

# DESIGN CHALLENGES OF ROAD WIDENING IN SOFT GROUNDS: CHARACTERISATION TO NUMERICAL ANALYSIS

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## ABSTRACT

This paper presents challenges involved in robust geotechnical design for upgrading an existing motorway, which is founded on soft soil. A critical review is presented on current approaches adopted for soft soil characterisation using results of laboratory and field tests commonly applied in Australian practice. Particular attention is given to assessment of undrained shear strength, overconsolidation ratio (OCR), primary and secondary compression (creep) indices and coefficient of consolidation. Some of the limitations of finite element analysis using 2D PLAXIS software, as a commonly used tool, and its 'soft-soil creep model' are also highlighted. Analytical approaches are presented which were used in a case study to overcome these limitations and help with deformation analysis of soft soils undergoing creep, as well as the design of rigid inclusions taking into account 3D effects.

The case study project upgrade works involved widening of the motorway embankment and in turn extension of existing culverts, which were located in a river floodplain. Preload and wick drains were considered for soft soil improvement to meet residual and differential settlement criteria over the design life of the new pavement. Rigid inclusions were also designed for the new culvert extension to reduce potential differential settlement between the proposed extension and existing culverts.

## 1 INTRODUCTION

Transport infrastructure in Australia has been mostly built along the congested eastern and southern coastal belt where soft and compressible alluvial and marine clay deposits are often encountered around watercourses and paleochannels. These soft deposits present significant challenges to design, construction and performance of major infrastructure currently being built or upgraded in Australia. This is due to their high compressibility and low bearing capacity that can adversely affect stability and settlement performance of structures.

To overcome these challenges and to enable a resilient geotechnical design for such infrastructure, it is crucial that fundamental features of soft soils are well understood and characterised through an appropriate laboratory and field testing program. Test results should be meticulously interpreted to extract design parameters, and appropriate material models should be applied for stability and deformation analyses. It is also important that limitations of present material models and commercial software are appreciated and appropriately dealt with to obtain reliable outcomes.

This paper presents challenges involved in robust geotechnical design for upgrading an existing motorway, which was founded on soft soil. The upgrade works involved widening of the motorway embankment and in turn extension of existing culverts, which were located in a river floodplain. Preload and wick drains were considered for soft soil improvement to meet residual and differential settlement criteria over the design life of the new pavement. Rigid inclusions were also designed for the new culvert extension to minimise potential differential settlement between the proposed extension and existing culverts.

A critical review is presented on current approaches adopted to extract design parameters using results of laboratory and field tests, which are commonly applied in Australian practice. Particular attention is given to assessment of shear strength, overconsolidation ratio (OCR), primary and secondary compression (creep) indices and coefficient of consolidation. Some of the limitations of finite element analysis using 2D PLAXIS software, as a commonly used tool, and its 'soft-soil creep model' are also highlighted. Analytical approaches are presented which were used in this design to overcome these limitations and help with deformation analysis of soft soils undergoing creep, as well as design of 3D rigid inclusions.

The reader should note that 'soft soil' terminology used in this paper represents very soft to firm soils.



undrained shear strength ratio,  $S_u / \sigma'_v$ , should also be studied ( $\sigma'_v$  is vertical effective stress or consolidation pressure). Experimental studies conducted since the early 1970's (e.g. Ladd and Foott, 1974; Larsson, 1980, Ladd, 1991) suggested that undrained shear strength ratio can be approximated by the following relationship:

$$\frac{S_u}{\sigma'_v} = S(OCR)^m \quad (1)$$

Where S=the normally consolidated value of  $S_u / \sigma'_v$ , OCR= overconsolidation ratio, and  $m$  =the strength increase exponent. S and  $m$  can vary significantly by soil type. Their recommended values for different soil types are presented later in this section.

Nevertheless, Ladd and Degroot (2003) and various other authors have shown that the undrained shear strength of normally consolidated inorganic clays can be approximated using the following relationship:

$$S_u = 0.22 \times \sigma'_v \quad (2)$$

To initial shear strength profile and the undrained shear strength ratio-OCR relationship can be evaluated using laboratory and field tests. The type and extent of testing are determined based on the degree of refinement required for the undrained strength analysis. Three levels of testing may be considered in regard to sophistication and expenses (Ladd, 1991), as follows:

**Level A** - for detail design of major projects and sites where foundation materials expected to show significant anisotropy in terms of shear strength and stress-strain relationship, or where soils have unusual features such as fissuring or high organic content, etc. This level also applies to the project where lateral deformations during construction are on interest (e.g. excavation adjacent existing sensitive structures).

**Level B** - for preliminary or tender design of major projects or detail design of less important projects involving "ordinary" soils with low to moderate anisotropy.

**Level C** - for preliminary feasibility or concept design and also to verify whether initial strengths inferred from in situ and laboratory Unconsolidated Undrained (UU) test programs are reasonable.

### 3.1.1 Laboratory Strength Tests

There are many types of laboratory testing equipment developed to measure soil shear strength under certain drainage conditions and stress paths. Nevertheless, variations of simple direct shear box and triaxial shear testing apparatus are commonly available in commercial laboratories and used in practice. Depending on the level of the project sophistication and importance, described above, the following laboratory test programme is recommended (Ladd, 1991):

**Level A** – Due to the effect of intermediate principal stress and anisotropy on the shear strength, a test program should ideally include  $CK_0U$  triaxial tests, where samples are consolidated under  $K_0$  (at rest) conditions to represent actual site conditions and sheared under undrained conditions. Samples should be tested for different modes of failure (e.g. compression and extension). Conventional  $CK_0U$  triaxial tests, however, generally would give conservative peak shear strengths for plain strain problems (Ladd et al., 1977). Alternatively, the test programme could include direct simple shear and plane strain compression/extension tests.

Testing of samples at different OCRs enables the site specific S and  $m$  in Eq. 1 to be established.

**Level B** -  $CK_0U$  direct simple shear testing is recommended for Level B to obtain  $S_u / \sigma'_v$  versus OCR relationships for stability analyses using isotropic strength profile (Ladd, 1991). Alternatively  $CK_0U$  triaxial compression and extension tests may also be used, however, they would require more soil, more experience and effort to obtain reliable results, and hence are more expensive.

Isotropically consolidated triaxial compression tests should not be relied on in Level B projects. It has been shown that these tests could significantly overestimate the in-situ average shear strength for most soils.

**Level C** – at this level the empirical correlation of Eq.1 can be used. In absence of site-specific correlations, Ladd (1991) suggests the following values for S and  $m$ :

- Sensitive marine clays (Plasticity Index,  $I_p < 30\%$  and Liquidity Index,  $I_L > 1$ ):  
 $S = 0.20$ , with normal standard deviation of 0.015, and  $m = 1$ ;
- Homogeneous sedimentary clays of low to moderate sensitivity ( $I_p = 20\% - 80\%$ ):  
 $S = 0.20 + 0.05I_p$ , or simply  $S = 0.22$ , and  $m = 0.8$ ;
- Varved clays:  
 $S = 0.16$ ,  $m = 0.75$ ; and
- Sedimentary deposits of silts and organic soils and clays with shells:  
 $S = 0.25$ , with normal standard deviation of 0.05, and  $m = 0.8$ .

### 3.1.2 Field Tests

Laboratory test results can sometimes be misleading for soil characterisation as they are based on testing small discrete samples collected from limited and specific depths which are also subject to sampling disturbance. In-situ field tests, however, enable assessment of a broader range of soil properties and provide an opportunity for an economical tool for soil profiling. The in-situ tests commonly used in Australia include Standard Penetration Test (SPT), Field Vane Test (FVT), Piezocone Penetration Test (CPTu) and the Marchetti Dilatometer Test (DMT). The SPT is not considered further in this paper as the authors consider it is of limited use for clays and gravelly soils and, in any event, is rarely used for soft soil characterisation. Other tests have also limitations in term of applicability as discussed below.

FVT provides a reasonable evaluation of undrained shear strength of relatively homogeneous clay deposits (i.e. clays without shells, granular layers varves, fibres, etc.) for preliminary design. However, it should be noted that the undrained shear strength assessed by FVT should be reduced by a correction factor as indicated by Bjerrum (1972, 1973). Chandler (1988), among others, suggested a correction factor as a function of Plasticity Index (PI) and time to failure, as presented in Figure 2.

