SOFT SOIL ENGINEERING IN PRACTICE

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ABSTRACT

Engineering development on soft soils has grown rapidly in recent years, spurred by an increasing demand on land space for infrastructure expansion. However, the combination of poor strength, high compressibility and low permeability characteristics inherent to soft soils form a problematic suite of conditions, placing overlying structures at risk of excessive deformation and instability. Managing these conditions is particularly challenging to designers and constructors. This paper presents a summary of the critical findings and conclusions gained through the author's own experience in designing embankments over soft soils. Particular emphasis is placed on the interpretation of geotechnical parameters using empirical correlations to validate test results and design assumptions, and specific design considerations that may impact on the performance of embankments built on soft soils. This paper also discusses design criteria, methods for settlement and stability control, commonly used ground stabilisation techniques, and approach to manage soft soil risks. The purpose of the paper is to provide suggestions towards a holistic approach to soft soil engineering based on the author's experience. Examples are also provided to illustrate the author's views and findings, which are not intended to be exhaustive.

1 INTRODUCTION

Over the past 30 years of my career, construction upon soft soils have played a prominent part in many projects I have participated in. In recent years, the recurrence of soft stratum underlying project sites has increased – with population expansion necessitating the development of previously marginal landscapes. Advances in ground stabilisation techniques have also presented opportunities for development upon weak ground which was once considered undesirable and avoided.

Soft soil engineering has often been branded as a subject synonymous with risk and uncertainty. Whilst design of soft soil generally follows the principles of classic soil mechanics, the highly compressible and variable nature of this material presents unique problems and difficulties, that can only be recognised through experience, trial and error, and lessons learnt.

In recognition of the challenges associated with these materials, this paper presents a brief accumulation of the findings from my personal experience in the design and construction of embankments on soft soils. The introductory sections below present the basic framework behind geotechnical design, including site investigation, geotechnical interpretation, and suggestions for developing geotechnical models from which designs are simulated and modelled. The latter sections provide discussion surrounding the application of design criteria for stability and settlement; and specific considerations in the development of stabilisation techniques, including assessment of high strength geotextiles, strength gain, wick drain design, and bridge transition treatments.

While this paper provides a collective summary of my personal experience, and have generally resulted in successful outcomes, I would urge practitioners to form their own opinions and conclusions based on sound research and practical experience. The information provided in this paper serves to demonstrate a holistic approach to soft soil engineering and is not intended to be exhaustive. I welcome any critical discussion about these issues to further our knowledge in tackling these problematic soils.

2 SOFT SOIL CHARACTERISTICS AND ISSUES

Soft soils are generally defined as clayey materials with a large fraction of fine-grained particles and high moisture content (Bergado et al, 1996). The "soft" aspect of these materials refers to the material consistency or shear strength, which is universally classified as an undrained shear strength, S_u, of 25 kPa or less.

Their high moisture contents typically relate to full saturation, whereby all voids in the soil matrix are filled with water. As such, they follow the behavioural framework of classic (saturated) soil mechanics – where the soil mass is represented as a two-phase material, comprising solid soil particles and water. The strength of the matrix is governed by the effective stress sustained by the soil particles.

Soft soils are characterised by poor strength and high deformation properties which present challenging construction conditions. A summary of these characteristics and consequential construction outcomes is provided in Table 1 below.

Table 1: Characteristics of soft soils and associated construction outcomes

Characteristic	Construction Outcome
Low shear strength	Instability
High compressibility	Large ground movement
Low permeability (or coefficient of consolidation)	Prolonged settlement / consolidation time

When soft soils are subject to additional overburden pressures, such as embankment fill, the applied stresses are initially carried by the incompressible pore water, leading to an instantaneous increase in pore water pressure equal in magnitude to the applied load. Over time, as the excess pore water pressure gradually dissipates, the applied external load then transfers to the soil particles resulting in an increase in effective stress, which in turn leads to soil settlement and strength gains. This time-dependent settlement behaviour comprising the transfer of load from pore water to soil particles is commonly known as “consolidation”.

To successfully engineer solutions in soft soils, ground improvements are required to address potential construction outcomes with due consideration of the following:

- Risk identification and mitigation
- Cost effectiveness

The following sections discuss measures that can be undertaken to overcome the challenges when constructing on soft soils.

3 GEOTECHNICAL INVESTIGATION AND INTERPRETATION

3.1 METHODS OF INVESTIGATION

Although it is widely acknowledged that soft soils are of low strength and high compressibility, the magnitudes and extents of these conditions vary greatly between different sites due to differing deposition methods, formation history, and material conditions, etc. As such, geotechnical investigations are required at every project site to provide critical information to quantify these characteristics and the extent of soft soils. The most common and popular investigation methods include:

- Field investigation: Boreholes (BH), Vane Shear Tests (VST), Cone Penetration Tests (CPT)
- Laboratory testing: Soil characterisation and classification, Atterberg Limits, Moisture Content, Oedometer, Triaxial tests

3.2 GEOTECHNICAL INTERPRETATION

The investigation results must be adequately interpreted to derive representative geotechnical models to be used for the purposes of analysing and predicting the soft soil behaviour (Hsi et al., 2005). The basis of a geotechnical model involves establishing two main components:

- Subsurface profile
- Geotechnical parameters

Details of geotechnical models are given in Table 2 below.

Table 2: Geotechnical model required for soft soil analysis

Components	Details
Subsurface profile	Geological units Material types Consistency / density Groundwater table
Geotechnical parameters	Strength – undrained shear strength (S_u) and drained shear strength (c' and ϕ') Stiffness – Young's modulus (E'), coefficient of volume change (m_v), coefficient of compression index (C_c), recompression index (C_r) and secondary compression index (C_α) Hydraulic conductivity – permeability (k_v in vertical direction and k_h in horizontal direction), coefficient of consolidation (C_v in vertical direction and C_h in horizontal direction) Other – unit weight (γ_t), void ratio (e), over-consolidation ratio (OCR), Atterberg Limits (LL, PL, and PI), Moisture Content (MC), etc.

In order to accurately determine geotechnical properties, data from investigations are generally plotted relative to depth to provide the practitioner with a visual presentation of the subsurface conditions, and identify where potential boundaries or outliers may lie. A typical way of presenting the collected geotechnical data can be seen in Figure 1 (Yoon et al., 2014) which gives a clear picture of the subsurface profile and material properties.

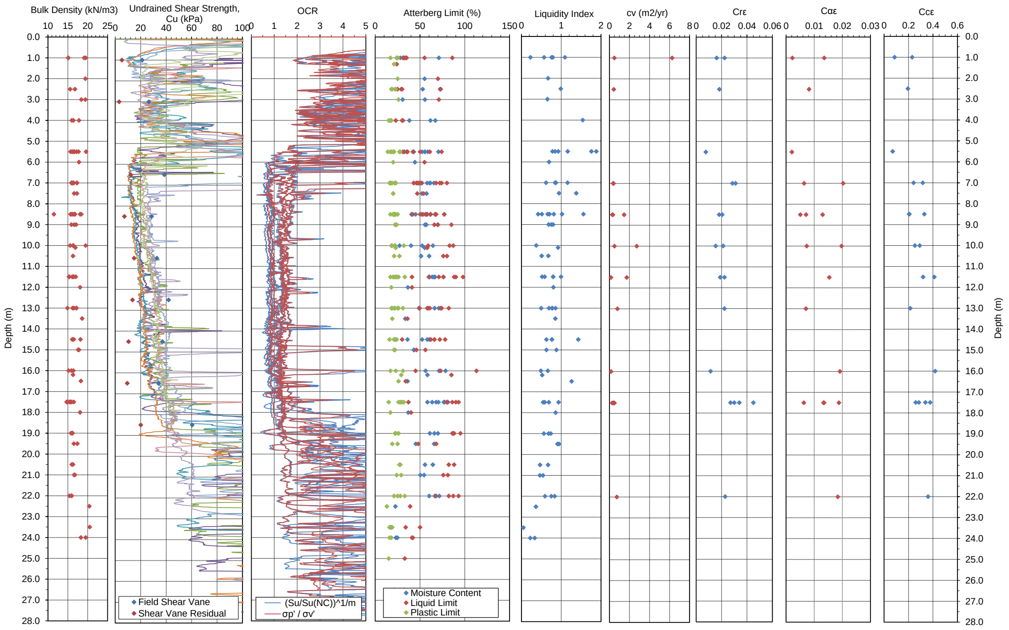


Figure 1: Presentation of geotechnical data (Yoon et al., 2014)

The quality of data obtained from the geotechnical investigations and the accurate interpretation of this information are paramount to developing a geotechnical model that is reflective of the actual ground conditions. Accurate representation of the geology will facilitate in the development of a solution best suited to the site conditions. As the success of a project hinges heavily on this process, particular attention should be paid to ensure due diligence is undertaken at this initial stage of the design.

3.3 EMPIRICAL CORRELATIONS

The geotechnical data derived from site investigation and laboratory testing are not always reliable and may not always be available. In these situations, empirical correlations can be used for validating the investigation results and/or providing possible ranges of parameter values. Some commonly adopted correlations are provided below.

3.3.1 Strength and over-consolidation ratio

Cone penetration tests (CPT) and vane shear tests (VST) are often used to determine the undrained shear strength, S_u, of soil. The continuous penetration of the CPT cone into soft soil provides a comparatively accurate and uninterrupted strength profile of the ground based on the resistance against the cone tip. However, the magnitude of resistance is not a direct measure of S_u and requires calibration with actual values. VST's offer a useful calibration, as the shearing motion of the test into the in situ soil provides a direct measurement of S_u. An example is given in the second column of Figure 1 where the CPT interpreted S_u profiles (lines) are adjusted based on the VST results (dots).

