

# GEOTECHNICAL ASSESSMENT OF PLEISTOCENE SOILS OF THE MULGRAVE RIVER DELTA, NORTH QUEENSLAND

C. Vincent and N. Murphy<sup>1</sup>

<sup>1</sup>WSP Golder, 900 Ann Street, Fortitude Valley, Queensland, Australia.

## ABSTRACT

The Mulgrave River Delta is a Quaternary age fluvial system in tropical North Queensland. The near-surface geology of the area comprises the Quaternary age (Pleistocene to Holocene age) Mulgrave River Delta soils. The soils accumulated to form deep infill of a distinct valley that is tens of metres deep. Geotechnical borehole, cone penetration test (CPTu) and geotechnical laboratory testing data were collected for use in geotechnical design strength assessment for displacement (driven) piles design for structures associated with the Edmonton to Gordonvale (E2G) Bruce Highway upgrade project, south of Cairns. This study focuses on the Pine Creek Yarrabah Road (PCYR) overpass bridge specifically. It is planned for the bridge abutments and piers to be supported on driven precast pre-stressed concrete (PSC) piles. As part of the site investigation twelve boreholes were drilled enabling standard penetration testing (SPT) and sampling. The SPT data identified a lower strength layer at depth. This was considered to be inconsistent with the initially anticipated geology, where increasingly stiff to hard cohesive soils had been expected. Supplementary geotechnical investigations were completed, including cone penetration tests (CPTu) and additional laboratory testing was scheduled on 'undisturbed' tube samples ( $U_{50}$ ). The aim was to assess implications (if any) of the identified lower strength layer on the design of PSC piles. Comparison of the field investigation and laboratory test data enabled greater confidence in the prediction of the soil strength profile and helped towards an optimised design for the PSC piles. To assist with the verification of geotechnical design strength, PDA testing (pile driving analyser – dynamic load testing) was completed during the driving of piles with CAPWAP analysis to help with confirming a prediction of shaft friction of the piles.

*Keywords: Mulgrave River Delta, cohesive soils, undrained shear strength, shaft friction*

## 1 INTRODUCTION

The Edmonton to Gordonvale (E2G) project will upgrade a 10.5 km section of the Bruce Highway between Gordonvale and Edmonton (south of Cairns) from a two-lane highway to a four-lane duplicated highway. The project will deliver significant safety improvements in this corridor which include signalised intersection and road upgrades, construction of overpasses (for vehicles and pedestrians), realignment of the Queensland Rail corridor, various cane rail relocations and construction of a separate cycle way adjacent to the new highway alignment.

The Maitland Road section of the E2G alignment (near Gordonvale) includes two new overpasses. The Pine Creek Yarrabah Road (PCYR) – North Coast Line (NCL) rail overpass is proposed over the existing rail alignment east of the Bruce Highway and the Pine Creek Yarrabah Road (PCYR) – Bruce Highway road overpass is proposed over the Bruce Highway. Both proposed bridges are located approximately 400 m south of the existing Maitland Road. This case study will focus on the PCYR road overpass which includes driven precast pre-stressed concrete (PSC) piles at the abutment and piers.

The site investigation included boreholes drilled at the piers and abutments where a thick layer of alluvium (up to 35 m) was identified overlying a layer of basalt at approximately 36 m depth. Between approximately 20 m to 26 m depth, standard penetration test (SPT) data identified a lower strength layer which was considered inconsistent with the anticipated geology.

Additional geotechnical investigations, comprising CPTs, were completed to help understand the impact (if any) of this perceived low strength layer on the proposed design of the PSC piles. During the pile driving, verification testing using PDA was used along with CAPWAP analysis to help confirm if the design geotechnical capacity was achieved. This case study details the findings of the geotechnical investigations carried out for the PCYR road overpass and the analysis of the data to create a design undrained shear strength model for driven PSC piles.

## 2 GEOLOGICAL SETTING AND STRATIGRAPHY

Reference to the published geological mapping (1:100,000 series Geological Map for Cairns Innisfail, by Queensland Government Department of Mines and Energy, 1989), indicates that the E2G corridor traverses a Quaternary age (Pleistocene overlain by Holocene age) alluvial flood plain, underlain at depth by the Hodgkinson Formation bedrock. The entire area represents a former delta of the Mulgrave River, where the Mulgrave River Delta soils represent deep infill of a down-cut gorge/valley that is tens of metres deep. The delta formed within the north to south oriented valley that is parallel to the coast, lying behind the coastal (former island) Malbon Thompson Range hills. The valley opens out both to the north and south, at the Trinity Inlet and the Mulgrave River respectively.

