Performance of timber piles in interbedded sands and clays

Performance des pieux en brois dans les sables et argiles

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SYNOPSIS Potentially aggressive groundwater conditions precluded use of conventional concrete cast-in-situ piles to support twin haul road bridges. Treated hardwood timber piles were adopted, and a programme of test piling carried out. The testing indicated conventional static analysis for piles in interbedded sands and clays would have been conservative. A Class A prediction of pile performance, which ignored the influence of layering, satisfactorily predicted the observed behaviour.

INTRODUCTION

Development of an open cut coal mine at Ulan in western New South Wales, Australia, was commenced in 1981, providing a considerable expansion to an existing underground operation. At the southern boundary of the proposed mine, the Permian Age coal measure rocks have been eroded and the area infilled by Tertiary Age deposits, consisting of interbedded sands and clays. Prior to development of the mine, it was necessary to divert the Goulburn River to the south of its existing course through the proposed mine area, via a 7km long diversion channel up to 12 metres deep through these Tertiary deposits.

Twin haul road bridges, to accommodate 170 tonne pay load haul trucks, were constructed over this diversion channel. The 40m long adjacent bridges were of two spans, with design abutment and pier loads of 10MN and 20MN respectively. The bridge decks are continuous and simply supported with horizontal loads transmitted directly into abutment headwalls. Differential settlement between abutments and pier of up to 25mm was acceptable.

A driven pile foundation system was considered to be appropriate for the structure, and given the high loads to be accommodated, maximizing the allowable pile loads was important in order to reduce the group size. Prediction of pile capacity with confidence was made difficult due to the interlayered soil strata, and a programme of test piling was agreed upon. Driven cast-in-situ piles were favoured at first, however, a routine check on the potential aggressiveness of the groundwater indicated severe corrosion potential towards both steel and concrete. As a result, treated hardwood timber piles were adopted. After test loading four piles, a confident prediction of performance was obtained, with the resulting foundation system being considerably less expensive than the original concrete pile proposal.

INVESTIGATION PROGRAMME

An extensive drilling, sampling and testing programme had been completed to allow design of the river diversion channel, thus allowing a good geological model of anticipated conditions to be established. Three additional boreholes were drilled at the actual bridge site, as shown on Fig. 1. A typical borehole log is presented as Fig. 2, together with a summary of test data obtained from both the bridges and diversion investigations. Three separate aquifers were encountered, as shown. The use of mud drilling below 8m depth precluded measurement of the piezometric level in each of the lower sand layers, but upon completion of the drilling the mud was "killed" and the level shown recorded after bailing the borehole.

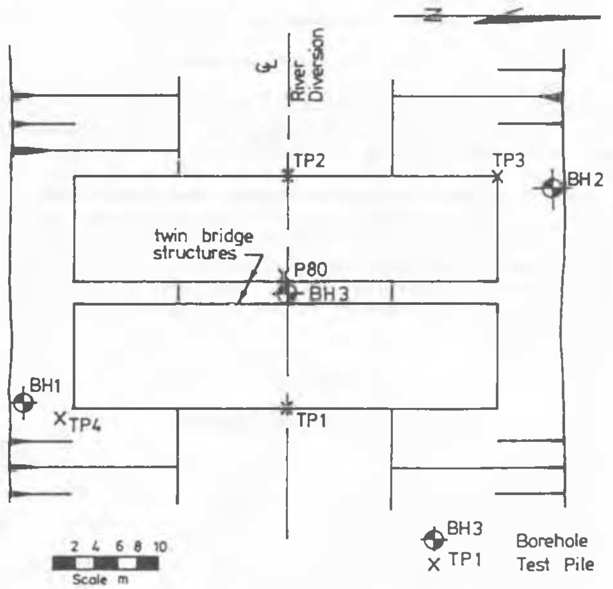

Fig. 1 Site Plan

Reasonable continuity of sub-horizontal layers corresponding to the two distinct material types shown in Fig. 2 was found to exist. As shown by the Standard Penetration Test "N" values, the sand varied considerably in density both between and within the layers. The sands within the aquifers are generally fine to coarse grained and contain only a trace of fines. The high plasticity clays varied in undrained shear strength from layer to layer, and were highly fissured. Some concern existed as to possible softening of these clays after the stress relief due to the diversion channel excavation, especially given the sub-artesian pressures which exist within the aquifers confined by these clays.

