DESIGN OF A PIPELINE PROTECTION STRUCTURE OVER COMPRESSIBLE GROUND

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ABSTRACT

This paper outlines the main geotechnical challenges associated with the protection of an existing major oil pipeline due to construction of an arterial highway within an area of compressible soils.

A detailed ground characterisation was carried out to understand the performance of foundation soils under embankment and traffic loading. This consists of a thorough interpretation of shear strength and consolidation characteristics to inform the design of a piled concrete slab protection structure. The design methodology was developed with the following three (3) key project drivers in mind:

- A solution that adopts piles to act as "settlement reducers" instead of a rigid piled alternative. The benefits of this approach are viewed through the lens of eliminating the formation of a so-called "hard point" within the road alignment measured at the pavement level.
- Mitigation of embankment fill stresses directly impacting on the performance of the pipeline.
- From a project perspective, the design aimed at achieving potential cost savings during the construction phase.

A 3D finite element model was developed in PLAXIS 3D to model the soil-structure interaction, coupled with a pipe stress analysis in ABAQUS 3D to model the soil-pile-pipe interaction and pipeline performance during primary and secondary consolidation.

1 INTRODUCTION

The current trend in Australia is to build additional capacity within the existing road network through re-development over suburban land. This will achieve improved transit times for communities, as well as ensuring the uninterrupted freight of goods between the main capital cities. Many of these future road developments will involve building high fill embankments over existing critical hydrocarbon pipeline routes which provide a supply of hydrocarbon for many industrial uses within the urban fringe of cities. The extent of the protection structure and pipeline alignment is shown in Figure 1.

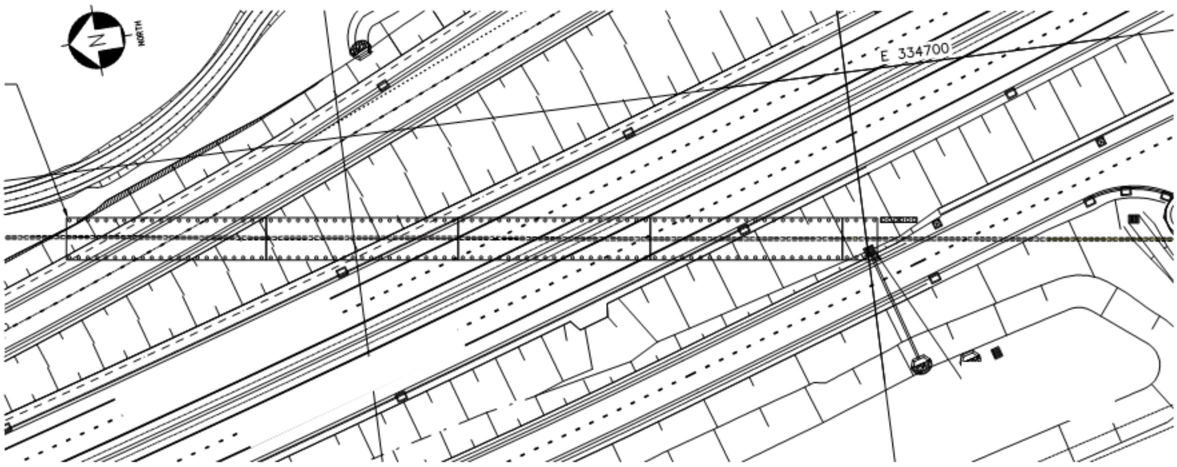

Figure 1: Alignment of existing pipeline and proposed freeway upgrade

The proposed road alignment consists of a dual carriageway, four lane highway, with fill embankments up to 7.5m high. The proposed pipeline protection structure consists of a concrete slab spanning across the buried pipeline, to be supported by concrete piles. The slab shields the dead and live loads from the pipe and it also minimises the associated ground movement that could affect the pipe.

Prior to the construction of the protection structure, it was required that the affected section of the pipeline be exposed, inspected and recoated. The old coal tar enamel coating was to be removed, and a new high-build epoxy coating (HBE 95 or equivalent) would be applied. This new coating has the capacity to tolerate a higher vibration level arising from the piling and nearby construction activities.