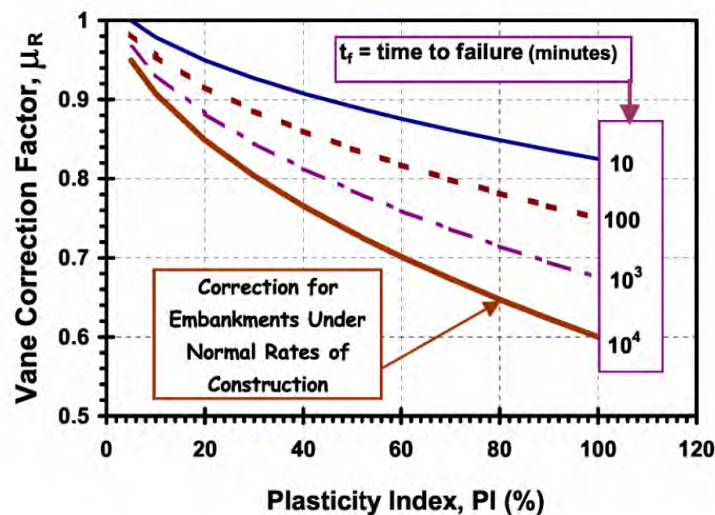


Figure 2: Proposed correction factor to raw field vane shear data from plasticity index (after Chandler, 1988)

CPTu is one of the most common field tests used for soft soil characterisation. It is relatively economic, quick and provides continuous results ideal for capturing spatial variations of soil properties (Campanella and Robertson, 1988). The peak undrained shear strength ratio can be estimated using CPT cone resistance as follows:

$$\frac{s_u}{\sigma'_{v0}} = \left( \frac{q_t - \sigma'_{v0}}{\sigma'_{v0}} \right) (1 / N_{kt}) = Q_{tn} / N_{kt} \quad (3)$$

where  $q_t$  is cone resistance, and  $N_{kt}$  is a cone factor that varies between 10 and 20, with an average value of 14. For sensitive clays ( $8 < S_t < 16$ )  $N_{kt}$  should be reduced (Robertson, 2009a). Sensitivity ( $S_t$ ) of clays may be estimated using:

where  $F_r = [f_s / (q_t - \sigma_w)] \cdot 100\%$  is CPT normalised friction ratio. For large and important projects (Levels A and B projects), site-specific  $N_{kt}$  should be determined using laboratory and FVT results. In the authors' experience, a reasonably accurate assessment of  $N_{kt}$  can be made by calibrating the results of the continuous CPT shear strength profile to discrete FVT measurements made in nearby boreholes or test holes.

**Marchetti DMT** is also inexpensive and relatively quick tool for soil profiling. However, the empirical relationships presented for DMT have limitations and need site-specific calibration for major projects (Ladd, 1991). Having said that, the original correlation for determining  $c_u$  from DMT recommended by Marchetti (1980) is as follows:

$$c_u = 0.22 \sigma'_{vo} (0.5 K_D)^{1.25} \quad (5)$$

where  $K_D$  is horizontal stress index from DMT test.

### 3.2 OVERCONSOLIDATION RATIO

Stress-strain behaviour of soils is highly stress history dependent. Therefore, it is crucial that the initial stress history of the deposit and its evolution during construction be comprehensively assessed. Pre-consolidation pressure ( $\sigma'_p$ ) is commonly used to evaluate the deposit initial stress history<sup>1</sup>.

Pre-consolidation pressure, known as 'maximum past pressure' that acted on the soil, represents a yield stress that divides recompression (elastic) from virgin compression (plastic or irrecoverable) during one-dimensional compression. Jamiolkowski et al. (1985) indicated four different mechanisms which can cause over-consolidation (OCR<sup>2</sup>) values greater than 1 within horizontal clay deposits: 1. *Mechanical*, due to removal of overburden pressure or lowering water table, 2. *Desiccation*, due to evaporation or freezing, 3. *Aging or secondary compression*, due to drained creep, and 4. *Physico-chemical*, due to natural cementation and related phenomena. Wong (2006a) provides further discussion on these mechanisms with particular emphasis placed on the effects of ageing.

For young man-made deposits such as dredged materials and mineral processing wastes, the aging and physico-chemical mechanisms can be neglected. While for natural deposits, all of the mechanisms could effectively contribute to actual OCR of the deposit (Ladd, 1991).

Determination of  $\sigma'_p$  profile plays an important role in analysis of soft soils, as it governs the initial in-situ undrained strength and represents the boundary between recompression with little deformation and small increase in  $S_u$  and virgin compression involving substantial rate of compression and strengthening of the soils. Different methods are used in practice to evaluate  $\sigma'_p$  including: laboratory consolidation tests and correlation with field test results.

#### 3.2.1 Laboratory Tests

The most common laboratory test used in practice is incremental oedometer tests (1D-consolidation test) using a load increment ratio ( $\Delta P/P$ ) of unity, which is often required to be reduced to about 0.5 to obtain a better fitted curve in the vicinity of  $\sigma'_p$  (Ladd, 1991). The test procedure and its result interpretation are well described in the literature (e.g. Casagrande, 1936, Schmertmann, 1955) and known by geotechnical engineers. Boone (2010) presents a critical appraisal of common methods used for interpretation of preconsolidation pressure using oedometer tests and proposes a new, simplified approach. However, it is worthwhile to highlight that users should be aware of the limitation of different interpretative techniques presented in the literature, as some might be only applicable to special classes of clays, e.g. structured clay, which could introduce significant errors if generalised to other types of clay. The authors consider Casagrande (1936) method is an appropriate method for ordinary clays, and for structured clays Schmertmann (1955) technique can be applied to correct the compression curves.

#### 3.2.2 Field Tests

FVT, as mentioned before, can provide a reasonable measure of undrained shear strength of homogenous clays. It can also be used to evaluate OCR using Eq. 1, as follows:

<sup>1</sup> In general, soil behaviour not only depend on can be stress path dependent

<sup>2</sup> Over Consolidation Ratio: ratio of  $\sigma_p$  to in-situ initial vertical effective stress ( $\sigma'_{v0}$ ).

$$OCR = \left( \frac{s_{u(FV)}}{\sigma'_{v0}} \cdot \frac{1}{S} \right)^{\frac{1}{m}} \quad (6)$$

where  $s_{u(FV)}$  is corrected undrained shear strength from FVT. As  $S$  and  $m$  could significantly vary for different soil types, site-specific correlations should be established.

There are also a few correlations presented in literature between OCR and CPT test results. The most common relationship is as follows (Robertson, 2009a):

$$OCR = k \left( \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \right) = k Q_{tn} \quad (7)$$

where  $k$  is the preconsolidation cone factor with an average value of 0.33, with an expected range of 0.2 to 0.5. A higher value of  $k$  is recommended in aged heavily consolidated clays. Its value should be adjusted based on previous experience in the same deposit, where possible.

Further to this, a quick assessment of pre-consolidation pressure and OCR for inorganic clays can be made by comparing the calibrated shear strength profile derived from the CPT to the theoretical normally consolidated shear strength profile calculated using Eq. 2. However, CPT interpreted OCR may not always be reliable because, as indicated by Blight (1986a, 1986b), no universal correlations can be made between penetration pore pressure-cone resistance data and stress history. This is because the shear induced pore pressure, which varies with OCR is small compared with the total pore pressure measured by the cone pressure cells. Also, cone-resistance-undrained shear strength correlations need site-specific data.

In the authors' experience, Marchetti DMT provides more reliable results than CPTu in evaluation of soil stress history (i.e. OCR) of cohesive deposits and  $K_0$  at-rest coefficient of lateral pressure. Nevertheless, it should be noted that some of the empirical relationship presented for DMT have limitations and need site-specific calibration for major projects (Ladd, 1991).

Marchetti (1980) presented an imperial relationship between OCR and  $K_D$  for fine-grained soils, based on DMT results carried out at over 40 Italian sites, which is still commonly used:

$$OCR = (0.5K_D)^{1.56} \quad (8)$$

Similar equations have been published by others based on DMT testing at different sites (Mayne and Martin, 1998; Kamei and Iwasaki, 1995), of which most have similar forms to Eq. 3. The equation form is also confirmed by analytical studies (Finno, 1993) and is influenced by soil shear strength, stiffness and compressibility (Robertson, 2009b). Therefore, for detailed design of large projects, the authors consider this equation may need to be compared to site-specific data and calibrated if required.

