

Pile Capacities for Echuca Rail Bridge

B.C. CHANDLER

Associate & Principal Engineer Geotechnical, Maunsell & Partners Pty Ltd, Melbourne

SUMMARY Two pile types were used for V/Line's Echuca Rail Bridge over the Murray River, 550 mm cast-in-place piles and 760 mm driven steel tube piles. The selection and installation of pile types was largely constrained by the requirement not to disturb the foundations of an old wrought iron bridge located only 15 m downstream. Details of the site stratigraphy and issues which influenced the choice, design and specification of these piles are given in this paper. Comparison of the design capacity of the piles with PDA dynamic pile testing and direct load test results obtained during construction revealed inaccurate but conservative design estimates of performance and capacity for the cast-in-place piles, reasonable correlation of design estimates of capacity for the driven steel piles and good correlation of PDA/Capwap test results with direct load testing.

1 INTRODUCTION

The Echuca Rail Bridge, currently under construction, is approximately 450 m long and passes over the Murray River at Echuca 15 m upstream of an old rail/road bridge built in 1878. The old rail/road bridge which is founded on vertical 1.4 to 3.0 m diameter concrete filled cast iron cylinders in stiff clays remains in service and hence its foundations could not be disturbed by the construction of the new bridge. A 15m centre line separation of the bridges was selected with this requirement in mind. The superstructure for the new bridge comprises a continuous 3 span section over the river and 13 simply supported approach spans over the flood plain.

As the bridge location and pier positions were finalised various foundation options were studied in the preliminary design phase and appropriate site investigations were carried out in stages to establish the stratigraphy and determine the soil properties and ground water conditions.

The final bridge design involved cast-in-place pile groups for the approach piers and open ended driven steel tube piles for the main piers. Individual pile capacities and pile group designs were developed based on the soil properties determined from the site investigations and current pile design practices. To confirm the design performance of the two pile types a programme of direct load pile testing and dynamic pile testing was specified in the contract documents.

This paper compares the performance of individual piles under test loading with the predicted performance and design philosophy adopted prior to construction.

2 GROUND CONDITIONS

Extensive borehole investigations disclosed approximately 20 m of variable interbedded silty sands and clays overlying about 12 m of very stiff to hard silty clay, in turn overlying an unproven depth of dense sands with some hard clay layers.

The soils below the river alluvium are part of the regional Shepparton Formation which is known to extend to depths of at least 100 m. A generalised view of the stratigraphy relative to the two pile foundation types for the Echuca Rail Bridge are shown in Figures 1 and 2.

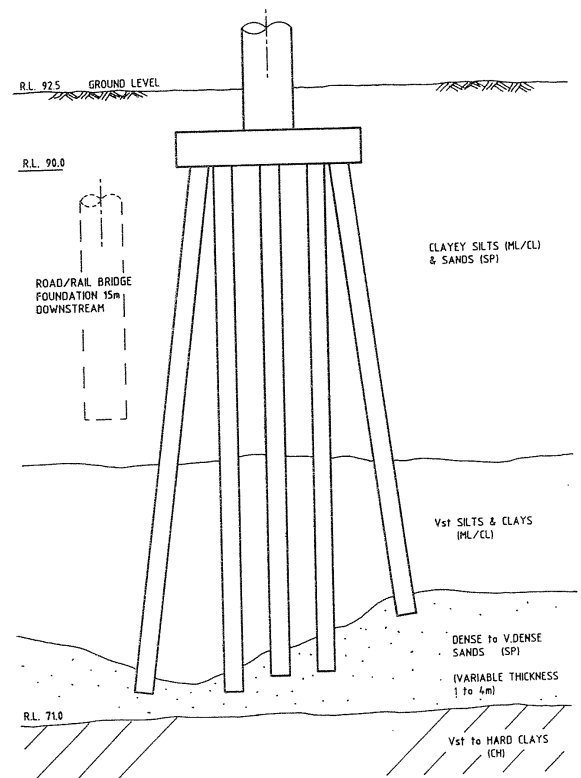


Figure 1 Cast-in-place Concrete Piles for Approach Piers

Strength and stiffness testing carried out during the site investigation indicated a significant degree of over consolidation of the clays and some natural cementation of the silts and sand layers below 15m. Confined compressive strengths of the clays and silts in most cases were over 400 KPa and some silty sand layers exhibited similar high compressive strengths. Pressuremeter testing confirmed the high strength of the soils and recorded soil stiffnesses nominally in the range of 150 to 200 MPa. The soils to depths of 15 m, comprising predominantly silts and sands, were generally of lower relative density with SPT values in the range of 10 to 20. These soils are alluvial having been deposited under the influence of the meanderings of the Murray River in more recent times.