2 GEOETCHNICAL CHARACTERSATION

3.1 REGIONAL GEOLOGY AND ENGINEERING GEOLOGICAL CROSS SECTION

The location of the existing oil pipeline sits within a complex and varied geological setting of broad downthrown areas bounded by uplifted fault blocks. These geological features are presented in Figure 2.

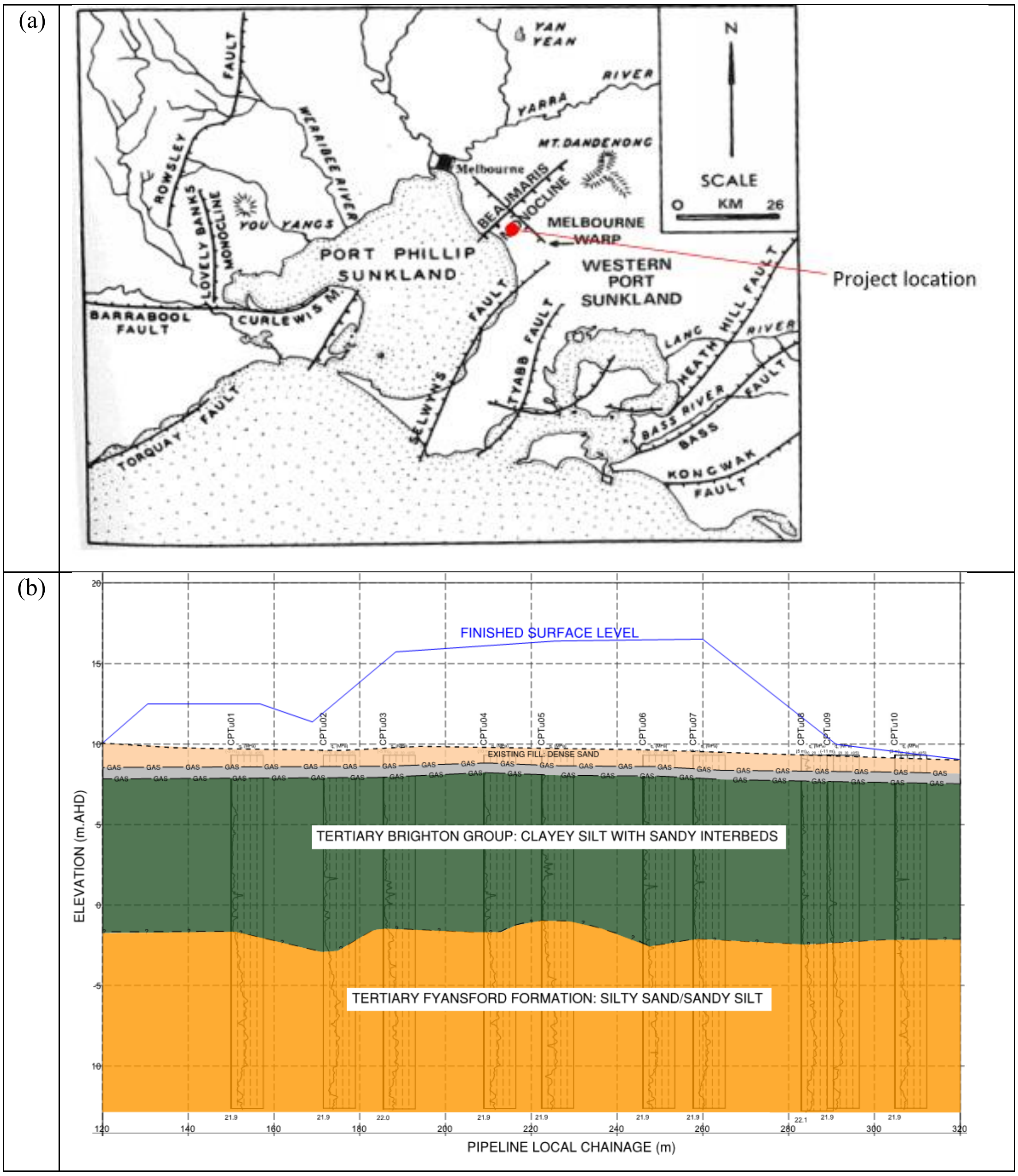

Figure 2: Published geological map and geological cross section, (a) project location, (b) finished surface level

The sedimentary soils developed in a depositional environment that was affected both by local tectonism and eustatic sea-level change, which resulted in a general fall in sea level and associated change from shallow marine to littoral to terrestrial environments.

