# Blanchetown Bridge Geotechnical Investigation

M.L. DUTHY, BE(Hons). PhD, MIEAust, CPEng
Geotechnical Engineer, PPK Environment & Infrastructure Pty Ltd

Summary: A geotechnical investigation for a proposed replacement bridge at Blanchetown, South Australia was carried out. The investigation comprised drilling, coring, field and laboratory testing, analysis, pile testing and reporting. The background to the geotechnical investigation, the investigation methodology and the investigation results are briefly outlined, and a model of the geotechnical profile along the proposed bridge alignment is presented. The main geotechnical issues for the design and construction of the proposed bridge are then discussed. These comprise design and constructability of the bridge pier and abutment footings; slope stability and excavatability of the western bridge approach cutting; use of the excavated material as fill; settlement performance of the eastern bridge approach embankment; and design or construction techniques to minimise the effects of embankment settlement.

## 1. INTRODUCTION

The existing bridge at Blanchetown, South Australia, carries road traffic across the River Murray. The bridge is located on National Highway 20, about 130 km north-east of Adelaide, South Australia. However, the existing bridge is unable to carry the predicted future traffic loadings and so a replacement bridge is being constructed nearby. Rust PPK Pty Ltd (now PPK Environment & Infrastructure Pty Ltd) were commissioned by the Department of Transport (now Transport SA) to undertake a detailed geotechnical investigation for the proposed new bridge, which comprised drilling, coring, field and laboratory testing, analysis, pile testing and reporting. The background to the geotechnical investigation, the investigation methodology and the investigation results are briefly outlined, and a model of the geotechnical profile along the proposed bridge alignment is presented. The main geotechnical issues for the design and construction of the proposed bridge are then discussed. These comprise design and constructability of the bridge pier and abutment footings; slope stability and excavatability of the western bridge approach cutting; the use of the excavated material as fill; settlement performance of the eastern bridge approach embankment; and design or construction techniques to minimise the effects of embankment settlement.

## 2. GEOLOGY OF THE AREA

Published information (Geological Survey of South Australia (1), (2)) shows that the site lies within the Murray Basin, which was formed at the beginning of the Tertiary. The oldest geological unit of interest is the Lower Miocene Mannum Formation. This unit is a marl (calcareous shelly sandstone of marine origin) that is expected to be present as a continuous layer at depth across the site area. The marl is massive, fossiliferous and of low strength, and is expected to be over 30 m thick. The marl is overlain by the Lower Miocene Morgan Limestone geological unit. This is a sandy limestone/calcareous sandstone that is also of marine origin, massive, fossiliferous, and of low strength and significant thickness (>15 m).

In areas not subjected to significant erosion, the sandy limestone is expected to be overlain by the Pliocene Norwest Bend Formation, which contains fossiliferous sandstone, limestone, calcareous sand and oyster beds. This formation is in turn overlain by Pleistocene Bakara Soil, which is a soil containing moderately hard, massive sheet or nodular calcrete and fossils. Recent aeolian sands are expected to cover the Bakara Soil and form the existing ground surface west of the cliff tops present on the western side of the River Murray in the general area.

Over the course of many years the River Murray has cut through the Morgan Limestone and into the marl for a depth of tens of metres, and this has resulted in the exposure of these rock strata on the face of the steep cliff that is present on the western (Adelaide) side of the river in the general area.

The relatively flat marl surface that resulted from the previous river scouring was then overlain by more recent sands. This material is likely to represent drift sands deposited by wind and then reworked by river flow. The sands belong to the Recent Monoman Formation. They are present east of the western cliff face to thicknesses of the order of 15 m, though over the existing river width some reduction in thickness due to river erosion is likely. The sands are fine to coarse grained and relatively loose in general.

Organic clays, silts, sands and muds several metres thick are expected to overlie the riverine Monoman Formation sands and form the surface strata for that part of the river bed within 100-150 m of the western bank. These soils are soft and loose, compressible and saturated and represent the riverine Coonambidgal Formation.

The land beyond the eastern river bank is low lying for a significant distance eastwards and contains a number of lagoons. The Recent surficial soils in this area are also expected to contain several metres of Coonambidgal Formation soils.