The undrained shear strength for normally consolidated (NC) clay $S_{u(NC)}$ can be estimated from the effective overburden stress, σ'_v, as summarised in Equation 1. The ratio of $S_{u(NC)}$ to σ'_v is dependent on the plasticity index (Ladd, 1991) and generally falls in the range of 0.2 to 0.25 for inorganic clays. A higher $S_{u(NC)}/\sigma'_v$ ratio is expected for organic clays (Ladd, 1991).

$$S_{u(NC)} = (0.20\ to\ 0.25) \times \sigma'_v \tag{1}$$

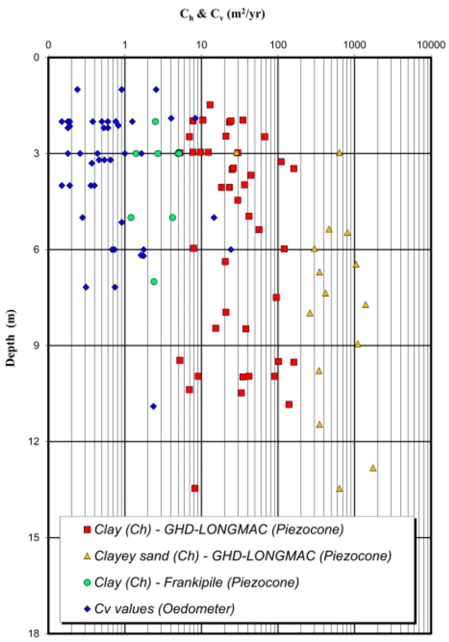
The over-consolidation ratio, OCR, can be calculated as shown in Equation 2, based on the SHANSEP approach as described in Ladd et al. (1977) and Jamiolkowski et al. (1985).

$$OCR = \frac{S_u}{S_{u(NC)}} \tag{2}$$

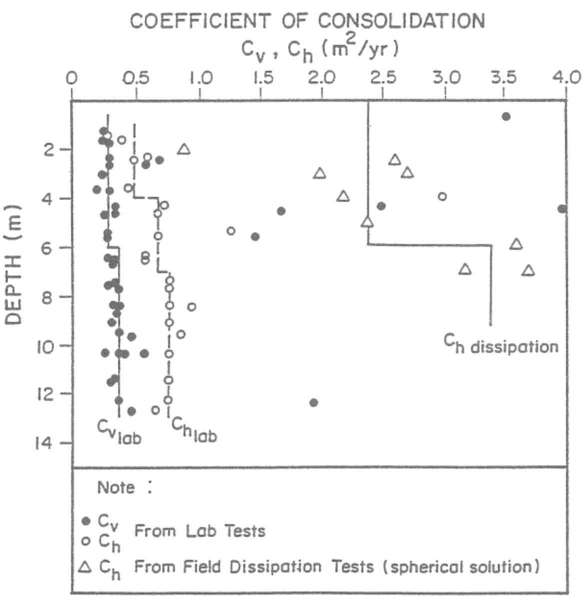
An example of an interpreted OCR profile is shown in the third column of Figure 1.

3.3.2 Coefficient of consolidation and permeability

Oedometer tests are often used to derive stiffness properties such as C_c, C_r, C_α, m_v as well as C_v values. However, due to scaling effects (i.e. small sample size), possible sample disturbance and inability to account for global effects, the C_v value derived from oedometer tests is often smaller than that obtained from in situ field tests, such as piezocones. Piezocones are used to undertake pore pressure dissipation tests (PPDT) from which C_h can be determined. Underestimation of C_v from oedometer tests has been documented in many publications, e.g. Fahey and Goh (1995), Noiray (1982), Orleach (1983), Jones and Rust (1993), Bergado et al. (1990), and Bergado (1992). The author has had similar experience as presented in Hsi (2003a), refer Figure 2a, and Hsi and Martin (2005). Figure 2a and Figure 2b indicate that results derived from oedometer tests produced C_v values that are much lower than those established from piezocone tests. Consequently, use of oedometer-derived C_v results correlated to an over-prediction of consolidation time.



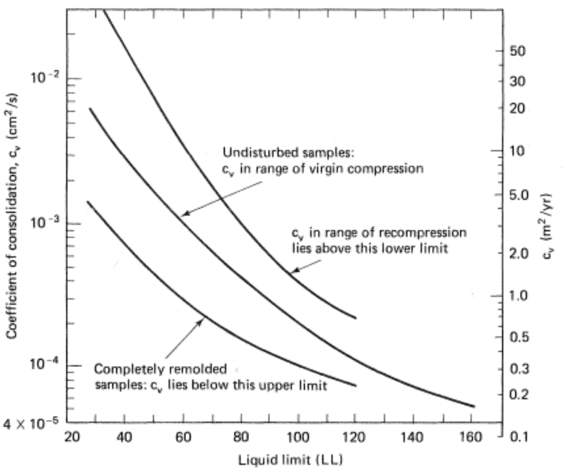
(a) C_v & C_h (Hsi, 2003a)



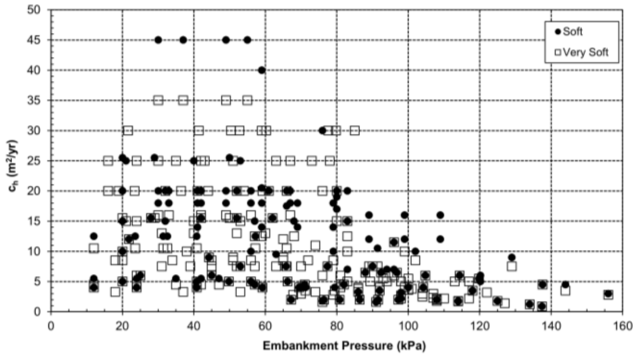
(b) C_v & C_h (Albakri et al., 1990)

Figure 2: Laboratory and field test results of C_v and C_h

In cases where piezocone field tests are not available, empirical correlations, such as C_v versus liquid limit, LL, (NAVFAC DM7.1, 1982), can be used (see Figure 3a). These correlations can also be used to verify the rationale of the C_v values assumed for design calculations. It is noted that the C_v value is higher when the soil is over-consolidated and smaller when it is disturbed.



(a) C_v vs LL (NAVFAC DM7.1, 1982)



(b) C_h vs pressure (Hsi and Martin, 2005)

Figure 3: Empirical correlations between (a) C_v and LL, and (b) C_v and embankment (total) pressure

Hsi and Martin (2005) reported that the back-analysed C_h values generally reduced as construction of the embankment proceeded, as shown in Figure 3b. This indicates that the rate of consolidation reduces as stress levels increase. The finding of reduced C_h with increased pressure is consistent with that presented by Mesri and Rokhsar (1974) and Mesri et al (1997). It also implies that C_h values reduce when the OCR reduces (as the pressure increases) and is consistent with the conditions presented in Figure 3a.

It is noted that oedometer tests produce results pertaining to the vertical coefficient of consolidation, C_v, as the water drains vertically; whilst the piezocone, PPDT, produces horizontal coefficients of consolidation, C_h, as the water drains horizontally. Due to the heterogeneity and deposition history of soil, C_h values are generally greater than C_v values.

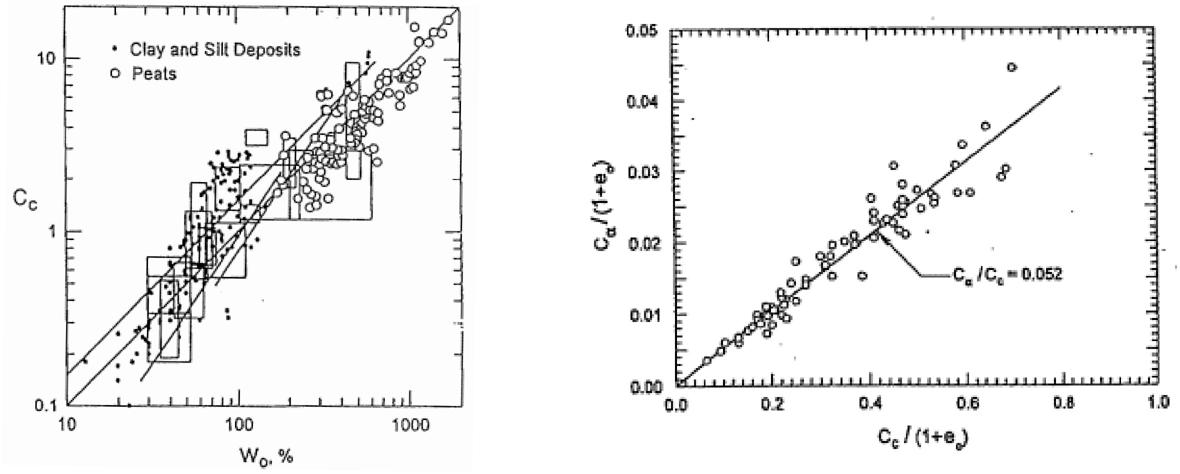
In-situ pump out or slug tests can also be carried out to determine soil permeability, k. Table 3 shows the ranges of k_h/k_v values for various natures of clay (Jamiolkowski et al., 1985). The ratio of C_h/C_v is equal to k_h/k_v, hence Table 3 also applies to the ratio of C_h/C_v. As such, the ratio of C_h to C_v could be up to 15.

Table 3: Range of possible ratio k_h/k_v for soft clays (Jamiolkowski et al., 1985)

Nature of Clay	k_h/k_v
No or slightly developed macrofabric, essentially homogenous 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

3.3.3 Deformation parameters

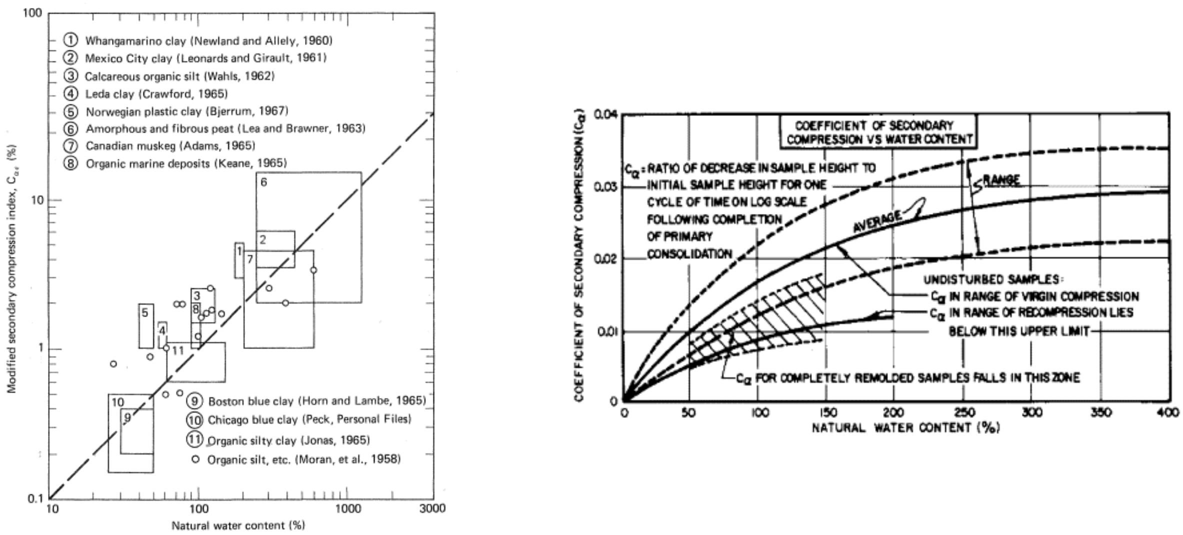
When there is lack of specific test results, published correlations with Atterberg limits (PL, LL and PI) and moisture content (MC) can be used as indirect derivations of soil deformation parameters. For example, C_c can be estimated from MC or LL values (Mesri et al., 1997; Terzaghi and Peck, 1967). Mesri et al. (1997) also presented a strong correlation between C_α and C_c as shown in Figure 4.