Based on published geological maps, the subsurface conditions for the site are shown to comprise Quaternary-aged swamp, lagoon and wind-deposited dune sediments. These sediments overlie Neogene Period sediments assigned to the Brighton Group which comprises the Red Bluff Sandstone and Beaumaris Sandstone. Underlying the Brighton Group is the Fyansford Formation, which in turn is underlain by the Silurian Period Melbourne Formation.

An intrusive CPTu testing regime was carried out adjacent to the existing pipeline structure. The interpreted geological long section along the pipeline alignment is shown in Figure 2.

3.2 BASIC SOIL INDEX AND MATERIAL BEHAVIOUR

The behaviour of the Tertiary Brighton Group Clays and Silts were assessed to be critical to the deformation of the pipeline due to the embankment fill loading. The variation of soil index properties is an important consideration for understanding the general behaviour of soil materials. Atterberg limits for the samples taken in the Brighton Group were plotted on a Casagrande Plasticity Chart. In general, this material mostly behaves as a low to medium plasticity CLAY or SILT, although, some higher plasticity samples were also recovered. The material classification according to the USCS was compared to typical CPTu soundings carried out within the project footprint. The interpretation is presented in Figure 3 below for comparative purposes.

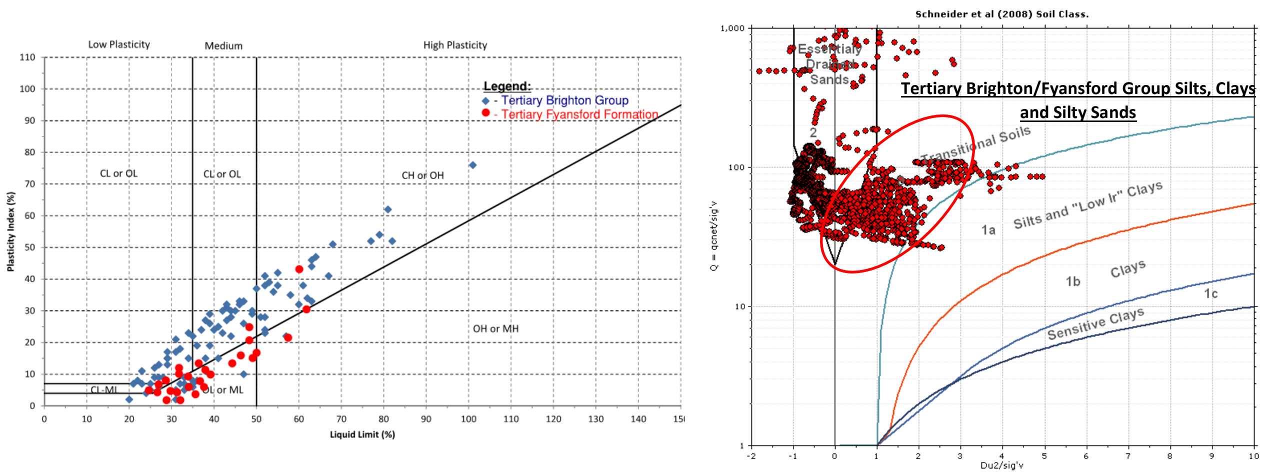

Figure 3: General soil index and behaviour

The Tertiary Brighton Group soil is clearly a transitional material but is likely to manifest in undrained behaviour. Subsequent sections discuss the interpretation of undrained shear strength, deformation and consolidation characteristics of the Tertiary Group Clayey Silts. For the underlying Fyansford Formation, the CPTu interpretation skews to the conclusion that this soil unit behaves mostly as a silty SAND mainly due to the normalised pore pressure ratio, B_q. However, the extensive laboratory testing campaign for the wider project footprint suggests that this material has a high fines content (>30%) and may also behave in an undrained manner. Subsequent foundation assessments have assumed that both soil units tend to behave as undrained, fine grained soils. Groundwater at 1.0m below existing ground level was considered in the following interpretations.

