Fifth Australia-New Zealand
Conference on Geomechanics
Sydney, 22-23 August 1988

Instrumented Dynamic and Static Pile Load Testing at Two Bridge Sites

I.J. HAUSTORFER
Geotechnical Engineer, Road Construction Authority
S. PLESIOTIS
Geotechnical Engineer, Road Construction Authority

SUMMARY This paper presents the results of static load testing and instrumented dynamic testing using a Pile Driving Analyser undertaken at two bridge sites. Two of the test piles were founded into weakly cemented limestone which was overlain with variable strength alluvial sands, while the third test pile was founded in alluvial clays, silts and sands. Predictions of load capacity, resistance distribution and settlements were made for the piles using traditional static design methods and also using the Pile Driving Analyser results. These predictions are compared with the static test loading measurements. At one of the sites, generally close agreement was shown between the predicted and measured values, while significant differences occurred at the other site where piles were founded into the weakly cemented limestone.

1 INTRODUCTION

The Road Construction Authority of Victoria has in recent years conducted a large number of instrumented dynamic tests on driven piles using a Pile Driving Analyser. Recently a programme of dynamic testing has been undertaken at two bridge sites to confirm requirements for driven pile foundations. Static testing has also been undertaken to provide a correlation with the dynamic test results.

The first site was at Barwon River in the City of Geelong, where a 520 metre long bridge is under construction. Eight of the piers are each supported on 24 prestressed concrete piles driven to found into limestone materials. A test piling programme, comprising i) dynamic testing during the installation and restriking of nine piles followed by ii) static loading to failure of three of the piles was undertaken prior to the construction contract. During construction the Pile Driving Analyser was used to determine the required pile driving criterion. The static test piles were restruck during the contract to assess pile set-ups. These restrikes enable a more comprehensive comparison to be made between the static and dynamic test results. Results for two of the three static test piles are presented.

At the second test site located at the Broken River bridges on the Hume Freeway near Benalla, instrumented dynamic testing was used as part of the foundation investigation to determine toe levels. One pile was subjected to a static loading test to failure. For the seven span twin bridges, a total of 300 piles comprise the abutment and pier foundations.

This paper examines the predictions made during the design and initial dynamic pile testing activities for these projects then compares the results with the performance in static load tests and subsequent long term dynamic testing.

2 FOUNDATION CONDITIONS

2.1 Barwon River Bridge

The foundation conditions at the two test sites consist of about 12 metres of mainly alluvial sands overlying Tertiary Age limestones known as the Waurn Ponds Formation. The alluvial soils generally consist of very loose to loose fine sands for the upper 5 to 6 metres with moderately dense to dense gravelly, fine to medium sands below these depths down to the limestone level.

The Waurn Ponds limestone, of about 10 metres thickness at the sites, comprises mainly silty fine to medium sized particles. The limestone has generally low strength, being mainly weakly cemented. At both test sites, the limestone is moderately dense in consistency down to the pile toe levels.

Geotechnical investigations undertaken at the sites to determine the foundation stratification and strength included electric friction-cone penetration tests located about 1.5 metres from the test sites and boreholes with Standard Penetration Tests conducted usually at 1.5 metre depth intervals in the alluvial and limestone materials. The penetration tests were conducted with a Road Construction Fugro type tip. A 20 tonne reaction was used for the tests to enable penetration within the cemented limestones. Cone penetration test results, showing the Standard Penetration Test 'N' values for the adjacent boreholes, are given in Figure 1.