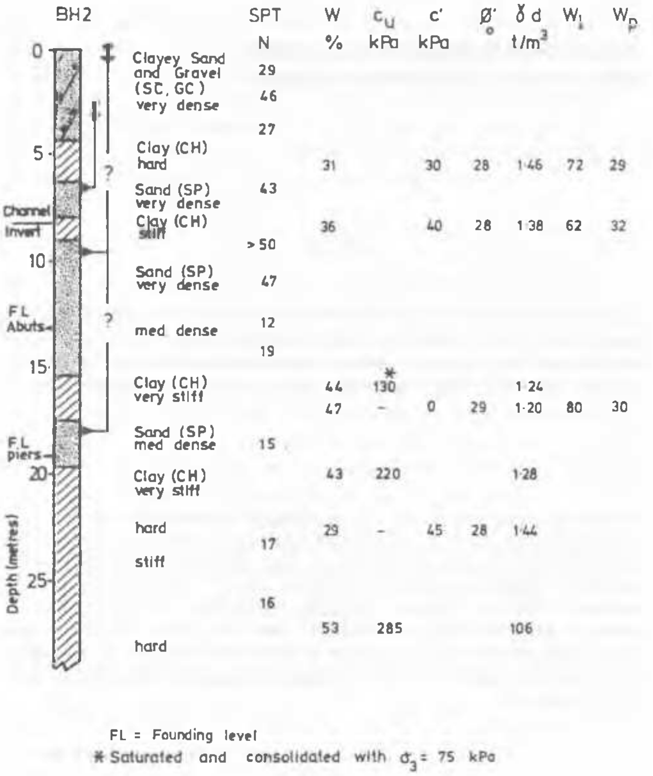

Fig. 2 Borehole Data

A sample of the groundwater was taken from the top aquifer prior to adoption of mud drilling, and of the groundwater recovered after bailing the borehole at completion of the drilling. These samples were submitted for analysis for potential aggressiveness to concrete and steel. The testing indicated a high level of aggressiveness, and it was therefore resolved to carry out additional testing on further more rigorously obtained samples. The testing on these samples, summarised below, confirmed the potential aggressiveness with low pH values and unacceptable levels of free CO_2.

TABLE I

Results of Chemical Analyses

Depth m	pH	Cl' mg/ℓ	SO_4" mg/ℓ	Free CO_2 mg/ℓ
7	5.25	1650	320	83
14	4.65	350	30	176
19	3.35	570	70	195

FOUNDATION DESIGN

Initial recommendations were for 500mm dia driven cast-in-situ piles, approximately 9 metres long. Estimation of allowable pile capacity required a rationalization of the influence of the two major soil types and of their layering. It is suggested by Meyerhof (1976) that a pile tip situated within 10 diameters of a weaker stratum is influenced by that stratum (Fig. 3). To maximize pile capacity at the site it was advantageous to found within the medium dense to dense sand layers, thus making use of the higher capacity offered by granular materials. The founding layers considered for the abutment and pier piles are only 6m and 2m thick respectively, corresponding to 12 and 4 pile diameters (if no base-enlargement). Hence using the Meyerhof criterion the adjacent clay layers dominate. For example, the allowable base resistance in the clay after excavation was considered to be 475kPa, compared to 1600kPa in medium dense sand at 9 metres depth (water table at surface). Applying the Meyerhof criterion, piles founding mid-depth on the upper founding layer may be proportioned for about 1100kPa base resistance, and in the lower layer negligible influence of the sand is allowed. Clearly in the upper layer overdriving would result in a reduction in allowable base capacity.

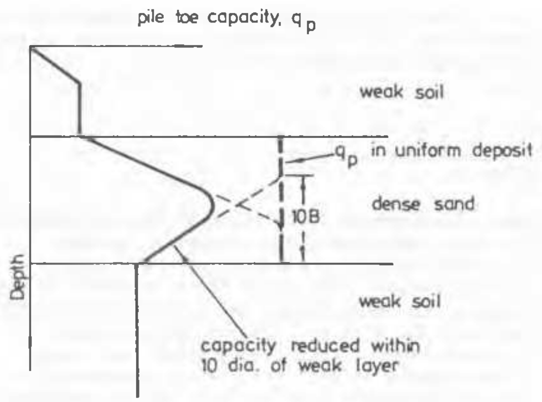

Fig. 3 Influence of weak layer (after Meyerhof, 1976)

Using this approach an allowable pile capacity of 650kN was decided upon. It was recognised this was well below the structural capacity of a 500 dia pile, and it was recommended that test piling be considered with a view to improving the allowable pile capacity.

Construction methods which would allow use of concrete cast-in-situ piles in these aggressive conditions were considered, but due to uncertainty and cost were rejected. As a result creosote treated hardwood timber piles were subsequently accepted as the most appropriate piling system for the site.

