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The Use of Large Scale Load Tests in Foundation Design and Construction

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SUMMARY Data from a number of pile load tests, and large scale plate bearing tests on rock are presented. The results are compared with predicted behaviour, and the variety of uses of the tests is discussed.

1 INTRODUCTION

Large scale load tests have historically been associated with the design of piles, because of the uncertainties involved in this type of foundation (Tomlinson, 1977). The trend towards much higher design loads for foundations on rock has also exposed an inadequacy in theory, and recourse may be made to large scale plate bearing tests in these cases.

Large scale load tests can therefore be seen to be mediating in the interaction between theory and practice in these types of foundation design.

The principal uses of large scale load tests are therefore:

a) To confirm design assumptions with respect to both capacity and load-deflection characteristics, and

b) To check on construction practices.

Other less common, but nevertheless real uses can be identified. These are:

c) As a site investigation tool, and

d) To comply with statutory requirements.

The results of large scale load tests at four separate sites within the Sydney Basin are presented in this paper, and the uses of the tests discussed. An attempt has been made to adequately represent the foundation conditions at each site, to describe the relevant installation procedures and to present the essentials of the test results. For clarity, intermediate load cycles which were carried out on many of the tests have been omitted from the figures.

2 SITE 1: BUILDING FOUNDATIONS - SYDNEY

2.1 Proposed Structure

A deep basement excavated through a residual clay/weathered shale bedrock profile was proposed for a multiple storey building on this site. Over approximately 50% of the site, the excavation intersected fresh Wianamatta Shale and pad footings were specified. It was proposed to use bored piers carried through the weathered layers to fresh shale as the foundations for the balance of the site.

2.2 Design Parameters

Following an initial site investigation, and in accordance with bearing stresses recommended by Pells et al (1978), a maximum allowable bearing pressure of 3.5 MPa for both pads and piers was proposed with a minimum depth of penetration of 0.5 m into fresh shale.

It was appreciated that the fresh rock had sufficient strength to enable even higher values of bearing pressure to be adopted. However, the additional work that would have been necessary to check for the effects of variability of the foundations, and the higher settlements which would have resulted were considered to outweigh the potential savings, and the value of 3.5 MPa was retained.

This bearing pressure is well in excess of that contained in Ordinance 70 (N.S.W. Government, 1976) which specifies statutory maximum limits. It was only approved by Council subject to the completion of a load test programme. It was arranged to carry out this programme at base level of the foundation, following bulk excavation.

2.3 Foundation Conditions at Test Locations

Three plate bearing test locations were chosen to give a good cover of the site. On the basis of visual appraisal of the excavated surface, Test A was located on fresh, intact shale, expected to easily carry the applied loads; Test B was located on fractured, slightly weathered to fresh shale, expected to be marginal; and Test C was located on weathered shale in an area where it was expected that bored piers would be required. Locations A and B were investigated by coring. Details of the rock strength, fracturing and defects at these locations are shown on Figure 1. Location C was investigated by excavation following the completion of the test.

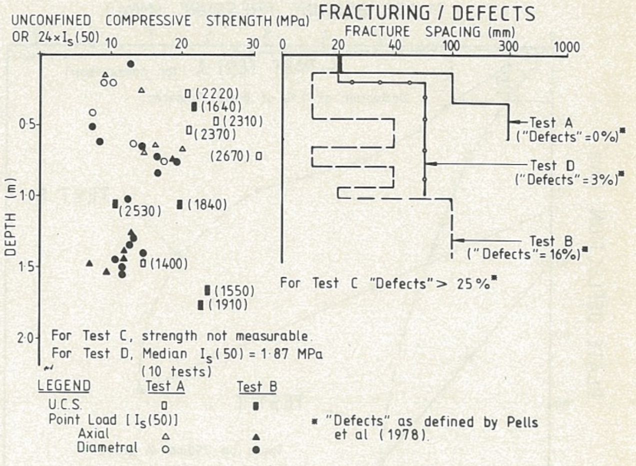

Figure 1 Foundation Conditions
for Tests, A, B (Site 1) and Test D (Site 2)

2.4 Test Procedure

Each location was initially tested to around 8 MPa, using a 750 mm diameter plate. Failure was not attained at site A, so the loading was repeated with a 305 mm plate. The test results are shown on Figures 2 and 3, together with the results of Test D, from Site 2 (see below).