Geotechnical information obtained by the Department of Transport (DoT) from the borehole drilling for the original bridge at Blanchetown, though not discussed in this paper, was consistent with the foregoing published information.

## 3. PROPOSED CONSTRUCTION

The proposed development comprises the provision of a new bridge and associated infrastructure over the River Murray at Blanchetown to replace the existing bridge. The new bridge and associated infrastructure will be similar to the existing bridge and associated infrastructure in its geometry, form and construction, and will be located slightly upstream (20-30 m) of the existing bridge. The proposed development will comprise the following elements:

(1) Western Approach

The western (Adelaide) approach road to the proposed bridge will be cut into the existing land from within about 500 m west of the proposed western abutment. The depth of the proposed limestone rock cutting will be similar to the existing cutting (approximately 9 m maximum depth) but the new north batter slope will be only about 1 vertical : 2 horizontal, instead of up to 1 vertical : 1 horizontal as for the existing cutting.

(2) Western Abutment

The western (Adelaide) abutment of the new bridge will comprise a shallow spread footing founded on the weathered limestone rock near the top of the 20 m high cliff that is present at this location.

(3) Bridge

The new bridge will span 407 m between abutments, and will be made up of end spans of 32 m and 25 m for the western and eastern ends respectively, and seven internal spans of 50 m. The two lane road carriageway will be supported by a post tensioned concrete box girder and 8 concrete piers, each of which are in turn supported by a pile cap and pile group. The piles are to be precast prestressed octagonal concrete piles driven to the marl. The elevation of the proposed b and its longitudinal gradient will correspond to that c existing bridge. The alignment of the proposed bric nearly parallel to that of the existing bridge. The bridg be incrementally launched from the embankment o eastern side of the river.

(4) Eastern Abutment

The eastern (Waikerie) abutment of the proposed bridg comprise a cellular raft retaining structure and piled foo the end of the eastern embankment, the piles of which be founded below the relatively weak surface soils ar the marl at depth.

(5) Eastern Approach

The eastern (Waikerie) approach road to the proposed will be supported by an embankment along its entire The height of the proposed embankment will be simila to 1 m higher than the existing embankment (approxim m maximum height) though the new north batter slope only about 1 vertical : 3 horizontal instead of up to 1 v 2 horizontal as for the existing embankment. Th embankment will traverse low lying ground and the e two lagoon pools along its length.

## 4. OUTLINE OF INVESTIGATION

The objectives of the investigation were to:

- identify the general sub-surface geological profi site;
- determine the strength parameters for the various
- develop a geological model for the site, basec field work and the results of previous investigati
- define appropriate footing solutions for the p bridge piers and abutments, and exami constructability of the various footing options;
- assess the stability and excavation methodolog proposed cutting on the western approach, suitability of excavated material for genera pavement use;
- assess the expected performance of the embankment on the eastern approach, in settlement potential for various design construction scenarios.

The fieldwork was carried out in March 1996 unde time supervision of experienced geotechnical engine test location plan for the fieldwork is given in Fig total of 7 land based boreholes and 6 over-water were drilled to depths between 5.8 m and 25.55 m. based boreholes were drilled by a truck mounted ri over-water boreholes were drilled by mounting th onto a floating steel pontoon assembly. The boreh positioned at each of the proposed pier and locations for the new bridge, as well as along th approach.

