

The Design of Frankipiles in Clays

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1 INTRODUCTION

This paper reports the results of a study of the performance of Franki cast-in-situ piles in clays based on the results of load tests.

There have been a number of studies of the performance of the Franki type pile in sands and gravels (Meyerhof, 1960, 1964; de Beer and Wallays, 1972; Nordland, 1982). A lesser amount of information has been published on the performance of Franki type piles in clays (de Beer et al, 1977).

It is normal for pile depths and capacity to be determined from dynamic measurements made during construction (pile set and basing resistance) but it is frequently necessary at the design or estimating stage to make predictions using static design methods based on shear strength parameters. It is known that most conventional methods of static analysis (e.g. using the Australian Piling Code 1978) will lead to an unduly conservative estimate of pile capacity, and this paper aims at producing a more realistic design method based on correlation with known test results.

2 PILE CONSTRUCTION METHOD

The construction sequence for a standard Franki hammered shaft cast-in-situ pile is illustrated in Fig. 1.

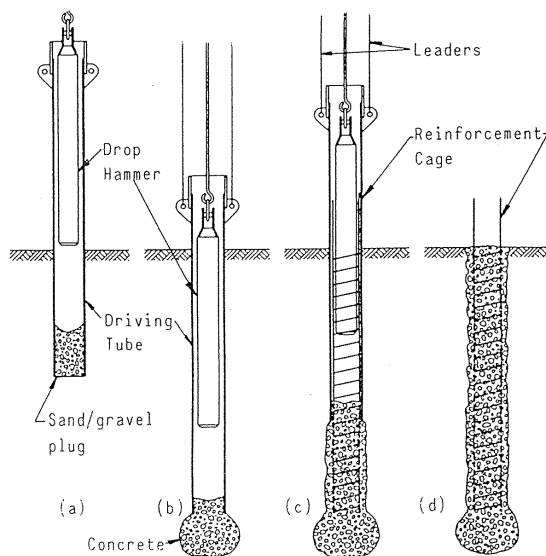


Fig. 1 CONSTRUCTION SEQUENCE FOR
HAMMERED SHAFT FRANKI PILE

A plug of sand and gravel is formed in the bottom of a heavy-walled steel tube.

The tube is then driven into the ground by the action of a drop hammer on this compacted plug (Fig. 1a), possibly with predrilling if very stiff or hard clays exist at shallow depth.

The anticipated founding depth is estimated from site investigation information and observations of the driving resistance of the pile tube. When suitable conditions are reached the tube is held in the leaders of the piling rig and the sand-gravel plug is expelled from the tube by the drop hammer. Dry cement-sand-gravel is then placed in the tube and expelled by the hammer to form a base enlargement in the ground. This process is continued until a predetermined basing resistance (i.e. energy per unit volume of displacement) is achieved (Fig. 1b). Thus, the base size will depend on the soil strength at the base location, with larger bases being formed in less stiff ground. (If the required basing conditions cannot be achieved, then the tube can be replugged and driven deeper to better ground).

The drop hammer is then withdrawn from the tube and a reinforcement cage is inserted. Small charges of zero-slump concrete are then placed in the tube and compacted under the action of the hammer, operating inside the reinforcement cage, to form the pile shaft. During this process the tube is progressively withdrawn, keeping some head of concrete inside it to prevent the ingress of groundwater (Fig. 1c).

3 LOAD TEST ANALYSIS

A study has been made of 55 load tests on Frankipiles founded in clays on 22 sites in Australia, predominantly in Queensland and South Australia. Pile load-deflection curves were analysed to determine shaft load capacity, base load capacity and base deflection, and reference was made to information from site investigation reports in which clay shear strength was determined or estimated. Most of the tests analysed were routine quality control tests, which were discontinued after reaching a predetermined load (usually twice design working load), and only six tests reached practical failure such that ultimate base load could be reliably determined.