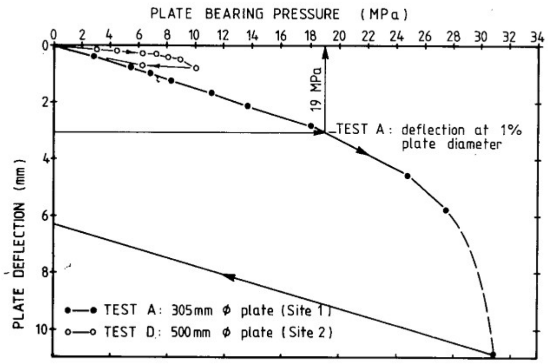

Figure 2 Results of Plate Test A (Site 1) and Test D (Site 2)

2.5 Discussion of Results

As expected, the rock at location A easily complied with the test requirements, and in fact can be classified as Class I material (using the classification of Pells et al, 1978). The results from locations B and C were interesting. Figure 1 shows that the strength results on intact core for location B were as high as for location A. However some joints with weathered surfaces were visible on the surface, and close assessment of the core defects and fracturing indicated that the rock should be classified as Class III, or even Class IV. This was confirmed by the magnitude of deflection sustained in the load test.

Similarly, test location C was initially classified as Class IV, but was shown by the test to be Class V. Subsequent excavation of the foundation exposed an inclined seam of clay/extremely weathered shale which would account for the large deformations measured.

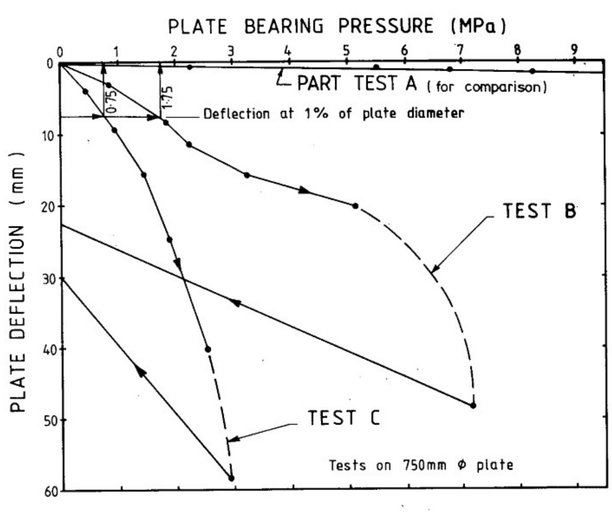

Figure 3 Results of Plate Tests B and C (Site 1)

3 SITE 2: MARINE STRUCTURE - PORT KEMBLA

3.1 Proposed Structure

Extremely high pile loads, ranging from 1200 kN to a maximum of around 7000 kN were proposed for this structure. A variety of pile sizes was included in the design, ranging from 600 mm to 1200 mm diameter hollow steel tubes. The smaller piles had L/d ratios in the range 30 to 45, and were designed to take load in both friction and end bearing, while the 1200 mm diameter piles had an L/d ratio of around 10 and were designed as end bearing only. Following driving, these larger piles were to be mucked out, and the ends plugged with concrete.

3.2 Subsurface Conditions

The full soil profile consisted of a zone of medium-dense to very dense sand, including some indurated (cemented) sand layers, overlying a clay zone, with a variable gravel zone in turn overlying bedrock (Figure 4). Bedrock was comprised of the Budyong Sandstone, which was generally a fine grained, grey, tuffaceous sandstone/siltstone. For the over-water locations which had been subject to dredging, varying proportions of the upper part of this profile were not present.

