Driven pile installations on Australia’s longest bridge
Jared Haube for Piling & Deep Foundations

The floodplain bridge over the Macleay River is the longest bridge in Australia, spanning 3.2 kilometres. Ahead of Piling and Deep Foundations 2013, I had the opportunity to speak with Paul Hewitt, Technical Executive at Parsons Brinckerhoff, about some of the features relating to the use of driven piles.

Paul Hewitt

What were the challenges associated with the design and construction and testing of driven piles on the project?

The project site is located in the Macleay River floodplain which is subject to frequent inundation with ground water levels close to the surface. This has driven the design to maximise the use of prefabricated elements such as steel piles to minimise site work.

Due to the length of the bridge, it was important to optimise the design and construction methodology, simplify typical details, enhance the speed of construction and reduce risks. Ground conditions are variable along the length of the bridge comprising compressible soils up to 50 metres deep, overlying weathered rock.

This was reflected in the selection of two different piling methodologies in the floodplain and in the river section. The floodplain section was founded on 328 steel driven piles, driven through alluvial sediments and weathered rock to nominal refusal on lower rock.

For the driven piles, initial challenges related to the driveability through thick layers of dense gravel overlying weathered rock, the need to achieve a design toe level in the lower rock, and achieving agreement between design and as-built toe levels.

The owner, contractor and their consultants addressed these challenges through careful, high quality geotechnical investigation, a very rigorous pile installation program and the owner’s peer review and verification program.

What did you do to mitigate potential risks of pile installation and supplement long term performance?

Extensive ground investigations were made before tender and during the detailed phase of the design, which were used to develop a more comprehensive ground model which mitigated installation risks, and realised significant efficiencies in the substructure design.

The project utilised the sonic drilling method to provide some production advantages and combined that with conventional core drilling in rock to better define the rock properties for the driven piles.
It was very important to demonstrate that the as-built toe levels of the driven piles were founding within rock. During the design and review process the definition of rock was refined to make it more realistic and specific, so it was tailored towards the project. We also carried out independent assessment of design toe-levels and demonstrated good agreement with as-built pile toe levels.

Risks associated with large deep excavations in acid sulphate soils with a high water table in the floodplain were overcome by eliminating pile caps and embedding the piles into the pier stems.

Can you talk about recommendations and interpretation of pile integrity testing results?

Geotechnical strength was assessed using dynamic load testing supported by signal matching. Pile Driving Analyser (PDA) techniques were used to verify the pile integrity and mobilised capacity.

The as-built founding toe levels were then assessed and reviewed by confirming rock levels, pile refusal levels and PDA data to satisfy performance criteria. By correlation with a single CAPWAP analysis in each pile group, it was also possible to infer capacities for other piles in the group by using a pile driving monitoring device - this increased construction confidence and expedited pile acceptance across the project.

At some locations results gave an indication that the pile toes might be damaged because we weren’t able to get a reflection from those pile toes. We extracted the piles, installed replacements and re-tested them to demonstrate that the integrity of the piles had not been compromised.

Back-analyses of design parameters from the testing will provide positive benefits for future projects, and helped both the owner and the contractor’s design and construction team obtain an efficient design through high level investigation and interpretation.

What was the quality of collaboration with the piling contractor?

Parsons Brinckerhoff was a part of the Roads and Maritime Services (RMS) Peer Review Team – our role was to ensure the design was reliable and addressed RMS quality requirements. Abigroup self-performed the piling work.

Quality results were achieved by a collaborative approach between the structural and geotechnical engineers, with the latter providing input on detailed foundation assessment, durability, validation testing, and construction monitoring.

