Geotechnical and Geophysical Site Characterisation 5 – Lehane, Acosta-Martínez & Kelly (Eds)
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The Smithfield Bypass Project – Justifiable Need for a Second Stage Piezocone Testing.

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ABSTRACT: The proposed Smithfield Bypass is a 4.15 km long dual carriageway spanning between Yorkeys Knob Road and Reed Road Roundabout in Cairns, Australia. On completion, the project will reduce traffic bottle necks currently experienced on the Captain Cook Highway and improve on the overall level of service of the highway. The proposed bypass will consist of two (2) new bridge structures over Avondale Creek, North and South, two (2) overpass bridge structures over Captain Cook Highway, five (5) major culverts, some retaining walls and associated motorway on and off ramps. Embankments heights are variable at the site reaching up to 9m at the approaches to the overpass structures. The project site traverses relatively flat alluvial coastal flood plain with thick sequence of alluvial deposits including up to 18m thick organic silty clays of high compressibility and low bearing strength. At the site, due to flooding concerns, there is only a 5-month window for ground improvement works. Using the coefficient of consolidation (Cv) values obtained from the first stage piezocone testing, the installation of perforated vertical drains (PVDs) and preload plus surcharge will have to be carried out to 100% of the southern section of the site with the exception of the at-grade sections. With improved c_v values obtained from a more careful and prolonged (occasionally overnight) second stage testing which ensured that 50% dissipations were reached, even on the basis of a low bound Cv value of 3m^2/yr., only 22% of the site would require surcharge and would meet the required 5-month target window for ground improvement.

1 INTRODUCTION

The proposed Smithfield Bypass is a 4.15km long dual carriageway spanning between Yorkeys Knob Road and Reed Road Roundabout in Cairns, Australia. On completion, the project will reduce traffic bottle necks currently experienced on the Captain Cook Highway and improve on the overall level of service of the highway.

The proposed bypass will consist of the following elements:

- overhead bridge structures, high embankments and retaining walls at McGregor roundabout and McGregor bypass connection road;
- two bridge structures and approach embankments over Avondale Creek;
- five major culvert structures;
- associated motorway on and off ramps; and
- at-grade and grade separated sections with variable embankment fill heights.

Based on existing information on the constructed Avondale Creek Bridge on the Captain Cook Highway located about 200m from the Smithfield Bypass site, the project area is on the Barron River flood plain with thick sequences of alluvial deposits. Soft to firm organic silty clays of up to 18m thick constitute a reasonable proportion of the alluvial deposits on the southern section between chainages 0m and 1650m (Ch. 0m and Ch. 1650m) of the alignment. These soft soils present potential stability and settlement risks to the project and need to be catered for in design as a means of ensuring that the strict in-service performance criteria on the project are met. Ground improvement has been identified as a viable option that could be used to manage long-term in-service settlement issues on the project but it has to be limited to a 5-month window due to flooding concerns. The flooding concerns and poor subsoil characteristics presented two major challenges to the project namely:

a) a reliable assessment of the properties of the subsoil such as in-situ shear strength, consolidation parameters such as compression and recompression indices, coefficient of consolidation, drainage path length, pre-consolidation pressure as well as thicknesses of the poor subsoil layers. A reliable estimate of these parameters will assist in the design of a competitive earthworks scheme;
b) how to construct the southern end of the bypass between Ch. 0m and Ch. 1650m, within the flood plain, as quickly as possible. The client had indi-

cated that should preloading due its cost competitiveness and proven efficiency compared to other ground improvement techniques be adopted as a means of ground improvement scheme on the project, that only preload can be left in place during periods of flooding. Should surcharge be required, as a means of facilitating settlement, they will be limited to a maximum duration of about 5 months during the non-flooding season (i.e. nominally May to October). With the formation level on the southern end of the project kept as low as possible in order to act as a weir to cater for flooding concerns and with poor subsoil thicknesses in the order of 18m to 20m the need for surcharge on the site becomes inevitable. This implies that early works (preload + surcharge construction) is anticipated to be carried out during the wet season.