The presence of sand layers between clayey soils created zones of sub-artesian ground water conditions depending on the fluctuating river level and hence a potential problem with uplift pressures for any deep excavations. Confined sand aquifers were identified immediately above and below the hard clays at RL 70 and RL 57 respectively, refer to Figure 2.

The concrete filled cylinders supporting the old rail/road bridge are founded between RL74 and RL84 in the stiff silts and clays and overlying alluvial soils. Refer to Figures 1 and 2. Construction reports indicate that the cylinders were hand excavated and problems with loss of ground and excess water inflows did occur when sand layers were intercepted.

3 CHOICE AND DESIGN OF PILES

3.1 Approach Piers

By adopting practical approach spans and limiting the pile working loads to 1,300 kN to allow a choice of conventional pile types, raked pile groups comprising 8 and 12 piles were required to support approach pier loads of 5,400 kN and 8,700 kN respectively. Refer to Figure 1.

Ultimate capacity estimates of a nominal 550 mm diameter pile founding on the dense sand at or above RL 72 ranged from 2,500 to 4,000 kN of which approximately 1,000 kN would be carried by pile shaft friction and the remainder in end bearing on the dense sand layer. If the piles penetrated through the sand layer, it being too thin or less dense in places, and entered the underlying clays, penetrations would have had to be increased by up to 8 m to achieve comparable pile capacities. To take advantage of the sand layer and avoid greater depth of pile penetration, which was beyond the reach of most conventional cast-in-place piling equipment, enlarged base piles were specified with a nominal base diameter of 800 mm to achieve minimum ultimate capacities of 3,000 kN.

Considerable design consideration was then given to how to identify and handle piles if they happened to be positioned in locations where the sand layer was too thin (less than 1 m) to generate the required 2,000 kN base capacity. The upper horizon of the underlying clays, i.e. at RL 71, was known to be slightly lower in consistency and hence could significantly reduce the base capacity of the piles. It was concluded that bigger enlarged bases and/or additional piles would be required to cater for these conditions.

The Australian Piling Code and published information on the design of Frankipiles in clays [1 & 2] were used together with the known soils test data to determine the above pile capacities.

The final design and specification was prepared adopting 550 mm diameter enlarged based cast-in-place piles. The piles were required to have ultimate capacities of 3,000 kN.

3.2 Main Span Piers

Larger spans were required over the river channel section to cater for river traffic and to complement the layout of the old bridge. The main span piers were 74 m apart and each had to be designed to carry a maximum vertical load of approximately 40,000 kN. In addition, both piers were adjacent to piers of the old bridge, the centreline of which was only 15 m down stream. Refer to Figure 2.