### 3.3 PRIMARY AND SECONDARY COMPRESSION INDICES

Relatively strict settlement criteria often apply to major roads and highways in Australia which sometimes are in effect up to a century after their construction. These criteria are used to essentially reduce maintenance costs for road pavements and to satisfy the drivers' comfort which is governed by change in grade of the pavement. Meeting these tight criteria is often a major challenge and can have a significant impact on the project cost. The reliable prediction of settlements in soft soils requires a reasonable understanding of their compressibility and time-dependent settlements (i.e. primary consolidation and creep) behaviour. This is achieved by undertaking laboratory and field tests or adopting empirical correlations to determine primary and secondary compression indices and consolidation coefficient ( $C_v$ ). The latter parameter is discussed in the next section. This section presents the most common approaches to assess primary compression ( $c_c$ ) and unloading-reloading compression ( $c_r$ ) indices as well as secondary compression or creep index ( $c_\alpha$ ) which define compressibility of soils under applied pressure.

1D-consolidation tests with incremental load stages are the most common and commercially available laboratory test used to determine soil consolidation parameters. Typically, end of primary consolidation (EOP) void ratios ( $e$ ) are plotted versus applied consolidation pressure ( $\sigma$ ) in the  $e$ -log  $\sigma$  plane.  $c_c$  and  $c_r$  are calculated as the slope of the normally consolidated (NC) compression curve and unloading-reloading curve, respectively.

For each loading stage, variation of  $e$  versus time is typically plotted in  $e$ -log  $t$  plane. If settlement reading is continued long enough after EOP, the shape of the  $e$ -log  $t$  plane often consists of three linear segments representing initial compression, primary consolidation and secondary consolidation. It is important that settlement reading is

continued at least for one log cycle of time after EOP, otherwise  $c_{\alpha}$ , the slope of the secondary consolidation segment  $e$ -log  $t$  plane, can be overestimated.

Some of commercial laboratories present only the assessed values of  $c_{\alpha}$  with test certificates and do not present the time plots. Nevertheless, the authors recommend that these plots are sought from commercial laboratories so geotechnical engineers can verify the reliability of assessed  $c_{\alpha}$  value; which its over or under estimation could result in significant design and maintenance costs, respectively.

Although consolidation testing can be expensive and time consuming, it is considered crucial in major projects (i.e. Level A and B projects). For less critical design, several empirical correlations have been presented in the literature to estimate compression indices using Atterberg limits and natural moisture content. These correlations provide an easy and inexpensive alternative method based on which the scope of the geotechnical laboratory program can be reduced to confirmatory investigation of the consolidation properties; hence reducing the amount of required consolidation testing. In the authors' experience, these correlations are used in less important projects (Level C) or to verify and obtain further data in combination with consolidation tests on more important projects.

**3.3.1 Primary Compression Index**

Correlations for primary compression index, which are widely used in practice and also considered in the case study presented later in this paper, are summarised in Table 1.

Table 1: Selected empirical correlations for primary consolidation index.

Soil	Equation <sup>1,2</sup>	Reference
All Clays	$c_c = 1.15(e_0 - 0.35)$	Nishida (1956)
Inorganic Clays	$c_c = 0.30(e_0 - 0.27)$	Hough (1957)
Undisturbed NC Clays	$c_c = 0.009(LL - 10) (\pm 30\% \text{ error})$	Terzhagi and Peck (1967)
All Clays	$c_c = 0.01325I_p$ or $c_c = 0.01W_N$	Koppula (1981)
All Clays	$c_c = 0.009W_N + 0.005LL$	Koppula (1986)
All Clays	$c_c = 0.012W_N$	US Army (1990)

- Note: 1. LL=Liquid Limit,  $I_p$  = Plasticity Index,  $W_N$ = Natural moisture content and  $e_0$ = initial (in-situ) void ratio.  
 2. In these equations, Atterberg limits and  $W_N$  are in percentage, not decimal.

**3.3.2 Secondary Compression Index**

Soil creep or secondary compression is considered one of the controversial concepts in geotechnical engineering practice. There are differing opinions on aspects ranging from the reference time when creep settlement is thought to begin to the general applicability of some of the  $c_c - c_{\alpha}$  correlations presented in the literature. Nonetheless, it is widely accepted that for the purpose of settlement analysis, the creep reference time is at EOP. Mesri and Godlewski (1977) introduced the concept of  $c_{\alpha} / c_c$  that for any natural soil, the ratio of  $c_{\alpha}$  to  $c_c$  is constant for any time, effective stress, and void ratio during secondary compression. Therefore, the value of  $c_{\alpha} / c_c$  together with EOP  $e$ -log  $\sigma$  curve completely defines the secondary compression behaviour of any consolidating soil. This concept has been validated for variety of soils and peat (e.g. Fox et al., 1992) and used for preloading design for soft ground (Mesri and Choi, 1985).

To assess site-specific  $c_{\alpha} / c_c$  values, special attention should be given to selection of  $c_c - c_{\alpha}$  pairs. They should be selected for the same time, effective stress and void ratio, as graphically depicted in Figure 3.

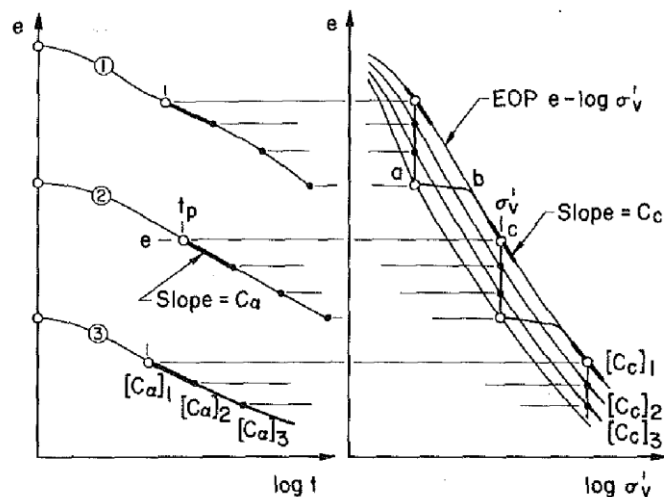


Figure 3: Corresponding values of  $c_a$  and  $c_c$  at any instant ( $e, \sigma'_v, t$ ) during secondary compression (after Mesri and Castro, 1987)

The ratio  $c_a/c_c$  is independent of time and its total range of values for all soils are in the range of 0.01 to 0.07 (Mesri and Vardhanabhuti, 2005). Table 2 shows the typical range of this ratio for different soils.

Table 2: Values of  $c_a/c_c$  for geotechnical materials (Mesri and Castro, 1987; Mesri and Vardhanabhuti, 2005)

Material	$c_a/c_c$
Granular soils including rockfill	0.02±0.01
Shale and mudstone	0.03±0.01
Inorganic clays and silts	0.04±0.01
Organic clays and silts	0.05±0.01
Fibrous and amorphous peats	0.06±0.01

### 3.4 COEFFICIENT OF CONSOLIDATION

Coefficient of consolidation controls the rate of consolidation settlement and in turn strength gain achieved over time. The laboratory 1D-consolidation test is used to estimate the vertical coefficient of consolidation,  $C_v$ , whereas a piezocone (CPTu) with pore pressure dissipation test data or flat dilatometer (DTM) can be used to estimate the horizontal coefficient of consolidation,  $C_h$ . The general trend associated with coefficient of consolidation is that the  $C_v$  values are stress dependent and that higher values are to be expected for effective stresses less than the preconsolidation stress.

Nonetheless, it is generally assumed to be constant within the interest range of stress. This is one of several simplifying assumptions made in the Terzaghi theory of 1D-consolidation. Therefore, even the best estimates of  $C_v$  or  $C_h$  from high-quality laboratory tests typically only result in predictions of consolidation settlement rate within one order of magnitude of observed values. The laboratory value of  $C_v$  is usually less than the value measured in the field (Sebatini et al., 2002). The authors, based on their previous experience, consider that actual  $C_v$  value can be 2 to 3 times larger than the value measured in the laboratory. Therefore, it is recommended that laboratory  $C_v$  values are compared to the values for  $C_h$  obtained from field dissipation tests to select a design value. Although the rate of horizontal consolidation can be greater than that in the vertical direction, for most relatively isotropic soils the ratio of lateral to vertical consolidation is not greater than two (Table 3). Nevertheless, it should be noted that in highly stratified soils such as varved clays a ratio of ten or more is anticipated (Sabatini et al., 2002). Furthermore, Figure 4

presents a correlation for  $C_v$  which can be used to judge the reasonableness of computed  $C_v$  values obtained from oedometer test data (U.S. Navy, 1971).

