Settlements of Embankments in Soft Soils

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ABSTRACT: The major part of this paper is on the challenges in predicting settlements in highway embankments and reclamation works in marine, deltaic and estuarine type of deposits. The emphasis is on practical aspects and the difficulties experienced in confidently estimating settlements even after a century of developments and contributions. The influence of the general geology and soil conditions is discussed in relation to the site investigation works and the establishment of soil profile models. The fundamentals of preloading techniques with and without prefabricated drains (PVD) as ground improvement measures are also included. The observational approach in evaluating PVD performance and settlement estimations is then made with emphasis on curtailing residual settlements. Finally, the latter part of the paper is devoted to the analytical and numerical solutions of the behaviour of piled supported approach embankments with transfer layers. Geogrid reinforced pile supported (GRPS) embankment system is studied with the analytical approaches of Terzaghi, BS8006 method and Hewlett and Randolph method. Additionally, numerical analyses are also made with the Plaxis software.

1. INTRODUCTION

The major part of the work presented in this paper relates to the challenges in predicting settlements in marine, deltaic and estuarine type of soft soils under road and highway embankments and during reclamation works. The early work on Soft Bangkok Clay is described by Muktabhant *et al.* (1967), Moh *et al.* (1969) and Eide (1968 and 1977). A comparison is then made with the practice in SE Asia and in Australia wherever possible. Concentration is made on the use of preloading with and without prefabricated vertical drains (PVD) and the restriction of post-construction settlements. Most of the material presented here is related to actual project works where the first author is involved over a period of forty years. In these activities classical and simple theories in Soil Mechanics are used and lesser emphasis is given to the recent excellent developments of more refined approaches. The primary and secondary settlements are calculated using the notable and well established works of Terzaghi, Mesri and Hansbo. The first comprehensive volume on soft clay engineering was by Brand and Brenner (1981) at AIT as based on the International Symposium on Soft clays held in Bangkok in 1977. The ground improvement conference held in 1983 at AIT also gave great impetus in the development of this field in SE Asia (see Balasubramaniam *et al*., 1984). Bergado *et al*. (1996) had an excellent volume on soft ground improvement. The recent volume on case histories by Indraratna and Chu (2005) has excellent contributions by Hansbo, Moh and Lin, Indratana, Hsi, Wong, Chu, Massarsch, Terashi, Kitazume, and others, which are very valuable for practicing geotechnical engineers.

The First Author’s experience generally lies in an observational approach and in interpreting laboratory test data and small scale and large scale field tests. The extensive use of CPT and CPTu have somewhat reduced the earlier emphasis made on the use of large number of boreholes to delineate the soft soil layer thicknesses. Also, it is the practice in SE Asia to conduct large number of index tests and water content determination and to use such data indirectly in estimating the soft clay layer thicknesses. While quality of undisturbed samples for laboratory tests and sophisticated triaxial stress path and other type of laboratory tests have been extensively researched, the current practice seems to avoid these sampling and testing procedures with a view to minimise the expenses on site investigation and design phases of the projects. However large scale field tests with the use of test embankments with and without PVD are still used heavily in practice. Here again perhaps the instrumentation used for the measurement of surface and sub-surface settlements, pore pressures and lateral movements are somewhat minimised to cope with the limited funds made available in site investigation works.

Differential settlement is also a major issue in approach embankments adjacent to bridge abutments, culverts and other structures founded on piles. Such settlements will affect the performance of the pavement and can cause negative skin friction which imposes additional loads on the piles. Stringent settlement criteria are now imposed on many major highway and expressway projects and this then led to the latter part of the paper in studying the available closed form solutions and numerical methods on the design of pile supported approach embankments with transfer layers. Geogrid reinforced pile supported (GRPS) embankment system is studied with the analytical approaches of Terzaghi, BS8006 method and Hewllett and Randolph method. Numerical analyses are also made with the Plaxis software.

2. GENERAL GEOLOGY OF MARINE, DELTAIC AND ESTUARINE CLAYS

The Soft Bangkok Clay (see Muktabant et al., 1967, Moh et al, 1969, Bergado et al., 1990, Moh and Lin, 2005, Seah, 2005) and the Muar Flat clays (Chan and Chin 1972, Ting and Ooi, 1977, Ting et al., 1987, Ting et al., 1989, Poulos et al., 1989, Nakase and Takemura, 1989; and Brand and Premchitt, 1989; Indraratna et al., 1992; and Loganathan et al., 1993) in Malaysia are marine clays and are very homogeneous and extend to great depths over a very large area. These deposits are studied extensively. The whole of Bangkok Plain has a carpet of soft clay spaning some 200 to 300km east west and some 400km north-south. Also, the thickness is relatively high.

The estuarine clays in Queensland differ markedly from the marine deposits in.

(i) They usually occur close to creeks and vary substantially in thicknesses and composition.
(ii) The strength is very low.

Notable highway projects in Thailand such as the Thon Buri Pak Tho Highway, the Bangna-Bangpakong Highway, Bangkok-Siracha Highway, Nakon Sawan Highway and others in the Bangkok Plain are such that the whole stretch of highway passes over the flat deltaic plain where the subsoil is soft marine clay. The age of this marine deposit is about 2000 years and it is considered as a recent deposit. The thickness of the soft clay in the Bangna-Bangpakong project varies from 15m at Bangna to 25m at km 28 from Bangpakong. This layer is underlain by stiff clay of 4 to 10m thick, followed by a dense to very dense sand. A weathered crust of varying thickness 1 to 3m forms the topmost layer. A longitudinal section of the Nakon Sawan Highway as presented by Eide (1977) is shown in Fig. 1.

The Queensland Department of Main Roads in Herston, under the leadership of Vasantha Wijekulasooriya has accumulated valuable wealth of Geotechnical information on Highway and Motorway construction. Similarly, the Port of Brisbane (POB) has done award winning work on reclamation works (see Ameratunga, 2010). Excellent expertises on the engineering geological aspects of soft soils in SE Queensland are there in well established geotechnical companies such as Coffey Geotechnics among others.

References can be made to the important pioneering works of Whitaker and Green (1980), Robertson (1984), Litwinowicz and Smith (1988) and Wijekulasooriya *et al*. (1999). The coastal plain in Australia has a low elevation and over the last 2 million years the coastline has changed as a result of sea level changes. The geology of the coastal area is substantially influenced by climate, water, tectonic and geological activity and vegetation. Along the alignments of highways and motorways in SE Queensland, the soft soils include a combination of alluvial, coastal and estuarine sediments; with sands, silts, and clays. The soil stratigraphy in some areas is very variable within very short stretches. Under these circumstances the evaluation of the thickness of the soft soil and its compressibility and drainage characteristics is a major challenge. Some of the routes comprised of extensive areas of Quaternary alluvial material forming tidal mangrove and mud flats. Also, embankments are situated on tidal flat of estuarine sediments with sandy beach ridges of Holocene age, which is underlain by older Pleistocene sediments. Basically two types of alluvium have been encountered in Brisbane area. Young alluvium consisting mainly of dark grey, soft to firm organic silty clay (OH); old alluvium consists of a series of layers including silty clay, sandy silt, silty sand, sand and gravel. Young alluvium is a very recent deposit and comprises mainly of soft silty clay (undrained strength 5 to 15kN/m^2). This in turn overlies a young alluvial deposit of soft to firm silty clay and older alluvium consisting of stiff to very stiff silty clay/sandy silty clay. This layer is assumed to have no effect on embankment stability or settlement.

The soft soils in Southeast Queensland and in New South Wales bordering with Queensland are of very low strength and high compressibility (see Wijekulasooriya *et al*., 1999; Hsi and Martin, 2005), thus there are potential risks with slope failures during the construction period. Additionally, there can be high settlements during the construction and service periods of the road. This will cause an increase in fill quantity during construction and problems of serviceability of the road under long term conditions due to residual (and differential) settlement. The low drainage characteristics of the soft soil will delay the consolidation process resulting in longer construction time.

There can be acid sulphate soils (ASS) as well and potential acid sulphate soils (PASS) present along the route. In such an environment, care should be taken to avoid the effects due to the formation of sulphuric acid and its impact during flood inundation periods, and potential degradation of structural elements, such as culverts, foundation piles, footings etc (Hsi and Martin, 2005, Ameratunga, 2009).

3. SOIL PROFILE MODEL

Before the introduction of CPT and CPTu, soil profiles in soft soils are entirely relied upon from borehole data and in-situ vane tests. Also, natural water content, liquid limit and plastic limit tests are carried out and these data are valuable in separating the soft clay from medium stiff and stiff clay. In the classical work at AIT and NGI (see Moh *et al*., 1969; Eide, 1977) the soft Bangkok clay is described as so homogeneous (at the Bangkok airport site and along the Nakhon Sawan Highway and the Bangkok Siracha Highway), they felt the undrained shear strength contours as established from vane shear tests will not vary more than 10 percent of the values. The soil profile as established from boreholes and the vane strength profiles are given in Figures 1 and 2. Also, profiles of water content are plotted along the longitudinal section of the routes (see Figure 3). Over the years, there seems to be a drastic reduction in the basic laboratory tests such as the Index tests and natural water content determination. These tests are most valuable when the quality of undisturbed soft clay samples is questionable when used in Oedometer tests to determine the compressibility characteristics. Recent work on Bangkok Clays by Sambhandharaksa (2006), Seah and Juirnarongrit (2003), Seah and Lai (2003), Seah and Koslanant (2003), Seah *et al*., (2004a and 2004b) has been based on good quality samples and refined testing methods.

CPT and CPTu tests are now used extensively in estuarine clays. The data are used in soil profiling as well. The undrained strength of the soft clay is established from the measured cone resistance and overburden pressure, together with the use of a cone factor N_k in the range of 15 to 20. In SE Queensland, the soft clay thicknesses as established from CPT and CPTu are found to be successful in line with those established from borehole profiles and in-situ vane tests. In some instances the strength derived from CPT are found to be lesser than those obtained from vane tests conducted in tube samples. Recently, T-bar tests were found to give more reliable strength than the CPT. But these tests are only carried out in limited projects.