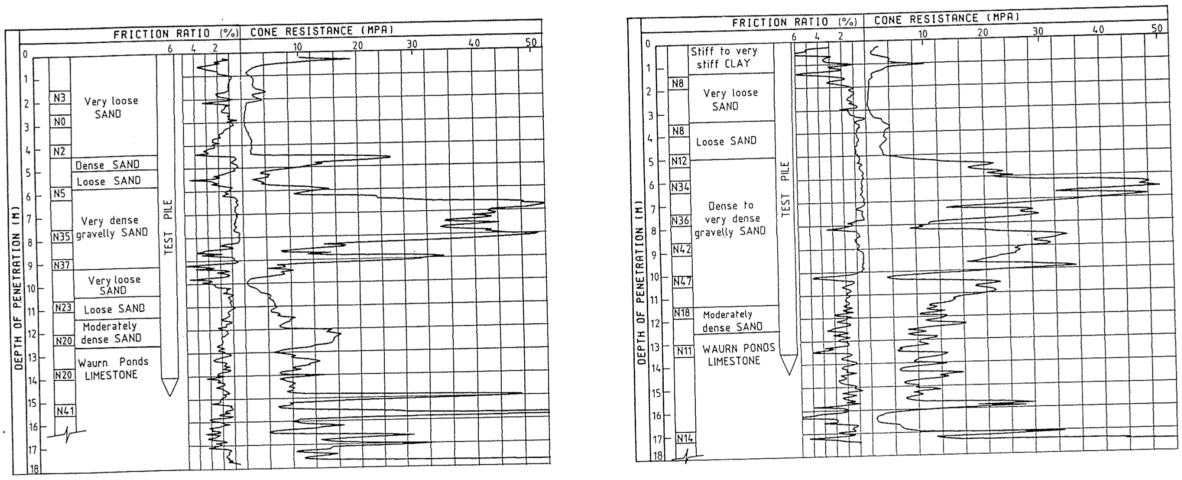
2.2 Broken River Bridges

At the Benalla site, the test pile was founded in alluvial deposits of clay, silt and fine to coarse sand being part of the Quaternary Age Coonambidgal and Shepparton Formations. The stratification of sedimentary deposits within the bridge site was found to be variable. To determine the foundation conditions relevant to the test pile a borehole and electric friction-cone penetration test were performed within 1.5 metres of the test pile. Test results are given in Figure 2.

3 STATIC TEST PILE DETAILS AND INSTALLATIONS

3.1 Barwon River Bridge

Static test loading was undertaken on vertical prestressed concrete piles located within separate pier pile groups. The piles tested were 450 mm



(a) Test No 1

(b) Test No 2

Figure 1 Subsurface profiles at Barwon River tests

square, with 102 mm diameter steel tubes of 4.9 mm wall thickness cast centrally to accommodate strain gauge instrumentation.

The test piles were installed to depths ranging from 14.45 to 14.75 metres. Both piles were founded about 2 metres into the limestone formation. The piles were driven with a 11.8 tonne drop hammer.

3.2 Broken River Bridges

At the Benalla site, the test pile was a 355 mm square reinforced concrete pile 12.0 metres in length. It was installed using a 2.5 tonne drop hammer with a 0.5 tonne helmet and timber packing.

The pile was driven to a level representing the approximate required ultimate capacity for the site, and where the available 200 tonne jacking system was capable of producing a failure.

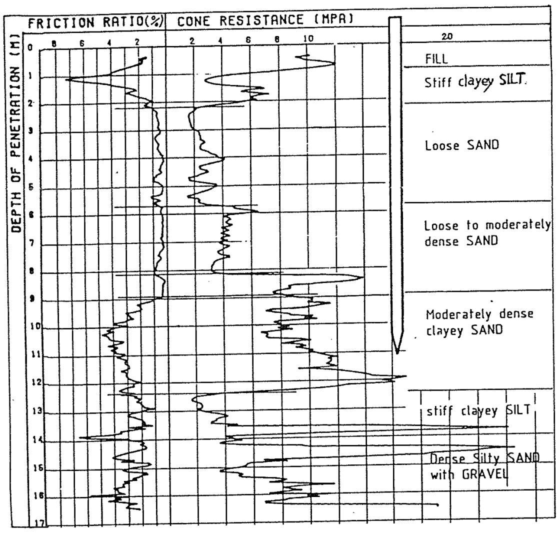


Figure 2 Broken River bridge penetration test

4 STATIC TEST PILE ARRANGEMENTS AND INSTRUMENTATION

4.1 Barwon River Bridge

For both tests the load was applied to the head of the pile through a reaction beam spanning between two vertical ground anchors. The anchors consisted of 12 prestressing tendons grouted into 200 mm diameter holes, located within the limestone horizon. The fixed anchor lengths ranged from 6 to 9 metres.