4 ULTIMATE SHAFT RESISTANCE

All of the 55 load tests were analysed to determine ultimate shaft resistance and the average cohesion acting along the pile shaft, using the expression

$$R_{us} = \pi d L c_a \quad \dots (1)$$

where R_{us} is the ultimate shaft resistance, d is the pile diameter, L is the pile shaft length and c_a is the average adhesion stress.

Analysis for all tests was carried out by the method of Whitaker and Cooke (1966) which is illustrated in Fig. 2. It is assumed that separate response curves for the pile shaft and the pile base can be combined to give the idealised load-deflection diagram comprising three straight line segments. If the second line segment is identified on an actual test load and produced backwards, it will cut the vertical or load axis at the value of ultimate shaft resistance.

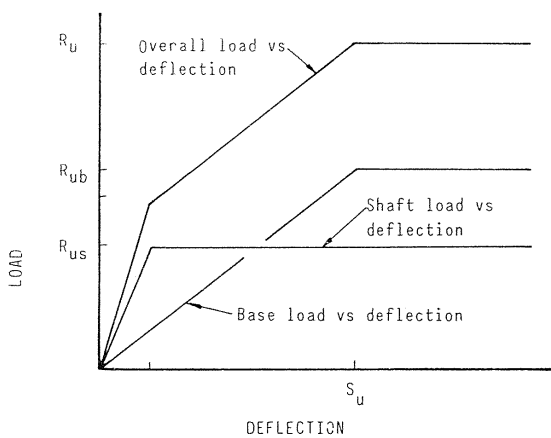


Fig. 2 IDEALISED LOAD DEFLECTION DIAGRAM

The values of average shaft adhesion (c_a) obtained from the analysis were then compared with values of undrained cohesion (c_u) measured by undrained triaxial tests on undisturbed samples. The measured cohesion values over the pile shaft depths for the sites in the study are summarised in Table 1.

TABLE 1 - AVERAGE COHESION FROM FROM SITE INVESTIGATION TESTS

Site	No. Tests	Ave. c_u (kPa)	Coef. Var'n
Tech. Coll. Mackay	10	109	0.42
Court House, Toowoomba	15	51	0.35
Toowoomba Hospital	14	68	0.42
Luggage Point	2	19	0.30
Qld Cultural Ctr.(A)	38	67	0.44
Q.G.R., Rockhampton	26	181	0.27
Bald Hills Tech. Coll.	13	236	0.61
Redcliffe Hospital	5	110	0.51
Regent Theatre, Brisb.	2	30	0.30
S.E.Q.E.B., Ipswich	17	62	0.49
Q.E.G.B., Rockhampton	9	228	1.10
Coles, Ipswich	2	75	0.30
Eventide, Brighton	9	61	0.42
Hospital, Mackay	4	78	0.31
Qld Cultural Ctr.(B)	38	67	0.44
Hastings River	2	250	0.30
S/P Hotel, Cairns	4	19	0.28
Naval Base, Cairns	9	16	0.32
Telecom, Bulimba	3	24	0.38
Consol. Fert. Pinkenba	1	21	
Various, Adelaide	16	171	0.19
Cairns Hospital	6	86	0.17

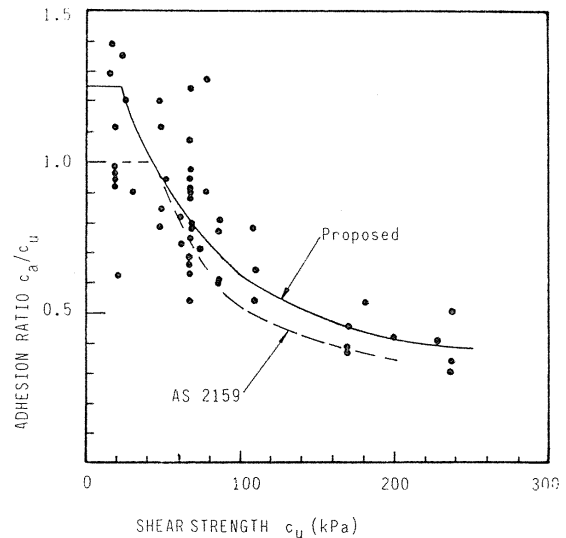


Fig. 3 c_a/c_u VERSUS c_u

Values of the ratio c_a/c_u for the individual load tests are plotted against the respective average measured values of c_u in Fig. 3. This plot indicates a range for the adhesion ratio of greater than unity for soft clay (c_u less than 50 kPa), reducing to about 0.4 for hard clays (c_u greater than 200 kPa). The reasons for this variation are a matter for speculation and may include the following factors:

