# On the Preconsolidation Pressure: Experience Based on Testing the Holocene Marine Clay of Peninsula Malaysia

Michael Dobie[1] BSc MSc DIC CEng FICE FCIHT
[1]*Regional Manager, Asia Pacific, Tensar International Limited, Jakarta, Indonesia*
*(Previously: Technical Adviser, Soil Centralab Sdn Bhd, Bangi, Malaysia)*
*E-mail*: mdobie@tensar.co.id

**ABSTRACT:** Prediction of the consolidation settlement of very soft alluvial clays in general requires knowledge of the compressibility characteristics of the deposit, but in particular it requires an accurate determination of the preconsolidation pressure. This defines the value of vertical effective stress where settlement behaviour changes from overconsolidated (OC) to normally consolidated (NC). In the OC stress range settlements are likely to be relatively small, but once into the NC range, they can become very large. Therefore the accurate determination of the preconsolidation pressure is essential if reliable consolidation settlement predictions are to be made. This is examined in detail by back-analysing settlement data from two trial embankments which were built over 13m of Holocene marine clay at Juru (south of Butterworth), as part of the geotechnical investigations carried out for the North-South Expressway project over the period 1990 to 1991, then making comparisons to settlement calculated from measured compressibility properties. The definitive determination of preconsolidation pressure is derived from the behaviour of the trial embankment itself, which is then compared with assessments based on undrained shear strength, oedometer test results and piezocone tests. Issues and potential misuse of all these test methods are examined, and test procedures described to minimise related inaccuracy.

**KEYWORDS:** Preconsolidation pressure, Holocene marine clay, Embankment settlement, Oedometer, Undrained shear strength, Piezocone

## 1. INTRODUCTION

The Central Soils Laboratory (known as CSL) was established near Kuala Lumpur in 1989 with the aim of providing high quality site investigation data for the Malaysian North-South Expressway project, in particular for the Holocene marine clays which underlie a substantial part of the route. Accurate data for these deposits was seen as key to optimising earthworks design, and the head of the supervising geotechnical group was specific that close attention should be given to the determination of the preconsolidation pressure of these deposits. More recently, keynote lectures on soft soil investigation by Professor Gholamreza Mesri (Mesri, 2007) have also stressed that the preconsolidation pressure is the most important parameter contributing to accurate prediction of the behaviour of such deposits. The preconsolidation pressure is defined as the value of vertical effective stress where settlement behaviour changes from overconsolidated (OC) to normally consolidated (NC). It may be determined directly or assessed indirectly from both laboratory and in-situ testing.

Geotechnical site investigation work carried out for the North-South Expressway provides an excellent opportunity to examine this parameter in detail, specifically at Juru (south of Butterworth) where two trial embankments were built in 1990 and then monitored after carrying out a detailed site investigation. Much of the data and experience gained from this work was published in the Proceedings of a Seminar on Geotechnical Aspects of the North-South Expressway held in Kuala Lumpur in November 1990.

The purpose of this paper is to provide a detailed description of the ground conditions and the trial embankments constructed at Juru, looking at the development of settlement and dissipation of excess pore water pressure with time, in order to establish the likely long term settlement. This is then compared to settlement calculated from measured compressibility data, but with specific attention paid to the influence of the preconsolidation pressure used in the calculation. The definitive value of the preconsolidation pressure comes from the trial embankment itself, and this is then compared with assessments based on oedometer test results, undrained shear strength and piezocone tests. Issues and potential misuse of all these test methods are examined, and procedures given to minimise test procedure related inaccuracy. It should be noted that this paper is concerned only with the magnitude of settlement, and not the rate at which the settlement takes place, so that the information presented is only relevant to the calculation of total settlement.

## 2. THE JURU TRIAL EMBANKMENTS

### 2.1 Ground conditions

Detailed information concerning the geotechnical ground conditions at the Juru site is provided by Wan Hashimi et al (1990) and Ramli et al (1991a & 1991b). The alluvial clay in areas occupied by the trial embankments has a relatively uniform thickness of 13 to 14m, but with some important features, as noted below:

- The clay was deposited in a shallow marine or estuarine environment during the Holocene, between 5000 and 9000 years ago. Pore water salinity was measured at around 12 gm/l (Nicholls & Ho, 1990), considerably less than sea-water, indicating that some leaching has taken place since deposition.
- The site of the Juru trial embankment was previously used as a pineapple plantation, with the ground level around 0.5m to 1.0m above MSL. There was no sign of any significant filling or other earthworks having taken place, so that the ground surface may be considered as "original" apart from disturbance due to agriculture, with a thin desiccated surface crust.
- The mineralogy is unusual, with the main features summarised on Figure 1. Kaolinite (K), illite (I) and montmorillonite (M) are present in roughly equal proportions (see Plate 1a, after Raj & Ho, 1990). However up to 45% of the deposit is quartz, although more typically around 30% (Raj & Malek, 1990).

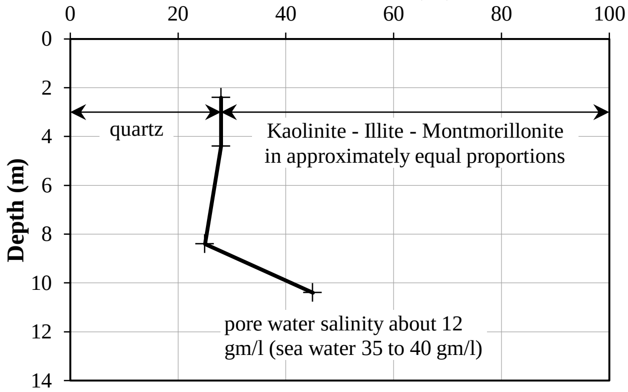
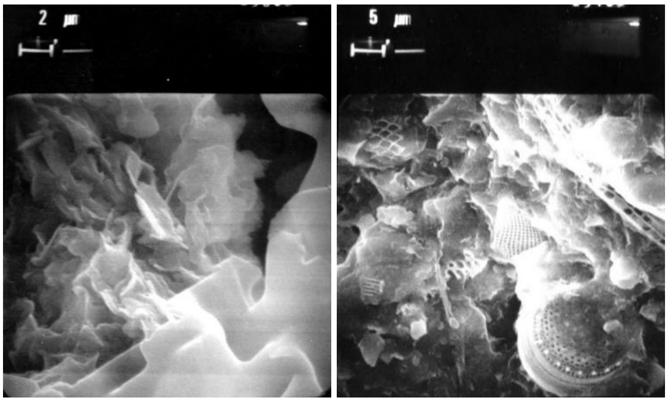



Figure 1 Mineralogy and pore water chemistry of Juru clay

- The explanation for the high quartz content is the plentiful presence of siliceous marine diatoms, which may be seen on Plate 1b (from Raj & Ho, 1990), with particle size in the clay to fine silt range. It should be noted that this high diatom content was only encountered at this one Holocene marine clay site among several studied along the alignment of the North-South Expressway.
- The high quartz content has a major influence on the drained shear strength of the clay which was consistently measured as $\phi' = 30°$ in CIU triaxial tests, a surprisingly high value for a clay with LL ≈ 125 plotting above the "A" line.
- The clay properties are remarkably uniform with plastic limit ≈ 45, liquid limit ≈ 125 and natural water content close to the liquid limit (see Figure 2). Void ratio is ≈ 3.0, and bulk unit weight ≈ 14 kN/m$^3$. This immediately suggests that the clay is highly compressible.
- Figure 3 shows a typical piezocone profile from the Juru site. From the upper part of the profile, the desiccated crust can be seen clearly, extending for about 1m. Below the crust the measured local friction, cone resistance and water pressure all increase with increasing depth as expected in such a deposit. The cone resistance plot includes a second trace labelled $q_T$ which is calculated from the measured $q_c$, but takes into account the water pressure on the back of the cone as described by Dobie & Wong (1990). This correction is essential when using cone resistance in correlations.




Plates 1a & 1b SEM images from the Juru clay: clay mineral platelets on the left, siliceous diatoms on the right

- Another unusual feature of the piezocone profile is the relatively "spikey" nature of each of the plots, which are normally much smoother in Malaysian marine clays. It is considered that this is caused by the presence of discrete organic debris, such as reeds and roots, which were frequently observed in sample

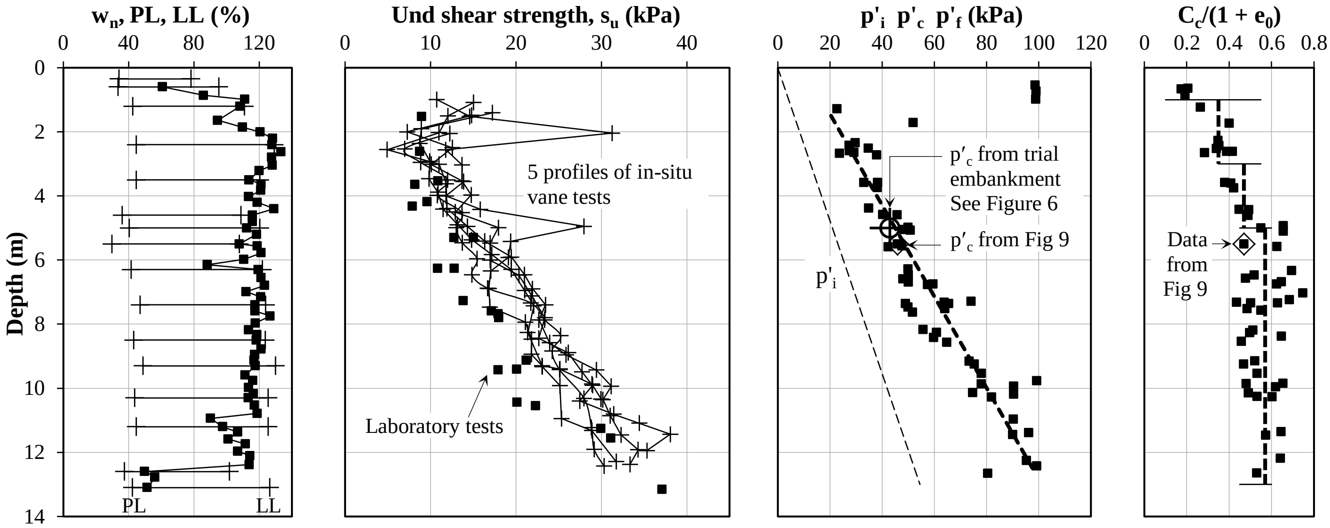



Figure 2 Index properties, undrained shear strength, preconsolidation pressure and compressibility for the Holocene marine clay at Juru