(a) C_c vs MC (W_o) (b) C_α vs C_c

Figure 4: Empirical correlations between (a) C_c and MC, and (b) C_α and C_c (Mesri et al. 1997)

Similarly, C_α values can also be correlated with MC values, as shown in Figure 5 (Mesri, 1973; NAVFAC DM7.1, 1982). Figure 4 indicates that C_α or $C_{\alpha\varepsilon}$ increases with increasing MC and C_α reduces when the soil is remoulded.



(a) $C_{\alpha\varepsilon}$ vs MC (Mesri, 1973) (b) C_α vs MC (NAVFAC DM7.1, 1982)

Figure 5: Empirical correlations between (a) $C_{\alpha\varepsilon}$ (creep strain) and MC, and (b) C_α and MC

4 DESIGN CRITERIA AND APPROACHES

4.1 DESIGN CRITERIA

As with all engineered structures, design criteria for embankments constructed on soft soils are centred around the prevention of two principle modes of failure, i.e. structural collapse and excessive deformation. Prevention of structural collapse is associated with maintaining the stability of the combined embankment and soft soil system. This is achieved through the adoption of factors of safety (FOS) against deep seated slips or sliding failure through the soft soil. Excessive deformation is prevented through adherence to deformation limits deemed acceptable to the supporting structures and rideability criteria. Such criteria are typically specified in the form of total and differential settlements.

4.1.1 Stability

Due to the time-dependent and stress-dependent nature of soils, assessment of collapse conditions requires consideration of the duration of applied loads as this impacts on the expected soil behaviour. Accordingly, stability analyses are usually delineated into “short term” and “long term” design cases, which relate to the available strength in the soil when insufficient time has elapsed for excess pore water pressures to dissipate for short term conditions – and conversely for long term conditions.

Three scenarios are generally considered for the short term, as listed below. Under these loading scenarios, built-up pore pressures will have insufficient time to dissipate, hence the available shear strength is expected to be characterised by undrained shear strength parameters, e.g. S_u, in isolation of changes to effective stress.

- During construction,
- Rapid drawdown during a flood event, and
- Earthquake

For long term post-construction stability analysis, it is conventionally recognised that sufficient time will have elapsed for pore pressures to dissipate, whereupon drained shear strength parameters, i.e. c' and ϕ', are used to characterise the available shear strength, and changes in effective stress are taken into account.

Typical FOS values (based on author's past experience) for the various stability assessment scenarios is given in Table 4.

Table 4: Typical stability assessment criteria

Term	Condition	FOS	Parameters
Short Term	Construction	1.2 –1.3	Undrained
	Rapid Drawdown	1.2 – 1.3	Undrained
	Earthquake	1.0 – 1.1	Undrained
Long Term	Post construction	1.4 –1.5	Drained

4.1.2 Settlement

Embankments constructed over soft soils will be subjected to various phases of settlement, including:

- Immediate settlement – occurs immediately after the loading is applied and settlement generally occurs under the undrained condition
- Primary consolidation settlement (i.e. consolidation settlement) – occurs when excess pore water pressure dissipates and is time-dependent.
- Secondary consolidation settlement (i.e. creep settlement) – occurs under constant effective stress and continues linearly on a logarithm-of-time plot.

Settlement criteria are usually time-dependent, with limiting magnitudes specified for certain periods or project milestones, depending on the requirements of the end user or application. Settlement criteria are generally specified in the form of total and differential settlements over defined periods. Based on author's past experience, criteria adopted for soft soils in NSW may be summarised as follows:

- Maximum total settlement between 100mm and 200mm over a pavement design life of say 20 to 40 years, i.e. post construction settlement
- Maximum differential settlement of 0.3% (for rigid pavement) to 0.5% (for flexible pavement) measured as Change in Grade (CIG).

The differential settlement (CIG) criterion is determined by the ride quality, road users' safety and pavement structural integrity. CIG is defined as the change of two adjoining slopes as shown in
Figure 6 and calculated as per Equation 3:

$$CIG = \left(\frac{y_1 - y_2}{x_1 - x_2} - \frac{y_2 - y_3}{x_2 - x_3}\right) \times 100\% \qquad \textbf{(3)}$$

Half chord length, T, is defined in Equation 4:

$$T = |x_1 - x_2| = |x_2 - x_3| \qquad \textbf{(4)}$$

The value of T on recent projects the author has partaken in, has generally fallen between 5m to 10m.

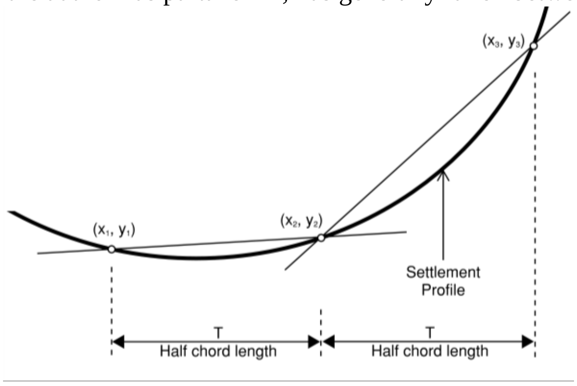


Figure 6: Measurement of Change in Grade, CIG

4.2 DESIGN APPROACHES

In recognition of the dual design criteria for stability and settlement, and numerous ground treatment options to satisfy these requirements, a design flow chart was developed for the Yelgun to Chinderah (Y2C) Freeway project in NSW, as shown in
Figure 7 (Hsi, 2003a; Hsi and Martin, 2005). This flow chart provides a systematic design process, ensuring both the stability and settlement criteria are achieved whilst the most cost-effective ground treatment options are chosen.

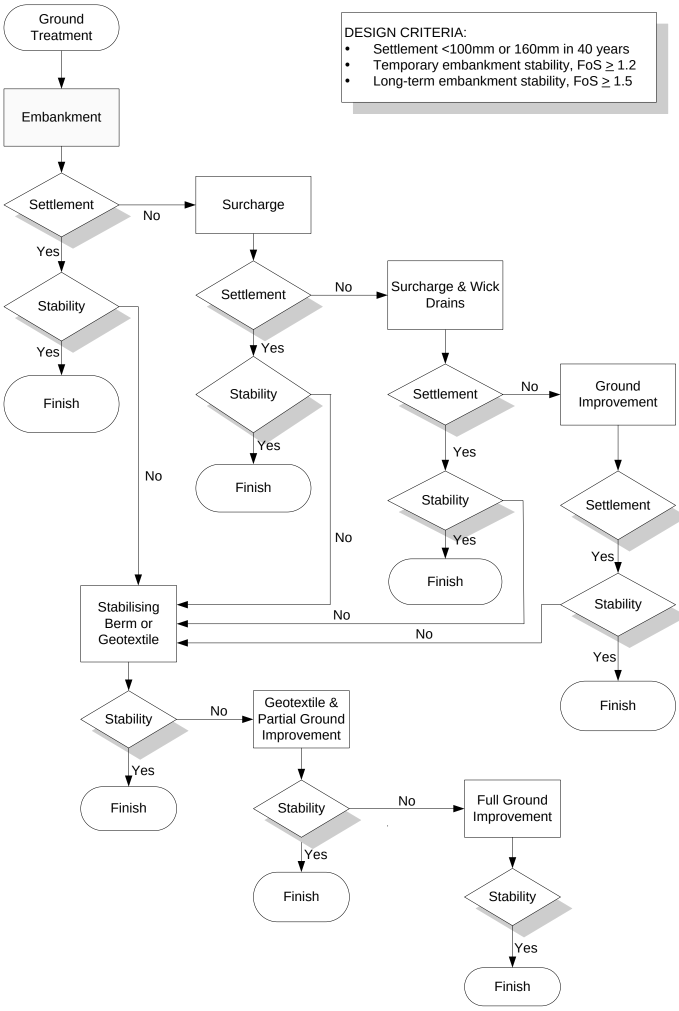


Figure 7: Design process for soft ground treatment (Hsi, 2003a; Hsi and Martin, 2005)

5 GROUND TREATMENT MEASURES

Due to the low strength and high compressibility characteristics of soft soils, embankments constructed on these materials can rarely attain the necessary stability and settlement requirements without some form of treatment. Commonly adopted measures can be broadly categorised as “above ground” and “below ground” solutions.

5.1 ABOVE GROUND SOLUTIONS

5.1.1 High Strength Geotextiles

Layers of polymeric grids or fabrics, i.e. high strength geotextiles (HSG’s), can be installed near the base of embankment fills to provide added resistance against deep-seated instabilities. Suitably installed HSG’s will intercept critical slope failure surfaces, allowing the transfer of embankment loads to the HSG by means of a frictional bond. The transferred forces are carried by the geotextile via tension within the polymeric fibres or grid – effectively restraining the slope (Exxon, 1994).

5.1.2 Stabilising Berms

Stabilising berms provide resistance against slope instability by effectively surcharging the toe of embankments and increasing the effective shear strength of the foundation soil below the berm footprint. This solution is considered relatively cost-effective provided sufficient space is available at the embankment toe and the cost of fill is not inhibitive.

5.1.3 Staged Embankment Construction

This process involves deliberately slowing down the embankment filling progress to allow sufficient time for dissipation of excess pore water pressures and hence increase in effective stresses and strengths, prior to installing the next layer of embankment fill. Adequate construction staging can mitigate against slope instabilities which may have occurred under more rapid embankment filling rates.