- Compaction of the shaft concrete produces a rough shaft of slightly larger diameter than the outside diameter of the pile tube. This would be expected to yield high adhesion resistance, unless countered by other effects such as discussed below.
- The lower strength clays would tend to deform plastically when displaced by the pile tube. There would initially be some remoulding and strength loss, but the stresses induced by the deformation would eventually produce some consolidation and strength gain. The tests analysed were carried out as quality control tests on piling contracts and were generally performed within one to two weeks of installation. Had a longer period been allowed, it is possible that adhesion stresses would have been greater, due to a regain of strength.
- In stiffer clays, "shattering" may occur, leading to a reduction in the mass strength (Tomlinson, 1977).
- In the very stiff to hard clays, preboring would have been used and the lateral of stresses on the pile shaft would be expected to be less than those associated with the driven tube.

For the Boom Clays of the Netherlands, which are stiff, fissured, over-consolidated clays, de Beer et al (1977) have recommended a value of ultimate shaft adhesion of 0.28 of the undrained strength estimated from in-situ vane tests. They also indicate that vane tests may give strength values 1.3 to 1.5 times higher than triaxial test values and attribute this largely to sampling disturbance. These observations correspond to the values of c_a/c_u shown in Fig. 3 for stiff to very stiff clays.

As a check on the method used for estimating shaft adhesion, selected test load results were analysed by two other methods:

- The Chin Method (Chin, 1979; Chin & Vail, 1973). The load-deflection curve is modelled as two hyperbolic arcs, so that for either arc

$$S = C_1 S + C_2 \quad \dots (2)$$

will be linear where S is the pile head deflection, Q is the applied load and C_1 and C_2 are constants. It is assumed that base resistance is negligible at small deflections (i.e. the first arc of the curve) and a plot of (2) will give two straight line segments. For the first line segment $l/Q = C_1$ for all values of Q if C_2 is negligible, and the value of R_{us} can be estimated from the initial slope of the plot of (2).

The Van Weele Method (Van Weele, 1957). If the load on the pile is cycled during the test, then rebound on load removal can be plotted against the corresponding load value. This plot will become linear with higher loads. If this linear plot is produced to cut the load axis, it is maintained that it will do so at the value of ultimate shaft resistance.

Values of R_{us} estimated by both these methods are plotted against the corresponding values estimated by the method of Whitaker and Cooke in Fig. 4. It can be seen that there is no major difference between the predictions of the three methods.

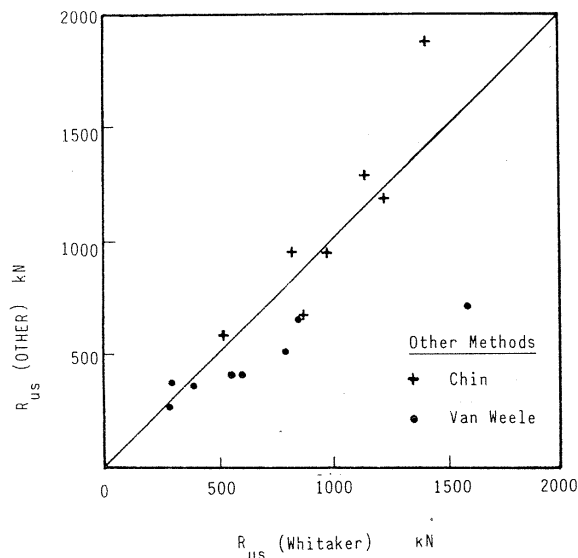


Fig. 4 COMPARISON OF ULTIMATE SHAFT LOAD PREDICTIONS

5 BASE RESISTANCE

The value of ultimate base resistance R_{ub} for a pile in clay is conventionally estimated from the expression

$$R_{ub} = c_u N_c A_b \quad \dots (3)$$

where N_c is a bearing capacity factor and A_b is the base area of the pile.

