

Characterization of Sensitive Soft Soils for the Waterview Connection Project, New Zealand

Caractérisation de sols mous sensibles pour le projet de raccordement Waterview en Nouvelle-Zélande

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ABSTRACT: The paper presents the results and interpretations of data collected during the procurement phase of SH16 motorway upgrade. The strength and consolidation characteristics are investigated for a soil unit, labeled AH, which is identified to manifest a response typical for a sensitive soft soil. The estimate of undrained shear strength based on empirical methods is found to have limitations to predict the undrained shear strength of the sensitive AH soil. The one-dimensional compression response of the virgin AH soil is proposed to estimate using a relationship between the liquidity index and the vertical effective stress. The predictive capability of this relationship is demonstrated by numerical simulations of settlement monitored during the construction and post-construction phase of the original SH16 motorway embankment.

RÉSUMÉ : Cet article présente les résultats et les interprétations des données recueillies lors de la phase de projet de mise à niveau de l'autoroute SH16. La résistance et les caractéristiques de consolidation sont étudiées pour un type de sol, noté AH, qui est représentatif d'un sol mou sensible. L'estimation de la résistance au cisaillement basée sur des méthodes empiriques est peu adaptée pour prédire la résistance au cisaillement non drainée du sol sensible AH. La réponse en compression unidimensionnelle du sol vierge AH est modélisée par une relation entre l'indice de liquidité et la contrainte effective verticale. La capacité prédictive de cette relation est démontrée par des simulations numériques des tassements mesurés pendant la phase de construction et post-construction du remblai original SH16 de l'autoroute.

KEYWORDS: clay, sensitive soil, critical state soil mechanics, SHANSEP, settlement.

1 INTRODUCTION

The Western Ring Route (WRR) is an ambitious project initiated by New Zealand Transport Agency (NZTA) to provide a 48km alternative route for improvement of traffic flow around the Auckland city center. One significant work component of the WRR project is the upgrade of the State Highway 16 (SH16), where part of the route is passing through an estuarine environment. This section of the motorway, referred to as the Causeway, has experienced significant settlements over a period of 60 years of service life. Today the traffic lanes are prone to flooding during storm and king tide events.

2 PAPER OBJECTIVES

NZTA has commissioned Aurecon to undertake an in-depth investigation into the soil conditions present along the Causeway Section. The geotechnical investigation includes over one hundred exploratory holes, in addition to numerous older holes drilled during the planning phase for the original Causeway in the late 1950s.

The field investigations were complemented by laboratory testing for the purpose to provide details on strength and compressibility of the Causeway estuarine soil materials. The compressibility and behaviour of AH soils is investigated in the critical state framework. The ability of SHANSEP formulation to predict the undrained shear strength is investigated based on the piezometer (CPTu) data.

Based on the non-linear one-dimensional compression manifested by the sensitive AH soil, a framework of analysis is proposed to predict the non-linear one-dimensional compression of AH soil. An example of soil settlement analysis is carried out to predict the magnitude and rate of settlement development of the original SH16 motorway embankment during the construction and post-construction stages.

3 GEOLOGICAL CONDITIONS

A detailed geological model was developed from the information collected during five (5) stages of site investigations. The ground profile consists of geological conditions which adopt a layering code system as summarized in Table 1.

Table 1. Geological layer codes adopted for SH16 Causeway alignment.

| Geologic Age | Unit | Layer Code | Description |
|-----------------------------|---------------------------------|------------|---|
| Late Pleistocene – Holocene | Marine Sediments | AH | Marine clays and silts – estuarine muds |
| | | ATcl | Clays and Silts |
| | | ATs | Sands and Silty Sands |
| Pliocene - Pleistocene | Tauranga Group | ATo/ATp | Organic Clay/Peat |
| | | ATv | Rhyolitic Silt and Sand (volcaniclastic) |
| | | ER | Residual ECBF Soil Weathered ECBF |
| Miocene | East Coast Bay Formation (ECBF) | EW | Sandstone – Soil and Rock Fractions (20 < N < 50) |
| | | EU | Unweathered ECBF Rock – Sandstone and Siltstone (N > 50) |

The marine sediments (AH soil unit) belong to the Late Pleistocene-Holocene age with a deposition environment starting between 8,000 to 14,000 years ago, which still carries on today.