For each test the applied load was measured using strain gauged load cells. These were located between the top of the reaction beam and the jacks. Vertical movements of the pile heads were monitored using a precision level.

Strain gauges were installed at seven levels in each test pile to determine the load induced strains down the piles. Two gauges were installed at each level. Weldable strain gauges were used, mounted onto fabricated steel sections and grouted into the central steel duct of the piles using a low shrinkage cement grout.

4.2 Broken River Bridges

At the Broken River test site the load was applied to the pile head using a reaction beam held down by two 27 metre long steel anchor piles located 2.4 metres from the test pile.

The load was measured using a strain gauged load cell placed between the jack and the pile head. Movements of the test pile head and the anchor piles were monitored using a precision level located 5 metres from the test pile, with a backsight benchmark founded in a concrete block a further 5 metres away.

5 STATIC TEST LOADING CYCLES

For both sites, maintained load tests with three loading cycles were used.

For the Barwon River Bridge tests, loading was undertaken in 300 kN increments. The first and third cycles involved loading to the design load of 1500 kN, with the second loading cycle being taken to failure. At 1500 kN, 3000 kN and near the failure load, between 3 to 5 hour hold periods were used to assess creep settlements. At the 300 kN loading stages, one hour hold periods were used.

For the Broken River Bridges test, the first cycle was loading to 400 kN (the design load) in 100 kN increments and maintaining the load until movements had become negligible, and then unloading in 200 kN increments.

The second cycle consisted of loading in 200 kN increments until the ultimate capacity was reached, and maintaining the load for on hour after each increment.

The third cycle was increasing the load in 250 kN increments, maintaining for 10 minutes at each increment and reaching the previously determined ultimate capacity and maintaining for an hour.

6 ULTIMATE PILE CAPACITY PREDICTION METHODS

6.1 Predictions Prior to Pile Installations

Prior to the installation of the test piles at the sites, several design methods were used to predict the load distributions and ultimate capacities based on the penetration and borehole test results.

6.1.1 Meyerhofs Method Using SPT Results

The method of Meyerhof (1956, 1976) for non-cohesive materials was used to make predictions of ultimate pile capacities at the sites, using the borehole Standard Penetration Test (SPT) results. For this method, the ultimate pile capacity (P_{ult}) is given by:

$$P_{ult} = \frac{40\ N\ D\ A_b}{d_b} + 2\ \bar{N}\ A_s \qquad -(1)$$

With

$$\frac{40\ N\ D}{d_b} < 400\ N \qquad -(2)$$

where N = SPT value near pile base
D = depth of pile penetration
d_b = diameter of pile base
A_b = pile base area
$\bar{N}$ = average SPT count along pile shaft
A_s = pile shaft area

6.1.2 Schmertmanns Method Using Penetration Test Results

The Road Construction Authority has developed a computer program, PENPILE, for the calculation of load capacity of driven piles using the method developed by L. Nottingham and recommended by Schmertmann (1975). Details of the method are contained in a report by Nottingham and Schmertmann (1975) and the manual by Schmertmann (1978).

The predictions presented in this paper have been made assuming sands when the friction ratio values are less than 2% and cohesive materials for friction ratio values greater than 2%. For determinations of the ultimate pile toe capacities, a weighted value was determined using the cone resistance values over the intervals of 8 pile diameters above the pile toe and between 0.7 and 3.75 pile diameters below the pile toe as described by Sanglerat (1972).

6.1.3 SAA Piling Code Methods

The methods presented in Appendix A of the SAA Piling Code, AS2159-1978, were used to determine the ultimate capacities for the piles at the three sites. For non-cohesive soils the Code design method is based on the work of Vesic (1967) and Kerisel (1961). The soil consistencies inferred from the penetration test results were used for these ultimate capacity predictions. The borehole test results usually showed close agreement with the consistencies as determined from the cone penetration tests.

