The paper was published in the proceedings of the 12th Australia New Zealand Conference on Geomechanics and was edited by Graham Ramsey. The conference was held in Wellington, New Zealand, 22-25 February 2015.

Geotechnical design of soft ground conditions

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ABSTRACT

Understanding of ground conditions and soil behaviour of soft highly compressible silts and peats in terms of deformations and shear strength is fundamental for the geotechnical design including their associated effects. This article presents the geotechnical design approach for soft ground conditions at the Domain Road Interchange of the Tauranga Eastern Link Project.
Significant geotechnical features are 6m thick soft peats underlain by a paleo channel comprising up to 17m thick soft estuarine silts and loose liquefiable sands. Preliminary design and monitoring records from previous nearby construction projects suggested that more than 4m construction induced settlements were to be expected.
A complex network of two bridges, three large EPS (expanded polystyrene) embankments and two roundabouts replace the existing road layout at State Highway 2. Smart construction staging was necessary to avoid damage caused by drag settlements to the existing and newly constructed infrastructure.
Main focus of this paper is the strength gain of the soft silt deposits as a result of increased overburden pressure using the *SHANSEP* (Stress History and Normalised Soil Engineering Properties) design approach. The strength gain of the normally consolidated silt deposits was considered as a function of the temporarily placed surcharges and the increased effective vertical stresses. Theoretical analyses and monitoring of actual settlements were conducted ensuring the required surcharge heights and periods meet the design and construction requirements.
Design limitations and review of construction observations will also be presented, as well as a brief overview of the adopted ground improvements and the EPS embankments.

Keywords: Tauranga Eastern Link, Domain Road Interchange, Ground Improvement, Soft Ground, SHANSEP

1 PROJECT OVERVIEW

The Tauranga Eastern Link (TEL) project is a four-lane 23km long NZTA (New Zealand Transport Agency) roading project between Tauranga in the west and Paengaroa in the east bypassing Te Puke. The project comprises four interchanges, 12 bridge structures and 6km upgraded and 17km newly constructed highway and more than 1.3 million cubic metres of earthworks. TEL was identified as strategic project for Tauranga and the Western Bay of Plenty region and was categorised as RoNS (Roads of National Significance) project with a construction cost of NZ$350 million. The design and construct contract for TEL was won by a construction alliance of Fulton Hogan and HEB. Early works construction commenced in mid-2011 and target completion is in mid-2015.
Geological conditions and geotechnical design challenges vary along the 23km TEL alignment. Main geological features include Aeolian dune sands, airfall Tephra deposits, up to 6m thick very soft highly compressible peats, deep paleo channels in-filled with soft Holocene estuarine silts. High groundwater levels close to the ground surface are present particularly in the low lying peat sections.
This paper focusses on the geotechnical design methodology in regards to improvements of soft ground at the Domain Road Interchange, where the TEL alignment diverges from the upgraded SH2 section to the new greenfield alignment.
Figure 1 shows the key features at the Domain Road Interchange, a 3-span flyover bridge over the main interchange roundabout, a single span bridge over the westbound off-ramp and three separate EPS (expanded polystyrene) embankments.
The geotechnical design challenges were managing the expected large settlements in excess of 4m during construction, mitigating long term creep settlements by comprehensive wick drain and surcharge schemes and the design of ground improvements to reduce the effects of seismic and liquefaction movements and settlements.

2 DOMAIN ROAD INTERCHANGE

The key design features at the Domain Road Interchange comprise a sequence of three separate EPS (expanded polystyrene) embankments, two bridge structures over connection roads. A complex arrangement of two new roundabouts, connections to local roads, SH2 and TEL provide the traffic continuity.
The previous roundabout linking Domain Road to SH2 has been removed. The PowerCo substation and East Coast Main Trunk (ECMT) railway culvert are indicated on the aerial photo. The SH2 link towards Te Puke remains in southern direction over the existing ECMT railway line.

