

A COMPREHENSIVE GEOTECHNICAL INVESTIGATION FOR THE NEW BRIDGE OVER THE CLARENCE RIVER AT HARWOOD

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ABSTRACT

The new bridge over the Clarence River at Harwood forms an integral part of the Pacific Highway upgrade in northern NSW and is currently under construction by the Acciona-Ferrovial Harwood Joint Venture (AFHJV). The proposed bridge extends 1.5km between abutments, spans the Clarence River with a water span of approximately 560m and is founded on 2m to 2.4m diameter driven hollow steel tubular piles to over 60m depth in some areas.

The underlying geology of the site consists of 30 to 40m of soft estuarine silts, clays and loose to medium dense sands underlain by saturated basal sands, gravel and cobbles up to 25m in thickness. Historical ground investigation information has thus far failed to provide confidence in the characterisation and engineering properties of this basal gravel and cobble layer due to limitations with conventional SPT, CPT and drilling techniques. The focus of this paper is primarily on characterisation of the basal sand and gravel units encountered in the River and North portions which largely drove the selection of innovative ground investigation techniques and methodologies.

A comprehensive ground investigation (GI) scope was proposed to support the bridge detailed design phase undertaken in parallel with the field works. This paper presents a perspective on the challenges of developing a ground model within this complex geological sequence and how this was addressed through a diverse, state-of-art GI campaign. It presents a methodology to derive ground profiles and soil parameters from advanced geotechnical testing and provides comparison of these innovative techniques against industry standard approaches.

1 INTRODUCTION

1.1 PROJECT DESCRIPTION

The new bridge over the Clarence River at Harwood forms an integral part of the Pacific Highway upgrade in northern NSW and is currently under construction by the Acciona-Ferrovial Harwood Joint Venture (AFHJV). The proposed bridge extends 1.5km between abutments, spans the Clarence River with a water span of approximately 560m and is intended to replace the existing lifting deck bridge located 30m upstream of the proposed alignment. The base of the proposed bridge deck is to be positioned at a level coincident with the top of the existing bridge lifting towers, some 30m above mean tidal water level. The bridge deck is supported by 35 piers at 36m to 45m spacing, each founded on two 2.4m diameter (land) or five 2.0m diameter (river) hollow tube steel driven piles. The bridge design evolved in parallel with the ground investigation (GI) which imposed further logistical constraints and prioritisation on the investigation program.

2 SITE GEOLOGY

2.1 GEOLOGICAL SETTING

The project site is located within the Clarence-Moreton Basin, an extensive on-shore Mesozoic sedimentary basin in northeast New South Wales and southeast Queensland. The basin comprises a sequence of fluvial sediments that rest on Palaeozoic basement of the New England Fold Belt. The site is situated within the southern region of the Clarence-Moreton Basin and is underlain by the bedrock of the Bundamba Group.

The portion of the Clarence River at Harwood is influenced by tidal movements and thus the site geology has characteristics of both a meandering river floodplain as well as estuarine environments. A deep paleochannel has been incised into the surface of the bedrock and is in-filled with Pleistocene age fluvial and estuarine sediments. The Pleistocene sediments have been further incised by a paleochannel and in-filled with younger Holocene age sands, silts and clays. The site is thus underlain by a complex sequence of fluvial and estuarine deposits of Holocene to Pleistocene age with soil depths ranging from 30m to over 80m in soil depth.

2.2 HISTORICAL GROUND INFORMATION

Given the long history of the Pacific Highway Upgrade project, ground investigation data at the site is available from historical GIs from 1962 through to 2016. Available data ranged from test pits and conventional boreholes to more advanced ground soundings such as cone penetration tests (CPT) and seismic dilatometers (sDMT).

Of interest for developing a suitable GI scope was the reported constraints regarding the deepest soil layer in the sequence. This material was summarised as medium dense to very dense sandy gravels and cobbles overlying bedrock. This basal sand and gravel layer was encountered in thicknesses from only a few metres at the midspan of the Clarence river to over 25m in thickness at the Northern abutment. It was noted that the depth and nature of the deposit resulted in poor sample recovery and therefore; no laboratory test data was available and significant uncertainty remained regarding the mechanical properties of the materials. The material was not encountered south of the Clarence River providing insight into the location and extent of the paleochannel incision and high energy flow environment required to deposit material of this nature.