Table 3: Suggested anisotropic permeability of clays (after Jamiolkowski et al., 1985)

Nature of clay	$k_h / k_v = c_h / c_v$
No evidence of macrofabric or layering, or only slightly developed macrofabric, essentially homogeneous deposits	1 to 1.5
From fairly well to well-developed macrofabric, e.g. sedimentary clays with discontinuous lenses and layers of more permeable material	2 to 4
Varved clays and other deposits containing embedded and more or less continuous permeable layers	3 to 15

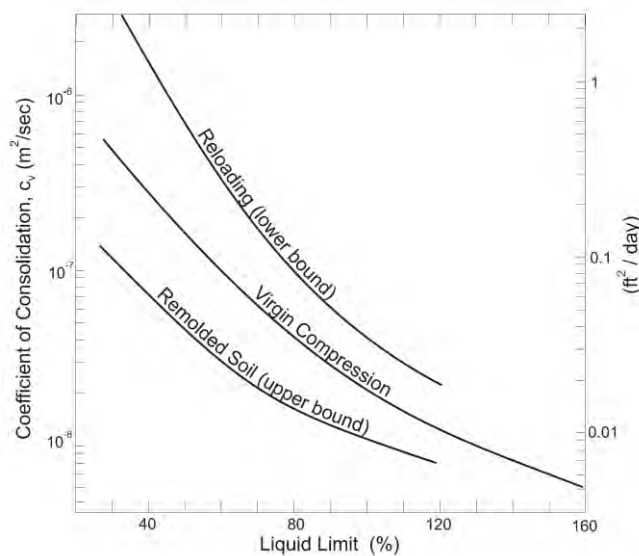


Figure 4: Correlation of coefficient of consolidation with liquid limit (after U.S. Navy, 1971 and Sebatini et al., 2002)

### 3.4.1 Field Dissipation Tests

#### *Piezcone (CPTu)*

During cone penetration excess pore pressures are generated in the soil around the cone. The dissipation rate of this excess pore pressure is controlled by soil permeability and its compressibility. Therefore, interpretation of its dissipation curve could enable estimation of quantities such as permeability and coefficient of consolidation. The initial cone generated pore pressure and its dissipation over time have been modelled based on the cavity expansion theory with critical state soil models, with coupled or uncoupled consolidation analysis. Different methods have been used to capture its initial distribution of the pore pressure around the probe such as strain path method (Baligh and Levadoux 1986), large strain finite element analysis (Houlsby and Teh 1988) or cavity expansion (Burns and Mayne 1998, Chang et al 2001) for which different equations have been presented in the literature relating dissipation data to coefficient of consolidation.

Teh & Houlsby (1991), based on the strain path method, provide the most reasonable results among others especially at  $U=50\%$  ( $U$ , being average consolidation ratio). They indicate that for monotonic pore water decays generally associated with soft to firm clays and silts, where the readings always decrease with time, the following expression may be used to determine coefficient of radial consolidation:

$$C_h = \frac{T^* r^2 \sqrt{I_R}}{t} \tag{9}$$

In which  $T^*$  is a modified time factor from consolidation theory,  $r$  is the probe radius (17.8 mm for 10-cm<sup>2</sup> cones),  $I_R = G/s_u$  ( $G$  being undrained shear modulus) is rigidity index (generally ranges between 50 and 500) and  $t$  is the

measured time during dissipation (usually at  $t_{50}$  representing 50% consolidation). There are other solutions presented for the modified time factor  $T^*$  based on different theories. For the particular case of 50% consolidation, the respective time factors are  $T^* = 0.118$  for the mid-face porous element ( $u_1$  position) and  $T^* = 0.245$  for the shoulder porous element ( $u_2$  position).

For relatively short dissipation tests, where determination of  $t_{50}$  from the normalized excess pore pressure curve is not possible, Teh (1987) proposed the following equation on the basis of square root time plot of dissipation results:

$$C_h = (m / M_T)^2 \cdot r^2 \cdot \sqrt{I_R} \quad (10)$$

Where  $m$  is initial slope of the linear part for square root time plot of normalised excess pore pressure curve,  $M_T$  is a coefficient equal to 1.15 for the typical  $u_2$  position and 10-cm<sup>2</sup> cone.

Determination of  $t_{50}$  is quite straight forward for monotonic dissipation curves when the ultimate pore pressure ( $u_0$ , hydrostatic water pressure at the time of the test) is known either from a long enough dissipation reading or measured groundwater level. Nevertheless, non-standard dissipation curves are observed, particularly for pore pressure transducers located at the shoulder ( $u_2$  position) or the shaft of a cone ( $u_3$  position). These differences may be due to shear-induced dilatancy of over-consolidated clays or sandy soils, and unloading effects for soil elements moving from the tip to the shoulder of a cone (Sully et al., 1999). In order to be able to use above equations, exclusively developed for monotonic dissipation curves, Sully et al. (1999) have proposed two correction methods to convert these curves to monotonic ones. The first method involves shifting the origin of time to that point where the measured pore pressure is a maximum (Figure 5a). The second method fits a square root of time plot to the post-maximum pore pressure dissipation curve in order to back-extrapolate the value of the initial pore pressure, as depicted in Figure 5b. However, care should be taken as shifting the origin of time ignores the effect of pore pressure redistribution in the vicinity of the cone before the measured pore pressure reaches its maximum, which could result in over-estimation of  $t_{50}$  and in turn underestimation of the value of  $C_h$ . Chai et al. (2012) have proposed an imperial relationship based on their numerical analysis of radial consolidation around piezocone which correct  $t_{50}$  measured using the above methods.

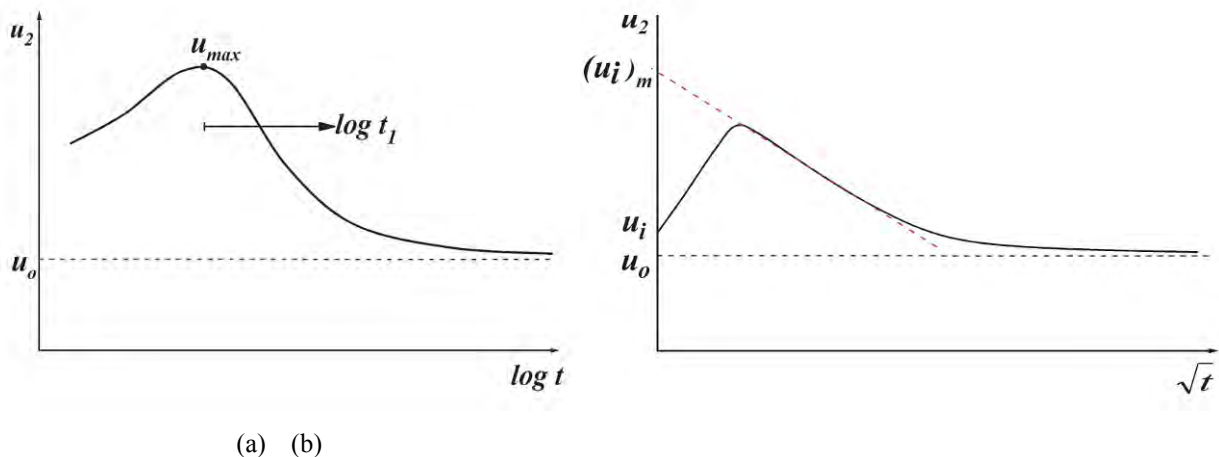


Figure 5: Correction for dilatatory dissipation: (a) logarithm of time plot, (b) square root of time plot (after Sully et al., 1999)

**Marchetti DMT Dissipation Tests**

Similarly, DMT can also be used to monitor dissipation rates of penetration pore water pressure. To the authors’ knowledge, there is no theoretical method published specifically for flat dilatometer penetration. However, the theoretical solutions developed for CPTu have been used to interpret DMT dissipation results by assuming an equivalent radius of the DMT blade and using equations similar to Eq. 9 with nominated empirical values for  $T^*$  to estimate  $C_h$  (e.g. Robertson et al., 1988).