The axial structural capacity of 280mm toe Australian hardwood piles corresponding to Strength Group S4 or better (SAA Timber Engineering Code, 1975) is 780kN. Clearly such piles had sufficient structural integrity but due to their smaller diameter their calculated static capacity was considerably less, despite the increased shaft resistance due to the pile taper (8mm per metre). The adoption of test piling to allow pile performance to be optimized was therefore considered essential.

TEST PILING

Four test piles were driven at the site, at the locations shown on Fig. 1. Due to industrial trouble it became necessary to drive the piles using a Delmag D12 diesel hammer, with a ram mass of 1.25 tonnes and a rated energy of 30.5kNm. It was initially proposed to use a drop hammer. Preboring was carried out through the upper cemented materials at the abutment pile locations.

Driving of TP1 was stopped at anticipated Contract level

(Fig. 4) at which stage its allowable capacity using the Hiley Formula and a F.O.S. = 3 was 530kN. It was also apparent that its capacity was still increasing. An additional pile was therefore pitched and driven (Pile P80) to assess driving resistance below Contract level. It was found (Fig. 4) that driving resistance improved up to 1.8m below Contract level, before decreasing significantly as the influence of the clay layer began to dominate. TP1 was then redriven, a further 1.5 metres, with a corresponding increase in dynamic capacity up to 610kN.

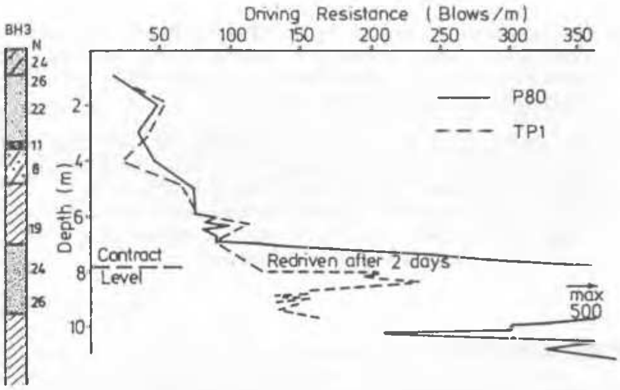

Fig. 4 Driving resistance - TP1 and P80

The remaining three piles were driven to a set corresponding to an allowable pile capacity of at least 650kN, the desired pile capacity.

The influence of the two soil types on driving resistance (and therefore capacity) is apparent in Fig. 4, particularly for P80 which was driven within a few metres of Bore 3. Only marginal increase in capacity results from driving through the clay, but once the underlying sand is penetrated the resistance increases dramatically. Heavy driving continues until the sand is penetrated when the resistance suddenly decreases.

It is interesting to note that the sand layer is about 9 pile toe diameters thick, yet its influence is almost immediate and continues to within a few centimetres of the clay layer. The subsequent increase again in resistance within the clay layer is judged to be an increase in shaft resistance corresponding to the taper effect within the sand layer.

Load testing was carried out using four 13m long 600 dia grout injected piles as reaction. Loading was carried out in increments, with time-displacement records kept at each increment. An unloading cycle was carried out at 650kN. TP1 failed by gross (40mm) deflection at 1175kN after behaving essentially elastically at earlier increments. The remaining three piles exhibited less dramatic behaviour, accepting between 1300kN and 1675kN for permanent pile head deflections of 4mm to 7mm. At the maximum applied loads, however, onset of signifncant settlement had occurred. Figs. 5 and 6 present load-deflection behaviour of TP1 and TP3. Table II presents the results of the test loading.

As noted, driving records were maintained to allow the dynamic capacity of the piles to be determined, therefore providing a basis for acceptance of the subsequent prototype piles. The dynamic capacity was determined using both the Hiley and Janbu formulae (refer Poulos & Davis, 1980). As shown in Table III below, the Janbu formula predicted the measured ultimate capacity within 12%, compared to 55% for Hiley. The Janbu formula has the added practical advantage of not requiring a graphical record of temporary compression and set to be obtained.

TABLE II

Results of Test Piling

Pile No	Maximum Load kN	Deflection at 650kN mm	Residual Deflection after test mm
TP1	1175	4.6	37.8
TP2	1300	3.9	4.0
TP3	1675	3.5	6.9
TP4	1450	3.6	5.5

TABLE III

Predicted Dynamic Capacity of Piles

Pile No	Ultimate Load (kN) Load Test	Hiley	Janbu
TP1	1180	1830	1240
TP2	1450 est	1980	1515
TP3	1675	2020	1475
TP4	1450	1960	1355

