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SITE INVESTIGATION AND GEOLOGICAL CHALLENGES FOR A RAIL LEVEL CROSSING PROJECT IN SEQ

Pramod Thakur[1] and Craig Brock[2]

1. Technical Executive, WSP Australia

2. Senior Associate Engineering Geologist, WSP Australia

ABSTRACT

An understanding of geology is very important to minimise design and construction uncertainties and challenges for any site. WSP was engaged to deliver a detailed design of an overpass bridge and associated structures to replace a very busy rail level crossing for a rail track in a metro area of Southeast Queensland (SEQ). The design services included design of an overpass bridge, earthworks, approach embankments, retaining walls, etc. Considering the design life of 100 years for these structures, a detailed geotechnical investigation (GI) and interpretation of the GI data were required to be fed into the design. As the site was located in an urban area, carrying out geotechnical investigation was not an easy task. It required dealing with various stakeholders, local community, and working night shifts to manage technical requirements and public expectations.

The project conceptual ground model benefited from some previous intrusive site investigations. However, WSP noted that the previous boreholes had limited rock coring and had relatively un-explained SPT refusals in soil material at depths of less than 10 metres below ground level. WSP scoped additional geotechnical investigation as per the Department of Transport and Main Roads (DTMR) geotechnical design standards and project specifications. When the geotechnical investigation was complete, the encountered materials were weakly and moderately cemented units, interlayered with poorly consolidated materials that are typically dominated by Residual (Sandy CLAY) to about 25 m below ground level overlying very low strength (UCS < 1.5 MPa) Sedimentary rock (Sandstone/Siltstone/Claystone) to the borehole termination depth of approximately 40 m bgl. This created additional challenges for pile foundation design and constructability issues.

This paper discusses how the challenges during geotechnical investigation were overcome and interpretation of the encountered strata for feeding into the design to manage geotechnical uncertainties considering constructability within a constrained environment.

1 PROJECT DESCRIPTION

WSP Australia was engaged to deliver a detailed design of a railway overpass bridge and associated structures to replace a busy rail level crossing for a rail track in a metro area of Southeast Queensland. The proposed development primarily comprises a new 92 m long, three span bridge, over the existing rail track including over 200 m of approach embankments either side of the bridge supported by Reinforced Soil Structure (RSS) walls, a pedestrian underpass, and an upgrade of an intersection and pavement. Both the geotechnical site investigation and the subsequent detailed design were completed in accordance with the relevant Department of Transport and Main Roads (DTMR) standards.

Figure 1 is an overlay of the project site showing the alignment of the proposed bridge and the upgrades to the surrounding approach roads.

This paper describes the site and some of the constraints in relation to completing the geotechnical investigation and the detailed design of the proposed structures. The geological setting and the initial project conceptual ground model are discussed followed by interpretation of the additional site investigation works which led to a modified 3D geotechnical design model. The apparent lack of correlation between in-situ dynamic cone penetration (DCP) tests and standard penetration tests (SPT) during the additional site investigation is discussed. Finally, the paper presents recommended geotechnical design parameters based on the 3D ground model and the relevant DTMR standards for geotechnical design.

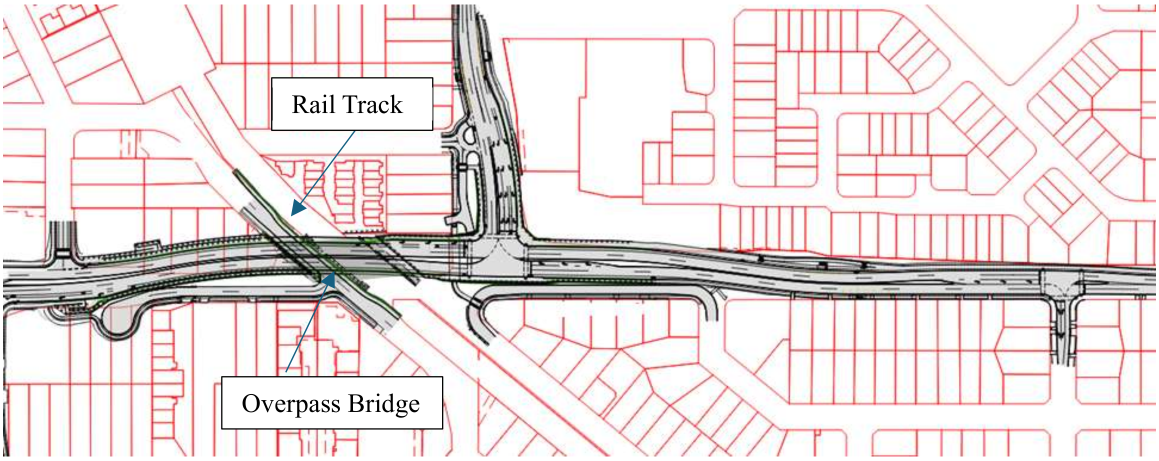

Figure 1: Proposed development layout.

2 SITE CONSTRAINTS

The project involves the construction of a bridge and approach works, removal of the existing level crossing, construction of a new intersection and road pavement upgrades. Existing infrastructure within the project site that had to be considered included an active petrol station, rail line with overhead electrical lines, active commercial/retail buildings, private properties and buried services. Figure 2 shows an aerial view of the project site centred on the existing rail crossing.

Figure 2: Existing conditions at project site (Google Earth, 2024).