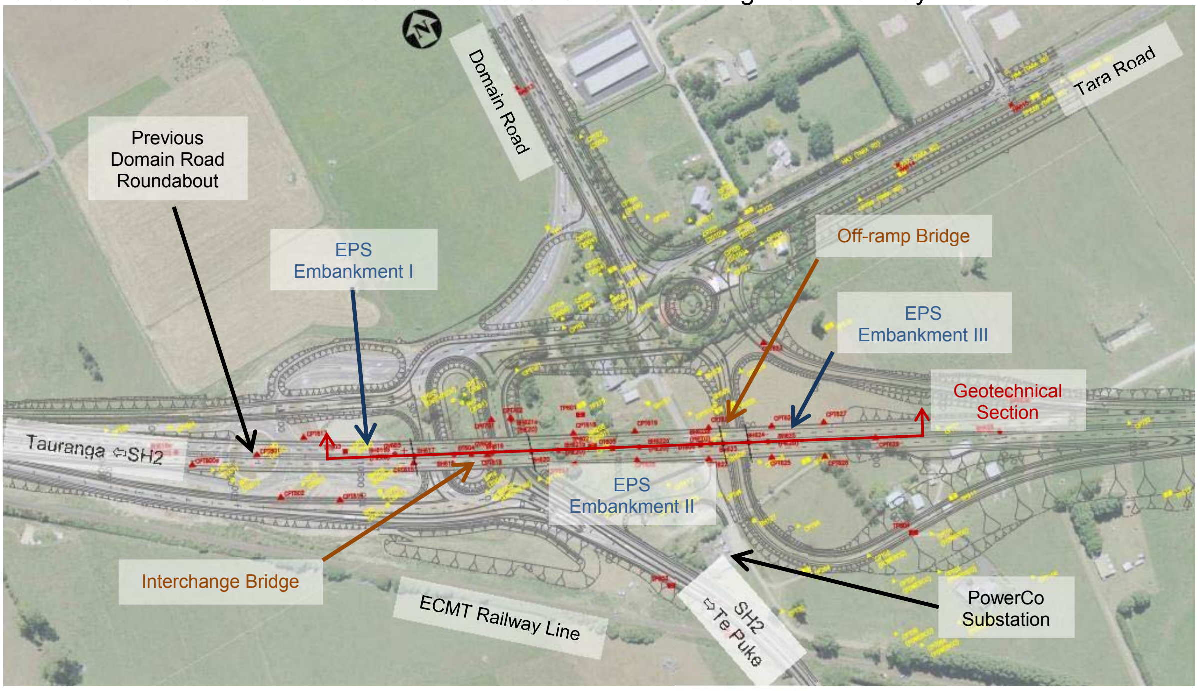

Figure 1. Site Plan Domain Road Interchange

The TEL alignment crosses at the Domain Road Interchange over a 23m deep paleo channel which is in-filled with soft estuarine silts and covered by highly compressible peat. The poor ground conditions would have required substantial deep ground improvements for the construction of conventional fill embankments.
In order to avoid extensive deep ground improvements, an approximately 400m long bridge was considered to carry the TEL alignment over the site. At concept (tender) design stage, both options were eliminated due to high costs and alternative solutions were explored. The existing roads, the ECMT railway line, the PowerCo substation and TEL alignment requirements were limiting possible options.
Replacing parts of the long bridge with expanded polystyrene (EPS) embankments was considered as the most cost effective solution. The final arrangement comprises an elevated alignment which carries TEL over the main interchange roundabout and west-bound off-ramp. The alignment features the following sequence for the two bridges and three EPS embankments in west to east direction.

- up to 3m high conventional sand fill embankment,
- 144m long, 3m to 7m high ‘EPS Embankment I’,
- 91m long 3-span bridge founded on up to 33m deep closed end driven steel tube piles,
- 146m long, 8m to 9m high ‘EPS Embankment II’ between both bridges,
- 20m long single span bridge founded on 45m deep piles which spans over the TEL west bound off-ramp,
- 65m long, 3m to 7m high ‘EPS embankment III’
- up to 3m high conventional sand fill embankment.

Geogrid and high strength geotextile reinforced foundation fills were designed to provide the necessary support for EPS embankments under design earthquake conditions.
The Domain Road Interchange site features further an 8m deep cut through alluvial Tephra deposits at the west-bound off-ramp, the upgrade and widening of the existing SH2 in the south of the TEL alignment and 4-lane widening of Domain Road and Tara Road in the north of the TEL alignment.