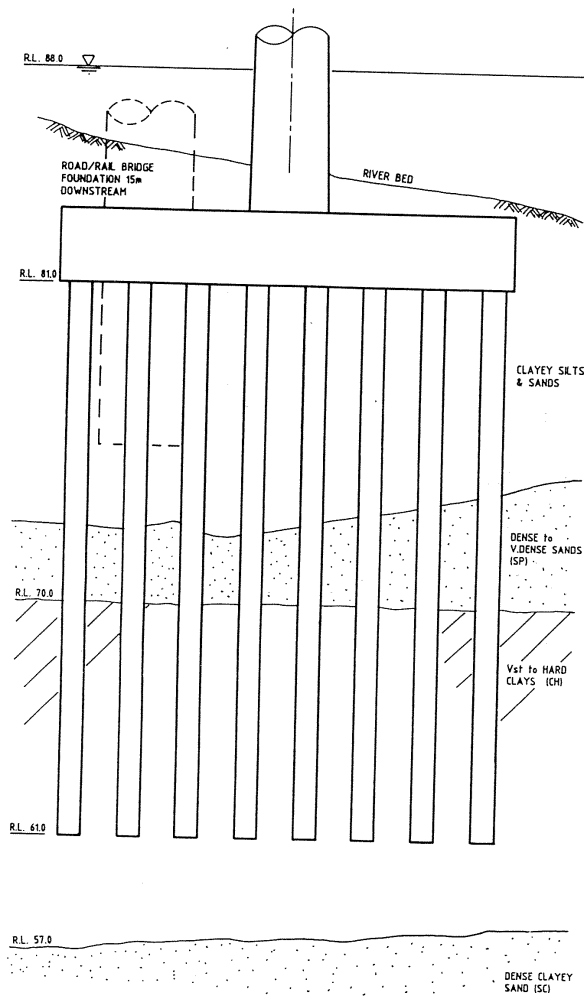


Figure 2 Steel Tube Piles for Main Piers

Bored piles up to 2 m diameter and founded in the hard clays were considered in the preliminary design stage. However, concerns with stability in the sand layers, the cost and time delays in proving pile capacities and the effects on the old bridge foundations led to the adoption of smaller piles in a larger group. Extensive design was carried out on group pile behaviour including DEFPIG analysis [3] of the final arrangement which comprised 30 No 762 mm diameter steel tube piles per pier.

The maximum loads applied to the outer piles in the group were determined for the various design load cases and an ultimate pile capacity of 4,600 kN was adopted to give an appropriate factor of safety for each load case. Based largely on guidelines in the Australian Piling Code [1] for estimating pile capacities using shear strengths and relative densities of soils, it was determined that the piles had to penetrate to about RL 61 to achieve the required ultimate capacity, i.e. well into the hard clays.

To minimise ground disturbance and hence any effects on the adjacent bridge piers the piles were specified to be driven open ended and to be installed to at least RL 70 without any significant soil plugging. Staged "mucking out" was specified to ensure that this was achieved. Concern for steel corrosion in the sand aquifers led to the requirement that the steel tubes be concrete filled down to a level of at least RL 69 after driving.

The pile toe level of RL 61 was specified as an anticipated level subject to direct load and PDA testing proving that the minimum ultimate capacity of 4,600 kN had been achieved at that level.

3.3 Alternative Piles for Approach Piers

An alternative to the enlarged based cast-in-place piles was submitted by the Contractor. His proposal was to use the GKN Keller system of driven cast-in-place piles. The system comprises top driving a 550 mm diameter steel tube with a slightly over-sized sacrificial steel end plate to a toe level sufficient to achieve capacity. When the tube is at the desired level the reinforcing cage and wet concrete are placed inside the tube and then the tube is progressively tapped with the driving hammer and extracted. More concrete is added as necessary to ensure that the tube at no stage is lifted above the top of the concrete.

Further calculations of capacity based on the soil test data and shaft adhesion coefficients for closed ended driven piles [1] were carried out. Consistent with the finding in the design stage assessment of cast-in-place piles, the alternative pile type would have to found on a substantial layer of dense sand above RL 71 to achieve the required capacity. The Contractor was advised that the system would be acceptable provided assurances were given that, in areas where the sand layer was too thin or of insufficient density to provide sufficient pile capacity, more piles would be driven or greater pile penetration achieved. These assurances were given and to substantiate his confidence the Contractor proposed a series of test drives adjacent to selected piers.

4 TEST RESULTS

The specification required a limited number of direct load tests supplemented by PDA dynamic pile testing [4] to prove the load carrying capacity of the piles.

Cast-in-place Piles

Trial driving of the cast-in-place pile installation tube was also monitored with the PDA. Although testing a temporary steel casing with oversize shoe could not be directly related to the capacity of the finished concrete pile, it did provide added confidence that the required capacity was achievable without excessive penetration. The tube testing indicated that even

with an oversize shoe a significant proportion of soil friction was generated along the shaft. In most cases the trial driving proved out at least two thirds of the required ultimate load with toe levels at or above the dense sand layer.

About 15% of the cast-in-place piles were PDA tested and two direct load tests were carried out on vertical piles. In many cases it was not possible to mobilise the piles when PDA testing and hence it was not possible to determine the full ultimate capacity. The testing did show that the shaft adhesion was well in excess of the 1,000 kN ultimate load determined in the design stage. In most cases shaft adhesion exceeded 2,000 kN. Base resistances were usually less than 500 kN owing to the inability to mobilise the piles and hence prove their full base capacity.

A record of the testing of pile 702 is shown in Figure 3. PDA testing of this pile was carried out before the load test. An ultimate capacity of 3,400 kN was recorded with a set of 4 to 5 mm per blow. CAPWAP analysis [4] determined that about 3,000 kN of this was generated in shaft adhesion, most of which was uniformly distributed over the lower half of the pile. Less than 400 kN was being carried in end bearing. With sets of 4 to 5 mm this result was considered to be close to if not equal to the ultimate capacity. The load test was taken up to 3,000 kN and sustained for 6 hours. Comparison of the load test and PDA/Capwap results is shown in Figure 3. Although the load test was not taken to failure the load deflection curve does indicate that the PDA estimate of ultimate capacity at 3,400 kN is reasonable.