Data obtained from the first stage piezocone testing at the site were considered unrealistic to enable a reasonable assessment of the consolidation characteristics of the compressible clay layer at the site. A second stage piezocone testing was embarked upon. The results of the second stage testing were very useful for the design of the ground improvement works and have been discussed in this paper.

2 SITE DESCRIPTION AND INVESTIGATION

2.1 *Site Description*

The locality and site plan showing the alignment of the proposed Smithfield Bypass is shown in Figures 1 and 2. It is located to the East of the Captain Cook Highway in the coastal plain. The coastal plain is mainly flat terrain, with alluvial deposits.

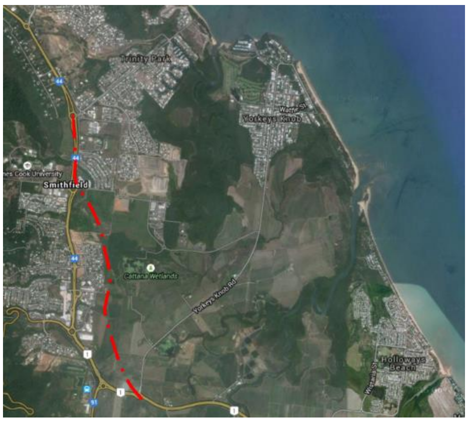

Figure 1. Google image of the site area with the proposed bypass alignment shown in broken line.

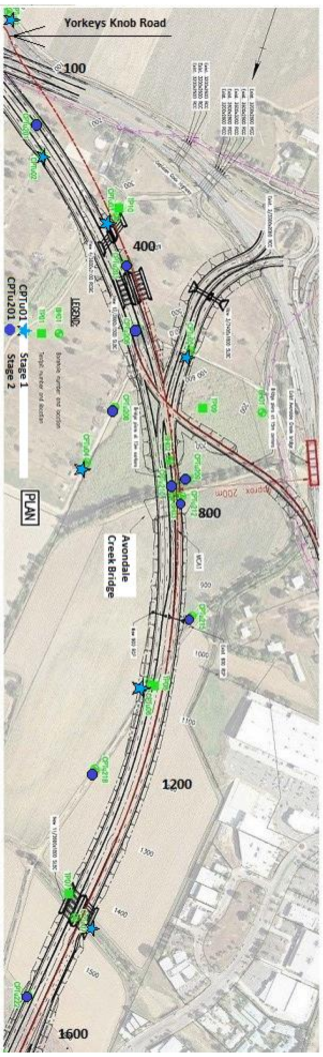

Figure 2. Site plan showing the extent of the project.

Figure 3. Site photo showing the relatively low elevation of the southern end of the site

The southern segment of the road alignment (Figure 2) from Yorkeys Knob roundabout (Ch. 0m) passes over Avondale Creek and continues to about Katana Road (Ch. 1600m) traversing the low lying Barron River delta, flood plain as shown in Figure 3. The ground elevation levels in this low lying area vary from RL 2m to RL 4m approximately. Vegetation in the flood plain consists mostly of grasses and commercially grown sugarcane.

Further to the north, the road alignment traverses gently undulating ground of elevation generally increasing from RL 4m to around RL 13m at the McGregor

Roundabout. The section of the site generally covered by moderate vegetation with medium to large sized trees, shrubs and grass and is outside the subject of this paper.

2.1 *Site Investigation*

The field investigation for this project was carried out in two (2) stages – Stages 1 and 2. The Stage 1 investigation consisted of the drilling of eight (8) number of boreholes at the proposed bridge structures, thirteen (13) number soil profiling using piezocones (CPTu) soundings and four (4) dissipation tests, the excavation of test pits and the execution of dynamic cone penetrometer (DCP) probing. The Stage 2 investigation consisted of ten (10) CPTu soundings and four (4) pore pressure dissipation tests to fill in the gaps in the Stage 1 investigation and most importantly, to re-evaluate the drainage characteristics of the clay layer due to perceived shortcomings in the results derived from the Stage 1 investigation.