3 SITE GEOLOGY AND GROUND CONDITIONS

The Tauranga Eastern Link project is located within a Pleistocene basin which was in-filled with alluvial and estuarine sediments during a period of rapid tectonic subsidence. *Figure 2* shows the geotechnical section as indicated in *Figure 1* along the TEL alignment at the Domain Road Interchange. For clarity, peat is indicated in brown, estuarine silt is shown in grey, sands in yellow and silts in blue. A detailed description of the geotechnical ground conditions and the complex layering are not presented here and only includes the main features relevant to the design methodology discussed in this paper.

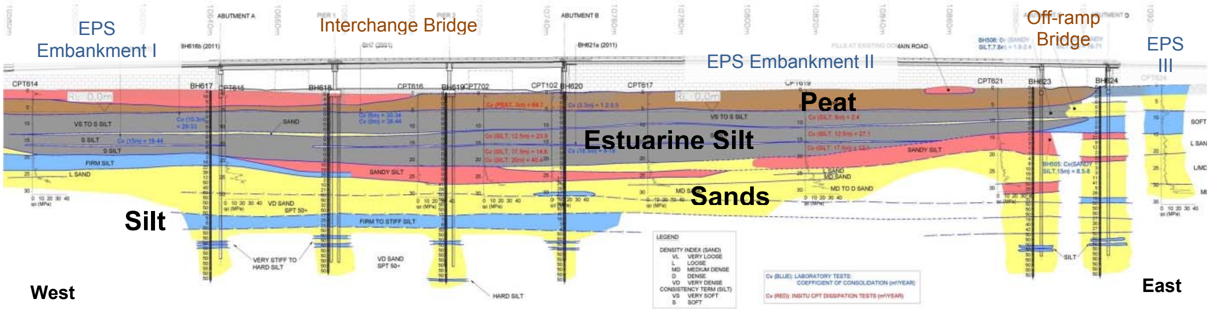

Figure 2. Geological Section

The ground conditions at the Domain Road Interchange comprise up to 6m of peat overlying a deposit of soft alluvial and estuarine silts. The peat layer extends over 600m along the alignment in east-west direction. Estuarine silt deposits within the paleo channel vary in thickness up to 17m and extend up to 23m below ground level at the Interchange Bridge and the central EPS Embankment II.

Cone penetration tests indicated cone resistances as low as 100kPa to 200kPa at upper estuarine silts and less than 500kPa at 20m depth (refer *Figure 4*). Low corresponding cone friction was indicative for very soft sensitive silts. The groundwater table is subject to seasonal variations of 500mm to 1m and is close to the ground surface at the low lying areas.

4 DESIGN AND CONSTRUCTION CHALLENGES

As deep ground improvements were excluded, management of more than 4m total settlements and subsequent drag settlements adjacent to existing roads and infrastructure were considerable challenges for the geotechnical design and the construction works.

It was required to divide the Domain Road Interchange site in more than ten earthworks zones with fill placement at various stages to allow traffic relocations, placement of underground services and to avoid and minimise damage to newly constructed roads and services caused by drag settlements.

In order to avoid the potential slope failures, staging of construction fills was necessary where fill side slopes faced greenfield sites. The existing embankment fills of SH2 and Domain Road provided a buttress and construction fills could be placed against the existing embankments without staging.

Surcharge fill heights varied from 1.5m to more than 4m across the site depending on the site specific conditions and whether wick drains were installed. Governing factors for surcharge heights and period were the mitigation of post construction creep settlements.

In addition, strength loss of loose saturated sands due to seismic liquefaction, liquefaction induced ground settlements and subsequent effects to the EPS embankments, the bridges and all interfaces between structural items required substantial design consideration.