Six of the available pile load tests were taken to a stage where the ultimate resistance-penetration slope could be confidently identified. Values of R_{ub}/A_b are plotted against the respective average measured values of c_u for these tests in Fig. 5. Average values of c_u for each of the sites considered are given in Table 2.

The limited information of Fig. 5 indicates a value of N_c generally greater than 20. This is appreciably higher than the value of 9 conventionally used for the design of deep foundations. Reasons for this could include the following:

Simpson et al (1980) have indicated that a value of 9 is appropriate for stiff clays where strength values are obtained from in-situ vane or static cone penetration tests. Where lower strengths are indicated by undrained triaxial testing due to disturbance effects, a higher value of N_c is appropriate.

Some consolidation and strength gain would be expected (and has frequently been measured) in the vicinity of the pile base resulting from the stresses induced by its formation.

TABLE 2 -BASE SHEAR STRENGTH SITE INVESTIGATION TESTS

Site	No. Tests	Ave. c_u (kPa)	Coef. Var'n
Toowoomba Hospital	6	82	0.32
Luggage Point	2	60	0.30
Qld Cultural Ctr.	13	91	0.29
Naval Base, Cairns	10	90	0.39
Cairns Hospital	4	119	0.38

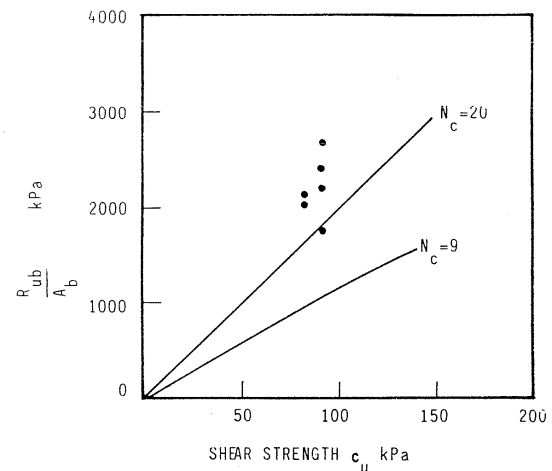


Fig. 5 BEARING CAPACITY COMPARISON

6 DEFLECTION CONSIDERATIONS

In most practical applications, failure of the supported structure will occur at some lesser deflection than that at which the ultimate base resistance is mobilised. Therefore it would be more reasonable to design the pile for a "limit state" corresponding to the failure deflection of the supported structure. For most structures, this will probably be in the range 25 to 50 mm and in the following analysis, a limiting deflection of 25 mm has been assumed.

At deflections of 25 mm, the majority of pile load tests were still characteristic of the second segment of the idealized load-deflection curve. The base resistance was still increasing quasi-elastically at this stage.

The elastic deflection of a pile base S_b would be given by

$$S_b = R_b \frac{(1 - \nu^2) I}{ED} \quad \dots (4)$$

where E is Young's modulus for the soil, ν is Poisson's ratio for the soil, R_b is the soil resistance at the base of the pile, D is the base diameter of the pile and I is an influence factor.

Randolph and Wroth (1978) have investigated the appropriate value for I for the settlement of pile bases and have proposed a value of 0.85.