The alluvium is anticipated to be substantially deep. It formed as a series of prograding alluvial fans derived from the mainland during the Pleistocene period. Deltaic alluvial soils may comprise a series of coarsening upward sequences, where the higher energy (proximal to the source) deposits spilled forward over previously deposited finer grained (distal) deposits, that can also be conjectured to dominate towards the east, north and south. Periodic falls in relative sea level during the later Pleistocene and Holocene periods may have caused erosional downcutting into 'older' alluvium, only for these downcut valleys to become filled with lesser consolidated soils as the sea levels re-bounded.

During the later Pleistocene period, crustal extension enabled episodic eruptions of basaltic lava, today represented by Green Hill (the Meringa Basalt) that rises within the otherwise broad alluvial valley near Gordonvale. It is probable that the Meringa Basalt could overly older Pleistocene alluvium and potentially a cover of hillslope derived colluvium at the margins of the Mulgrave River valley corridor.

The surrounding and underlying geology is underlain by the Devonian age Hodgkinson Formation. The Hodgkinson Formation is dominated by folded and cleaved partly metamorphosed sediments and interspersed granite bodies. The bedrock surface was potentially eroded (scoured) during episodes of low relative sea level, such that a relatively thin (or absent) bedrock weathered profile may be anticipated under the alluvial deposits (W. F. Willmott, P. J. Stephenson, 1989).

### 2.1 Site Geology

The site geology (based on geotechnical investigation data) at PCYR road overpass typically comprises fine grained alluvial soils with beds of sand and gravel, underlain by a thin basalt (Meringa Basalt) flow (2.8 m to 4.1 m thick) at approximately 36 m depth (Golder Associates, 2020). Atterberg Limits test data and site observations (including SPT N values) indicate a layer of silt (typically 60-85% fines content) between approximately RL 1 m AHD and RL -6 m AHD. The inferred stratigraphy is illustrated in Figure 1.

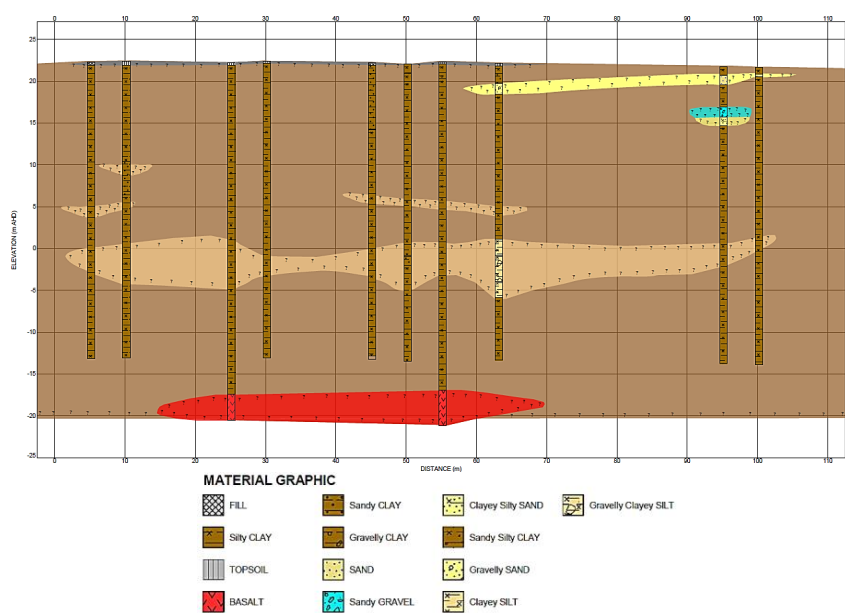


Figure 1: PCYR Road Overpass Inferred Stratigraphy

### 3 GEOTECHNICAL SITE INVESTIGATION DATA

#### 3.1 In-situ Testing (SPT and Vane Data)

The site investigation prior to and during the detailed design at the PCYR road overpass included borehole drilling and laboratory testing on disturbed and undisturbed soil samples (Golder Associates, 2020). SPTs were performed at 1 m intervals (commencing at 1 m depth) in all boreholes. Field SPT N values were corrected to  $N_{60}$  values to account for hammer efficiencies and equipment variables. The results are shown in Figure 2 below. Between about RL 1 m AHD and RL -6 m AHD, a layer of low SPT N values is evident (values as low as 0).