The boreholes were drilled using a track mounted rig under the supervision of geotechnical staff from the Department of Transport and Main Roads (TMR) and generally involved the augering and casing of the first 3m followed by wash boring to completion of the hole. SPTs were undertaken at intervals of one metre for the first 5m and then at an intervals of 1.5m up to a maximum of 32m. From there onwards SPTs were undertaken at 3m intervals to termination depth. In one of the boreholes, field shear vane tests were alternated with U50 thin wall push tube undisturbed samples at 1.5m intervals.

The CPT's and CPTu's were undertaken mainly to establish the thickness of the soft to firm clay layer as well as the in-situ consolidation properties of the layer. The second stage CPTu was necessitated by the poorly executed dissipation tests in Stage 1 which led to drainage characteristics which were considered erroneous. A combined piezocone (CPTu) plot which gives an indication of the soft clay profile at the site is given in Figure 4. Plotted on the figure is also the over-consolidation ratio (OCR) as proposed by Chen and Mayne (1994):

$$\mathrm{OCR} = 0.32\left(\frac{q_c - \sigma_0}{\sigma'_0}\right) \qquad \text{.......... (1)}$$

where q_c = uncorrected cone resistance; σ_0 = total overburden stress; σ_0'= effective overburden stress.

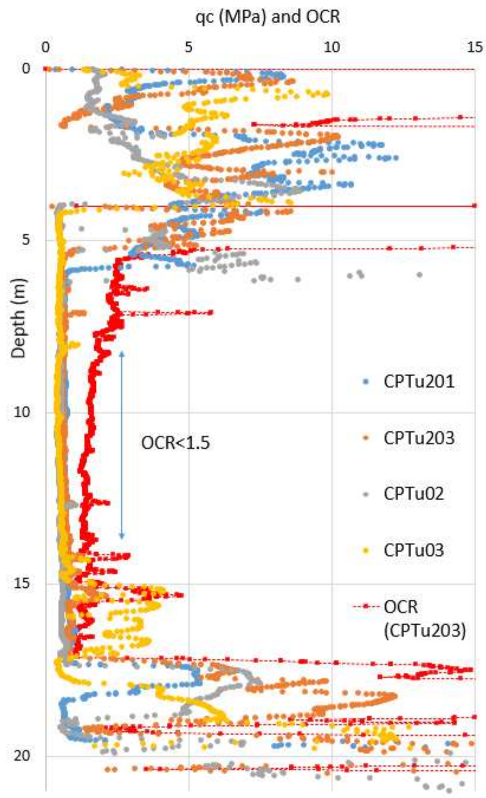

Figure 4. Combined plot of piezocone profiles between chainages 300m and 500m showing typical thickness of soft to firm clay at the Smithfield Project site.

Typical dissipation test results re-plotted in excel format from Stages 1 and 2 tests are shown in Fig. 5.

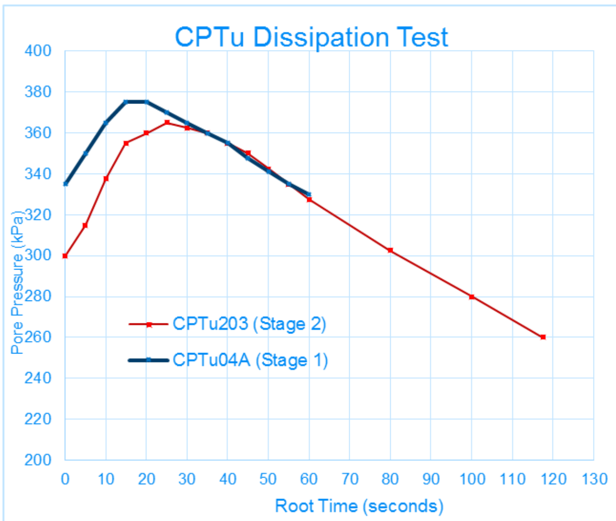

Figure 5. Typical dissipation test results from Stages 1 and 2 investigations.

3 GEOTECHNICAL MODEL AND KEY DESIGN PARAMETERS

The subsoil profile at the southern section of the project between Ch. 0m and Ch. 1650m consist mainly of deep deposits of alluvium. The maximum depth of the drilled holes during the investigation was 50m.