3 GEOLOGICAL SETTING

The project is located in the Oxley Basin, the Oxley Group of rocks comprises three sedimentary formations that are said to be lateritised towards the basin's southern extent (Geological Survey of Queensland, 2023). The geological map of the project site indicates that the majority of the road alignment is underlain by the Sunnybank Formation (of the Oxley Group) unconformably overlying the Triassic/Jurassic age Woogaroo Subgroup. Figure 3 shows the project site with the local geology overlay, and Table 1 provides a description of the geological units expected to be encountered on the project site.

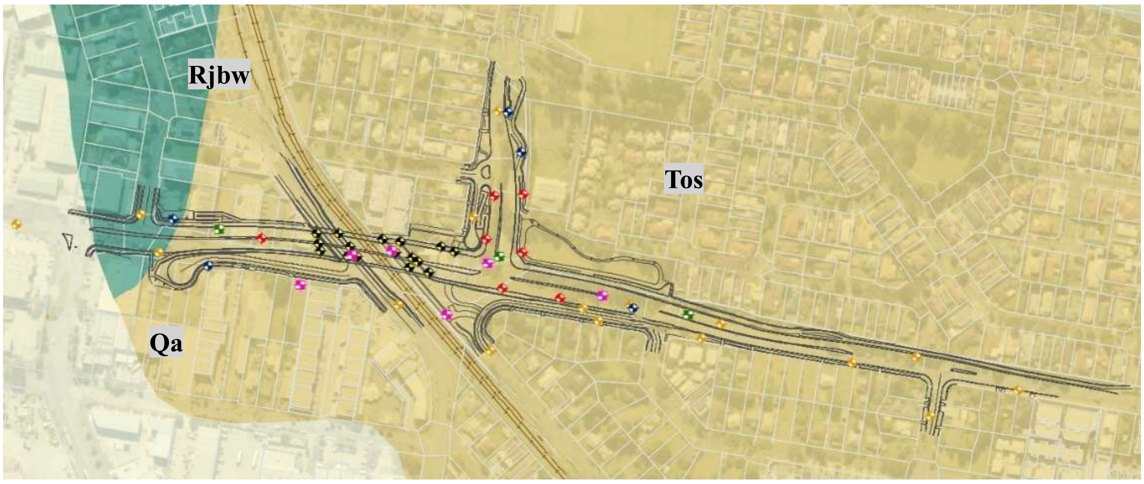

Figure 3: Project layout, local geology, and proposed and existing borehole locations.

Table 1: Geological unit description

Geological Unit	Description
Rjbw	Woogaroo Subgroup (Stratified unit including Volcanic and Metamorphic) – Sublabile to quartzose sandstone, siltstone, quartz – rich granule to cobble conglomerate and coal.
Tos	Sunnybank formation (Stratified unit including Volcanic and Metamorphic) – Lacustrine and fluviatile quartzose to sublabile sandstone, conglomerate, silty clay, siltstone, clayey mudstone: mildly lateritised.
Qa	Alluvium – Clay, silt, sand, and gravel: flood-plain alluvium.

The Sunnybank Formation is said to consist of up to 164 m of claystone, sandstone, conglomerate and limestone deposited in a fluvial to lacustrine environment (Geological Survey of Queensland, 2023). The sedimentary rocks of the Oxley group have been undisturbed by folding and are considered to be relatively unconsolidated and soft (Willmott, 2012).

The Sunnybank Formation includes weakly and moderately cemented units, interlayered with poorly consolidated materials that are typically dominated by sandy clay. The Formation was subject to deep (lateritic) weathering processes when it was subaerially exposed. Superficial soils are typically red earth (lateritic red soils and laterite podzol) that may be described as clay, with irregularly shaped nodules of limonite and haematite.

The Sunnybank Formation includes vertically and laterally non-persistent beds of quartzose (grading up from pebbly conglomerate) sandstone, clayey/weathered feldspathic sandstone and sandy mudstone, any of which may be moderately cemented, typically low and partly medium strength.

Towards the western end of the project site the geology was anticipated to transition to the Woogaroo Subgroup and surficial deposits of quaternary aged alluvium. The Woogaroo Subgroup is dominated by grey interbedded sandstone and siltstone, with some interbedded conglomerate and some carbonaceous shaly mudstone.

4 GEOTECHNICAL INVESTIGATION

4.1 INVESTIGATION REQUIREMENTS

Knowledge of the properties of the subsoil layers and their lateral variation are important for an accurate site characterisation. According to the Department of Transport and Main Roads (DTMR) Geotechnical Design Standard Minimum Requirements (DTMR, 2020a), geotechnical investigations are required to inform the design of the foundations of all bridges and other structural elements. The scope of work for any site investigation needs to ensure that the site geological model can reasonably be established and adequately inform the design. Additional site investigation requirements are also specified based on the proposed structure types. DTMR (2020a) requires a minimum of two boreholes to be drilled at every abutment and pier location. Also, the boreholes to be drilled should be at intervals not exceeding 10 m along the width of every abutment and pier of all bridges. For bored piles, the boreholes are to be extended to a minimum of 5 m into competent bedrock (moderately weathered and medium strength or better rock).

In addition, the Main Roads Technical Specification MRTS63 (DTMR, 2020b) requires the depth of at least three boreholes or 50% of all the boreholes at a proposed bridge location to be a minimum of 2 x pile diameter deeper than the proposed pile base with no weak layers identified. MRTS63 further states that the maximum distance between the boreholes and proposed pile location to be as per the following pile ultimate loading:

- ≤2,000 kN: borehole can be 10 m away from the pile location.
- ≤10,000 kN: borehole to be within 10 m from the pile.
- >10,000 kN: borehole to be at the pile location.