Through teamwork with the contractor, their design team and the owner, we were able to identify risks and opportunities, resolve technical issues in a practical manner, and help achieve early project delivery. This was due in part to piling efficiencies identified and realised by the design and construct team, RMS and their advisors.
Aspects of driven piles on Macleay River and floodplain bridge

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SUMMARY: The bridge over the Macleay River and floodplain (MRFB) will be 3.2 km long on completion, which will make it the longest bridge in Australia. The floodplain section was founded on a combination of steel tubular piled footings 750 mm and 825 mm in diameter, designed to be driven through alluvial sediments and weak rock to nominal refusal on ‘sound and unyielding rock’. Dynamic load testing using the pile driving analyser (PDA) was used to verify the pile integrity and mobilised capacity. To address the RTA’s Scope of Works and Technical Criteria requirements and help select pile toe levels, boreholes were drilled before pile driving. Sonic drilling was used for some of the boreholes. Although fast, this technique presented difficulties in assessing design toe levels. Variations in rock level, degree of weathering and strength, and dense gravels above rock added additional risks to driven pile design and construction.

This paper describes the following aspects that had to be addressed in the requirements for reliable bridge foundations on the MRFB:

• design and construction review challenges of driven piles
• measures to mitigate the potential risks of pile installation and long-term performance
• driven piles capacity in weak rock based on PDA results
• recommendations on interpretation of pile integrity testing results, such as the BTA value
• limitations of pile testing methods
• recommended design and performance criteria of driven piles
• comparison between concept design, detailed design and construction
• recommendations for RMS technical direction on pile testing
• lessons learnt.

Keywords: driven tubular steel piles, weak rock, PDA, sonic drilling, plugging, and driveability.

1. INTRODUCTION

This paper discusses geotechnical aspects associated with design and construction of driven piles on the floodplain section of Macleay River and Floodplain Bridge (MRFB). It presents the authors views from the perspective of peer review of geotechnical aspects of a particular element of the MRFB and describes the approaches used in this review function to assist all parties in the MRFB team to deliver a successful project. Due to the nature of the review role, only a limited perspective of the overall project can be provided in this paper.

Although part of the Kempsey bypass, the bridge over the Macleay River and Frogmore Floodplain forms a separate design and construct project. The contract was awarded to Abigroup in December 2010 and permanent work began in July 2011. The bridge is being constructed concurrently with the Kempsey Bypass Alliance works (by others) to the south. When the bridge is built its overall length of 3.2 km will make it the longest bridge in Australia, extending over the Frogmore Floodplain and crossing the Macleay River east of Frederickton.

Parties in the MRFB project include Abigroup Contractors with its consultants on the design and construct (D&C) team. Roads and Maritime Services (RMS), which was then the Roads and Traffic Authority (RTA), provided design reviews to ensure the bridge design conformed to the RTA’s Scope of Works and Technical Criteria (SWTC). The Principal, RMS, and consultant Parsons Brinckerhoff developed the route and concept design. Parsons Brinckerhoff also provided RMS with ongoing peer review services for the Kempsey Bypass advising on geotechnical aspects of alliance and associated D&C contracts over 14.5 km of the project, which included the 3.2 km MRFB.
2. BACKGROUND

2.1 Project history

In September 2001, the then RTA began investigating a route for a four-lane divided road for the Pacific Highway from Kempsey to Eungai. Preliminary route options and a description of the environmental, social and engineering aspects of each route were displayed for public comment in October 2002. The final preferred route (Eastern option) is a bypass to the east of Kempsey and Frederickton and was announced in July 2004 (Figure 1). Design development and further field investigations took place during 2005.

The reference design prepared by Parsons Brinckerhoff proposed a combination of embankments and bridges founded on driven and bored piles in the floodplain areas and raised concerns about whether they could be driven through dense floodplain gravels that occurred in some places. Multiple bridging configurations were investigated to minimise afflux/flood effects upstream of the highway. Minimising the number of bridge to embankment transitions was recognised as being beneficial as it would reduce the future maintenance of the upgrade.

The original EIS concept involved three sets of twin bridges with a total length of 2.15 km (Fig. 3). During detailed concept design development, the Kempsey Bypass Alliance reduced this to two single bridge structures by deleting the short embankment between the floodplain bridges and combining each of the pair of twin bridges to one single structure. These changes were approved based on a whole of life economic analysis which reduced the overall project cost. The total length of the proposed bridging then became 2.45 km (Fig. 2). During post tender negotiations with the contractor and further economic analysis and costing (including earlier completion), approval was given for elimination of the central embankment and the construction of a single 3.2 km bridge. As this omitted the risk of potentially adverse interaction effects due to ground treatment of embankments in soft ground at bridge pile locations, it accelerated the construction program.