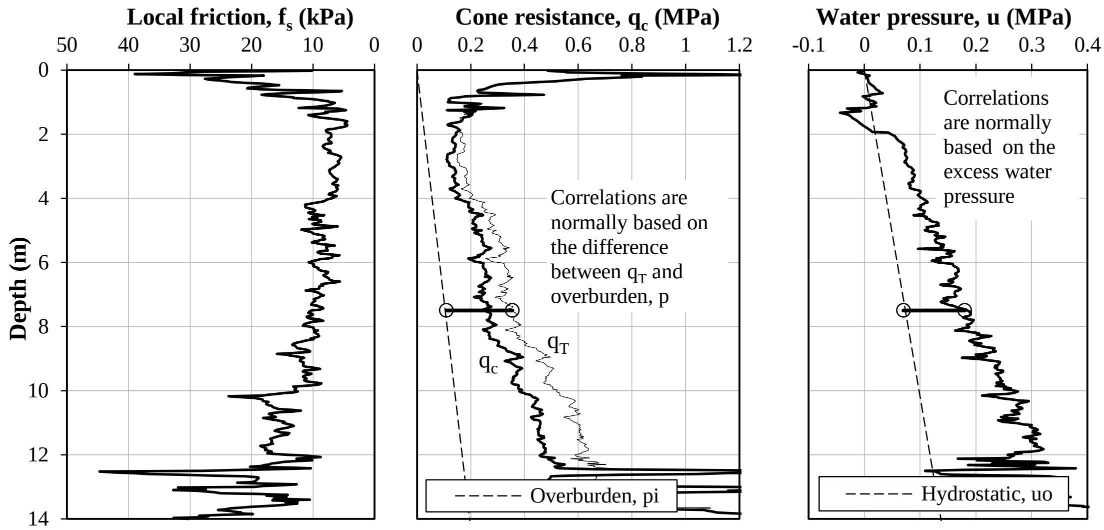



Figure 3 Typical piezocone (CPTu) profile through the Holocene marine clay at Juru

Figure 2 presents profiles of index properties, undrained shear strength from in-situ vane and laboratory UU triaxial tests, as well as preconsolidation pressure and compression ratio from oedometer tests. It should be noted that the taking, storing, transportation and later preparation of undisturbed samples by CSL to obtain this data was carried out to the very highest standards (Nicholls, 1990).

### 2.2 Trial embankment layout and performance

Two trial embankments were built at Juru, both with a cross-section as shown on Figure 4, one with vertical drains under the high part of the embankment and one control embankment without vertical drains. The embankments were built on the alignment of the North-South Expressway, each 100m long separated by 60m. Figure 4 also shows the instrumentation installed in order to monitor the behaviour of the embankments, principally deformation and pore water pressure. Full details are provided by Wan Hashimi et al (1990) and Ramli et al (1991a). Importantly the instrumentation included a reference piezometer group well remote from the embankments and a deep datum to provide a reference for settlement measurements. The reference piezometer group confirmed that the water pressure profile was hydrostatic, with the phreatic surface very close to ground level.

The main performance results of interest to the subject of this paper are shown on Figure 5. The upper part shows the construction history versus time, namely the embankment load calculated as a total vertical pressure. This graph also shows the maximum excess pore water pressure, measured by the piezometers installed at 5m depth.

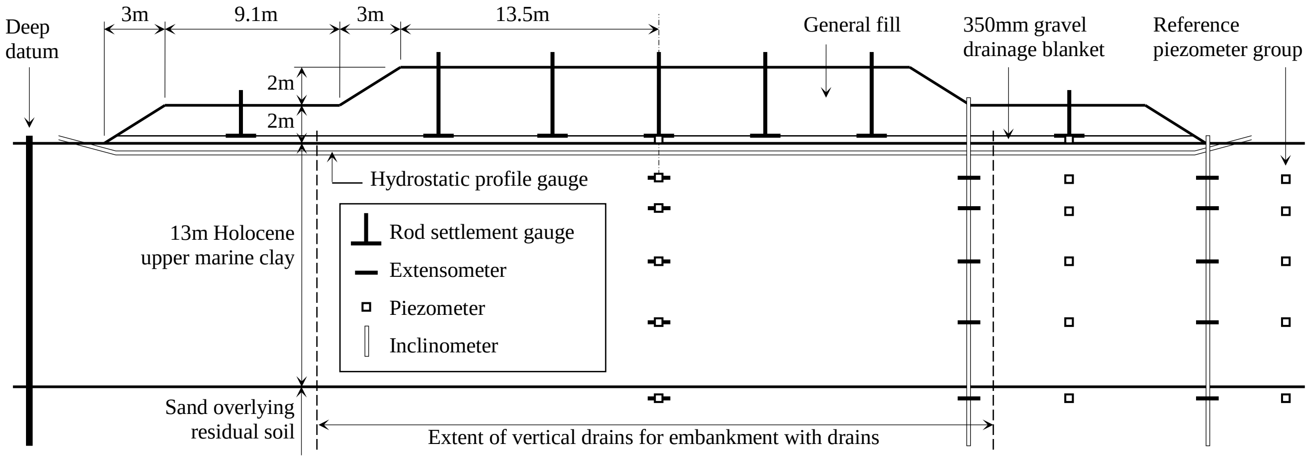



Figure 4 Cross section through the trial embankments constructed at Juru, showing the layout of instrumentation

Comparing each excess pore water pressure trace with the corresponding embankment load trace indicates that the magnitude of the pore water pressure responded very closely to changes in the total embankment load during construction. For the embankment with drains a distinct spike in both traces may be seen at Day 147. This was caused by a rapid removal of 0.5m of fill following the addition of 1m of fill in a few days. Settlement had reached 20mm/day, so the unloading was carried out to avoid a possible failure. However this incident confirmed how very well the piezometers were reacting to changes in total load from the embankment.

The lower part of Figure 5 shows the maximum vertical settlement of each embankment (at the centreline) and the maximum lateral displacement at the toe of each embankment (a short way below the original ground surface). The values of U indicated at the end of each settlement trace give the degree of consolidation at that time based on the distribution of excess pore water measured in November 1991, and reported by Ramli et al (1991b).

Figure 6 shows a plot of excess pore water pressure at 5m depth versus the total embankment load, corrected for submergence. This method of examining the undrained behaviour of the clay foundation is based on Leroueil et al (1985). The parallel nature of the trace with the 45° line above 50 kPa indicates that Skempton's pore pressure parameter B $\approx$ 1.0, and the (p - u) shift of 22 kPa provides a method of assessing the preconsolidation pressure.

### 2.3 Assessment of settlement

One weakness of the Juru trial embankments was that they were built on the alignment of the highway, so that the post-construction performance was only recorded for about one year, as per Figure 5. Completion of the highway effectively terminated the trial, so that the record on Figure 5 is all that exists to judge the likely long term behaviour. It is interesting to note that by Day 500, the embankment with drains had settled 1.9m with a rate of 0.9 mm/day, whereas the control had settled 1.1m with a rate of 0.8 mm/day.

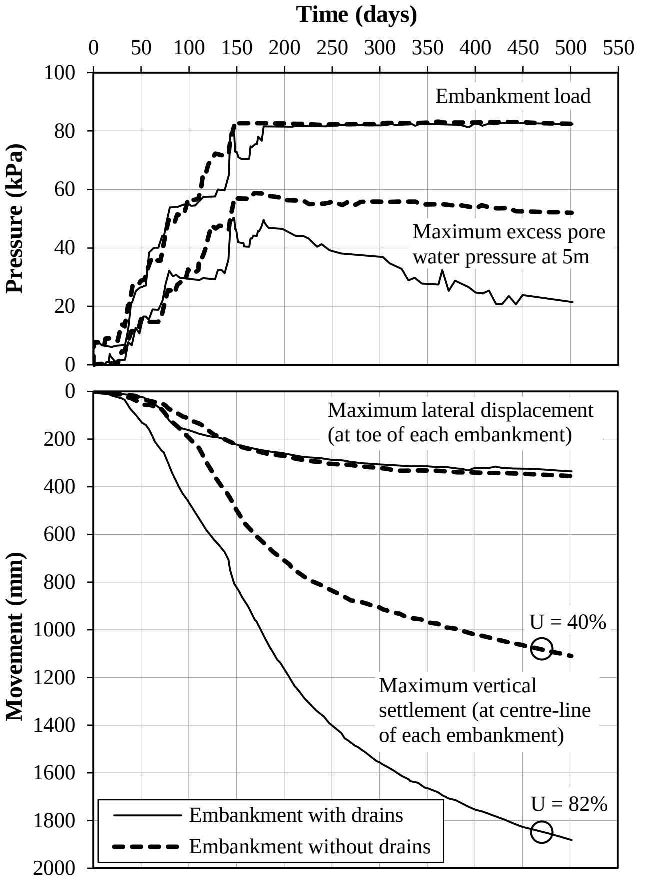



Figure 5 Embankment load, maximum excess pore water pressure and embankment deformation versus time

