2	12	1.6	810	2010	9630
E3	33	4.1	270	515	1790
E5	35	2.2	250	550	995
E6		1.1		1270	2210
E7	15	1.5	710	1220	3100
E8	33	1.5	280	1090	2350

Notes: (1) The average joint frequency is a weighted average determined over the length of the socket plus one diameter below the socket.
(2) The maximum settlement is the settlement at a load of 6700 kN after one previous load-unload cycle.
(3) The estimated rock modulus has been determined from Figure 2.
(4) The measured rock moduli have been determined from the pile tests as the secant moduli between 0 and 6700 kN.

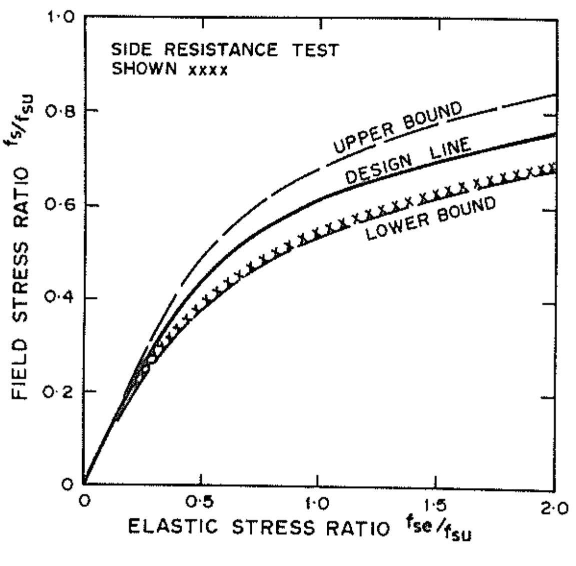

Figure 13 A comparison of the side resistance test with the normalized design curve proposed by Williams *et al.* (1980)

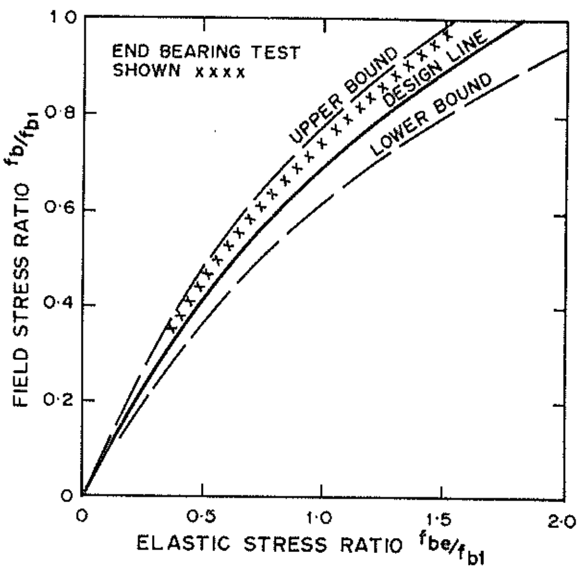

Figure 14 A comparison of the base resistance test with the normalized design curve proposed by Williams *et al.* (1980)

rock may have caused a modulus anisotropy which has been identified only with the comparison of the horizontal pressuremeter modulus and vertical pile modulus. Thirdly, the pressuremeter moduli were measured over large strains whereas the pile moduli have been determined from very small displacements. However, although there is a difference in magnitude between the pressuremeter and pile moduli, the variations in pile moduli do appear to reflect the variations in the joint frequency of the rock.

6 PILE DESIGN METHOD

The 1.52 m diameter service piles for the Eastern Freeway were designed on the basis of the results of the test piles and an assessment of the effects of jointing as indicated by Figures 2 and 3. However, rather than review the design of the Eastern Freeway piles, it is intended to assess the applicability of the pile design method proposed by Williams *et al.* (1980) to the extreme case of the extensively jointed rock encountered at the Eastern Freeway site.

The load-settlement curves obtained from the test piles have been normalized according to the elastic and plastic stress ratios, as defined in Figure 12 and plotted in Figures 13 and 14. The normalized load-settlement curves have been determined on the basis of the actual values of initial tangent moduli, maximum side resistance, f_{su}, and the base resistance at a settlement ratio of 1%, f_{b1}, as calculated in Section 4. A comparison of the normalized curves obtained from the test piles and the general envelopes suggested by Williams *et al.* (1980) indicate that the side resistance curve lies along the lower bound of the suggested envelope and that the base resistance curve lies slightly above the suggested envelope although in both cases the variation from the mean design basis is not large.

7 CONCLUSIONS

The construction and subsequent loading to failure of two test piles in extremely jointed rock has demonstrated that it is practicable and economic to use rock socketed piles in such conditions, provided that the investigation, design and construction stages are carefully matched to the rock conditions. In order to provide the basis for a sound design, it was found useful to relate the strength and modulus properties of the rock mass to the joint frequency, and it was found that the pressuremeter test was an appropriate means of achieving this.

The results of the test piles and the proof loading of the service piles indicated that proof loading the piles caused a marked increase in the rock modulus and a corresponding decrease in subsequent pile settlement, thus demonstrating the usefulness of proof loading foundations in extremely jointed rock to ensure that settlements under service conditions do not exceed the settlement criteria.

Although the two test piles were constructed in extremely jointed rock, the load-settlement curves were found to agree well with the normalized design curves suggested by Williams *et al.* (1980).

8 ACKNOWLEDGEMENT

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