Measured values of E are not generally available to the designer and he must rely on presumptive values. It is generally accepted that there is an approximate proportionality between E_u and c_u . Therefore

$$s_b \frac{E_u}{c_u} = R_b (1 - \nu_u^2) I / c_u D \quad \dots (5)$$

and substituting for $\nu_u (= 0.5)$ and $I (= 0.85)$

$$s_b \frac{E_u}{c_u} = 0.64 R_b / c_u D \quad \dots (6)$$

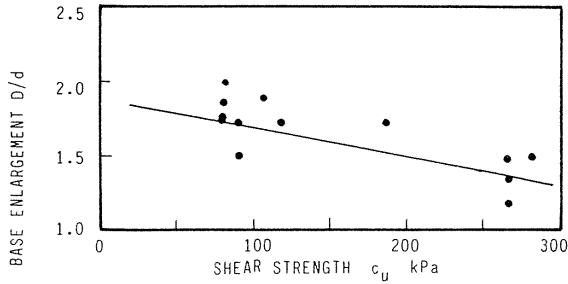


Fig. 6 BASE ENLARGEMENT vs SHEAR STRENGTH

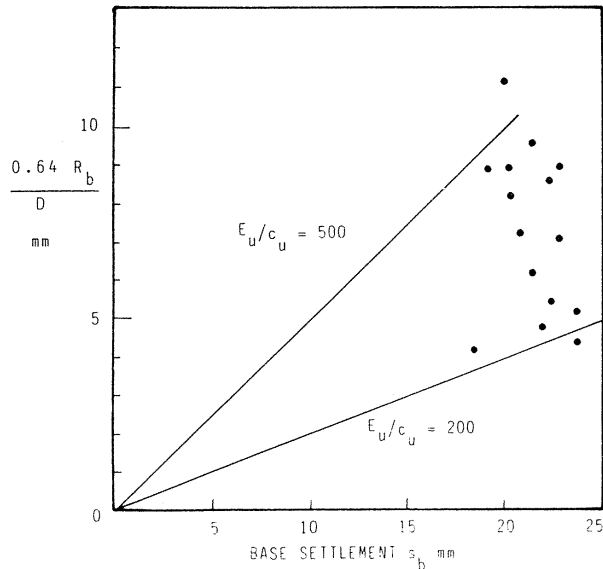


Fig. 7 SETTLEMENT RESPONSE

In order to make predictions of base resistance from (6) it is necessary to know the base diameter of the pile. Values of base diameter were estimated from the data recorded during pile construction and are presented as values of D/d vs c_u in Fig. 6. It should be noted however that Fig. 6 represents "normal practice" for Frankipile construction and that the base size can be (and is) varied to suit particular soil and load conditions.

Estimated values of S have been plotted against $0.64 R_b / c_u D$ in Fig. 7 for deflections at the pile heads of 25 mm. The values of S_b were estimated on the basis that shaft resistance is developed uniformly along the length of the pile shaft. i.e.

$$S_b = S - (R_b + R_{us}/2)L/E_p A_p \quad \dots (7)$$

where S is the settlement at the head of the pile, L is the length of the pile shaft, E_p is the Young's modulus for the pile shaft and A_p is the cross-section area of the pile shaft.

Fig. 7 shows that the values of E_u/c_u from the load tests fall in the range 200 to 500 at this order of deflection. This is the range of values commonly quoted for clays.

7 PILE DESIGN

7.1 Load For Limiting Deflection

Using a typical value of E_u/c_u of 350, base resistance can be related to base settlement for undrained conditions by (6). If S is the deflection which will produce a limiting deflection in the supported structure, then the expected value of base resistance to give this deflection R_{bf} will be given by

$$R_{bf} = \frac{547 c_u D S_f (1 - R_{us} L/2 E_p A_p S_f)}{1 + 547 c_u D L/E_p A_p} \quad \dots (8)$$

As (8) has been derived for values of base deflection of up to 25 mm, and as the extent of further quasi-elastic compression of the soil supporting the base is not known, this expression should not be used for designs with base deflections above this value.

For most practical applications, the influence of the terms involving pile modulus is relatively small and equation 8 can be rewritten

$$R_{bf} = 547 c_u D S_f K \quad \dots (9)$$

where K varies between about 0.82 for long, minimum diameter piles in strong soils up to about 0.98 for short, maximum diameter piles in soft soils. There is little error induced by adopting an average value of 0.9 for most conditions, leading to the expression

$$R_{bf} = 500 c_u D S_f \quad \dots (10)$$

Adopting S_f as 25 mm further simplifies the expression to

$$R_{bf} = 12.5 c_u D \quad \dots (11)$$

7.2 Safety Factor

The value of safety factor chosen must cover:

- . Uncertainty in loads
- . Uncertainty in soil strength due to spatial variability
- . Uncertainty in the design value of soil strength due to limited test information.
- . Uncertainty in predicted foundation response due to limitations and approximations in the design model, e.g. bias due to sample disturbance.