5 GEOTECHNICAL DESIGN PHILOSOPHY

To facilitate a robust and cost effective design, design solutions where the expected large settlements provide beneficial effects and could be utilised as integral part of the ground improvement scheme were investigated. This approach required comprehensive understanding of the settlement behaviour at the site. Therefore, as part of the geotechnical tender and detailed design, the settlement monitoring data and reconstruction records of the Domain Road Roundabout were thoroughly reviewed, back analysed and compared with the new geotechnical investigation data at the existing SH2 embankment fills and at greenfield locations.

Additionally, a fully monitored trial embankment was implemented during the design stage to simulate the settlement behaviour and including further assessment of geotechnical consolidation parameters. *Section 6* presents a brief summary of the trial embankment details and settlement monitoring results.

The final design at in situ sites comprised an approximately 5m thick geogrid reinforced foundation ‘raft’ of compacted sand which provides sufficient support for the EPS embankments and lateral

resistance for the bridge piles. The existing embankment fills were also incorporated in the foundation raft and where required strengthened with additional geogrids.
At sections outside of the paleo channel where the ground was void of peat and estuarine silts, settlements were expected to be less than 1.5m and the details of the foundation raft were modified accordingly to suit the site specific conditions.
In order to meet the design requirements, the construction works of the compacted sand foundation fills had to achieve the following items:

- Compression and consolidation of the peat layer to provide strength increase, mitigate long term post construction creep settlements to acceptable limits.
- Increase of the undrained shear strength within the estuarine silts due to higher overburden pressure and increased over consolidation ratio due to surcharge fills.

6 TRIAL EMBANKMENT

Main purpose of the trial embankment was obtaining in situ load deformation behaviour of the peat and estuarine silt layers at various depths before the construction at the Domain Road Interchange commenced.
The trial embankment was located at a selected in situ site with subsoil conditions comprising approximately 5.5m peat and 17m soft to firm estuarine silts. A typical CPT plot is presented in *Figure 4*.
Wick drains at 1.6m triangular distances were installed to 23m to 25m depth below ground level. Geotechnical instrumentation including magnetic extensometers, vibrating wire piezometers, settlement plates, one profilometer and one inclinometer were implemented in the trial embankment. After a baseline reading was established, monitoring was carried out up to twice daily during the 1-month loading (filling) phase indicated by the dashed lines in *Figure 3*.