Figure 1 Route study in Kempsey to Eungai concept design showing depth of compressible soil (Source: Parsons Brinckerhoff 2004)

Figure 2 Route plan showing preferred route – 2.45 km bridge length (RTA 2010)
2.2  Site description

The project is in the local government area of Kempsey and lies on the floodplain of the Macleay River valley about 400 km north of Sydney and about 55 km north of Port Macquarie. The site is about 9 km from the nearest coastline between Hat Head and Crescent Head.

The landscape is characterised by the floodplain, which has been largely cleared of natural vegetation. The project area is generally flat and used mainly for grazing livestock with infrequent houses.

3.  FLOODPLAIN SECTION

The 2790 m long floodplain section, which is the subject of this paper, comprises 81 spans, supported on 324 driven piles. Typical pile foundation detail is shown in Figure 4. The foundation at Abutment A (see Figure 2) comprises four driven, steel tubular piles, with a composite reinforced concrete plug for the top 15 m of the piles that is connected to the abutment headstock.

The foundation from Pier 1 to Pier 81 comprises two driven, steel tubular piles; 750 mm and 825 mm diameter, connected directly into each of the two pier columns (see Figure 4). The foundation for the river section from Pier 82 to Pier 93 consists of four or six bored piles and a single pile cap supporting the two columns at each pier location. The foundation at Abutment B comprises four reinforced concrete bored piles connected to the abutment headstock. Total piling length was about 11 km.

Figure 4 Issued for construction (IFC) pile foundation section (Arup drawing no MRFB-SBR-DRG-2020)
4. GEOTECHNICAL DESIGN

4.1 Ground conditions
Subsurface conditions at the bridge site comprise compressible, estuarine and alluvial soils up to 50 m deep, overlying weathered rock. The ground conditions, from youngest to oldest, generally comprise:

- very soft to firm Holocene clay
- firm to very stiff Pleistocene clay
- medium dense to dense Old Alluvium gravel (absent in some locations),
- residual soils.

They also include the Kempsey beds, which are mainly siltstones often with interbedded or interlaminated sandstones, from extremely low to very high strength. The degree of weathering in the bedrock tends to decrease with depth, and the weathering ranges from extremely weathered at the top of bedrock to slightly weathered at depth. The bedrock dip is approximately 20° north to north-east. The bedrock is relatively deep and ranges from 12 m to 50 m, with an average depth of 30 m below existing ground level.

Scouring of the rock mass is evident beneath much of the floodplain and includes several deep palaeochannels where scouring has reduced rock level to RL -40m or deeper. For example, a palaeochannel exists from Pier 51 to 56, with steeply dipping rock around Pier 56/57.

The inferred geotechnical profile between Abutment A to Pier 82 is shown in Figure 5.

4.2 Geotechnical investigations
Extensive ground investigations were made before the tender and during the detailed phase of the design in accordance with SWTC requirements. Additional tests and technology that would maximise productivity were used to produce more detailed information critical for the design. The additional tests were intended to:

- fill in gaps in the existing data and provide a more detailed model
- refine geotechnical models where necessary to include additional geotechnical investigation data
- provide larger and higher quality samples to minimise effects of sampling disturbance
- measure in situ strength and stiffness.

One of the key ground investigation tests used was the Seismic Dilatometer Marchetti Test (SDMT) to assess in situ the small strain modulus that is important for soil-structure interaction under seismic loading. SDMT combines the standard flat dilatometer (DMT) with a seismic module consisting of a probe outfitted with two sensors, spaced at 0.5 m, for measuring the shear wave velocity Vs, used to assess the small strain shear modulus $G_0$.

The detailed investigation undertaken by the D&C team was used to develop a more comprehensive ground model which improved the quality of the design, and realised significant efficiencies in the substructure design.

4.3 Sonic drilling
To meet the site investigation requirements of the SWTC and AS5100, at least one borehole was required at each abutment and pier location. Given the need to complete the project within 104 weeks, conventional rotary coring methods presented a challenge in terms of rate of production. Consequently, it was decided to supplement conventional methods with sonic drilling.