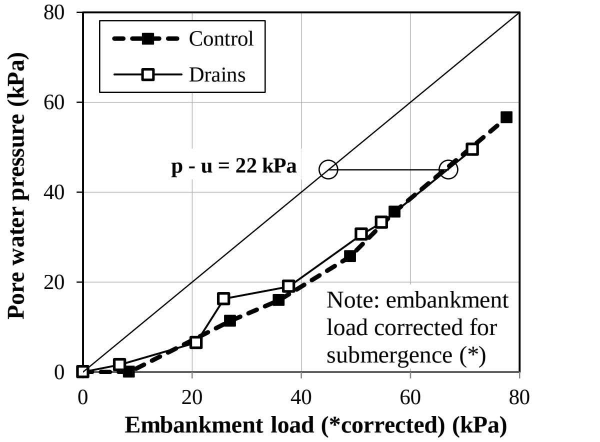



Figure 6 Excess pore pressure (at 5m depth) versus embankment load for both embankments

The settlement data for the embankment with drains is amenable to analysis using Asaoka's method, and the result is shown on Figure 7. The analysis was performed with a time interval $\Delta t$ = 20, 40 and 60 days, all suggesting a final settlement of around 2100mm. For the control embankment, the last 200 days of settlement increased almost linearly with time, so that the Asaoka method is not reliable

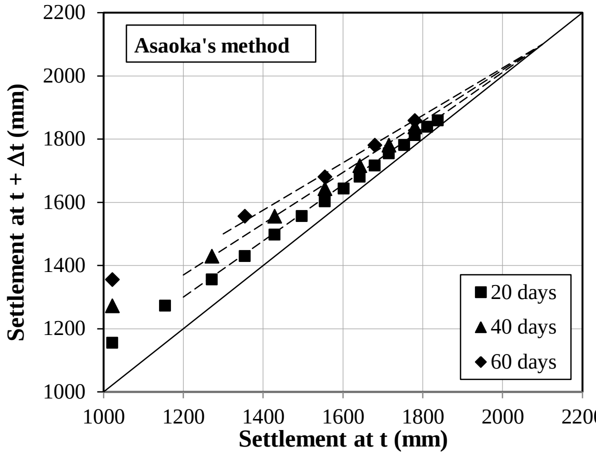



Figure 7 Assessment of final settlement of embankment with drains using Asaoka's method

Calculation of the centreline settlement of the Juru trial embankment has been carried out as follows, dividing the settlement into three components:

(1) The instantaneous undrained settlement ($\Delta H_i$) due to distortion or lateral flow of the clay beneath the embankment under conditions of zero volume change. This settlement cannot be distinguished in embankment settlement records, because the fill is built up gradually. A number of methods are published to assess this, for example Leroueil at al (1985) give this approximate expression:

$$\Delta H_i = (0.07 \pm 0.03)(H - H_{crit}) \tag{1}$$

Where: H is the embankment height, and $H_{crit}$ is the critical height where settlement behaviour switches to normally consolidated. In the case of the Juru embankments $H_{crit} \approx 1.0$m, so assuming a mean value, $\Delta H_i \approx 210$mm.

(2) Consolidation settlement in the overconsolidated range of stress ($\Delta H_{OC}$), given by:

$$\Delta H_{OC} = H_0 m_{vOC}(p'_c - p'_i) \tag{2}$$

Where: $H_0$ is the initial layer thickness and $m_{vOC}$ is the coefficient of volume compressibility in the OC range.

(3) Consolidation settlement in the normally consolidated range of stress ($\Delta H_{NC}$), given by:

$$\Delta H_{NC} = H_0 \frac{C_c}{1+e_0} \log_{10} \frac{p'_f}{p'_c} \tag{3}$$

Where: $C_c$ is the compression index in the NC range.

In order to carry out the calculation of consolidation settlement following the method given in (2) and (3) above, the values of $m_{vOC}$ have been taken from typical consolidation tests results, as shown on Figure 9. The values of $C_c/(1 + e_0)$ are taken as means of measured data according to depth, shown as the dotted lines on the right-hand profile of Figure 2. To complete the calculations, the pressures $p'_i$, $p'_c$ and $p'_f$ are required. Profiles of these pressures are shown on Figure 8, and are determined as follows. The initial vertical effective stress ($p'_i$) is based on a unit weight of 14 kN/m$^3$ with the water table at ground level. $\Delta p'$ due to the embankment is calculated using an elastic solution from Gray (1936) which permits modelling of the stress increase due to an embankment of the shape shown in Figure 4, which gives $p'_f$. The profile of $p'_c$ is the mean of the data shown, taken from 56 oedometer tests, omitting any obvious outliers. The plot also includes $p'_c$ determined from the trial embankment itself, based on Figure 6, which plots very close to the mean line.

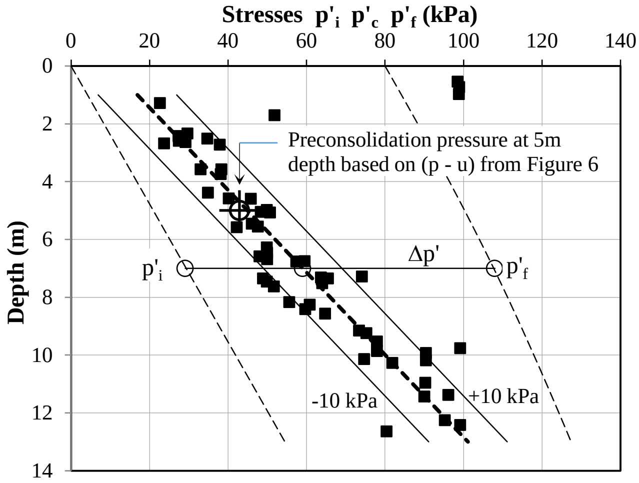



Figure 8 Stresses which control the settlement calculation

The calculation of consolidation settlement based on the methods and data described above is tabulated in Table 1. The Holocene marine clay is divided into 13 layers each 1m thick, with the top 1m representing the desiccated crust. The right-hand part of Table 1 is an adjustment made to allow for submergence, because the lower part of the fill will be below the phreatic line once settlement has taken place. This results in $\Delta H_{OC}$ = 274mm and $\Delta H_{NC}$ = 1465mm, so the total consolidation settlement $\Delta H_C$ = 1739mm.

This calculated value may be reconciled with the settlement record shown on Figure 5 (lower) for the control embankment as follows:

- By Day 470, total settlement reached 1080mm, with U = 40%
- At this time $\Delta H_i$ (= 210mm) and $\Delta H_{OC}$ (= 274mm) are complete
- The remaining settlement (1080 - 210 - 274 = 596mm) is part of $\Delta H_{NC}$
- Making the assumption that U = 40% applies only to $\Delta H_{NC}$, then the total $\Delta H_{NC}$ would be 596/0.4 = 1490mm
- This compares very well with the calculated $\Delta H_{NC}$ of 1465mm

Table 1 Settlement calculation for the Juru control embankment based on the mean preconsolidation pressure from oedometer tests

| **Layer details** | | **Stresses** | | | **Compressibility** | | **Settlement** | | | **Correction for submergence** | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Layer** | **Thickness** | **$p'_i$** | **$p'_c$** | **$p'_f$** | **$m_{vOC}$** | **$C_C/(1+e_0)$** | **$\Delta H_{OC}$** | **$\Delta H_{NC}$** | **$\Delta H_C$** | **$\Delta\sigma'_v$** | **$p'_{fs}$** | **$\Delta H_{OC}$** | **$\Delta H_{NC}$** | **$\Delta H_C$** |
| No | mm | kPa | kPa | kPa | $m^2$/MN | | mm | mm | mm | kPa | kPa | mm | mm | mm |
| 1 | 1000 | 2.1 | **high** | 82.1 | 0.4 | 0.334 | 32 | | 32 | -18.4 | 63.7 | 25 | | 25 |
| 2 | 1000 | 6.3 | **20.4** | 86.3 | 0.7 | 0.356 | 10 | 209 | 219 | -16.9 | 69.4 | 10 | 177 | 187 |
| 3 | 1000 | 10.5 | **27.4** | 90.4 | 0.7 | 0.404 | 12 | 184 | 196 | -15.4 | 74.9 | 12 | 155 | 167 |
| 4 | 1000 | 14.7 | **34.5** | 94.4 | 0.7 | 0.522 | 14 | 177 | 190 | -14.0 | 80.5 | 14 | 149 | 163 |
| 5 | 1000 | 18.9 | **41.5** | 98.4 | 0.7 | 0.567 | 16 | 196 | 212 | -12.5 | 85.9 | 16 | 165 | 181 |
| 6 | 1000 | 23.0 | **48.5** | 102.2 | 0.7 | 0.567 | 18 | 184 | 201 | -11.0 | 91.1 | 18 | 156 | 173 |
| 7 | 1000 | 27.2 | **55.5** | 105.9 | 0.7 | 0.567 | 20 | 159 | 179 | -9.6 | 96.3 | 20 | 136 | 156 |
| 8 | 1000 | 31.4 | **62.5** | 109.5 | 0.7 | 0.567 | 22 | 138 | 160 | -8.1 | 101.4 | 22 | 119 | 141 |
| 9 | 1000 | 35.6 | **69.5** | 112.9 | 0.7 | 0.567 | 24 | 119 | 143 | -6.6 | 106.3 | 24 | 104 | 128 |
| 10 | 1000 | 39.8 | **76.6** | 116.3 | 0.7 | 0.567 | 26 | 103 | 129 | -5.1 | 111.1 | 26 | 92 | 117 |
| 11 | 1000 | 44.0 | **83.6** | 119.5 | 0.7 | 0.567 | 28 | 88 | 116 | -3.7 | 115.9 | 28 | 80 | 108 |
| 12 | 1000 | 48.2 | **90.6** | 122.7 | 0.7 | 0.567 | 30 | 75 | 104 | -2.2 | 120.5 | 30 | 70 | 100 |
| 13 | 1000 | 52.4 | **97.6** | 125.9 | 0.7 | 0.567 | 32 | 63 | 94 | -0.7 | 125.1 | 32 | 61 | 93 |
| Total | 13000 | | | | | | 281 | 1694 | 1976 | | | 274 | 1465 | 1739 |

The same exercise may be carried out for the embankment with drains, which reached 1850mm total settlement by Day 470 and U = 82%. This leads to a prediction of $\Delta H_{NC}$ = 1666mm based on the measured settlement, which is more than 200mm greater than the calculated value given in Table 1. However the resulting total is 1666 + 274 + 210 = 2150mm, which is very close to the prediction given by the Asaoka method on Figure 7.