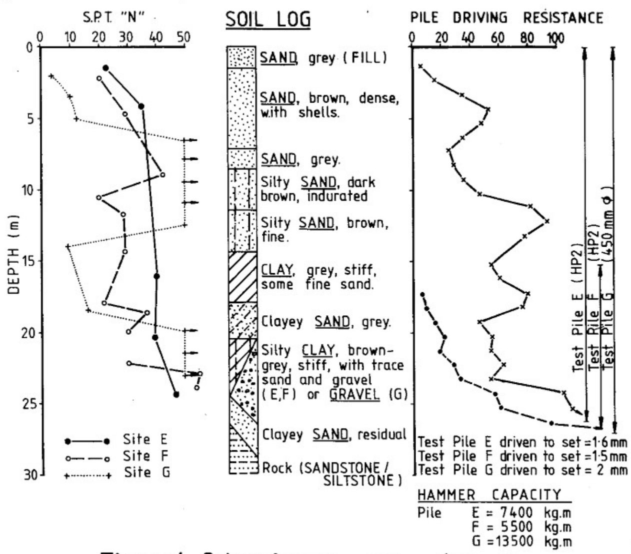

Figure 4 Subsurface Conditions (Site 2)

3.3 Plate Bearing Test (Test D)

In order to check the end bearing capacity of the 1200 mm piles an initial pile was driven to the design depth and set, mucked out, and a 500 mm diameter plate test carried out on the rock at the base of the pile. The rock below the test level was cored for assessment and testing (Figure 1). A thin highly fractured zone near the pile toe may have been due to the effects of driving and mucking out, but beyond that, the fracture spacing increased, and was largely confined to bedding plane partings. The mean Point Load Strength Index of 1.87 MPa implied unconfined rock strengths of around 45 MPa, which are at least twice as high as those obtained for the Wianamatta Shale at Site 1.

Because of physical limitations of the test it was only possible to obtain a maximum pressure of 10 MPa, which is an indicated factor of safety of around 1.6. Only very low deflections were recorded at this load.

The specified foundation conditions of pile penetration, set, and rock strength and quality were confirmed by the test. A close check was retained on these factors throughout the ensuing construction by means of coring the toe of each pile prior to pouring the concrete plug.

3.4 Pile Load Tests

Parts of the site, and adjacent sites had been the subject of previous investigations. The initial stage of the geotechnical investigation for this project was therefore to obtain and collate the existing information. As part of this exercise, attention was directed towards the results of two series of pile load tests, Pile Tests E and F, which had been carried out some years previously in connection with a nearby structure.

Pile tests E and F were carried out on BHP HP2 hollow segmented piles. The data provided showed that they were both initially driven and tested at shallow depths, then subsequently driven to refusal on rock and tested again.

Only the final tests on rock are shown here (Figure 5). These tests were carried out concurrently with site investigation drilling, and were primarily designed to determine the depth of penetration which would be necessary to carry the imposed loads. The use of these tests was therefore basically a site investigation technique.

The results of tests E and F were re-analysed, and utilized in the design of the new work. The new work was sufficiently different, however, to necessitate additional pile testing. Test G was carried out during the early stages of construction on a 450 mm diameter, hollow steel pile driven to rock (Figure 5). The test provided a check on the design strength values, and the specified driving sets for the piles. Similar results were obtained to those from Tests E and F, and confirmed the design parameters of the smaller piles in the structures (see Table 2).