5.1.4 Preloading/Surcharging

Should excessive settlement occur post-installation of the overlying pavement or structure, their integrity and serviceability can be compromised. Preloading and surcharging techniques can be used to accelerate the settlement time, forcing the expected settlement to occur within a much shorter period of time. This is achieved by applying overburden loads in excess of the final embankment profile during construction. Prior to installation of the pavement or structures, the excess overburden can be removed, thus ensuring only minimal residual settlement remains when the settlement sensitive structures are applied (Hsi and Martin, 2005).

5.1.5 Lightweight Fill

Lightweight fills, including materials such as geofoam or bottom ash, can be used in place of conventional earthfill to construct the embankment. These materials weigh approximately 1%-2% of earthfill for geofoam and 60%-70% for bottom ash. As destabilising forces are directly proportional to the density of embankment fill, reduced fill weights will increase the margin of safety against instability, and associated settlements will diminish proportionately. Reference can be made to Yoon et al. (2014) for a documented application of geofoam embankments on soft soil. Despite their attractiveness in mitigating conventional earthfill embankment issues, geofoam has a number of limitations and restrictions, including its high cost compared with earthfill, susceptibility to floatation during flooding, and vulnerability to chemical and fire attack.

5.2 BELOW GROUND SOLUTIONS

5.2.1 Remove and Replace

In situations where the soft soil layer is at a shallow depth, a simple solution would be to remove it and replace with compacted engineered fill. This solution would be recommended where soft soils exist in isolated pockets, but would be cost and time-prohibitive if removal and replacement were required for significant extents and depths along the site.

5.2.2 “Soft” Inclusions

In order to realise potential strength gains and settlement magnitudes quickly, the consolidation process can be accelerated through the introduction of soft inclusions into the soil, such as prefabricated vertical drains (PVD) or vacuum consolidation (Hsi and Martin 2005; Hsi and Lee 2010). PVD’s are essentially drainage conduits allowing the built up pore water to dissipate at a much accelerated rate compared with untreated ground. Vacuum consolidation involves application of vacuum pressures in the soil, resulting in increases in effective stress and hence strength gains. Consequently, soft soils no longer pose constraints on the embankments being built at an accelerated rate.

5.2.3 "Semi-rigid" Inclusions

"Semi-rigid" inclusions provide partial strengthening of the ground with or without added drainage capacity. For example, sand piles and stone columns have high drainage capacities to accelerate consolidation, and hence hasten the strength gain and settlement process, whilst also providing partial support to the embankment. Deep soil mixing, on the other hand, relies solely on the improved strength of mixed cement-soil columns to carry the embankment loads.

5.2.4 "Rigid" Inclusions

"Rigid" inclusions provide structural support, by introducing stiff vertical elements into the soft ground, such as concrete injected columns, CFA piles, timber piles and precast concrete piles (Hsi, 2001, Hsi and Martin, 2005, Hsi et al. 2008; Hsi et al. 2013). Installations are formed in a regular grid pattern, and when viewed holistically, rigid inclusions effectively improve the strength and reduce the compressibility of the treated ground mass. With these elements, stability and settlement issues are significantly reduced. These methods are normally used as a final resort due to their high cost, in areas where settlement criteria are particularly stringent or when scheduling pressures do not allow any time for consolidation to take place.

6 DESIGN CONSIDERATIONS

6.1 SPECIFIC CONSIDERATIONS

This section discusses some specific considerations that are often incorporated into soft soil treatment design, including:

- Strength gains of soft soil associated with consolidation
- High strength geotextile for embankment basal reinforcement
- Smear effects caused by wick drain installation
- Creep settlement influenced by over-consolidation ratio
- Lateral soil movement associated with embankment settlement
- Delayed pore water pressure measurement during soil consolidation
- Load transfer platforms over rigid inclusions
- Transition treatment at bridge approaches

6.2 STRENGTH GAINS

The shear strength of soil, τ, is governed by the effective stress experienced between the solid particles of the soil matrix and its stress history, i.e. the stresses previously experienced by the material, expressed in the form of an over-consolidation ratio (OCR). The shear strength of soil can be expressed as shown Equation 5:

$$\tau = S_{u(NC)} \times OCR^m \quad \textbf{(5)}$$

where, $S_{u(NC)}$ = shear strength of normally consolidated clay and can be approximated as $R_{(NC)}$ (normally 0.2 to 0.25) multiplied by the effective overburden pressure σ_v', as expressed in Equation 1. The exponent, m, is a constant value ranging between 0.8 and 1.0. It is noted that when m = 1.0, Equation 5 is identical to Equation 2. Equation 5 can be re-written as Equation 6:

$$\tau = R_{(NC)} \times \sigma_v' \times OCR^m \quad \textbf{(6)}$$

As mentioned in Section 2, the application of external loads onto soft soils initially translates into a build up of pore water pressure. Consolidation of the soil ensues, whereupon the excess pore water pressure dissipates, followed by an increase in effective stress, $\Delta\sigma$, experienced by the soil matrix. Such an increase in effective stress will result in strength gains in the soil. The magnitude of strength gain and available shear strength in the soil can be quantified numerically as shown in Equations 7 to 9:

$$\tau_i = R_{(NC)} \times \sigma_{vi}' \times OCR_i^m \quad \textbf{(7)}$$

$$\tau_t = R_{(NC)} \times (\sigma_{vi}' + \Delta\sigma) \times OCR_t^m \quad \textbf{(8)}$$

$$\Delta\tau = \tau_t - \tau_i \quad \textbf{(9)}$$

where, the subscript i represents the initial condition before loading and t represents the condition when an increase in effective stress $\Delta\sigma$ is achieved. The magnitude of $\Delta\sigma$ experienced by the soil matrix at any point in time, relates to the progress of consolidation, i.e. degree of consolidation, U%, refer to Equation 10 below. The process continues until a point where 100% consolidation has taken place, and the full magnitude of the applied overburden stress, i.e. $\sigma'_{vf} - \sigma'_{vi}$, is fully translated into an increase in effective stress felt by the soil matrix.

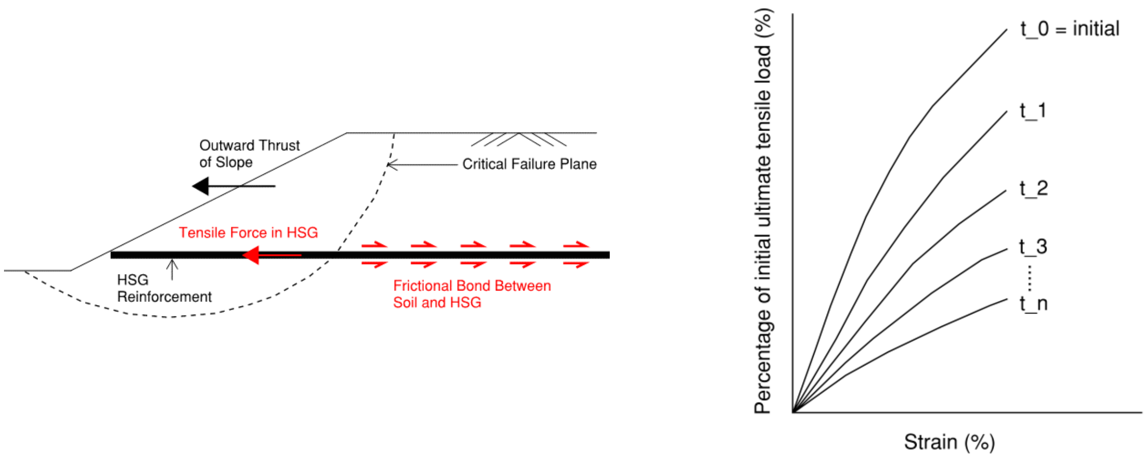
$$\Delta\sigma = \sigma'_{vf} \times U\% - \sigma'_{vi} \tag{10}$$

The above calculations demonstrate that strength gains are related to the effective stress increase and the OCR achieved before and after the stress increase.

6.3 HIGH STRENGTH GEOTEXTILES

High Strength Geotextiles (HSG's) are commonly used as basal reinforcement within embankments founded upon soft soils. The polymeric composition of these products bring high magnitudes of tensile strength to the system to prevent against slope instabilities and bearing type failures.

Suitably installed HSG's will intercept critical slope failure surfaces, allowing the transfer of embankment loads to the HSG by means of a frictional bond. The transferred forces are carried by the geotextile via tension within the polymeric fibres or grid – effectively restraining the slope (Exxon, 1994). The reinforcement mechanisms are illustrated in Figure 8a, where the geotextile is shown intersecting a critical failure surface in an embankment founded upon soft soil.



(a) Load transfer in HSG (b) Isochronous creep curves of HSG

Figure 8: Mechanisms of HSG (Exxon, 1994)

HSG's are "passive" forms of reinforcement, whereby activation of their capacity occurs whence destabilising forces are applied, causing the reinforcement to undergo tensile strain. The polymeric materials in HSG's undergo creep, i.e. deformation under sustained application of constant tensile loads. The combination of strain-dependent and time-dependent behaviours in HSG's create a complex mix of material characteristics, which can be captured in three main design considerations according to Li and Hsi (2016):

- Time, t
- HSG strength, T
- HSG strain, ε

These attributes are neatly summarised in isochronous creep curves, a typical example of which is illustrated in Figure 8b. The y-axis relates to the product strength, x-axis to the corresponding strain, and different curves are presented at various times post load application.

The ubiquity of these products has spawned numerous guidelines and codes covering the design and selection of proprietary geotextile products. However, the design of basal reinforcements still has elements of ambiguity, resulting in confusion and inconsistency of design outcomes in the industry today.

For the design of HSG's, reference can be made to Li and Hsi (2016) which discusses the various approaches used in Australia, and suggests following a limit state approach that is largely based upon the principles of basal reinforcement design documented in British Standard, BS8006 – Code of Practice for Strengthened/Reinforced Soils and other Fills (BS8006, 1995 and 2010) and recommendations found in literature.