Marchetti and Totani (1989) proposed a method consisting of sequential ‘A readings’ (the contact pressure measurement) at several times, while the blade is stopped at a specific depth, until the ‘A reading’ is stabilised. They suggest that  $C_h \cdot t_{flex}$  is constant ( $t_{flex}$  being the time associated with the contraflexure point in the A-log t curve), and would range between 5 to 10 cm<sup>2</sup>. Marchetti and Totani’s method or other DMT based methods may also be used to

estimate consolidation properties, however, it should be noted that the empirical factors used in these methods show some notable variations and extra care should be taken to select and verify appropriate site specific factors.

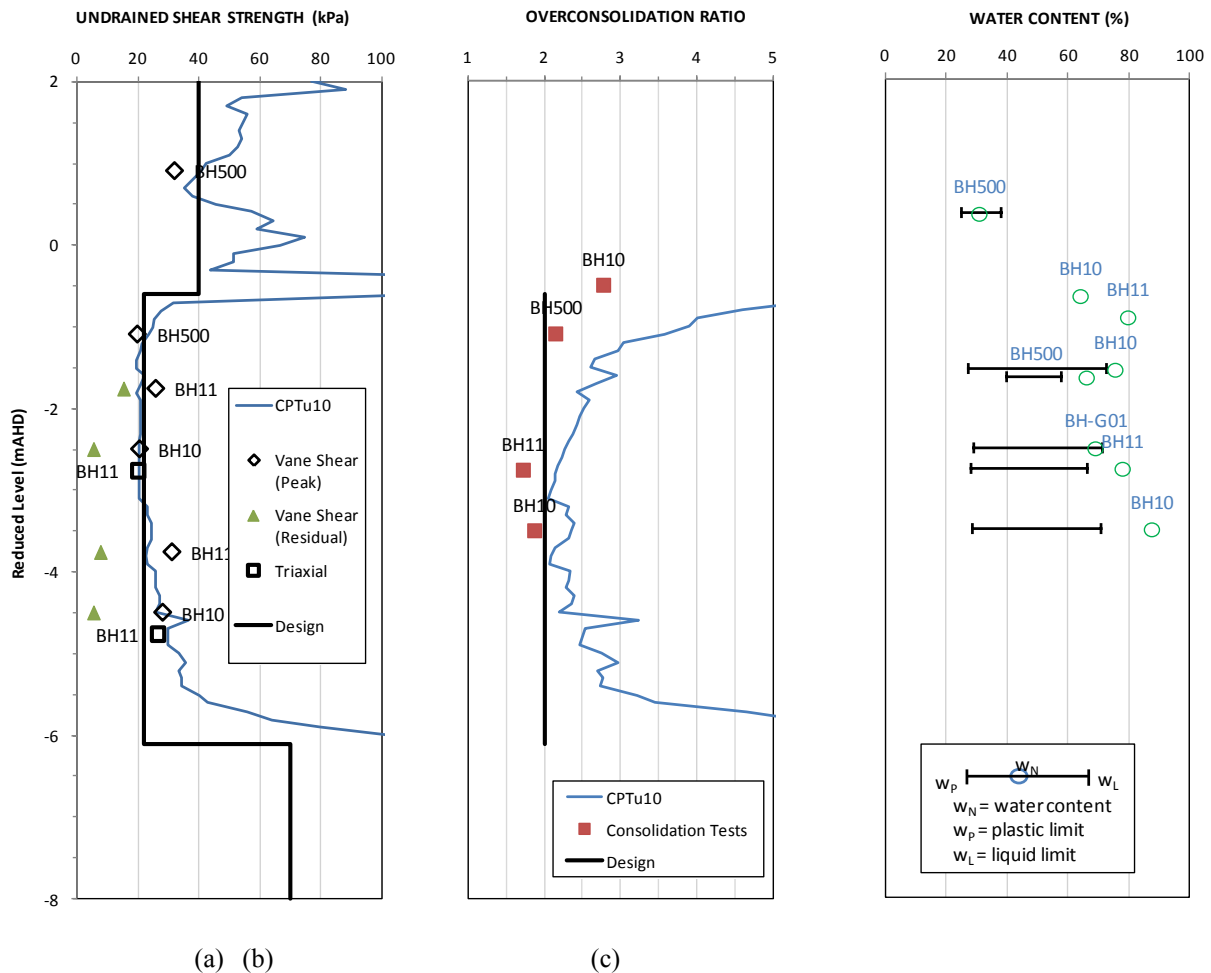
### 3.5 SELECTED DESIGN PARAMETERS USED IN CASE STUDY

The inferred geotechnical data in the vicinity of the case study as well as selected design values are shown in Figure 6. The geotechnical parameters for design were based on the assessment of field and laboratory tests, as described above, as well as from experience in similar geological deposits in the project region.

The undrained shear strength of the Holocene alluvium determined from field vane shear tests (peak values corrected for plasticity) and laboratory triaxial tests indicated good correlation with CPTu adopting a  $N_{kt}$  of 12 (Figure 6a). However, overconsolidation ratio assessed from laboratory consolidation tests were slightly lower than inferred from CPTu (Figure 6b).

There appears to be a large scatter in results of empirical relationships used for determining the modified primary compression index. However, the published relationship considered in this paper by US Army (1990) indicates the best correlation with project specific laboratory data, as depicted in Figure 6d.

A  $c_u/c_c = 0.04$  is assessed based on limited consolidation test results available in this area, which shows a good agreement with the typical ratios presented in Table 2 for inorganic clays. The ratio of 0.04 is used to correlate  $c_u$  based on some other older consolidation test results with no secondary compression index reported (Figure 6e)



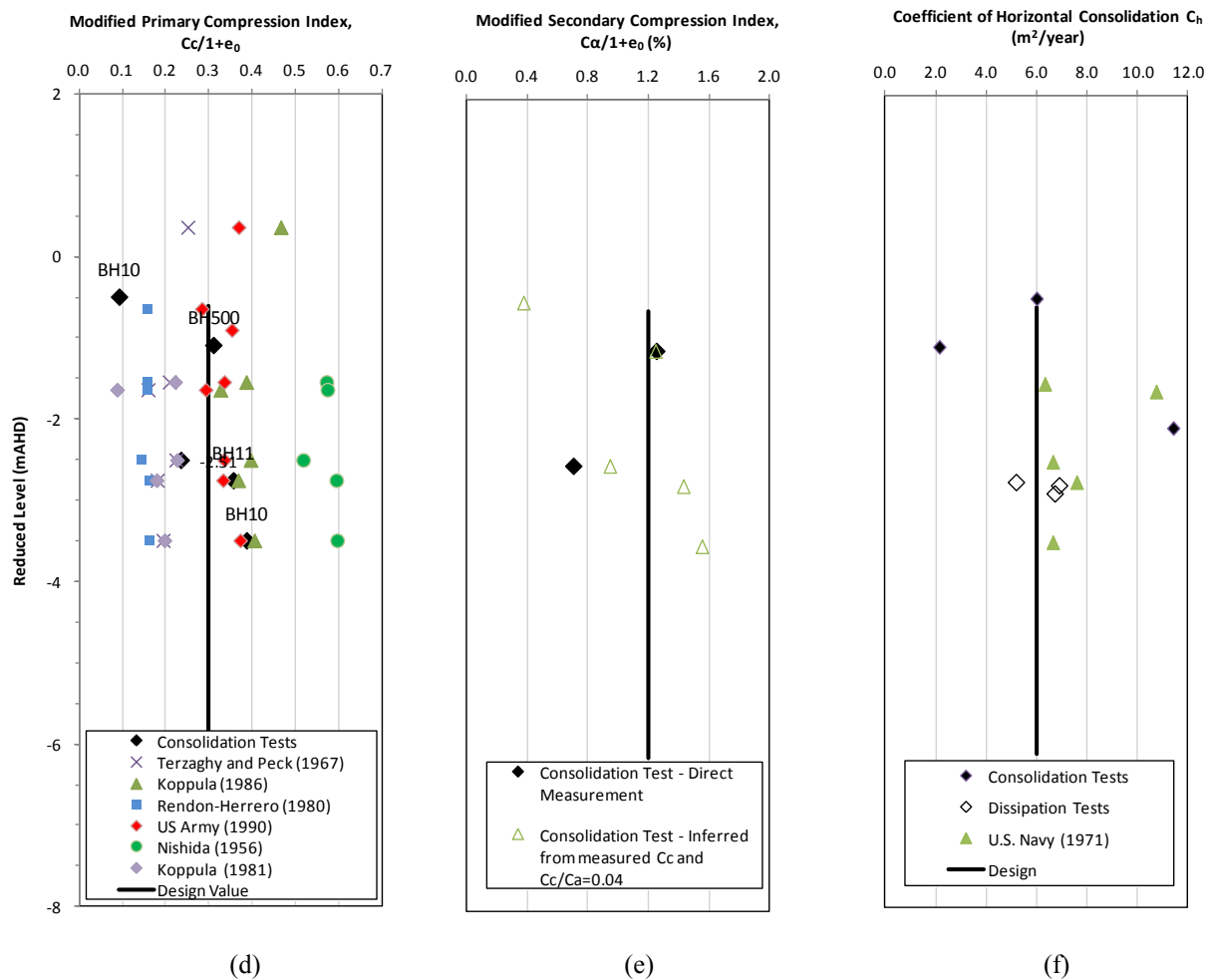


Figure 6: Typical material parameters obtained through laboratory and field testing and selected design values.