4.2 CONCEPTUAL GROUND MODEL

There were some existing boreholes available from the Business Case design of the project (Figure 4). The conceptual ground model was developed based on this data. As there were only two boreholes (BH6 and BH7) at the bridge location, additional investigation was required. Moreover, these boreholes had limited rock coring and had relatively un-explained Standard Penetration Test (SPT) refusals in soil material at depths of less than 10 metres below ground level. These previous intrusive works were not sufficient to meet the DTMR and project requirements. Therefore, an additional geotechnical investigation scope was developed with consideration of the project objectives and the locations of the previous investigation completed at the site.

Figure 4: Existing geotechnical investigation locations.

4.3 ADDITIONAL INVESTIGATION AND METHODOLOGY

An additional geotechnical investigation was proposed to inform the detailed design of the bridge and associated structures in accordance with the project specification. The investigation included the following number of additional boreholes, and the locations of these boreholes are shown in Figure 3.

- 14 boreholes for the proposed bridge.
- 14 boreholes for the proposed retaining walls and pedestrian underpass.
- 19 boreholes to investigate the pavement subgrade of the road alignments.

In addition, a number of laboratory tests (e.g. Atterberg Limits, Aggressivity, Point Load Index, Uniaxial Compressive Strength, 4 Days-soaked California Bearing tests for pavement, etc.) were conducted on the materials retrieved during the investigation.

Given the urban environment there were many site constraints to consider. Constraints to the site investigation included:

- Dense urban environment comprising residential and light industrial / commercial properties adjoining the road corridor.
- Potentially contaminated soils (including a petrol station).

- A number of buried services.
- A busy road where the site investigation had to be completed with no disruption to the daytime traffic.

Careful planning and coordination were required to carry out the proposed investigation. In addition, the following methodologies were implemented to minimise the disruption to the public and manage the site constraints:

- Public consultation was conducted before any works commenced on site.
- Boreholes along the existing road alignment were completed at night under lights and with all relevant traffic management and permits in place.
- For the boreholes located within 3 m of underground services, hand auger and/or non-destructive drilling (NDD) techniques were used from the surface through the fill material until natural soil was confirmed.
- When drilling at the petrol station, gas monitoring was carried out and the drilling rig was shut down during re-fuelling activities.

Following the NDD of the boreholes, drilling was advanced through the soil using auger and rotary wash boring techniques until rock was encountered. At the bridge borehole locations, the boreholes were advanced through rock using diamond rotary coring techniques. The encountered materials in the boreholes were logged in accordance with AS1726-2017.

Standard Penetration Tests (SPT) were completed in the bridge and retaining wall boreholes at approximately 1.5 m intervals until refusal (SPT, N > 50). Dynamic Cone Penetration (DCP) tests were completed adjacent to each borehole to assess strength of soils within top 3 m.

5 SUBSURFACE PROFILE AND GROUND WATER

Based on the materials encountered in the boreholes, the typical subsurface profile at the project site consists of the following:

- Fill – Generally comprising firm Gravelly CLAY, medium dense to dense Clayey and Gravelly SAND up to 1 m below ground level (bgl). In absence of compaction tests results, the fill encountered was generally considered to be un-controlled as per AS3798-2007.
- Alluvium – Encountered as a thin layer in a small number of borehole locations. Where encountered it was described as grey Sandy CLAY or Clayey SAND.
- Residual Soil / Extremely Weathered Material – generally stiff to hard CLAY and dense to very dense Clayey SAND, encountered up to 31.5 m bgl. Inferred lateritised zones at shallow depths is shown in Figure 5.
- Rock – highly weathered to fresh Siltstone, Sandstone, and Claystone, generally very low to low strength.

Figure 5: Inferred lateritised zone in cored soil (WSP, 2024).

The ground conditions were generally consistent with the published geology. The SPT refusals (> 50 blows/150 mm penetration) at shallow depths (< 10 m bgl) from the conceptual ground model were encountered during the additional ground investigation. These refusals have been interpreted to represent weakly cemented or lateritised horizons. In places, these layers have been described as rock (unable to disaggregate by hand) in accordance with AS1726-2017.

Groundwater was encountered during the investigation and data from groundwater monitoring wells installed as part of the project indicated that groundwater was generally < 2 m below the existing ground surface.

6 GEOTECHNICAL GROUND MODEL

6.1 GEOTECHNICAL UNITS

When the geotechnical investigation was complete, the encountered materials were summarised as weakly and moderately cemented units interlayered with poorly consolidated materials that are typically dominated by Residual (Sandy CLAY) to about 25 m bgl overlying very low strength (UCS < 1.5 MPa) sedimentary rock (Sandstone/Siltstone/Claystone) to the borehole termination depths of approximately 40 m bgl.

In general, the geotechnical units interpreted for the geotechnical model and design were:

- Fill overlying
- Residual Soil overlying
- Extremely Weathered Material overlying
- Weathered Sunnybank Formation (Sandstone/Siltstone/Claystone).

6.2 3D MODEL AND GEOLOGICAL SECTIONS

The borehole data from the site investigations was imported into Leapfrog, a leading 3D geological modelling software, to assist with the development of the 3D ground model. The geotechnical units were imported based on the geological sequence interpreted from the borehole logs. The 3D geological model for the site is shown in Figure 6.