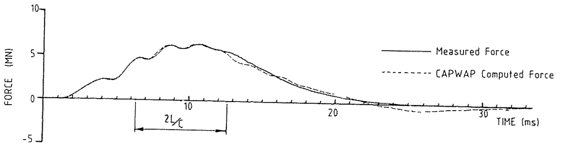
6.2 Predictions Based on Pile Driving Measurements

During driving and restriking of the test piles, ultimate capacity predictions were made by the instrumented dynamic testing technique using the Pile Driving Analyser (PDA). The traditional dynamic formula, Hiley, using measured sets and temporary compressions was also used for the Broken River test.

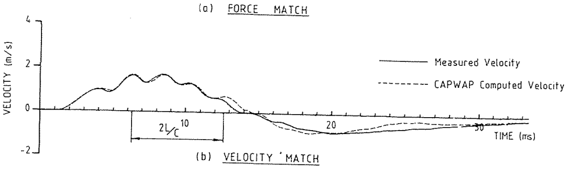
6.2.1 Instrumented Dynamic Testing

During driving and restriking the test piles were instrumented with accelerometers and strain transducers and the RCA's Pile Driving Analyser (PDA) used to record the force and velocity traces produced for selected hammer impacts. These traces were then used to carry out a wave equation analysis with the programme known as CAPWAP (Case Pile Wave Analysis Programme, after Goble et al, 1980).

Basically the CAPWAP analysis uses one of the measured field traces, say velocity, as a boundary condition in a wave equation analysis to compute the other trace (force). The computed force trace is compared to the field measured force trace, and if they do not match, the operator interacts with the program altering parameters such as ultimate resistance, resistance distribution along the pile shaft, dynamic damping factors and quakes. This process is continued until the match between the computed and measured force traces cannot be further improved. Typical CAPWAP matches are shown in Figure 3.



(a) FORCE MATCH



(b) VELOCITY MATCH

Figure 3 CAPWAP matches for Test No. 2 at Barwon River

At the Barwon River bridge, CAPWAP capacity was determined from restriking data obtained prior to, and also after the static loading tests. The restrikes prior to the static loading tests were carried out about 6 days after initial driving, while the later restrikes were undertaken about 540 days after driving, during the main construction contract. The static loading tests were commenced about 100 days after initial driving.

For the Broken River bridges site, the CAPWAP prediction was based on data obtained from restriking the test pile 47 days after the initial driving and 45 days before the static loading test. This was done to allow an equal time for the dissipation of pore pressures between the three events. No restriking was carried out after the static loading test.

6.2.2 Hiley Formula

Traditionally, in the Road Construction Authority, the allowable capacity of driven piles has been assessed by the Hiley formula using a factor of safety of 4.

It is possible to improve the accuracy of the Hiley formula by eliminating one of the major sources of error in its application. With the Hiley formula, the energy transfer to the pile is evaluated from the assumptions regarding hammer efficiency and coefficients of restitution. With dynamic testing the PDA measures the energy transfer to the pile and this value can be used to re-evaluate the Hiley formula prediction as follows:

$$P_{ult} = \frac{\text{Measured Energy}}{S + 1/2\ (C2 + C3)}$$

where S = measured set

C2 + C3 = temporary compression of pile and ground

7 PILE SETTLEMENT PREDICTION METHODS

7.1 Use of DEFPIG program

Predictions of pile head settlements were made using the Deformation Analysis of Pile Groups (DEFPIG) program written by Poulos (1980). The program considers a pile to be elastic with a constant axial stiffness. The soil is assumed to be a linear elastic medium, but allows for slippage at the pile-soil interface when the limiting soil resistance is reached.

For the settlement predictions presented, Youngs Modulus (E_s) values for the soils as given in Table A3.1 of AS2159-1978 were used. For the Barwon River bridge Test Pile No. 2, the limiting side resistance (f_s) and end bearing resistance (f_b) from the static test measurements were used, while for the Broken River bridges test pile, the limiting resistance values were derived using the design methods given in Appendix A of the SAA Piling Code. For very loose sands, a Youngs Modulus value of 25 MPa was adopted.