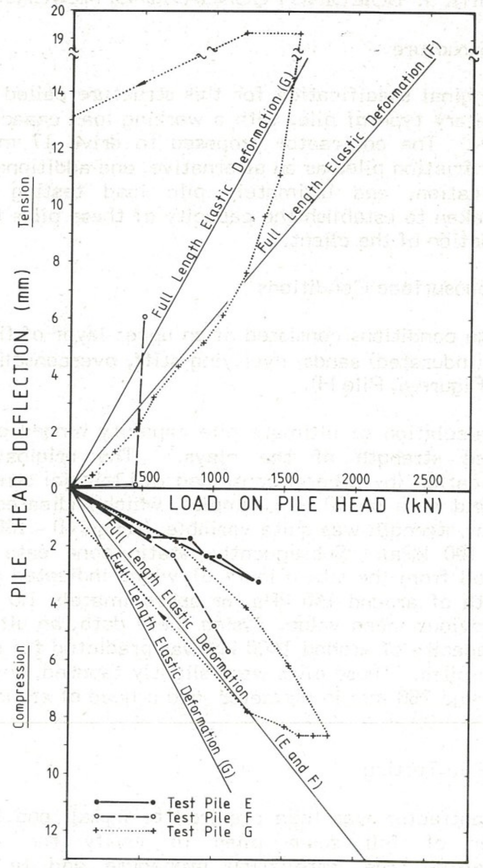

Figure 5 Pile Load Tests E, F and G (Site 2)

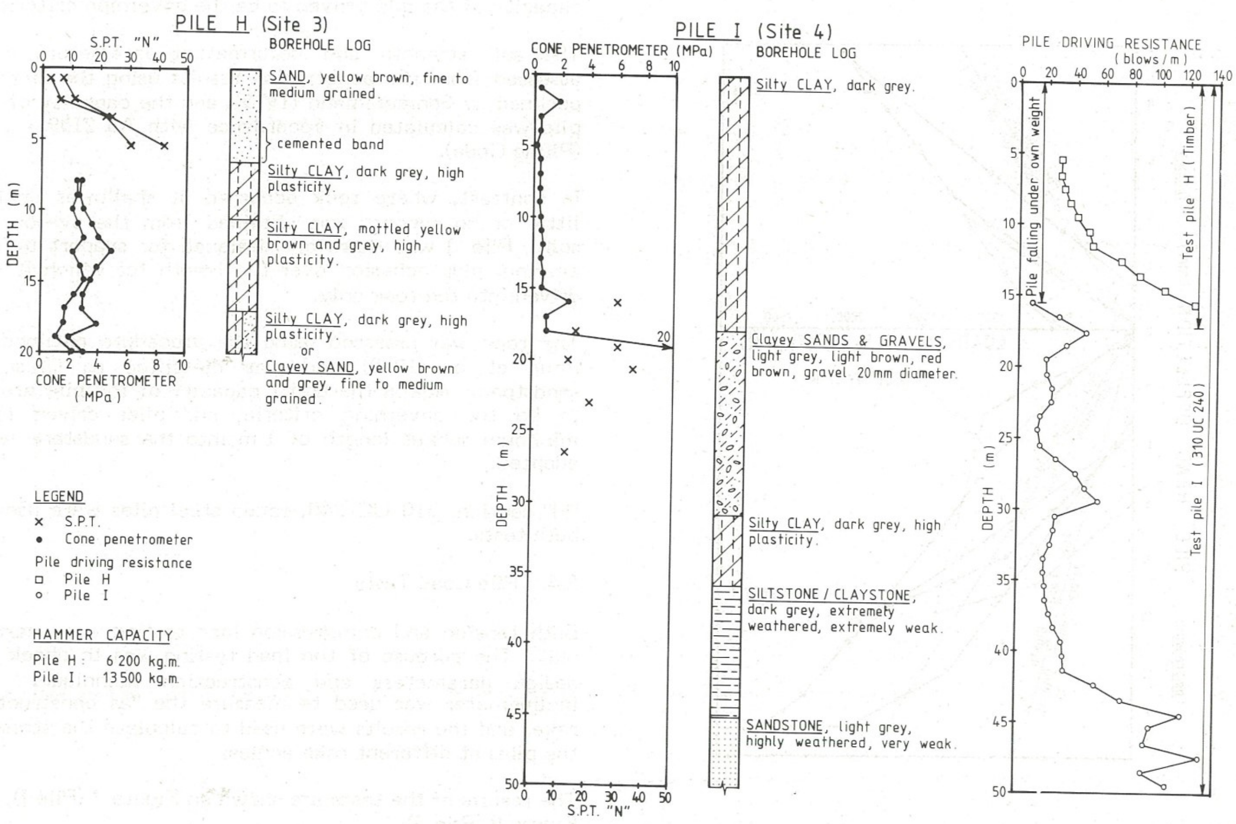

Figure 6 Foundation Conditions - Pile H (Site 3) and Pile I (Site 4)