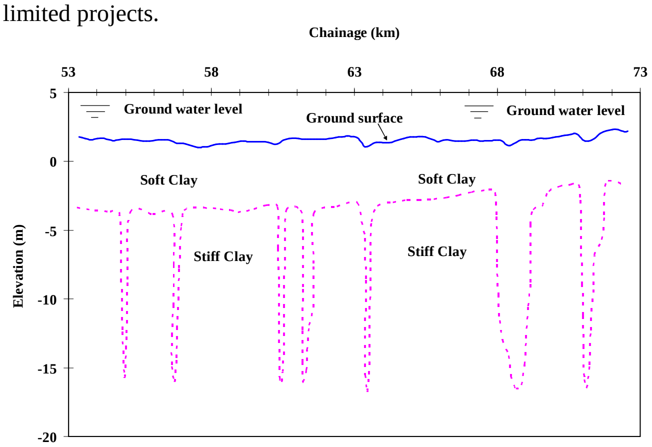


Figure 1. Longitudinal section of the Nakon Sawan Highway (Eide, 1977)

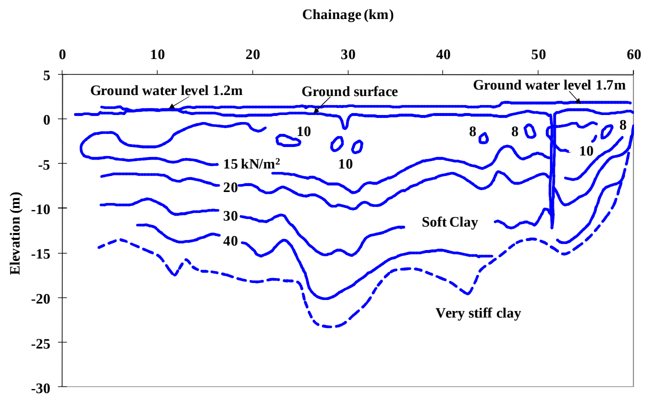


Figure 2. Vane strength profiles (Eide, 1977)

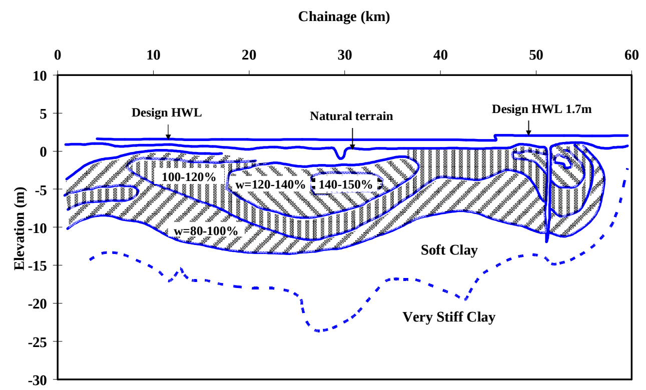
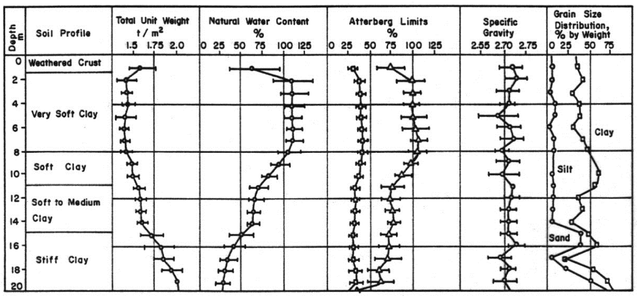


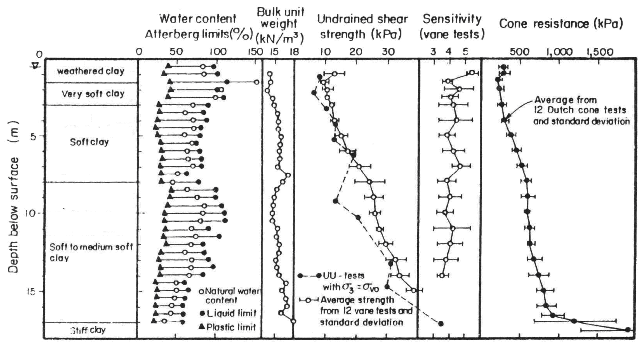
Figure 3. Water content profile (Eide, 1977)

Figures 4(a) and (b) indicate the use of water content and index tests in delineating the stiffnesses of clays as soft, medium soft and stiff.

In marine and deltaic deposits of Bangkok and Muar Clays etc., the soil model of the subsurface is generally of weathered crust and soft clay. The thicknesses of these layers are the same over very large areas with little variation. However, when estuarine clays are experienced, there can be more than one or two layers interbedded with layers of sand and firm clay etc. In reclamation works, some time the number of layers encountered can even range up to six or more. Also within very short distances, the layer thicknesses can change substantially. The CPTu tests are found more reliable in identifying the various layers encountered and their types. Figure 5 shows the soil profile as established from CPTu and Boreholes.



(a) Bangkok Swarnabhumi Airport site
(Moh et al., 1969, Moh and Woo, 1987)



(b) RTN Dockyard site in Bangkok Plain
(Balasubramaniam et al., 1980)

Figure 4. Profiles of water content and index tests

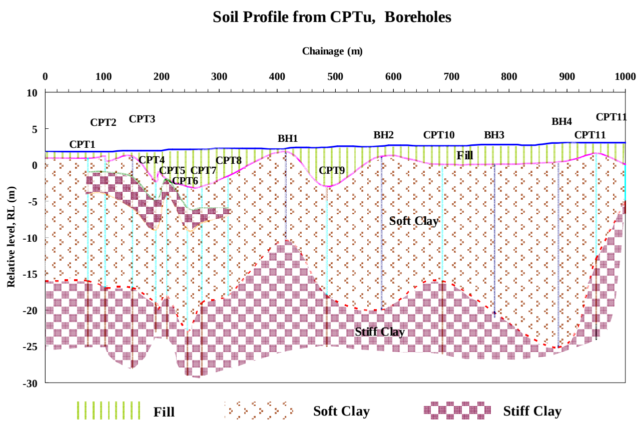


Figure 5. Soil profile as established from CPTu and Boreholes

4. PRE-LOADING TECHNIQUE IN GROUND IMPROVEMENT

Pre-loading is a common method used to improve soft clayey soil deposits. The effective surcharge pressure for preloading can arise from either the weight of the imposed fill material (example an embankment) and or the application of a vacuum pressure applied to a soil. Excellent contributions on the use preloading with PVD have been made by Hansbo (1960, 1979, 1981 and 1987); Holtz *et al*. (1988); Mesri (1991, 1994); Balasubramaniam *et al.* (1995); Bergado *et al*., (1991, 1998, 1999 and 2002); Sambhandharaksa *et al*. (1987); Moh and Lin, (2005); Choa *et al*. (1979a and 1979b); Lee *et al*. (1985); Tan *et al*. (1987); Yong and Lee (1997); Chu *et al.* (2004); Bo and Choa (2004); Arulrajah *et al*. (2007); Chu *et al.* (2009a and 2009b); Ooi and Yee (1997); Yee (2000); Masse *et al*. (2002); Varaksin and Yee (2007); Indraratna *et al*. (2005a and 2005b); Balasubramaniam *et al*., (2004); Oh *et al.* (2004); Long *et al*. (2006) among others in many countries.

4.1 Primary Consolidation

The magnitude of primary consolidation settlement under embankment loading is calculated now for many decades, using classical theory of one-dimensional consolidation and the strain-based recompression ratio (*RR*) and the compression ratio (*CR*). The existing vertical stresses and the anticipated increase in stress under embankment loads were calculated in a classical manner from layer thicknesses, position of ground water table and unit weights as well as simple expressions on stress distribution based on elastic soil behaviour.

4.2 Secondary Consolidation

Classical and scholarly contributions on primary and secondary consolidation are made by Mesri and Castro (1987), Mesri (2001), and Mesri and Vardhanabhuti (2005) among other authors. Secondary compression is the slow compression of soil that occurs under constant effective stress after the excess pore pressures in the soil dissipated.

The magnitude of secondary compression is a direct relation both to the soil's susceptibility to secondary compression as measured by the secondary compression index ($C_{\alpha\varepsilon}$) and by the time ratio (ratio of total time from load application to the time required to complete primary consolidation). By shortening the time required to complete primary consolidation (t_p) with the use of PVD, the ratio of total time (design life) to time for primary consolidation increases and would by itself cause the amount of secondary compression to increase.

Data gathered from the field and laboratory test program in SE Queensland showed considerable scatter in $C_{\alpha\varepsilon}$ with elevation. In general $C_{\alpha\varepsilon}$ ranged from 0.5 to 0.25%, and the ratio of $C_{\alpha\varepsilon}$ to *CR* varied from 0.015 to 0.07. $C_{\alpha\varepsilon}$ generally increased with Liquid limit and natural water content. One of the factors reported for the scatter of $C_{\alpha\varepsilon}$ was that the time interval was short as adopted in practice to determine this parameter from laboratory consolidation tests.

4.3 Reducing Secondary Consolidation Effects by Surcharging

When a clay soil is subjected to increased stress under new loading and also receives a surcharge loading, there will be some amount of rebound when the surcharge is removed. At some time after the rebound occurred, the clay will experience creep compression under constant effective stress, but the rate of secondary compression is slower than that would have occurred without surcharge. Thus when the secondary compression resumes, the secondary compression ratio $C_{\alpha\varepsilon}$ will be less than $C_{\alpha\varepsilon}$ of the clay before the surcharge was removed, and less than that would be present for, if no surcharge had been applied. It demonstrates that the longer the surcharge can be left in place, the greater the effect of the surcharge in delaying the time to onset of post-surcharge secondary compression until a time after the rebound has occurred, when the surcharge has been left in place for some time after its t_{100}. Therefore, two factors must be determined in applying the surcharge to reduce later secondary compression. The magnitude of reduction in $C_{\alpha\varepsilon}$ must be estimated and the time of delay in the on set of secondary compression.

Figure 6 illustrates the e-log p' curve and the time effects. In this figure:

(a) Point A represents the initial in-situ stress conditions where voids ratio is e_0.
(b) Point B represents the final stress condition after applying the surcharge.

(c) Point C corresponds to field settlement matched point on AB, prior to the removal of excess surcharge.
(d) Point M corresponds to the swelling line when the service load and the stresses due to the ground water lowering are incorporated (M is mostly in an over-consolidated state).

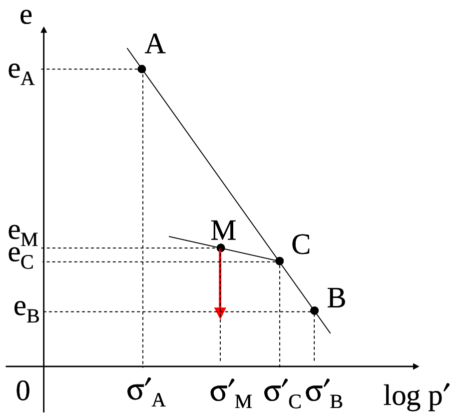


Figure 6. e-log p' relationships

Residual settlement is calculated as the difference of the creep settlement for 20 year period from the state M and the swelling due to stress release CM.

In this idealisation, if normally consolidated $C_{\alpha\varepsilon}$ is used, it is very conservative as $C_{\alpha\varepsilon}$ is very much dependent even on low values of OCR.