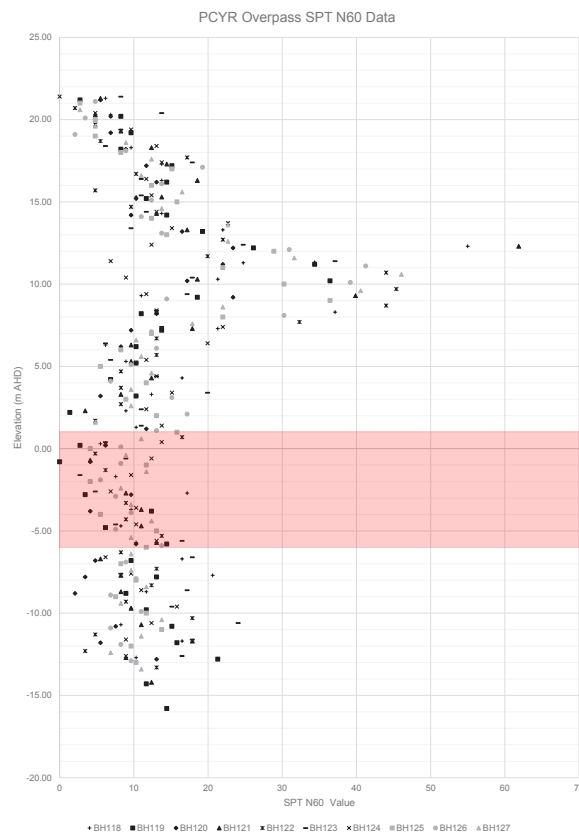


Figure 2: SPT  $N_{60}$  Data at PCYR Overpass

The SPT  $N_{60}$  values were used to estimate undrained shear strength values using a correlation factor of 7 ( $S_u \approx 7 \times N_{60}$ ). Results are displayed in Figure 4 and where the inferred silt layer is present (between RL 1 m AHD and RL -6 m AHD),  $N_{60}$  values indicated the undrained shear strength varies from 25 kPa to 100 kPa (soft to firm).

In some undisturbed tube samples collected during the borehole investigation, vane shear tests were completed. These results indicated undrained shear strength values ranged between 40 kPa and 80 kPa (corrected for plasticity using the correction factor from Bjerrum, 1973) between RL 1 m AHD and RL -6 m AHD. Due to the low  $N_{60}$  values encountered within this layer, additional field investigation was completed comprising a CPT at the proposed abutment location (Golder Associates, 2020), discussed in Section 3.3 below.

#### 3.2 Laboratory Test Data (Atterberg Limit, Particle Size Distribution and Triaxial Compression Test Data)

Atterberg Limit and Particle Size Distribution (PSD) tests were completed during the site investigation prior to the detailed design at the PCYR road overpass (Golder Associates, 2020). PSD and Atterberg Limit test results completed on samples collected between RL 1 m AHD to RL -6 m AHD are presented in Figure 3. Results indicate the soil at this elevation is predominantly low to high plasticity silt.

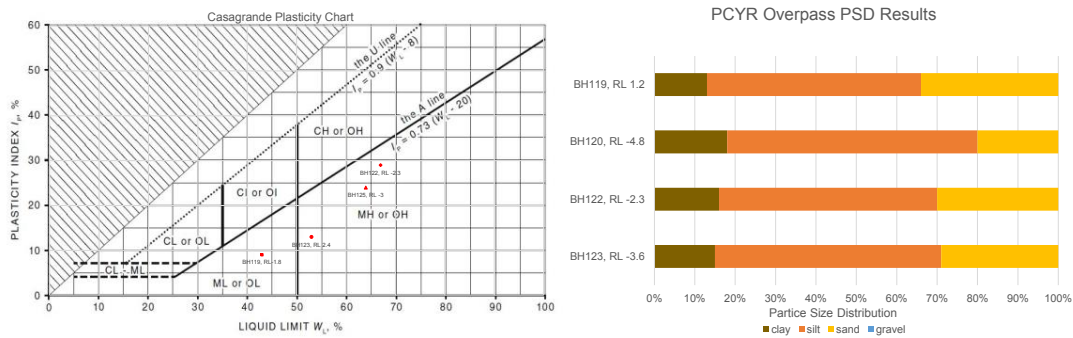


Figure 3: Atterberg Limit and PSD Test Data PCYR Road Overpass

Laboratory testing undertaken on undisturbed tube samples collected within the boreholes included single stage and multi-stage triaxial compression testing. The undrained shear strength at the select test locations (inferred from the test results) varied between 114 kPa and 398 kPa. The samples from BH120 to BH125 indicate the undrained shear strength of the soil between RL 1 m AHD and RL -6 m AHD is very stiff to hard. The triaxial test results are plotted in Figure 4.

