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Foundation Aspects of Mount Henry Bridge, Perth

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SUMMARY This paper presents information on the geotechnical investigations undertaken, the soil profile determined and the foundation systems reviewed for the Mt Henry Bridge over the Canning River in Perth, Western Australia. Site investigations, which included pressuremeter testing, SPT and vane shear tests, in addition to triaxial and other laboratory testing, established the soil parameters for the foundation design. Bored piles and various types of driven piles, approximately 40 m long, are discussed and details given of an adopted composite pile founded in hard sandy silty clay. Full scale pile load tests are also reported.

1. INTRODUCTION

The Mt Henry Bridge over the Canning River, forms part of the southern extension of the Kwinana Freeway and is situated approximately ten kilometres south of the city of Perth. The structure is a nine span prestressed concrete continuous box girder 660 m between abutments and provides dual three lane carriageways with nominal shoulders and a central concrete median barrier. Pedestrian and cycleway facilities are provided at the bridge soffit level (Figure 1).

Extensive site investigations were carried out for this bridge, which enabled assessment of a range of possible piling systems. The main systems studied were bored and driven piles founded in a hard sandy silty clay (siltstone) located approximately 26 m below water level.

2. SITE INVESTIGATION

The investigation of this site was conducted in several stages. The initial investigation was undertaken to gain a broad appreciation of the soil profile together with sufficient data to enable an evaluation of the various types of foundations considered for the site. Subsequent investigations were undertaken to provide data when it was required for specific purposes, and to complete an understanding of the site as it related to the foundations finally selected.

Preliminary borings showed that the abutments were sited on approximately 20 m of interbedded sands and clays overlying black clayey sand and silty clay which merges into a hard black sandy silty clay known geologically as the King's Park Formation. This is referred to as siltstone throughout this paper. The mid-section under the river consists of approximately 14 m of very soft clay overlying the sandy and silty clays.

On the basis of that information, a more detailed investigation was undertaken to provide data to enable an assessment of potential settlements of a shallow foundation at each abutment and of probable pile lengths where deep foundations would be required. Field SPT and laboratory consolidation data were obtained for the settlement predictions. SPT data and unconfined compression (UCC) test results were used to determine the upper level of the siltstone. Menard pressuremeter tests were used to determine the shaft resistance offered to piling by the siltstone.

A bored pile alternative to driven piles was considered in the design phase and included in the tender documents. During assessment of tenders, this alternative became questionable due to the concern that the sides of holes drilled into the siltstone might close-in or spall. To better evaluate that possibility, further insitu parameters of the siltstone were obtained using a self boring pressuremeter.

2.1 Field Work

Cased boreholes were drilled at each pier and abutment location. SPT tests were conducted at regular intervals in the overlying sediments and continuously when the clayey sand/silty clay overlying the siltstone was encountered. In adjacent holes, samples were obtained for laboratory testing in thin wall sampling tubes in the soft sediments and from diamond drill cores in the siltstone. On the clay samples, consolidation testing as well as a limited programme of triaxial testing was undertaken. On the siltstone core samples, UCC tests were conducted.

The Menard and self boring pressuremeter tests were carried out on the Mt Pleasant shore. These provided siltstone data including undrained shear strength, stress strain relationship and values of the insitu lateral earth pressure coefficient (K_o). This latter parameter was important in assessing the feasibility of bored piles as, based on an analysis by Davis (1979), with K_o values greater than 1.1 yielding would occur in siltstone excavations.

In one of the earlier investigations of the site some limited vane shear testing was conducted in the upper clays approximately 40 m west of the centreline of pier 7.

2.2 Results

The strata encountered in the investigation are shown schematically on Figure 1 in relation to the profile of the bridge and are described in greater detail on Table I which presents a brief summary of some of the properties determined from the field and laboratory testing.