#### 4 RECOMMENDATIONS FOR SITE INVESTIGATION

A comprehensive geotechnical site investigation (SI) is one of the crucial initial steps in every project which enables safe, resilient and economic design, and identifies potential mechanical and chemical hazards related to the ground. No universal approach can be recommended for site investigations as their extent, planning, approach and sampling and testing methods adopted will depend on the site-specific circumstances and the project requirements. Standards such as BS5930-1999 'Code of practice for site investigations' or, to some extent, AS1726-1993 'Geotechnical site investigations' set out the minimum requirements for a geotechnical site investigation and provide some guidelines for planning and design of SI, methods and testing, etc. However, further to specifying type, location and target depths of field and laboratory tests, there are other factors to be considered. Such factors could significantly increase usefulness of SI results and improve soft ground characterisation, and potentially result in a more reliable and economic design with significant potential savings in the project construction or maintenance costs. Some of these factors are highlighted as follows:

- Site investigations for soft ground, in particular for the detailed design stage, should allow for a combination of CPTu (with dissipation tests), DMT, FVT, and appropriate laboratory shear and consolidation tests. As discussed in Section 3, some of these tests have advantages over others and it would be useful to consider a balance of different field and laboratory test results;
- It would be useful that the above field and laboratory tests are undertaken in the vicinity of each other at a few locations within the same soft soil deposit and at similar depths. This would enable selection and validation of site-specific correlations, where appropriate, as discussed in Section 3;

- The quantity of tests will not guarantee a successful site characterisation, but the quality of test results is also important. It is critical that field and laboratory test equipment is calibrated and standard testing procedures are strictly applied in the field and laboratory. For instance, it is important that the CPT and DMT equipment is appropriately calibrated and piezocone pore pressure porous elements are saturated and de-aired before testing; and
- It is then important that geotechnical engineers supervising site investigations be completely familiar with the standard field testing procedures and calibration of field test equipment, and periodically request field testing contractors to demonstrate calibration of their equipment, as appropriate.

Furthermore, to facilitate the interpretation of test results, the authors also recommend:

- Groundwater level is measured at the time of CPT tests and be reported with CPT results;
- $e$ - $\log t$  curves for each loading stage of consolidation tests are also presented with the test results to enable geotechnical engineers to verify the assessed consolidation parameters; and
- Consolidation reading at each stage is continued at least one log cycle of time after EOP (and preferably more within the range of applicable stresses) to enable reasonable estimation of the secondary compression index ( $c_\alpha$ ).

## 5 FINITE ELEMENT ANALYSIS

Common practices in soft ground profiling and characterisation were reviewed in Section 3. The final part of a geotechnical design would be geotechnical modelling and numerical analysis (if warranted). One of the most commonly used commercial finite element analysis software products used in the geotechnical practice is 2D PLAXIS. In this section, comments are provided about the PLAXIS material models which can be adopted for consolidating soil deposits, different approaches used to model wick drains (usually used in soft ground improvement) and modelling of rigid inclusions or piles using 2D PLAXIS.

### 5.1 MATERIAL MODEL

One of the most important steps in setting up a numerical model for an engineering problem is adoption of appropriate material models. 2D PLAXIS provides users with different models for soils among which *Soft Soil* (SS) and *Soft Soil Creep* (SSC) models can be used to simulate consolidation of isotropic soft soils. The SSC model also incorporates concepts of viscoplasticity in order to capture time-dependant creep deformations. Both models adopt Mohr-Coulomb (MC) failure criterion.

These models can predict soft soil behaviour with reasonable accuracy. However, this not only depends on the quality of basic input parameters, which include compressibility indices and shear strength parameters, but also the accurate calibration of the model as further discussed below:

#### 5.1.1 Calibration for Rate of Consolidation

It is a common practice that a constant value is nominated for coefficient of consolidation ( $C_v$ ) of soil units, based on Terzaghi's theory of 1D-consolidation. Nevertheless, there is no input  $C_v$  value to the SS or SSC models. PLAXIS calculates rate of consolidation directly based on coefficients of permeability ( $k_h$  and  $k_v$ ), which are model inputs, and coefficient of volume compressibility ( $m_v$ ), which is assessed at each loading step, based on the current stress state and the input compressibility parameters.

To achieve similar consolidation results as those anticipated based on the design  $C_v$  value, the input coefficient of permeability for each material should be estimated using the following equation:

$$k_v = C_v \cdot m_v \cdot \gamma_w \quad (11)$$

Where,  $m_v = \frac{\Delta e}{(1 + e_0) \Delta \sigma'_v}$ ,  $\Delta \sigma'_v$  is the applied pressure (e.g. average surcharge due to a new embankment),  $e_0$  is initial void ratio,  $\Delta e$  is the change of the void ratio due to the applied pressure, and  $\gamma_w$  is unit weight of water. It should be noted that  $m_v$  is not constant and depends on the current value of the effective stress. However, its average value may be simply calculated using a formulated spread sheet when initial OCR and compressibility indices of the layer are known. Then the equivalent  $k_v$  can be estimated using Eq. 11.

Alternatively, for preliminary analysis and in the absence of test results, a  $k_v = 10^{-9} m/sec$  can be considered for soft clays, as  $k_v$  for the majority of soft clays ranges between  $5 \times 10^{-9} m/sec$  and  $5 \times 10^{-8} m/sec$  (Mesri and Choi, 1985).

### 5.1.2 Calibration of M-parameter

The SS and SSC models have a cap yield surface to delineate elastic and plastic strains. In both models, a M-parameter determines the height of this elliptical cap yield surface and in turn controls the coefficient of lateral stress in normal consolidation ( $K_0^{nc}$ ). By default,  $K_0^{nc} = 1 - \sin\phi$  is considered in both models ( $\phi$  is the friction angle of the material). On this basis the M-parameter is calculated automatically such that the simulation of an oedometer test will give the most realistic results. It should be noted, however, that the SSC model in PLAXIS V8 (or older versions) adopted an M-parameter such that the simulation of an undrained triaxial test gave the best results (Waterman and Broere, 2004), using the following equation:

$$M = \frac{6 \cdot \sin(\phi)}{3 - \sin(\phi)} \quad (12)$$

This equation yields a smaller value than the automatically calculated default value. It should be noted that the M-parameter would affect calculated displacements but would not affect undrained shear strength in these models, as the both models use a MC failure criterion. Therefore, to obtain more realistic results, the M-parameter can be adjusted by changing default  $K_0^{nc}$  value in regard to the purpose of the analysis.

### 5.1.3 Calibration of SSC model for ageing effect

The cap yield surface in the SSC model continuously expands over time with a decreasing expansion rate. This enables the model to simulate the ageing effect in the soft soils (Schmertmann, 1991). The rate of the cap expansion is affected by any change in the stress state; an increase in stress, even when the stress remains within the elastic region, increases the rate of the cap expansion. Similarly, a decrease of stress decreases in the velocity of the cap expansion. However, the cap never stops expanding.