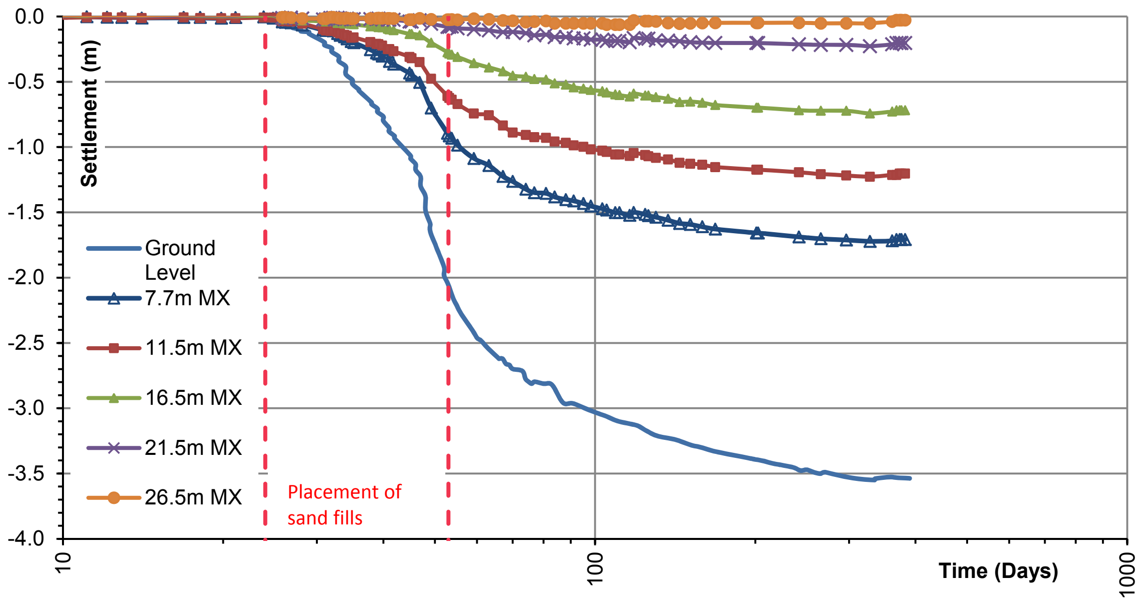

Figure 3. Trial Embankment Settlement Monitoring

The monitored data was used to validate and verify design assumptions adopted for the surcharge design. Calibration of laboratory test results was carried out based on this information and site specific compressibility and consolidation parameters were derived for peat and estuarine silts.
Four other trial embankments were constructed along the TEL alignment to determine site specific parameters and for the comparison and establishment of a project wide parameter set. The Domain Road trial embankment was a 'single cell' 6m high and 30m by 30m wide trial embankment while the other four trial embankments had up to 4 different fill heights.
As shown in *Figure 3*, the monitored ground settlements in the centre of the trial embankment were 3.55m caused by 6m of fill. The magnetic extensometers which were also installed in the centre recorded 1.72m settlements at 7.7m depth suggesting 1.83m compression within the upper 7.7m.

Relatively large settlements of 742mm and 227mm were recorded at 16.5m and 21.5m depth respectively.
At the end of the 1-month construction phase of the trial embankment 2.06m settlement occurred. Further 1.49m of total settlement was recorded over the subsequent 9 month holding period prior to surcharge removal. The maximum monitored ground settlement at the trial embankment was 3.89m at an outside corner. At the end of the 9 month settlement period, approximately 1.5m of surcharge fills were removed and cut to the final ground level which resulted in 30mm rebound.

Figure 4 shows a comparison of cone penetration tests carried out on in situ ground conditions prior to construction and after surcharge preloading.
The approximately 5.5m thick in situ peat layer can be inferred by the friction ratio in the middle graph and the compacted sand fills are obvious by the CPT cone resistance on the left-hand side. The ground surface and subsoil settlements as presented in *Figure 3* can be inferred by distinct thin soil layers, i.e. sand layers within the estuarine silts.