3.3 UNDRAINED SHEAR STRENGTH, DEFORMATION AND CONSOLIDATION PROPERTIES

Undrained shear strength (s_u) has been derived using the CPTu testing carried out within the project footprint. For the Tertiary Brighton Group soil unit, undrained shear strengths were determined using location specific CPTs, with s_u estimated by adopting the correlation by Lunne et. al. (1997):

$$s_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{1}$$

where:

where:	q_t	=	Corrected cone resistance
	σ_{vo}	=	Total overburden pressure
	N_{kt}	=	Cone factor

An empirical methodology was proposed by Ladd (1991) to provide a platform for a unified framework of correlation of soil strength with stress levels and soil compressibility characteristics. Mathematically such a relationship may be expressed as:

$$\frac{s_u}{\sigma_v'} = S \times OCR^m \tag{2}$$

where:

- OCR is the over-consolidation ratio of the soil.
- σ_v' is the vertical effective stress; and
- S and m are empirical derived coefficients to consider the effect of clay plasticity and soil compressibility characteristics. The relationships proposed to derive these coefficients are as follows:

$$S = 0.2 + 0.05 \times PI \text{ or simply } 0.21 \text{ for PI} = 20\% \tag{3}$$

$$m = 0.88\left(1 - \frac{C_r}{C_c}\right) \text{or simply } 0.8 \tag{4}$$

In-situ Marchetti Dilatometer readings were carried out to calibrate the in-situ CPTu cone factor. The adopted N_{kt} value of 25 was found to calibrate well with the dilatometer results. It is noted that the N_{kt} = 25 adopted for this project is on the higher end of published data by Lunne (1997) and other researchers. This suggests that the soil is likely to be lightly cemented or structured. This conclusion is in line with the documented geological history of the site, as well as interpretation of previous stress history (by assessing the over-consolidation ratio), discussed in Section 3.4. Deformation properties have been interpreted from laboratory testing and in-situ testing CPTu testing. The interpreted strength and deformation parameters are presented below in Figure 4:

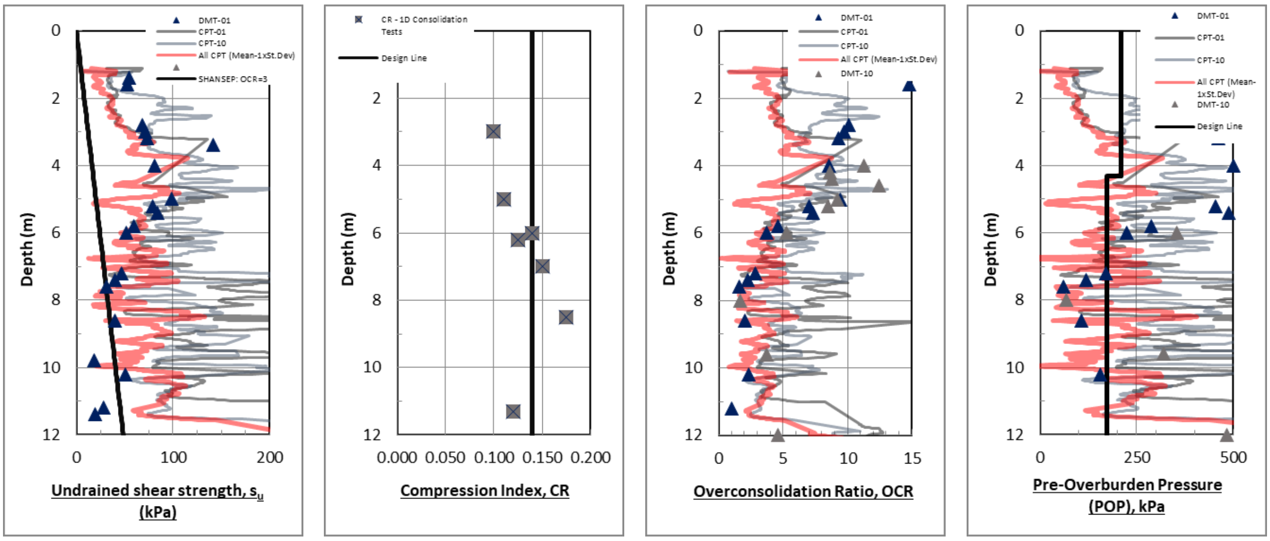

Figure 4: Interpretation of geotechnical parameters

- CR = Compression Index. Soil compressibility interpreted from Oedometer test results on undisturbed soil samples.
- CRR = Recompression index, adopted as 0.15xCR
- Cαe = Creep Index, adopted as 0.04xCR adopted based on creep consolidation testing and previous project experience.
- OCR = kQ_{t1} where Q_{t1} is the normalised CPT cone resistance and k = 0.15 (calibrated using in-situ DMT and oedometer results).

3.4 SUMMARY OF GEOTECHNICAL PARAMETERS

The adopted geotechnical parameters for the pipeline protection structure are summarised below in Table 1:

Table 1: Summary of geotechnical design parameters

Soil Unit	Depth to base of unit (m)	Bulk Unit Weight, γ_b (kN/m^3)	Undrained shear strength, s_u (kPa)	Drained friction angle, ϕ' (deg.)	CR	CRR	Secant modulus, E_{50}^{ref} (MPa)	Pre-overburden pressure, POP (kPa)	Vertical permeability, k_v (m/day)
Existing Fill	1.2	20	-	34	-	-	30	-	8.64
Tertiary Brighton Clay/Silt – Upper	4.3	18	50	-	0.14	0.042	-	210	0.0007
Tertiary Brighton Clay/Silt –Lower	12.0	18	50	-	0.14	0.042	-	172	0.0004
Tertiary Fyansford Formation	-	19	150	-	-	-	30	-	0.0008

3 FOUNDATION DESIGN

3.1 DESIGN PHILOSOPHY

The design of the pipeline protection structures forms a small part of the overall project upgrade, that is, the design of a major highway and all its associated works. As such, an important consideration for this design package is the interaction of the pipeline protection structure with the overall road project. The following key design considerations were of utmost importance when designing the protection structure:

1. Ensuring that the protection structure does not form a significant “hard point” along the final pavement level;
2. Accommodating fill long term settlement (residual primary consolidation and creep settlement) of embankments adjacent to the protection structure;
3. Cost effective design; and
4. Minimising pipe stress within the existing pipeline to acceptable limits.

With these design intents it was decided (through interaction of consultants and client’s engineers) that the piles would be designed to behave more like “settlement reducers” rather than rigid piles (bridge abutment piles, for example). This approach proves to be very cost effective in this scenario as the structure is around 200m long and pile lengths could be significantly reduced. When adopting this approach however, it is prudent that detailed numerical analysis is undertaken to model the soil-structure-pipeline interaction. Subsequent sections discuss these assessments in greater detail.

3.2 DESIGN LOADS, STRUCTURE CONFIGURATION AND PILE FOUNDING LEVELS

The protection structure was designed by the structural discipline to minimise the embankment fill stresses developing large deformations within the foundation soils under the pipeline. The design entails the construction of a concrete slab supported by piles on either side along its entire alignment. It was proposed to adopt the Contiguous Flight Auger (CFA) piling technique to install:

- 600mm diameter piles under the fill embankment founded on the Tertiary Fyansford Formation Very Stiff to Hard Clay Unit; and

- 750mm diameter piles under the embankment sides to accommodate the lateral foundation loading.