Sonic drilling offered rapid formation penetration through the substantial gravel layer; rotary drilling would be switched back to continue coring into bedrock. It increased production and reduced fieldwork time, and produced a continuous core which allowed better definition of the gravel/rock boundary.

At some locations, sonic drilling was continued beyond the gravel layer into the bedrock. Figure 6 shows an example where the sonic drilling was able to detect the boundary of weathered Old Alluvium and rock. While sonic drilling offered advantages in terms of production rate it presents some limitations in understanding rock mass characteristics due to vibration induced mechanical fractures. Disturbance to weathered rock required extensive interpretation to achieve accurate rock classification.
To mitigate the cutting and vibratory impact on soil and rock samples, limited conventional core drilling methods were used to assess the sample disturbance and supplement the sonic drilling data. Figure 6 shows a comparison of cored borehole and sonic drilling borehole photos. Sonic drilling assisted the project team to achieve desired project outcomes on this project and was well suited to defining the extent of the gravel, which would have been difficult with traditional rotary coring. When used in conjunction with rotary coring methods it is an effective method to sample materials that would otherwise be lost using conventional drilling practice.

Figure 5 Floodplain bridge geological long section (Arup 2011b)
5. FLOODPLAIN PILES

5.1 Driven pile design

The floodplain section was founded on a combination of steel tubular piled footings 750 mm and 825 mm in diameter, designed to be driven through alluvial sediments and weak rock to nominal refusal on ‘sound and unyielding rock’. Geotechnical strength was assessed using dynamic load testing to failure supported by signal matching. PDA was used to verify the pile integrity and mobilised capacity (see Figure 7). The adopted geotechnical strength reduction factor was 0.78 according to the piling code AS2159. Static load testing, which would have allowed a higher geotechnical strength reduction factor to be used, was not required.

To assess the lateral capacity of the piles a full pier (two pile group of two connected to the columns connected via the headstock) was modelled using a 3-D finite element modelling program.

Driven piles were designed using Imperial College Pile (ICP) methods (Jardine et al 2005), which were developed from detailed field investigations, laboratory testing, analysis and verification against a large database of piles in sand and clay. The verification studies showed this design approach was more reliable than traditional ones and could provide opportunities to reduce costs or improve foundation performance. Although it is widely used for driven piles in sands and clays, there is limited information of its use for rocks, uncertainty over whether a rock plug can be formed and how much penetration is needed to form a rock plug. The peer reviewer examined the suitability of ICP for driven piles in weak rock at MRFB, and cross-checked suitability against more usual methods such as Tomlinson & Woodward (2008), and FHWA (1998).

Cone penetration tests (CPTs) typically refused above the gravel layer. The designer assumed cone resistance $q_c$ values for gravels, weathered rock and founding rock of 30 MPa, 50 MPa and 100 MPa respectively. With rocks assumed as gravel, unit shaft resistance and unit base resistance were then calculated using the ICP method. The ultimate shaft resistance and base resistance of weathered rock were around 700 kPa and 17 MPa respectively.

The vertical geotechnical capacity of the piles was achieved by a combination of end bearing and shaft resistance. The lateral capacity of the piles is governed by the structural element, as they behave as long piles.

The maximum ultimate vertical load per pile is 8007 kN. Twenty-five per cent of the driven piles were dynamically tested using PDA to verify the pile capacity.

The piles were designed to be driven to nominal refusal (13 mm/10 blows, or 77 blows/m) on ‘sound and unyielding rock’, which was not precisely defined in the SWTC at the time of contract award. It was later defined as distinctly weathered and low to medium strength or better rock (UCS>5 MPa). Driving into relatively stronger rock (UCS>5 MPa) presented an increased risk of pile damage due to hard driving. The acceptance of the founding level was then determined by the confirmation of bedrock level, pile refusal level and PDA data.
5.2 Design development

The tender design (resubmitted at the 15% submission) was for four steel piles 1200 mm in diameter at every pier location, in a single row alignment. The top of the piles and the pile cap would be connected by welding the reinforcement to the inside for the tube and casting a 700 mm concrete plug in the pile, to embed the full length of the welded connection.