The AH deposit generally consists of uniform normally consolidated Silty Clays. The very soft strength characteristic of the AH soils is in direct contrast to the typically

overconsolidated soils present in the underlying Tauranga Group alluvium.

In the Causeway Section, the depth of the Holocene layer is typically 12.0m to 13.0m, with a reduced depth of 1.5m where the ECBF rock head is present at a shallow depth.

4 UNDRAINED SHEAR STRENGTH

The undrained shear strength was assessed by in-situ testing methods such as: (a) field shear vanes; and (b) piezocones (CPTu).

The field shear vanes are commonly used to determine the undrained shear strength (s_u) of soft to medium stiff clays. In this project, a typical field vane was used with a diameter of 50 to 65mm and a height to diameter ratio of 2.

The piezocone testing was conducted using a standard cone: cone angle of 60 degrees, a cross-sectional area of 10 cm², and a porous element located immediately behind the cone. The cone was advanced during field probing at a standard rate of 20mm/sec. The data was measured electronically to include the tip resistance (q_c), sleeve friction (f_s) and the penetration pore water pressure (u). The cone correction factor (N_k) for estimates of s_u values, was assessed based on correlations against the field shear vane test results.

The shear vane profile for the virgin AH soil is presented in Figure 1a. All shear vane readings were corrected adopting the empirical correction factor (μ) recommended by (Bjerrum 1973). The undrained shear strength is showing an increasing trend with depth in accordance with a linear relationship that can be expressed as follows:

$$s_u = 6 + 1.9 \times \text{depth} \quad (1)$$

5 SOIL SENSITIVITY

The soil sensitivity represents an indicator of soil micro-structural bonding or development of inter-particle forces between particles or their aggregates. In this study these effects are referred to as structural bonding. The disturbance to the soil structural bonding during loading could have some serious consequences such as: (a) strength reduction; and (b) changes in the overall soil behaviour due to an increase in soil compressibility properties.

The measure of soil sensitivity (S_t) adopted in this study is based on the ratio between peak undisturbed strength (s_u) and the remould strength (s_r) when the soil reaches its residual state. The results of shear vane tests were interpreted to determine the strength sensitivity manifested by virgin AH soil as shown in Figure 1b.

Several classifications of soil sensitivity have been proposed in the technical literature. According to (Rosenqvist 1953) the AH soil falls in the range of a Very Sensitive soil (i.e. $4 < S_t < 8$).

6 ATTERBERG LIMITS

The consistency limits, liquid limit (LL) and plasticity limits (PL), besides serving the basic means of soil classification, they have also been shown to provide estimates of strength and deformation parameters via empirical correlations (Wroth and Wood 1978, among many others).

For this project, the method to assess the LL has adopted the “fall cone” method for the AH soils. The PL was determined by the method of thread rolling according to BS 1377-2:1990.

Figure 2a shows the liquidity index (LI) for a soil profile at Chainage 1,860m. The values of LI are generally greater than 1 which is indicative of a soil micro-fabric that is able to accommodate additional resistance over the remoulded state due to development of structural bonds.

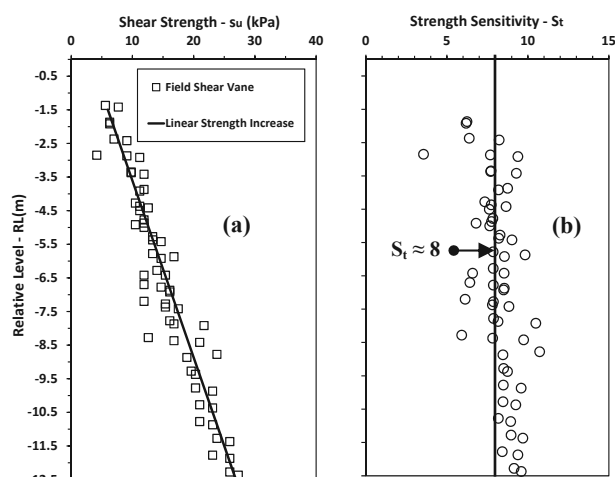


Figure 1. (a) Undrained shear strength profile; and (b) Soil sensitivity profile.