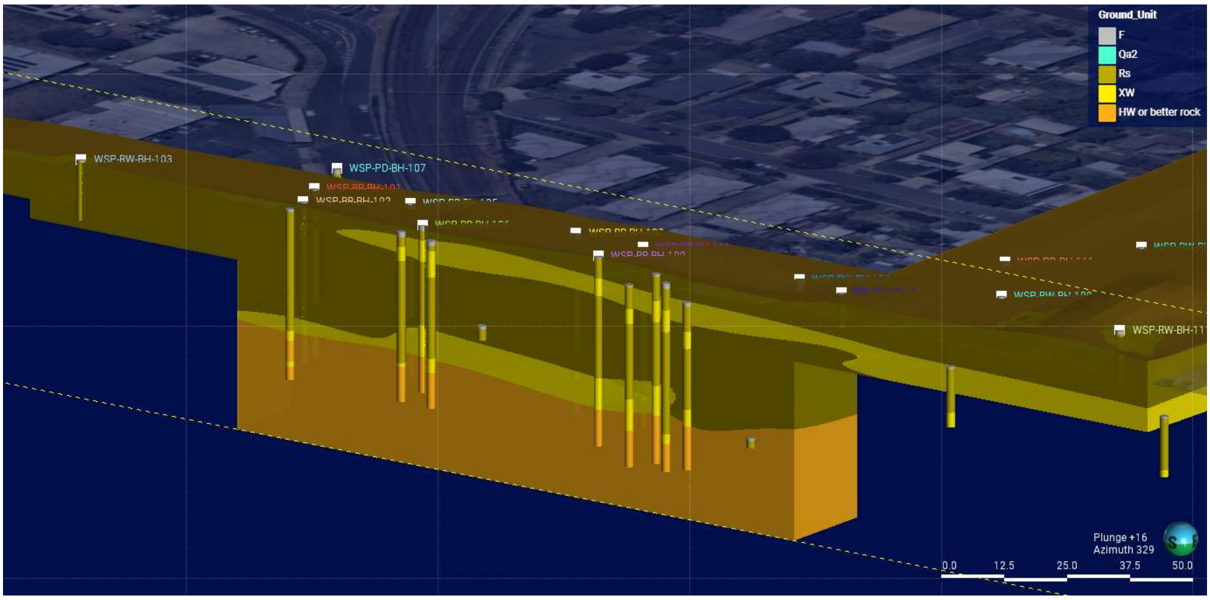

Figure 6: 3D Geological model of the site.

The geological sections along the bridge abutments and piers, and RSS walls were extracted from Leapfrog Works in 2D. These inferred geological sections formed the basis of the geotechnical model for design. Figure 7 shows an example of a geological sections at one of the bridge abutments.

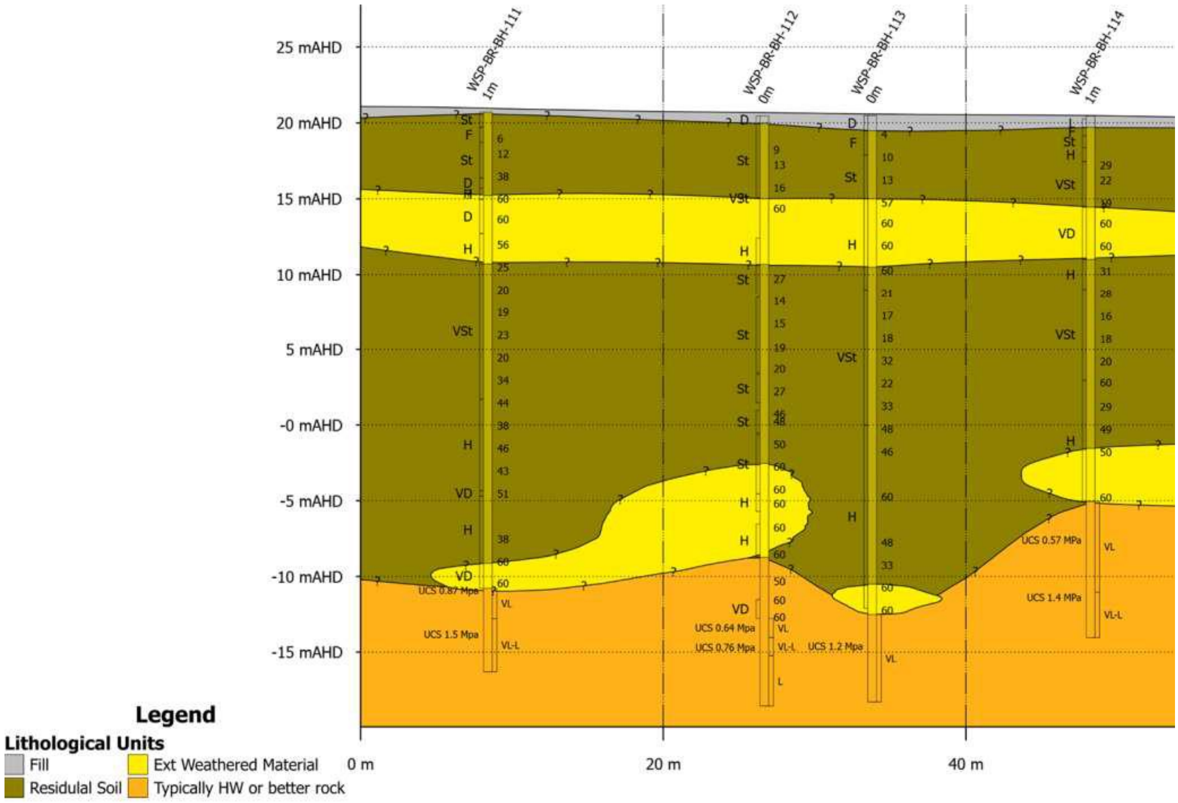

Figure 7: Inferred geological section through a bridge abutment from Leapfrog.