Experience indicates that a safety factor of 2.5 provides an acceptable level of reliability in bearing capacity pile design. In this proposed design method, settlement considerations have been substituted for bearing capacity considerations in determining base resistance. Inspection of the variability of the information in Figs. 6 & 7, on which the design method is based, indicates that it is the same order of magnitude as the variability of soil strength requiring a safety factor of 2.5. Therefore it is considered that a safety factor of 2.5 should still be used to give the normally acceptable level of reliability in foundation design. As with bearing capacity design, a lower safety factor can be justified if confirmatory information on pile performance is obtained from load tests at the design stage.

7.3 Example

The following is an example of the design of the piles for the Toowoomba Hospital site by the proposed simplified method.

Design for 400 mm diameter pile with 7 metre shaft length.

1. Calculate ultimate shaft resistance:
Over the shaft depth $c_u(av) = 68$ kPa giving $c_a/c_u = 0.80$ (Fig. 3)
 $R_{us} = \pi d L c_a = \pi \times 0.4 \times 7 \times 0.8 \times 68 = 479$ kN
2. Calculate nett base resistance for base deflection of 25 mm:
At the base level $c_u(av) = 82$ kPa
 $D/d = 1.7$ (Fig. 6) $D = 0.68$
 $R_{bf} = 12.5 c_u D = 12.5 \times 82 \times 0.68 = 697$ kN
3. Estimated total load = $479 + 697 = 1176$ kN
4. For safety factor of 2.5:
Working load = $1176/2.5 = 470$ kN

Figs. 8 & 9 show the "best" and "worst" results from two sites on which a number of load tests were carried out on similar piles. Also shown on these figures are the safe working loads for these piles, estimated as in the above example. It can be seen that these working loads are related in a reasonably conservative manner to the load-deflection behaviour of the piles tested on the sites. Deflections at working load are in the range 2 - 5 mm.

8 SAA PILING CODE

It is of interest to compare these values of working load with the values which would be estimated from the Australian Piling Code at the same safety factor. These latter values are also shown on Figs. 8 & 9, and it can be seen that the Piling Code produces significantly lower working loads. This is not unexpected, since the Piling Code aims at producing a safe "lower bound" result, whereas the design method here is based more on the average of observed test results.

The proposed design method does not require an independent settlement check. However, it is of interest to compare the measured pile deflections at working loads with those which would be estimated on the basis of the SAA Piling Code. The Piling Code gives only three presumptive values for E for the soil, related to undrained strength. In order to improve the sensitivity of this comparison, a similar but continuous relationship of E to c_u proposed by Poulos and Davis (1980) was used. The results of this comparison are plotted in Fig. 10 for the first segment of the load-deflection curves (i.e. the range of approximate linear elastic response). It can be seen that agreement is reasonably good, considering the uncertainties in the presumptive values of E .

9 CONCLUSIONS

From the analysis of load test results a design method has been developed for the Franki type enlarged base cast-in-situ pile in clay.

Ultimate shaft adhesion values have been found to be somewhat higher than the design values recommended by the SAA Piling code in the absence of any other information. However, these values are not as high as intuition might expect for a hammered shaft in stiff clay; the reasons for this cannot be fully explained at this stage.