### 3.3 CPT Data

Undrained shear strength was calculated from the CPT data using corrected cone resistance ( $q_c$ ) and a  $N_{kt}$  factor of 15. The undrained shear strength inferred from in-situ test results (i.e. CPT data, SPT data, vane shear and pocket penetrometer tests) and from laboratory tests (i.e. triaxial tests) are presented in Figure 4. The CPT results correlate with the results from the triaxial compression tests and vane shear tests and indicate the soil strength between RL 1 m AHD and RL -6 m AHD varies from 50 to >200 kPa (stiff to hard). Note that CPT201 was pre-drilled from RL 15 m AHD to RL 5 m AHD (refer to Figure 4).

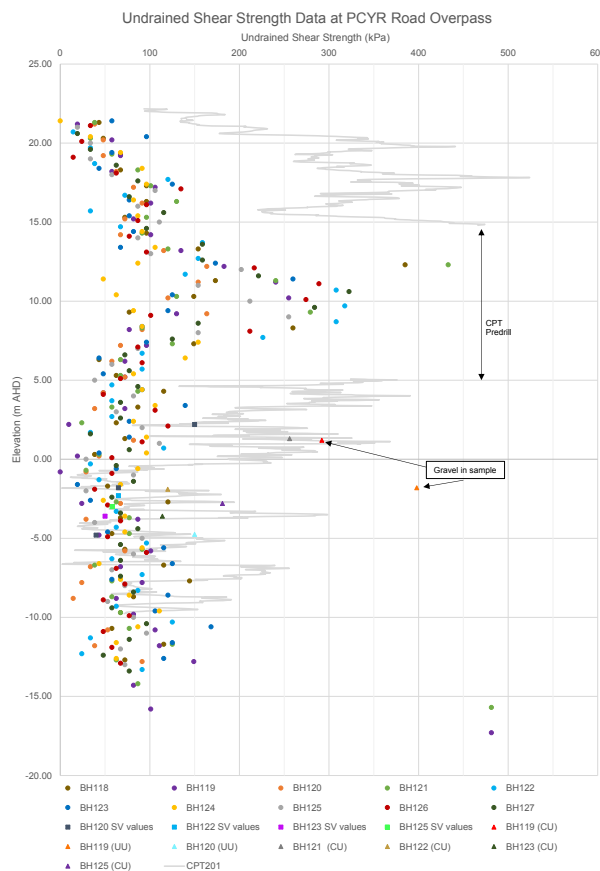


Figure 4: Undrained Shear Strength Data at PCYR Road Overpass

#### 4 DESIGN OF DRIVEN PSC PILES

Between RL 1 m AHD and RL -6 m AHD the results from CPT201 indicate the undrained shear strength varies from 50 kPa to > 200 kPa whereas the  $N_{60}$  results indicate the undrained soil strength ranges from <25 kPa to 100 kPa (soft to stiff). Triaxial compression test results correlated with the CPT results and indicates the soil is stiff to hard at this elevation. The low SPT values could have been caused by realignment of the soil structure due to repetitive shearing by the SPT hammer and appear to correlate with the zone of high silt content (illustrated in Figure 1). The design shear strength profile adopted for the piles at PCYR road overpass was estimated using CPT and triaxial compression test data. Design shaft friction was then developed using the adopted design shear strength profile and adhesion factors from Tomlinson and Woodward (2008). Table 1 below presents the adopted design parameters for alluvial cohesive soils at PCYR road overpass. The Unit 1B parameters were adopted within the silt layer between RL 1 m AHD and RL -6 m AHD.