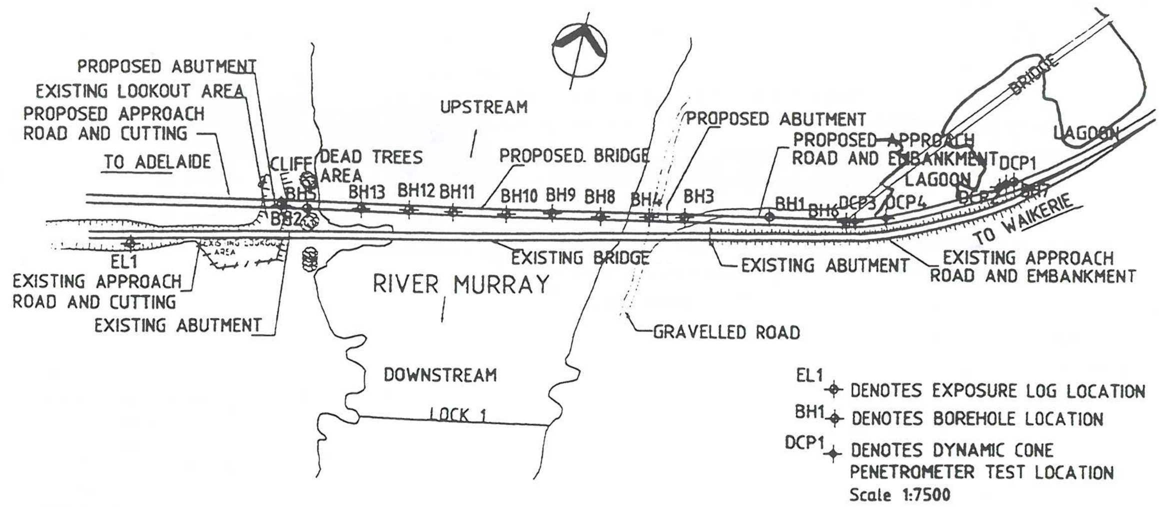


**Figure 1: Test Location Plan**

In general, each land based borehole was drilled using the hollow spiral auger technique with split spoon sampling at regular intervals initially, and then wash boring. For the over-water boreholes, wash boring was generally used for approximately the uppermost 10 m, before diamond coring was adopted for the weathered marl rock that comprised the remainder of each borehole.

The soil and rock samples recovered from the drilling were logged by the visual manual method cited in AS1726-1993 "Geotechnical Site Investigations", with hand held Geotester readings taken in all cohesive strata in order to estimate their consistency. Selected thin walled push tube samples were taken of the soft compressible clay soils encountered in the boreholes located along the proposed eastern approach embankment of the new bridge, in order to obtain hand held Geotester readings of the clay in an undisturbed state and hence more accurately estimate the in situ consistency of the clays. Standard Penetration Tests (SPTs) were also carried out in the granular soil layers in each borehole in accordance with AS1289 test 6.3.1, in order to determine the relative densities of the sand and gravel soil strata encountered in the drilling. SPTs were also completed in the weathered marl rock strata that were encountered, though only when wash boring was used as the drilling technique.

Oedometer testing in accordance with AS1289 test 6.6.1 was undertaken for three thin walled push tube (undisturbed) samples of the soft clay soils encountered in the Waikerie embankment boreholes, in order to determine the consolidation properties of these soils. Point load strength index testing ($I_S(50)$) of rock specimens from all boreholes where diamond coring was used was also undertaken in accordance with International Society for Rock Mechanics (ISRM) standards (3).

The exposed face of the existing Adelaide approach cutting was logged using the visual manual method cited in AS1726-1993 "Geotechnical Site Investigations", in order to provide information about the expected subsurface profile along the alignment of the proposed Adelaide cutting. The existing cutting was logged at the approximate location where the cutting depth was greatest. Samples of the soil and rock materials exposed along the depth of the cut face at this location were collected for Estimated CBR testing in accordance with the Department of Road Transport test method DRT-MAT-TP133. The purpose of this testing was to determine the utility of the cutting material as general fill or pavement fill for the proposed Waikerie embankment.

## 5. RESULTS OF INVESTIGATION

The stratigraphy of the subsurface strata encountered along the alignment of the proposed replacement bridge confined the published information and the results of the drilling for the original bridge. Sands, clayey sands and some clay layers were encountered across the river and the land to the east, from the ground surface to depths between 7.6 m and 15.7 m. These deposits principally comprised sand layers, but some surface or near surface clays were present within the shallow, western-most 100-150 m width of the river and within the land east of the river. The upper clay layers were generally very soft in consistency over water, and firm to stiff in consistency over land. The lower sand layers were generally medium dense over dense. The marl stratum that underlay the river soil deposits was very similar in appearance and properties across the site, and the marl horizon showed only a slight dip to the west. The marl was found to generally be a very low strength rock, and equivalent to a dense gravelly sand in strength. The surface of the marl rises steeply at the western bank of the river, so that the thickness of overlying riverine deposits is significantly reduced. Weathered limestone of low strength directly overlay the marl on the western bank of the river and formed the nearby steep cliff.