Although not a main aim of this paper and the analyses presented, this does lead to the suggestion that the embankment with drains, although settling more rapidly, is also likely to settle more than the control embankment. This seems to match well with the end-of-trial settlement behaviour summarised in the first paragraph of this section, indicating that disturbance caused by installation of the drains may have "damaged" the clay structure, resulting in greater total settlement.

The profile of preconsolidation pressure values used in Table 1 was taken as a mean regression from 56 oedometer consolidation tests. This test method is discussed in the next section, while the aim of the remainder of this paper is to investigate the effect of different profiles of $p'_c$ based on alternative methods of test or interpretation. However to get a general idea of sensitivity, the calculations in Table 1 have been repeated using profiles of $p'_c$ both 10 kPa higher and 10 kPa lower than the mean regression. These profiles are also shown on Figure 8 and define quite well upper and lower bounds to the measured data. The calculated consolidation settlement is given in Table 2.

Table 2 Consolidation settlement (mm) for $p'_c$ ±10 kPa tolerance

| **Determination of $p'_c$** | **$\Delta H_{OC}$** | **$\Delta H_{NC}$** | **$\Delta H_C$** |
|---|---|---|---|
| Oedometer mean +10 kPa | 359 | 1043 | 1402 |
| Oedometer mean | 274 | 1465 | 1739 |
| Oedometer mean −10 kPa | 188 | 2005 | 2193 |

This gives the interesting outcome that varying $p'_c$ by ± 10 kPa results in a range of calculated total consolidation settlement from 1402mm to 2193mm, a difference of about 800mm. An error of 10 kPa in assessing $p'_c$ might be considered quite small, however the resulting error in the calculated settlement is large. This immediately confirms the observations made in the opening paragraph of the introduction to this paper, namely the importance of accurate determination of the preconsolidation pressure, and the remaining sections of the paper will investigate this in more detail.

## 3. ASSESSMENT OF PRECONSOLIDATION PRESSURE

### 3.1 The oedometer consolidation test

The test method normally used to assess the preconsolidation pressure in the laboratory is the oedometer consolidation test. However in order to carry out such tests it is first necessary to obtain soil samples in order to prepare the required tests specimens. These samples should be of the highest possible quality. Minimising sample disturbance was a specific aim of the equipment and procedures set up and used by CSL, as described by Nicholls (1990). Therefore when assessing data from consolidation tests, the entire procedure should be taken into account, including the method of sampling, as well as the techniques used to transport and protect the samples before they are finally extruded in the laboratory to obtain test specimens.

Undisturbed samples of the marine clay were taken using 75mm diameter piston samplers, with stainless steel sample tubes, of which the cutting edge and entrance diameter were carefully prepared and adjusted based on the quality of the samples obtained.

After sampling, the tubes were immediately sealed with wax and rubber end caps, and then transported to the laboratory. To make allowance for the long distances from most sites to the laboratory in Bangi, special transportation equipment was developed, including foam lined boxes to hold the sample tubes, as well as vehicles with a purpose built cradle to hold the sample boxes, such that they "floated" during transportation in order to minimise sudden shocks and jarring. The boxes and cradle may be seen on Plate 2.


Plate 2 Sample protection and transportation as practised by CSL

Figure 9 shows a typical oedometer consolidation test result obtained from a specimen taken from the Juru site at a depth of 5.5m. This type of test is referred to as a maintain load (ML) test. The test procedure and method of presentation used are the "standard" method, in which the load increment ratio is 1.0, in other words each test pressure is double the previous pressure, so loads might be 6.25, 12.5, 25, 50, 100 kPa, etc. Furthermore, the horizontal pressure axis used for the graphical result is logarithmic. This results in the plotted data points being uniformly spaced in the horizontal direction, and the upper graph is referred to as the e-log p graph.

With regards to sample quality, Mesri et al (1994) describe a method of assessing disturbance in the oedometer test, referred to as Sample Quality Designation or SQD, ranging from A to E, and based on the strain in the sample when the pressure reaches the effective overburden pressure, $p'_i$. Table 3 defines SQD, and lines have been added to the upper graph in Figure 9, showing the SQD categories for that particular test, based on $e_0$. At 5.5m depth, $p'_i$ = 23 kPa at which point vertical strain is about 1%, so that this test specimen is on the boundary between SQD = A and B, therefore of a very high quality.

Table 3 Sample Quality Designation, SQD (Mesri & Yong, 1975)

| SQD | Strain at p′ | Description |
|---|---|---|
| A | 0 - 1% | Desirable |
| B | 1 - 2% | Desirable |
| C | 2 - 4% | Borderline |
| D | 4 - 10% | Unacceptable |
| E | > 10% | Unacceptable |

The lower part of Figure 9 shows various derived parameters, the coefficient of consolidation $C_v$, as well as the compressibility parameters $m_v$ and $C_c$. This method of presentation is very helpful, because it shows how these parameters change with pressure, especially as the preconsolidation pressure is passed. In this case the value of $p'_c$ was assessed as 46 kPa by the laboratory technician, which is highlighted on the overall data profile in Figure 2. The chosen value of $C_c$ is taken from the first full pressure cycle after $p'_c$, in order to represent NC behaviour.

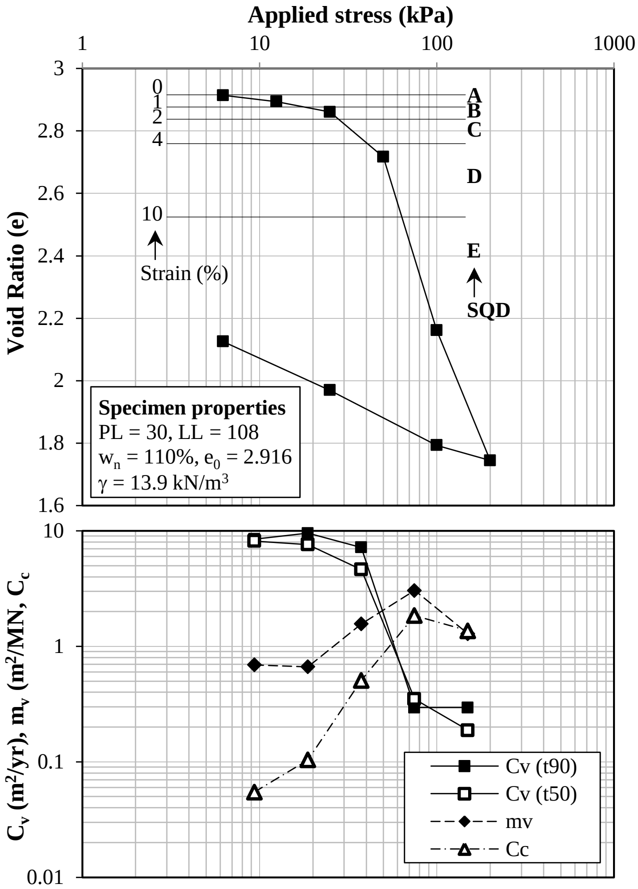



Figure 9 Oedometer consolidation test on specimen from 5.5m

The oedometer tests which formed part of the site investigation for the Juru trial embankment site were initially carried out using the standard procedure described above. As values of $p'_c$ became available they were plotted on a profile, shown as the solid square symbols on Figure 10. It soon became clear that there was a discontinuity in the data profile, between 5m and 8m depth, which did not seem to match other data profiles, and it occurred at a pressure of 50 kPa.

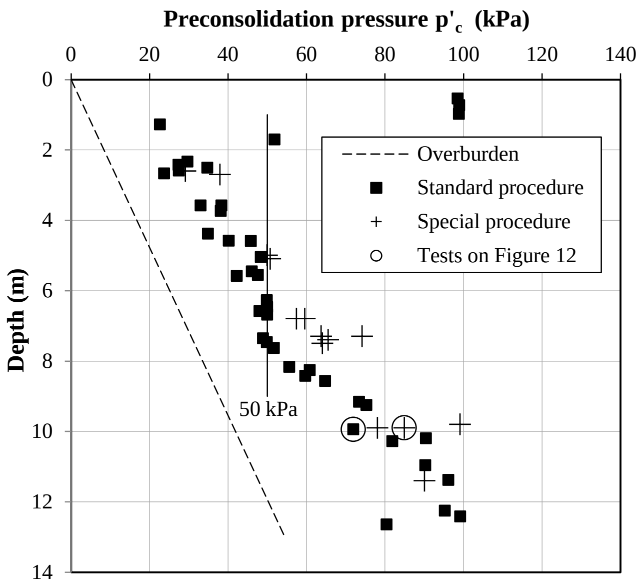



Figure 10 Profile of preconsolidation pressure at Juru based on oedometer consolidation tests

The reason for this discontinuity became clear on discussion with the CSL laboratory technicians who carried out and reported the results of the oedometer consolidation tests. They used the normal Casagrande construction to find $p'_c$, which worked well if it was close to or just less than a pressure stage. However if $p'_c$ was a bit higher than a pressure stage, then the technicians would tend to report it as the same as the pressure stage, because the Casagrande construction could not discern this small difference.