5. SETTLEMENT CALCULATION WITH AND WITHOUT PVD

5.1 Ultimate Primary Settlement without Using PVD

In natural deposits of lightly overconsolidated soft clays, the primary settlements are calculated using the traditional expressions:

Table 1. Expressions for primary consolidation settlement with stress history

Stress history	Primary consolidation settlement
$\sigma'_i + \Delta\sigma < \sigma'_p$	$\rho = \frac{C_r}{1+e_0} H \log \frac{\sigma'_i + \Delta\sigma}{\sigma'_i}$ (1)
$\sigma'_i < \sigma'_p < \sigma'_i + \Delta\sigma$	$\rho = (\frac{C_r}{1+e_0} \log \frac{\sigma'_p}{\sigma'_i} + \frac{C_c}{1+e_0} \log \frac{\sigma'_i + \Delta\sigma}{\sigma'_p})H$ (2)
$\sigma'_p < \sigma'_i$	$\rho = \frac{C_c}{1+e_0} H \log \frac{\sigma'_i + \Delta\sigma}{\sigma'_i}$ (3)

The major issue is then in determining the compression ratio *CR* ($= C_c/(1+e_0)$) and the recompression ratio *RR* ($= C_r/(1+e_0)$). These values are determined from laboratory consolidation and swelling tests, but the values are found to have very large scatter in estuarine clays.

5.2 Calculation of Settlement after Time *t* Prior to PVD Installation

The settlement at any time t is calculated using the expression;

$$\rho_t = \rho_\infty U \tag{4}$$

The degree of consolidation varies with the time factor T_v for one dimensional consolidation with vertical drainage as:

$$U = \left(\frac{1}{1 + \frac{1}{2T_v^3}} \right)^{\frac{1}{6}} \tag{5}$$

$$T_v = \frac{c_v t}{h_e^2} \tag{6}$$

5.3 Calculation of Settlement at Time *t* after PVD Installation

The currently adopted common expressions in ground improvement works with PVD are well established and generally the work of Hansbo (1981) is cited in using these expressions.

$$\rho_t = \rho U \tag{7}$$

$$U = 1 - (1 - U_h)(1 - U_v) \tag{8}$$

$$U_h = 1 - e^{-\frac{8T_h}{F}} \tag{9}$$

$$T_h = \frac{c_h t}{D_e^2} \tag{10}$$

The value of *F* is given by:

$$F = F(n) + F(s) + F(r) \tag{11}$$

$$F(n) = \frac{n^2 \ln n}{n^2 - 1} - \frac{3n^2 - 1}{4n^2} \tag{12}$$

$$F(s) = ((k_h/k_s) - 1)\ln(d_s/d_w) \tag{13}$$

$$d_s = 2d_m \tag{14}$$

$$d_m = \sqrt{\frac{4}{\pi} wl} \tag{15}$$

$$F_r = \pi z(L - z)(k_h/q_w) \tag{16}$$

5.4 Calculation of Residual Settlement (*RS*)

The *RS* values are calculated by two methods - (1) The *RS* values depend on the effective stress before stress removal with the appropriate *DOC* and the final stress level; the *RS* values are generally high as calculated by this method; (2) This method uses the $C_{\alpha\varepsilon}$ value in the over-consolidated range and it is noted that $C_{\alpha\varepsilon}$ in the over-consolidated range reduce sharply even with small values of *OCR*. The *RS* values depend on the structures built; typical values can be 150mm settlement under 15kPa service load in 20 years; these values can increase to 250mm settlement under 25kPa service load in 20 years.

$$\rho_{sec} = \frac{C_\alpha}{1+e_0} \log(\frac{t + t_p}{t_p}) \tag{17}$$

Table 2. Residual settlement calculation by two methods

Method I	Method II
$t_p = 10^{\left(\frac{\frac{C_c}{1+e_0} \log \frac{\sigma'_C}{\sigma'_M}}{\frac{C_\alpha}{1+e_0}} \right)}$ (18)	$t_p = 10^{\left(\frac{\frac{C_c}{1+e_0} \log \frac{\sigma'_C}{\sigma'_M}}{\frac{C_\alpha}{1+e_0}} \right)}$ (19)
	$\frac{C_{\alpha\varepsilon(oc)}}{C_{\alpha\varepsilon(nc)}} = \frac{1-m}{e^{(OCR-1)n}} + m$ (20)
	(*m* = 0.05, *n* = 6) (Wong, 2008)

$$\rho_{\sec} = \frac{C_\alpha}{1+e_0}\log(\frac{t+t_p}{t_p}) \qquad (21)$$

$$\rho_{\sec} = \frac{C_{\alpha(oc)}}{1+e_0}\log(\frac{t+t_p}{t_p}) \qquad (22)$$

6. PRELOADING WITH AND WITHOUT PVD

Two approaches are adopted in highway and motorway construction for preloading with and without the use of PVD. Some time it is wrongly conceived that settlement can be specified as the criterion for the removal of surcharge after pre-loading. It is emphasised that the *DOC* values must be generally higher than 90 pc and even as much as 95 pc or so prior to the surcharge removal. Otherwise substantial left over primary settlement can add to the secondary settlement and makes the post-construction settlement much higher. Before proceeding with the estimate of the post construction settlement (*PCS*), the need for the preloading time as based on *DOC* is important. These cases are illustrated below.

6.1 Preloading without PVD

Figure 7 illustrates the total stress due to the application of embankment and surcharge loading, the undissipated pore pressure at the time of removal of the preload. In this explanation:

(a) Double end drainage, of a clay layer of thickness H_c is considered;
(b) The horizontal axis refers to the stresses;
(c) The vertical axis AD refers to the layer thickness;
(d) AB = DC is the total stress from embankment load and additional surcharge during preloading: H_e is embankment height; h is surcharge height during preloading; γ unit weight of embankment material;
(e) Curve AGKD represents the excess pore pressure before removing preload h;
(f) At that time when the preload h is removed, the effective stress increase is represented by the hatched area AEBCFD;
(g) The settlement at this time is due to the increase in effective stress as represented by the area AEBCFD;
(h) However it must be ensured that the effective stress increase as represented by EB and FC should be higher than the value $H_e\,\gamma$.

Such a criterion seems logical to be adopted in practice.

6.2 Preloading with PVD

(a) Figure 8(a) shows the elevation and plan of the clay bounded between adjacent drains AD and BC (in elevation).
(b) The soil bounded by EH and FG (in elevation) shows the central annulus where the *DOC* is less than target 90 pc. The Ring of clay bounded between the boundaries ABCD and EFGH will have *DOC* higher than 90 pc.
(c) The plan view in Figure 8(a) shows the effective stress $H_e\gamma$, due to embankment load is reached at the boundaries dm and cl; but at the edges of the central annulus the values hp and gq correspond to values smaller than $H_e\gamma$.
(d) In Figure 8(b), the settlement time graph OABC corresponds to embankment load $H_e\gamma$. Point A in this graph is at 90 pc *DOC*. Point D on AD corresponds to the settlement at 90 pc *DOC*, with the embankment load $H_e\gamma$, while Point E on EC corresponds to the settlement at 100 pc *DOC* under the embankment load $H_e\gamma$.
(e) If the primary settlement beyond 90 pc *DOC* is not to be included in the residual settlement, then the embankment height must be increased by an additional surcharge h (that is total load corresponds to $(H_e+h)\gamma$), so that the settlement - time plot OFGJ will have a 90 pc consolidation settlement corresponding to the Point F on this graph; the settlement at F is the same as the settlement at E under 100 pc DOC for the embankment load $H_e\gamma$ only.
(f) Further to erase the creep settlement by a prescribed amount then an additional surcharge h_{creep} must be added, so that the total load is $(H_e+h+h_{creep})\,\gamma$. The time settlement graph for this loading is OKLM.

In practice the *DOC* before the removal of surcharge must be higher than 90 pc even as much as 95 pc.

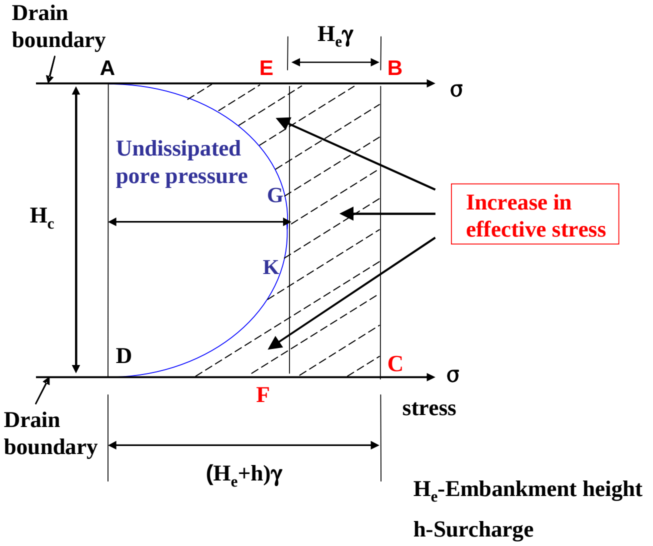


Figure 7. Total stress due to the application of embankment and surcharge loading, the undissipated pore pressure at the time of removal of the preload

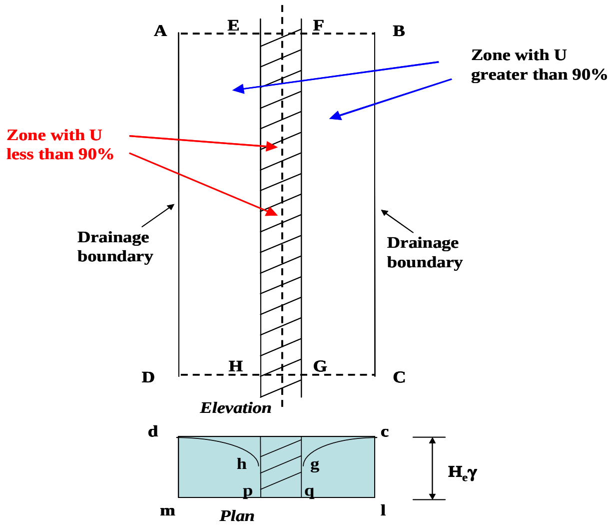


Figure 8(a) Elevation and plan of the clay bounded between adjacent drains AD and BC

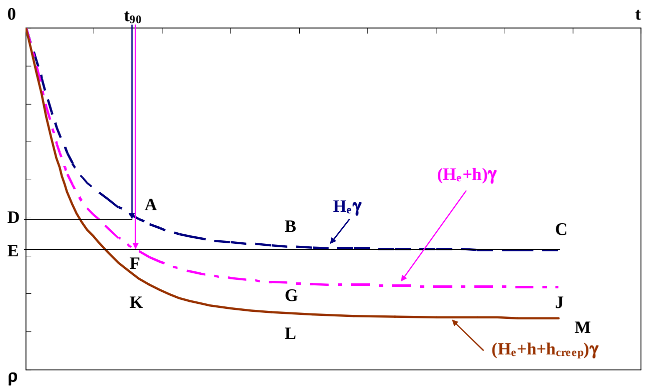


Figure 8. (b) Settlement-time graph corresponds to embankment load $H_e\gamma$

7. OBSERVATIONAL APPROACH IN EVALUATING PVD PERFORMANCE AND RESIDUAL SETTLEMENTS

Infrastructure developments in soft clays have always been based on observational approaches with test embankments constructed on full scale. Extensive studies were made with such embankments in Bangkok clays (Moh *et al.*, 1969; Balasubramaniam *et al*., 1995) and Muar clays (Poulos *et al.,* 1989; Brand and Premchitt, 1989). These test embankments are also fully instrumented. In Southeast Queensland as well (see Wijekulasooriya *et al*., 1999; Ameratunga, 2010), there are many instances in which test embankments were built and the data analysed and used in design and construction monitoring.