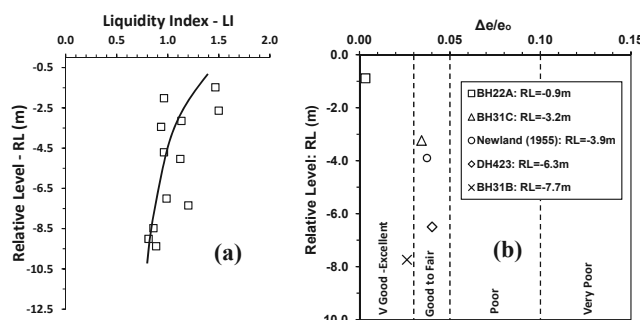


Figure 2. (a) Variation of liquidity index with depth; (b) Sample quality assessment based on (Lunne et al. 1997) classification system.

7 SAMPLE DISTURBANCE

An evaluation of the soil sample quality is essential, as disturbance may lead to a laboratory measured soil behaviour that is different from its in-situ response. Efforts were made to minimize the sample disturbance, but it inevitably occurs due to stress changes associated with soil sampling.

(Lunne et al. 1997) proposed to estimate the quality of soil samples based on the ratio $\Delta e/e_0$, where the change in void ratio Δe is measured during soil re-consolidation phase to in-situ vertical stress σ'_{v0} with consideration of void ratio e_0 at the beginning of this phase.

Figure 2b presents an assessment of soil sample quality of “reliable” oedometer tests (i.e. sample designation falling in the range of good to excellent). Given the very soft consistency of the virgin AH soil, a significant number of samples were found to have an undesirable quality.

For brevity and clarity of the paper presentation these values have not been included in Figure 2b, and the associated oedometer and triaxial test results have been discarded from further consideration.

8 OEDOMETER TESTS

The tests were carried out using a fixed-ring oedometer with drainage allowed at top and bottom of the test soil sample. The soil samples were 31mm in diameter and 16mm in height. The settlement of the soil sample was monitored by a Linear Vertical Displacement Transducer (LVDT). Each load increment was applied over a time period of 24 hours, with the end of primary consolidation determined using the square-root-of-time method (Taylor 1942).

An AH sample was vigorously remoulded on a glass plate using a spatula to form a soil slurry at a water content equal to the liquid limit (Burland 1990).

The results of AH oedometer tests are presented in Figure 3 in a semi-logarithmic plot of void ratio, e , against the logarithm of the vertical effective stress, $\log(\sigma'_v)$. The virgin AH soil manifests initially a negligible amount of compression before reaching the pre-consolidation stress, σ'_p . When the stress values exceed σ'_p , the consolidation curve displays a non-linear response with a gradient significantly higher compared to the remoulded sample. At high pressures, the compression curve of virgin soil is seen to approach the remould consolidation line in an asymptotic trend.

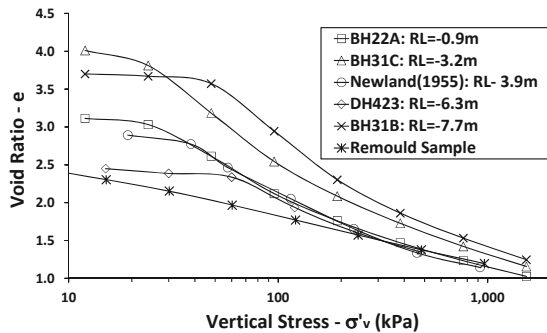


Figure 3. One-dimensional consolidation test results.

9 TRIAXIAL TESTS

The triaxial tests were carried out on virgin soil samples of diameters varying between 50 and 70mm. For all the tests the sample saturation was achieved by raising the back pressure to a maximum of 300kPa. Over the duration of saturation, the stress state was maintained at an effective stress of 20kPa. Full saturation was achieved when Skempton's pore pressure parameter B has achieved a value equal or greater than 95%.

On completion of soil saturation, the samples were isotropically consolidated before the start of undrained shearing. The undrained shearing was conducted in a deformation controlled mode to large axial strains between 15 to 20%.