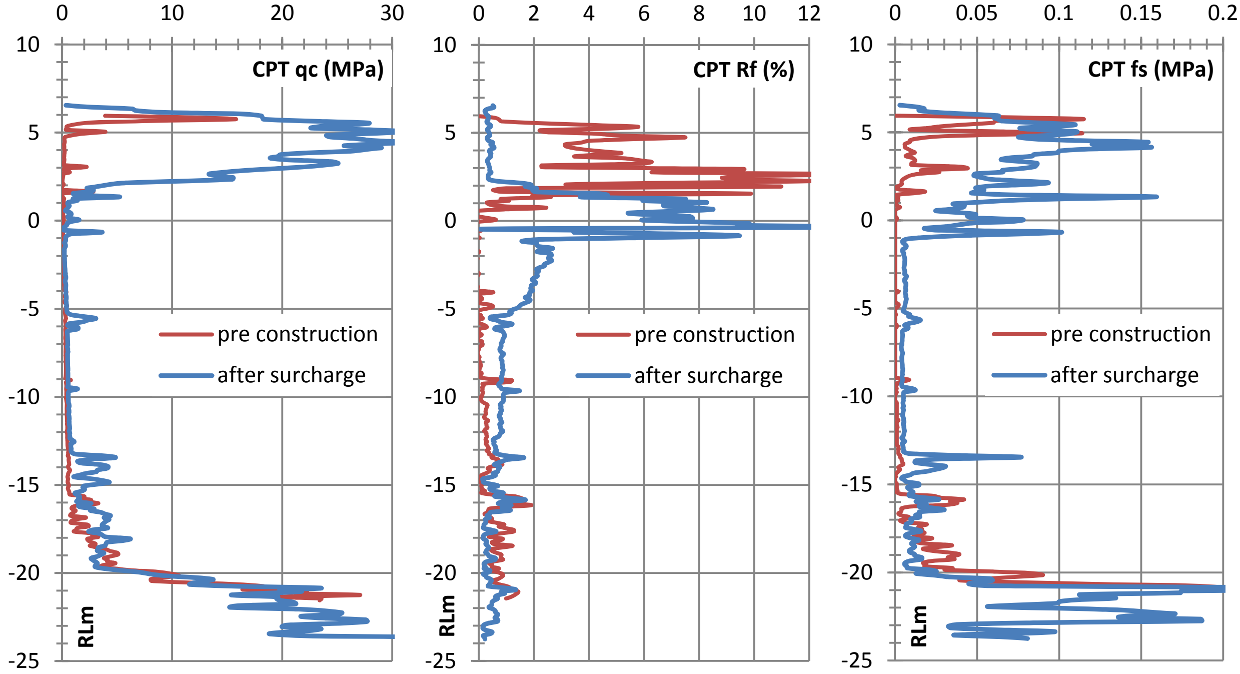

Figure 4. CPT Investigation Data prior and after Surcharging

7 *SHANSEP* – STRESS HISTORY AND NORMALISED SOIL ENGINEERING PROPERTIES

For the slope stability and bearing capacity design of short term static and seismic load cases undrained shear strength is typically adopted. It is appropriate to use in situ shear strength parameters for short term conditions, but in situ parameters may be conservative at sites where significant improvement of the in situ ground conditions are expected due to changes of the stress state, i.e. as a result of embankment fills and large settlements.
The in situ undrained shear strengths were determined based in situ Geonor vane tests and correlations with in situ CPT tests.
For static slope stability analyses using effective stress parameters, the shear strength of the subsoil is increased as a function of the soil friction angle and the additional embankment overburden pressure. Undrained shear strength is similarly increased due to the additional overburden pressure and consolidation. The strength gain occurs gradually during the consolidation process and the dissipation of porewater pressures.
The strength gain of the estuarine silts at the Domain Road Interchange as the result of increased overburden pressure was calculated according to the *SHANSEP* (*Stress History And Normalised Soil Engineering Properties*) approach.
For the assessment of the *SHANSEP* design approach, the estuarine silts were determined to be normally consolidated to slightly over-consolidated based on one-dimensional consolidation tests. The normalised soil parameters for the assessment of the undrained shear strength ratio are a function of the pre-consolidation pressure and the over consolidation ratio (OCR).

Site specific ratios for undrained shear strength over pre-consolidation pressures can be assessed based on undrained triaxial test. *Ladd*'s research and recommendations for *SHANSEP* parameters are summarised in *Figure 5*. In-depth background is provided in the referenced research publications at the end of this paper.

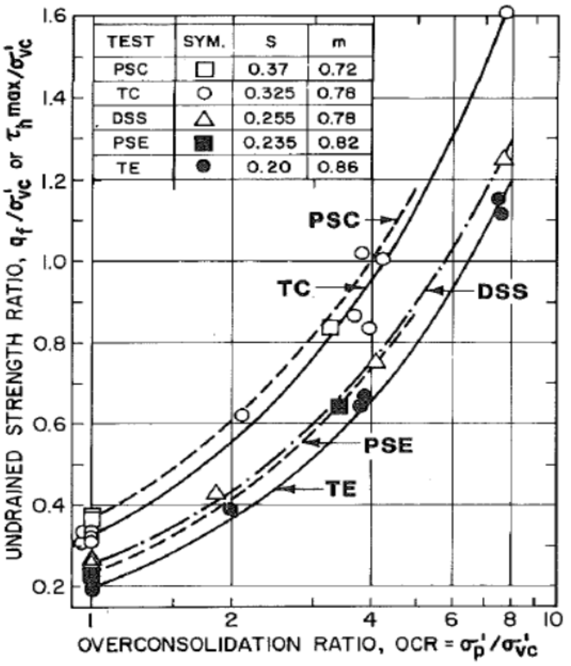

Figure 5. SHANSEP Parameters (Koutsoftas & Ladd, 1985)

$$\frac{s_u}{\sigma'_v} = S(OCR)^m = S\left(\frac{\sigma'_p}{\sigma'_v}\right)^m$$

where s_u undrained shear strength
σ'_v vertical effective stress
S undrained shear strength ratio for normally consolidated
σ'_p effective pre-consolidation pressure
OCR over-consolidation ratio
m exponent as per *Figure 5*