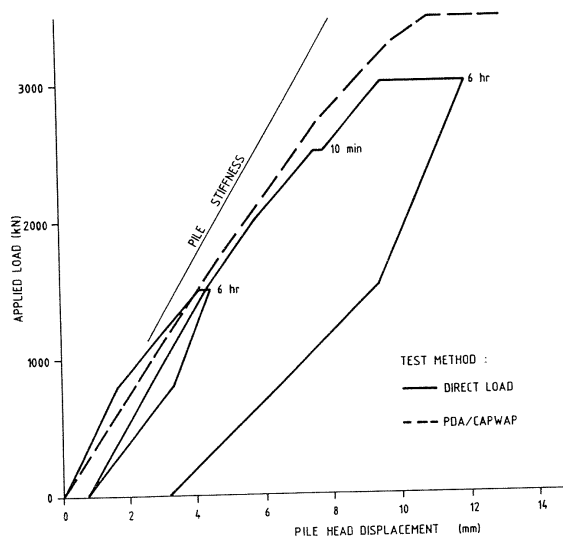


Figure 3 Concrete Pile No. 702 Load Test Performance

The recorded load deflection stiffness of the pile head was similar for both test methods, about 0.33 MN/mm. In comparison, the pile stiffness over its full length is about 0.45 MN/mm and therefore, with the centre of pile support at about the lower 2/3 point, about half of the elastic deflection was in the pile and half in the soil.

It is of interest to compare this result with the estimate of ultimate capacity based on the soil test data. Using the adhesion coefficients, F and α , given in Appendix A of the Australian

Piling Code [1] the ultimate shaft capacity of pile 702 is in the range of 100 - 160 tonne depending on the ground water level. These figures were arrived at using the F values for driven piles, Table A1.1.2 and the confined undrained shear strengths of the clays as equal to C_u in Figure A1.1. The calculated end bearing capacities for the pile toe were 250 kN and 2,000 kN in the stiff clayey silts and in the medium dense to dense sands respectively. The range in capacity is large and this was reflected by increased driving resistance when installing the temporary casing but not to the same order of magnitude as these figures would suggest.

Steel Tube Piles

The steel tube piles for the main piers were driven with a K45 diesel hammer in the bottom of sheet pile cofferdams after excavation to the underside of the pile caps. Selected piles were monitored with the PDA during the last few meters of driving and again by restrike testing to determine set-up. Typical set-up recorded one day after first driving gave a 10 - 15% increase in capacity. One pile was load tested to further confirm pile capacity. All testing was carried out after the piles were "mucked out" to RL69 but before they were concrete filled.

A record of the testing of pile 514 is shown in Figure 4. Again PDA testing was carried out before the load test on this pile. The PDA testing by restrike with sets of 0.5 to 1.0 mm per blow estimated an ultimate capacity of 4800 kN. Owing to the sets being less than 3 mm per blow this estimate of capacity was considered to be a lower bound. CAPWAP analysis confirmed the capacity estimate and showed a build up in skin friction from excavation level to about RL73 and a relatively uniform skin friction distribution below that level with little or no toe resistance. The latter is indicative of the soil inside the pile not being plugged.

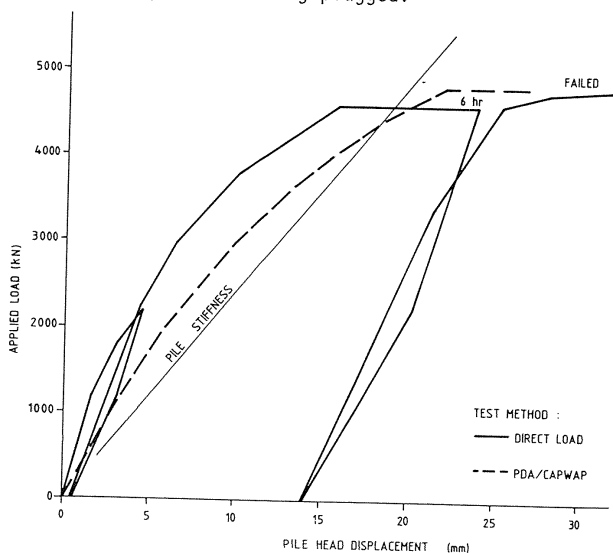


Figure 4 Steel Pile No. 514 Load Test Performance

After loading to 4600 kN for 6 hours the pile sustained a permanent head displacement of 14 mm indicating a stable but significant creep rate at this load level. Upon reloading the pile failed at a load of between 4800 kN and 4900 kN. Hence

the PDA/Capwap estimate of 4800 kN was not a conservative lower bound but was more indicative of the behaviour of a predominantly friction pile.