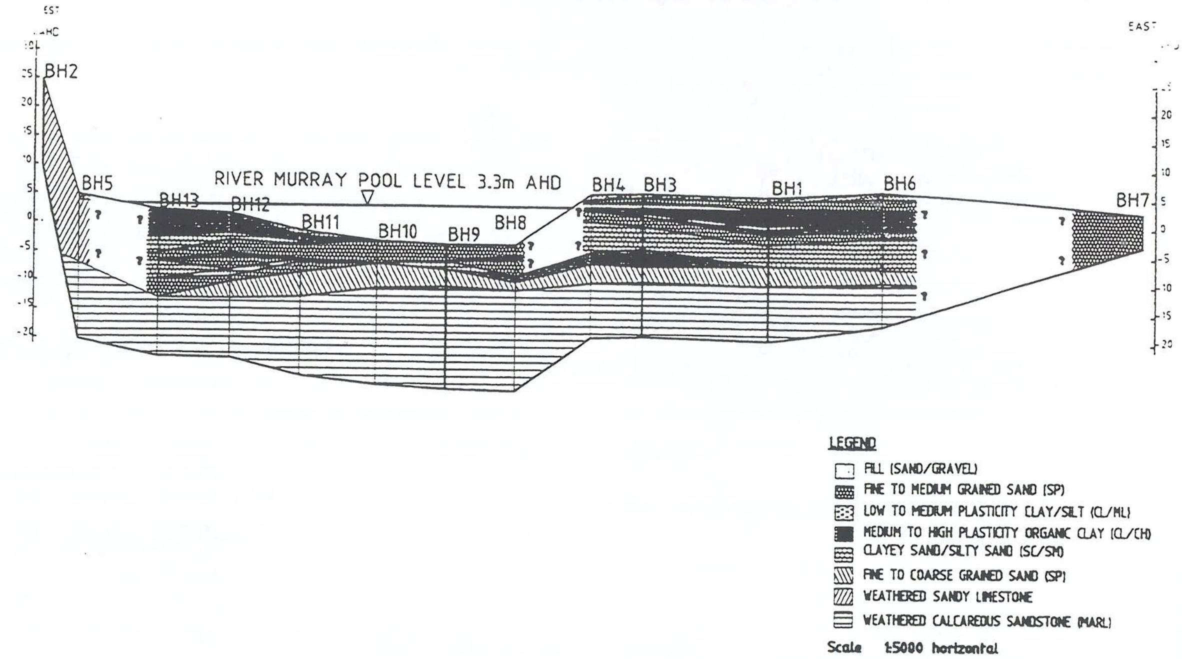


**Figure 2: Inferred Geological Transect Along Bridge Alignment.**

Figure 2 summarises the results of the soil drilling and rock coring in an inferred geological transect along the alignment of the bridge.

The results of the oedometer testing indicated that all three clay samples were actually heavily over-consolidated, with over-consolidation ratios of the order of 15 due to periodic dessication. The coefficient of consolidation was found to be around 10 $m^2/yr$ and the coefficient of volume change was found to be around $1x10^{-4} m^2/kN$ over the stress range of interest.

The exposure log of the existing cutting confined the published geology. Assessment of the true nature of the exposed strata was made difficult by the fact that the cut face has been exposed for about 30 years and therefore has been subjected to appreciable weathering. However, the limestone material appeared massive and without obvious discontinuities. The strength of the material was judged to be very low to low. SPT testing at the proposed western abutment showed that the weathered limestone was equivalent to a very dense sand in general. Diamond coring and point load strength index testing of the weathered limestone below 5 m at this location confirmed that the rock strength was very low to low in general.

Prior to particle size distribution and Atterberg Limits testing, each sample of the approach cutting material received a set amount of mechanical energy to try to break up the larger sized fragments. This was done to simulate the fracturing of the cut face materials that would occur upon excavation and subsequent placement and compaction as fill.

The results of the testing indicated that all samples were st not sufficiently fine sized for Estimated CBR testing, despi the pre-test mechanical energy treatment. This implied th the Estimated CBRs of the tested material exceeded 20. T test results indicated that the weathered limestone degraded a gravelly silty sand material under moderate mechanic effort, and therefore was of low strength at most.