Figure 4 presents the results of undrained shearing as plots of effective stress paths: deviatoric stress $q = \sigma'_1 - \sigma'_3$ vs. mean effective stress $p' = (\sigma'_1 + 2\sigma'_3)/3$.

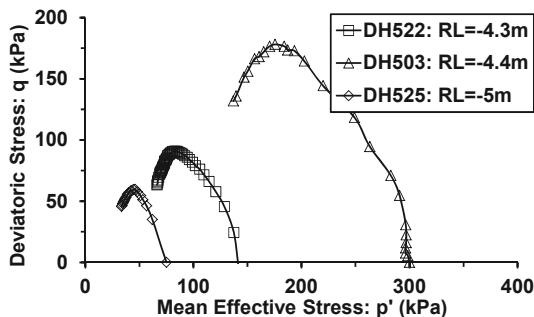


Figure 4. Effective stress paths of AH soil samples in undrained shearing for a range of confining pressures $p'_0 = 75$ to 300 kPa.

The behaviour manifested by the virgin AH soil is of a contractive type. The stress paths climb to a maximum deviatoric stress, q_{max} , before plummeting towards the origin of the stress path. The strain softening past q_{max} is a distinctive feature of this suite of tests.

10 CRITICAL STATE

The critical state is a fundamental concept in soil mechanics as it represents a reference state to assess the state and behaviour

of soil under loading. It was firstly introduced by (Roscoe et al. 1958) to describe the behaviour of remoulded clays, and nowadays the concept was extended to represent a more general framework of soil behaviour for: (a) Sands (Been et al 1991); and (b) Combinations of sand with various percentages of plastic and non-plastic fines (Bobei et al. 2009).

The results of undrained triaxial tests conducted on AH soil are interpreted in the framework of critical state as illustrated in Figure 5. The soil samples were considered to reach the critical state when the following conditions are satisfied: $dq=0$, $dp'=0$, $du=0$ while $d\epsilon_q \neq 0$.

The CS is found to plot along a linear relationship. On consideration of initial soil state being located above the CS line, and the contractive behaviour manifested during undrained loading (refer to Figure 4), the virgin AH soil appears to fully conform to the framework laid out by CS.

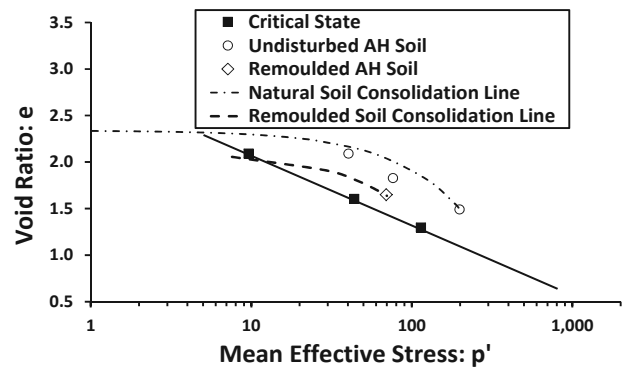


Figure 5. Critical state and soil state of AH soil before undrained loading.

11 SHANSEP PROCEDURE

SHANSEP (Stress History and Normalized Soil Engineering Properties) was proposed by (Ladd and Foot 1974) as an empirical method to adopt in engineering practice to estimate the undrained shear strength on consideration of stress history effects arising from geological unloading. The over-consolidation ratio ($OCR = \sigma'_p/\sigma'_v$) is chosen as a parameter to encapsulate the stress history effects, with the OCR values determined by high quality oedometer data. In mathematical terms, SHANSEP correlates the undrained shear strength with the soil OCR as follows:

$$\frac{s_u}{\sigma'_v} = S \times OCR^m = \left(\frac{s_u}{\sigma'_v} \right)_{OCR=1} \times OCR^m \quad (2)$$

where: S = intercept with vertical axis at $OCR = 1$; and m = gradient of linear relationship.

A plot of undrained shear strength ratio (s_u/σ'_v) with OCR based on CPTu data is shown in Figure 6. The CPTu data was found to manifest a linear response, with slope gradients within the bounds of linear relationships shown with dashed lines. In the normally consolidation range, all linear relationships intersect the vertical axis at a constant $(s_u/\sigma'_v)_{OCR=1} = 0.35$.