The design then progressed to the 85% stage and the revised foundation evolved to driven steel piles 750 mm and 825 mm in diameter (Fig 4), connected directly to the pier columns with a reinforced concrete plug (12–15 m long), but without the use of pile caps (Gentile & Canceri 2011).

In the floodplain there were 324 driven steel tube piles 16 mm thick, 750–825 mm in diameter and between 20 m and 47 m long, driven to nominal refusal into sound and unyielding rock in accordance with the SWTC. There are two pier columns per headstock and two piles per pier column, with these columns varying in height from about 4–6m. The upper 12–15 m of the piles were filled with reinforced concrete for connection and load transfer from the columns. The toe of the piles was extended using a 25 mm thick tubular section welded to the 16mm thick tubular pile toe to form a cutting shoe to strengthen the pile so that high driving stresses could be accepted.

As the floodplain’s water table is very close to ground level, during the design development phase it was determined the pile caps from P1 to P81 could be eliminated by connecting the piles directly to the reinforced concrete columns. This decision was made to speed up construction, as excavations for pile caps would have had to be pumped out continuously and disposing of the excavated acid sulfate soils would have been problematic.

Significant efficiencies were gained from deleting the pile caps, which required additional precision on site, with much tighter construction tolerances for the 750 mm and 825 mm diameter piles to align within the columns than would conventionally be the case with pile caps. To achieve the very tight tolerances for the location of the driven steel piles, a specially designed pile frame was fabricated as shown in Figure 8.

Figure 7 Pile testing

Figure 8 Crane and frame for pile installation
6. CONSTRUCTION ASPECTS

This section describes the equipment and methods used during pile driving operations. It discusses construction aspects and includes a general overview of impact hammers, how piles were installed, and discusses results of regular monitoring and audits to check that construction complied with the design requirements in the SWTC.

6.1 Construction equipment and methods

Impact hammers were used to drive all of the floodplain piles. The impact hammer consisted of a heavy ram weight that is raised mechanically or hydraulically to some height (termed ‘stroke’) and dropped onto the head of the pile. During impact, the kinetic energy of the falling ram is transferred to the pile, causing the pile to penetrate the ground. With such hammer energy, steel tubular piles were driven to refusal (13 mm/10 blows) in accordance with the RMS specification B54.

Many different pile-driving hammers are commercially available; the major distinction between them is how the ram is raised and how it impacts the pile. The size of the hammer is characterised by its maximum potential energy, referred to as the ‘rated energy’. The rated energy can be expressed as the product of the hammer weight and the maximum stroke. A 14-tonne hammer (Junntan HHK14s) and 1.5-m stroke was used to test the pile at the end of driving. The hammer efficiency generally ranged from 85% to 95%. Driving stresses were limited to 90% of the yield stress of the steel.

6.2 Toe levels

During pile driving, some piles reached nominal refusal above design toe levels and some of the piles terminated in extremely low to very low strength rock. Concerns on the pile capacity (plug formed or not) and long-term performance (total and differential settlements) were examined. At Pier 11 for example, all the four piles achieved nominal refusal about 5.3–6.2 m above the respective design toe level. The actual pile was 20 m long compared with a design length of 25–26 m. Figure 9 shows the borehole core photo. On further review of sonic drilling data and rock mass characteristics, a reassessment of design toe levels provided better agreement (usually within 1 m) with pile driving records.

![Figure 9 Borehole core photos of Pier 11](image)

The CAPWAP analysis on P11B showed that 9860 kN of shaft resistance (mainly in rock) and 1200 kN toe resistance were mobilised with toe movement of 1.5 mm. With a value of geotechnical strength reduction factor, \( \phi_g \) of 0.78, the design geotechnical strength was achieved. Assuming full section area (0.44 m\(^2\)), the mobilised unit base resistance \( f_b \) is about 2.7 MPa. Using \( f_b = 3 \times \text{UCS} \) (Stevens et al., 1982), the back calculated rock strength is about 0.9 MPa. Under this non-fully mobilized condition, if only net cross-section area (0.11 m\(^2\)) is adopted, that is, unplugged, the back calculated rock strength would be about 25 MPa, which seems too high for low to medium strength rock with \( \text{Is}(50) \) of 0.3–0.6 MPa. This implies that a rock plug was formed.
6.3 Driveability