## 6. DEVELOPMENT OF GEOTECHNICAL MODEL

Based upon the published geological information for the are and the investigation for the proposed bridge, an idealis geotechnical model along the alignment of the propos bridge and approaches was developed. This geotechnic model was used as a basis for the geotechnical desi recommendations and construction advice for the project th are discussed in Section 7 of this paper.

### 6.1 Western Bridge Abutment

The conceptual geotechnical model for this location (on t of western cliff) comprised very dense sand representi weathered limestone to 5 m depth, and limestone rock bel 5 m.

### 6.2 Bridge Pier 1

The conceptual geotechnical model for this location ( western river bank at base of cliff) comprised loose grave sand to 2 m depth, over firm clay/silt to 5 m, over medi dense gravelly sand to 12 m depth, over marl.

### 6-3 Bridge Piers 2-4

The conceptual geotechnical model for these locations (over western part of River Murray, and encompassing the shallow portion of the river) comprised 3 m thickness of soft clay, over 1 m thickness of clayey sand, over 3 m thickness of medium dense sand, over 2 m thickness of clayey silt, over 3 m thickness of medium dense sand, over 2 m thickness of coarse grained dense sand, over marl.

### 6.4 Bridge Piers 5-7

The conceptual geological model for this location (over eastern part of River Murray, encompassing the deeper portion of the river bed) comprised 4 m of medium dense sand, over 1 m of clayey sand, over 3 m of coarse grained, dense sand over marl.

### 6.5 Bridge Pier 8 and Eastern Bridge Abutment

The conceptual geological model for this location (on eastern river bank) comprised 1 m of loose gravelly sand, over 2 m of stiff clay, over 2 m of medium dense sand, over 4 m of medium dense clayey sand, over 3 m of stiff clay, over 3 m of medium dense coarse grained sand, over marl.

### 6.6 Eastern Approach Embankment

The conceptual geological model for this location (land to east of River Murray) comprised 2 m of medium dense gravelly sand, over 4 m of stiff clay, over 2 m of medium dense sand, over 5 m of medium dense clayey sand, over 3 m of medium dense coarse sand, over marl.

## 7. DISCUSSION OF GEOTECHNICAL ISSUES

### 7.1 Western Approach Cutting

Geotechnical issues for the western approach cutting comprised slope stability, excavatability and use of excavated material as fill. The faces of the existing cutting have slopes of up to 1 vertical to 1 horizontal (1V:1H). Though the existing cutting was approximately 30 years old, it appeared to be performing satisfactorily apart from some apparent weathering and erosion of the exposed cut face that would have resulted in periodic minor sloughing and slumping of surficial material. It was very likely that the new cutting would be excavated in soil and weathered rock that was similar to the material excavated for the existing cutting. Since the proposed cutting slope was approximately 1V:2H, the western approach cutting could clearly be constructed without an unacceptable risk of slope instability failure (an approximate factor of safety of 2.0 results if a friction angle of 45° is assumed for the weathered limestone), provided that the proposed cut face is suitably protected from erosion due to water.

It was expected that the new cutting would be able to be excavated by a combination of digging and ripping. This was confirmed through use of a Franklin (Franklin et al (3)) chart, which relates excavatability of rock to the point load index strength and fracture spacing. An average point load index strength of 0.1 MPa (very low to low strength rock) was assumed for the material to be excavated, based on observation of the rock material exposed in the existing cutting, and the field and laboratory test results associated with the borehole drilled at the proposed western bridge abutment location. Although the limestone rock was massive, it was easily fractured. Adopting an average fracture frequency of 10 fractures per lineal metre based on Rock Quality Designation values from the cored rock samples, the Franklin chart showed that the limestone rock could still be ripped with a Cat D9 equivalent dozer.

Most of the material to be excavated from the proposed cutting was weathered limestone, that is equivalent to a gravelly silty sand when broken down. Based on the results of the Estimated CBR testing, the weathered limestone rock from the cutting was expected to break down under the mechanical energy of excavation and then placement and compaction, to form sand to gravel sized particles. Based on the foregoing qualities, the material that was excavated from the proposed cutting was expected to be suitable for pavement fill, as well as for general fill. A CBR value of 20 for excavated material comprising a sand/gravel mixture was suggested for preliminary pavement design purposes. A later compaction trial performed by DOT confirmed the foregoing recommendations, and also showed that the bulking factor for the material was 0.87.