The soil types encountered on site have been

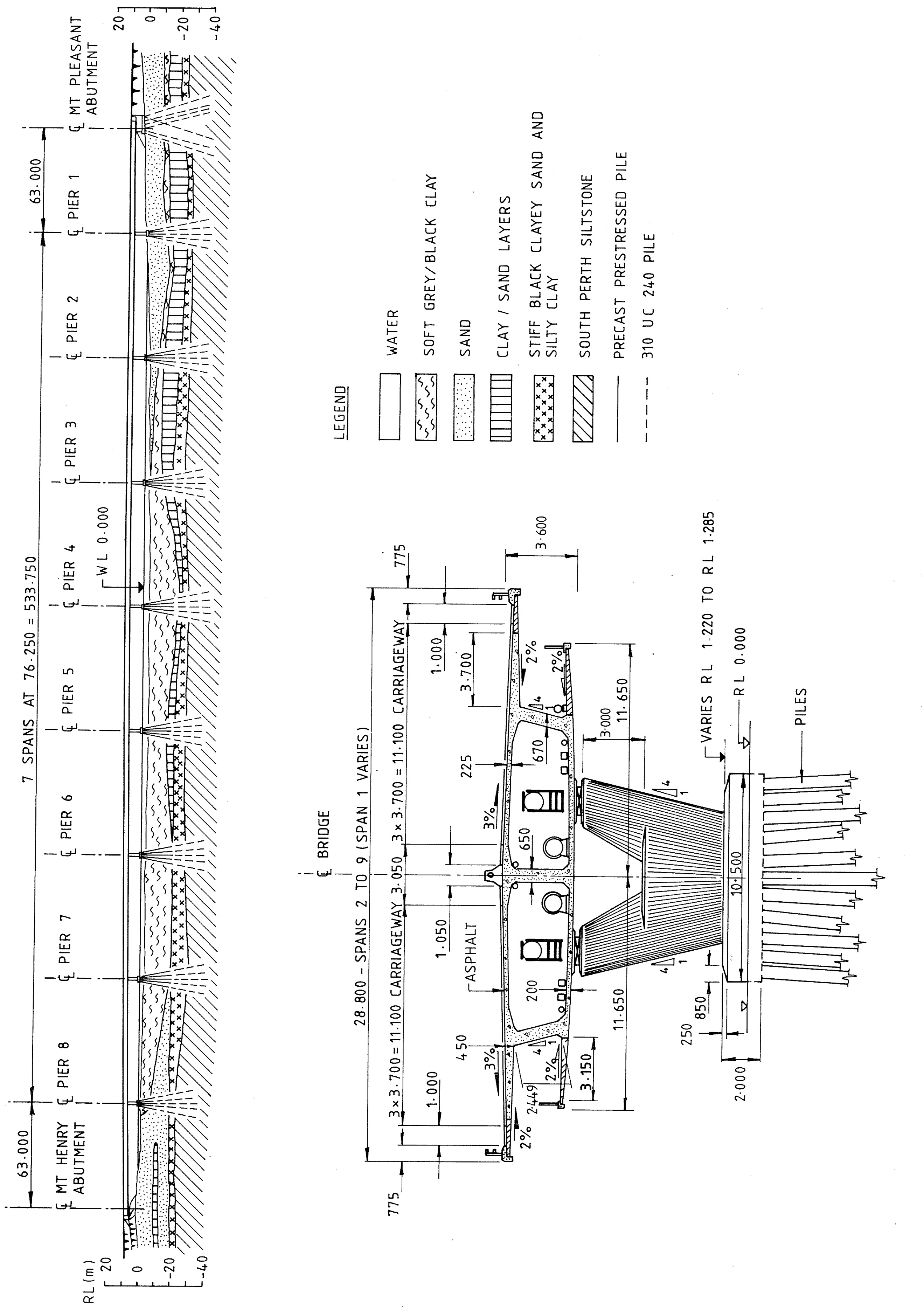

Figure 1 General details

TABLE I

STRATA DESCRIPTION AND PROPERTIES

Stratum Description	Consistency	Unified System Classification	Average Stratum Thickness (m)	SPT (Blows/ 300 mm)	Miscellaneous Properties	Remarks
Coarse to medium fine sand — Mt Henry abutment	Firm	SP	20	20-35	$\phi = 35^o$	Lower 3 m compact with SPT $\simeq$ 45
Coarse to medium fine sand — Mt Pleasant abutment	Reasonably firm	SP	10	15-30	$\phi = 30^o$	Upper 3 m loose with SPT $\simeq$ 5
Soft grey/black clay	Very soft	CH	14	1	PI = 150-50 LL = 210-80 LS = 25-15	Range of values generally correspond to top-bottom of stratum
Clay/sand layers — Mt Henry abutment	Soft	SC/CH	3	10	PI = 59-65 LL = 82-91 LS = 18-20	Variability of properties seemingly unrelated to depth
Clay/sand layers — Mt Pleasant abutment	Firm to compact	SC/CL/CH	8	10-20	PI = 40-92 LL = 57-130 LS = 20-31	
Medium to coarse black clayey sand and silty clay	Reasonably compact and stiff	CL/CH	6	30-70	PI = 21-33 LL = 41-53 LS = 7-10	This layer appears to be weathered siltstone
Siltstone	Very compact and stiff	CH	> 20	> 80	ϕ = 25-27^o c_u = 500-700 kPa PI = 27 LL = 51 LS = 12 Passing % / Sieve (µm): 99 / 300 95 / 150 50 / 75 Void ratio = 0.64 UCC=800-1400 kPa with average = 1045 kPa and 3.4% strain at failure	Tests indicated material is highly impermeable. Largest particles retained on 600 µm sieve.

classified into five basic groups as follows:-

(a) Coarse to medium/fine sand
(b) Soft grey/black clay
(c) Clay/sand layers
(d) Stiff medium to coarse black clayey sand and silty clay
(e) Very compact and hard black sandy silty clay (siltstone)

Considerable information was obtained from testing concentrated in the area of the Mt Pleasant abutment for a determination of pile capacity and consequent pile test analysis. An appreciation of the site can be obtained from a review of the data in that area and this presentation concentrates on that data. The only other results included are from the vane shear tests which were conducted in the soft grey/black clay in the vicinity of pier 7 which is towards the Mt Henry abutment.

The results obtained from boreholes located between the Mt Pleasant abutment and pier 1 are presented on Figures 2 and 3. For clarity and convenience the results in the siltstone are presented on Figure 3 and for the softer overlying material on Figure 2. At the Mt Pleasant shore, the upper surface of the siltstone was estimated to be at RL-28 m and Figures 2 and 3 present data from RL 0 to -26 m and RL-26 to -42 m respectively. This division, while not corresponding exactly with the assessed siltstone interface, is chosen to suit the data available. Availability of data was influenced by the difficulty in obtaining competent samples and hence