It is common in such studies with test embankments to measure surface settlements, sub-surface settlements and sub-surface pore pressures. The deep settlements are measured with settlement gauges and magnetic extensometers. The test embankments can use surcharge as fill material or partial surcharge and vacuum. Early studies carried out in Bangkok with vacuum consolidation experienced substantial difficulties in maintaining the vacuum, but lately the sealing techniques have improved substantially and the vacuum can be maintained satisfactorily over a very long period of even a year or more. The surface settlement measurements are always found to be more reliable and accurate than the sub-surface measurements.

The sub-surface measurements do indicate that the full soil profile is consolidating in any project with wick drains. Extensive studies carried out with the Second International Airport works in Bangkok has clearly indicated that very little differences are noted in the performance of most drains when selected in a proper manner following the specifications and with guaranteed performance. In Southeast Queensland as well, studies have revealed similar observations.

In evaluating the PVD performance and the influence of the installation pattern and spacing, the degree of consolidation (DOC) and the rate of pore pressure dissipation are used as indices to compare the relative merits of each drain and the spacing adopted. There are a number of methods for predicting the 100pc primary consolidation settlement. Asaoka (1978) Method and Hyperbolic Methods are the most widely used methods by engineers. Asaoka plots are found to be more reliable to estimate the ultimate settlements both from the surface and sub-surface settlement measurements. Undoubtedly, the surface measurements are more accurate while the sub surface measurements are also important as they are useful in the estimation of the consolidation settlements in the deeper layers. The pore pressure measurements and their rate of dissipation though very consistent in their values, can at times be subject to doubts if the measurements are taken very close to the drains. The RS calculations need the DOC of the sub-soils at the removal times and this is achieved by using the Hansbo theory and matching the measured surface settlements with the predicted values.

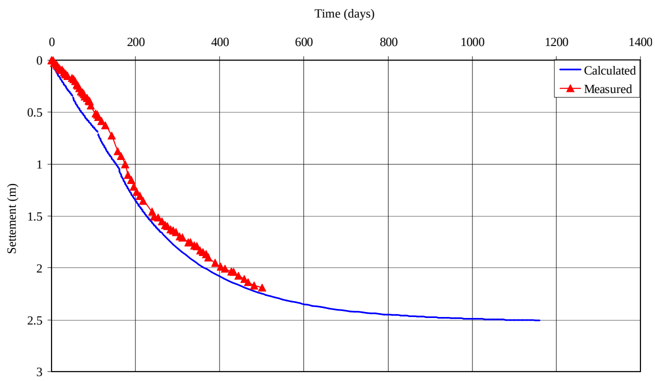


Figure 9. Calculated and measured settlement

Figure 9 presents typical calculated (using Hansbo theory) and measured settlement used to estimate the hundred percent primary settlements for DOC estimation.

In planning such field trial, the earlier studies indicated that it is good to separate the test embankments, rather than having them all side by side, but still many prefer to have them side by side. The trials are generally carried out like a parametric study, where one variable is change at one time, to see the influence of this variable. When triangular and square pattern of drains are used, the equivalent diameter D_e of the drain is found to be a good parameter to quantify the drain spacing and drain pattern.

8. DETERMINATION OF GEOTECHNICAL PARAMETERS

8.1 Geotechnical Parameters

Even though extensive site investigation works are carried out, engineers nowadays tend to keep the variation of each geotechnical parameter to a minimum degree.

In Brisbane (SE Queensland), the natural clay profile is divided into an Upper Holocene Clay and a Lower Holocene Clay. The compression ratio for these clays are generally 0.235, lower values can be encountered if the OCR values are high and these values can be as low as 0.18. Basic studies carried out on soft clays generally indicate that the recompression index RR range from 0.2 to 0.1 of CR in these clays. Generally a lower value of 0.1 is adopted. The greatest variation is noted in the coefficient of consolidation c_v. In most instances, the laboratory consolidation test values of c_v are always very low. CPTu tests at times seem to give very high c_h values. The realistic estimation of this parameter is always difficult; this is where the field settlement-time plots from test embankments could help in estimating field c_v values. It appears the field values are generally taken as 5 to 10 times the lab values. Long term consolidation tests are seldom performed and as such the estimation of C_{α} values also have great uncertainty. The C_{α} values seem to range from 0.005 to 0.008.

8.2 Soil Parameter Determination

This section is discussed under two sub-headings. First the commonly carried out laboratory tests and secondly the in-situ tests.

8.2.1 Soil Parameters from Laboratory Tests

There seems a substantial reduction in the laboratory tests performed. Even the natural water content and index tests are trimmed to the very minimum; and also, particle size distribution. Continuous borehole logging is also not any more in practice.

Consolidation tests are mainly stress controlled tests. But triaxial tests are seldom or never carried out. Even if there is some, it is multi-stage triaxial tests. Stress paths tests and even K_o consolidated triaxial tests are virtually not done. The Index tests are used primarily to determine the strength and compressibility parameters from empirical correlations.

(a) Water content vs. compression index correlations

When the laboratory consolidation tests are few and also, the results are affected by sample disturbance, empirical correlations are often used as fall back to estimate compression ratios. Four such correlations of CR with water content are by Simons and Menzies (1975), Simons (1957), Wilkes (1974) and Lambe and Whitman (1969). The expressions are summarised in Table 3 for CR values.

(b) Empirical equations for *OCR* from Plasticity index

When there are doubts on the quality of samples and the reliability of Oedometer results, empirical relations are relied on for the estimation of OCR values in settlement calculations. Three such classical relations are by Skempton and Henkel (1953), Osterman (1959) and Bjerrum and Simons (1960).

Table 3. Compression Ratio from moisture content

Compression Ratio from moisture content		
Authors	**Formula**	**Range of w_n**
Simons and Menzies (1975)	$CR = 0.006w_n-0.03$	$20 \le w_n \le 140$ (23)
Simons (1957)	$CR = 0.006w_n^{1.68}$	$28 \le w_n \le 57$ (24)
Wilkes (1974)	$CR = 0.26\ln(w_n)-0.83$	$30 \le w_n \le 90$ (25)
Lamb and Whitman (1969)	$CR = 0.12\ln(w_n)-0.28$	$10 \le w_n \le 100$ (26)

(c) Values of $C_{\alpha e}/C_{c\varepsilon}$ for Geotechnical Materials

In geotechnical engineering practice the scholarly work of Mesri *et al*. (1994) is used extensively in estimation of primary and secondary settlements. Values of $C_{\alpha e}/C_{c\varepsilon}$ as given by Mesri *et al.* (1994) are given in Table 4.

Table 4. OCR from Plasticity index

OCR from Plasticity index	
Authors	**Formulae**
Skempton and Henkel (1953)	$OCR = 0.0017I_P+0.5$ (27)
Osterman (1959)	$OCR = 2\times10^{-6}I_P^3-3\times10^{-4}I_P^2+3.1\times10^{-2}I_P+0.41$ (28)
Bjerrum and Simons (1960)	$OCR = 2\times10^{-6}I_P^3-4\times10^{-4}I_P^2+3.35\times10^{-2}I_P+0.28$ (29)

(d) Secondary consolidation parameter from Compression ratio (Mesri *et al.*, 1994)

The secondary compression ratio is estimated from the work of Mesri *et al*. (1994) as follows:

$$\frac{C_\alpha}{1+e_0} = (0.04 \pm 0.01)\frac{C_c}{1+e_0} \tag{30}$$

Lambe and Whitman (1969) also estimated the secondary compression ratio from water content as follows:

$$\frac{C_\alpha}{1+e_0} = (0.002\ to\ 0.01)\frac{w_n\%}{100} \tag{31}$$

Table 5. Values of $C_{\alpha e}/C_{c\varepsilon}$ for Geotechnical Materials (Mesri *et al*., 1994)

Material	**$C_{\alpha e}/C_{c\varepsilon}$**
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
Peat and muskeg	0.06 ± 0.01

(e) Empirical equations for undrained shear strength from Plasticity index

For shear strength the mostly used empirical formula is from Skempton and Henkel (1953). The other classical relations are by Osterman (1959) and Bjerrum and Simons (1960). These expressions are given in Table 6.

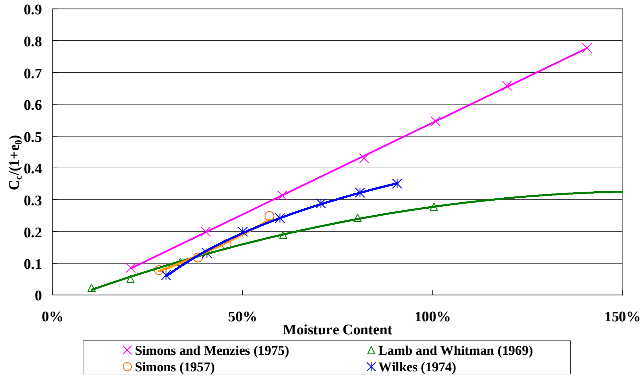


Figure 10. Relationship between $C_c/(1+e_0)$ and natural moisture content, after Simons (1974)

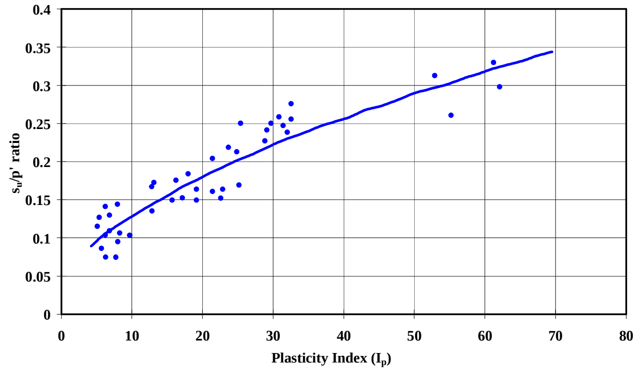


Figure 11. Relationship between s_u/p' and plasticity index, after Bjerrum and Simons (1960)

Table 6. Undrained shear strength from plasticity index

Undrained shear strength from plasticity index	
Authors	**Formulae**
Skempton and Henkel (1953)	$s_u/\sigma'_{vo} = 0.004I_P+0.1$ (32)
Osterman (1959)	$s_u/\sigma'_{vo} = 5\times10^{-7}I_P^3-8\times10^{-5}I_P^2+6.8\times10^{-3}I_P+0.08$ (33)
Bjerrum and Simons (1960)	$s_u/\sigma'_{vo} = 5\times10^{-7}I_P^3-8\times10^{-5}I_P^2+7.4\times10^{-3}I_P+0.06$ (34)

8.2.2 Soil parameter from In-situ tests

Because of sample disturbance in soft soils, in-situ tests are more relied on in practice than laboratory tests. Vane shear tests are traditionally relied upon for undrained shear strength. In the last two to three decades CPT and CPTu are well advanced and heavily relied upon for the estimation of strength and compressibility parameters in soft soils, Lunne *et al*. (2002), Mayne (1986, 1991, 1993).