### 7.2 Western Abutment

Geotechnical issues for the western abutment comprised excavatability, vertical bearing capacity and slope stability. Excavatability of the weathered limestone rock was discussed in Section 7.1 of this paper, and it was expected that the cliff top material above the abutment footing would be able to be excavated by a combination of digging and ripping.

An allowable bearing capacity of 500 kPa was recommended for a spread footing founded in the weathered limestone material. This was obtained by equating allowable bearing capacity to 0.3 times unconfined compression strength (correlated from a mean point load index test result of 0.164 MPa) and then selecting approximately half this value to allow for rock weathering and proximity to the cliff face.

The proposed spread footing to support the Adelaide abutment was to be located close to the top edge of the steep cliff face that was present in the area. The existing cliff face appeared to be stable, although there was evidence of shallow surface sloughing and slumping due to weathering and erosion. Based on the survey information provided by the DOT, the cliff height was approximately 20 m and the average cliff slope was about 45°. The centerline of the proposed abutment was approximately 5 m west of the edge of the cliff.

The slope stability of the cliff was analysed using Bishop's simplified method of slices as implemented in computer program XSLOPE (University of Sydney, 1986). Provided that the foundation material for the abutment was no weaker then very dense sand or very weak to weak limestone, as encountered in the borehole at this location, an adequate factor of safety against slope instability of 1.6 was indicated by the computer analysis, assuming that the foundation soils and cliff face are protected from softening and erosion due to water.

### 7.3 Bridge Piers

The geotechnical issues for the proposed bridge piers 1-8 included the capacity of the footings and how this varied with founding material, depth, footing type and footing size; and the proposed construction technique for the footings. A deep footing must be used to support the bridge piers both on land and over-water, as the near surface soils were not sufficiently strong to support the loads from the bridge superstructure if a spread footing was used. Driven preformed piles were considered to be the most appropriate pile type for the proposed bridge in terms of constructability and performance, given the need to install piles over water and through weak ground with a high water table. A number of different driven preformed pile types were available, though it was considered that precast prestressed concrete piles or steel tube piles were most likely to be used. A nominal 450 mm size pile was expected.

The piles would generally have to be driven so as to penetrate or at least reach the very low to low strength weathered marl rock that underlay the sand and clay soils across the extent of the bridge site. In areas where the marl strength is relatively high and significant driving resistance of the piles in this stratum is achieved, little penetration of the marl is then needed and the axial pile capacity will be mainly from toe bearing. However, for lower strength marl, significant penetration of the pile into the marl stratum may be required to develop the required pile set. In this case, a greater proportion of the pile axial capacity would develop from shaft friction. Significant toe and shaft capacities may also develop in the dense, fine to coarse grained gravelly sand that overlay the marl in some locations (bridge piers 5-7 for example).

The predicted ultimate pile capacities for vertical compressive loading were about 2000-2500 kN (depending on the depth of overburden) for 450 mm size piles founded on the marl surface when the simple calculation methods and parameter values presented in Appendix A of AS2159-1978 "SAA Piling Code" were used. The marl was considered as equivalent to a dense sand with $N_q$ value of 180 for the toe capacity calculations. However back-calculated ultimate compressive capacities from the driving records of the existing bridge piles, using wave equation analyses, were 4800-5000 kN for a 400 mm dimension octagonal concrete pile founded near the marl surface. Therefore, the theoreti pile capacities were less than half the back-calcula capacities. There were a number of possible reas suggested for this. Firstly, the maximum values for sh friction and toe bearing suggested by the "limiting dep concept in AS2159-1978 may be unnecessarily conservati The magnitude of shaft friction and toe bearing are sensiti to the value of limiting depth, and therefore the actual valu could be significantly higher than those suggested by t theoretical design. Secondly, the nature of the subsurfa soils may be such that the soil strength after pile drivi decreases with time. This is because for founding str equivalent to dense sands, dilatant soil behaviour could expected when the in situ soils are disturbed by pile drivi leading to temporary negative pore pressures. If t foundation soils were dilatant, the pile capacities b calculated from the pile driving records would be up bounds to the true in service pile capacities.