7 INTERPRETATION OF THE GI DATA

Accurate geotechnical interpretation of the encountered materials at the site is very important for a robust and economical design. Therefore, a hand-drawn geological section was developed to commence the design prior to the completion of laboratory testing and 3D modelling. These sketches in combination with SPT and core photography helped to confirm the lateral extent of both the cemented horizons near the surface and the very low to low strength rock at depth.

As part of the review of the field testing, it was observed that the SPT, N values indicated the materials in the upper layer (within top 3 m) to be of firm to stiff consistency. Considering the height of the approach embankments (up to 10 m), foundation improvement was required to minimise the differential settlements and satisfy the bearing capacity. Remove and replace was considered the economical option. However, considering the close proximity of rail track, buildings, and underground services; excavating up to 3 m to carry out remove and replace was not viable. Therefore, Dynamic Cone Penetration (DCP) test data available for the top 3 m was reviewed. A summary of the SPT, N and DCP values at each borehole location is presented in Table 2. The material consistency was interpreted from the SPT, N and DCP values as per HB 160 (AS 2006).

Table 2: Summary of SPT, N and DCP values with corresponding material consistency

BH ID	Depth (m bgl)	Material Type	SPT N value	SPT corelated Consistency	DCP blow count/100 mm penetration	DCP corelated Consistency
103	1	CLAY, high plasticity	11	Stiff	5	Very stiff
108	1	CLAY, high plasticity	8	Firm-Stiff	5	Very stiff
108	2.5	CLAY, high plasticity	12	Stiff	14	Hard
110	1.5	Sandy CLAY, high plasticity	2	Soft	7	Very stiff
111	1.5	Clay, low plasticity	6	Firm	8	Very stiff
112	2	CLAY, medium to high plasticity	9	Stiff	19	Hard
101	1.5	Silty CLAY, medium plasticity	8	Firm-Stiff	17	Hard

BH ID	Depth (m bgl)	Material Type	SPT N value	SPT corelated Consistency	DCP blow count/100 mm penetration	DCP corelated Consistency
102	1.5	Clay, medium plasticity	6	Firm	3	Stiff
103	1.5	Clay, high plasticity	5	Firm	9	Very stiff
105	1.5	Clay, high plasticity	9	Stiff	8	Very stiff
106	1.5	Clay, med plasticity	6	Firm	20+	Hard
108	1.5	Clay, low Plasticity	Refusal	Hard	6	Very stiff
109	1.5	Sandy CLAY low plasticity	Refusal	Hard	17	Hard
110	1.5	Clay medium to high plasticity	6	Firm	6	Very stiff
111	1.5	Sandy Clay low to medium Plasticity	8	Firm to stiff	7	Very stiff
112	1.5	Sandy Clay (low to medium plasticity	5	Firm	4	Stiff
113	1.5	Clay, high plasticity	15	Stiff to Very stiff	8	Very stiff

SPT, N values indicated the encountered ground to be firm to stiff consistency while the DCP values indicated the soils to be primarily very stiff to hard consistency with some localised stiff materials. It is envisaged that the materials may have lost strength due to ground disturbance during borehole drilling resulting in lower SPT, N values. As a result, the materials' consistency based on the DCP values was adopted in the design which helped to minimise the ground improvement requirement and saved construction time and cost.

8 GEOTECHNICAL DESIGN PARAMETERS

Once the field and laboratory tests were completed, the geotechnical design parameters were determined to complete the geotechnical design of the approach embankments, pile foundation, and retaining walls.

8.1 GEOTECHNICAL PARAMETERS FOR SOIL

8.1.1 Unit weight

Soil unit weight can typically be related to the consistency / density of the material. AS4678-2002 recommends a typical unit weight of the materials. A summary of the design values for bulk unit weight for various materials encountered at the site is presented in Table 3.

Table 3: Adopted unit weight of the soil/rock encountered

Materials	Consistency/Density/Strength	Unit Weight (kN/m^3)
Cohesive materials	Firm	18
	Stiff to very stiff	19
	Hard	20
Granular materials	Medium dense	19
	Dense	20
	Very dense	21
Extremely weathered materials	Hard/Very dense	21
Siltstone/Claystone/sandstone	Moderately to highly weathered, very low strength	21
	Slightly weathered to fresh, low strength	22

8.1.2 Undrained shear strength

The undrained shear strength (s_u) of cohesive materials was determined using the SPT, N values based on the relationship given by Terzaghi and Peck (1967).

$$s_u = 6 \times N \quad (kPa) \tag{1}$$

8.1.3 Effective strength parameters

The effective strength parameters (cohesion, c', and friction angle, ϕ') for cohesive soils were determined based on engineering judgement and correlations between plasticity index (I_p) and friction angle (ϕ') shown in Figure 8 obtained from AS4678-2002.

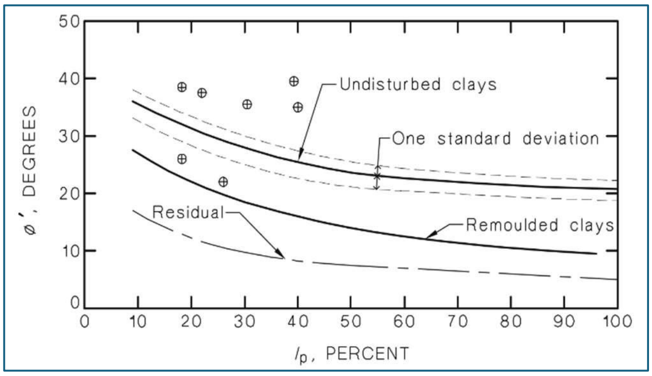

Figure 8: Correlation between ϕ' and plasticity index I_p (AS4678-2002).

