# THE PERFORMANCE OF A PREFARICATED VERTICAL DRAINS TRIAL EMBANKMENT

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## ABSTRACT

*Two full scale trial embankments were constructed to a height of 3.65 m over the soft marine clay at Juru, Butterworth, Pulau Pinang. One of the embankments was improved using prefabricated vertical drains (PVD), and the other one without treatment was a control embankment. Monitoring instruments such as hydrostatic profile gauges, settlement gauges, inclinometers and piezometers were installed in the subsoil to observe the deformation behavior of the embankments under loadings. The hyperbolic and the Asaoka's methods were used to predict the total primary consolidation settlement. The embankment with PVD helped to accelerate the consolidation settlement as compared to the control embankment. It can be seen from the back analysis that the performance of the vertical drains is well predicted with smear effect taken into consideration using $K_L/K_S = 10$, $s = 2.5$ and $C_h = 4.5$ m$^2$/year. In addition insitu vane shear strength, the compression ratio (CR) and preconsolidation pressures of the subsoil before and after the embankment loadings were tested. It was confirmed that the insitu vane shear strength, the compression ration (CR) and the preconsolidation pressures of the subsoil after treatment with PVD had improved substantially. The hyperbolic equation was also used to predict the ultimate lateral deformation for both the treated and control embankments. The "R" squares of the linear regression showed a reasonable linearity between t/d versus t where t is time and d is the lateral deformation.*

**Keywords:** *Compression Index, Lateral Deformation, Prefabricated Vertical Drain, Primary Consolidation, Smeared Effects, Trial Embarkment*

## 1.0 INTRODUCTION

Extensive deposits of low strength and compressible soft soils are found in the western coast of Peninsular Malaysia. Their general distribution is shown in Figure 1. This low strength and compressible soft soils known as the marine clays were found in portion of the North South Expressway from Taiping to Butterworth. The length of this section of the expressway is approximately 80 km and traverses across mainly soft marine clays.

Embankment height over soft flat ground is usually controlled by the flood levels and bridge clearance when it traversed across existing roads or existing railway lines. This resulted in high embankments which can be as high as 7 to 8 meters. This gives rise to problems of instability during construction and excessive and persistent settlement subsequently.

There were reported cases where treated and untreated embankments had failed during construction on soft clays. Examples of these failures can be found in [1, 2] and in other countries in [3].

With the problems as mentioned above, some methods of soil improvement are generally required for this marine clay, and the type soil improvement methods to be adopted depend on the height of the embankments and the sub-soils properties. The purpose of the soil improvement work is to ensure the stability of these embankments and also to minimise the post construction tolerable total and differential settlement.