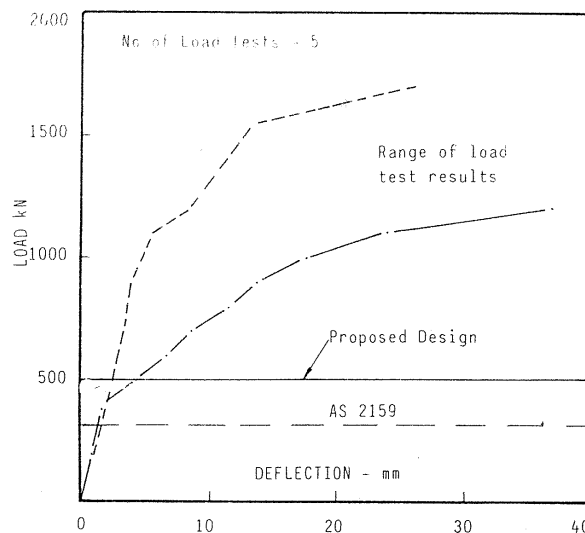


Fig. 8 LOAD TEST RESULTS - TOOWOOMBA HOSPITAL

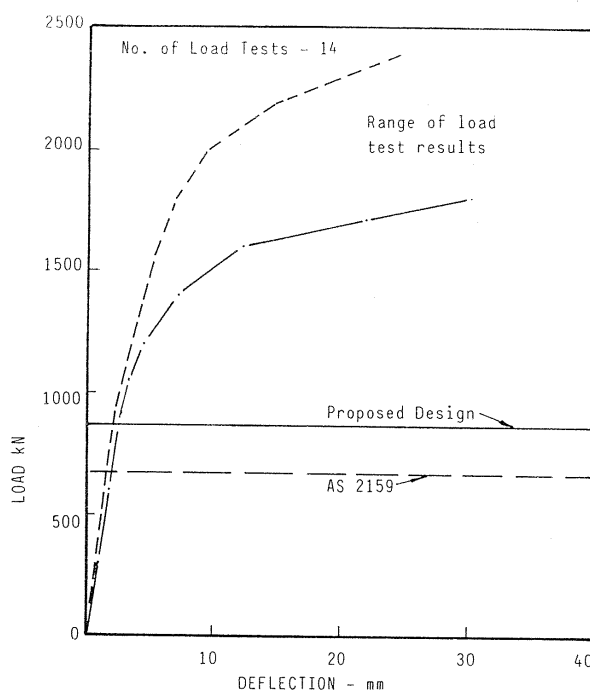


Fig. 9 LOAD TEST RESULTS - QLD CULTURAL CENTRE

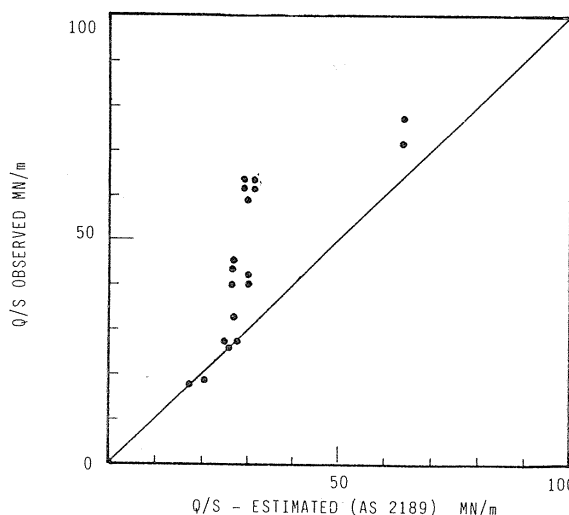


Fig. 10 COMPARISON OF OBSERVED and ESTIMATED SETTLEMENTS

Nett ultimate base resistance appears, on limited information, to be considerably higher than the value which would be conventionally predicted. However, unacceptable deflections of the supported structure would occur in many applications before this condition was reached. A more reasonable design procedure has been developed, based on deflection considerations, and limiting deflections may be selected to correspond to those of the supported structure.

In practice, pile construction, depth and capacity are usually determined by dynamic considerations in driving and basing, the static design methods being used more as a means of predicting or estimating pile depth than as a construction control. This method provides a realistic means of prediction and estimation for the normal range of conditions under which they are installed. It also provides a supplementary control method for use where deflection limitations are outside the range of normal acceptance.

10 ACKNOWLEDGEMENTS

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D.J.D.

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