7.2 CAPWAP Predictions

Predictions of pile head settlements were also derived from the CAPWAP analyses. For these analyses, an elasto-plastic soil model is assumed and the load settlement relationship is derived using the determined resistance distribution and the assumed soil quakes.

8 STATIC TEST LOAD RESULTS

8.1 Barwon River Bridge

The ultimate loads achieved for Test Pile Nos 1 and 2 were 3900 kN and 4200 kN respectively.

Both tests showed essentially linear-elastic response up to the design load of 1500 kN. Beyond 1500 kN, creep settlements became significant and the pile responses were noticeably non-linear, indicating the occurrance of some shaft slip. The load-settlement response for Test Pile No. 2 is shown in Figure 4.

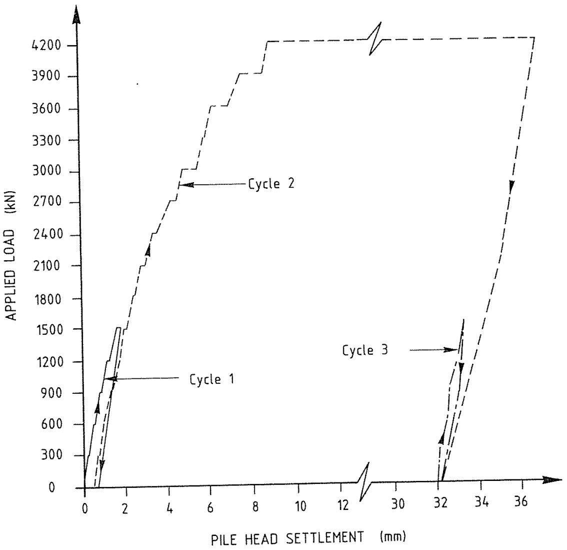


Figure 4 Load - settlement response for Test No. 2 at Barwon River

For the three cycles of loading, measured pile head settlements at 1500 kN ranged from 2.3 to 3.1 mm for Test Pile No. 1 and from 1.2 to 1.9 mm for Test Pile No. 2. Settlements decreased for each successive load cycle, indicating an increase in the overall pile-soil stiffness with load cycling.

For both tests, maximum creep rates of 0.1 mm per hour were measured during loading up to the design load of 1500 kN. Beyond 1500 kN, creep settlement rates increased approximately linearly to be about 0.5 mm per hour at 3000 kN. Negligible long term creep is expected for the pier foundations, we the maximum sustained loading (ie. dead loads) per pile will be about 800 kN.

8.2 Broken River Bridge

The measured load settlement response is shown in Figure 7. The curve demonstrates typical behaviour in that it is essentially linear to 500 kN and then time dependant effects become more pronounced leading to a plateau at the ultimate capacity (1270 kN).

The total pile head movement after completion of the third cycle was 60 mm, whilst at the design load of 400 kN, it was 4.5 mm.

9 PREDICTION VERSUS PERFORMANCE

9.1 Ultimate Capacities and Resistance Distributions

Values of ultimate capacities predicted for the three test piles, using the methods outlined in Section 6, are summarised in Table 1. Figure 5 shows the resistance distribution comparisons for the two Barwon River bridge test piles. For the static load test distributions shown in Figure 5, the average strain measurements for the two strain gauges at each level was adopted. For the measured distributions, no adjustments have been made for the initial residual stresses in the piles following installations.