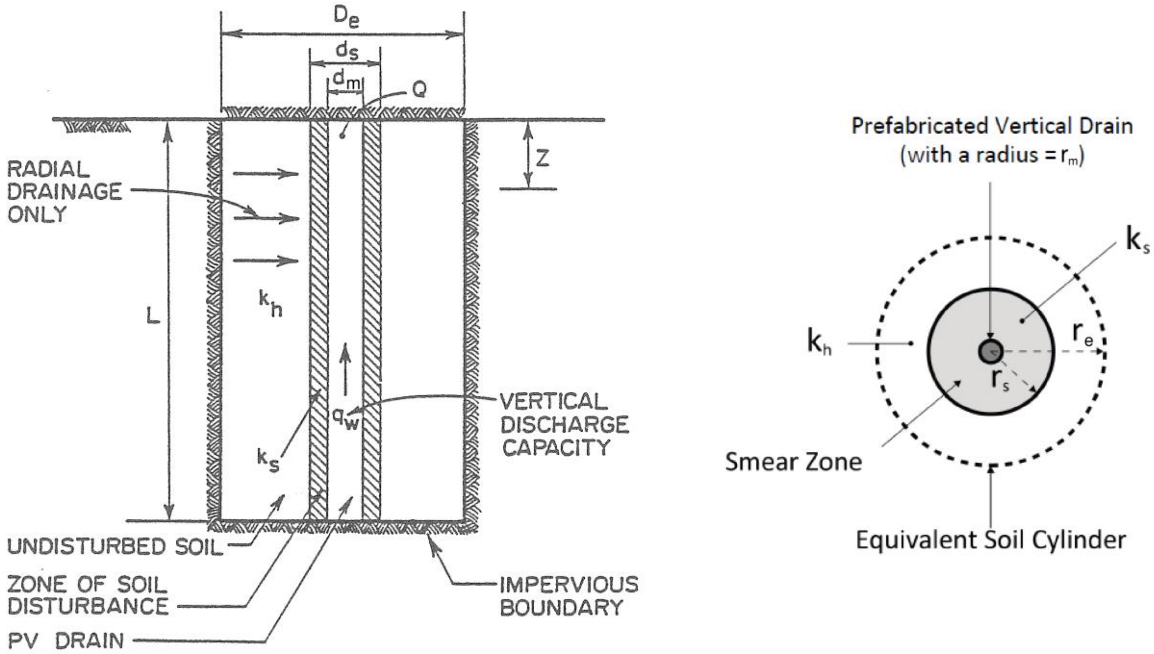
6.4 SMEAR EFFECT OF WICK DRAINS

Wick drains are a commonly adopted form of soft soil treatment used to accelerate the consolidation process. Wick drains comprise a synthetic core with moulded channels wrapped in filtration fabric. The composite strip is installed vertically into soft ground, using a crane or excavator with a steel mandrel attachment. The drains are effectively driven in place using vibratory hammer of static method techniques, whereupon the mandrel is withdrawn and the wick drain is left in place. The drains are installed at regular intervals, forming a grid like arrangement of vertical conduits over the area of interest.

Each wick drain provides a drainage path through which built up pore water pressures can be readily discharged. Due to the shortened radial distance through which pore pressure is required to travel prior to discharge, the dissipation time and consolidation settlement are effectively hastened as a consequence. In the absence of wick drains, pore pressure is limited to dissipation only through the upper and lower bounds of the soft soil layer, forcing pore water to travel longer distances in low permeability soil, thereby lengthening dissipation times.

When quantifying the accelerative impact of wick drains, practitioners must account for the alteration of in situ soil conditions created by the installation of these elements. The "smear effect" is a well-documented phenomenon used to describe reduced soil permeability at the interface between the wick drain and soil. This condition is caused by disturbance of the soil during installation. Consequently, expected consolidation rates are reduced, leading to prolonged consolidation times. Smear effects are defined by the following factors (also see Figure 9):

- Extent of the smear zone
- Reduced permeability within the smear zone



(a) Vertical section (Rixner et al.1986) (b) Cross section

Figure 9: Smear zone around a wick drain

Published information (refer Table 5) indicates that permeability of the soil before disturbance, k_h, may be up to six times the permeability of the disturbed (smeared) soil, k_s. The extent of the smear zone r_s (radius from centre of the wick drain) can be measured up to five times the equivalent radius of the wick drain r_m (also see Table 5).

Table 5: Smear zone parameters (Indraratna et al., 2005)

Source	Extent	Permeability	Remarks
Barron (1948)	$r_s = 1.6r_m$	$k_h / k_s = 3$	Assumed
Hansbo (1979)	$r_s = 1.5\text{-}3r_m$	Open	Based on available literature at that time
Hansbo (1981)	$r_s = 1.5r_m$	$k_h / k_s = 3$	Assumed in case study
Bergado et al. (1991)	$r_s = 2r_m$	$k_h / k_v = 1$	Laboratory investigation and back analysis for soft Bangkok clay
Onoue (1991)	$r_s = 1.6r_m$	$k_h / k_s = 3$	From test interpretation
Almeida et al. (1993)	$r_s = 1.5\text{-}2r_m$	$k_h / k_s = 3\text{-}6$	Based on experience
Indraratna et al. (1998)	$r_s = 4\text{-}5r_m$	$k_h / k_s = 1.15$	Laboratory investigation (for Sydney clay)
Chai and Miura (1999)	$r_s = 2\text{-}3r_m$	$k_h / k_s = C_f\,(k_h / k_s)$	C_f the ratio between lab and field values
Hird et al. (2000)	$r_s = 1.6r_m$	$k_h / k_s = 3$	Recommend for design
Xiao (2000)	$r_s = 4r_m$	$k_h / k_s = 1.3$	Laboratory investigation (for Kaolin clay)

r_s = radius of smear zone

The variation of permeability adjacent to a wick drain due to smear effects is presented by Onoue et al. (1991), as shown in Figure 10. It is clear that permeability of soil is most affected immediately adjacent to the wick drain (Zone III) and gradually increases away from the wick drain (Zone II) until full permeability is reached in the undisturbed zone (Zone I).

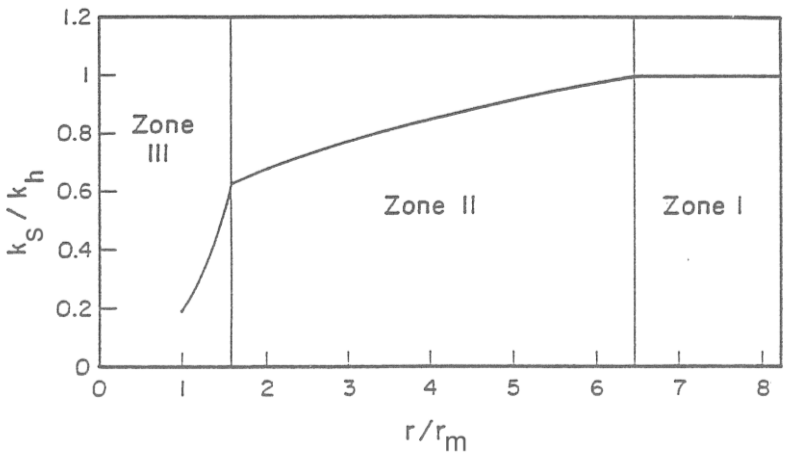


Figure 10: Variation of permeability with radius of wick drain (based on Onoue et al. 1991)

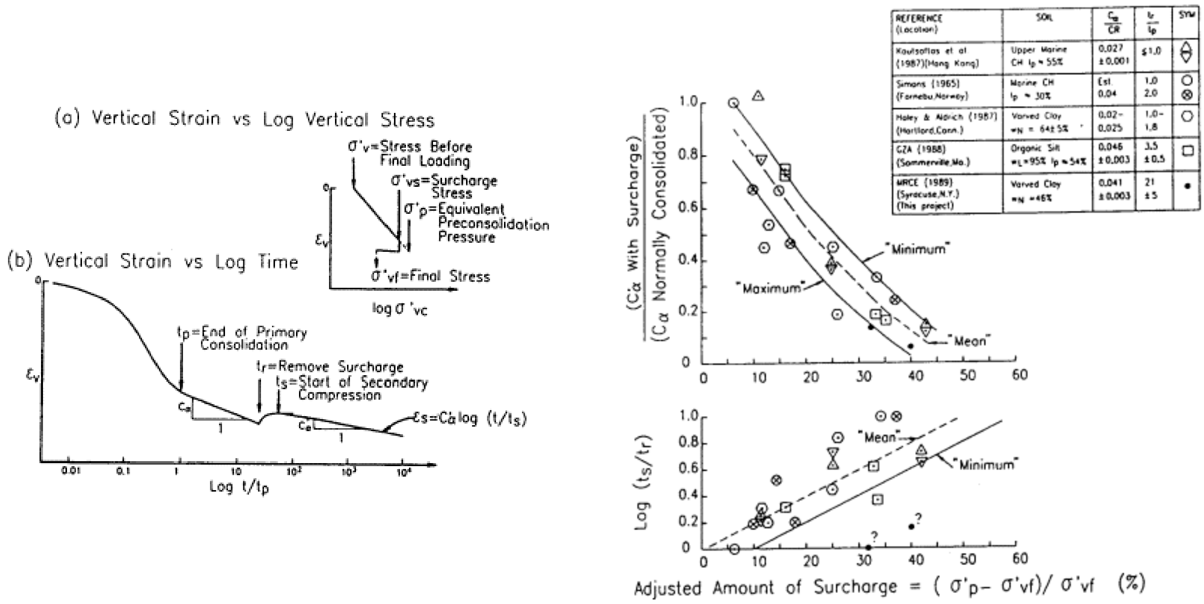
Hsi and Lee (2015) discuss various methods for simulation of radial consolidation accounting for smear effects, including, axisymmetric finite element analysis, an analytical solution published by Hansbo (1981), and the 2D plane strain finite element analysis based on geometric and/or permeability matching, as presented by Hird et al. (1992), Chai et al. (2001), and Indraratna et al. (2005). The parametric studies indicated the consolidation ratio based on settlement can be reasonably predicted by all methods, whilst the calculated rates of consolidation were more scattered based on excess pore water pressure dissipation.

6.5 CREEP SETTLEMENT

Creep settlement, sometimes referred to as secondary consolidation, can continue for decades after primary consolidation is complete. Creep settlement is a term used to describe the consolidation of soft soil under constant effective stress. The defining factors of creep settlement are the soils' stress history, i.e. OCR, and the passage of time, t, since the additional load was applied to the ground.