The Atterberg Limit tests on the encountered cohesive materials indicated the Plasticity Index (I_p) in a range of 30 to 50 %.

The friction angle of cohesionless soils were determined using empirical correlations between SPT, N values and ϕ´, as indicated in published literature (Bowles 1997) and engineering judgment. In general, Equation 2 was used for determining the friction angle of cohesionless soils.

$$\emptyset' = (0.36 \times N) + 27 \tag{2}$$

Equation 2 may result in high ϕ' values for SPT completed within very dense granular materials. Therefore, a maximum friction angle (ϕ') of 39° has been used for very dense materials at this site.

Based on this, the recommended effective strength parameters for the encountered materials are summarised in Table 4.

Table 4: Recommended value of effective strength parameters

Materials	Consistency/Density	Effective Cohesion, c' (kPa)	Effective Friction, ϕ' (°)
Cohesive materials	Stiff	3	26
	Stiff to very stiff	4	28
	Hard	5	30
Granular materials	Medium dense	-	33
	Dense	-	36
	Very dense	-	39

8.1.4 Elastic modulus and Poisson's ratio

For cohesive soils, Duncan and Buchignani (1976) recommended that undrained Young's modulus (E_u) was linearly proportional to s_u and proposed the Equation 3.

$$E_u = Ks_u \quad (kPa) \tag{3}$$

Where K is the constant of proportionality and is a function of Plasticity Index (PI), and Overconsolidation Ratio (OCR) as shown in Figure 9. As the encountered soil has PI in a range of 30 to 50, K = 350 was adopted.

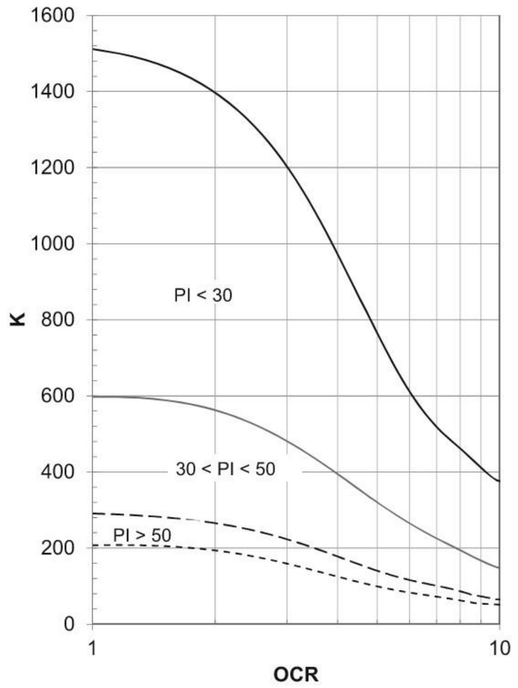

Figure 9: K-factor variation based on OCR (Duncan and Buchignani, 1976).

Drained elastic modulus (E’) for the cohesive soils were estimated from the E_u through an elastic relationship with the drained Poisson’s Ratio (ν'):

$$E' = \frac{2(1+\nu')}{3} E_u \tag{4}$$

For the cohesionless soils, a correlation between elastic modulus and SPT, N value was selected as follows, which is recommended for clayey sands (Clayton 1995):

$$E' = 2000 \times N \quad (kPa) \tag{5}$$

A Poisson’s ratio (ν) of 0.3 has been adopted for all soil and rock layers.

8.2 GEOTECHNICAL PARAMETERS FOR ROCK

8.2.1 Strength and weathering characteristics

The Sunnybank Formation includes weakly and moderating cemented units, interlayered with poorly consolidated materials. The formation was subject to deep (lateritic) weathering processes, with the weathering being both contemporaneous with and subsequent to its period of deposition. Rock recovered in the boreholes as slightly weathered and fresh, was still very low to low strength.

Generally, the extremely weathered material encountered in the boreholes was recovered as hard clay / sandy clay (inferred as residual soil and extremely weathered material) generally up to a depth of 24 m at Abutment A and to 30.5 m bgl at Abutment B. Below this depth, the rock (Siltstone, Sandstone, Claystone) was recovered as very low to low strength through to the termination depths of the boreholes.

8.2.2 Rock Strength

The bridge boreholes encountered extremely weathered materials with SPT refusal (N > 50) at shallow depth of around 5 m bgl and before rock coring at a depth between 24 m and 30.5 m bgl. Ignoring the contribution of this materials for pile skin resistance would have made the pile very deep and uneconomical. Therefore, equivalent uniaxial compressive strength (UCS) for this material was estimated as per the recommendations made in CIRIA

(1999). Figure 10 presents relationship between SPT and equivalent compressive strength for extremely weathered material with rock structure (Weak Rock).