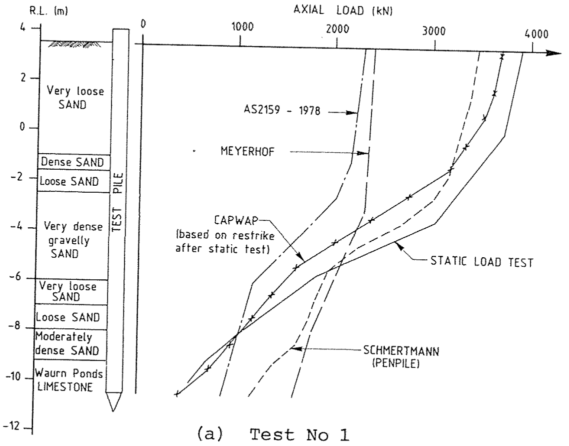
Significant variations between the measured and predicted values are noted for the Barwon River bridge tests. Use of Meyerhofs method and the

TABLE I

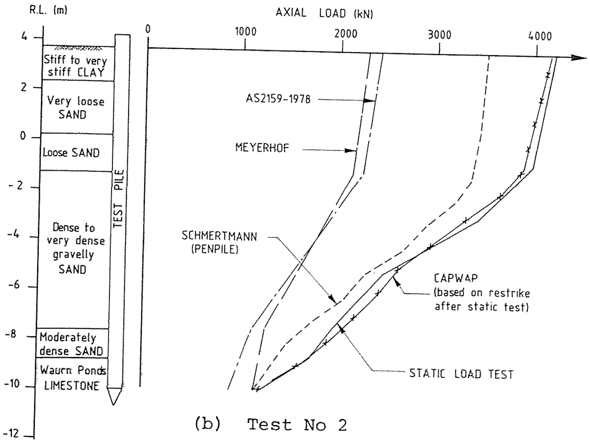
ULTIMATE PILE CAPACITY COMPARISONS

		BARWON RIVER BRIDGE				BROKEN RIVER BRIDGES	
		TEST PILE NO. 1		TEST PILE NO. 2			
BASIS OF DATA	METHOD	ULTIMATE CAPACITY (kN)	% AT TOE	ULTIMATE CAPACITY (kN)	% AT TOE	ULTIMATE CAPACITY (kN)	% AT TOE
Foundation Investigation	Meyerhof - SPT	2410	67	2315	49	1226	78
	Schmertmann - CPT	3445	34	3505	33	1310	64
	SAA Code AS 2159-1978	2300	39	2420	37	1232	63
During Pile Restriking	Hiley Formula					882	-
	Adjusted Hiley Formula					1410	-
	CAPWAP - Prior to Static Load Test	2438 (6)	18	3290 (7)	14	1200 (47)	79
	CAPWAP - After Static Load Test	3700 (543)	12	4118 (535)	29		
Static Loading Test	Brinch Hansen's 80% Criterion (Brinch Hansen, 1963)	3900 (97)	14	4200 (111)	32	1270 (92)	-

NOTE: Numbers in brackets indicate days after pile installations



(a) Test No 1



(b) Test No 2

Figure 5 Ultimate capacity distributions - Barwon River bridge tests

AS2159-1978 Code design methods gave similar ultimate capacities and distributions, but underpredicted ultimate capacities by between 38.2 and 44.9 per cent. In contrast, Schmertmanns method underpredicted capacities by between 11.7 and 16.6 per cent. For Test Pile No. 1, Schmertmanns method predicted the toe capacity to be about twice the measured value, while for Test Pile No. 2, there was close agreement between the predicted and measured toe capacities.

CAPWAP analyses based on pile restrike tests undertaken about 3 months prior to static load testing and then about 14 months after testing at the Barwon River bridge site showed that significant set-ups occurred. For the two test piles, increases in load capacities of 25 and 52 per cent between restrikes were observed, which are unusually high for the non-cohesive foundation types present. CAPWAP analyses based on the later restrikes showed close agreement with the static test results.