(a) Coefficient of volume change from CPT

CPT and CPTu tests data are used to obtain the coefficient of volume decrease. The cone resistance q_c and a parameter defined as α_m are used as:

$$M = 1/m_v = \alpha_m.\ q_c \tag{35}$$

(b) Compression index from coefficient of volume change

The coefficient of volume decrease is related to the compression index, C_c as:

$$m_v = \frac{0.435C_c}{(1+e_0)\sigma_{va}} \tag{36}$$

(c) T-Bar tests

T- bar tests are relatively new in onshore geotechnical engineering practice. They are claimed to give better performance than the CPT and CPTu and the results compare well with the strengths obtained from vane tests.

Table 7. Relationship of q_c and q_m (Mitchell and Gardner, 1975)

Soil type	**q_c (MPa)**	$3< \alpha_m <8$
Low plasticity clay	$q_c<0.7$	$2< \alpha_m <5$
	$0.7<q_c<2.0$	$1< \alpha_m<2.5$
	$q_c>2.0$	$3< \alpha_m <6$
Silts of low plasticity	$q_c>2$	$1< \alpha_m <3$
	$q_c<2.0$	$2< \alpha_m <6$
Highly plastic silts and clays	$q_c<2.0$	$2< \alpha_m <8$
Organic silts	$q_c<1.2$	$q_c<0.7$
Peat and organic clay	$q_c<0.7$	
	$50<w<100$	$1< \alpha_m <1.5$
	$100<w<200$	$0.4< \alpha_m <1$
	$w>200$	$3< \alpha_m <8$

(d) Undrained shear strength from CPT

CPT and CPTu tests are first used to estimate the undrained shear strength s_u in clays. In this paper, the compressibility parameters are first presented and then the strength. The expression used for s_u determination is as follows:

$$s_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{37}$$

N_{kt} varies from 15 to 20.

(e) Undrained shear strength of inorganic soft clay and silt deposits

There is a correlation to estimate s_u (Mesri, 1975) from the maximum past pressure or conversely the maximum past pressure from s_u as

$$s_u = 0.22\ \sigma'_p \tag{38}$$

(g) Undrained shear strength of organic soft clay and silt deposits

The coefficient varies from soil to soil (Mesri, 1993), for organic soils, the expression is:

$$s_u = 0.26\ \sigma'_p \tag{39}$$

9. TYPICAL SETTLEMENT AND STABILITY EVALUATIONS IN ESTUARINE CLAYS

9.1 Geotechnical Parameters

(a) Compression ratio (*CR*)

Figure 12 gives the compression ratio. In this plot the laboratory CR values and the values determined from water content are shown. The design CR values are estimated from these data and often compared with back calculated CR values from the test embankment.

(b) Overconsolidation ratio (*OCR*)

The OCR values are determined from the consolidation tests and CPT, CPTu. The values are presented in Figure 13. The CPT tests are found to give higher OCR values than the laboratory consolidation tests.

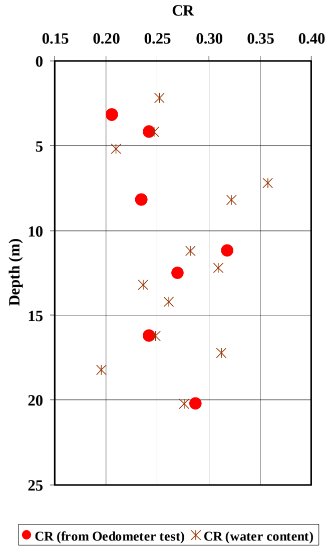


Figure 12. Compression ratio - *CR*

(c) Coefficient of consolidation (c_v)

The coefficient of consolidation is the most difficult parameter to be reliably estimated. The laboratory tests normally under-estimate the c_v values while the c_v values as obtained by scaling down the CPTu c_h values are always found to be much higher. In the test embankment, the back calculated values are found to be higher than the laboratory values but generally smaller than the CPTu values. These results are presented in Figure 14.

(d) Secondary compression parameters ($C_{\alpha\varepsilon}$)

The $C_{\alpha\varepsilon}$ values were determined from the water content as well as from the *CR* values as obtained in the laboratory tests. A Mesri coefficient of 0.035 was used to multiply the *CR* values to obtain $C_{\alpha\varepsilon}$. These values are presented in Table 8.

Generally lower C_α values are used in practice and is in the range 0.003 to 0.0045.

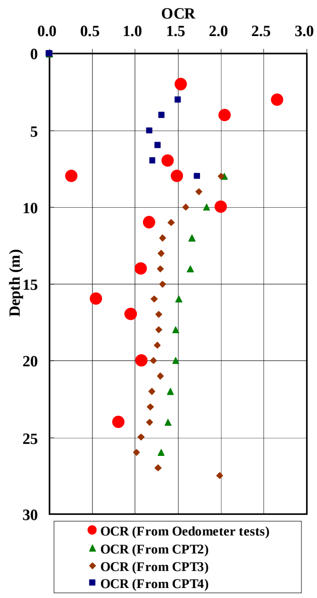


Figure 13. *OCR* with depth

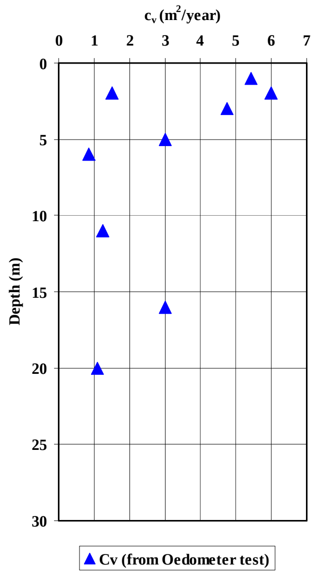


Figure 14. Coefficient of consolidation (c_v) with depth

Table 8 $C_{\alpha\varepsilon}$ values

$C_{\alpha\varepsilon}$ =0.035×(*CR* from water content)	$C_{\alpha\varepsilon}$ =0.035×(*CR* from consolidation test)
0.0104	0.0098
0.0095	0.0088

9.2 Calculation for Settlement

9.2.1 Time to reach ninety percent consolidation

Generally, the c_v values of soft clays encountered in estuarine clays are low and the normal time taken for 90 pc DOC is found to be very high and exceeds the surcharge period adopted in practice as nine months. Figure 15 illustrates that even for a surcharge height of 2m; the time for 90 pc DOC exceeds 9 months. If the post construction settlement is limited to 100mm, then it is noted PVD with spacing of 1.5 m is needed to limit the post construction settlement with 1m surcharge.

9.3 Residual Settlement Criteria

9.3.1 Methods to calculate residual settlement

The RS values are calculated generally by two methods - (1) In Method 1, the RS values depend on the effective stress before stress removal with the appropriate DOC and the final stress level; the RS values are generally high as calculated by this method. (2) This method uses the *C*□□□value in the over-consolidated range and it is noted that *C*□□□in the over-consolidated range reduce sharply even with small values of OCR. Method 2 normally gives much lower values of RS. The RS values depend on the structures built; typical values for building works are 150mm settlement under 15kPa service load in 20 years; these values increase to 250mm settlement under 25kPa service load in 20 years.

9.3.2 Embankment settlement criteria

The critical factors governing the design of road embankments are: stability, total/differential settlement and time for settlement.

The settlement criteria will be discussed herein. Normally settlement criteria for embankments are defined in terms of the allowable total settlement and differential settlement over a given time frame. The time frame is typically the design life of the embankment. The embankment change in grade due to differential settlement is generally anywhere within 20m of the approach of any structure (the "structure zones") must be limited to 0.5% in both the longitudinal direction and transverse direction of the embankment.

The settlement criteria adopted by different countries are summarised in Appendix.

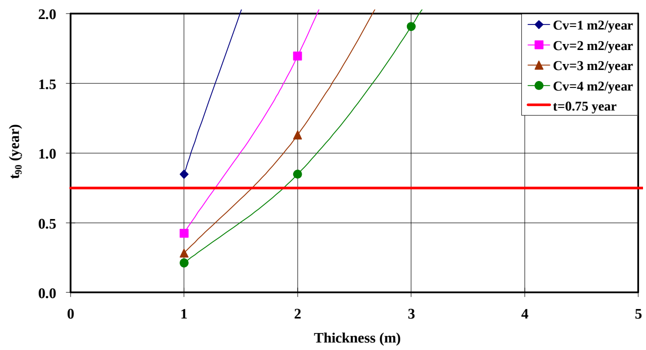


Figure 15. t_{90} with layer thickness for different c_v values

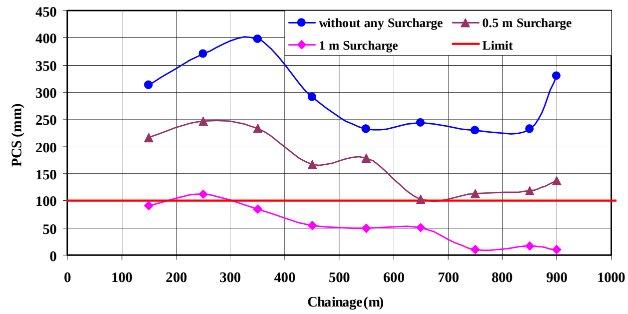


Figure 16. PCS under different surcharge

9.3.3 Zones of approach embankments

The region which extends from structures such as bridges to the normal highway embankment is normally divided into three zones especially when the approaches involve high embankments. Zone 3 is on the side close to the low embankment side. Zone 1 is closer to the structure. In Zone 1, it is recognised that the differential settlement limits may not be met, and the design will be based on total settlement only. However, a hinged approach slab, and possibly with other measures such as reinforced mattress, the differential settlement can to reduced.

9.4 Stability analyses

Stability analyses were carried out on the embankments along the motorway and at the interchange alignments. The side slopes of the embankment are 1V:2H. Both short term and long term stability analyses are carried out for the worst case scenarios in terms of embankment height, water table and underlying soil layers.

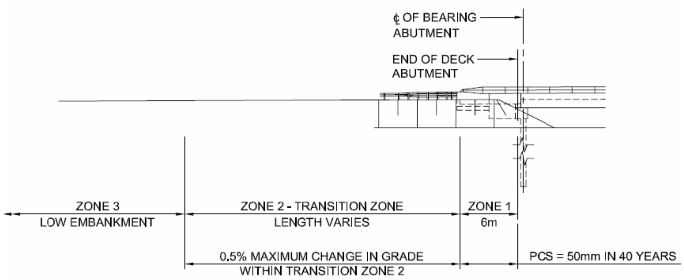


Figure 17. Zones of approach embankment close to bridge culvert and other structures (Hsi and Martin, 2005)

For soft clay stability under short term conditions, the undrained strengths are used in the analysis. For long term conditions, the drained strength is used after taking into account of the effective vertical stress under the embankment loading. The construction loading is taken as 10kN/m² under short term conditions and the traffic loading under long term conditions is taken as 20kN/m^2. The factor of safety under short term and long term stability are taken as 1.3 and 1.5 respectively. In the case where geotextile reinforcement is used a line load is applied at the base equivalent to the ultimate strength of the membrane.