2.3 SIMILARITIES TO MACLEAY RIVER FLOODPLAIN BRIDGE

Supplementing site specific ground information, prior experience from the Macleay River floodplain bridge, part of the Kempsey Bypass Project (2011). This experience identified similarities in geological conditions between the two sites where at both locations, a deep gravel and cobble dominant unit was encountered logged as medium dense to very dense. However, key field observations such as drilling resistance and observations of pile driving behaviour at the Macleay River site suggested the material performed as a loose formation under pile driving conditions, providing a contrast of information to that presented on borehole logs.

2.4 SUMMARY OF GROUND CONDITIONS

Ground conditions varied across the site however were generally distinct between the South, North and River crossing locations. Soil depth generally increased towards the north with average soil depths of 32m, 38m and 56m for the South, River and North locations, respectively. The focus of this paper is primarily on the basal sand and gravel units encountered in the River and North portions, which largely drove the selection of innovative ground investigation techniques and methodologies.

The underlying geology consists of 30 to 40m of soft estuarine silts and clays and loose to medium dense sands of Holocene to Pleistocene age. This is underlain by a layer of saturated basal sands, gravel and cobbles up to 25m in thickness with gravel grains and cobbles comprised of rounded high strength meta-siltstone.

The final interpreted geological long section is presented in Figure 1 (Arup, 2017) with simplified geological units descriptions listed in Table 1 to provide context for the remainder of this paper.

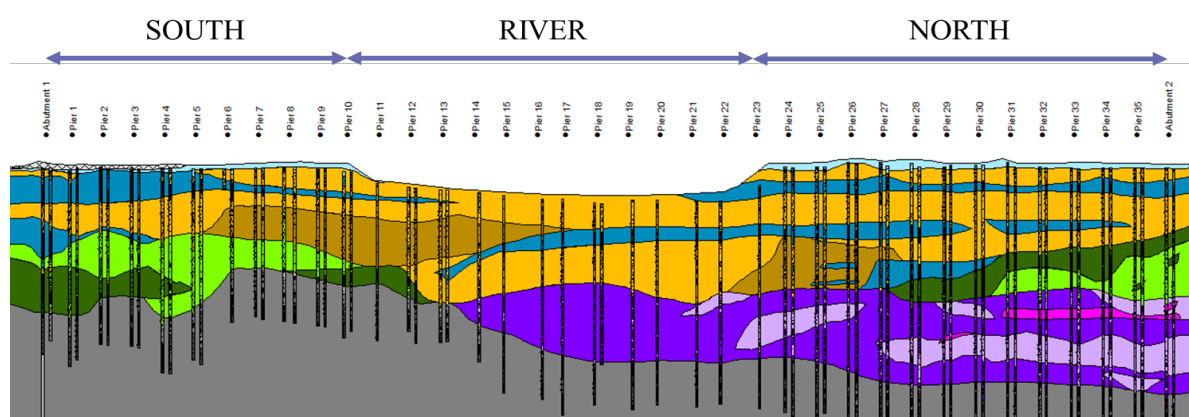
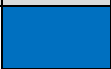










Figure 1: Final interpreted geological long section exaggerated by five times in vertical orientation (Arup, 2017)

Table 1: Summary of geological units at the site in relation to Figure 1 (Arup, 2017)

Geological sequence	Colour	Unit	Material description
Holocene (Clarence River floodplain)		Estuarine Clay	Dark grey silts and clays, very soft to stiff, with shells and minor organics
		Estuarine/Alluvial Sand and Clayey Sand	Grey to dark grey sands and clayey sands, very loose to medium dense, with shells and minor organics
		Estuarine/Alluvial and Clayey Sand	As above – medium dense to dense
Pleistocene (Clarence River floodplain)		Pleistocene Clay	Grey, pale grey and orange/red brown clay (often mottled and fissured), typically stiff to very stiff
		Pleistocene Sand and Clayey Sand	Clean grey, brown and yellow/orange brown sands and dark grey estuarine sands and clayey sands with minor organics, loose to very dense
		Basal Clays	Dark grey and grey-brown clay, trace of gravel, variable consistency
		Basal Sands	Pale grey to dark grey sands, silty sands and gravelly sands, medium dense to dense
		Basal Gravels	Gravels and sandy gravels to cobbles, rounded to sub-rounded, comprised of extremely high strength meta-siltstone grains
Bundamba Group		Rock	Interbedded siltstone and sandstone

3 GROUND INVESTIGATION SCOPE

A critical design uncertainty was the suitability of the basal sand and gravel layer for pile founding capacity and drivability assessments. Ground information available at the start of design was deemed insufficient to provide confidence in engineering design for foundations within the basal layer and thus the proposed GI was critical.