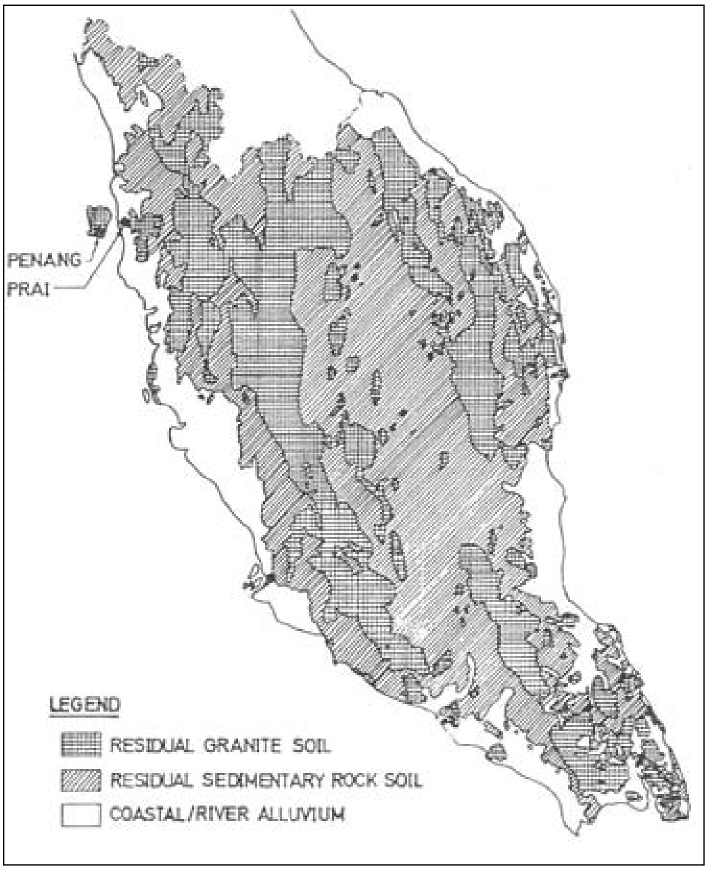


***Figure 1: General surface soil distribution***

There are many million meters of prefabricated vertical drains that had been installed in Malaysia for the treatment of marine clay and alluvial soils in road construction. In order to study the effectiveness of these prefabricated vertical drains in accelerating the consolidation settlement and also the improvement of the consolidation characteristics of the subsoils, full scale trial embankments were constructed to investigate the performance of these prefabricated vertical drains. A well known example of the full scale trial embankment was the one carried out in the Muar flat, Johor. In this trial a numbers of soil improvement methods were proposed. Prefabricated vertical drain with surcharging was one of the methods that were carried out in this full scale trial. It was reported that vertical drains helped to accelerate the consolidation process [4].

## 2.0 SUBSOIL PROPERTIES

A comprehensive site investigation was carried out at the area where the trial embankment works were conducted to determine the subsoil profile and properties of the trial site. This site investigation revealed the presence of a desiccated upper crust of about 1.5 meter thick. Beneath the upper crust is a layer of about 12.5 meters thick very soft to soft clay. The clay has been identified as part of a Holecence marine deposit formed after the last period of low sea level between 14,000 to 18,000 years ago [5]. Below the clay stratum is a layer of loose to medium dense sand of about 2 meters thick, which is underlain in turn with residual soil deposits. Figure 2 summarised the profile and properties of the soft stratum of the Juru trial site as outlined from both field and laboratory tests. The laboratory tests included consolidated isotropic undrained tests and oedometer tests. The marine clay was shown to be of high plasticity limit. Liquid limit in the range of 80 % to 120 %, plasticity index varies from 40 % to 80 % and the moisture content closed to the liquid limit.

The consolidation characteristics and other soil properties of the marine deposit for the control and treated embankments were summarised in Tables 1 and 2 respectively. The coefficient of consolidation, $C_h$, in the horizontal direction derived from Piezocone dissipation tests however, are considerably higher than those of the laboratory values. The $C_h$ values derived from Piezocone dissipation tests varied from 2.5 m²/year to 3.5 m²/year.

## 3.0 THE TRIAL EMBANKMENTS AND PREFABRICATED VERTICAL DRAINS INSTALLATION

In order to ascertain the effectiveness of prefabricated vertical drains in accelerating the consolidation settlement and in the improvement of the sub-soils properties in this section of the expressway, it was decided to carry out a full scale trial embankment treated with prefabricated vertical drains in Juru, Butterworth. The trial involved the construction of two embankments to an elevation of 3.6 meters above the existing ground level. One being a control embankment (untreated) and the other one was treated with prefabricated vertical drains. Instruments such as hydrostatic profile gauge, settlement gauges, inclinometers, piezometers and others were installed in both the treated and control embankments. Details of these