Consideration of creep settlement is vital to the assessment of post-construction settlement, given that total creep magnitudes increase with time, progressing well beyond construction timeframes and into the serviceable life of embankments, which may be 50 to 100 years. In practice, creep is considered to start after full primary consolidation has occurred. However, it is also reported by Bjerrum (1967) that there is a "delayed" compression in response to an increase in effective stress which implies that creep settlement takes place in conjunction with primary consolidation settlement.

To ensure long term settlement criteria can be met, creep settlements often require curtailing. An effective and commonly adopted solution is to over-consolidate the soil using surcharge loads. This has been documented to create a reduction in C_α in proportion to the increase in OCR, as shown in
Figure 11 (Ladd, 1989; Stewart, 1994).

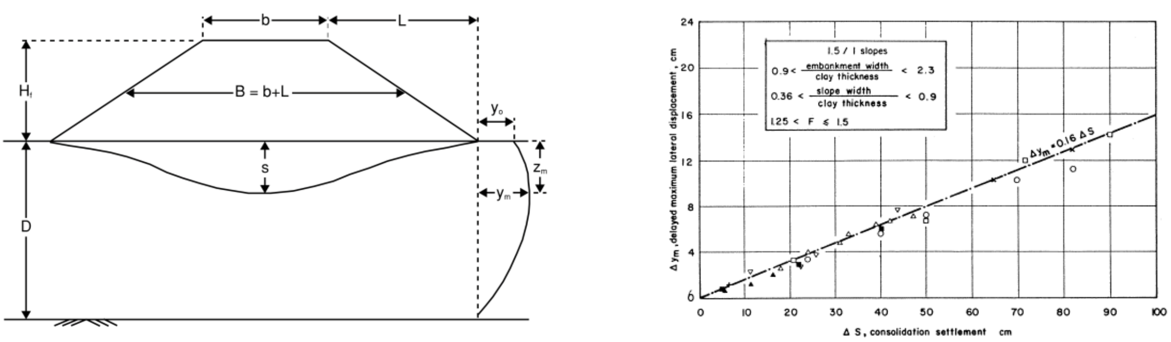


(a) Effect of surcharge (b) Creep reduction

Figure 11: Creep reduction as a result of surcharging (Ladd, 1989, Stewart, 1994)

From Figure 11 it can be seen that the magnitude of creep reduction and the commencement of creep after surcharge removal can be assessed based on the OCR (i.e. σ'_p/σ'_{vf}) achieved as a result of surcharging, where σ'_p is the pre-consolidation pressure and σ'_{vf} is the final stress. Note that this approach assumes creep commences after primary consolidation is complete.

Numerical modelling software, such as PLAXIS, are often used to simulate the behaviour of embankments on soft soil, due to the computational ability of such programs to model stress-dependent and time-dependent characteristics of saturated soil. One of the available features of PLAXIS is the Soft Soil Creep (SSC) model, which allows the calculation of ground settlement due to primary and secondary consolidation. However, the PLAXIS SSC model assumes that creep occurs concurrently with primary consolidation and the calculated settlements do not appear to be consistent with those derived from the conventional method using C_c, C_r, C_α and OCR. The author suggests that unless the SSC model is well understood and correct input parameters are adopted, use of this model should be handled with caution.



(a) Settlement induced lateral soil movement. (b) Lateral soil movement vs settlement

Figure 12: Correlation between lateral soil movement and embankment settlement (Tavenas, 1979)

6.6 LATERAL SOIL MOVEMENT

Settlement in soft soil is often accompanied with lateral extrusion of the foundation material, as illustrated in Figure 12. The ratio of lateral soil movement, δ, at the embankment toe, relative to the maximum vertical settlement, d, can provide insight into the potential for embankment instability/slope failure, as shown in Figure 13 (Matsuo and Kawamura, 1977). In this figure, P_j equates to the embankment load and P_f equates to the embankment failure load. Instability is defined when the combination of d and δ/d exceeds the P_j/P_f = 1.0 curve. For example, if embankment settlement, d, is recorded to be 1m, the embankment may experience instability if the lateral movement exceeds 0.7m, i.e. $\delta/d \geq 0.7$.

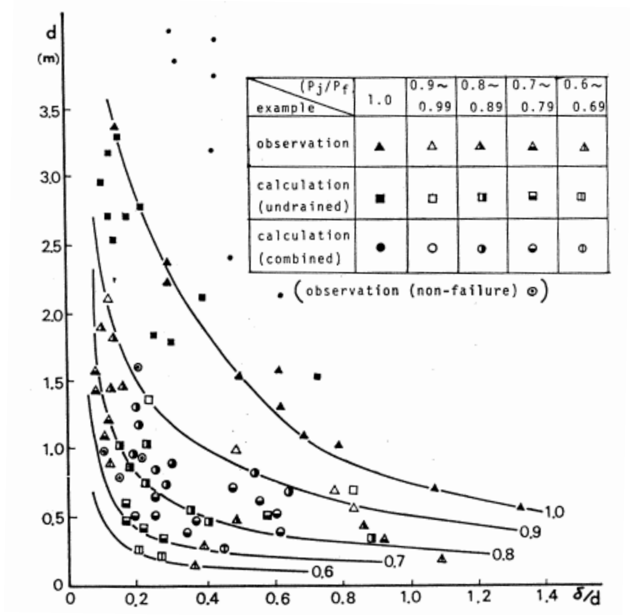
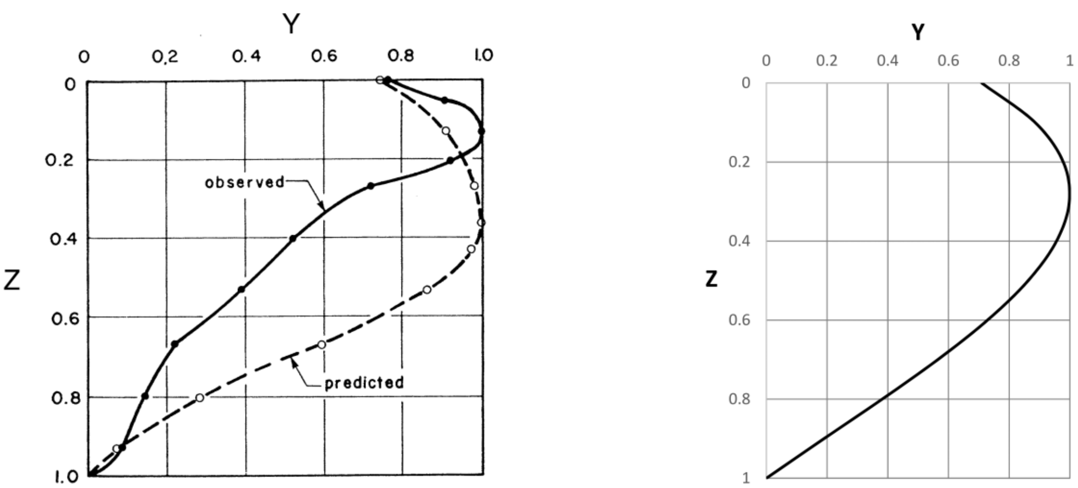


Figure 13: Ratio of lateral soil movement to vertical settlement and relation to slope instability. (Matsuo and Kawamura, 1977)

Prediction of lateral soil movement can also be carried out using finite element modelling (FEM) software, such as PLAXIS. However, analyses carried out with these methods often lead to over predicted movements (Poulos, 1972). Despite the ability to develop somewhat accurate vertical settlement predictions with FEM, the combination of heterogeneity and anisotropy of the soil, and the inability to accurately determine Poisson's ratio, lateral soil deflections are recognised as being difficult to predict (Poulos, 1972). Furthermore, FEM also fails to predict the distribution with depth of the lateral soil displacement, as shown in Figure 14a (Poulos, 1972).



(a) Predicted and observed distribution of normalised lateral soil displacement (Poulos, 1972).

(b) Normalised lateral soil movement distribution. (Bourges and Mieussens, 1979)

Figure 14: Lateral soil movement distribution with depth

Using results obtained from field observations, Tavenas (1979, 1980) reported that a linear relationship exists between the maximum lateral displacement Δy_m and the settlement ΔS after the end of construction, as shown in Figure 12b. This relationship can be expressed as per Equation 11.

$$\Delta y_m = (0.16 \pm 0.02)\ \Delta S \qquad \textbf{(11)}$$

Bourges and Mieussens (1979) have shown that the normalised deformation profile at the end of construction can be expressed as per Equation 12.

$$Y = 1.78Z^3 - 4.7Z^2 + 2.21Z + 0.71 \qquad \textbf{(12)}$$

The variables in Equation 12 can be defined as $Y = y/y_m$ and $Z = z/D$. D is the thickness of soil subjected to lateral soil movement.

The maximum lateral soil movement y_m can be calculated as 0.16 times the maximum settlement S based on Equation 11. Lateral soil movement y at depth z can then be calculated from Equation 12. With the above empirical correlations, the lateral soil deflection and distribution below the embankment toe can be realistically predicted, provided that the settlement of the embankment is known (see Figure 14b).

6.7 DELAYED PORE PRESSURE MEASUREMENT

Measuring the progress of consolidation during construction plays a critical role in the long term success of embankments founded on soft soil. Records of its actual performance allow engineers verify their design assumptions, ensuring its as-built behaviour matches their predictions. One of the most critical predictive decisions to be made in embankment construction is the deduction of when residual settlements are sufficiently small to allow the construction of settlement-sensitive structures above the embankment, such as the overlying pavement. Accurately recorded consolidation measurements provide engineers with critical information to predict embankment behaviour, and provide meaningful estimates of residual settlement.

In theory, measuring the progress of consolidation can be achieved in two ways

- Measuring the actual embankment settlement through the use of settlement plates, survey markers, etc.
- Measuring excess pore water pressures with piezometers, standpipes, etc.

However, inconsistent results between these measurement records are sometimes obtained between settlement plates and piezometers, resulting in confusion about the status of consolidation. It is recognised that piezometers sometimes provide a delayed response to pore pressure dissipation.

The recognition of time-lag in pore water pressure measurements with piezometers is well documented, e.g. Gibson (1963), Fell et al. (2005), and Schultheiss (1990). For accurate readings, equilibrium of pore water pressure needs to be achieved between the water pressure within the soil and that within the measuring device. However, such equilibrium may not be achieved instantaneously because of the following:

- Presence of air bubbles and gas within the measuring device, resulting in blockage of water flow
- Low permeability of soil resulting in slow water flow rate into the measuring device
- Compressibility of soil – volume change of soil causing change in pore water pressure, etc.