Although driveability was analysed to assess the toe levels, sloping rock surfaces and medium to high strength rock bands made it difficult to achieve exact agreement between design and as-built toe levels. Sonic drilling results made it challenging to accurately characterise rock mass strength, which added additional complexity to pile installation. Apparent penetration, as observed at the head of the pile during driving, does not necessarily mean penetration of the pile toe is achieved. In medium-strength rock, the risk that steel may buckle or split increased (Figure 10). Two piles P56B and P56D at Pier 56 were suspected to be damaged. The CAPWAP analysis at the end of driving indicated a loss of impedance in the bottom 5 m of the pile, which suggested damage had occurred in each of the bottom five 1 m segments. A PDA Integrity Factor BTA value of 61% also suggested damage at 44.7 m below the gauge location. Figure 11 shows the borehole logs and design versus actual toe levels.

The two damaged tubes were extracted without removing the soil and rock inside.

New piles were driven at the locations of P56B and P56D. Subsequent use of PDA confirmed the replacement piles were sound and achieved the design capacity. Borehole log BH11-P56-NB shows high rock strength at 47.4 m depth (RL-45.3). The replacement pile P56B stopped just above this high strength rock.

Observations from boreholes at Pier 56 and 57 indicate: from P56 to P57, in transverse direction, rock top levels are similar; longitudinally there is about 12 m level difference over 34 m distance, which gives about 1:3 slope or 18° angle by linear interpolation. It is postulated that damage may have been associated with refusal of a section of the pile shoe on the sloping rock surface (see Figure 5) and driving in medium to high strength rock to achieve the design toe level.

Figure 10 Pile damage: P56B (left) and P56D

Figure 11 Borehole logs for Pier 56: P56-NB (left) and P56-SB
6.4 Testing and interpretation

The PDA was used to record, digitise, and process the force and acceleration signals measured at the pile head. The signals used the Case Method (a simplified field procedure for estimating pile capacity) to estimate static capacity, as well as the more rigorous CAPWAP. A description of the fundamentals of dynamic testing, including CAPWAP, is presented in Design and construction of driven pile foundations (FHWA 1998).

To verify the pile capacity and integrity, 85 piles (25%) were tested using PDA. The following findings can be made from CAPWAP analyses:

- Mobilised unit shaft resistance in alluvial, estuarine, and residual soils is relatively low compared with that in weak rock.
- The ICP method generally gave reasonable estimates on shaft resistance.
- As an alternative, curve 1 (for driven steel and concrete piles in clay with \( q_c < 0.7 \) MPa and sand with \( q_c < 3.5 \) MPa) of Laboratoire de Ponts et Chaussées (LCPC) method (Figure 12) may be used. A conservative approach may ignore shaft resistance in soft clay. Shaft resistance is mainly from weak rock.
- Unit shaft resistance in weak rock was fully mobilised and ranged from 100 to 800 kPa, depending on weathering degree, rock strength and penetration. Interpreted design (mobilised) values, given in Table 1, based on peer reviewer’s back analysis of the PDA results, may be used for guide purposes.
- Ultimate unit base resistance \( (f_b) \) in weak rock may not be fully mobilised due to the small set (a few mm) when using the PDA. Mobilised unit base resistance ranges from 3 to 11 MPa; assuming \( f_b = 3q_u \), the equivalent \( q_u \) ranges from 1 to 3.7 MPa.
- It is reasonable to assume plugging was formed in weak rock (EL-L strength) if the minimum penetration was about two times pile diameter. Use the gross base area to calculate the total base resistance.

Following installation of replacement piles, the PDA test data indicated the piles P56A and P56B have adequate capacity. PDA CAPWAP analyses indicate that no reflection was detected before the pile toe. This implies the PDA detected no damage for an interval of a few pile diameters above the pile toe. It was concluded the integrity of the two piles had not been compromised.