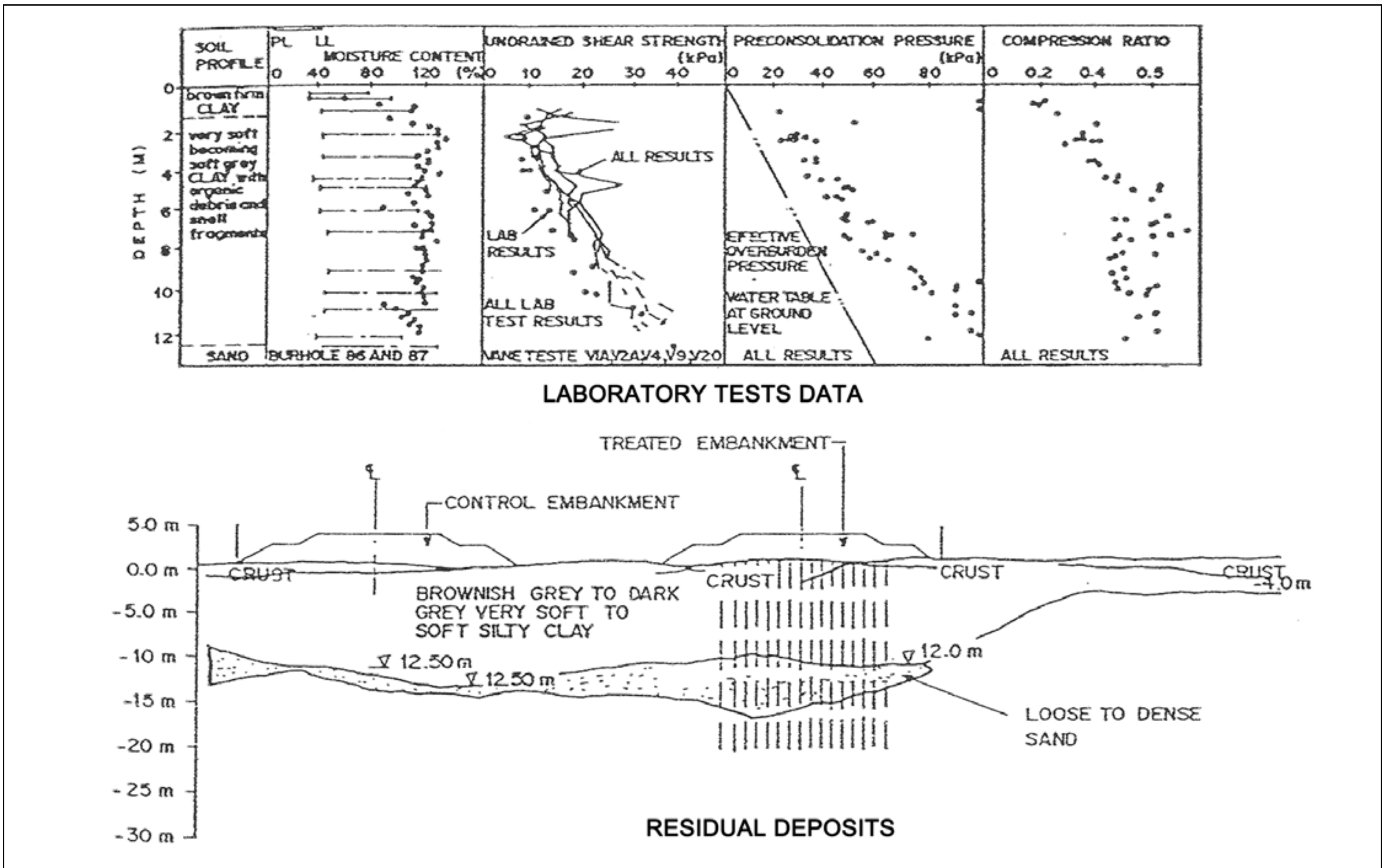


*Figure 2: General surface soil distribution*

*Table 1: Oedometer test results before soil loading*

| Control Embankment | | | | | | |
|---|---|---|---|---|---|---|
| Depth (m) | Cc | CR | eo | Pc' | $C_v$ | $C_h$ |
| 2.5 | 1.50 | 0.35 | 3.27 | 35 | 0.30 | - |
| 3.1 | 1.70 | 0.39 | 3.36 | 42 | 0.30 | 0.45 |
| 6.4 | 2.00 | 0.46 | 3.34 | 50 | 0.35 | - |
| 8.4 | 2.00 | 0.47 | 3.26 | 65 | 0.40 | - |
| 10.5 | 2.60 | 0.61 | 3.23 | 90 | 0.45 | - |
| 12.0 | 2.60 | 0.61 | 3.23 | 90 | 0.45 | - |