A typical result carried out from Slope/W is presented in Table 9.

Table 9. Slope/W parameters for stability analysis

Embankment height (m)	Surcharge height (m)	Soft clay (m)	FOS
2.5	0	5.6	1.83

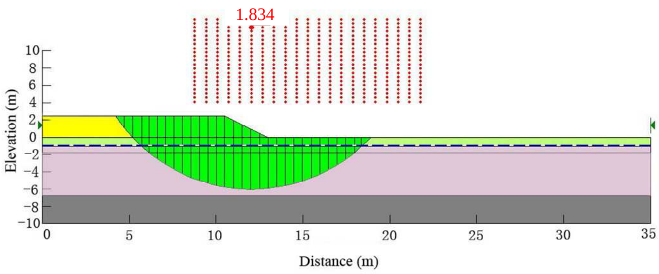


Figure 18. Stability analyses from Slope/W

In most instances circular type of failure is appropriate for both types of analyses. The FOS values are generally the same for Slope/W and Plaxis. However, the Plaxis analyses gave lower FOS values especially when wedge type of failures occurred. Wedge failure occurs when soft clay layer is thin (see Table 10). Figure 15 shows wedge type of failure in Plaxis.

For the cases where the FOS is lower than the stipulated design values, stage loading with waiting periods is recommended. Typical recommendations are contained in Table 11.

Table 10. Embankment details for Plaxis analyses

Embank-ment height (m)	Sur-charge (m)	Berm height (m)	Berm width (m)	Reinforcement	FOS
9.3	0	3.5	6m of berm +3.5 of Rock armour	150 kN @1m above ground surface	<1

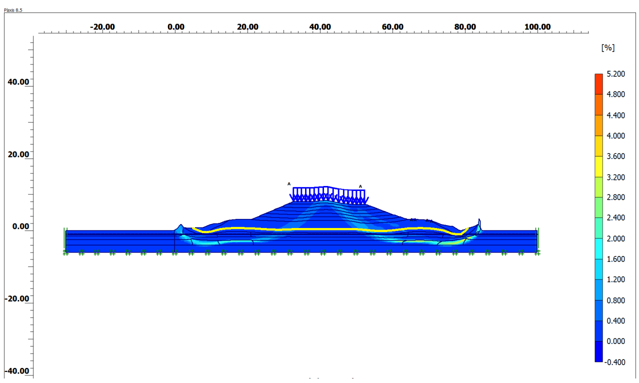


Figure 19. Wedge type of failure from Plaxis

Table 11. Recommendation for FOS<1

Stage	Loading (m)	Loading period including waiting time 30 days (days)	FOS	
			Short-term	Long-term
1	3.5	45	1.77	1.72
2	3	45		
3	2.8	-		

9.5 Reinforced Embankment

The early works carried out on reinforced embankments at AIT are those of Bergado *et al.* (1993a and 1993b) and Long *et al.* (1996). Ting *el al.* (1984), Ting *et al.* (1989), Ting *et al.* (1994), Broms (1986), Chin *et al.* (1989), Jones *et al.* (1990), Han (1999) and Han and Wayne (2000), Poulos (1998), Li *et al.* (2002) have also done pioneering works on reinforced embankments.

9.5.1 Load transfer mechanism for Geogrid reinforced pile supported embankment

In Geogrid Reinforced Pile Supported (GRPS) Embankment system, the load from the embankment fill due to its self-weight will be transferred onto the underlain layers following four paths (shown in Figure 20). The loads are transferred as: load (W_1) transfer to pile supports directly; load transferred to pile supports through arching effect; loads transferred to the pile supports through the geotextile membrane or grid and the load transferred to soil masses under the embankment fill between the pile supports.

Figure 21 shows the stress generated in GRPS embankment, σ_c, τ, σ_{sr} and σ_s, representing stresses due to the force carried by the pile supports, the soil arching, geogrid and soil masses under embankment between pile supports respectively. T_{rp} is the tensile force in the geogrid due to the vertical load.

The column supported embankments consist of vertical columns that are designed to transfer the load of the embankment through the soft compressible soil layer to a firm foundation. The selection of the type of column used for column supported embankments will depend on the design load, the constructability of the column, the cost, etc. The load from the embankment must be effectively transferred to the column to prevent punching of the column through embankment fill creating differential settlement at the surface of the embankment. If the columns are placed close enough together, soil arching will occur and the load will be transferred to the columns. In order to minimize the number of columns required to support the embankment and increase the efficiency of the design, a load transfer platform reinforced with geogrid reinforcement is being used on a regular basis. The load transfer platform consists of one or more layers of geogrid reinforcement placed between the top of the columns and the bottom of the embankment (Collin, 2007).

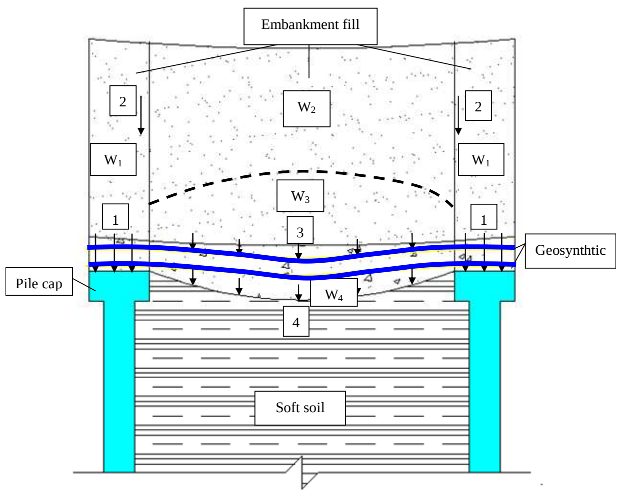


Figure 20. Load transfer mechanisms of GRPS embankment

where:

1 Load (W_1) transfer to pile supports directly.
2 Load transfer to pile supports through arching effect.
3 Load transfer to pile supports through geogrid.
4 Load transfer to soil masses under embankment fill between pile supports

Generally speaking, material used to build Load Transfer platform requires better engineering property than the material used to build the embankment, in addition to that, the LTP needs to be well compacted, thereby enhancing the interaction between the geogrid reinforcement and the fill material; consequently, minimising the settlement occurring at the surface of the embankment, as well as to constrain the deformation of the reinforcement.

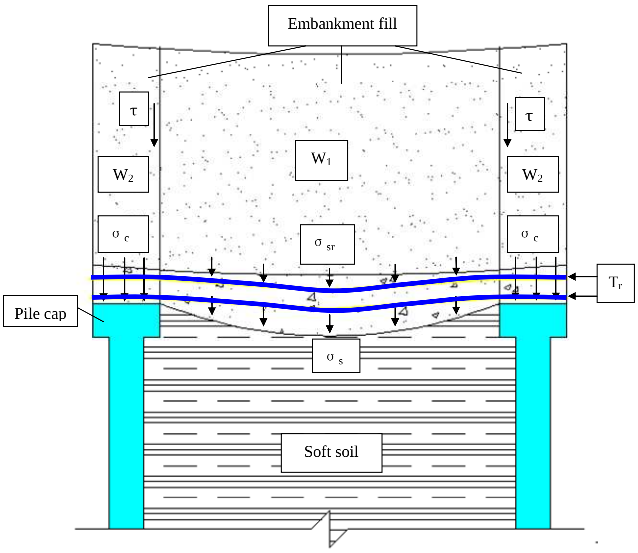


Figure 21. Stress and force generated in GRPS embankment

Russell and Pierpoint (1997) pointed out that due to uncertainty of the foundation behaviour it is generally assumed that the entire vertical load of the embankment is carried by the piles either by soil arching or transferred by the reinforcement. The main difficulty in the design calculation is the assessment of the proportion of the vertical load carried by the reinforcement between the pile supports.

9.5.2 Stress reduction ratio

In order to compare the various design methods, a parameter called the stress reduction ratio (S_{3D}) has been defined (Russell and Pierpoint, 1997). The stress reduction ratio is defined as the ratio of the average vertical stress carried by the reinforcement to the average vertical stress due to the embankment fill.

$$S_{3D} = \frac{\sigma_{sr}}{\gamma H} \tag{40}$$

As shown in Fig. 23, British Standard Method and Terzaghi Method (1943) give close value of stress reduction ratio, S_{3D}, while Hewlett and Randolph Method generates the most conservative value.

Additionally, Fig. 24 shows the comparison of tensile stress in geogrid, the pattern of which is identical to that of the stress reduction ratio plot, only with different magnitude. It is expected when all the parameters are fixed.

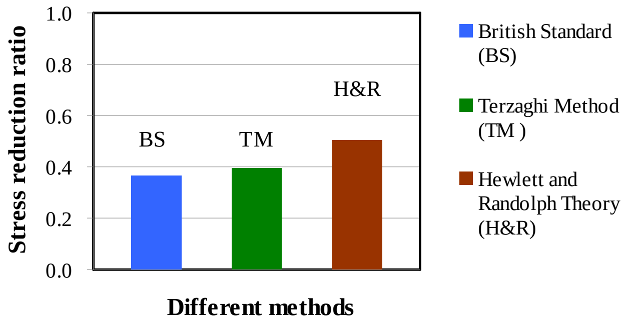


Figure 23. Comparison of stress reduction ratio calculated from four analytical methods

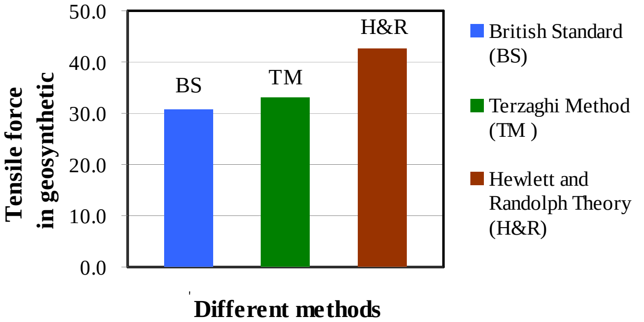


Figure 24. Comparison of tensile force in geogrid calculated from four analytical methods