Table 6 shows that equalisation time lags vary substantially between sand and clay, and delays of up to hundreds of days may be observed in clays.

Due to this delayed response, physical measurements of settlement are often used as the primary indicator when assessing the progress of consolidation. Pore pressure measurements are only used as a guide.

It should be noted that when wick drains are used, the distance between the piezometer and the wick drains will affect the pore water pressure measurement. For example, fast dissipation of excess pore water pressure may be measured because the piezometer is installed close to a wick drain. The measurement may not reflect the average degree of consolidation of soil between wick drains. Use of piezometers in this capacity can be misleading and hence the results need to be treated with caution.

Table 6: 95% and 99% equalization time lag for Casagrande Piezometers (Standards Association Australia 1979)

Material	Sand	Silt			Clay		
Permeability cm/s	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Average time lag for 95% equalization	12 s	2 min	20 min	3.5 h	36 h	14 days	150 days
Average time lag for 99% equalization	18 s	3 min	30 min	5.2 h	54 h	21 days	225 days

6.8 LOAD TRANSFER PLATFORMS

When rigid inclusions are adopted as “below ground” treatments, such as columns or piles (refer Section 5), a load transfer platform (LTP) is required above the inclusions to distribute the overburden pressure evenly on to the columns / piles and the underlying soils. The absence of this intermediate layer can lead to a “mushrooming” effect, whereby surficial undulations can be formed due to differential settlement between the rigid inclusions and untreated soft soil.

Layers of high strength geotextile (laid in orthogonal directions) are often embedded within the LTP to assist with load redistribution (see Figure 15). Available design specifications, standards/guidelines, e.g. BS8006 (1995, 2010), can be used to determine the LTP requirements.

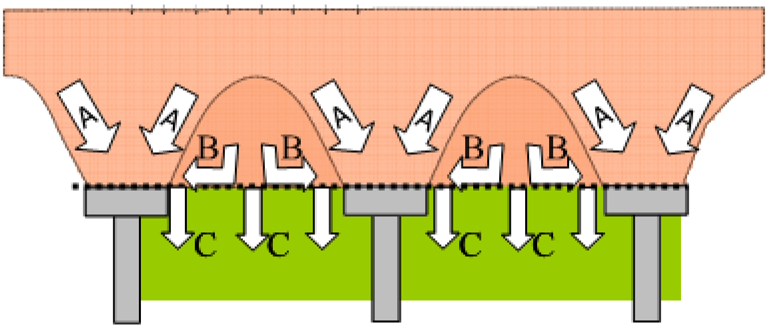


Figure 15: Load distribution through LTP (Van Eekelen and Bezuijen, 2012)

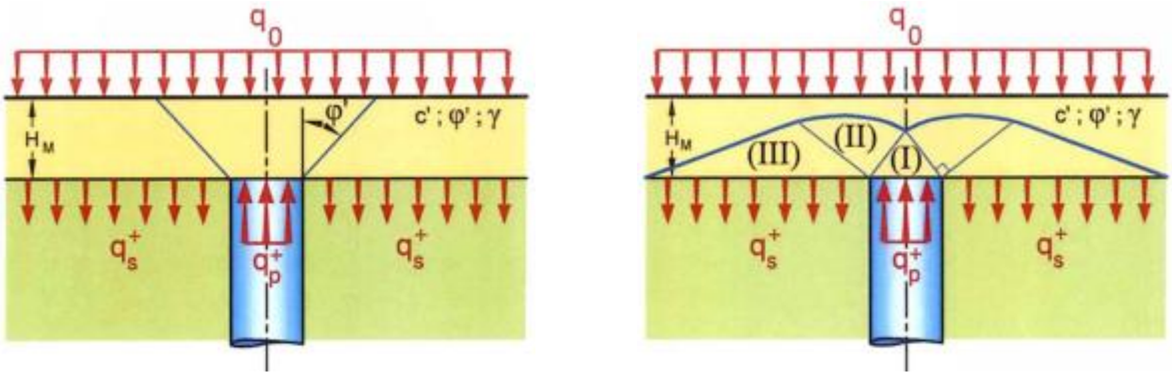
There have been many discussions about the requirement of HSG within LTP’s (Simon, 2012; BS8006, 1995 and 2010; Love and Milligan, 2003; Russel and Pierpoint, 1997; Ghosh et al. 2015) – with each of the design methods leading to different design outcomes. The main point of difference lies in the assumption of load sharing among the following components:

- Overburden fill
- Columns/piles (below the fill)
- HSG within the LTP
- Soils surrounding the columns/piles

As the interaction between each component is complex, numerical modelling is often adopted to simulate their behaviour. In author’s view, 3D numerical modelling would better simulate such complex interactions. Whilst simplified 2D or axisymmetric analysis is often used, caution should be exercised and the results calibrated against corresponding 3D simulations. It should be noted that the LTP (except the HSG) simulated in the numerical modelling should not possess any tensile capacity, i.e. zero tension cut-off, as it generally comprises granular materials. Failing to do so would lead to under-prediction of tensile stresses within the HSG.

BS8006 (1995, 2010) provides an LTP design methodology which assumes that the embedded HSG behaves as an elastic structural member between piles, essentially carrying the full overburden load, and assuming no support from the underlying ground. The overburden pressure is then transferred entirely to the rigid inclusions from the HSG. The dismissal of support provided by the soil is a conservative assumption and will lead to the over-prediction of the required HSG strength.

When the soil between piles/columns is considered to provide partial support to the overburden pressure, the required HSG strength can be reduced or completely removed depending on the fill height. ASIRI (2012) presented the various modes of failure, such as punching and bearing failure, which can occur between the pile and the load transfer platform., as shown in Figure 16. ASIRI (2012) also presented the assessment of minimum LTP thickness when HSG's are not used.

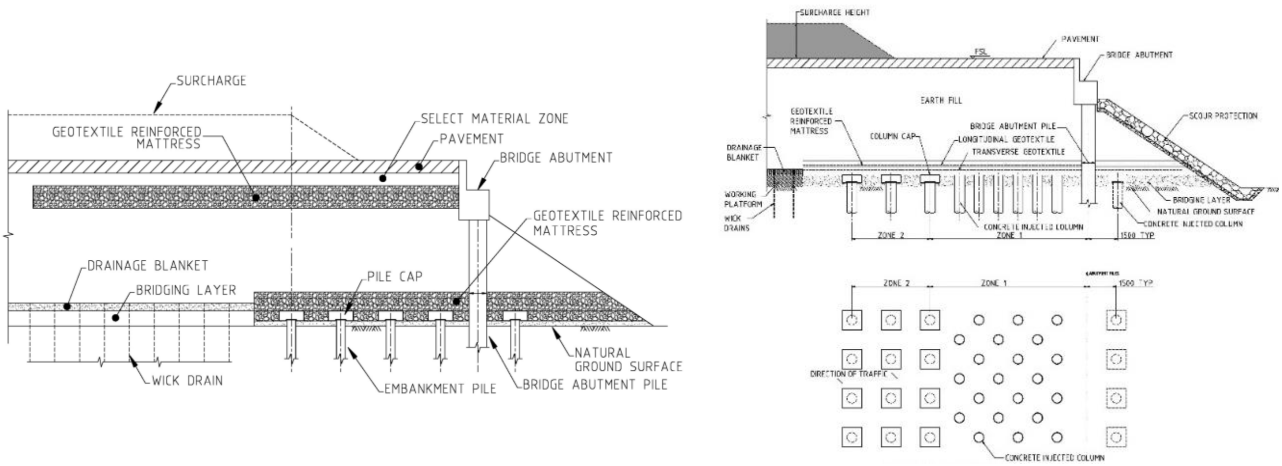


(a) Punching failure (b) Bearing failure

Figure 16: Different types of failure mechanism of LTP (ASIRI, 2012)

6.9 TRANSITION TREATMENT

Differential settlements are likely to occur between approach embankments constructed over soft soils and structures supported on stiff, rigid foundations, e.g. bridges or culverts. Such differential settlements may render loss of serviceability of the road, damage to the pavement and risks to the safety of road users. A successful mitigating solution involves implementation of transition treatment along approaches to bridges or culverts, such that differential settlement can be controlled and reduced.



(a) Timber piles (Hsi and Martin, 2005) (b) CIC's (Hsi et al., 2008)

Figure 17: Typical arrangements of bridge approach transition treatments

There are many different ways to arrange transition treatments depending on the ground condition, height of embankment and construction program. Some treatment measures that have been used recently include timber piles, precast concrete piles, CFA piles, concrete injected columns (CIC), preloading/surcharging with wick drains and vacuum consolidation (Hsi, 2001; Hsi and Martin, 2005, Hsi et al., 2008, Hsi and Lee, 2010; Hsi et al., 2013). Typical arrangements that have been used successfully by the author include (see Figure 17):

- A forest of piles/columns (with an LTP) installed immediately adjacent to the bridge abutments which would eliminate most of the settlement. The piles/columns may be arranged differently away from the abutment to allow slightly increased settlement (see Figure 17b)
- Heavy surcharge of the area (with wick drains) adjacent to the pile / column supported area to accelerate consolidation and over-consolidate the soft soil, hence minimising creep settlement (see Figure 17a and b).

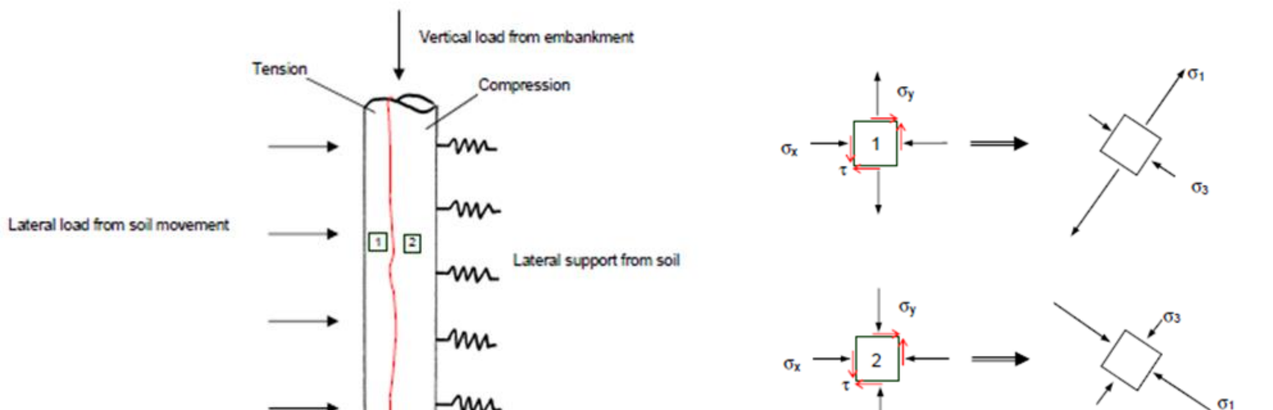
- A geotextile reinforced rock mattress (GRRM) can be lain below the pavement after completion of preloading/surcharging across the pile/column and preload/surcharge treated areas. The GRRM will eliminate any sudden differential settlement between different treatment zones (see Figure 17a).