*Table 2: Oedometer test results before soil loading / treatment*

| Prefabricated Vertical Drain Treated Embankment | | | | | | |
|---|---|---|---|---|---|---|
| Depth (m) | Cc | CR | eo | Pc' | $C_v$ | $C_h$ |
| 2.5 | 1.20 | 0.31 | 2.86 | 30 | 0.30 | 0.6 |
| 3.1 | 1.70 | 0.41 | 3.13 | 38 | 0.26 | - |
| 6.4 | 2.80 | 0.65 | 3.31 | 50 | 0.35 | 0.6 |
| 8.4 | 2.10 | 0.52 | 3.05 | 56 | 0.35 | 0.8 |
| 10.5 | 2.00 | 0.50 | 3.02 | 75 | 0.30 | 0.6 |
| 12.0 | 2.60 | 0.65 | 2.99 | 75 | 0.40 | 0.8 |

instrumentations can be found in [6]. Figures 3 and 4 showed the instrumentation locations for these two trial embankments.

The prefabricated vertical drains spaced at 1.2 meters centre to centre in square grid were installed by means of a mandrel which was statically stitched into the ground with an anchor plate of size 120mm x 50mm attached at the bottom of the mandrel.

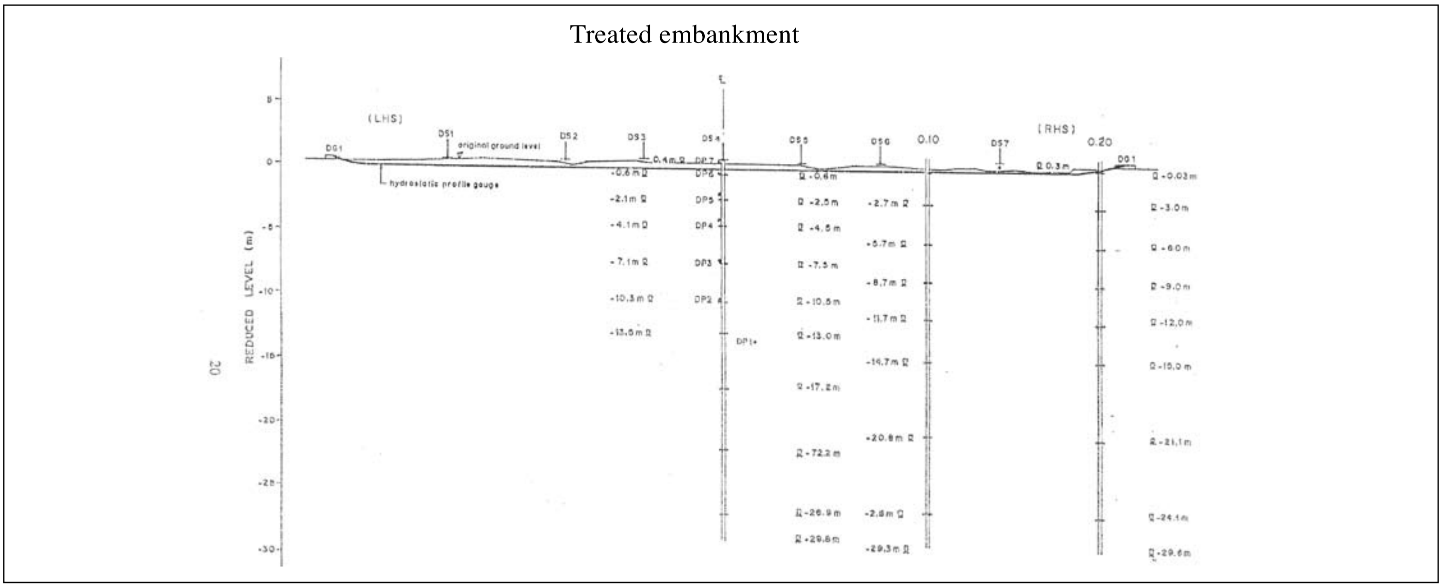


*Figure 3: Instrumentation layout (Treated embankment)*

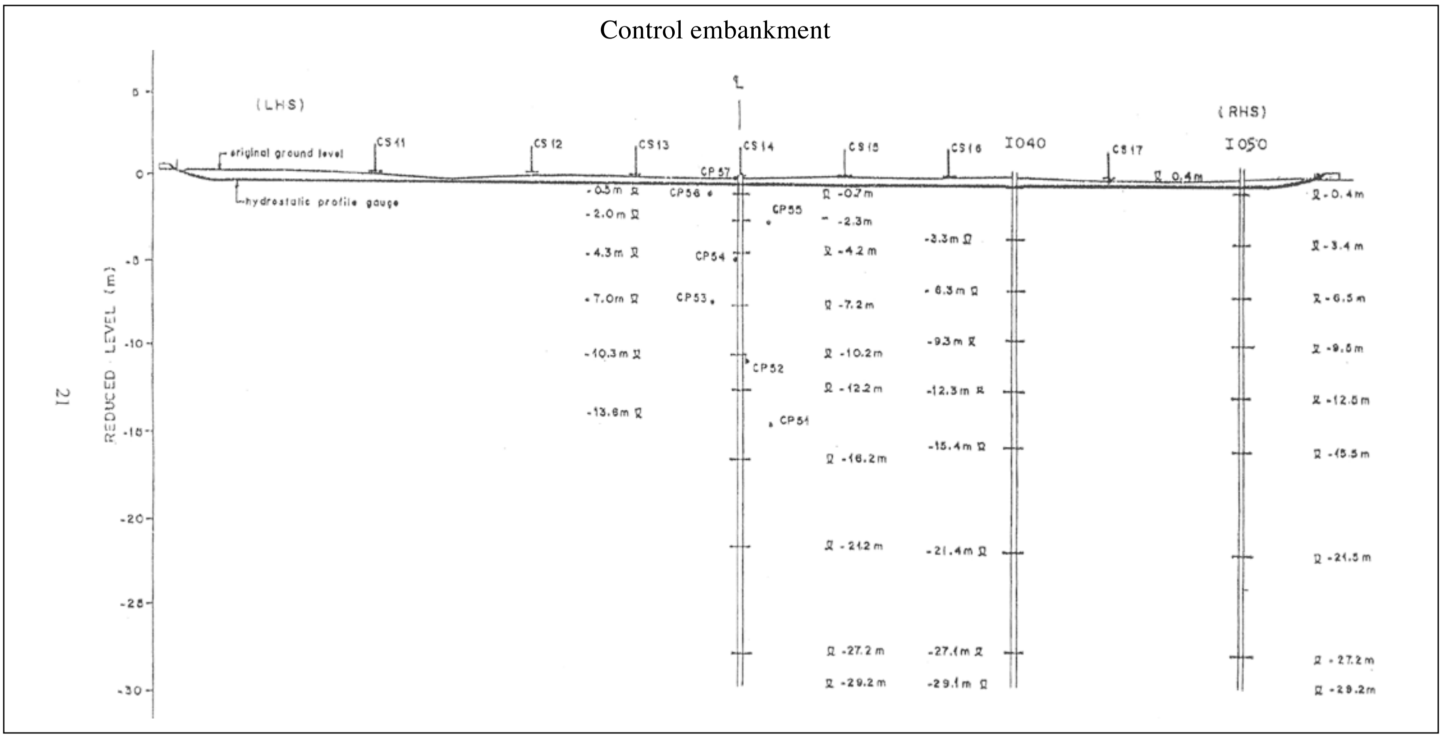


*Figure 4: Instrumentation layout (Control embankment)*

This is to prevent the mandrel from being filled with earth as well as to anchor down the drains. The mandrel containing the prefabricated vertical drain was driven by a crawler crane. When the mandrel had reached the required depth, the prefabricated vertical drain will be cut and the mandrel will then be withdrawn progressively. The anchor plate to which the prefabricated vertical drain was attached remained below the ground. After this operation the equipment would then slew around to the next location and the process will be repeated. After the installation of the prefabricated vertical drains, both the control and treated embankments were built up to 3.6 meters high with a 9 meter long and 2 meter high counterweight berm.