This issue became known as the "50 kPa problem", and was particularly clear in the Holocene upper marine clay, where it might affect parameters over the 5m to 8m depth range. With reference to Table 1, this is the depth range where the contribution to settlement is likely to be largest. The 50 kPa problem is described in more detail by Ho and Dobie (1990). In order to mitigate this effect, an alternative loading procedure was developed, using small uniform increments (5 kPa or 10 kPa) up until $p'_c$ was passed. This required that special sets of weights were made, because the standard weights are arranged to provide the load increment ratio of 1.0. It was also necessary to develop a technique for observing the settlement behaviour in order to decide when to add the next weight. It was found that this was best done using the root-time plot, and examples of the root-time plots for a test on a specimen from around 10m depth are shown on Figure 11. One drawback of this procedure was that the early part of the test needed constant supervision by the technician, however by observing these results, it became very clear when $p'_c$ was reached and passed.

In the test shown in Figure 11, the load stages from 10 to 70 kPa all behaved much the same, with the movement of the settlement dial gauge needle quickly slowing to a near halt, and being complete in about 10 minutes. At 80 kPa the first slight change in behaviour was seen, then at 90 kPa the needle on the settlement dial gauge continued to turn as $p'_c$ was passed. An observer could "see" $p'_c$. The value of $p'_c$ was assessed as 85 kPa. The result in terms of void ratio versus applied stress is shown in Figure 12, however in this case with stress plotted on a linear axis.

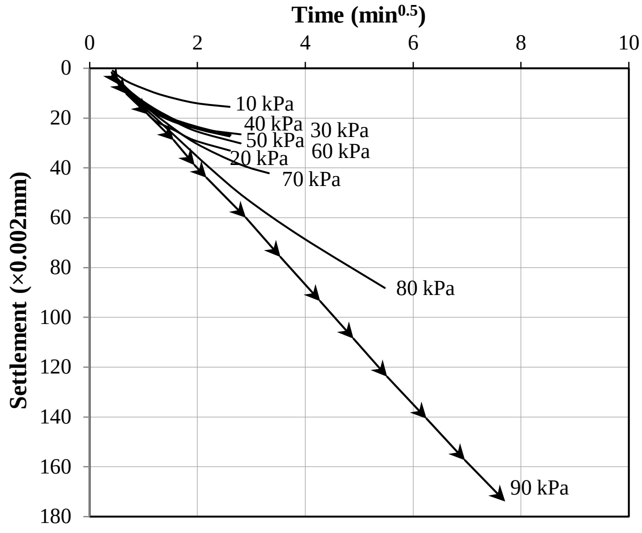



Figure 11 Settlement versus root-time for the special oedometer test procedure on a sample from 10m depth

Figure 12 includes the lines indicating SQD for the test carried out following the special procedure, and with $p'_i$ = 42 kPa, it can be seen that SQD is within Category B. The applied pressure has been plotted on a linear axis with reference to discussion by Wesley (2010), who emphasises issues which may arise when the "traditional" logarithmic scale is used, in particular reporting a preconsolidation or yield pressure which does not actually exist. In the case of this Holocene marine clay, it does exist, but looking at the data from the standard test, $p'_c$ could be almost anywhere between 50 and 90 kPa, and the technician actually assessed it as 72 kPa. However using the special procedure leaves no doubt that $p'_c$ is between 80 and 90 kPa, and it was taken as 85 kPa.

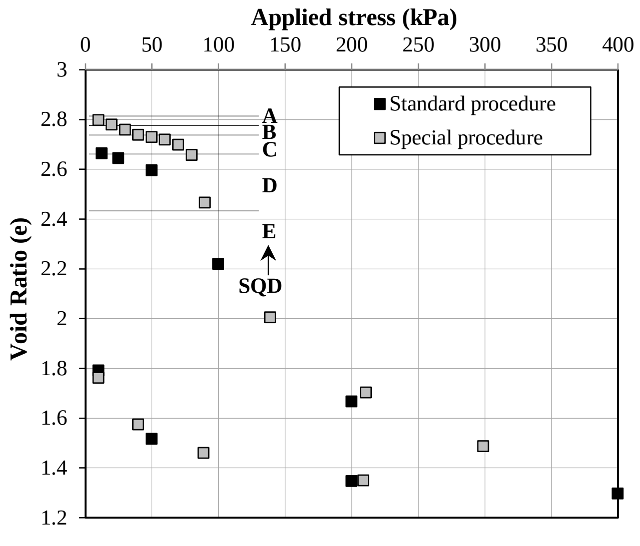



Figure 12 Void ratio versus applied stress plotted on a linear scale from oedometer tests on samples from 10m depth

A number of special procedure tests were carried out on specimens taken from the Juru site, and the resulting values of $p'_c$ are plotted on the Figure 10 profile as crosses. From this it is clear that the apparent discontinuity between 5m and 8m depth was an anomaly, caused by the test procedure and method of interpretation, and was not a soil property. Due to the large number of oedometer tests carried out as part of the investigation of the Juru site, even without the special procedure tests, the 50 kPa problem does not result in a significant error in the calculation of consolidation settlement. However if far fewer tests had been carried out, it might well be of major significance, taking into account the observation in the previous section, namely that a ±10 kPa error in $p'_c$ could result in a major error in calculated settlement.

### 3.2 Assessment based on undrained shear strength

Undrained shear strength of the Holocene marine clay at the Juru site was measured using the in-situ field vane (Geonor vane borer) as described by Dobie (1990), as well as by carrying out unconsolidated undrained (UU) triaxial tests on specimens taken from the piston samples. Profiles of both sets of data are included on Figure 2. There are five profiles of vane tests which were carried out at 0.5m vertical spacing, resulting in 97 measurements (termed $s_{uv}$). There are 18 results from laboratory UU triaxial tests. The scatter in data is significant, but this is consistent with the behaviour seen in the piezocone tests, as discussed in Section 2.1.

A common method of normalising undrained shear strength is against the vertical effective stress at the depth of testing or sampling (denoted as $p'_i$ in this paper). This is shown on Figure 13 following the method described by Mesri et al (1994). The undrained shear strength is based on the field vane tests, corrected by the Bjerrum correction factor, $\mu_B$. For the Juru site, with reference to Figure 2, PI is consistent with depth and just over 80, so that $\mu_B = 0.65$ has been used to correct $s_{uv}$ to give values suitable for embankment design.

The y-axis of Figure 13 is $\mu_B s_{uv}/p'_i$ and the x-axis is the overconsolidation ratio or OCR, namely $p'_c/p'_i$. Data is taken from the mean profile of the oedometer test results to give $p'_c$ and the mean of the field vane test results to give $s_{uv}$. Values have been calculated at 1m intervals to match the layers used in Table 2. Based on this, the range of OCR is 3.4 near the surface reducing to 1.95 at the base of the layer, and $\mu_B s_{uv}/p'_i$ varies from 0.73 to 0.44. However based on the method of presentation shown on Figure 13, the data falls very close to the inclined line given by $m_0 = 1.0$, intersecting the x-axis at 0.22. This leads to the elegant result that:

$$\mu_B s_{uv} / p'_c = s_u / p'_c = 0.22 \tag{4}$$

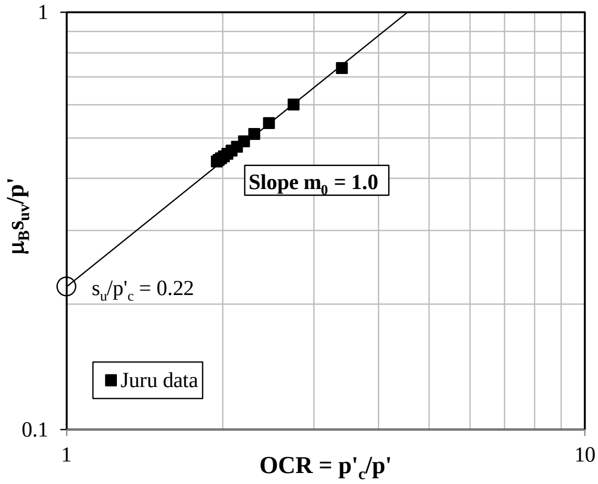



Figure 13 Independence of $s_{uv}/p'_c$ from OCR, based on oedometer and corrected vane test data for the Juru site, after Mesri et al (1994)

Terzaghi et al (1996) present extensive data relevant to the relationship in Equation 4. In particular they demonstrate that, based on a large database, while the normalised undrained shear strength from vane tests ($s_{uv}/p'_c$) varies with PI, the Bjerrum correction factor $\mu_B$ also varies with PI by a similar but opposite trend. So when combined, the resulting relationship is as given in Equation 4, where the value 0.22 is independent of PI, and valid for a wide range of soil types.

The 0.22 Mesri factor in Equation 4 is extremely useful in soft soil engineering, and the results on Figure 13 show that the Holocene marine clay at Juru fits very closely with this general result. It has been applied to the undrained shear strength profiles from Juru, assuming three possible cases: (1) $s_{uv}$ directly as measured (ie. assuming that the Bjerrum correction has not been applied), then (2) $s_{uv}$ corrected, and finally (3) $s_u$ taken directly from the UU triaxial tests. These three profiles are shown on Figure 14.