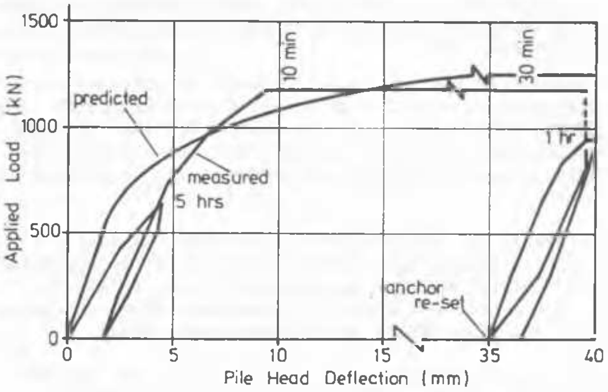

Fig. 5 Predicted and Actual Performance - TP1

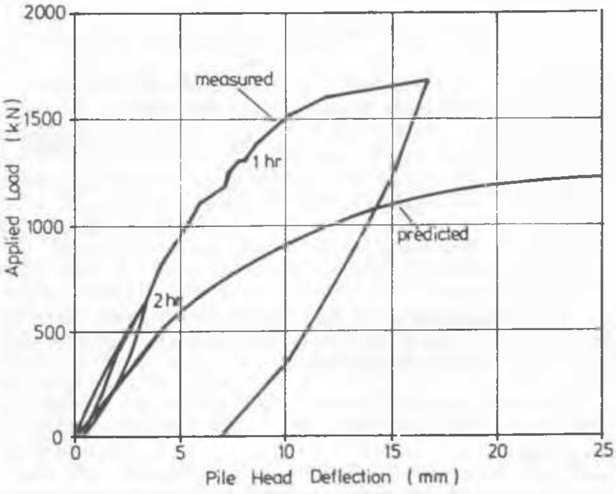

Fig. 6 Predicted and Actual Performance - TP3

PREDICTION OF PILE PERFORMANCE

Some months after completion of this project the data were used in a Class A prediction exercise. Various engineers, including the second author of this paper, were provided with the site investigation data and pile installation details. Predictions had to be made of the static pile load performance of the four test piles. The following is a brief description of the approach adopted by the second author of this paper.

* Formulate a simplified geotechnical model at each location assuming that clayey sands would behave as sands, and that localised lower density or weaker horizons would be bridged by the pile and hence would have little effect.
* Determine ultimate side shear values in sands and clays. The method presented by Nordlund (1965) was used for the sand layers but with a check against the data presented by Tomlinson (1977 - Fig. 4.19). Adhesion values for the clay horizons were taken from Tomlinson (Fig. 4.7).
* Determine ultimate end bearing component assuming 9 c_u for clay and the end bearing values of Berezantsev (1961) for sand.
* Calculate the elastic settlement of the pile using the method of Mattes (1972) as given in Poulos and Davis (1980), assuming that the effective short term modulus would be that of a very dense sand (E=200MPa) and that the load-settlement curve would be elastic up to 40% of ultimate load.
* Assume that ultimate capacity would be obtained at a displacement of 10% of the nominal pile diameter. The predicted load-displacement curves were then produced by linking the elastic portion with the ultimate load using curves obtained from previous pile load tests in sand.

In carrying out these analyses no account was taken of the interaction between the sand and clay layers as suggested by Meyerhof (1976). It was considered that, unless the base of the pile was within 1½ diameters of an underlying soft layer, there would be no significant effect.

Table IV gives the predicted ultimate loads and the displacements at 650kN. These can be compared with the measures values given in Table II. Figures 5 and 6 compare the predicted load-displacement curves for TP1 and TP3 with the measured data.

TABLE IV

Pile No	Predicted Ultimate Load kN	Predicted Deflections at 650kN	
		mm	% of measured
TP1	1250	2.0	44
TP2	1600	2.0	51
TP3	1200	5.5	166
TP4	1500	4.4	122

The predictions were generally quite satisfactory with the worst being TP3 where the predicted ultimate capacity was about 70% of the measured value.

CONCLUSIONS

1. The value of a comprehensive test piling programme was clearly demonstrated for this project, with adopted design pile capacities considerably exceeding those which would have been adopted from static analysis. Considerable cost savings over the original foundation scheme resulted, highlighting the advantages over concrete and steel which can be offered by timber piles if the structural capacity of timber is utilized. The structure has been in service for some two years and is performing satisfactorily.
2. Based on the observed performance of the test piles, particularly during driving, it appears that the Meyerhof criteria regarding the influence of adjacent weak strata is conservative.
3. The approach presented by Nordlund (1965) for calculating side shear values for tapered piles in sand provided a satisfactory basis for predicting the behaviour of these timber piles.
4. The adhesion factors given by Tomlinson, and adopted in the prediction, are higher than presented in many texts and codes. Adoption of such factors for prediction purposes appears to have been satisfactory, although in static analysis without supporting test pile data caution would be required.

ACKNOWLEDGEMENTS

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