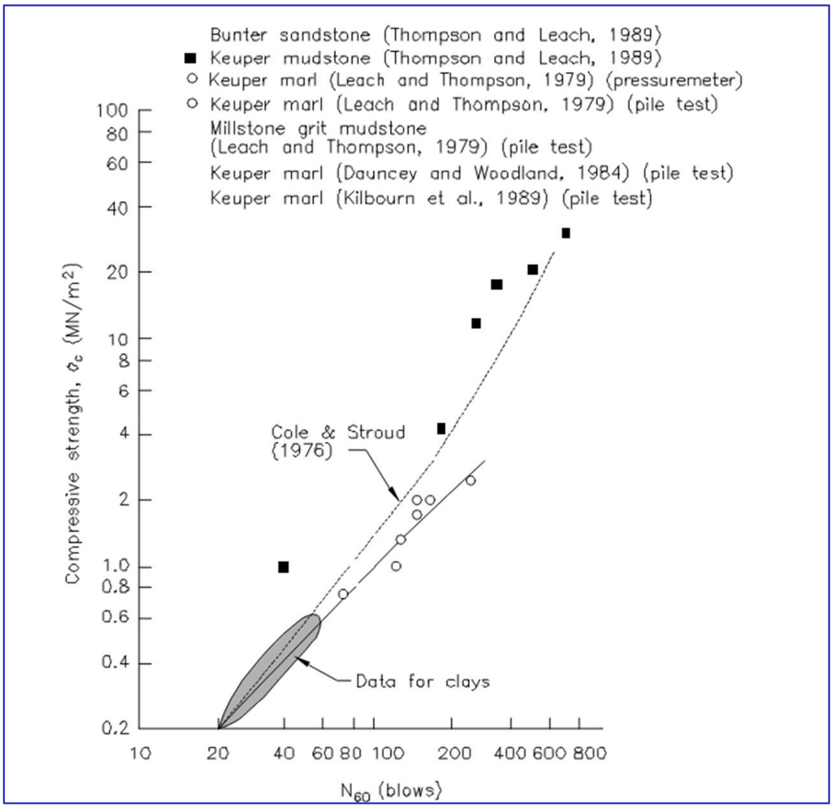

Figure 10: Correlations between SPT N_{60} and Compressive Strength of Weak Rock (CIRIA, 1999).

For the extremely weathered materials encountered at the site with lower bound SPT N of 60, a design UCS of 0.6 MPa was adopted.

The point load strength (Is_{50}) and UCS at respective depth from bridge boreholes were plotted and are presented in Figure 11. The line of best fit indicates a factor of 10 between Is_{50} and UCS (i.e. UCS = 10 x Is_{50}). The relationship is lower than the ratio suggested in AS1726-2017; however, it is in line with the relationship reported by Look (2007) for low strength rock.

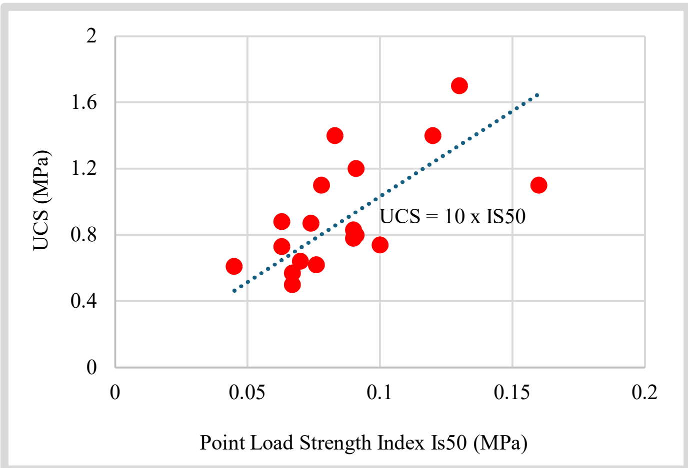

Figure 11: UCS vs Point Load Index strength correlation

This suggests that the inferred UCS of the rock encountered at the bridge site ranges between 0.6 to 1 MPa.

In addition, the UCS tests were plotted against depth at the bridge location and is shown in Figure 12. Based on the results of the UCS tests, it was found that the Rock UCS ranges between 0.6 MPa to 2.3 MPa. Moreover, the data are concentrated between 0.6 MPa and 0.9 MPa. Therefore, a design UCS of 0.75 was adopted for determining pile design parameters. This was consistent with the inferred values of UCS from Is_{50}.

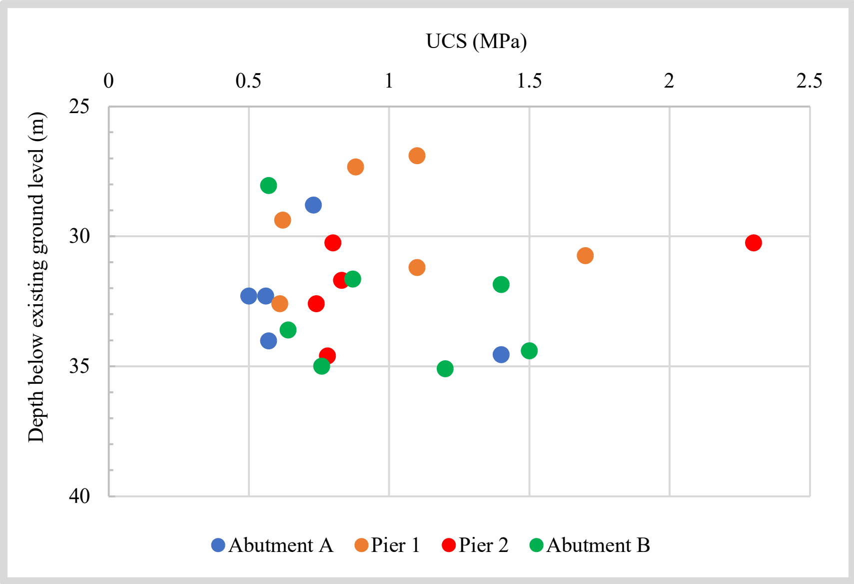

Figure 12: Variation of Rock UCS with Depth at Bridge Location

8.2.3 Pile design parameters

Based on the materials encountered at the bridge site, majority of the pile embedment is expected to be within the Residual Soil/Extremely weathered materials of very stiff to hard consistency and very low to low strength rock. The piles are expected to be terminated in very low to low strength rock.