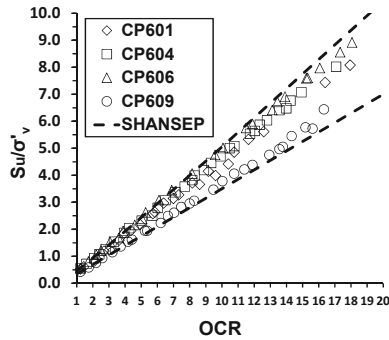


Figure 6. Interpretation of CPTu data based on SHANSEP normalization procedure.

12 FRAMEWORK OF SETTLEMENT ANALYSIS

Previous examinations of the one-dimensional response of a large number of clay soils (Skempton and Northey 1952), suggest the possibility to develop a correlation between the soil sensitivity and liquidity index. The oedometric results of virgin AH soils are interpreted in Figure 7 using LI as a normalizing parameter.

The one-dimensional compression curves are observed to “bundle together” in the stress range greater than pre-consolidation stress, σ'_p . Such normalization procedure may be modeled by a non-linear relationship expressed as follows:

$$\sigma'_v = \frac{90}{(LI + 0.21)^2} \quad (3)$$

The predictive capability of equation (3) is illustrated in Figure 7 by the dotted line.

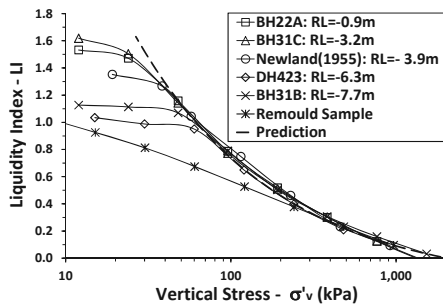


Figure 7. Prediction of non-linear oedometer compression curves.

13 SETTLEMENT PREDICTION

The embankment construction progressed gradually on the surface of soft marine muds to form a series of containment cells, to be later in-filled with granular (sand and shell) and cohesive clay material.

An in-house spreadsheet was developed to incorporate the non-linear relationship (3) to describe the compression response of the virgin AH soil. The main features embedded into the spreadsheet include: (a) multi-layered soil configuration; (b) vertical stress increase with depth calculated based on 2D embankment geometry; (c) reduction in σ'_v due to submergence of fill embankment below the ground water table; (d) calculation of primary consolidation using Terzaghi’s 1-D consolidation theory; and (e) calculation of creep settlement.

The prediction is found to simulate considerably well the magnitude and rate of settlement development with time as shown in Figure 8.

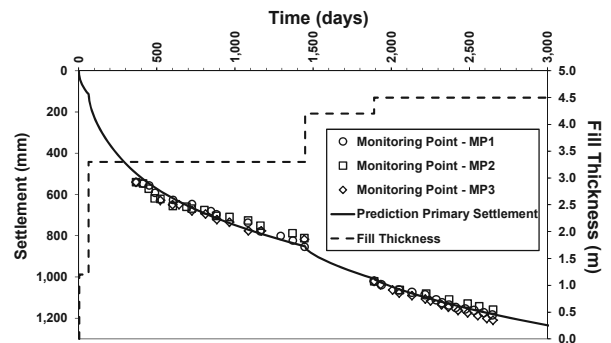


Figure 8. Settlement prediction at the centerline of motorway embankment.

14 CONCLUSIONS

The paper presents some of the results collected as part of an extensive geotechnical investigation carried out into the sub-surface conditions of the Causeway Section of SH16. The main findings of the paper are summarized below:

1. The undrained shear strength of virgin AH soils manifests a linear increase with depth.
2. The compressibility of virgin AH soil in one-dimensional testing displays non-linear characteristics when stresses exceed the pre-consolidation pressure.
3. The assessment of undrained shear strength of virgin AH soil is not readily predicted by methods such as SHANSEP.
4. The one-dimensional response of virgin AH soil is found to uniquely relate LI and σ'_v . The predictive capability of a proposed relationship is demonstrated by numerical simulations of settlement monitored during the construction and post-construction phase of SH16 motorway embankment.

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