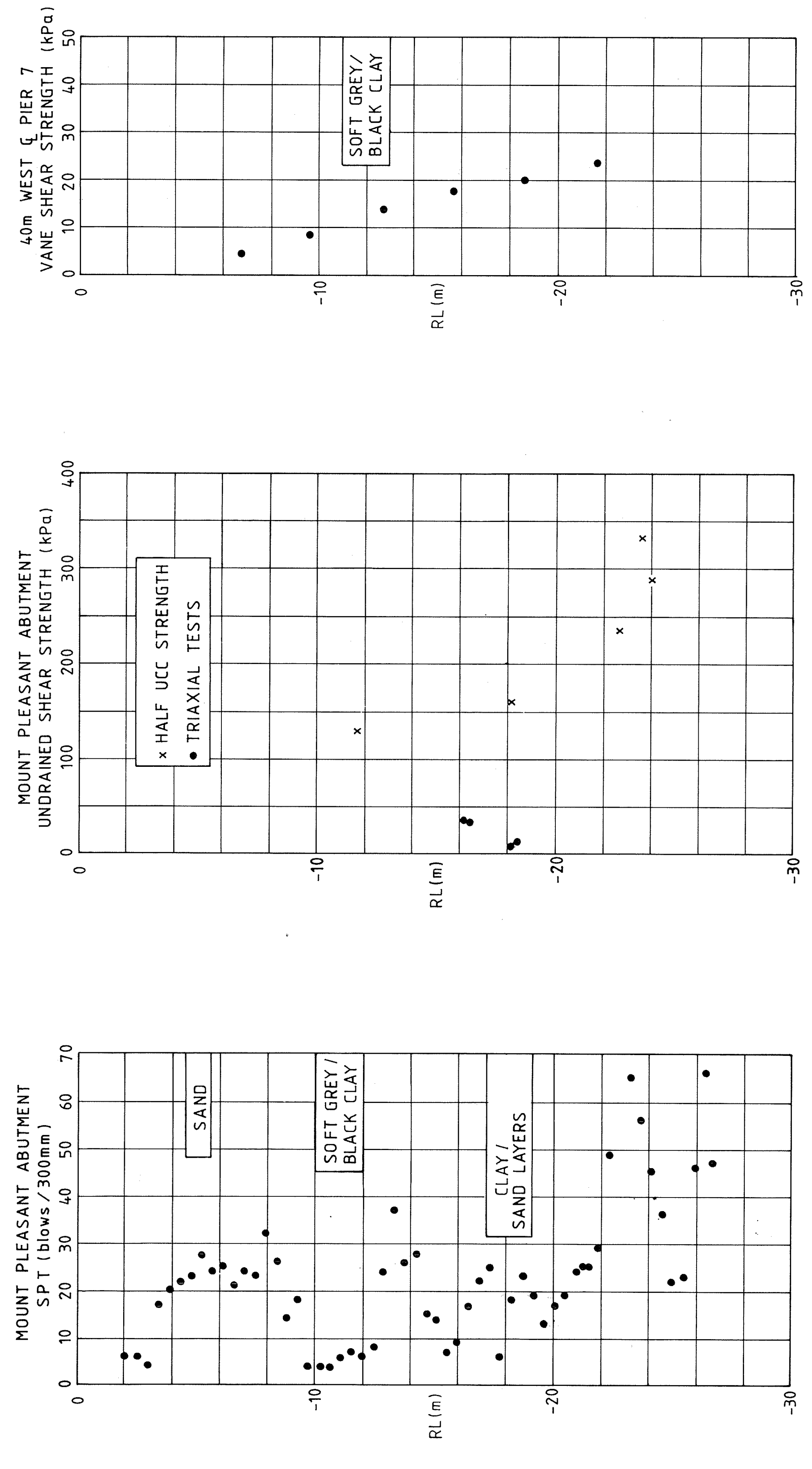

Figure 2 Soil parameters in material overlying siltstone

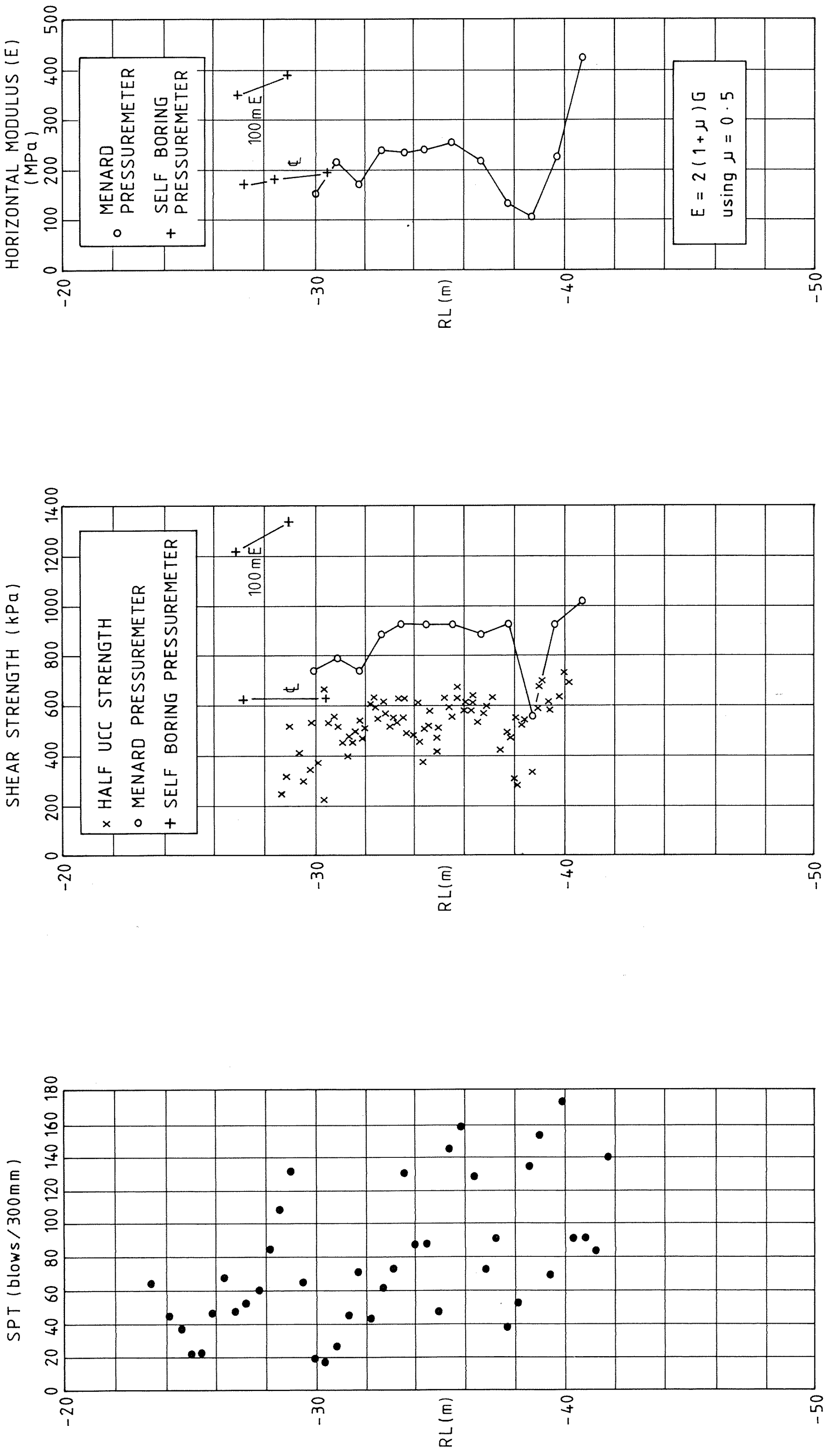

Figure 3 Siltstone parameters at Mt Pleasant abutment

undertaking meaningful laboratory tests in the upper softer material layers.

Figure 2 presents the SPT profile and associated shear strength data. The consolidated undrained triaxial test values of undrained cohesion (c_u) and undrained friction angle (ϕ_u) have been converted to an undrained shear strength value (τ_u) using the Mohr Coulomb relationship $\tau_u = c_u + p \tan\phi$ and an overburden pressure (p) equal to the geostatic stress at the appropriate depth. On the other hand the UCC results have been converted to a shear strength value simply by dividing by 2. Also on Figure 2 are the vane test results. As previously explained these were conducted in the very soft clay in the river bed and which only appears in the vicinity of RL-9.5 to -12.5 m in the Mt Pleasant abutment profile.

Figure 3 presents the SPT profile and the shear strength values estimated from UCC results and from the pressuremeter testing for the siltstone. It also presents values of horizontal elastic modulus (E) estimated from the pressuremeter testing. The self boring pressuremeter analysis (Jewell, 1980) actually provides values for shear modulus (G) which have been converted here to elastic modulus for comparison with the Menard pressuremeter results (Golder Associates, 1977) by assuming a value of Poisson's ratio (μ) = 0.5 for application to the quick undrained pressuremeter tests.

The two separate sets of values for shear strength and elastic modulus attributed to the self boring pressuremeter were from separate boreholes. The very much stiffer results were obtained 100 m east of the Mt Pleasant abutment where the siltstone interface was encountered at an RL 4 m higher and proved to be much more competent than at comparable depths in the other borehole.

2.3 Discussion

The difficulty in delineating the upper surface of the siltstone can be observed from results in Figure 3. This was first defined as the level at which the SPT blowcount initially exceeded 80 even though in most boreholes considerably lower values were obtained at various intervals throughout the siltstone. An average value of UCC strength of 800 kPa was also used to differentiate between the siltstone and the overlying clayey sand and silty clay stratum.

The SPT blowcounts in the siltstone below RL-28 m average 88 with a standard deviation of 43. The UCC results over the same stratum average 1045 kPa with a standard deviation of 224 kPa and as can be seen on Figure 3, are much more consistent that the SPT tests. These UCC results have also shown up the variability of the layer of siltstone at around RL-39 m which appears in the SPT and Menard pressuremeter data.

Shear strengths derived from UCC tests are generally expected to be low according to Lambe and Whitman (1969) whereas pressuremeter results are often high as outlined in Gibson and Anderson (1961). This would suggest that the actual shear strength of the siltstone should lie between the two sets of values on the appropriate plot on Figure 3. A comparison between those two sets of data provided an average value of shear strength from the UCC equivalent to 61% of the Menard pressuremeter shear strength at the same depth.