A cross section of the proposed protection structure is shown below in Figure 5, with design loading summarised in Table 2:

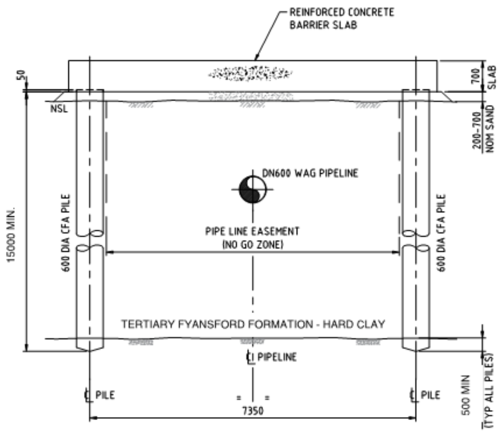

Figure 5: Proposed pipeline protection structure cross section

Table 2: Summary of structural loads

Load Case	Main Piles under Full Height
ULS Axial Force (kN)	1390
ULS Bending Moment (kN.m)	510
ULS Shear Force (kN)	470

3.3 COMPARISON OF SIMPLIFIED α METHOD AND CPT LCPC METHOD

The anticipated Factor of Safety (FoS) of the piles under compressive loading was assessed using the simplified approach and compared to the LCPC CPT method (Bustamante, M., and L. Gianeeselli, 1982). The comparison for 600mm CFA piles and the associated Factor of Safety is presented in Figure 6.

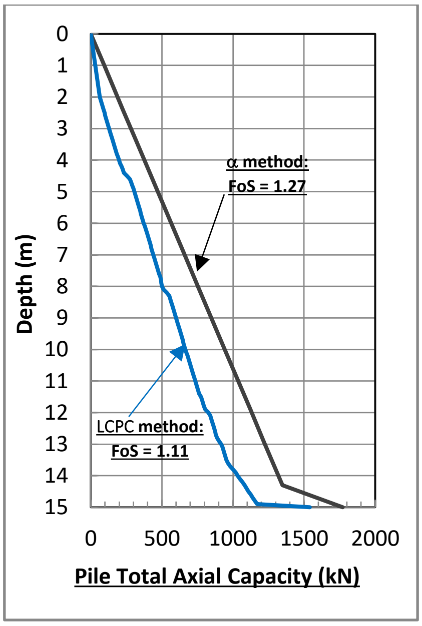

Figure 6: Comparison of α and LCPC method for pile geotechnical capacity

The contribution of end bearing (Q_b) is calculated based on $9xs_u$ for the Tertiary Fyansford formation, for both methods. It was found that the CPT method generally produced very high values of end bearing in this material, most likely due to its cementation. It is anticipated that these cementitious bonds will be destroyed within the region where effective end bearing can manifest, as a result of the CFA piling technique. On the contrary, the LCPC method predicted a lower shaft adhesion contribution compared to the α method, even though a conservative unified undrained shear strength of 50kPa was adopted for the Tertiary Brighton Group Clays.

4 NUMERICAL ANALYSIS

4.1 PIPELINE DEFORMATION ANALYSIS AND SOIL STRUCTURE INTERACTION: PLAXIS 3D

The pipeline settlement due to embankment loading and traffic loads over the proposed protection structure have been assessed using a full soil-structure interaction model using commercial software PLAXIS 3D. The model considers the following effects:

The 3D model mesh consists of 105253 finite elements and 158541 nodes. 10 nodded tetrahedral finite elements were used in the model. The model is normally fixed on all boundaries, except the maximum vertical boundary (fully free in all directions) and the minimum vertical boundary (fully fixed in all directions). The soft soil constitutive model was used to model the deformation of compressible soils, with parameters summarised in Table 1 adopted in the analysis.

The concrete slab was modelled as a plate element and CFA piles modelled using embedded beam rows with axial skin and base resistance derived according to the LCPC method for axial skin resistance and $9s_u$ for base resistance;

The staged construction loading conditions are summarised as follows:

1. K_0 procedure to determine in-situ stress conditions;
2. Installation of protection structure (plastic load step);
3. Embankment fill construction with a maximum fill thickness of 7.5m (plastic load step);
4. Application of a 10kPa traffic load (plastic load step); and
5. Consolidation of excess pore pressure (up to 95% degree of consolidation) to model the long-term primary consolidation of the pipeline and protection structure.

The 3D finite element model of the pipeline, the protection structure and the proposed embankment loading is shown in Figure 7.