Even up to this depth, no bedrock was encountered. The top layer at the site consists of variable thickness of crustal layer which comprise of silty clay and loose sand mixtures. Underlying this layer is a soft to firm clay layer of variable thickness ranging from 9.5m to 18m with Standard Penetration Test (SPT) blow count generally less than 1 (SPT N <1). Within this thick clay layer a continuous sand lens with variable thickness (generally <1.5m) was encountered, especially between Ch. 550m and Ch. 700m. Underlying the consolidating clay layer is interbedded sand and silt of medium dense consistency but occasionally frim to stiff clay.

The consolidation characteristics of the clay layer namely, void ratio (e_0), compression index (Cc), recompression index (Cr), pre-consolidation pressure (Pc') and coefficient of consolidation (Cv) were established based on laboratory test results backed by correlations with published data. The coefficient of secondary compression (ε_α) was based on published correlation by Mesri (1973). Apart from the Cv values, the values of the other parameters from the two stages of investigations were in agreement. Given in Table 1 is a summary of the interpreted Cv values from Stages 1 and 2 investigations.

Table 1. Interpreted coefficient of consolidation (Cv) from Stages 1 and 2 investigations

S/No.	Test Location	Depth (m)	C_h (m^2/yr)	C_v (m^2/yr) k_h/k_v=2	C_v (m^2/yr) Average
	STAGE 1				
1	CPTu2A	12.3	1.92	0.96	1.03
2	CPTu3A	9.6	2.54	1.27	
3	CPTu4A	10.9	1.25	0.63	
4	CPTu7A	5.9	2.49	1.25	
	STAGE 2				
5	CPTu201a	7.0	8.4	4.2	2.58
6	CPTu203a	6.0	5.4	2.7	
7	CPTu203a	11.0	2.1	1.05	
8	CPTu209a	8.0	4.7	2.4	

As indicated earlier, the estimated value of Cv based on Stage 1 investigations were considered to be too low. The values were even lower than laboratory test values (1.42 to 2.46 m^2/yr) that were carried out on 50mm samples. They were deemed to be erroneous. As shown in Figure 5, the estimated Cv values were based on poorly conducted dissipation tests that never reached t50 values. Stage 2 tests were carried out to fill some gaps left in the first stage investigation and as a verification tool and were continued to ensure that t50 pore pressure dissipations were reached. In some cases, the dissipation continued overnight. The results were generally higher than the laboratory values without accounting for scale effect. Generally, it is widely accepted that actual (field operating) Cv values could be as high as 5 to 10 times those of laboratory values. The adopted Cv values based on Stage 1 test was 1.4m^2/yr, a median value of the test results from piezocone and laboratory test results. For the Stage 2, values ranging between 3m^2/yr and 5m^2/yr were adopted. The 3m^2/yr was considered lower bound in recognition of the sand lenses observed from the field data.

Summarized in Table 2 are the design parameters adopted for the consolidating clay layer for the Smithfield Project.

Table 2. Adopted design parameters for consolidating clay layer

Material	Unit Weight (kN/m^3)	Moisture Content (%)	Liquid Limit (%)	e_o	c_c
Soft to firm clay	18.0	*60 to 77	*60 to 97	1.82	0.7
	c_r	P_c' (kPa)	ε_α (%)	Cv (m^2/yr) Stage 1	Cv (m^2/yr) Stage 2
	0.07	100 to 110	0.008	1.4	3 to 5

Note: * Based on results from BH01

3 DESIGN CRITERIA FOR SETTLEMENT

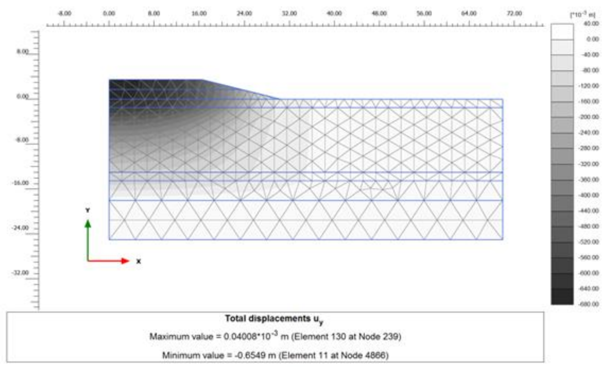

Figure 6. Estimated primary consolidation settlement between Ch. 550m and Ch. 700m.