The design of the strength gain of the soft silt deposits was considered as a function of the temporary overburden pressure due to the surcharge fills placed above the foundation raft (permanent fills) and the increased effective vertical pressure resulting from the sands fills (higher density of sands compared to peat).
With equation *(1)* and the in situ over consolidation ratio determined from consolidation tests, which was in the order of 1 to 1.3, the undrained shear strength ratio ***S*** and the exponent ***m*** were determined. The upper and lower limits for ***S*** and ***m*** as per *Figure 5* were considered for our sensitivity analyses. *Figure 5* indicates undrained shear strength ratios ***S*** values of 0.2 to 0.37 and ***m*** values of 0.72 to 0.86 respectively. Combined with the low in situ OCR, the sensitivity of the parameters ***S*** and ***m*** was small in the curve fitting process which was carried out to match the in situ measured undrained shear strength.
SHANSEP consideration postulates that these values are site and soil type specific. Thus, once a parameter set was determined, it was adopted for the assessment of improved undrained shear strength using increased stress state and over-consolidation ratio.

The following design procedure was adopted:

- Assessment of the ratio of undrained shear strength over effective vertical stress s_u / σ'_{v0}, which was depth depending in the order of 0.45 to 0.50 for the in situ estuarine silts.
- The in situ stress state of the estuarine silts is normally consolidated to slightly over-consolidated. An over-consolidation ratio of 1.1 was adopted for the design.
- The new fills and surcharge fills increase the stress state of the subsoils. The wick drain scheme and combined with the settlement monitoring ensured that the primary consolidation is completed prior to surcharge removal. At this stage, the silts are normally consolidated. An apparent over-consolidation of the silts due to secondary compression (creep settlement) is ignored for this assessment.
- After removal of the surcharge fills, the silts are over-consolidated, decreasing with depth. The over-consolidation ratio can be determined depth dependant.
- *Figure 6* shows that the undrained shear strength of the soil is a function of effective vertical stress and over-consolidation ratio. The calculated post surcharge undrained shear strength is then calculated using the pre-determined values of ***S*** and ***m***.

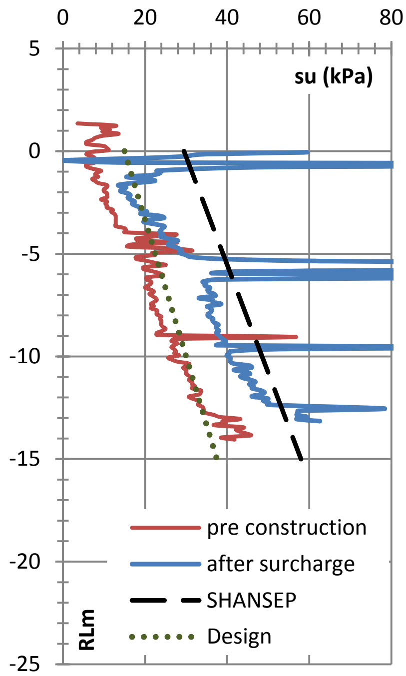

Figure 6. Improvement of Undrained Shear Strength

Based on sensitivity analyses and the recommended values, the undrained shear strength ratio ***S***=0.37, the exponent ***m***=0.8 and the over-consolidation ratio OCR=1.1.
The in situ (pre-construction) undrained shear strength as shown in *Figure 6* was correlated with 8 Geonor vane tests at the site to determine a CPT cone factor correlation, which was depth depending. The best fit CPT cone factor ranged between 12 and 14.
The estuarine silts within the paleo channel were divided by two distinct thin interbedded sand layers at approximately 10m and 15m depth, which are shown by the s_u peak values in *Figure 6* at approximately RL-5m and RL-10m. The three estuarine silt layers had slightly different geotechnical characteristic in respect of depth dependant strength increase.
The in situ undrained shear strength for the upper and middle silt layer was approximately 12kPa plus 1kPa strength increase per 1 metre depth. The lower layer below 15m depth had an undrained shear strength of 24kPa plus 3.6kPa strength increase per 1m depth.
Due to the risks that the improved *SHANSEP* undrained strength could not be verified during construction, a more conservative '*agreed*' undrained shear strength of 15kPa plus 1.5kPa strength increase per metre depth was adopted as indicated by the green dotted line ('*Design*') in *Figure 6*.
However, the improved undrained shear strength according to the *SHANSEP* methodology was determined to be in the order of 27kPa to 30kPa with a strength increase of 1.9kPa per 1m depth (refer black bold dashed line '*SHANSEP*' in *Figure 6*).
Back analyses comparing the undrained shear strength calculated based on CPT test prior to construction and after surcharging show significant improvements. The initial assessment at design stage of improved undrained shear strength could not be verified by the post surcharge assessment. It can be seen that the initially calculated improved *SHANSEP* undrained shear strength is lower than the calculated shear strength using post construction CPT data and equivalent CPT cone factors.
However, the adopted depth depending undrained shear strength of 15kPa plus 1.5kPa is conservative.