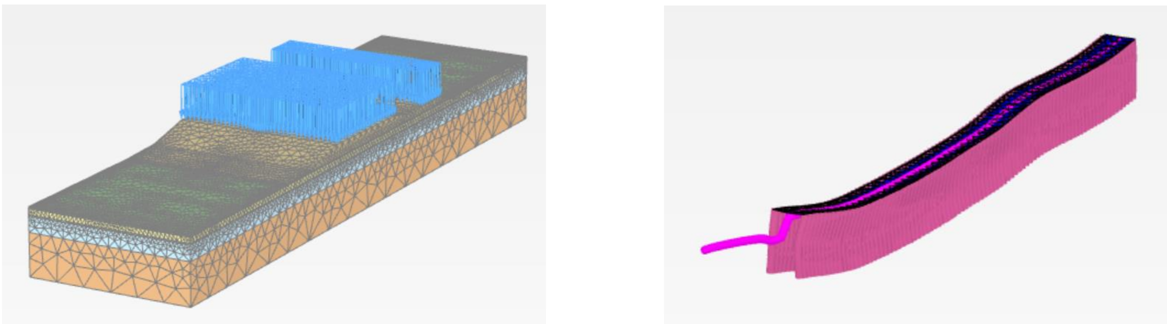

Figure 7: PLAXIS 3D soil-structure interaction model

The long-term secondary consolidation (creep) was assessed separately using a calculation methodology developed by Wong (2007), which was found to fit reasonably well based on oedometer testing results for the Tertiary Brighton Group Clays.

The results of settlement calculations to predicted long-term vertical deformation of the pipeline (including creep effects) is presented in Figure 8. The settlement diagram predicts a gradual change in settlement along the pipeline alignment. A maximum of 120mm settlement is anticipated to develop at pipeline chainage 210m.

The results of the 3D finite element analysis were validated with a settlement calculation using Terzaghi's One Dimensional consolidation theory. Due to the semi-rigid pipeline protection structure, it is difficult to ascertain exactly how much load is transferred to the soil. An estimate of 20% load transfer was considered, based on experience with load transfer to soil from concrete injected columns.

The magnitude of settlement assessed using 1D consolidation theory was **133 mm** for the 7.5m fill thickness and 10 kPa traffic load, assuming a 20% transfer of load to the soil. This demonstrates a reasonably fair comparison between the finite element analysis and the 1D hand calculation and validates the 3D finite element analysis.

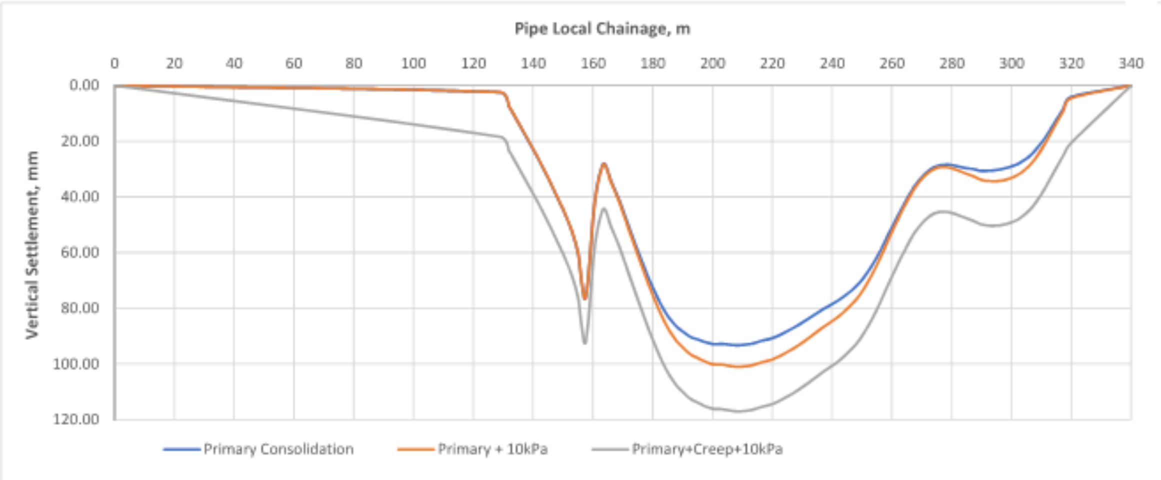

Figure 8: Vertical settlement profile of pipeline

The pipeline deformation profile is provided as input for the pipeline stress analysis for capacity checks. Of particular importance for pipeline stress analyses will be the sudden change in deformation gradient at pipeline chainage 160m. The sudden increase in settlement is caused by interfacing issues with the Freeway off-ram ramp which caused the protection slab to finish early under the embankment batter. A provision was only made to retain the fill batter with 750mm diameter CFA piles to restrict the lateral deformations impacting on the pipeline.