4 SITE 3: BUILDING FOUNDATIONS, NEWCASTLE

4.1 Structure

The original specification for this structure called for a proprietary type of pile, with a working load capacity of 700 kN. The contractor proposed to drive 17 m long timber friction piles as an alternative, and additional site investigation, and ultimately pile load testing were undertaken to establish the capacity of these piles to the satisfaction of the client.

4.2 Subsurface Conditions

The site conditions consisted of an upper layer of fill and dense (indurated) sands, overlying stiff, overconsolidated clay, (Figure 6, Pile H).

The calculation of ultimate pile capacity hinged on the assessed strength of the clays. The original site investigation (by others) contained UU triaxial strength, and field data on 50 mm samples, which indicated that the clay strength was quite variable, (range 40 - 160 kPa, mean 100 kPa). Subsequently, static cone data were obtained from the site (Figure 6), which indicated a clay strength of around 150 kPa, or approximately 1.5 times the previous mean value. Using these data, an ultimate pile capacity of around 1400 kN was predicted for driven timber piles. These piles were slightly tapered, having a tip around 280 mm in diameter, and a head of around 550 mm.

4.3 Pile Testing

The contractor was then required to install and test a number of full scale piles to verify the design calculations, the installation procedure and to some extent, the variability across the site. Only compression tests were performed. The site proved to be quite uniform, and a typical test result (Test H) is shown on Figure 7.

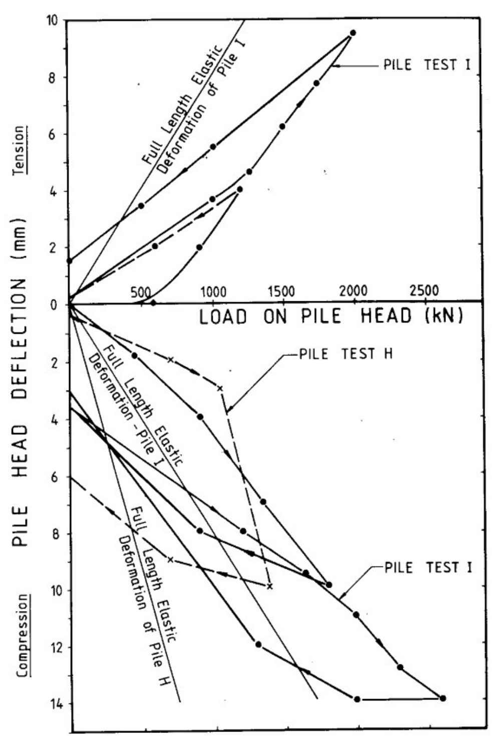

Figure 7 Pile Load Test H (Site 3) and Test I (Site 4)

4.4 Discussion of Results

The test results were remarkably close to the design ultimate load value which was predicted on the basis of the static cone data. The piles had a factor of safety of at least 2 at maximum working load, and were accepted by the client as an alternative foundation type.

This case illustrates the importance of adequate site investigation technique, and highlights the value of in-situ penetration testing.

5 SITE 4: TOWER FOOTINGS, LOWER HUNTER VALLEY

5.1 Proposed Structure and Loads

The footings at this site were required to carry tower legs subjected to high reversible uplift and compression loads of the order of 1600 kN and 2400 kN respectively. The structures could only accommodate very small total and differential settlements.

5.2 Foundation Conditions and Investigation

The stratigraphy consisted of highly variable alluvium comprising soft normally consolidated clays and medium dense sands, overlying weathered siltstone/sandstone bedrock at depths between 5 and 35 metres. The alluvials were investigated using Dutch Cone probes and SPT's (Figures 6 and 8), while the weathered siltstone/sandstone was cored and tested for strength using the Point Load Test (Figure 8).