4.1 Design Criteria

As per the Queensland Department of Transport and Main Roads Geotechnical Design Standard (GDS) - Minimum Requirements (2015), the following settlement criteria should be met to ensure minimisation of whole-of-life costs and maximise whole-of-life benefits for road pavements:

- maximum total in-service permissible settlement within 40 years of construction (post construction settlement – PCS) shall not be more than 50mm within

The structure zone and 200mm away from structure-zone. The structure zone is a length not less than 25m within the approach to any structure;

- the structure zone shall be constructed to the requirement of the GDS;
- design change in grade due to the differential settlement over any 5m length of pavement must be limited to 0.3% to 0.5%, depending on pavement type; and
- post-construction in-service movements must not impair or compromise pavement support.

3.1 *Design for Settlement*

3.1.1 *Construction Program*

Critical to the delivery of this project is the need to cater for hydraulic (afflux) issues that may arise due to embankment placement. The Smithfield Bypass project is located in the Barron River floodplain and strict controls are required to be put in place to minimise any impacts due to afflux.
To cater for hydraulic concerns, the vertical alignment between Ch. 0m and Ch. 550m has been lowered to enable this section of the bypass act as a “weir” and is overtopped in a flood event equal to or greater than an annual exceedance probability (AEP) = 20% (5yr ARI). Further to this, it is a requirement on the project to construct the southern end of the bypass (Ch. 0m to Ch. 1650m), within the flood plain, as quickly as possible. Any preload works can be left in place during periods of flooding, however, surcharge works are limited to a maximum duration of approximately 5 months (i.e. nominally May to October). To meet the May to October target, it implies that early works construction (preload + surcharge) has to be carried out in wet season and the surcharge removed before the flooding season.

3.1.2 *Design and Design Outcomes*

Initial estimates of the settlement magnitude and degree of consolidation were carried out using Terzaghi’s I-Dimensional consolidation theory implemented using excel spread sheet. The calculations were optimized in order to accommodate multiple soil layers efficiently as well as simulate staged construction procedure using Plaxis 2D (2014) and Settle 3D v. 3.0 (2009) softwares. Both the Mohr-Coulomb and soft soil creep advanced model in Plaxis were used in settlement estimates. The Mohr-Coulomb model was used to model the behavior of the embankment and sandy layers whereas the soft soil creep model was used to model the behavior of the compressible clay layer. The results of the analyses are summarized in Table 3 and Figures 6 and 7.

Table 3. Summary of the results of the analyses

Chainage (m)	**Max. Embankment Height (m)**	**Stress History (Expected Settlement)**	**Estimated total settlement Primary (Creep*)**
0 to 300	At - Grade	-	-
300 to 550	2.5	POP << ΔP (High)	470 (153)
550 to 700	3.5	POP << ΔP (High)	605 (191)
700 to 800	Bridge over Avondale Creek		
800 to 1060	1.6	POP < ΔP (Low)	157 (211)
1060 to 1550	1.3	POP > ΔP (Negligible)	-
1550 to 1650	At - Grade	-	-

Chainage (m)	Maximum Embankment Height (m)	Time in months to achieve 90% consolidation under preload $C_v\ (\frac{m^2}{year})$			Time (months to achieve 90% consolidation under preload + PVD $C_v\ (\frac{m^2}{year})$		
		1.4	3	5	1.4	3	5
0 to 300	At - Grade		-	-	-	-	-
300 to 550	2.5	>5	>5	>5	>5	<5	<5
550 to 700	3.5	>5	>5	>5	>5	>5	<5
700 to 800	Bridge over Avondale Creek						
800 to 1060	1.6	>5	>5	>5	>5	<5	<5
1060 to 1550	1.3	-	-	-	-	-	-
1550 to 1650	At - Grade	-	-	-	-	-	-

Note: *Creep settlement within 40 year service life.