The BTA factor for restrike was assessed according to Rausche’s (1979) Beta Method Damage Classification. The Beta Method was not considered reliable for pile damage classification because there is no experimental proof available justifying Beta Method Damage Classification (Verbeek and Middendorp, 2011).
Table 1. Interpreted Design Parameters for Open-Ended Driven Tubular Steel Piles

<table>
<thead>
<tr>
<th>Soil/rock¹</th>
<th>s-f clay/ vl-l sand</th>
<th>st-h clay/ md-d sand</th>
<th>EL rock (Is(50)&lt;0.03)</th>
<th>“Weak” rock (4)</th>
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<tr>
<td></td>
<td>20</td>
<td>30–60 (40)</td>
<td>100–200</td>
<td>200–400</td>
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<td></td>
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<td>3–6</td>
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<td>8–15</td>
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Notes:

1. Consistency/ strength: s-f = soft to firm; vl-l = very loose to loose; st-h = stiff to hard; md-d = medium-dense to dense; EL = extremely low strength; VL = very low strength; L = low strength; Is(50) – point load test strength (MPa)

2. Plug is formed if minimum penetration of twice the pile diameter is achieved; gross cross-section should be used.

3. The above values are based on mobilised values back calculated from PDA results and are for preliminary design purposes only. The actual values under static load conditions may be different.

4. For the purpose of this paper, the term “weak” is partly defined by a lower-bound uniaxial compressive strength of intact specimen of 0.6MPa (Gannon et al, 1999) – corresponds approximately to Is(50)>0.03MPa.

At time of writing this paper the project was well advanced as shown in Figure 13, due in part to piling efficiencies identified and realised by the design and construct team, RMS and their advisors.

![Figure 13 Work progressing on the Macleay River and floodplain bridge, June 2012 (RMS)]](image-url)
7. CLOSING

The success of this project has provided some very positive benefits and lessons learnt for future projects for RMS, its contractors and consultants. By working together with the D&C team, a common understanding of the project objectives was created, potential risks identified, opportunities realised, and valuable infrastructure delivered to the community. Particular key thoughts on aspects of the piling work on MRFB include:

- The project successfully demonstrates the importance of a comprehensive geotechnical investigation to characterise ground conditions before the piles are designed. Some of the efficiencies in design perhaps may not have been realised without the level of investigation and interpretation that was applied in this case.
- High quality investigation data and interpretation, and progressive refinement during design development, provided sufficient information to allow design approvals which facilitated reduction of pile diameter from 1200mm to 750/825mm.
- The term ‘sound and unyielding rock’ to define the founding material was not defined in the SWTC at the time of contract award. As this terminology is not explicitly defined in geotechnical engineering practice, it is difficult to apply in design. It would be more appropriate to define the founding material as low to medium strength rock (per an agreed standard or code) or an equivalent rock class that considers rock strength, defect spacing and weak seams, and addresses constructability.
- The current practice of using a minimum of one borehole at each pier and abutment location is considered to be prudent. Sonic drilling has been observed to provide some production advantages particularly if a substantial layer of gravel is present. It also arguably provides increased recovery of materials that may be lost using conventional methods. However, we suggest that it be used in conjunction with conventional methods where rock core recovery is required for detailed rock mass assessment.
- In this project driven tubular steel piles can penetrate 2–3 m into extremely low to low strength rock, and refused on low to medium strength rock, although the amount of penetration may ultimately be governed by shaft resistance if there is a thick layer of dense gravel or lower strength rock.
- There is increased risk of pile toe buckling for pile sections driven into medium to high strength rock. With PDA testing, nominal refusal criteria and accurate ground characterisation, such as that carried out on this project, these issues can be addressed.
- PDA testing was valuable in providing increased confidence that the design complied with the requirements of SWTC, addressing pile integrity, and providing data for verification of design.
- The Beta Method is not considered reliable for pile damage classification because there is no experimental proof available justifying Beta Method Damage Classification.
- Field data from this project suggests plugging can be formed if minimum penetration into weak rock of twice the pile diameter is achieved.

8. ACKNOWLEDGEMENTS

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9. REFERENCES