For detailed pile design, it was recommended that minimum ultimate bearing capacity of a 450 mm diame pile be taken as 2000 kN for a pile bearing on the surface the marl, based on a static analysis and the minimum depth soil overburden to the marl. A simple elastic analysis gav predicted pile settlement of around 5 mm under corresponding working load of 800 kN.

However, due to the uncertainty associated with the predic pile bearing capacity, it was recommended that a pile driv test be undertaken prior to construction piling.

A 400 mm diameter steel tube test pile was therefore dri initially open ended, and then with a gravel toe plug, in low lying land adjacent to the proposed location of eastern abutment of the replacement bridge.

The dynamic load test including CAPWAP analysis indica that an ultimate geotechnical strength for a 450 mm diam pile driven closed ended was at least 2580 kN for a penetration into the marl. A design geotechnical strengt 2000 kN was recommended to be adopted for the bri design.

The pile test indicated that any hollow pile to be used for bridge must be driven closed ended, as a pipe pile dr open ended could not be expected to form an adequate plug at the site.

The pile test also indicated that pile "set up" could anticipated at the site and that the measured resistance a time of driving will be the lowest achieved stre throughout the life of the pile. Since the restrike resistanc the pile was found to be higher than the end of dri resistance at this site, this finding was contrary to ea predictions. An explanation could be related to the natu the medium dense sand and marl occurring at depth. T materials were highly calcareous with a high proporti shell particles, so that their behaviour would be differe that previously observed on mainly quartzitic sands.

The test result provided site specific data for a back analysed wave equation analysis of a 450 mm closed ended pile. For a pile driven to the interface of the medium dense sand and marl at the site of the eastern abutment, at an ultimate resistance of 2580 kN, a set of 5 mm per blow was expected for the pile driving equipment that was likely to be used.

It was recommended that the piling specification for the project states a target depth to which each pile is to be driven but that the contractor ensures that the ultimate geotechnical capacity of each pile is to be at least 2500 kN (2000/0.8), and the contractor provides site sets and necessary calculations to verify the required capacity of every pile installed.

### 7.4 Eastern Embankment

The geotechnical issues for the proposed eastern embankment included the magnitude and time of settlement of the foundation strata under the embankment surcharge, and construction techniques for the embankment. Based upon the results of oedometer tests for the alluvial clays along the proposed embankment alignment, and the idealised geotechnical model presented in Section 6 of this paper, a consolidation settlement of 110 mm over 150 days was calculated for the maximum height of embankment (10 m). An immediate settlement of 140 mm was also estimated from simple elastic theory, based on elastic moduli values for the foundation strata that were derived from approximate correlations with SPT N values, and conceptualising the embankment foundation surface as a flexible rectangular loaded area. Thus, a maximum total settlement of 250 mm was suggested for the embankment construction. It was recommended to construct the embankment a suitable time period (say several months) prior to the start of bridge construction, such that the calculated remaining consolidation settlement at the end of that time period could be accommodated by the bridge construction activities. Thus, ground improvement techniques such as provision of drains to the consolidating strata and/or placement of large additional fill surcharge were not considered necessary.

The proposed embankment should be constructed using a clean gravelly material similar to a quarry waste, that would be strong, relatively incompressible and erosion resistant. The material to be excavated from the western approach cutting was considered suitable for this purpose. Placement and compaction of the embankment fill material in layers to achieve a specified compaction performance was recommended. The proposed embankment side slopes of 1 vertical to 3 horizontal were expected to have an acceptably low risk of slope instability if the foregoing procedures were followed, since for a friction angle of only 35°, the approximate factor of safety is still 2.1.

### 7.5 Eastern Abutment

The geotechnical issues for the proposed eastern abutment were the type of footing and abutment, the means of carrying the applied vertical and lateral loads, and the interaction between the abutment and embankment. The eastern abutment will be the launching abutment during bridge construction and the fixed abutment in the final structure. This abutment must therefore be able to carry the vertical loads from the end bridge span during launching and after construction, as well as longitudinal launching reactions and longitudinal forces due to earthquake effects.