For soil materials (i.e. residual soil and extremely weathered material), the ultimate skin friction (f_{su}) was determined using the following relationship:

$$f_{su} = \alpha \times s_u \quad (kPa) \tag{6}$$

Where α was obtained as recommended in MBIE (2014). From Figure 13 below, a value of 0.4 was adopted for the bored piles within the very stiff to hard residual soils.

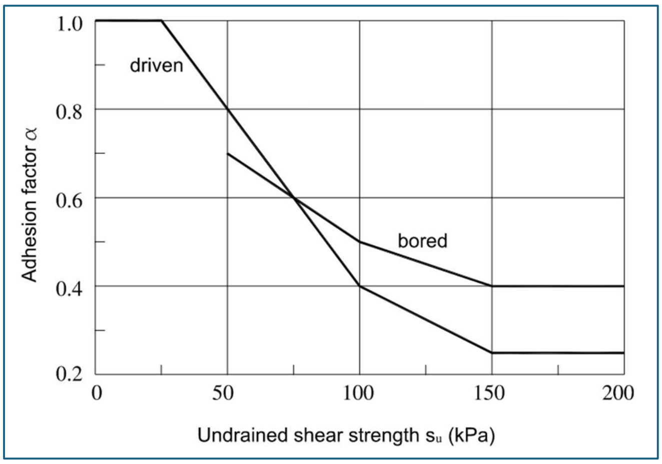

Figure 13: Adhesion factor for piles in cohesive soils (MBIE, 2014).

For very low to low strength rock, the f_{su} and ultimate base bearing (f_{bu}) were determined as per Zhang and Einstein (1998).

$$f_{su} = \beta \times \sqrt{UCS} \quad (MPa) \tag{7}$$

$$f_{bu} = \gamma \times \sqrt{UCS} \quad (MPa) \tag{8}$$

Where β varies between 0.3 and 0.8 depending on the rock socket roughness whereas γ ranges between 3 and 6.6. For robustness, β = 0.4 and γ = 4 was adopted for this site.

Based on the above, the recommended pile design parameters for the site are summarised in Table 5.

Table 5: Recommended pile design parameters

Materials Unit	Design s_u (kPa) or *UCS* (MPa)	Ultimate Skin Friction, f_{su} (kPa)	Ultimate Base Bearing, f_{bu} (kPa)
Residual (very stiff)	s_u = 190	75	-
Residual (hard)	s_u = 250	100	-
Extremely weathered materials	UCS = 0.6	300	-
Sandstone/Siltstone/Claystone (very low strength)	UCS = 0.75	350	3500

Rock mass modulus (E_r) is another important parameter for estimating pile settlement. For very low strength rock, the rock mass modulus was determined as per Rowe and Armitage (1987) using Equation 9.

$$E_r = 215 \times \sqrt{UCS} \quad (MPa) \tag{9}$$

The recommended value of E_r was 185 MPa.

9 CONSTRUCTABILITY CONSIDERATIONS

As the project is nearby a rail track and other structures, a steel liner to 10 m bgl has been recommended to provide pile hole stability. In addition, a recent project in similar materials encountered borehole instability due to groundwater and very low strength Sunnybank Formation rock, so a polymer supported pile hole has been proposed. The project is yet to be constructed, no construction issues have been discussed.

10 CONCLUSION

The paper presented the importance of good understanding of the site geology and having sufficient geotechnical investigation data to inform the detailed design of a railway overpass bridge and associated structures. Review of the existing data indicated the existing boreholes were limited and does not satisfy the project specification. Therefore, additional geotechnical investigation comprising of 14 bridge boreholes, 14 retaining wall boreholes, and 19 pavement boreholes was proposed and conducted to inform the design and minimise the construction risks. The bridge boreholes were drilled up to 40 m bgl, retaining wall boreholes were drilled up to 15 m bgl, and pavement boreholes were drilled to 2 m bgl. SPT tests were conducted at 1.5 m intervals to the top of rock. In addition, DCP tests were conducted at each borehole in upper 3 m or earlier refusal. Soil/rock samples retrieved were sent for laboratory testing.

The geotechnical investigation campaign required meticulous planning and managing stakeholders' expectations. The project being in urban setting the road was required to be opened during the day so most of the investigations were conducted during the night under traffic control to keep the road partially open.

Comparison of inferred soil strength from SPT, N and DCP values indicated reasonable discrepancy. The undrained shear strength of the soil in upper strata determined from the SPT, N values showed to be firm to stiff consistency requiring foundation improvement using remove and replace. However, the soil consistency based on the DCP tests indicated very stiff to hard consistency. As the DCP measures soil strength nearly in undisturbed state, it was decided to adopt in the design.

A 3D geological model was developed using Leapfrog to estimate the extent of soil/rock strata. Then 2D geological sections were produced to prepare a geotechnical model at each abutment, pier, and wall location. Field and laboratory test data were used to determine geotechnical design parameters using empirical correlations published in literatures and was recommended for detailed design.

The train track needed to be in operation and due to presence of buildings and services nearby the bridge site, steel liner was recommended to be screwed or inserted in pre-augured borehole to 10 m bgl. Moreover, polymer supported pile hole was advised considering the Sunnybank Formation may soften under water.

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