8 GROUND IMPROVEMENTS – REVIEW

The strength gain of the estuarine silts and the reduced thickness of the peat layer were considered as ground improvements and implemented in the slope stability design. Both items are basically non specifiable design items and therefore require either *(a)* reasonably conservative considerations and/or *(b)* appropriate verification testing during construction.
Option *(a)* was adopted for the design at the Domain Road Interchange, but the '*agreed*' parameters were only marginally above the in situ strength. However, there was no need for potentially disruptive and time delaying verification testing and subsequent assessments. In case that the predicted

improvement strength would not be verified, additional mitigation measures would have been necessary.
The adopted ground improvement scheme comprised various items including wick drains, surcharge fills and geogrids, which were interacting together and were partially relying on each others performance.
For example, at some areas where the predicted settlements were less than 1.5m, but the geogrids requried sufficient overburden depth to the perform efficiently in order to minimise the development length, undercutting was required prior to placement of the geogrids.
Comprehensive settlement monitoring, associated reviews and assessments were undertaken to verify the design assumptions and to demonstrate the long term performance.
Implementing the three EPS embankments signifcantly minimised the need ground improvements and eliminated the deep ground improvements entirely and reduced the bridge length.

9 CONCLUSIONS

Based on the comparison of the cone penetration tests carried out prior to construction and after surcharge removal at similar locations, only minor increase in CPT cone resistance was observed. However, the CPT sleeve friction increased significantly which is reflected in the friction ratio plot shown in *Figure 4.* The improvement undrained shear strength based on CPT data correlations is in the order of 40% to 50%.
Theoretically, the new permanent construction fills would have provided a strength of the estuarine silts based on the increased vertical effective stress. The additional temporary surcharge fills induced a depth dependant over consolidation which provided further improvement.
Geonor shear vane tests to demonstrate the increase of undrained shear strength were not carried out. Thus, the assessment of strength gain in accordance with the *SHANSEP* approach remains theoretical. The adopted improvement of undrained shear in the design is well below the calculated value and therefore provides the required robustness for the design.
Geotechnical design typically requires the use of '*moderately conservative*' soil parameters based on in situ or laboratory testing. If the calcuated shear strength based on the *SHANSEP* theory would have been adopted, large amount of verification testing would be required after the surcharge removal, which is impractical and would have imposed a significant risk to design. Therefore, lower '*agreed*' design parameters were used.
In hindsight, the use of more conservative undrained shear strength proved to be the correct decision.

10 ACKNOWLEDGEMENTS

I would like to thank all members of the Design Team (URS, Bartley Consultants, Opus and Gaia Engineers formerly Peters & Cheung), the construction team Fulton Hogan and HEB and finally, NZTA and their agent Beca.
Special recognition goes to Ian Manley and Leon Cho, both Gaia Engineers, for their research on the trial embankments which provided invaluable contribution for the assessment of consolidation and creep settlement behaviour of soft ground conditions for and beyond the TEL project. I would like acknowledge Dr Ka-Ching Cheung (Gaia Engineers) for support and technical advice, and Aidan Nelson and Dr Rob Davis, both Earthtech Consulting Limited, for their input to the project.

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