## 4.0 TOTAL PRIMARY CONSOLIDATION SETTLEMENT

In this trial embankment, because of the loaded length is much larger than the compressible layer, the loading in this case can be considered as one dimensional [7]. In the calculation of the one dimensional consolidation settlement, Terzaghi's theory is normally adopted.

For the general case where both recompression and virgin compression are included, the following formula can be used to calculate the total primary consolidation settlement $\rho_T$. [7]

$$\rho_T = \Sigma[C_r/(1 + e_o)*\Delta z*\log(\sigma'_{vc}/\sigma'_{ov}) + C_c/(1 + ec)*\Delta z*\log(\sigma'_{vf}/\sigma'_{vc})] \quad (1)$$

where $C_r$ = Recompression index
$C_c$ = Compression index
$e_c$ = Void ratio corresponding to intersection of recompression to virgin line
$e_o$ = Initial void ratio
$\sigma'_{vf}$ = Final vertical stress
$\sigma'_{vc}$ = Preconsolidation pressure
$\sigma'_{vo}$ = Initial vertical effective pressure
$\Delta z$ = Thickness of compressible layer

Based on the above formula and the consolidation parameters from Tables 1 and 2, the calculated total primary consolidation settlement (using e – $\log_{10}$p curves) for the control and the treated embankments are shown in Table 3.

***Table 3: Calculated total primary consolidation settlements***

| | | |
|---|---|---|
| Control Embankment | 1748 mm | Embankment thickness = 3.60m |