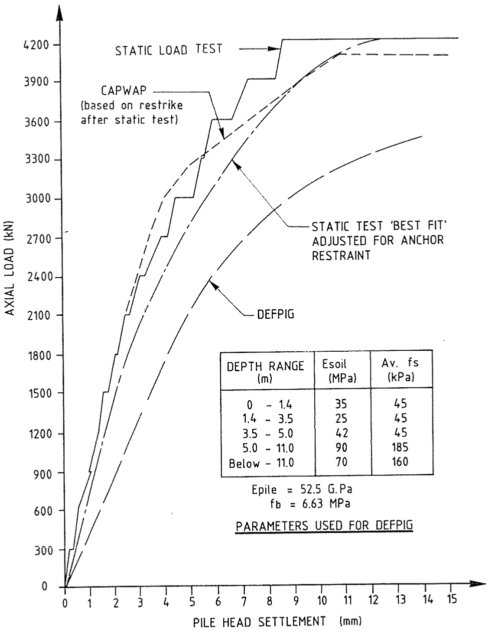
Predicted ultimate capacities for the Broken River site were generally in close agreement with the static test results, being usually within 10 per cent of the measured value. The most significant variation occurred for the Hiley formula prediction, which underestimated the capacity by about 30 per cent. Use of the adjusted Hiley formula provided a closer prediction, over estimating the measured capacity by 11 per cent.

The Broken River test pile was not instrumented with strain gauges and so the shaft resistance distribution was not measured. The percentages of load carried by the toe as predicted by the various design methods and CAPWAP are presented in Table 1. CAPWAP and the Meyerhof SPT method predicted approximately 80 per cent at the toe whilst Schmertmanns method and the SAA Code methods predicted lower, at about 65 per cent.

9.2 Pile Settlements

Comparisons of the predicted pile head settlements with measured settlements for two of the test piles are given in Figures 6 and 7. The measured settlements were adjusted for the estimated anchor restraint effects determined using the method given in Section 16.5 of Poulos and Davis (1980).

For the Barwon River Bridge Test Pile No. 2, the CAPWAP prediction showed close agreement with the

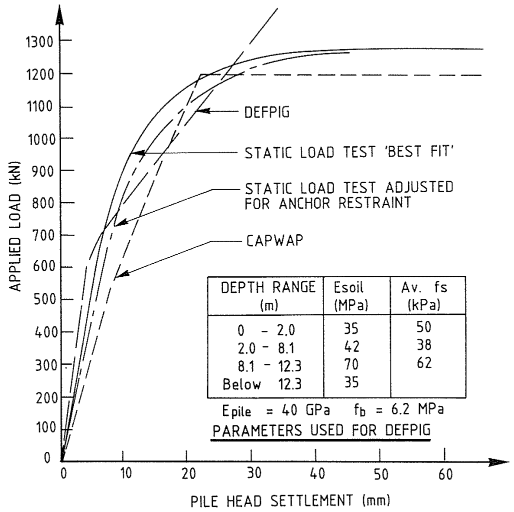


DEPTH RANGE (m)	Esoil (MPa)	Av. fs (kPa)
0 - 1.4	35	45
1.4 - 3.5	25	45
3.5 - 5.0	42	45
5.0 - 11.0	90	185
Below - 11.0	70	160

Epile = 52.5 G.Pa
fb = 6.63 MPa

PARAMETERS USED FOR DEFPIG

Figure 6 Pile settlements - Barwon River Test No. 2



DEPTH RANGE (m)	Esoil (MPa)	Av. fs (kPa)
0 - 2.0	35	50
2.0 - 8.1	42	38
8.1 - 12.3	70	62
Below 12.3	35	

Epile = 40 GPa fb = 6.2 MPa

PARAMETERS USED FOR DEFPIG

Figure 7 Pile settlements - Broken River bridge

actual measured settlement, while the DEFPIG prediction, using AS2159-1978 Code values of soil modulus, overestimated the adjusted measured settlement by about 50 per cent. The DEFPIG analysis indicated that slippage at the soil-pile interface was minor up to 1500 kN. At 2400 kN, slippage occurred for about 70 per cent of the pile shaft length. The departure from essentially linear-elastic behaviour was in close agreement with the static test measurements. The larger settlements predicted using DEFPIG show that the SAA Code values of soil modulus are conservative at this site. The smaller measured settlements may also be due in part to the effect of residual stresses in the pile following driving.

For the Broken River test, the static loading results generally fall between the CAPWAP and DEFPIG predictions of 2.4 mm from DEFPIG and 5.6 mm from CAPWAP. For agreement with the measured settlements, the DEFPIG analysis showed the backfigured soil Youngs Modulus values for the loose and moderately dense sands were about half of the typical values given in the SAA code.