The major objective of the self boring pressuremeter work was an evaluation of K_o in order to test the analysis of Professor Davis. The five tests undertaken were in the upper level of the siltstone, and a K_o value of around 2.5 was obtained in each test. This indicated that the material was heavily overconsolidated and that substantial lateral movements and possibly spalling into open excavations would have occurred had bored piles been chosen for the bridge foundations.

It had been hoped that the self boring pressuremeter results would enable an assessment of the full stress strain relationship for the siltstone. The pressuremeter (developed for softer material) has a cell pressure limit of 2.6 MPa which was only sufficient to produce a radial deformation of less than 2% in four of the five tests. However, one test deformed to 5% strain and showed that the siltstone is a strain softening material with a peak stress at approximately 2% strain. The shear strength derived from these tests is presented on Figure 3 and shows that the results from the borehole on the bridge centreline are of the same order as those derived from the Menard pressuremeter. The results from the other borehole 100 m east of the bridge are much higher as previously noted.

3. FOUNDATION CONCEPT

It was desirable from a structural point of view to carry all longitudinal loads at the Mt Pleasant abutment. Accommodation of these loads by spread footings would have been a problem as would the long term settlements in the clay layers from the embankment and bridge loadings. Piling beyond the consolidating strata was therefore necessary. Although drag-down forces could develop on these piles, magnitudes were assessed to be of minor significance for the pile types considered.

The Mt Henry abutment, being sited predominantly on firm sand with SPT values ranging from 20 to 35, presented the opportunity to use a spread footing foundation. Allowable bearing pressures could be satisfied with a footing area the size of the abutment chamber and in this case, estimated residual settlements could be accommodated within the structural design.

All piers were located above soft consolidating materials leaving little choice but to adopt piling for these foundations. Except for pier 8, the only suitable founding material was the siltstone, located at an average depth of 26 m below water level. The possibility of founding pier 8 piles in the 15 m layer of sand was investigated. However, there are limited pile types that could develop sufficient support in the sand layer and it was considered inappropriate to attempt a different piling system for this one pier.

4. REVIEW AND SELECTION OF PILE TYPES

Points considered during review of the various pile types included:

(a) Design unknowns and the time available to clarify these.
(b) Size of pile cap required.
(c) Settlement behaviour.
(d) The ability to match ultimate soil and structural pile capacities.
(e) Feasibility in regard to installation methods.
(f) Construction risks.
(g) Durability and likely corrosion problems.
(h) Costs.

Consultation with the construction industry indicated that assessment of some of these points depended to a large degree on a Contractor's particular equipment and expertise. It was therefore decided to detail for tender, piling systems that were considered suitable and competitive, and to also include information in the contract documents to allow submission of alternatives.

The following comments summarise the proposals and points considered and the decisions made in determining the adopted piling systems.

4.1 Mt Pleasant Abutment Piling

With an abutment area of approximately 17 m x 11 m, it was preferable to choose a piling system with sufficient number of piles to allow concentration at zones of greatest load and a distribution such as to minimise bending and shear in the abutment footing.

In view of the necessity to penetrate reasonably firm layers to reach the siltstone, a small displacement driven pile became attractive. Favourable experience with steel piling in the past, led to the adoption of a 310 UC 240 steel section.

This foundation involved 43 piles of 43 m average length, with rakes of 4:1 and 3:1 as shown in Figure 4a. The raking was governed mainly by the longitudinal force acting away from the abutment since the passive pressure developed on the back wall of the abutment contributed significantly to resisting the reverse of this force. It was considered sufficient to use a coal tar epoxy coating for protection from corrosion. This was applied over the top section of piles to a depth not less than 3 m below the fresh water table level.

4.2 Pier Piling

One of the main objectives in the substructure design was to limit the obstructions at water level for the benefit of the river users. A single large caisson at each pier or a pilecap at riverbed level could have satisfied this requirement, but both of these were considered to be too expensive and impractical. Therefore, piling systems requiring relatively small pilecaps at water level were the main foundation proposals studied for the piers. It was also preferable for such piling systems to be feasible for all pier locations and this required significant versatility in view of the different sub-soil characteristics at piers 1, 2 and 8 compared with piers 3 to 7.

4.2.1 Displacement piles

Pile cap sizes are directly related to the number of piles required for each pier and hence low load capacity piles were not favoured.

Simple high load capacity precast concrete piles were not ideal due to problems in splicing, making length adjustments and difficulties in penetrating some of the upper soil layers.

Closed ended steel casings, filled with reinforced concrete after driving, were thought to be the most promising of the displacement type piles. Such piles could develop full end bearing without excessive penetration and would not require a cleaning-out operation. Piles of this type were used successfully at the Stirling Bridge, Fremantle, but the largest of these was only 730 mm in diameter with an ultimate capacity of 4500 kN.

The problem in developing a larger driven closed ended casing was to find a hammer that could set such a pile to an ultimate soil capacity equivalent to the ultimate structural capacity of the infilled casing. This aspect was studied and it was determined that the practical limit for top driving a casing with an acceptable wall thickness was a 6500 kN ultimate capacity pile, 800 mm in diameter. A Kobe KB60 pile driving hammer or equivalent, developing in the vicinity of 150 kJ per blow, would be the capacity of equipment required.

Because of these limitations and the likely problem in penetrating the upper sand layers at piers 1, 2 and 8, the alternative of the open ended driven or bored casing became more attractive.

4.2.2 Bored piles

The bored pile proposal studied consisted of a permanent casing installed into the siltstone, cleaned-out to allow socketing beyond the casing and infilled with reinforced concrete. An uncased bored pile was not considered feasible in view of the significant layer of very soft clay.

Whilst such a cased bored pile overcame the limitations of the closed ended casing, there were more unknowns in estimating ultimate capacities and hence the depth of socketing required. The shear strength of the siltstone estimated from UCC and pressuremeter tests was quite variable as discussed in Section 2. Also, the proportion of shear strength developed in shaft resistance, α, could not be estimated with any great certainty. For the siltstone in question and allowing for a reduction in shaft resistance due to wetting from concreting, a resultant value of 0.3 for α was applied to the pressuremeter shear strength. After allowing for base resistance, a socket depth in the order of 12 m was estimated.

Such a piling solution was detailed as one of the tender alternatives recognising the need for an insitu test should the detail be competitive. In fact there was little difference in the tender prices for the various pier foundation systems and the bored pile alternative was eventually eliminated. There would have been little time to conduct insitu tests to confirm shaft resistance and the possibility of collapse during socketing, as discussed in Section 2, also became a concern.

The possible collapse of the siltstone socket also eliminated a Contractor proposed belled end-bearing bored pile.