A cellular raft supported by piles founded on the weathered marl was considered to be the most appropriate abutment configuration for the site conditions. Piles were required to support the cellular raft base, due to the low strength and relatively high compressibility of the near surface clay soils in the area of the abutment. Driven preformed piles, as per the proposed internal bridge pier footings, were considered to be the most suitable pile type. The piles would carry a portion of the lateral loading applied to the abutment as well as vertical loads, and for this reason they were to be raked. Due to the large size and mass of a cellular raft, abutment stability against lateral forces could be at least partly achieved by gravity retaining action. For horizontal forces from the bridge acting eastwards, the passive pressure mobilised in the embankment fill behind the abutment, as a result of lateral deflection of the cellular raft, would also contribute to lateral abutment capacity. For horizontal forces from the bridge or the embankment fill that act westwards, "dead man" anchors connected to the cellular raft would increase the lateral capacity over that offered by gravity retaining wall action and lateral pile capacity alone.

The load actions and magnitudes of loading that will act on the proposed piled cellular raft abutment will depend, among other factors, on the interaction between the abutment and the adjoining embankment. The interaction, in turn, depends on the relative sequence of construction for the abutment and embankment. Of particular concern is any part of the abutment footings or superstructure that is constructed prior to the completion of consolidation settlement of the foundation soils under embankment surcharge. Such elements must then be designed for additional horizontal forces due to lateral spreading of the embankment fill and foundation soils as a result of vertical settlement and self weight of the fill, as well as vertical drag down forces due to vertical settlement of the embankment fill and foundation soils. The solution adopted in the detailed bridge design was to construct the fill embankment, then at the appropriate construction stage, locally excavate at the proposed abutment location in order to allow the piles to be driven and the abutment to be built, before backfilling around the abutment.

## 8. CONCLUSIONS AND RECOMMENDATIONS

A geotechnical investigation for a proposed replacement bridge at Blanchetown was undertaken. The investigation involved a review of the geology of the area, a program of soil drilling, rock coring, field and laboratory testing, the development of a geotechnical model for the site, a pile test, and geotechnical design recommendations and construction advice for the bridge approaches, abutments and internal piers. The key conclusions and recommendations based on the results of the geotechnical investigation were as follows:

- The limestone at the location of the proposed western approach cutting was expected to be rippable by dozer, and suitable for use as fill for the proposed eastern approach embankment.
- A spread footing founded near the top of the weathered limestone cliff was appropriate for the proposed western abutment, with the slope stability of the nearby cliff face expected to be adequate despite the abutment surcharge.
- Driven preformed piles founded on or within the weak marl rock at depth were required for the pier footings, with a design geotechnical strength of 2000 kN suggested for a 450 mm size pile driven to the marl surface.
- The maximum total settlement of the foundation strata to the proposed eastern approach embankment was calculated as 250 mm, with a maximum time period for practical consolidation of 150 days. This settlement behaviour was expected to be accommodated by constructing the embankment sufficiently early in relation to the bridge construction, to allow for most or all of the predicted settlement to occur prior to bridge construction.
- A piled cellular raft with "dead man" anchors extending back into the approach embankment fill was recommended for the eastern bridge abutment, to carry all of the vertical and horizontal loads imposed during and after bridge construction. The interaction between the abutment and embankment will determine some of the loads that the abutment will carry, with the nature of the interaction being in turn dependent on the relative construction sequence for abutment and embankment.

## 9. ACKNOWLEDGMENTS

The author gratefully acknowledges the permission of then South Australian Department of Transport to publish paper, and in particular wishes to thank project personnel Richard Herraman, Geotechnical Engineer, and Mr Ror Washyn, Project Manager. The views expressed in this pa are those of the author and PPK Environment Infrastructure Pty Ltd, and do not necessarily reflect views of the Department. Professor Ian Donald, and Dr P Mitchell of PPK, are thanked for their review of the paper.

## 10. REFERENCES

1. Geological Survey of South Australia. "Renmai Sheet SI 54-10, Department of Mines, 1971.
2. Geological Survey of South Australia (LW Parkin, "Handbook of South Australian Geology". Departm of Mines, 1969.
3. International Society for Rock Mechanics (ISR Commission on Testing Methods. "Sugges Methodfor Determining Point Load Strength", Int Rock Mech. Min. Sci & Geomech. Abstr., Vol 22, No pp 51-60.
4. Franklin JA, Broch E and Walton G. "Logging Mechanical Character of Rock". Transactions of Institution of Mining and Metallurgy, Volume Section A, pages A1-A9, 1971.