The above artificial ageing effect could affect the creep settlement analysis as the creep rate at which PLAXIS displacement calculation starts depends on the current value of OCR, which is continuously changing over time in this model. So to obtain more realistic creep settlements, it is important that the ageing effect, or in other words the amount of creep settlements occur due to the self-weight of the layer before a new load applies, is taken into the account (Wong, 2006). It is proposed then the default OCR=1 is selected for the soft soil, and after the stress initialisation of the PLAXIS model, a plastic nil step is considered with a time period of  $\Delta t$  (may be called 'geological time period') estimated as follows (Waterman and Broere, 2004):

$$\Delta t \cong \exp\left(\frac{\lambda^* - \kappa^*}{\mu^*} \cdot \ln(OCR)\right) \quad (13)$$

In which  $\lambda^* = c_c / [2.3(1 + e_0)]$ ,  $\kappa^* = 2 \cdot c_r / [2.3(1 + e_0)]$  and  $\mu^* = c_\alpha / [2.3(1 + e_0)]$  are modified compression index, modified swelling index and modified creep index, respectively. The value of OCR in this equation is the design value selected through the soil characterisation practice. In the cases where the design value for soft soil OCR changes with depth or the geotechnical model comprises more than one soft soil unit with different compressibility indices and/or OCR, one may resort to considering an average OCR value in Eq. 13 with or without an initial OCR>1 profile for different units to account for the ageing effect.

Alternatively, the SS model can be adopted for soft soils to calculate primary consolidation settlements using PLAXIS. Creep settlements can then be separately calculated using other methods (e.g. Mesri and Choi, 1985, Wong 2006b, etc.), possibly with a formulated in-house spreadsheet, where the user could appropriately control the creep starting time and the creep rate, depending on the loading history. In the authors' experience, this latter approach can provide more realistic results when compared to the PLAXIS SSC model, particularly for modelling staged construction involving unloading (e.g. surcharge removal). In several case studies assessed by the authors, the current PLAXIS formulation seems to overestimate the creep rate during and after unloading. Whilst acknowledging this may be associated with improper use or calibration of the geological time step, the concept can be difficult to rationalise and a simpler analytical approach based on well established principles is therefore appealing. The authors note a similar finding by Wong (2006b).

## 5.2 WICK DRAINS

Wick drains are installed by pushing the wick drain (and anchor plate) into the ground using a mandrel. The process can disturb a zone of soil around the centre of the drain referred to as the *smear zone*. This smearing can break down the natural structure and drainage pathways in the soil, reducing horizontal permeability and effectively reducing some of the benefits the wick drains are designed to provide.

The diameter of the smeared zone,  $d_s$ , can vary substantially from the equivalent diameter of the wick drain,  $d_w$ , depending on the soil type, the size and type of equipment used to install the wick drains and the experience of the contractor installing them. Likewise, the horizontal permeability of the smeared zone,  $k_s$ , can be significantly lower than that of the parent soil,  $k_h$ . Therefore, design is not just sensitive to the soil parameters adopted, but is also heavily dependent on construction practices.

There are various methods available to design wick drain treatments. The most straightforward typically comprise analytical solutions that are based on radial consolidation theory and apply corrections for the diameter and permeability of the smeared zone. As an example, Yeung (1997) provides a practical analytical design approach by presenting design curves used to calculate wick drain spacing, assuming different ratios of  $k_h/k_s$  and  $d_w/d_s$ . Values of  $k_h/k_s$  and  $d_w/d_s$  range from 5 to 10 and 2 to 3, respectively, depending on the level of conservatism warranted in design.

Finite element approaches can be adopted where more robust analysis is required. This is typically undertaken by modelling wick drains using one of the following approaches:

- Modelling wick drains as individual drains in plane-strain and correcting for 3D effects.
- Modelling the soil mass with a vertical permeability equivalent to that treated with wick drains ( CUR 191).
- Modelling the soil mass with a horizontal permeability equivalent to that treated with wick drains, after CUR 191, Indraratna (2000) and other works by the same author.

Modelling of individual drains is time consuming. In the authors' experience, reasonable results have been yielded more efficiently using the equivalent vertical and horizontal permeability approaches set out above.

## 5.3 RIGID INCLUSIONS

Box culvert structures for the case study project were considered to be structural elements with strict settlement criteria. After a period of ground treatment options assessment the client required that culvert extensions be supported on piles embedded beneath the soft soil deposits in deeper stiffer/denser materials. Piles typically comprised 0.6 m diameter CFA piles on a 2.4 m centre-to-centre grid. In addition to vertical loading from the box culverts, the piles needed to be designed to resist bending moments and shear forces due to lateral displacements in the soft soils underlying adjacent non-pile supported embankments.

One method to estimate bending moments and shear forces in the piles involves modelling pile rows in 2D plane-strain finite element analysis (e.g. PLAXIS) using plate elements. Pile properties are 'smeared' in the out-of-plane direction based on the pile spacing, and thus each row of piles is effectively analysed as a 'wall'. This method has the clear drawback of not allowing for 3D effects associated with soil continuity between piles. Plate elements in 2D models completely separate the soil at both sides of the element. This may be an appropriate simplification when piles have a small out-of-plane spacing compared to the pile diameter (e.g. spacing-to-diameter ratio < 2 to 3). However, for piles with a larger spacing-to-diameter ratios, and particularly in soft soil deposits where soil can more readily 'flow' between, and independent from the pile, such a method does not accurately resemble pile-soil interaction. The outcome is that bending moments and shear forces in the pile may be overestimated.

The selected design method adopted for the project allows for 3D effects associated with soil flow around the piles by first calculating the 2D free-field soil displacement below the culverts (i.e. without piles) in a plane-strain finite element PLAXIS analysis. The soil displacements are then input into the program PYGMY, which analyses the behaviour of single piles under general lateral loading. In this case, the user applies soil displacement and peak soil pressure to the pile. Soil resistance is represented by p-y springs. To perform analysis of pile groups, different multiplying factors (termed p multipliers) are applied depending on the individual piles position relative to the group (e.g. leading pile, trailing pile, side-by-side pile, etc.). The p-multipliers are based on the research of Dunnivant and O'Neill (1986) and O'Neill et al. (1990).

The two-stage process of using PLAXIS and PYGMY results in estimated bending moments and shear forces which are in general less than the method using plate elements in PLAXIS alone. It is noted that PLAXIS 2D (2012) includes a relatively new material model called 'embedded pile row' which is intended to improve the modelling of pile-soil interaction of a row of piles in the out-of-plane direction, which was not utilised in this design. A 3D

approach, for instance using 3D PLAXIS, can also be adopted to overcome the above issue by allowing soil flow around the piles, but 3D models are more time consuming to build and compute and are mostly considered for detailed design on complex projects.

## 6 ANALYSIS RESULTS

Preliminary settlement analysis in the vicinity of the proposed culvert extension indicated that 2 m surcharge (in addition to preload) would be required for a period of 9 months to meet the settlement criteria. Construction considerations in this area mandated that staging comprised placement of a 0.7 m working platform, pile construction and wick drain installation on both sides of the culvert, immediately followed by placement of the preload and surcharge. A PLAXIS model (Figure 7) was developed to assess: a) internal forces of piles induced by lateral displacement of soft soils underlying adjacent non-pile supported embankments, and b) post construction total and differential settlements. Another model was also developed at a transverse direction to assess potential impact on the existing embankment, and to assess the stability of the new embankment and filling rates, which is not presented in this paper.

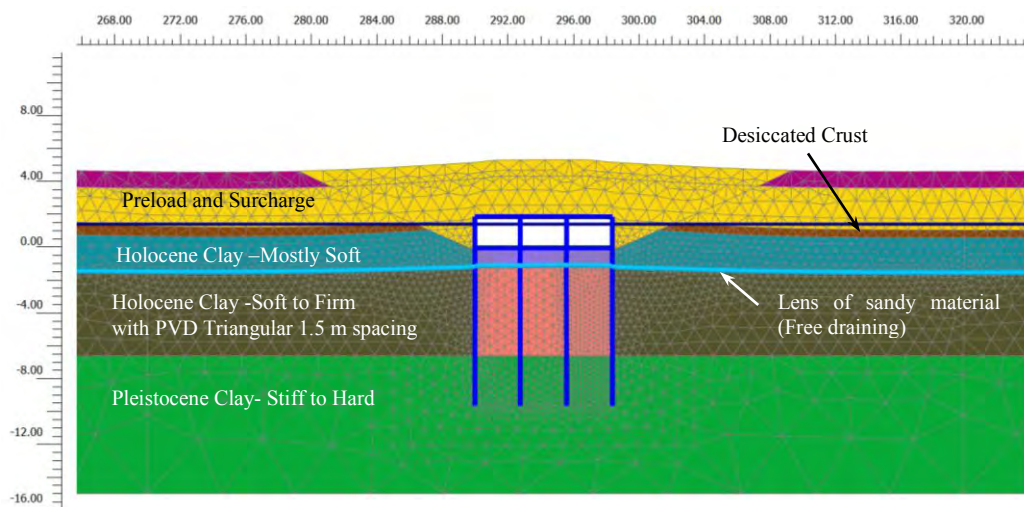


Figure 7: PLAXIS geotechnical model.