4.2.3 Steel UC and composite piles

The greater significance of corrosion in the marine environment of the piers, higher pile bending moments and buckling in the soft clay layers, suggested that a normal steel UC pile as determined for the Mt Pleasant abutment, would not be suitable for the piers. The small displacement steel section would, however, overcome the problem of penetrating the relatively dense layers at piers 1, 2 and 8 to reach the siltstone. These observations led to the development of a 6000 kN ultimate capacity composite pile comprising a 310 UC 240 steel lower section and a 550 mm square prestressed concrete upper section. The precast prestressed concrete section incorporates a cast-in short length of the 310 UC 240 thus enabling a welded splice to the lower steel section (Figure 4b). The joint detail is similar in principle to the composite piles used in the Captain Cook Bridge at Taren Point, New South Wales (Barmby and Miller, 1969) and was considered to be superior to the

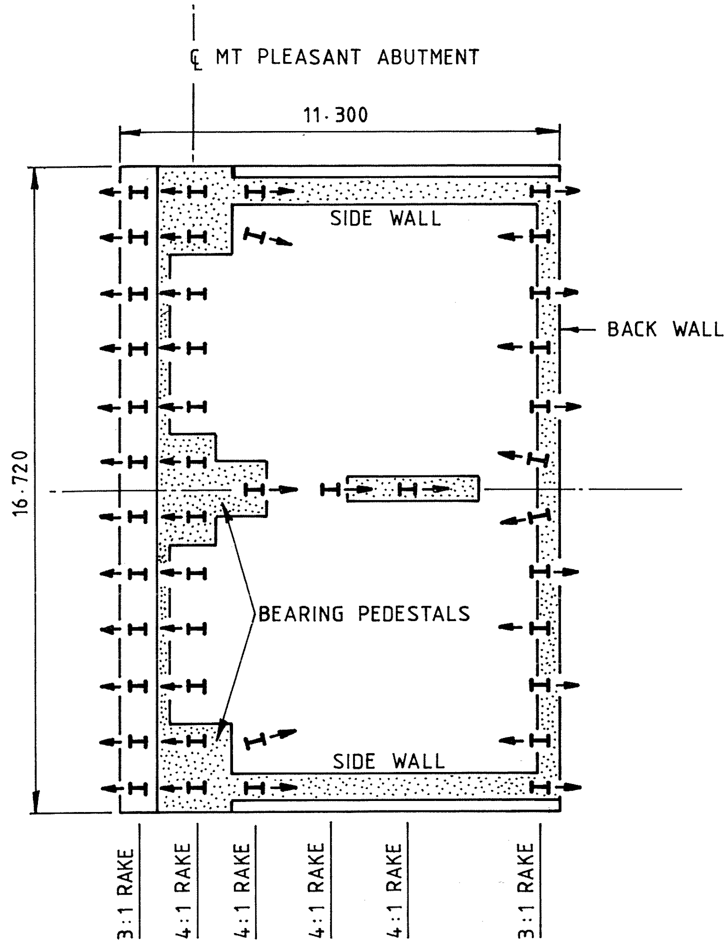

Figure 4a

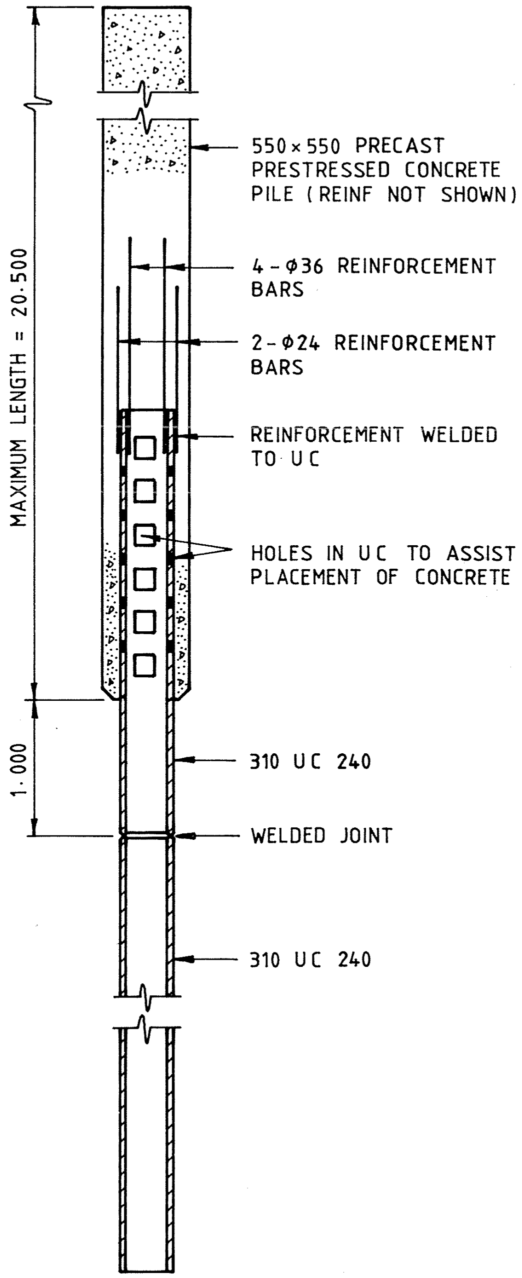

Figure 4b

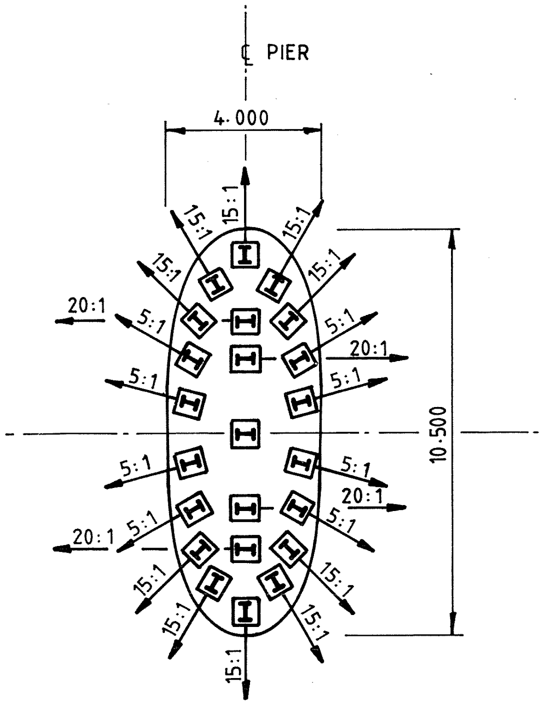

Figure 4c

Figure 4 Adopted piling details

alternative composite form involving simply a concrete encasement to the upper length of a continuous UC pile.

Determination of the length of the upper pre-stressed concrete section at each pier involved a compromise to satisfy acceptable handling weights and lengths, corrosive zones, pile bending moments, and buckling in weak substrata. There was also a need for this concrete section to avoid firm material which would cause difficulties in achieving the required UC penetration into the siltstone. This latter requirement was difficult to satisfy at piers 1, 2 and 8 and the necessity for jetting at pier 1 and to a lesser extent at piers 2 and 8 was considered a distinct possibility.