Table 1: Adopted Design Parameters

Unit	Unit General Description	$\gamma_b$ (kN/m <sup>2</sup> )	$c_u$ (kPa)	$f_s$ (kPa)*
1B	Stiff	19.0	80	40
1C	Very Stiff to Hard	19.5	140	80
1D	Hard	20.0	250	125

Note: \* -  $f_s$  calculated based on adhesion factors from Tomlinson and Woodward (2008)

#### 5 DESIGN SOIL STRENGTH PROFILE VERIFICATION

Verification testing using PDA was completed during the driving of piles with CAPWAP analysis to analyse the achieved shaft friction at Pier 3 for PCYR road overpass (Golder Associates, 2020). Pre-drilling to 7 m depth was carried out on site to assist with pile installation. Figure 5 presents the piles at Pier 3 achieved shaft friction results from the CAPWAP analysis along with the adopted design shaft friction for Pier 3. The CAPWAP results typically indicate a higher achieved shaft friction than the design shaft friction profile. The results indicate that shaft friction through the inferred silt layer below RL 1 m AHD decreases from approximately 100 kPa to as low as 30 kPa, but is still generally higher than the design shaft friction (40 kPa) for this layer. Note that the pile design was based on static analysis, adopting the full frictional resistance of the soil. This full friction resistance will occur over time (set-up) after pile driving and installation. The CAPWAP results below are presented for re-strike testing (at between 6 and 23 days post pile installation) and allow for some set-up.

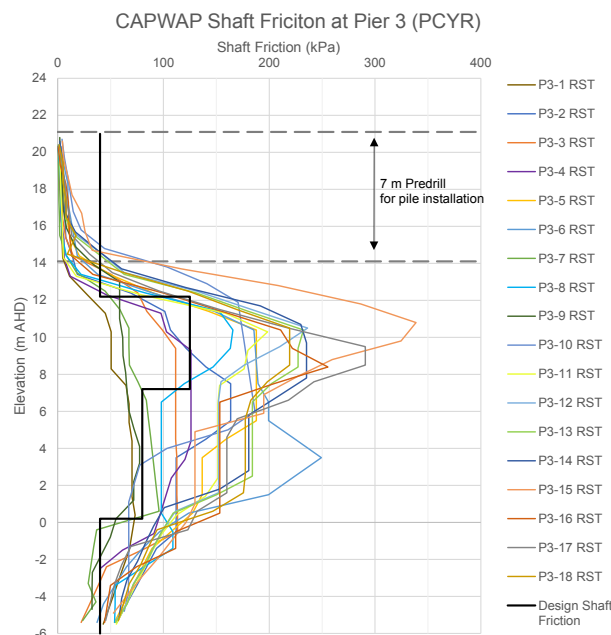


Figure 5: CAPWAP Shaft Friction Results

## 6 CONCLUSIONS

The Pine Creek Yarrabah Road (PCYR) Bruce Highway road overpass site investigation data (including laboratory test and SPT results) indicated a bed of silt was present between approximately RL 1 m AHD and RL -6 m AHD at PCYR road overpass. SPT results from borehole investigations indicated the undrained shear strength of the silt was much lower than the results of the triaxial compression test and CPT data. The low SPT results (and inferred undrained shear strength) are inferred to be caused by realignment of the soil structure due to repetitive shearing caused by the SPT hammer (i.e. cyclic loading).

The adopted undrained shear strength design profile (and later shaft friction profile) considered the results of the triaxial compression test data and CPT data. PDA results combined with CAPWAP analysis from driving of the PSC piles was used to help confirm the design shaft friction. The CAPWAP analysis indicated a lower achieved shaft friction at the elevation of the silt layer (compared to other alluvial layers in the profile), but still achieved close to the design skin friction for this layer.

The lower strength layer was identified during the site investigation, and the adopted design undrained shear strength relied on testing from CPT, vane shear tests and laboratory test data, rather than using correlations with SPT  $N_{60}$  values. Due to the sensitive nature of this layer at depth, SPT  $N_{60}$  values were considered not appropriate for the design. Furthermore, consideration must be given to site investigation methodology and laboratory testing when investigating Pleistocene soils of the Mulgrave River Delta. These soils are not consistent in nature over the entire depth of the profile, and some layers of 'sensitive' cohesive soils with high silt content may be present. The design methodology must consider different investigation techniques, including in-situ and laboratory testing. PDA testing is also crucial for confirmation of geotechnical resistance and confirmation of the design capacity of piles.

## 7 ACKNOWLEDGEMENTS

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