3.1 BOREHOLE DRILLING TECHNIQUES

Given the well-known difficulties penetrating gravel and cobble materials via conventional drilling methods, it was proposed that two boreholes be drilled at each bridge pier (one per pile, 10m spacing) consisting of one conventional borehole and one sonic borehole. Sonic boreholes allowed for continuous large diameter (80 to 120mm) soil cores to be recovered and sampled, appropriate for the larger grain sizes of gravel and cobble materials. This method provided an improved resolution of all soil units and in many instances identified lenses and layers, which would otherwise be missed by 1.5m interval SPTs. Conventional HQ sized boreholes were drilled to provide mechanical SPT testing as well as piston and push tube samplers for granular and cohesive materials respectively. Both conventional and sonic boreholes were switched to HQ sized rock coring below soil horizons. Table 2 summarises the borehole diameters and purposes utilised on the project.

Table 2: Summary of geological units at the site in relation to Figure 1 (Arup, 2017)

Borehole Technique	Size	Casing Outer Diameter (mm)	Core Outer Diameter (mm)	Application
Conventional Rig	HQ / HW	114.3	88.9	SPT tests and piston/tube sampling. PVC casing installed for Crosshole geophysics receiver
Sonic Rig	6inch	160.0	107.0	Continuous geological profile
Sonic Rig	8inch	210.0	152.0	Continuous geological profiling and installing PVC casing for Crosshole geophysics source

Wherever possible, conventional boreholes were scheduled in advance of sonic at each pier to limit potential disturbance and impact to SPT testing from the high frequency sonic vibration. It is acknowledged that limited published data is available on the zone of influence of sonic drilling, which may adversely impact subsequent SPT, CPT

or other ground sounding test results; and may be an opportunity for further study. The paired conventional boreholes provided SPT information for design engineers estimates of relative density of granular horizons and other soil units.

3.2 GEOPHYSICAL TESTING

A key limitation of both conventional and sonic drilling methods is the inability to provide reliable information on the density or mechanical properties of gravel and cobble dominant layers. Traditional SPT tests are well known to be unsuitable, CPTs and DMTs are unable to penetrate, and the high frequency vibration of sonic drilling effectively liquefies the soil matrix making meaningful measurements of density impossible.

As such the following series of geophysical test methods were proposed to provide insight into the density, stiffness and spatial characteristics of these materials:

- Cross-hole seismic testing at five selected land-based piers in the North site. PVC casing was installed in paired conventional boreholes (65mm inner diameter PVC) and sonic boreholes (100mm inner diameter PVC) to house the seismic geophone receiver and signal source, respectively.
- Natural gamma logging (QL40-GR Probe) within select boreholes to provide a mapping of relative levels of potassium, thorium and uranium gamma radiation signals. These elements typically occur in higher concentrations within clay soils and conversely in lower concentrations in clean sands (correspondence with Alpha Geoscience). Logging was undertaken through PVC casing for all cross-hole locations and groundwater monitoring wells and prior to pulling steel drilling casing from select over-water boreholes.
- Marine geophysical suite including sub-bottom profiling (SBP), continuous marine seismic refraction (CMSR) and the mapping of riverbed levels using bathymetric survey. This study provided a sub-surface profile by density, which was correlated against borehole information and natural gamma logs to support ground model development.

4 DEVELOPING THE GEOTECHNICAL MODEL

The geotechnical model summarises the spatial distribution and engineering properties of all materials below the ground surface. It is the fundamental basis for all geotechnical analysis and design. A high quality geotechnical model, which addresses the key geotechnical aspects of a project is essential to undertaking meaningful, efficient and cost-effective design. This section summarises how the ground investigation techniques described above were used to develop key aspects of the geotechnical model, focusing in particular on the basal sands and gravels.

A preliminary set of characteristics for the basal sand and gravel layer was based on the regional geology, tender stage ground investigation and previous projects in similar geology such as Kempsey Bypass. It was theorised that this unit comprised a high energy meandering river deposition comprising predominantly gravels, and likely including lenses of sands and/or clays. It was unknown whether the material would behave as loose or dense under pile driving conditions. The ground investigation was targeted to develop a sufficiently detailed ground model, addressing the following key uncertainties:

- Stratigraphy - soil grading, heterogeneity, clast supported or matrix supported gravels;
- Engineering properties - strength and stiffness;

This geotechnical model was developed in parallel with the ongoing field investigation and to inform design of pile capacity and settlement and drivability assessments (details of these calculations are not covered in this paper).