The piles of bridge abutments built next to settling ground will be subjected to vertical and lateral soil movement (Hsi and So, 1999). As such, an important consideration when designing transition treatments is to prevent soil movement from negatively impacting the bridge abutment piles. Solutions such as the forest of piles adjacent to the abutment, refer Figure 17b, will provide a "buffer zone" between the settling ground in the preload/surcharge area and the bridge abutment piles. Buffer zones prevent the bridge abutment piles being impacted by either ground settlement or lateral soil movement (Hsi et al. 2008). With this arrangement, the bridge abutment and the bridge can be built at the early stages of the project whilst preloading/surcharging takes place.

In situations where adequate time is allowed for partial preloading/surcharging, "pre-treatment" methods can be used to remove part of the ground settlement. As a result, reduced transition treatment is achieved. Hsi et al. (2013) presented a case study using CFA columns for transition treatment after the ground had been partially preloaded and surcharged.

The use of plain concrete columns to support embankments in transition zones is growing in popularity, with options such as concrete injected columns and CFA plain concrete columns. When placed beneath embankment batters or adjacent to settling ground, these columns will be subject to lateral loads, inducing bending and shear forces within the columns. However, due to the absence of steel reinforcement, these columns have limited bending capacity, and careful assessment of their structural integrity will be required. The author suggests undertaking the following checks on the column integrity:

- Use of numerical modelling software to simulate the transition treatment with plain concrete columns
- Concrete columns simulated as Mohr-Coulomb material and assumed to have low tension cut-off, i.e. no tensile stresses can develop within the column beyond the specified tensile capacity
- Limitation of the maximum principle compressive stress within the columns to the unconfined compressive strength, UCS, of the concrete column (see Figure 18)



(a) Stress zones (b) Principal stress

Figure 18: Principal stresses within plain concrete column (tension in Zone 1 and compression in Zone 2)

7 RISK MANAGEMENT

Due to the variability and uncertainty of soft soil behaviour, it is prudent for practitioners to validate design predictions during construction against the actual performance. Vigilant record-keeping and monitoring will ensure remedial action can be undertaken in a timely manner, where necessary.

The validation process can be realised via an "Observational Approach" (Hsi and Lai, 2015), which includes the following steps:

- Instrumentation and monitoring, to record the actual behaviour of soft soil during construction (e.g. Hsi and MacGregor, 1999; Hsi, 2003b)

- Back analysis of the monitoring data
- Calibration of design assumptions and geotechnical models by matching predictions with measurements
- Prediction of future soft soil performance, using the calibrated models and assumptions
- Modification of design and construction, if required, to meet the design criteria

An example of the back analysis where predictions match actual measurements is shown in Figure 19 (Hsi and Martin, 2005).

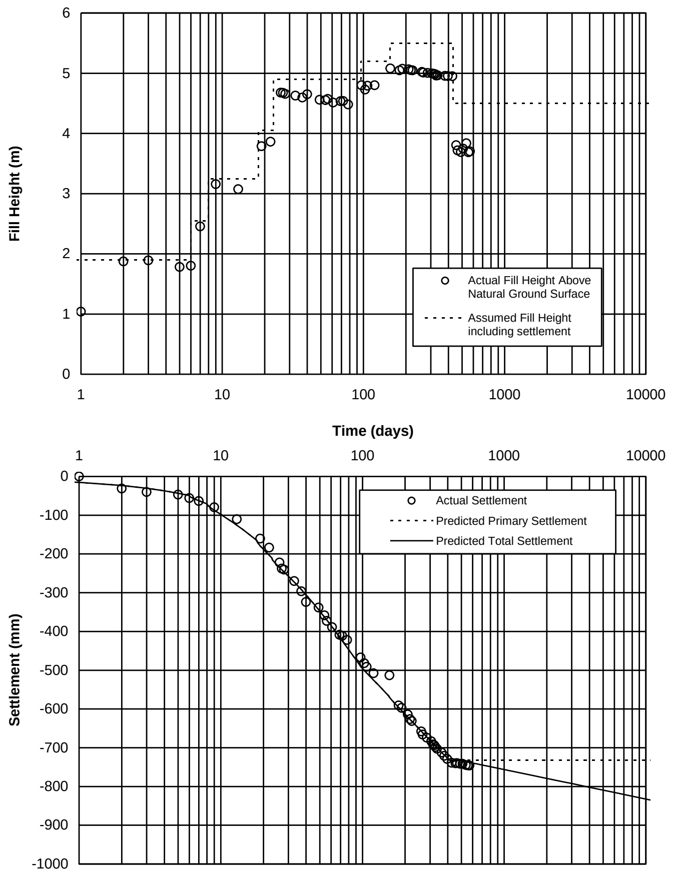


Figure 19: Back analysis of embankment filling and associated settlement

A risk management system was developed by Hsi (2003a and 2003b) and Hsi and Martin (2005) for a section of the Pacific Highway Upgrade project in NSW, as shown in Figure 20. Risk mitigation strategies including geotechnical investigation prior to design, and site validation using the Observational Approach, were implemented to manage and minimise soft soil risks. Implementation of these systems are critical to the success of soft soil projects.

8 CONCLUSIONS

Soft soil engineering is a discipline shrouded in risk and uncertainty. The highly compressible and variable nature of this material presents unique problems and difficulties, that can only be recognised through experience, trial and error, and lessons learnt.

The importance of ground interpretation cannot be over-emphasised. However, due to budget and time constraints in collecting field information, the available test data often cannot account for the enormous variabilities in soft soil. Practitioners must be aware of the limitations of the available geotechnical information and techniques, and make due allowance for possible conditions that may exist in reality, such as varying vertical and horizontal permeability, and the inability of oedometer tests to capture scaling effects.

Client-prescribed design criteria underlie the basis behind most soft soil designs. Practitioners must take time to fully comprehend the conditions which these criteria represent and the purpose of these limitations. For example, the background of the differential settlement criterion defined as Change in Grade and how it is measured.

There are a vast number of treatment solutions available for soft soil. A design process has been presented which allows a streamlined approach to achieve cost-effective designs which encompass both settlement and stability considerations. The chosen solution may comprise a suite of treatment options to fully meet the design criteria, such as plain concrete columns in conjunction with heavy surcharging and wick drains at the bridge approaches to meet differential settlement criteria.

Instrumentation and monitoring are an integral part of the design and construction in a soft soil environment which is dominated by variable and uncertain conditions. The use of the Observational Approach has allowed added flexibility during construction to address unexpected behaviour and implementation of mitigation measures to rectify the as-built condition. Through this approach, the long term project performance can be improved.

Despite working with soft soils for a large part of my career, understanding and appreciating their behaviour remains elusive and often surprising at times. It is this behaviour that suggests engineering of these materials is not an absolute science, but to a certain extent, an art form based on a combination of theoretical soil mechanics and engineering judgement. Despite having drawn my own conclusions from past experience documented herein, this is by no means a comprehensive list and I would welcome an open discussion of experiences and methods to the contrary.

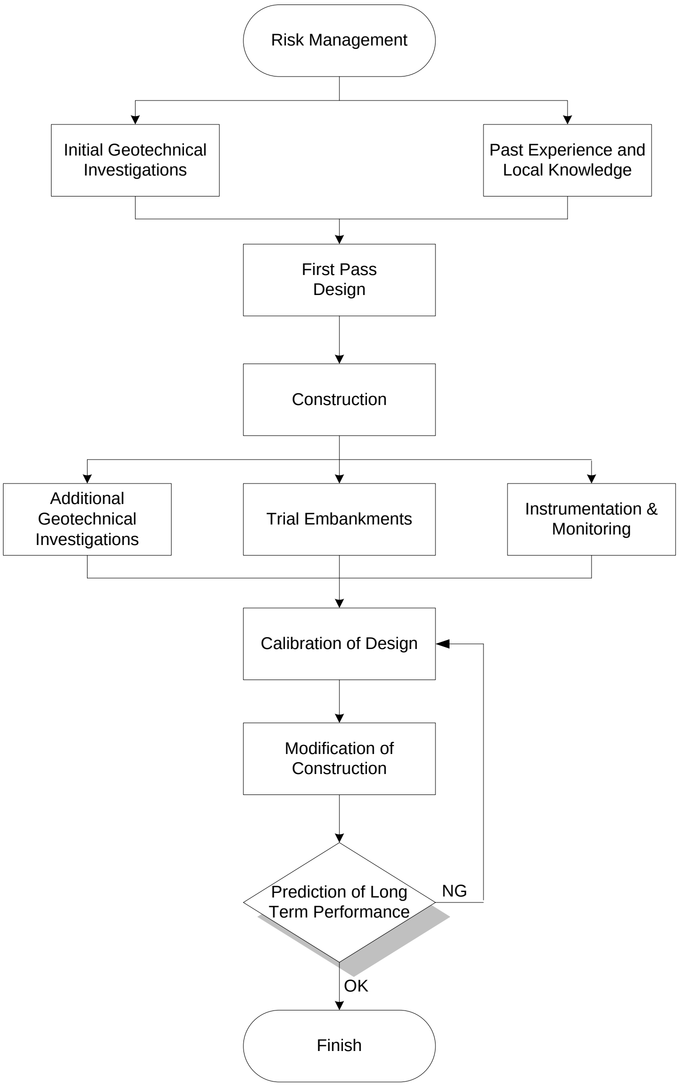


Figure 20: Risk management system

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