The composite piling system as finally developed, comprised 23 piles per pier of 39 m average length and rakes varying up to 5:1. The piles were accommodated in a 10.5 m x 4.0 m pile cap as shown in Figure 4c. Jetting was in fact required for pier 1 and a Kobe KB60 (150 kJ) pile driving hammer was used for all piers. Wave equation calculations had indicated that at least a Kobe K42 (100 kJ) size hammer would be necessary.

5. PILE TESTS

Because of the unconventional detail of the composite pile, driving and load tests were carried out. Two locations were selected for the load tests, one near the Mt Pleasant abutment (Test Pile 1) and the other in the general vicinity of the first pier (Test Pile 2). The ultimate capacity being sought was 6000 kN for both the pier composite piles and the abutment steel piles.

5.1 Test Pile 1

Test Pile 1 was carried out before a decision on the pile type for the pier foundations had been made. This test was essentially in two parts. Initially the steel section was driven to a penetration depth of 4.0 m into the siltstone and, subsequently, load tested. A 600 mm square prestressed concrete section 8 m long was then connected to the steel section and the composite pile driven a further 3.0 m into siltstone, followed by a further load test.

The objectives of the tests conducted on this pile were to:

(a) Determine the driving parameters for use in the wave equation analysis.
(b) Determine the depth of penetration into the siltstone and the associated "set-up" factor that would be necessary to achieve the desired ultimate capacity.
(c) Examine the performance of the connection between the prestressed concrete and steel sections of the composite pile.

As shown in Figure 1, there is approximately 10 m of sand overlying the clayey materials, which would affect both the driving characteristics of the pile as well as its resistance. It was decided to eliminate the frictional resistance of the sand by driving into a bentonite filled hole, with the intention of modelling the majority of the pier piles.

Because of the anticipated "set-up" properties of the siltstone, each driving phase was followed by a "set-up" test approximately 24 hours later, which involved re-driving the pile over a short distance. The resulting increased pile set gave a more representative assessment of capacity and provided a check on the calibrated running set. This particular test was included in the specification for the bridge contract.

Following the initial driving of the steel section a further four driving phases were undertaken with the composite section to study the degree of "set-up" in relation to the depth of penetration. The loading schedule for each pile test was generally based on AS 2159-1978 Type B Loading. Also the constant rate of penetration (CRP) test was used to provide confirmatory results. Tension piles were used as the reaction system for the first load test. However, they did not have sufficient capacity for the magnitude of the load estimated for the second load test and a kentledge system was adopted. The results of this series of driving and load tests can be summarised as follows:

5.1.1 Test load results

With a penetration of 4.0 m into the silstone, the failure load recorded was 4150 kN. A further 3.0 m penetration resulted in an increase of 660 kN to 4810 kN. This increase was lower than anticipated and could not be satisfactorily explained. This situation contributed to the decision to carry out a second test pile.

5.1.2 Driving characteristics

Three different hammers (Kobe K32, K35 and K45) were used, but because of the short distance driven for each operation and with the exception of the initial driving phase, continuous driving conditions were generally not achieved. The sets recorded after the initial phase were affected by "set-up" and without the corresponding continuous driving case, the results had limited application. The test did, however, justify the earlier design estimate that at least a hammer equivalent to a Kobe 42 would be required to drive the composite pile.

5.1.3 Composite pile connection

The connection between the prestressed concrete section and the steel section was exposed at all times and showed no visible signs of distress after a total of 3700 blows had been struck. It was anticipated that this figure would be greater than the number of blows required to drive the bridge piles to level and set.

5.1.4 Concrete pile head

The original pile head fractured before the desired penetration was achieved. An excessive amount of concrete cover was considered to have contributed to this failure. Redetailing to overcome this gave more satisfactory results.

5.1.5 Discussion of results

Because the number of piles at the Mt Pleasant abutment were not severely limited by pile cap size or by economy for this one foundation, it was considered satisfactory to extrapolate results conservatively from Test Pile 1 for this design. This was done by extrapolating from the second test load result (4810 kN) which indicated that a penetration of about 10 m into the siltstone would be required. The relatively higher safety factor associated with this conservative assessment provided some flexibility in the degree of penetration and set to be achieved for the abutment piles.

Further calculation based on the second test load,

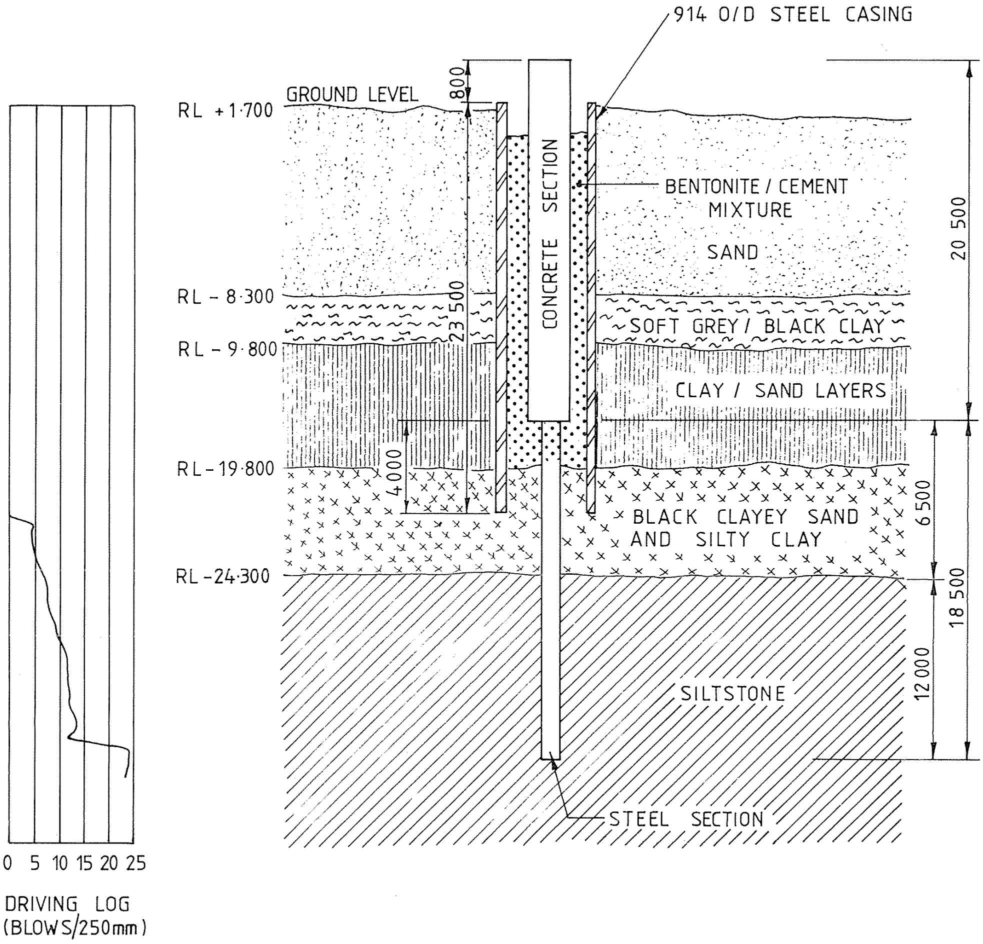

Figure 5 Details of Test Pile 2

which involved subtraction of the estimated skin friction of the upper clay layers and the end bearing component, gave an implied adhesion factor α of 0.55 using shear strengths derived from the UCC tests on the siltstone.