4.1 STRATIGRAPHY

A model of the basal sand and gravel stratigraphy was developed by plotting the borehole stratigraphy, SPT 'N' values and shear wave velocity measurements in 2D and 3D. Geological interpretation was undertaken between data points to understand the distribution of ground materials.

The recovered sonic cores confirmed that the basal sand and gravel unit comprised predominantly rounded coarse gravel to cobbles, which appeared to be clast supported. Extensive lenses of saturated sands and occasional 1m to 2m thick lenses of clay were also identified as part of the soil matrix. The gravel and cobble layers were noted to vary in thickness

from a few metres in the southern portion of the river to over 20m at the northern abutment. The composition of these gravel and cobble materials was later observed to be partially metamorphosed siltstone displaying extremely high strength based on point load testing of some suitably sized cobbles where failure was not achieved. Example sonic cores containing basal gravel, sand and clay material are shown in Figure 2.

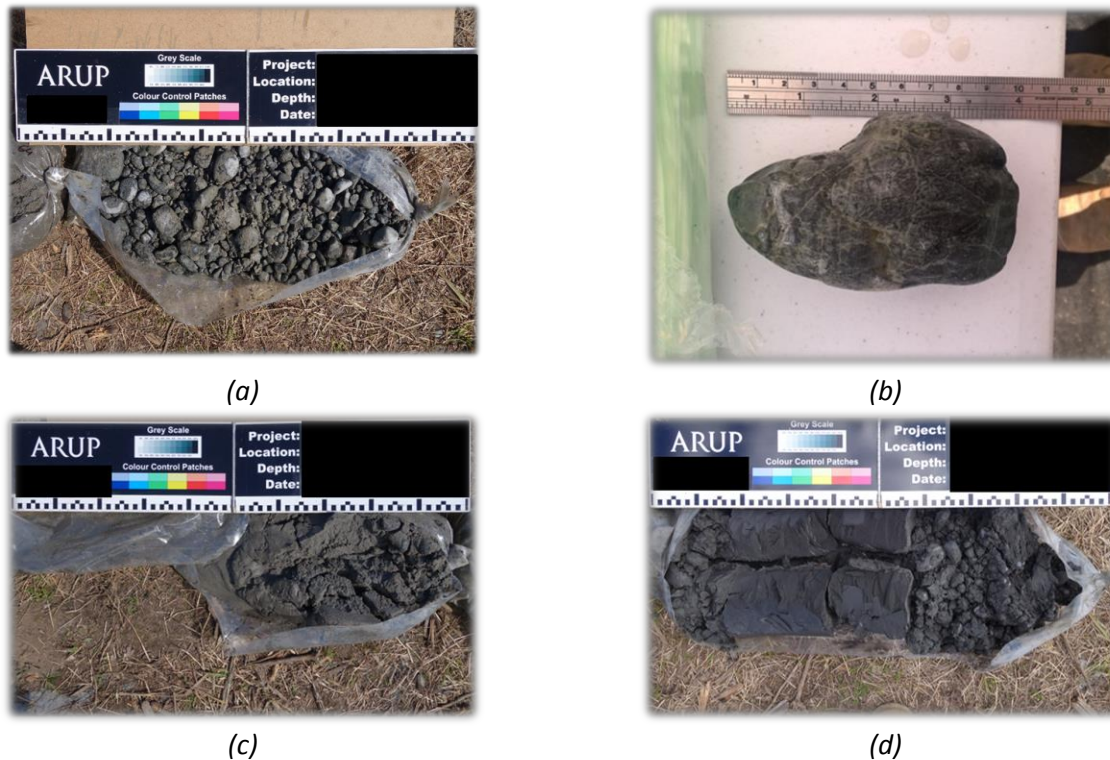


Figure 2: Example sonic soil core containing (a) gravel and cobble material, (b) recovered cobblestone, (c) sand material and (d) silty clay material

The sonic core provided a high resolution, continuous geological profile. The technique was ideal for the gravelly material which is challenging to penetrate and sample using conventional wash bore or coring techniques and ground soundings using cone penetration or dilatometer probes. The adjacent conventional wash-bored boreholes with SPT samples under-represented sand lenses and did not identify any of the clay lenses, which were able to be detected using sonic drilling.

4.2 STIFFNESS

The stiffness of the basal sand and gravel unit was assessed using typical empirical correlations and using the cross-hole and marine geophysics measurements. In this section stiffness will be discussed using the working shear modulus (G) and small strain shear modulus (G_{max}).