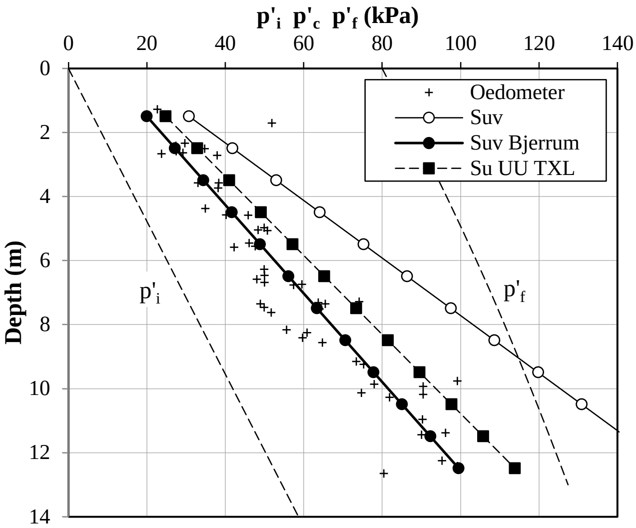



Figure 14 Profiles of $p'_c$ derived from undrained shear strength following three different assessments

The consolidation settlement calculations in Table 1 have been repeated using the three profiles of $p'_c$ shown on Figure 14, and the results are given in Table 4. It is clear from this that omitting the Bjerrum correction results in a major error, while using the laboratory UU data is less reliable than using the corrected field vane results.

Table 4 Consolidation settlement (mm) with $p'_c$ based on $s_u$

| **Determination of $p'_c$** | **$\Delta H_{OC}$** | **$\Delta H_{NC}$** | **$\Delta H_C$** |
|---|---|---|---|
| Based on $s_{uv}$ | 506 | 504 | 1010 |
| Based on corrected $s_{uv}$ ($\times\mu_B$) | 281 | 1438 | 1719 |
| Based on laboratory UU triaxial | 362 | 1063 | 1425 |

### 3.3 Assessment based on piezocone test results

A large number of piezocone tests (or CPTu) were carried out at the Juru trial embankment site. Figure 3 shows a typical result, and this profile of measured test parameters is used in the analysis which follows. The relatively "spikey" nature of each of the plots is discussed in Section 2.1, and it is considered that this is caused by the presence of discrete organic debris, such as reeds and roots, which were frequently observed in samples. In the analysis which follows, no attempt is made to smooth out these profiles.

Interpretation of the data from piezocone tests in the Holocene marine clay is discussed in detail by Dobie & Wong (1990), including assessment of the degree of overconsolidation, therefore $p'_c$. There are several published relationships for assessing $p'_c$ from piezocone results, but here two are considered, which are based on parameters $A_q$ and $B_q$. These are defined as follows:

$$A_q = \frac{q_T - p_i}{p_i - u_0} \quad \& \quad B_q = \frac{u - u_0}{q_T - p_i} \tag{5a, 5b}$$

Where $q_T$ denotes the corrected cone resistance (see Figure 3 and Section 2.1), $p_i$ denotes the total vertical stress, u denotes the instantaneous pore pressure measured during the piezocone test and $u_0$ denotes the hydrostatic pore pressure. Profiles of $A_q$ and $B_q$ derived from the piezocone profiles on Figure 3 are shown on Figure 15. It is not surprising that these profiles are also "spikey", taking into account the nature of the original profiles, and comments above.

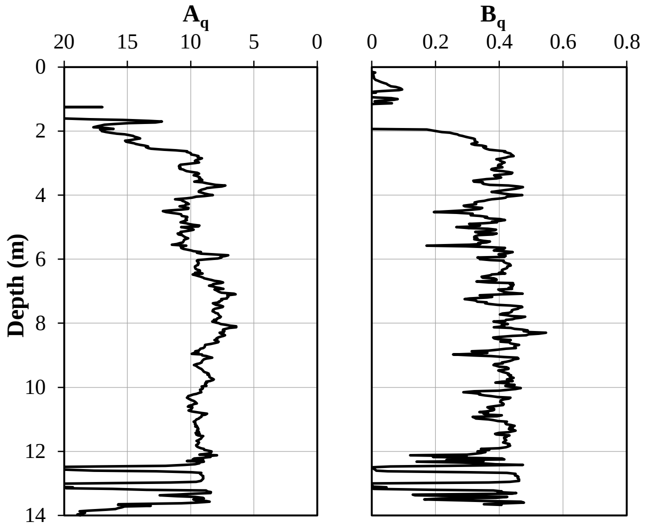



Figure 15 Profiles of $A_q$ and $B_q$ derived from the piezocone test shown on Figure 3

Wroth (1988) terms the parameter $A_q$ as the normalised cone resistance and $B_q$ as the water pressure ratio, and relates them to OCR as follows:

$$A_q = N_{kT}(s_{uv}/p'_i)_{nc}OCR^m \tag{6}$$

Where $N_{kT}$ denotes the cone factor, which is 13.1 for the Holocene marine clay at Juru when $s_u$ is based on vane testing, as derived by Dobie & Wong (1990). The term $(s_{uv}/p'_i)_{nc}$ is the undrained shear strength from vane testing normalised to the effective vertical stress for the NC condition, and may be taken from the relationship given by Skempton (1957) as equal to (0.11 + 0.0037 × PI), so 0.406 for the Juru marine clay. The power m is taken as 0.8. This gives:

$$A_q = 5.32 \times OCR^{0.8} \tag{7}$$

$B_q$ is related empirically to OCR as per Figure 16. The actual OCR data for the Holocene marine clay at the Juru site is also shown on Figure 16 plotted against $B_q$, where OCR is based on the values of $p'_i$ and $p'_c$ given in Table 1, 3rd and 4th columns. It can be seen that the data is not that close to the empirical relationship, with all data falling below the line. Indeed the prediction of settlement based on $B_q$ is not very good, as discussed below.

Based on these relationships between OCR and the parameters $A_q$ and $B_q$, profiles of $p'_c$ have been calculated and are presented on Figure 17. The values of the piezocone test parameters required to do this ($q_T$ and u) have been taken as means over the 1m intervals as used in the settlement calculations given in Table 1, and no attempt has been made to average the data beyond this. Therefore the resulting profiles of $p'_c$ are not smooth lines as per Figure 14, but reflect the slightly erratic nature of the plots of $A_q$ and $B_q$ shown on Figure 15.

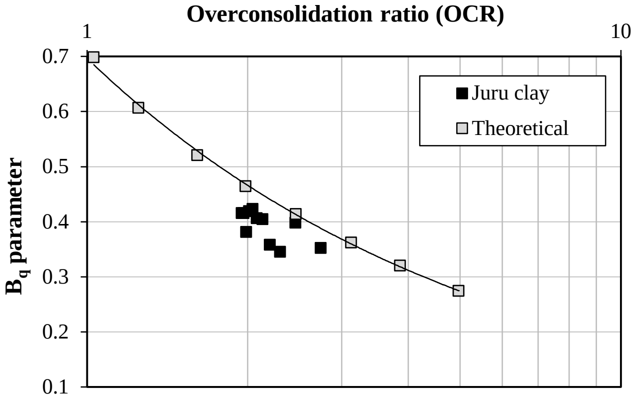



Figure 16 Relationship between OCR and $B_q$ according to Wroth (1988) including data from the Juru site

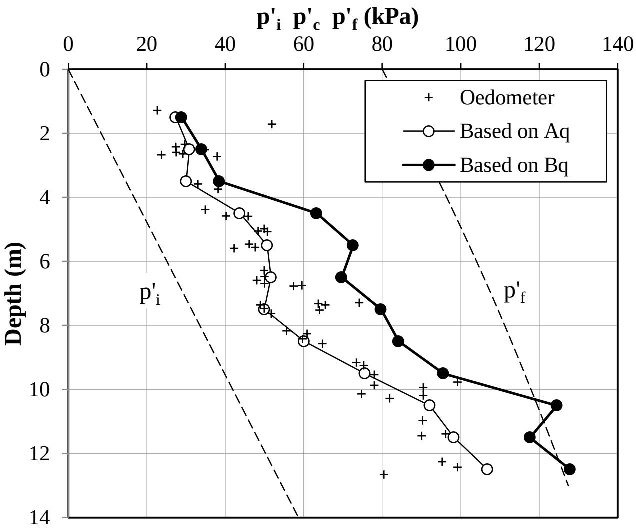



Figure 17 Profiles of $p'_c$ based on piezocone parameters $A_q$ and $B_q$

It can be seen that the profile of $p'_c$ on Figure 17 which has been derived from the normalised cone resistance ($A_q$) fits within the scatter of the values measured directly in the oedometer tests. However the profile derived from the water pressure ratio ($B_q$) is well above the oedometer data. Consolidation settlement has been assessed using the approach in Table 1, but with $p'_c$ given by the plotted data points on Figure 17. The resulting settlement predictions are given in Table 5, where the predicted total consolidation settlement based on $A_q$ is very close to the Table 1 calculation, but the value based on $B_q$ is 500mm less.

Table 5 Consolidation settlement (mm) with $p'_c$ based on piezocone test data comparing the use of $A_q$ and $B_q$

| **Determination of $p'_c$** | **$\Delta H_{OC}$** | **$\Delta H_{NC}$** | **$\Delta H_C$** |
|---|---|---|---|
| Based on $A_q$ | 281 | 1451 | 1732 |
| Based on $B_q$ | 429 | 803 | 1232 |

This is happening because $B_q$ is over-predicting $p'_c$ by quite a large amount. However looking at Figure 16, this is not surprising, with all actual data points well below the theoretical line. This suggests that establishing site-specific determinations of the $B_q$ relationship would be wise, however in terms of this paper, settlement predictions are based on taking the data and published relationships at face value, and this indicates that using $B_q$ may not be reliable.

One possible explanation could be the unusual properties of the Juru marine clay (high silica content) as described in Section 2.1.

When using data from piezocone tests for the interpretations and predictions for very soft clays as made above, it is very important to be aware that the equipment is being used at the very lowest limit of its working range and sensitivity, especially with regards to the measurement of cone resistance and local friction. This issue is further exacerbated by the fact that the $A_q$ and $B_q$ definitions both consist of two subtractions and a division. It might be assumed that total vertical stress and initial (hydrostatic) water pressure are well known, but this is not always the case.