In the light of the client request to limit surcharge on the project to no more than 5 months and considering the settlement criterion on the project, the following deductions are made from the results of the analyses.

a) In order to satisfy the permissible post construction settlement criterion on the project, ground improvement is required;

b) With only the preload in place, 90% consolidation settlement will not be achieved within 5 months;
c) On the basis of the adopted c_v value obtained from the 1st Stage site investigation, to meet the 5 months window allowed for ground improvements, surcharge in addition to the use of PVD and preload will be required for a 660m length section of the site with embankment heights ranging from 1.6m to 3.5m;
d) On the basis of the revised Cv that was obtained from a more realistic dissipation test in the Stage 2 investigation, the use of surcharge is limited to the 150m length between Ch. 550m and Ch. 700m for a lower bound Cv of 3m^2/yr. This equates to about 22% of the 660m length of the site that are not at-grade. Should the operating c_v be up to 5m^2/yr, the 90% consolidation target will be achieved without the need for surcharge for the 660m length of the project;
e) Considering the cost of re-mobilizing and carrying out additional testing at the site for the second stage testing and the cost implication of limiting the area to be surcharged that resulted from the revised Cv values, there is significant cost benefit to the project. The second stage testing is therefore justifiable;
f) As part of a validation tool on the project, the use of observational approach is recommended to be implemented and monitored before, during and after construction.

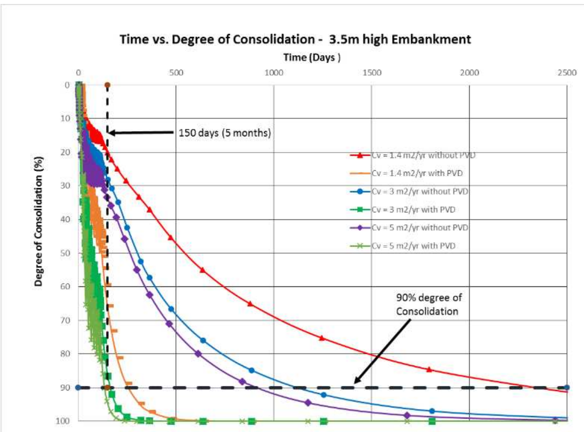

Figure 7. Estimated degree of consolidation with and without PVD for varying coefficient of consolidation (Cv) between Ch. 550m and Ch. 700m.

4 CONCLUSIONS

Due to other competing demands in government spending, funding for road projects do not come so easily in recent times. Where poor subsoils of low bearing capacity and compressibility are encountered and stringent settlement criteria are to be met, cost of ground improvement works add on to the overall cost of the project. The Smithfield Bypass project falls into this category due to additional cost imposed on the project by poor subsoil consisting of 9.5m to 18m thick very soft to firm compressible clays.

A re-appraisal of the coefficient of consolidation (Cv) obtained from the 1st Stage investigation carried out on this project and used in the initial estimates of the ground improvement works was carried out due to perceived inconsistencies in the test results. A 2nd Stage investigation aimed at establishing a more realistic Cv was carried out and the obtained values used in further analyses.

Based on the 1st Stage Cv = 1.4m^2/yr, the recommended ground improvement works at the project site will consist of the installation of PVDs, preload plus surcharge and would have been carried out to 100% of the southern section of the site except for the at-grade sections. The total length of the southern section requiring ground improvement is equal to 660m. Using the revised Cv values, on the basis of a lower bound Cv of 3m^2/yr, only 22% section of the 660m length would require surcharge. With a Cv of 5m^2/yr, only the PVD and preload is required without surcharge to achieve a 90% consolidation within the required 5 month period imposed on the project by flooding risks.

The revised Cv on the basis of the 2nd Stage investigation has significant cost benefit for the project.

5 ACKNOWLEDGEMENTS

The author would like to express his gratitude to his colleagues in the Geotechnical Engineering Section of the Department of Transport and Main Roads, especially the Director Lalith Welikala and Engineers Siva Sivakumar and Dr Preba Nadarajah who reviewed the geotechnical design report from which the contents of this paper were largely drawn from. Likewise my thanks also goes to Engineers Devinder Pal, Jeremy Kirjan, Mali Yagama, Michael Sun and Vernon Pereira, Regional Geologist, TMR, Far North, Region for their assistance in the preparation of the technical report and this paper.

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