10 CONCLUSIONS

(1) Predicted ultimate capacities and resistance distributions based on Schmertmanns method and CAPWAP analyses (long term) showed close agreement to the static load test measurements at both sites.

(2) Meyerhofs method using borehole SPT results and the design methods incorporated in the SAA AS2159-1978 Piling Code gave ultimate capacity predictions that were between 38 and 45 per cent lower than the measured values at the Barwon River bridge site. The main difference at this site was the underprediction of side resistance in the dense to very dense gravelly sands. In contrast, these methods gave predictions that were within 4 per cent of the measured values at the Broken River site.

(3) The Hiley formula underpredicted ultimate capacity by 30 per cent at the Broken River bridge site. Use of the adjusted Hiley formula resulted in a closer match with the measured value.

(4) Some significant variations occurred between predicted pile head settlements and measured settlements at both sites. For the Barwon River Test No. 2, DEFPIG analyses using SAA code values of soil Youngs Modulus overpredicted settlements by about 50 per cent, while CAPWAP analyses based on restriking after the pile loading test showed close agreement. For the Broken River test the measured settlement was approximately midway between the DEFPIG and CAPWAP predictions. At the design load of 400 kN, the predictions varied by up to about 50 per cent from the static load test measurements.

(5) When allowing for creep and shaft slip effects, CAPWAP analyses closely predicted the static load-settlement response for the Barwon River bridge tests. CAPWAP overpredicted pile head settlements by about 25 per cent at the 400 kN design load for the Broken River bridges site.

(6) The need to undertake restrike testing to allow for the dissipation of dynamically induced effects during driving and set-up of resistance, has been demonstrated at both sites, particularly at Barwon River. This is an important aspect when considering dynamic testing programs.

11 ACKNOWLEDGEMENTS

The authors gratefully acknowledge the efforts of engineers and technicians who have contributed in providing the information necessary for this paper, in particular Mr M Ravichandra who undertook the CAPWAP analyses. The paper is presented with the permission of of Mr I F X Stoney, Chairman and Managing Director of the Road Construction Authority of Victoria. The views expressed are those of the authors and do not necessarily reflect the views of the Road Construction Authority.

12 REFERENCES

Brinch Hansen, J. (1963). Discussion, "Hyperbolic stress - strain response. Cohesive soils", ASCE, J SMFD, Vol. 89, SM4, pp. 241-242.

Goble, G.G., Rausche, F. and Likins, G.E. (1980). The analysis of pile driving - A state of the art. Proc. Int. Sem. on the Application of Stress - Wave Theory on Piles, Stockholm.

Kerisel, J. (1961). Foundations Profondes en Milieu Sableux, Proc. 5th Int. Conf. S.M. and F.E., Vol. 2, pp. 73-83.

Meyerhof, G.G (1956). Penetration Test and bearing capacity of piles, ASCE, J. SMFD, Vol. 85, SM6, pp. 1-29.

Meyerhof, G.G. (1976). Bearing capacity and settlement of pile foundations, ASCE, J.GED, Vol 102, No. GT3, pp. 195-228.

Nottingham, L.C. and Schmertmann, J.H. (1975). An investigation of pile capacity design procedures. Final Report D629, Dept. of Civil Eng., University of Florida.

Poulos, H.G. (1980). Users Guide to Program DEFPIG, University of Sydney.

Poulos, H.G., and Davis, E.H. (1980). Pile foundation analysis and design, John Wiley and Sons, New York, NY.

Sanglerat, G. (1972). The penetrometer and soil exploration. Elsevier Pub. Co., Amsterdam.

Schmertmann, J.H. (1975). Measurement of in-situ shear strength. Proc. Conf. In-Situ Measurement of Soil Properties, ASCE Speciality Conf., Vol 2, p.57.

Vesic, A.S. (1967). A study of bearing capacity of deep foundations. Final Report, Proj. B-189, School of Civ. Eng., Georgia Inst. Tech., Georgia.