However of far greater significance to the calculated values of $A_q$ and $B_q$ is the accuracy of the cone resistance measurement, which is used in both. Mohd Pauzi et al (1990) and Dobie (2014) describe a case where three adjacent cone resistance profiles show a significant lateral shift from each other, as shown on Figure 18. As mentioned above, a large number of piezocone tests were carried out as part of the investigation of the Juru site, but when some of the early tests were compared and superimposed, it was noticed that, although of a similar form, they could be displaced laterally from each other, as seen on Figure 18.

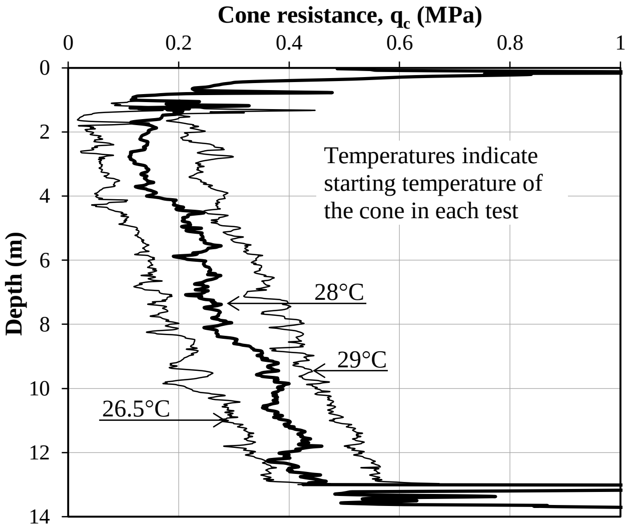



Figure 18 Profiles of $q_c$ measured at the Juru site affected by the preparation temperature of the cone

Further investigation of this behaviour identified the cause of this shift to be the initial preparation temperature of the cone, and in the case of the profiles on Figure 18, these temperatures were as shown.

Mohd Pauzi et al (1990) describe this investigation in detail as well as cone preparation procedures using a temperature controlled water bath to minimise the effects. It was found that for the typical Holocene marine clay at the Juru site, the initial cone temperature should be 28°C to minimise any possible errors due to temperature changes. Therefore on Figure 18, the middle profile is the representative profile of $q_c$ (much the same as that shown on Figure 3), and the other two have been affected by preparing the cone either too hot or too cold.

The normalised cone resistance $A_q$ has been determined from all three profiles on Figure 18, from which OCR and hence $p'_c$ have been derived using the relationship given by Equation 7. The resulting profiles of $p'_c$ are plotted on Figure 19, which also includes the data from the oedometer tests. The $p'_c$ values based on the piezocone prepared at 28°C fall well within the scatter of the oedometer data. However the profiles derived from the piezocone tests prepared either too hot or too cold are clearly well outside the oedometer data. In particular the 26.5°C plots suggest that the deposit is normally consolidated, which is not the case (see Figure 6).

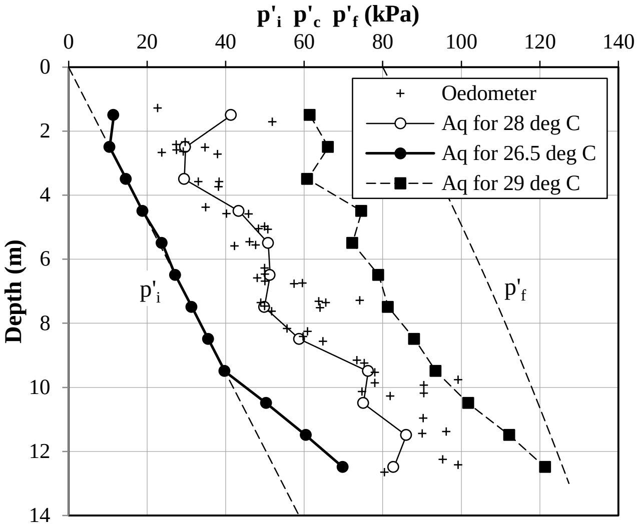



Figure 19 Profiles of $p'_c$ derived from piezocone test parameter $A_q$ but investigating the effect of cone preparation temperature

The resulting settlement predictions are given in Table 6, where the predicted total consolidation settlement based on $A_q$ determined using the piezocone prepared at 28°C is very close to the Table 1 calculation, but the other two are wildly inaccurate.

Table 6 Consolidation settlement (mm) for $p'_c$ based on piezocone data using $A_q$ with cones prepared at different temperatures

| Determination of $p'_c$ | $\Delta H_{OC}$ | $\Delta H_{NC}$ | $\Delta H_C$ |
|---|---|---|---|
| Based on $A_q$ with cone at 28°C | 251 | 1544 | 1795 |
| Based on $A_q$ with cone at 26.5°C | 49 | 3002 | 3051 |
| Based on $A_q$ with cone at 29°C | 490 | 513 | 1003 |

## 4. SUMMARY AND DISCUSSION

Section 2.0 of this paper outlines the measured behaviour of the Juru trial embankments, which indicates that the eventual consolidation settlement of the control embankment would have been about 1750mm. Unfortunately this condition was never reached, and the consolidation settlement of the embankment with drains, although more advanced at the termination of the trial is likely to have been about 200mm greater than the control, possibly due to disturbance of the very soft Holocene marine clay caused by drain installation. Therefore the assessments of consolidation settlement provided in Section 3.0 are only relevant to the control embankment. A simple calculation taking into account a range of ±10 kPa in the preconsolidation pressure profile based on oedometer tests, results in consolidation settlement from 1400mm to 2200mm, making it clear that an accurate and representative assessment of preconsolidation pressure is of the utmost importance.

Section 3.0 describes methods of assessing $p'_c$ based on the oedometer, undrained shear strength and parameters derived from piezocone test data. In each case potential errors in the test method or interpretation are considered, with the predicted consolidation settlement given in Tables 4, 5 and 6, all summarised on Figure 20.

Importantly, Figure 20 indicates that all methods, with proper execution and interpretation of $p'_c$, are capable of giving much the same result as the trial embankment itself (EMB). However Figure 20 also provides a warning that poor test procedure or incorrect interpretation may result in major errors in the calculated consolidation settlement. In particular it can be seen that $p'_c$ based on undrained shear strength determined from laboratory UU triaxial test results (UU TXL) or uncorrected vane shear strength data (Suv) may result in predicted settlements which are far too small. In the case of using parameters derived from piezocone test data, the normalised cone resistance ($A_q$) appears to offer a better prediction than the water pressure ratio ($B_q$), although the latter could be improved by site-specific correlation. However a far greater issue relates to using an adequate cone preparation procedure, in terms of initial cone temperature, in order to minimise the potential for a lateral shift in the cone resistance profile (as per Figure 18). This can be seen very clearly on Figure 20, where the extreme results represent only a 2.5°C variation in initial cone temperature, but the range of calculated consolidation settlement is from 1000 to 3050mm.

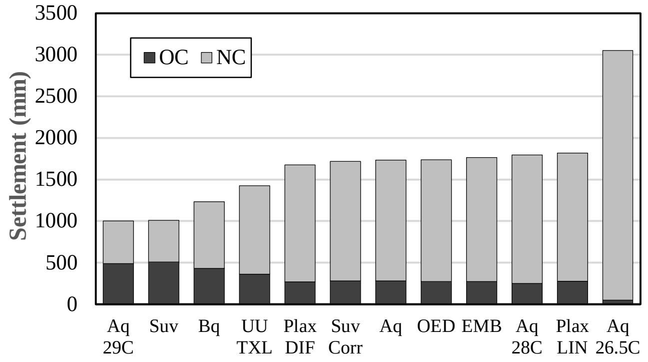



Figure 20 Range of predicted consolidation settlement based on various methods used to assess the preconsolidation pressure

A final discussion point is relevant to the increasingly popular use of numerical methods to carry out settlement calculations of the type summarised in Table 1. The finite element program Plaxis® provides two methods of defining $p'_c$, (Plaxis, 2015) as follows:

$$p'_c / p'_i = \text{OCR} = \text{constant OR } p'_c - p'_i = \text{constant} \qquad (8)$$

Therefore assuming that the Holocene marine clay is modelled as a single layer, and basing the profiles on the centroid of the oedometer data, the two possible distributions of $p'_c$ are as shown on Figure 21. Neither profile is very close to the mean profile of $p'_c$ from the oedometer, although the line of open symbols based on constant OCR appears to fit better visually.

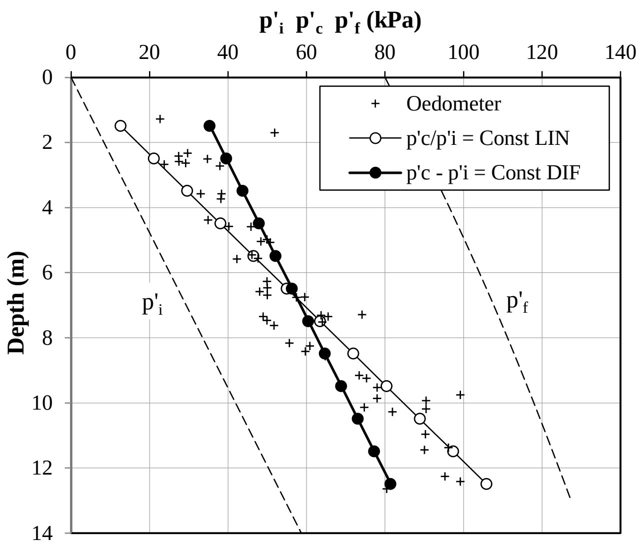



Figure 21 Profiles of $p'_c$ which may be defined using Plaxis®

The calculated consolidation settlements based on the procedure summarised in Table 1, combined with $p'_c$ taken from the two profiles in Figure 21, are given in Table 7. These values are also represented on Figure 20 (Plax DIF and Plax LIN). Here it can be seen that the two values of $\Delta H_C$ straddle the actual embankment settlement, as well as those predicted based on $p'_c$ assessed from corrected vane shear data or the normalised cone resistance from well executed piezocone tests. The errors are not major, but it would seem wise to use the constant OCR approach, being both on the conservative side and closer to the measured result. An alternative approach would require that the clay deposit is sub-divided into a number of thinner layers.