Dynamic analysis of this test pile was carried out using the wave equation model. However the discontinuous nature of the driving procedure made it difficult to determine a sufficiently accurate penetration or set for the less conservatively designed pier pile. This also influenced the decision to install a further test pile.

5.2 Test Pile 2

Test Pile 2 was a composite pile located near pier 1. The head detail of the pile had been modified and the concrete section reduced following the completion of Test Pile 1. A thick layer (not less than 110 mm) of softwood cushioning was used which was found to yield the most satisfactory results with respect to performance of the pile head.

As for Test Pile 1, there was a significant depth of sand overlying the clay layers (Figure 1). In order to simulate the river pier conditions, a steel casing was driven into the black clayey sand and silty clay layer and the material within the casing removed.

The composite pile was then driven within the casing to a penetration depth of 12 m into the siltstone using a Kobe KB60 hammer (Figure 5). Based on data from Test Pile 1, an allowance for "set-up" was conservatively estimated at 20%. Wave equation studies had suggested a pile running set of 30-35 blows/250 mm to achieve an ultimate capacity of 6000 kN. Either this set or a penetration depth not greater than 12 m was selected to determine the final level of the test pile. In this case a maximum set of 23 blows/250 mm was achieved (Figure 5), with the pile penetrating 12 m into the siltstone.

A "set-up" (redrive) test was carried out approximately 24 hours later and yielded an equivalent set of 155 blows/250 mm measured over a penetration depth of 50 mm. This value was used to determine wave equation parameters for the pier piles.

To better simulate the river conditions and to provide lateral stability of the pile during load testing, a bentonite/cement mixture was pumped into the casing to surround the pile. The composition of the bentonite/cement mixture was adjusted to have a similar shear strength to the soft clay.

The load test was carried out using a kentledge system (Figure 6). Again a loading schedule based on AS 2159-1978 Type B Loading was adopted, followed by a CRP test.

The load/settlement and time/settlement curves for the design ultimate capacity of the pile (6000 kN) are shown in Figure 7. There is clearly negligible residual settlement, nor any sign of a levelling off on the load/settlement curve, demonstrating that the pile was supporting its ultimate design load.