5.3 Design Parameters

Where bedrock occurred at depth, the soft upper clay layer was underlain by more competent soil materials as at the location of Test Pile I. This pile was driven to set on rock, but because of the high L/d ratio, was nevertheless designed as a friction pile. The uplift capacity of the pile proved to be the governing criterion.

The soil strength and deformation parameters were assessed from the penetration results using the analysis outlined by Schmertmann (1975), and the capacity of the pile was calculated in accordance with AS 2159 - 1978 (Piling Code).

In contrast, where rock occurred at shallower depths, little or no support was obtained from the overburden soil. Pile J was therefore designed for support by end bearing, plus adhesion over the length for which it was driven into the rock only.

The rock was assessed using the procedure outlined by Pells et al (1978) and was classified as Class IV sandstone. Again, the uplift capacity of the pile proved to be the governing criteria, and piles driven to a minimum socket length of 1 m into the sandstone were adopted.

"H" section, 310 UC 240, raked steel piles were used in both tests.

5.4 Pile Load Tests

Both tension and compression load testing were carried out. The purpose of the load testing was to check the design parameters and construction technique. An inclinometer was used to measure the "as constructed" rake, and the results were used to calculate the stress in the piles at different rake angles.

The results of the tests are shown on Figure 7 (Pile I), and Figure 9 (Pile J).

TABLE I: SUMMARY OF PLATE LOAD TEST RESULTS

DESIGN/PREDICTED VALUES				TEST RESULTS			
Test Identification	Allowable Bearing Pressure (MPa)	Rock Modulus (MPa)	Rock Class/ Type	Ultimate Capacity (MPa)	Allowable * Bearing Pressure (MPa)	Rock Modulus (MPa)	Rock Class
A (305mm plate)	3.5	-	I - II Shale	32	19.0	1350	I
B (750mm plate)	1.0 - 3.5	-	III - IV Shale	7.1	1.75	130	IV
C (750mm plate)	0.7 - 1.0	-	IV - V Shale	2.9	0.75	70	V
D (500mm plate)	gt 6.0	2000 - 3000	II Shale	gt 10.26	N.D.	5000	I - II

* = Determined for settlement = 0.01 x Plate diameter
N.D. = Not determined
gt = Greater than

TABLE II: SUMMARY OF PILE LOAD TEST RESULTS

TEST	DESIGN PREDICTED VALUES						TESTED VALUES					
	Shaft Adhesion (kPa)	Allowable End Bearing Pressure (MPa)	Design Capacity (kN)	Modulus (MPa) Shaft	Base	Rock Class	Ultimate Shaft Adhesion (kPa)	Ultimate End Bearing Pressure (MPa)	Ultimate Capacity (kN)	Modulus (MPa) Shaft	Base	Rock Class
E (Steel, HP2)	-	-	-	-	-	-	gt 30 (average)	gt 2.0	gt 1315 (C)	30	3000- 6000	ND
F (Steel, HP2)	-	-	-	-	-	-	gt 30 (average)	gt 6.0	gt 1015 (C)	20	3000	ND
G (Steel, 450mmØ)	30	gt 6.0	2500 (C,U) 1300 (T,U)	30	2000	-	41	gt 2.0	gt 1800 (C) 1600 (T)	22	gt 10000	ND
H (Timber, tapered)	72 (U, average)	-	1480 (C,U)	32	-	-	70	-	1400 (C)	45- 80	-	-
I (Steel, 310UC240)	8-20 (A,Clays) 20 (A,Sands) 50 (A,T,Socket) 100 (A,C,Socket)	1.0	2000 (C,A) 900 (T,A)	-	-	V (Base)	gt 60 (average)	ND	gt 2600 (C) gt 2000 (T)	8	200	V
J (Steel, 310 UC 240)	175 (A,T,Socket) 350 (A,C,Socket)	3.5	800 (C,A) 400 (T,A)	-	-	IV (Socket + base)	790 (T, socket)	ND	gt 2000 (C) 1200 (T)	-	400	IV