4.2.1 Derivation of stiffness using empirical correlations

Empirical correlations are widely adopted in industry for classifying soil stiffness. The working elastic modulus (E) was approximated by empirical correlation of SPT 'N' values from the conventional boreholes using Eq. (1) below:

$$E = 1000N_{60} \text{ (CIRIA, 1995)} \quad (1)$$

The working shear modulus was then calculated using Hooke's Law:

$$G = \frac{E}{2(1 + \nu)} \quad (2)$$

4.2.2 Derivation of stiffness using geophysics

The small strain shear modulus of the soil (G_{max}) can be directly calculated from the shear wave velocity. The shear wave velocity of a homogenous, isotropic solid is related to the small strain shear modulus (G_{max}) using Eq. (3) below:

$$G_{max} = \rho V_s^2 \text{ (Lunne et. al. 1997)} \tag{3}$$

Where ρ is the soil mass density and V_s is the shear wave velocity.

The small strain shear modulus is related to the working shear modulus by shear modulus ratio G/G_{max} . The measured shear wave velocity and constrained modulus values from available seismic dilatometer testing were used to establish a site-specific shear modulus ratio for each unit using the approach presented in Varcoe et al. (2015).

4.2.2 Comparison of empirical correlation and geophysics

The cross-hole and marine geophysics indicate a much greater for the basal unit’s in-situ stiffness than that derived from the SPT data, shown in Figure 3 below. Due to difficulties in obtaining accurate measurements of basal gravel densities, sensitivity checks were performed assuming various densities from very loose to very dense based on correlation with the SPT data. The increase in stiffness inferred from the geophysical testing was up to 8 times greater at lower relative densities (very loose to loose) and 1.5 to 3 times greater at higher relative densities (medium dense to very dense).

The higher soil stiffness calculated from geophysics was believed to be more representative as the calculation of G_{max} is not reliant on empirical correlations. This has been demonstrated on previous projects in similar geology such the Kempsey Bypass (Terzaghi et al. 2015). The higher soil stiffness was important to prove that the proposed pile design satisfied the strict project settlement criteria. The higher stiffness also attracts additional bending moments for the lateral pile design. Adopting the lower stiffness from empirical correlations can under-estimate the bending moments generated in the piles.

The ability to determine stiffness parameters directly using geophysics for coarse grained materials is valuable as it is site specific, provides higher confidence in the determined parameters and offers programme and cost savings where higher stiffness allows for a more efficient design. The empirical correlations commonly used in industry are typically conservative, in many cases are derived from case histories, which are not representative of the project.

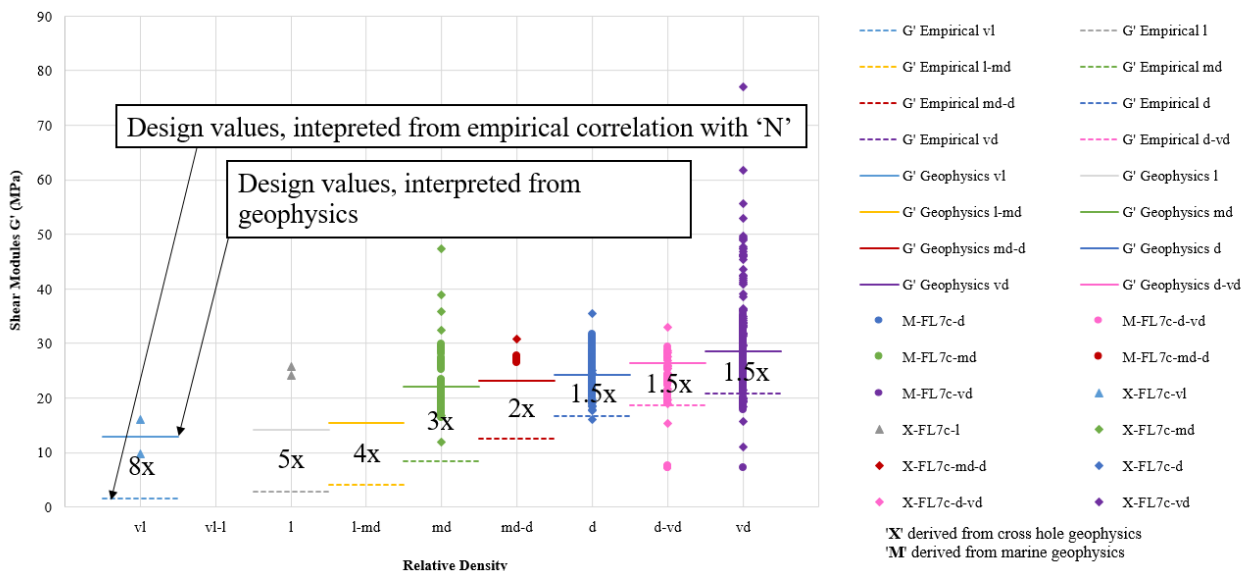


Figure 3: Comparison of stiffness estimate obtained from empirical correlation with SPT ‘N’ and geophysics for Basal Gravel unit