The pile was then reloaded using the CRP method in

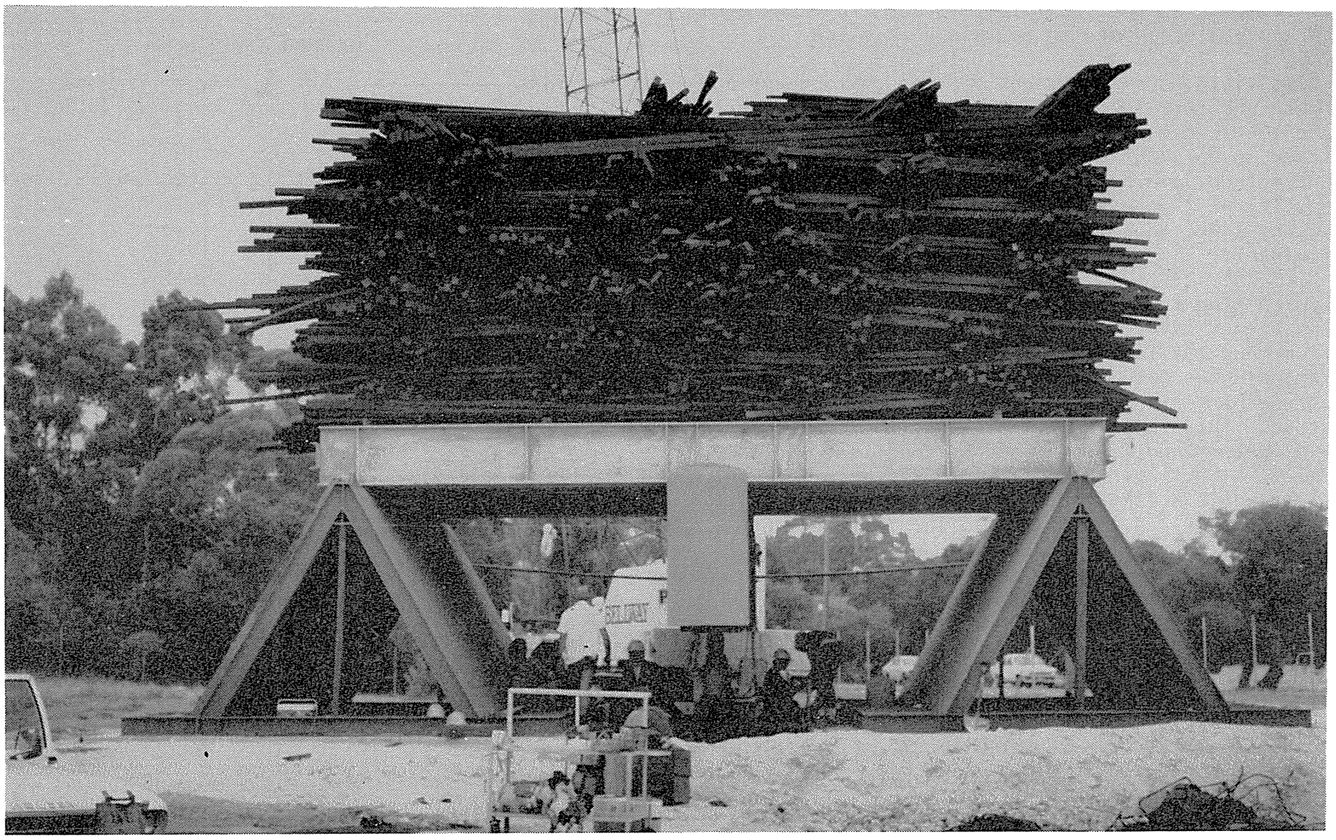

Figure 6 View of pile load test

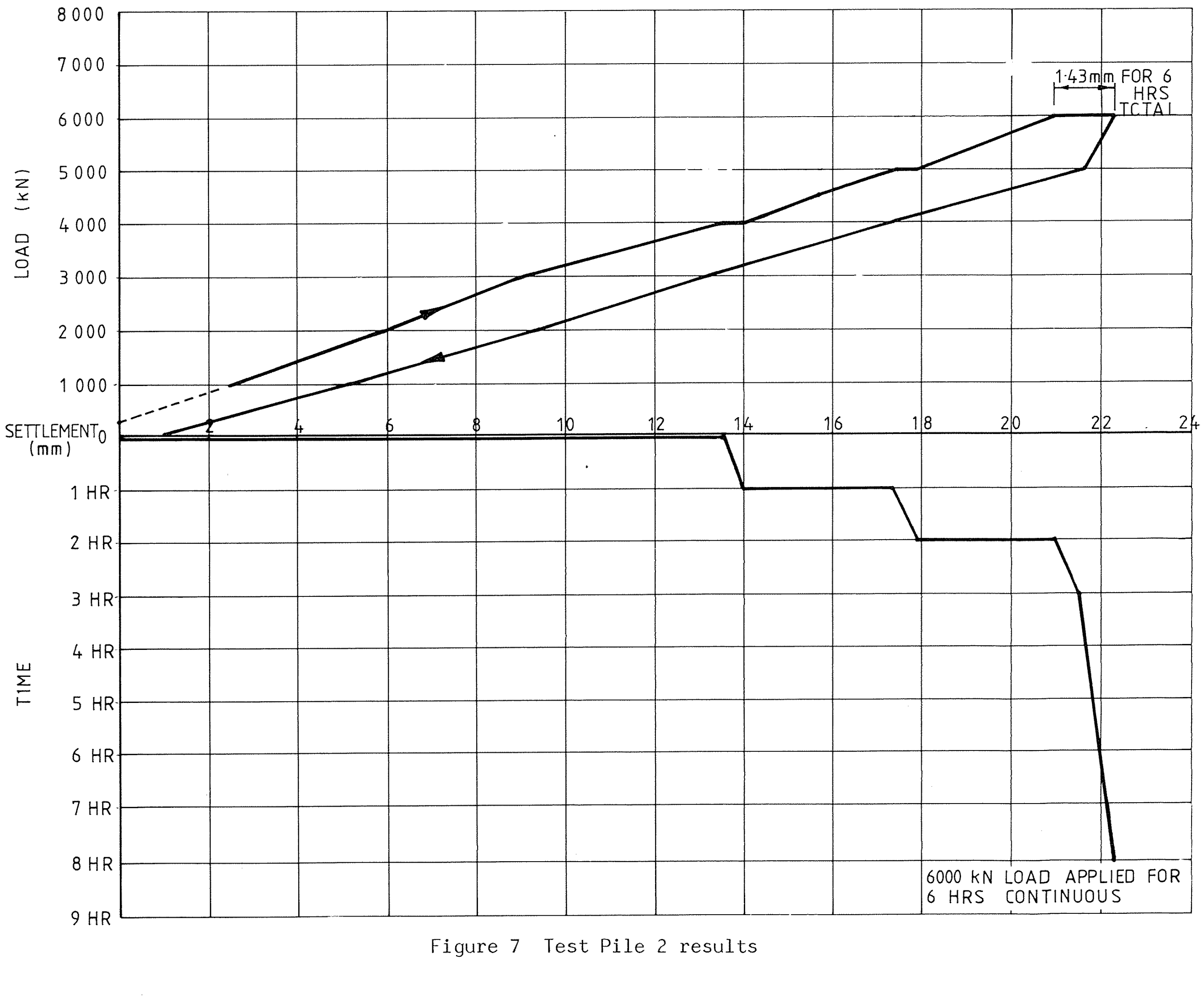

Figure 7 Test Pile 2 results

order to determine its failure load. The kentledge system was preloaded to sustain a reaction of 7400 kN. This load was close to the point (7650 kN) where yielding of the steel section could occur and there was therefore little to be gained by increasing the load much above this figure. The load was taken up to 7400 kN but failure, as defined by the CRP method, did not occur. However, the load/settlement characteristics of the pile in this higher load range indicated that failure was imminent.

Subtracting the estimated skin friction of the upper clay layer and the end bearing component from the test load of 7400 kN, the implied adhesion factor α was calculated as 0.45, using shear strengths derived from the UCC tests on the siltstone. This compares with the previous value of 0.55 determined from the second load test of Test Pile 1. For material of similar strength to the siltstone, AS 2159-1978 suggests values of α in the order of 0.3. This is similar to the value which was used to estimate the capacity of bored piles (Section 4.2.2), but is significantly less than the value estimated from Test Pile 2. However, this assumes a shear strength based on a purely cohesive material. The results from the pressuremeter indicate that the shear strength is higher than that given by the UCC results as noted in Section 2.3. Hence based on the UCC strength as a parameter, it is not unreasonable to expect the implied value of α to be higher.

The ultimate capacity/blow count curves for Test Pile 2 are given in Figure 8. The set achieved on redriving (155 blows/250 mm) corresponded to a load of 7400 kN, which was accepted as the maximum load the pile could support. This then suggested that the estimated capacity corresponding to a set of 23 blows/250 mm would be 4400 kN. This implies a "set-up" factor in the order of 1.66.

Based on the results of the load tests, pile lengths were determined using 12 m penetration into the siltstone. The driving set was estimated from a wave equation analysis for the pile, using the calibrated parameters determined from the pile tests.

6. CLOSURE

The detailed site investigations, soil testing, pile testing and foundation reviews for the Mt Henry Bridge brought forward some interesting aspects in relation to foundation alternatives. The possibility of soil failure during socketing into an over-consolidated hard sandy silty clay, the development of pile shaft resistance in such a material, the detailing of an efficient composite pile and the determination of driving characteristics to produce the required ultimate load capacity, were the main results of this exercise.

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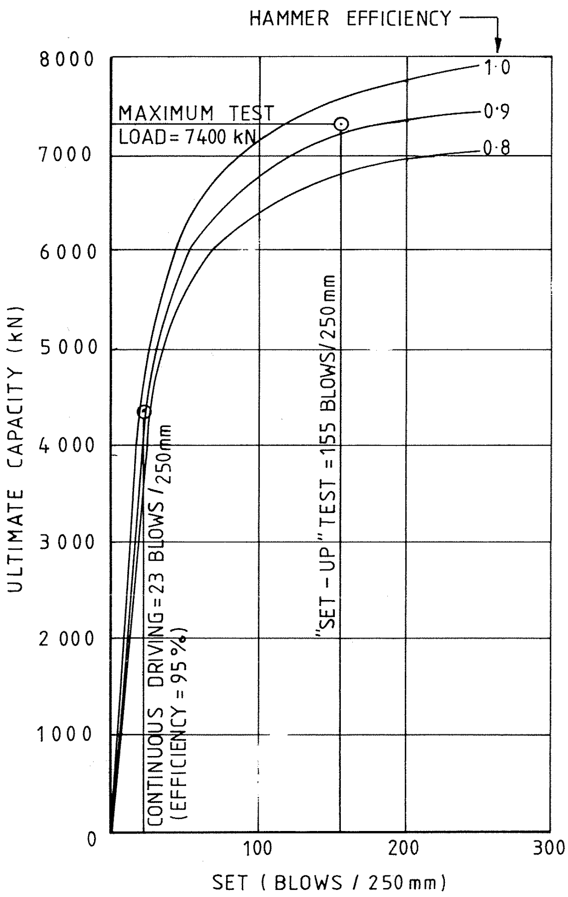

Figure 8 Wave equation results Test Pile 2

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