The recorded load deflection stiffness of the pile head was again similar for both test methods, 0.35 MN/mm. In comparison, the pile stiffness over its full length was only 0.25 MN/mm likewise confirming that the load was predominantly supported in skin friction with a centre of support at working load levels at approximately the mid point of the pile.

A comparison of the test results for pile 514 with estimates of load capacity based on the piling code guidelines shows the latter to be slightly conservative. The ultimate capacity estimated using the guidelines and soil test data was initially calculated to be in the range of 3450 to 3900 kN assuming the shear strength of the hard clays was 200 kPa on average. If a more realistic shear strength of 250 kPa and an adhesion coefficient α of 0.3 were adopted for the hard clays, i.e. an adhesion of 75 kPa, the ultimate capacity estimate would be increased to a maximum of 4100 kN. This was still less than the test load result of 4800 kN. To cope with the difference between these figures and the required ultimate load prior to construction, the Specification was written to provide scope for driving the piles beyond the anticipated toe levels. The test load result of 4,800 kN, obtained with the pile at the anticipated toe level, indicated an ultimate shaft adhesion in the hard clays of at least 80 kPa. This result confirmed the preconstruction judgement that the design, based on the piling code guidelines, was conservative and slightly greater capacities would be achieved.

5 CONCLUSIONS

A number of project specific conclusions as well as more general conclusions can be drawn from this comparison of design versus performance of cast-in-place and driven steel tube piles.

(i) It is clear that the design significantly underestimated the ultimate shaft capacity that can be developed by cast-in-place piles on this site. About 15% of the cast-in-place piles were PDA tested and all showed shaft capacities at least 50% higher than those estimated in the design. A direct load test on one pile confirmed the PDA capacity and indicated an ultimate shaft friction of approximately twice the design estimate. The reason for this high shaft friction is believed to be due to the roughness of the shaft and the expansive effects of the tapping and hammering installation technique. Adhesion coefficients F and α as recommended in the Australian piling code do not appear to be appropriate for this type of pile installation.

(ii) Despite the need to extend pile tops and provide a drop hammer to carry out PDA testing on cast-in-place piles, the PDA proved to be a practical and economic means of testing cast-in-place piles. In being able to test raked piles and to test more piles than could be achieved by direct load techniques, the PDA provided greater quality assurance to the piling.

- (iii) The application of adhesion coefficients, F and c in estimating the capacity of the steel tube piles gave slightly conservative results. The differences were not significant enough to suggest that higher coefficients should have been adopted on this site. However, it is significant to note that confined undrained shear strengths were used to determine the pile capacities and hence the design conservatism may have been higher if unconfined soil test data had been adopted.
- (iv) Direct load testing of a steel pile to failure indicated the accuracy of the PDA/Capwap estimate of ultimate capacity by restrike testing to be within a few percent. The testing revealed that for purely friction piles sets of less than 1 mm/hammer blow can be sufficient to mobilize the ultimate shaft friction and hence the ultimate capacity of the pile.
- (v) The need to provide a specification which recognises the shortcomings of estimating pile capacities based on soil strength data and hence gives scope for varying pile penetration during construction in a fair and equitable manner was essential on this project.

6 ACKNOWLEDGEMENTS

The author wishes to thank V/Line and Maunsell & Partners Pty. Ltd. for their permission to publish this paper.

7 REFERENCES

- [1] STANDARDS ASSOCIATION OF AUSTRALIA (1978). SAA Piling Code. AS2159-1978.
- [2] MCANALLY P.A. & DOUGLAS D.J. (1984). The Design of Frankipiles in Clays, Fourth Aust. N.Z. Conf. on Geomechanics, Perth 14 - 18 May '84 p. 402.
- [3] UNIVERSITY OF SYDNEY, Dept. of Civil Engineering (1980). Computer program Deformation Analyses of Pile Groups, Code DEFPIG. Written by H.G. Poulos.
- [4] CHAPMAN G.A. & CHANDLER B.C. (1986). Dynamic Pile Analysis Theory & Techniques, Leighton Mudajaya JV Seminar on Teluk Intan Hospital, Malaysia.