4.2 PIPELINE STRESS ANALYSIS: ABAQUS

The 3D ground displacement profiles computed in PLAXIS 3D were applied as prescribed displacement input to a Pipe-Soil-Interaction (PSI) model in Abaqus to determine the stress state in the pipe. The pipe element in this model can account for the internal pressure, thermal effects, Poisson's ratio effect of a restrained pipe, and soil deformation. The pipe-soil interaction was represented by a series of bi-linear springs along the pipe. The methodology and the advantages of using this approach to analyse pipe stress caused by ground deformation are detailed in Ho and Dominish (2005).

A typical pipe stress plot is shown in Figure 9. The computed pipe stresses were assessed against the requirements in AS 2885.1 to ensure they are compliant.

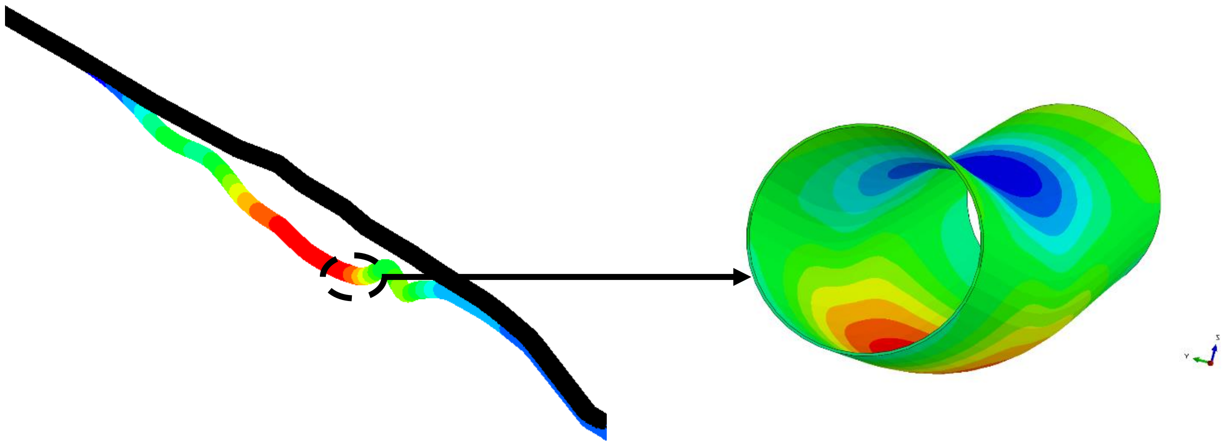

Figure 9: Results of pipe stress analysis

5 DISCUSSION AND CONCLUSIONS

In line with the above findings, the following recommendations were provided for consideration of the construction team:

- The design of the piled foundation system was found to limit the settlement development at the finished pavement surface level to alleviate concerns of potentially creating a "hard point" to exceed the grade change requirements outlined project scope of works at technical criteria;
- This project was delivered in close collaboration with the structural team to design the appropriate connection between the piles and concrete protection slab. A number of iterations were conducted with final design to include a "pinned" connection to prevent development of large shear stresses and bending moments exceeding the pile structural capacities;
- Pre-fabricated steel connections were specifically designed for this project to bring an innovative alternative to address the pile structural design considerations;
- The assessment of the piled protection structure on the basis of "settlement reducers" resulted in a cost-effective design for such a large structure. This design approach was found to eliminate the need for additional ground treatment at transition zones between the protection structure and general embankments;
- A detailed numerical analysis has combined the computational efficiencies of a finite element model (PLAXIS 3D) to predict the deformations of foundation soil materials, with a specialist pipeline stress analysis model carried out in Abaqus 3D.

6 ACKNOWLEDGEMENTS

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