ND = Not determined T = Tension gt = greater than U = Ultimate
\- = Not applicable C = Compression A = Allowable

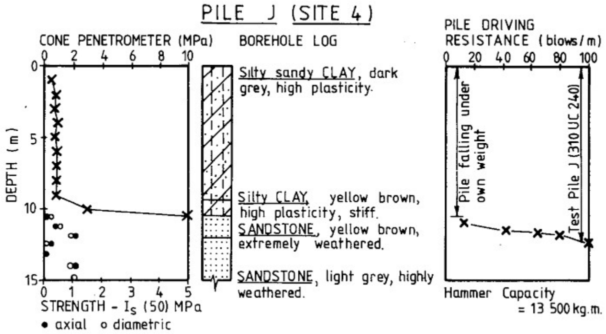

Figure 8 Foundation Conditions
Pile J (Site 4)

5.5 Discussion of Results

Inspection of the results shows that a clearly defined failure was attained for the tension test on Pile J, while some reserve capacity remained at the termination of the tension test on Pile I. It is considered that there may be inaccuracies apparent in the deflections recorded in the latter stages of the compression test on Pile J, but it is apparent that for both piles a failure condition was not attained in compression.

The test results confirmed the design parameters. The results of the tension tests indicate that the parameters for socket adhesion in sandstone (Pells et al, 1978) apply to driven as well as bored piles, at least for the weaker classes of rock.

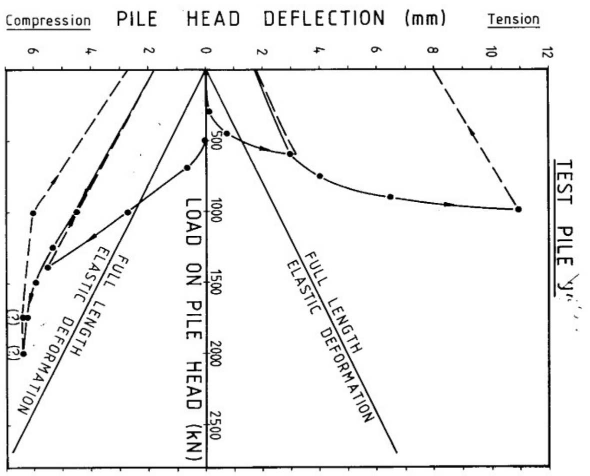

Figure 9 Pile Load Test J (Site 4)

6 COMPARISON OF RESULTS

The results of the plate bearing tests are summarized in Table 1, and where possible, are compared with the original design values or predicted conditions. Similarly, the results for the pile load tests are summarized in Table II.

Space precludes a complete discussion of these tables, but it is apparent that in many cases reasonable to quite good agreement was obtained between predicted and test results. It should be noted that the predicted/design values have been based on a mixture of analytical and empirical approaches. For foundations on rock, extensive use has been made of the recommendations of Pells et al (1977), while piles in general were designed in accordance with the SAA Piling Code, occasionally modified by other accepted methods in respect of the shaft adhesion factors adopted (e.g. Tomlinson, 1977). Elastic method were invariably adopted for the estimation of settlement (Poulos and Davis, 1980).

7 CONCLUSIONS

1. Large scale load tests can be used in a variety of ways. In nearly all cases however, the tests act to supplement or confirm the design values. The increased confidence gained from the tests may permit higher design bearing values in two distinct ways. Firstly (mainly in soils), design estimates of ultimate load may be confirmed, so that lower overall factors of safety may be adopted, resulting in higher design loads. Secondly (mainly in rock), values of ultimate load may be established so that realistic design parameters can be adopted.
2. Reliable site investigation techniques are very important for adequate foundation design. One of the examples presented shows the value of penetration testing in clays, but the techniques adopted will vary with the site conditions. For foundations on rock, it is imperative that due account be taken of the effects of discontinuities, as well as the intact material strength.
3. Certain of the data presented support the design values recommended by Pells et al (1978). Results also indicate that piles driven into the weaker classes of rock develop similar shaft adhesion over the socket length to those obtained from bored piles.

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