4.3 STRENGTH

4.3.1 Derivation of strength using SPT 'N'

The Mohr-Coulomb framework is widely adopted in industry for classifying soil strength using effective friction angle (ϕ') and effective cohesion (c'). The effective friction angle of granular materials is often estimated empirically using published empirical correlations with SPT 'N'. The relationship presented by Peck et al. (Peck 1963, cited in Tomlinson, 2001) has been adopted and is shown in Figure 4 below. As the basal sand and gravel layer is predominantly granular, the effective cohesion was not assessed.

4.3.2 Derivation of strength using geophysics

The critical state framework provides an alternative method of deriving soil strength from the shear wave velocity measured by cross-hole and marine geophysics. In the critical state framework, strength is understood using the critical state parameter ψ , which can be empirically correlated to a Mohr-Coulomb friction angle.

The soil void ratio (e) was calculated empirically from the small strain shear modulus derived using Eq. (1). A number of empirical relationships exist from small strain shear modulus to void ratio. A modification of the Hardin & Black equation (Hardin and Black 1968, cited in Das 1993) presented in Eq. (4) has been adopted for sands and gravel units at the project site.

$$e = \frac{2\omega + \kappa \pm \sqrt{(-2\omega - \kappa)^2 - \omega(\omega^2 - \kappa)}}{2} \quad (4)$$

Where $\kappa = \frac{G_{max}}{A} * \left(\frac{p'}{p_{ref}}\right)^{-m}$, $\omega = 2.17$, $A = 80$, $m = 0.5$, p' = mean effective stress and p_{ref} = reference pressure

The state parameter can then be calculated at any depth using Eq. (5). The critical state void ratio (e_c) can be obtained by laboratory testing. To allow for comparison with conventional Mohr-Coulomb strength parameters, the peak effective friction angle can then be approximated using Eq. (6)

$$\psi = e - e_c \quad (\text{Jeffries and Been, 2016}) \quad (5)$$

$$\phi' = -45 \times \psi + 32 \quad (\text{Jeffries and Been, 2016}) \quad (6)$$

4.3.3 Comparison of friction angle derived from SPT 'N' and geophysics

A comparison of ϕ' derived from empirical correlation with SPT 'N', and geophysical testing at five pier locations is presented in Figure 5. A number of useful observations can be made:

- The estimated friction angle is loosely correlated between the two approaches, however; the SPT 'N' indicates a denser soil profile and geophysics indicates a looser soil profile;
- High estimated friction angles generally corresponded with gravel layers. Where SPT refusal occurred (represented by the horizontal cluster of data points at a friction angle of 54°), the estimated friction angle is typically larger than the estimated friction angle derived from geophysics;
- Low estimated friction angles generally corresponded with sand lenses. Where low SPT 'N' values were obtained (SPT 'N'=0-5) geophysics confirmed that these values were representative of loose layers within the profile. Care should be taken in the interpretation of occasional low SPT 'N' values within a gravelly profile. Low SPT 'N' values should not be simply dismissed as outliers or errors in testing;
- Sonic drilling was essential to understanding friction angles interpreted from both methods within the context of the geological model. The ground model derived from widely spaced SPT tests can be misleading as significant sand and clay lenses are not able to be identified.

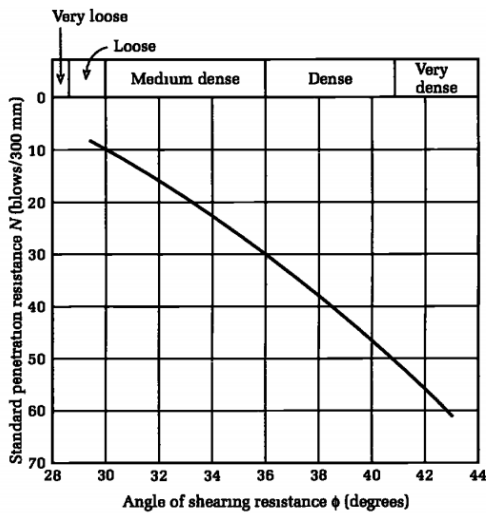


Figure 2.13 Relationship between SPT *N*-values and angle of shearing resistance (after Peck *et al*²⁶)