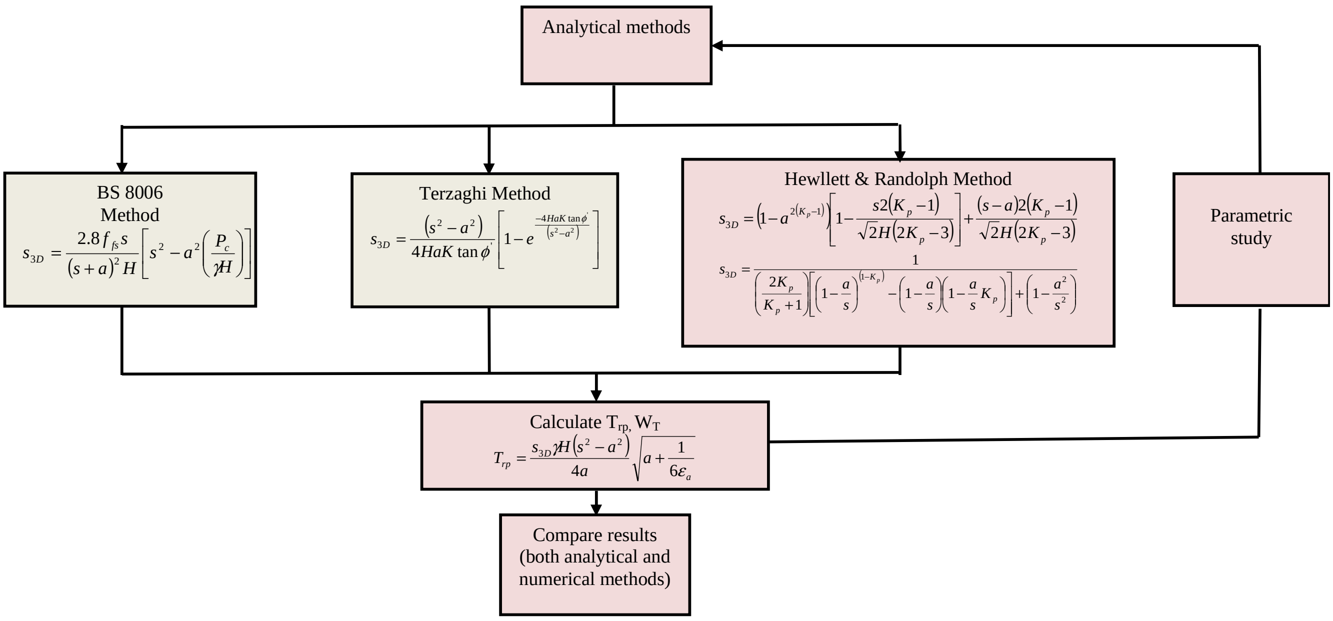


Figure 22. Procedure adopted in the use of analytical methods in GPRS embankments

Figure 25 illustrates the procedure adopted in numerical analysis using the Plaxis program.

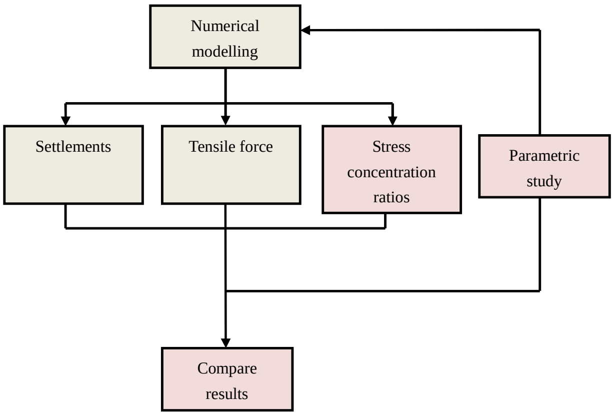


Figure 25. Illustration of the numerical procedure

9.5.3 Influence of support spacing

To analysis the influence of support spacing on stress reduction ratio, the support spacing was varied from 0.5 to 2.5 while other parameters are fixed. Then stress reduction ratio was calculated by using the three analytical methods, and results are plotted below in Fig. 26.

The graph suggests that there is a positive relationship between stress reduction ratio and support spacing. Terzaghi Method (1943) and Hewlett and Randolph Method (1988) produce similar trend in S_{3D} versus support spacing plot. When British Standard Method was used, the calculated stress reduction ratio is sensitive to spacing changing.

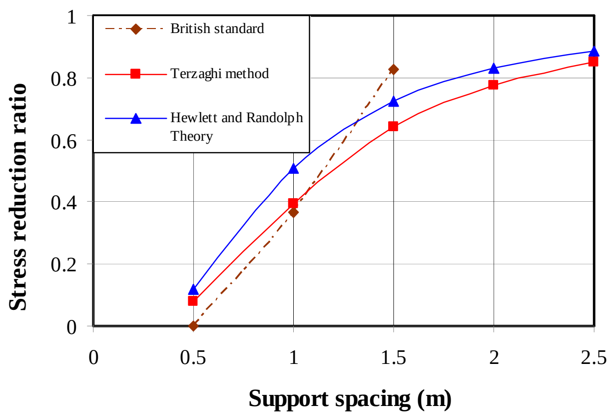


Figure 26. Influence of support spacing on stress reduction ratio

9.5.4 Influence of s/a ratio

As discussed before, to control the design value of tensile force in the geogrid, larger support should be used corresponding to larger spacing, especially when spacing exceeds 2m. Analysis was conducted to check S_{3D} under fixed *s/a* ratio condition (see Figure 27).

S_{3D} of Hewlett and Randolph Method experienced a sudden increase after spacing reaches 2m. It is because S_{3D} at top of support dominates the results until the spacing exceeds 2m, and S_{3D} at top of the support is subject to *s/a* ratio, therefore it remains the same provided that *s/a* ratio is fixed. On the other hand, S_{3D} at the crown of arching soil keep increasing as the spacing of the support increase, and beyond the constant value at the top of the support after 2m spacing; then becomes the larger of the two values and take control of final S_{3D}.

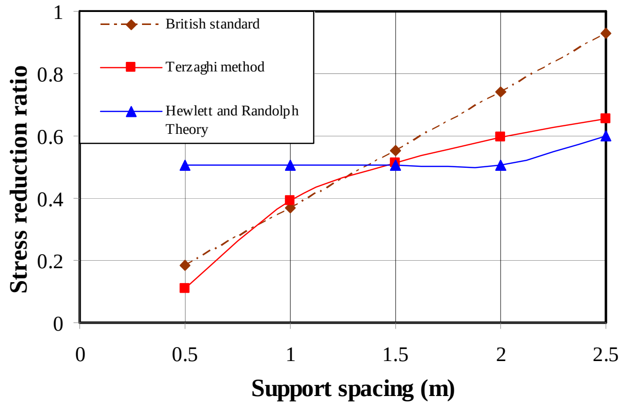


Figure 27. Influence of support spacing on stress reduction ratio with fixed s/a ratio

9.5.5 Influence of embankment material

Only Terzaghi Method (1943) and Hewlett and Randolph Method (1988) consider the effect of embankment fill material on S_{3D}, hence only those two methods were studied by varying friction angle of embankment fill, results are demonstrated in Figure 28.

9.5.6 Results from numerical method (Plaxis)

(a) Axi-symmetric model

The axi-symmetric model is adopted in the Plaxis analysis to perform a parametric study. The four important materials involved in the geometry are the piles, the geogrids, the foundation soil and the embankment fill as shown in Figure 30. Input parameters are shown in Table 11.

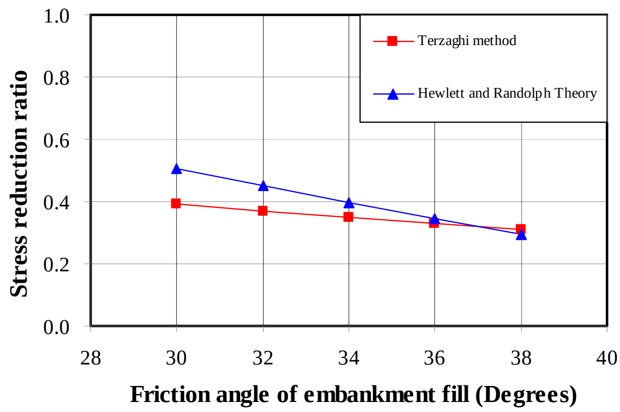


Figure 28. Influence of friction angle of embankment on S_{3D}

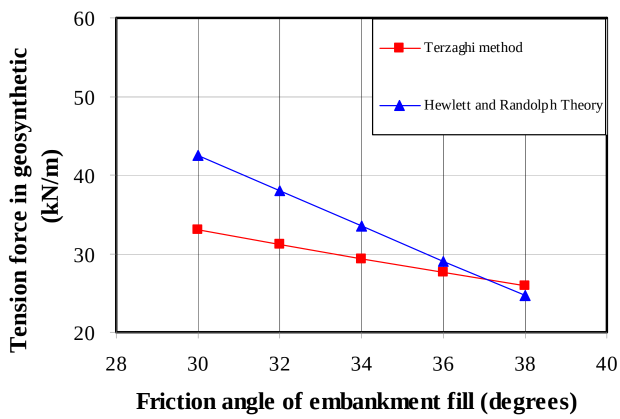


Figure 29. Influence of friction angle of embankment on tensile force in geogrids

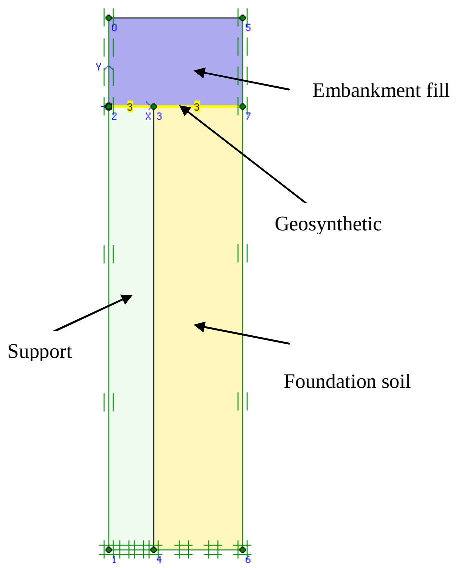


Figure 30. Geometry of axi-symmetric model

Table 11. Properties of soft clay for axi-symmetric modelling

λ*	0.2
κ*	0.05
ν_{ur}	0.15
ϕ'(degrees)	22
c'(kN/m^2)	5

The "Mohr-Coulomb Model" was used for the embankment fill, and input parameters are shown in Table 12. The geogrid is represented by a geotextile element in Plaxis. These are flexible elastic elements that represent sheet of fabric in out of plane direction. They can sustain tensile forces but not compression. The factors those are varied in the parametric study are geogrid stiffness, the height of the embankment, the position of the geogrid layer and the modulus of elasticity of the pile.

Table 12. Properties of embankment fill for axi-symmetric modelling

Unit weight of embankment fill (kN/m^3)	19
ϕ' (degrees)	30
c'(kN/m^2)	1
Elasticity modulus of the fill (kN/m^2)	2000

The maximum settlements at the pile head are studied. The maximum settlements decreased with an increase in the pile modulus. It can also be seen in Fig. 31 that the inclusion of the geogrid layer reduced the maximum settlements greatly. The stress concentration ratio is improved with the inclusion of the geogrid layer; this confirms previous discussion that when the embankment is rigid, stress will concentrate on supports.

As can be seen in Fig. 32, the maximum settlement increased with an increase in the height of embankment. It can be showed again that the presence of the geogrid helps in reducing the maximum settlements by supporting the embankment soil masses. The position of the geogrid with respect to the pile head is also considered and the results are presented in Table 13. The geogrids are placed on the top of the pile had or at some distance from the top of the pile head. It is seen that as the position of the geogrid from the pile head increases, the maximum and differential settlement continue to increase. However, there is a decrease in the tensile stress in the geogrid. This suggests that the efficiency of geogrid is at the highest level when reinforcement is located at the top of the support, and keeps reducing as it is moved up.