Table 7 Consolidation settlement (mm) with $p'_c$ based on the two models provided in the FEM program Plaxis®

| **Determination of $p'_c$** | **$\Delta H_{OC}$** | **$\Delta H_{NC}$** | **$\Delta H_C$** |
|---|---|---|---|
| Based on $p'_c/p'_i$ = Constant DIF | 269 | 1407 | 1676 |
| Based on $p'_c$ - $p'_i$ = Constant LIN | 276 | 1542 | 1818 |

## 5. CONCLUSIONS

The Juru trial embankment provides an excellent opportunity to examine in detail the calculation and prediction of an important performance feature of any embankment built over a compressible Holocene marine clay, namely its total settlement. The trial embankment should provide the definitive value of total settlement, but in the case of Juru this was not possible due to the short duration of measured performance data. However a reasonable assessment can be made indicating a likely consolidation settlement of about 1750mm.

In order to predict this settlement by calculation based on typical measured soil properties, of greatest importance is the preconsolidation pressure ($p'_c$), being the vertical yield stress at which settlement behaviour changes from overconsolidated to normally consolidated. Based on the extensive site investigation carried out at the Juru trial embankment site, assessment of $p'_c$ has been examined based on three approaches: directly from oedometer tests, based on undrained shear strength and based on parameters derived from piezocone data. Important observations are as follows:

- For any method which relies on laboratory testing, it is vital that equipment and methods are used which minimise sample disturbance, and this should include the method used to transport the samples from the site to the laboratory.
- The maintained load oedometer consolidation test provides the most common and direct method of predicting $p'_c$, however using the "standard" load increment ratio of 1.0 may give rise to non-representative values, especially when $p'_c$ is in the range from just greater than 50 kPa to just less than 100 kPa. This issue can be improved by using an alternative loading procedure consisting of uniform pressure steps up to the point where $p'_c$ is passed.
- The Mesri relationship that $p'_c = s_u/0.22$ for a wide range of soft clays gives a completely independent method of establishing the preconsolidation pressure based on undrained shear strength. Experience from Juru indicates that $s_{uv}$ from the in-situ vane test provides a reliable assessment but it is very important that the measured values are corrected using Bjerrum's correction factor, for which PI data are required. The direct use of $s_u$ from laboratory UU triaxial tests was less reliable for applying this method, despite the very high quality of samples recovered from the site.
- Two parameters derived from piezocone tests were used to assess the preconsolidation pressure: the normalised cone resistance ($A_q$) and the water pressure ratio ($B_q$). In the Juru case, $A_q$ gave far better and more consistent results. The use of $B_q$ could be improved based on site-specific correlation, but this would require that high quality oedometer test data were also available.

However the important reminder when using these techniques is that the piezocone is operating at the very lowest part of its operating range, and attention to cone preparation, in particular the initial cone temperature, is vital in order to obtain reliable results.

- An important question based on the points above might be: is there

a preferred approach? The answer to this is that it is wisest to use a number of approaches, and come to a balanced decision. However it is vital that all parts of the procedures used, both on site and in the laboratory, are carried out diligently to the highest standards, with awareness of the possible effects of inadequate procedures.

- The discussion in the previous section also points to a possible issue with regards to techniques available in FEM programs used to model the profile of preconsolidation pressure. In the case of the Juru deposit neither of the available models in Plaxis® matches the measured data, but using constant OCR appears to provide the better option. An alternative approach would be to divide the deposit into thinner layers in order to match the model to the data as closely as possible.

The final conclusion is to stress that all of the geotechnical techniques and test methods described in this paper may be considered as appropriate for commercial site investigation, although the intensity of testing used at Juru would be a luxury for a typical highway embankment investigation. Probably the only equipment not readily available in commercial soil laboratories would be the weights required to carry out the special consolidation test procedure, but these can easily be manufactured at a small cost. Beyond this it is a question of training and supervision of operators and technicians, which should include an appreciation of the importance of their work. The additional effort and cost to turn a mediocre test result into one of high quality is not that great. However the return in terms of confidence in using the data is major.

## 6. ACKNOWLEDGEMENTS

The measured data presented in Figures 2, 3, 9, 11, 12 and 18 were determined from laboratory and field testing carried out by the technicians of CSL. The Juru trial embankment data on Figure 5 was assembled from the extensive monitoring programme conducted by the CSL monitoring team. The author was fortunate to spend 1989 and 1990 working at CSL, and witness the young graduates selected to become technicians develop into accomplished professionals, whose diligent and capable work provided the data used to create this paper.

## 7. REFERENCES

Dobie, M. J. D. (1990) "In situ testing Methods for Soft Ground". Proceedings Seminar on Geotechnical Aspects of the North-South Expressway, Kuala Lumpur, Malaysia, pp9-24.

Dobie, M. J. D. and Wong, J. T. F. (1990) "Piezocone Testing: Interpretation in Malaysian Alluvial Clays". Proceedings Seminar on Geotechnical Aspects of the North-South Expressway, Kuala Lumpur, Malaysia, pp75-86.

Dobie, M. J. D. (2014) "Site Investigation of the Holocene Marine Clay of Southeast Asia Using the CPT". Proceedings Southeast Asia Conference on Soft Soils Engineering and Ground Improvement, SoftSoils 2014, Bandung, Indonesia, ppE1-1 to E1-11.

Gray, H. (1936) "Stress Distribution in Elastic Solids". Proceedings 1st International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Massachusetts, USA, Vol. 2, p157.

Ho, H. S. and Dobie, M. J. D. (1990) "Consolidation Testing: Alternative Loading Procedure for Soft Clays". Proceedings Seminar on Geotechnical Aspects of the North-South Expressway, Kuala Lumpur, Malaysia, pp135-140.

Lerouil, S., Magnan, J. P. and Tavenas, F. (1985) "Remblais sur Argiles Molles". LCPC et Techniques et Documentation (Lavoisier), Paris, France.

Mesri, G. (2007) "Strength and Compressibility of Soft Soils in Settlement and Stability of Structures for Highways and Motorways". Lectures given at Griffith University, Gold Coast, Australia.

Mesri, G., Lo, D. O. K. and Feng, T. W. (1994) "Settlement of Embankments on Soft Clays". Keynote Lecture, Settlement '94, Texas A&M University, USA, Geotechnical Special Publication No. 40, Vol. 1, pp8-56.

Mohd Pauzi, I., Kassim, K. and Dobie, M. J. D. (1990) "Piezocone Testing: Effect of Temperature". Proceedings Seminar on Geotechnical Aspects of the North-South Expressway, Kuala Lumpur, Malaysia, pp47-52.

Nicholls R. A. (1990) "Sampling Techniques in Soft Ground and Residual Soils". Proceedings Seminar on Geotechnical Aspects of the North-South Expressway, Kuala Lumpur, Malaysia, pp1-8.

Nicholls R. A. and Ho, H. S. (1990) "Clay Salinity ". Proceedings Seminar on Geotechnical Aspects of the North-South Expressway, Kuala Lumpur, Malaysia, pp173-176.

Plaxis (2015). Manual for Plaxis®, Edited by Brinkgreve, R. B. J., Kumarswamy, S. and Swolfs, W. M., Delft, The Netherlands.

Raj, J. K. and Ho, H. S. (1990) "Clay Mineralogy of Holocene Marine Clays along the North-South Expressway". Proceedings Seminar on Geotechnical Aspects of the North-South Expressway, Kuala Lumpur, Malaysia, pp177-185.

Raj, J. K. and Malek, M. (1990) "Scanning Electron Microscopy of Clay Samples from the Butterworth Trial Embankment Site". Unpublished Internal Report, Kuala Lumpur, Malaysia.

Ramli, M., Nicholls, R. A., Wan Hashimi, A., Dobie, M. J. D. and Khalit, O. (1991a) " Vertical Drains Trial at Juru, Malaysia: Performance during Construction". Proceedings 9th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, Thailand, Vol. 1, pp509-514.

Ramli, M., Nicholls, R. A., Wan Hashimi, A., Dobie, M. J. D. and Khalit, O. (1991b) " Vertical Drains Trial at Juru, Malaysia: Performance for the First Year after Construction". Proceedings 9th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, Thailand, Vol. 2, discussion.

Skempton, A. W. (1957). Discussion on the Planning and Design of the new Hong Kong Airport. Proceedings ICE, No. 7, pp305-307.

Terzaghi, K., Peck, R. B. and Mesri, G. (1996) "Soil Mechanics in Engineering Practice", Third Edition. John Wiley & Sons, Inc., New York, USA.

Wan Hashimi, A., Khalit, O., How, K.T. and Chan, P. C. (1990) "Vertical Drain Embankment Trial at Sungai Juru". Proceedings Seminar on Geotechnical Aspects of the North-South Expressway, Kuala Lumpur, Malaysia, pp195-206.

Wesley, L. D. (2010) "Fundamentals of Soil Mechanics for Sedimentary and Residual Soils". John Wiley & Sons, Inc., New York, USA.

Wroth, C. P. (1988) "Penetration Testing – a more Rigorous Approach to Interpretation". Proceedings 1st International Conference on Penetration Testing (ISOPT-1), Orlando, Florida, USA, Vol. 1, pp303-311.**a**