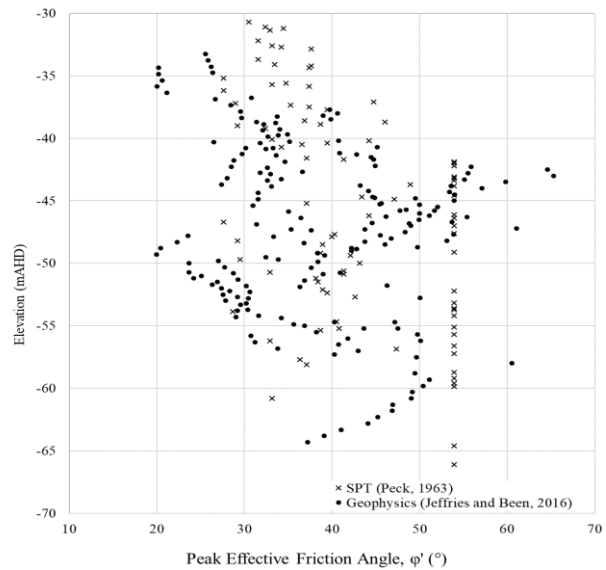


Figure 4: (Left) Relationship between SPT ‘N’ value and angle of friction (Peck 1963, cited in Tomlinson, 2001) (Right) Peak effective friction angle versus elevation for the basal sand and gravel unit, derived using SPT N60 (Peck *et. al*) and geophysics (Jeffries and Been, 2016)

5 TECHNICAL SUCCESSES

5.1 DRILLING RESISTANCE OF BASAL GRAVELS

Despite the physical difficulties progressing conventional drilling methods in gravel materials, valuable observations could be made and recorded by the supervising engineer and drillers of drilling resistance through various strata. The interbedded nature of the basal deposits and distinctive ‘crunch’ on first strike of gravel or cobbles, allowed relatively accurate measurements of layer thickness and transitions.

As the drill string was advanced, it was often necessary to pull back and re-drill to prevent locking up of the drill bit between large cobbles. It was often the case that after multiple drill passes, the similarities in resistance of each pass suggested that the gravel and cobble grains were merely being pushed aside and thus the material was sufficiently loose to allow for this movement. These observations were recorded on borehole logs as a valuable piece of information, which would be otherwise ignored to the design engineer scanning SPT values of refusal.

Both tri-cone and diamond tipped coring drill-bits were trialled in the basal gravels, each with degrees of success. Whilst tri-cone bits are more suitable for progression and tend to survive longer, soil returned in drilling fluid is generally crushed and provides little information on the true size of the gravel and cobble grains. Diamond tip drill bits on the other hand can core through larger cobbles and pickup gravels providing some indication of size up to the core barrel diameter (PQ in this case), albeit with a shorter lifespan and thus additional cost.

5.2 NATURAL GAMMA LOGGING

One of the more unique techniques employed was natural gamma logging, which is more often deployed in the minerals, oil and gas industries. The method relies on the detection of gamma emission spectra of primarily potassium occurring naturally in clay dominant rocks such as siltstones, and shales. Whilst natural gamma logging does not provide distinct ‘geological logs’, the interbedded nature of the soil sequence at the site coupled with complete sonic soil cores provided a supplementary source of information regarding layer transitions. Seawater was observed to have a relatively insignificant gamma emission signature and thus the depth to mudline could be easily confirmed from gamma traces.