Initial analyses indicated that the outer row of piles were likely to experience un-factored bending moments of 460 kN.m for 0.6 m diameter piles, which were greater than the design limit. As a result one of the recommended options was that one outer row of 'shielding' piles be installed. The shielding piles were to be installed beyond the edge of the culvert at the same spacing as the main culvert piles (i.e. 2.4 m c/c spacing), and be provided with a minimum 2.5 m wide top slab (beyond the edge of the culvert) to support the overlying soil and traffic loads.

The shielding piles would provide settlement reduction as well as some protection from lateral soil movements to the piles supporting the culverts. In this sense they were considered ground improvement, and did not require design in strict accordance with AS 2159. The piles could therefore carry higher structural stresses and lower factors on geotechnical strength. It was also considered prudent to design a joint detail into the connection between the culvert base slab and the slab over the shielding piles, to allow some rotation while transferring horizontal loads (a pin connection).

Further to Section 5.3, this analysis utilised PYGMY to calculate pile internal forces using 2D free-field soil displacement. Figure 8a shows a comparison between free-field soil lateral displacements, and pile displacements predicted by PLAXIS and PYGMY. The bending moment diagram of outer piles (at the edge of the culvert), with pinned head, estimated by PYGMY has also been compared with those estimated by PLAXIS in Figure 8b. As shown in this figure, the corrected pile bending moments, estimated by PYGMY, are over 20% less than those predicted by PLAXIS for this case where piles have 4d (i.e. 2.4 m) spacing (d is pile diameter). This reduction of pile bending moment due to 2D to 3D correction would increase to a certain threshold value when the pile spacing increases. It is anticipated that the reduction would be negligible when the pile spacing is 3d or less (Carder and Easton, 2001).

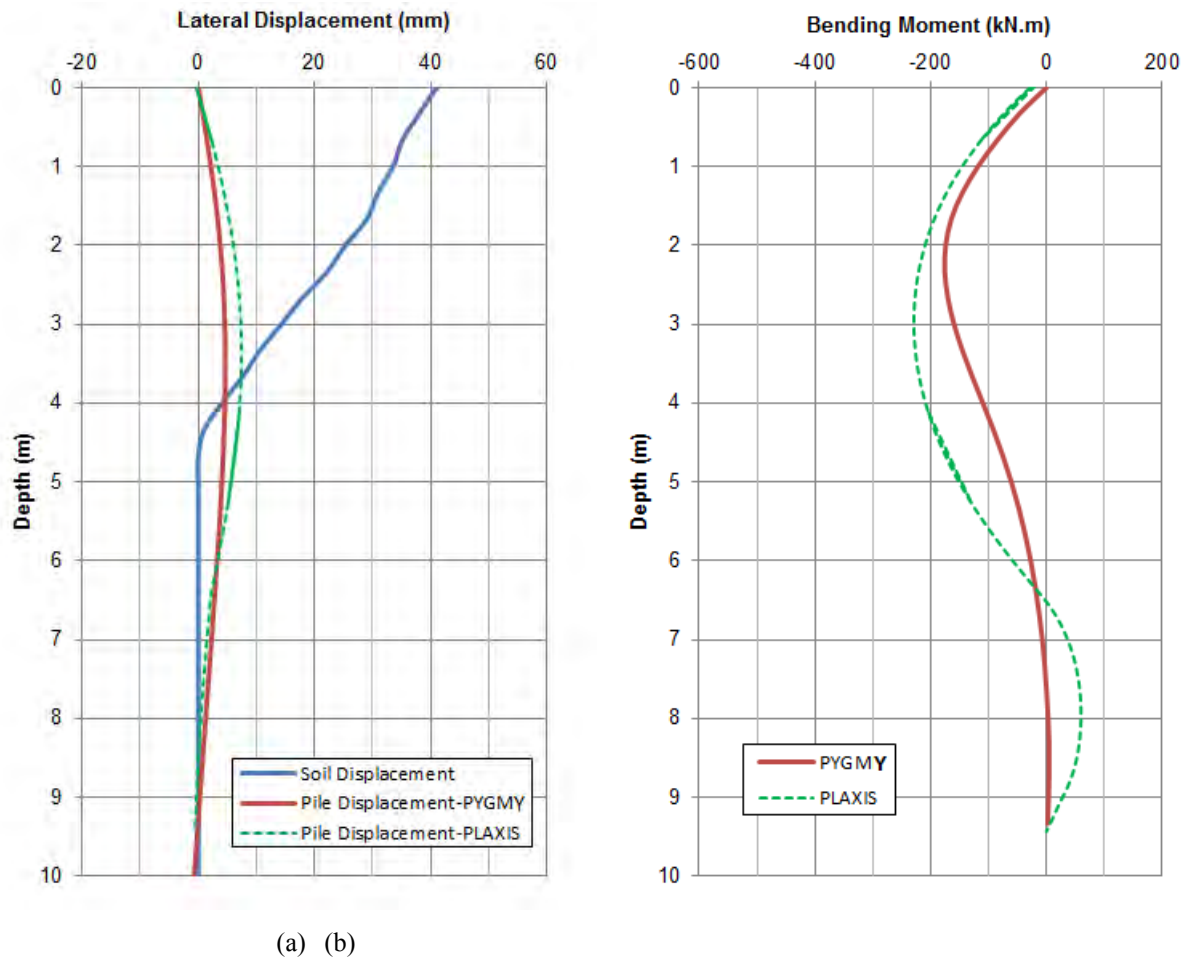


Figure 8: (a) Lateral soil and outer pile displacements, (b) PYGMY vs PLAXIS pile bending moment diagram (600 mm dia. piles with 2.4 m c/c spacing).

## 7 CONCLUSIONS

Embankment construction or widening over soft ground presents several geotechnical design challenges including stability and settlement. This paper provided a critical literature review of the current approaches to characterise soft soils and select key design parameters. The advantages and limitations of the major laboratory and in-situ field tests were discussed, as follows:

- Different field and laboratory test methods may be used to determine undrained shear strength including CPT, field vane shear, DMT and laboratory triaxial or direct shear testing. The undrained shear strength profile could be assessed using undrained shear strength ratio according to Ladd (1991) to assess the undrained shear strength profile.
- The test program and types of testing selected as part of site characterisation would depend on the level of project sophistication and importance and recommendations were provided in this regard.
- Assessment of overconsolidation ratio is generally made through interpretation of 1D laboratory consolidation tests. However estimates of OCR can also be obtained from field vane, CPT and DMT, although in these cases, correlations usually need site-specific data.
- Compression indices are assessed from interpretation of 1D laboratory consolidation tests. A number of empirical correlations have also been presented which provide an easy and inexpensive alternative method based on which the scope of the geotechnical laboratory program can be reduced to confirmatory investigation of the consolidation properties.

- The constant  $c_\alpha/c_c$  concept introduced by Mesri and Godlewski (1977) defines secondary compression behaviour of any soil. Site specific values should be determined by selection of  $c_\alpha/c_c$  pairs, selected for the same time, effective stress and void ratio.
- Coefficients of consolidation are usually determined using laboratory 1D-consolidation tests. However, interpretation of CPTu and DMT dissipation tests could also be used to determine  $C_h$  and/or  $C_v$ . It is highlighted that the laboratory value of  $C_v$  is usually found to be less than the value measured in the field. Even the best estimates of coefficients of consolidation from high-quality laboratory tests could result in estimation of the rate of consolidation settlement with an order of magnitude difference from observed values in the field.

Furthermore, recommendations were provided for site investigations mainly emphasizing on:

- Inclusion of variety of laboratory and field tests within the program, and to undertake different types of tests at a few locations in vicinity of each other and at the same depth. This is to facilitate establishment of site-specific correlation factors, as discussed in this paper.
- Quality of test results which could be affected by the test procedures, calibration of test equipment and operators skills.

Finally material models of 2D-PLAXIS and their limitations were discussed. Some methods were proposed for correct calibration of the models to obtain reasonable results. Different approaches in numerical modelling of wick drains were presented, and some of the drawbacks of modelling rigid inclusions or piles using 2D-PLAXIS were highlighted and recommendations were provided to overcome such issues.

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