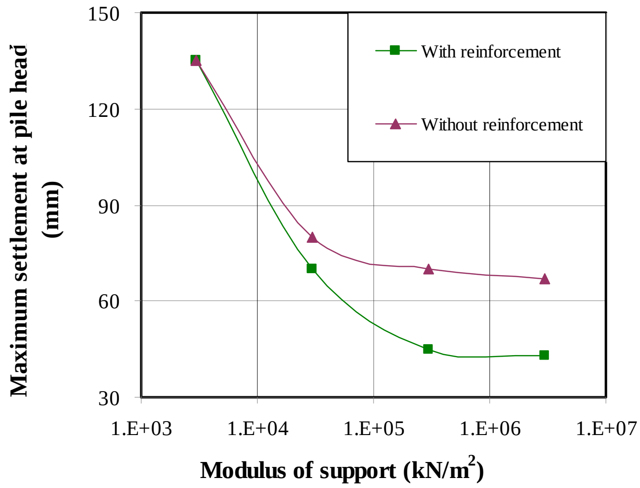


Figure 31. Influence of modulus of support on maximum settlement

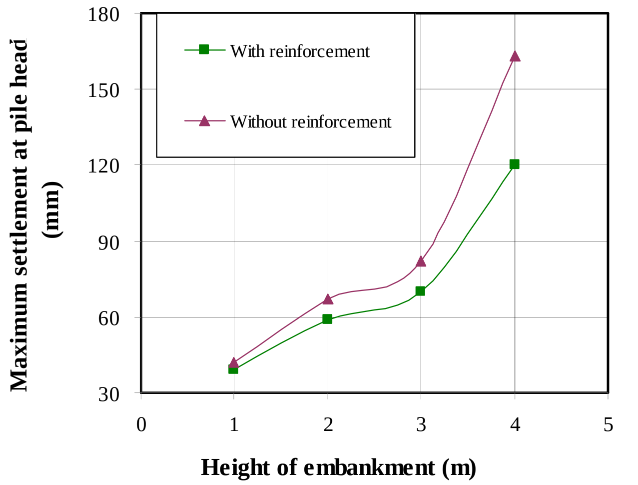


Figure 32. Influence of embankment height on maximum settlement

Table 13. Effect of position of the geogrid on settlements and tensile force in geogrid

Position of geogrid	**On pile head**	**0.1m above pile head**	**0.2m above pile head**
Maximum settlement (mm)	93	94	109
Differential settlement (mm)	92	94	109
Tension force (kN/m)	91	63	45

(b) Plane strain model

When axi-symmetric model is used, the lateral movement of GRPS embankment, the bending moment in the support, and the actual tensile force distribution and deformation shape of the whole system cannot be studied. Therefore, the plane strain model is more frequently used in GRPS embankment design. A deformed mesh can be inspected from Plaxis output as shown in Figure 33.

A comparison between the predicted values from the analytical methods with numerical methods is made in Table 14. Terzaghi and Hewlett and Randolph Methods seem to give results close to those given by Plaxis.

Bergado *et al*. (1999) and Seah *et al*. (2000) reported the use of deep mixing method and cement piles in highways and airport works.

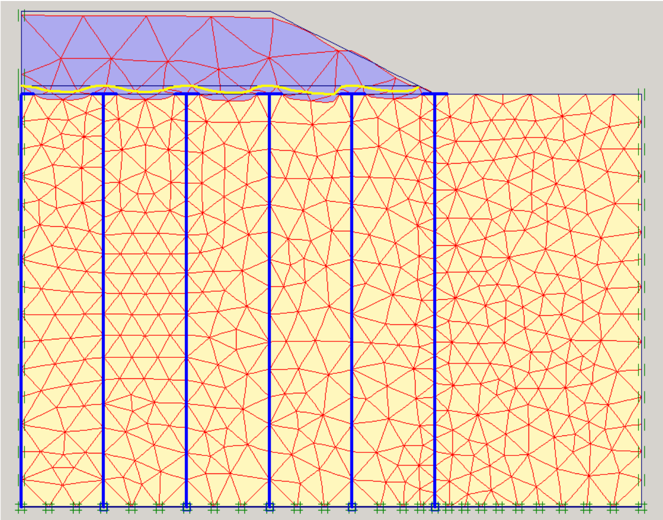


Figure 33. Deformed meshes after calculation (Plane strain model)

Table 14. Comparison between predicted values for tensile force in geogrid reinforcement

Methods	Tensile force (kN/m)
Terzaghi	55
Hewlett and Randolph	47
Plaxis with void between supports	33

10. CONCLUSIONS

The major part of this paper is devoted to the evaluation of settlements in embankments constructed in marine, deltaic and estuarine soft soils. The estuarine deposits are more heterogeneous with the soft soil layer thicknesses relatively small and at times the thicknesses change rapidly within short distances. Over the years, there is a substantial reduction in boreholes and laboratory tests as carried out in site investigation works in soft soils. In situ tests and in particular CPT and CPTu tests now play a dominant role in all site investigation works. A simple voids ratio-logarithmic effective stress relationship is shown to be very helpful in understanding the degree of consolidation (DOC) and the OCR during removal of surcharge and in estimating residual settlement arising from secondary settlement. The classical expressions used in the evaluation of settlements with and without PVD are tabulated. The role of DOC in curtailing excessive residual settlement for preloading with and without PVD is shown with diagrammatic sketches using the pore pressure isochrones. The observational approach in designing embankments and reclamation works as based on fully instrumented test embankments is recommended. The Asaoka method of estimating ultimate settlement from measured surface settlement and then estimating DOC is recommended. The Hansbo method is found to be adequate in works related to preloading with PVD. The current geotechnical practice seem to need the classical work done on shear strength and compressibility of soft soils as there is a drastic reduction in traditional laboratory tests in estimating these parameters and also the doubt on the quality of samples. The use of CPT and CPTu tests is also emphasised. In analysing the slopes of embankments, wedge type of analysis is recommended when the thickness of the soft soil is relatively small and the soft soils are underlain by hard layers.

Preliminary works on the geogrid reinforced pile supported (GRPS) embankments is presented. BS8006, Terzaghi and Hewllett and Randolph methods are found to make similar predictions in line with the numerical analyses using Plaxis software for the behaviour of GRPS embankments as used in approaches closed to structures.

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12. NOTATION

12.1 Notations for settlement

c_v	Coefficient of consolidation in vertical direction
C_r	Recompression index
C_c	Compression index
CR	Compression ratio
$C_{\alpha e}$	Secondary compression parameter
$C_{\alpha e(oc)}$	Overconsolidated secondary compression parameter
d_m	Equivalent diameter of mandrel
d_s	Diameter of the cylinder of influence of the drain (drain influence zone)
d_w	Equivalent diameter of mandrel
D_e	Diameter of a circular drain
e_0	Initial voids ratio
F	
$F(n)$	Drain spacing factor
$F(r)$	Drain resistance factor
$F(s)$	Soil disturbance factor
h	Surcharge height during preloading
h_{creep}	Surcharge to erase creep settlement
h_e	Drainage height/length
H	Layer thickness
H_c	Layer thickness
H_e	Embankment height
I_p	Plasticity index
k_h	Coefficient of permeability in the horizontal direction in the undisturbed soil
k_s	Coefficient of permeability in the horizontal direction in the disturbed soil
l	Width of vertical drain
m	Power
m_v	Coefficient of volume compressibility
N_{kt}	Cone factors
OCR	Overconsolidation ratio
q_t	Corrected cone resistant
$q_{T\text{-}Bar}$	T-bar resistance
s_u	Undrained shear strength
t_{100}	Time for 100% primary consolidation
t_p	Time to complete primary consolidation
T	Consolidation time
T_h	Time factor for horizontal consolidation
T_v	Time factor for vertical consolidation
U	Degree of consolidation
U_h	Degree of consolidation due to horizontal drainage
U_v	Degree of consolidation due to vertical drainage
w	Thickness of vertical drain
w_n	Natural water content
σ_{v0}	Initial vertical stress
σ_{va}	Average vertical stress

12.2 Notations for GPRS embankment

a	the size of the pile caps
c	Cohesion
C_c	Arching coefficient
f_{fs}	Partial factor for soil unit weight
H	Height of embankment
K_0	Coefficient of earth pressure at rest
K_p	Coefficient of Passive earth pressure
S_{3D}	Stress reduction ratio
s	Spacing of piles
ε_α	Axial strain
ϕ	Friction angle
γ	Unit weight of soil
κ^*	Modified swelling index
λ^*	Modified compression index
ν_{ur}	Unloading reloading Poisson ratio
σ_c	Load supported by column
σ_s	Load supported by foundation soil
σ_{sr}	Load supported by reinforcement
τ	Shear stress

13. APPENDIX

Country		Settlement Criteria
USA		(a) A slope of 1 in 200 is typically accepted. Such a slope tends to create a bump generating a dynamic factor for trucks of the order of 1.5 at most at highway speeds when the truck “takes off” and lands on the bridge deck. The bridge beams should be designed for this increase in dynamic transient load. It is not easy to minimize the bump. (b) 0.5% is typical for an approach relative gradient in the design even though a lot of approach embankments in practice do not meet this criterion. Studies showed that 0.5 in (12mm) differential settlement at the interface likely require maintenance but not intolerable.
Australia		There are standard criteria for the RTA and QMR highways with a design speed greater than 100 km/hr. Essentially these criteria are to satisfy the riders’ comfort which is governed by change in grade of the pavement. They are indeed tight and have to be met.
Singapore		In Singapore, the post construction settlement for road embankment should not be more than 50mm (the above criteria is 100mm - 0.5% of 20m) and differential not more than 1:200 Since the magnitude is small, in all projects, near 100% consolidation under design load is emphasised- this is achieved either by more surcharge, closer drain spacing, longer consolidation time or a combination these three design factors - i.e. easier and cheaper to “over-treat” than to do remedial measures when post-construction settlement exceeds the design criteria. If the soils are prone to secondary compression, treatment must also mitigate the secondary settlement.
Malaysia	**North- South Highway Concessionaire's) design criteria**	The design of embankments, particularly in areas of soft ground for which special ground treatment is either shown in the existing design or is indicated as a result of supplementary soils investigations, is to be reviewed. The minimisation of ground treatment work and expense are the prime objectives consistent with providing a satisfactory level of service in terms of acceptable post-construction settlements of the pavement, both differential and overall. In this regard the following criteria shall apply. *Total Settlement:* (i) Following the opening of the Expressway for public use the settlement within the first seven years of service shall not exceed 10% of the sum of the total theoretical primary consolidation settlement and secondary settlement, the latter being assessed for a period of 20 years; (ii) In addition, settlement within the first seven years of service shall nowhere exceed 400mm. *Differential Settlement:* (i) In areas of transition between piled approach embankments and general low embankments differential settlement within the first seven years of service shall not exceed 100mm within a length of 50m; and (ii) In areas remote from structures and transition zones differential settlement shall not exceed 100mm within a length of 100m.
	JKR (PWD) design criteria	Total post construction settlement < 250mm except for approach embankment. For embankment within 10m from bridge abutment, the above settlement criteria should be reduced to 15%.