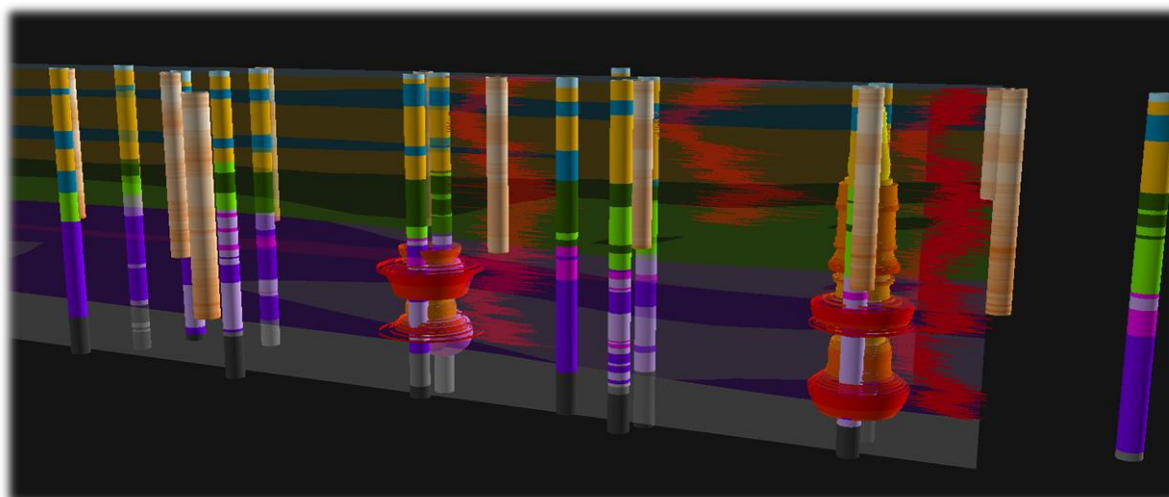


Figure 5: Example of natural gamma traces (red lines) overlaid on a 3D geological model

Natural gamma logging was undertaken within boreholes following installation of PVC for cross-hole geophysics or groundwater monitoring wells as well as prior to pulling up steel casing for overwater boreholes. To test the potential signal attenuation through steel, a comparison was undertaken on a land-based hole through steel casing and then again through PVC with determined a minor attenuation of about 2%.

The equipment was easily operated by a field engineer or geologist following supplier training requiring only operation of a wireline winch and data logger, powered by a 12volt battery or mains supply. For the probe supplied for this project, a winch speed of 3m/min was used, and a trace was obtained on both the up and down legs and undertaken twice at each location. Comparison of the up, down and different passes showed only natural variability with the overall trace shapes largely consistent.

A few useful observations of trends and patterns of natural gamma traces on the project were:

- Generally higher emission signatures at depths coinciding with logged clays and sandy clays and conversely lower signatures in sand dominant materials. Very low relative signatures were recorded for clean sands observed on the South portion of the site.
- A relatively larger range and scatter of emission signatures at depths coinciding with logged gravel and cobble materials. The authors suspect that the scatter is a result of the relatively high concentration of clay particles within the meta-siltstone gravel contrasted with void/water space between the grains.

An example of natural gamma logs within the context of a 3D ground model is presented in Figure 5 showing higher relative readings within clay dominant materials (shown in darker colour sets, see Table 1).

The typical time from setup to pack-down for a 65m borehole was one hour; a month hire cost of the equipment totalled A\$6000 thus making this technique remarkably efficient. This leads the authors to conclude that for complex interbedded sequences, the return on investment and value add to geological logs from an accompanying natural gamma log is substantial and hopes to see the technique more widely researched and employed for use in the geotechnical industry.

6 CONCLUSIONS AND RECOMMENDATIONS

A comprehensive and diverse GI campaign was completed for the proposed bridge over the Clarence River at Harwood which included conventional and sonic drilling and geophysical test methods. The scope of works was developed and delivered to manage the technical and physical challenges of obtaining reliable material information of deep gravel deposits. Reflecting on these challenges and successes, the author draws the following conclusions and recommendations:

- Substantial value can be added for designers, clients and projects by pursuing non-traditional investigation techniques such as sonic drilling and geophysical surveys. The best outcomes occur where emphasis on a ‘right tool for the right job’ approach is adopted and by utilising a diverse range of techniques to build sound evidence-based ground models.
- Sonic drilling coupled with cross-hole seismic testing and marine geophysics is a viable strategy for obtaining meaningful engineering properties for gravel and cobble dominant beds where conventional SPTs are unsuitable. Sonic cores provide engineers and geologists with meaningful observation of grain size and characteristics whilst geophysics can provide small strain material behaviour properties and densities.
- Natural gamma logging presents a time and cost-efficient technique for sub-surface profiling and is a valuable supplement to a borehole log, particularly in complex and interbedded sequences.
- Observations of drilling resistance and physical challenges of penetrating gravel and cobble materials should always be recorded on logs as they can provide insight into relative density to contextualise refusing or outlier SPTs.

6.1 AREAS FOR FURTHER RESEARCH

- Investigation of the zone of influence of sonic drilling vibration to determine whether results of SPT, CPT or geophysical tests adjacent to a previously drilled sonic borehole are compromised.
- Further research into the application of natural gamma logging and other easily deployed and efficient geophysical test methods to extract additional geological and geotechnical data from drilled boreholes.
- Feasibility of positioning geophone arrays at ground surface to detect SPT blows at depth as a seismic source providing a “reverse down-hole seismic” test during conventional borehole drilling. Such a method could be reasonably and cost efficiently deployed for many ground investigations to capture data on material properties.
- Evaluation of design methodologies adopted and comparison with results of pile driving.

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