| Treated Embankment | 1724 mm | Embankment thickness = 3.60m |

## 5.0 PREFABRICATED VERTICAL DRAIN DESIGN

The design of vertical drain system is based on the classical theoretical solution developed by Baron [8] in which the drains are assumed to be functioning as an ideal well. The differential equation in term of polar coordinates for a vertical drain behavior is as shown in the following formula:-

$$\delta u/\delta t = C_h * (1/r * \delta u/\delta r + \delta^2 u/\delta^2 r) \quad (2)$$

where $C_h$ = Coefficient of horizontal consolidation
u = Excess pore pressure
r = Radial distance of the considered path from centre of the drained soil cylinder
t = Time after an instantaneous increase of the total vertical stress.

The average degree of consolidation $U_h$ at a depth z due to the effect of radial drainage only can be expressed as follow:-

$$U_h = 1 - \exp(-8 * T_h / F_n) \quad (3)$$

where $T_h$ = $C_h * t / d_e^2$ = Time factor
Fn = $n^2/(n^2-1)* Ln(n)-[(3*n^2-1)/4*n^2] \approx Ln(n) - 0.75$ for n >20
n = Drain spacing ratio = $(d_e/d_w)$
$d_e$ = Equivalent diameter of soil cylinder = 1.13 of drain spacing for square grid
$d_w$ = Equivalent drain diameter = 2*(a+b)/π where a and b = width and thickness respectively of the band shaped drain

Equation 3 does not include the effect of smear and drain resistance. Similar equation (equal strain) was developed by Hansbo [9] where smear and drain resistance effect was taken into consideration and the average degree of consolidation is given as below:-

$$U_h = 1 - \exp(-8 *_{Th} / F) \quad (4)$$

where F = Fn + Fr + Fs
Fn = Ln(n) – ¾
Fr = $3.142 * z*(L-z)*(k_w/q_w)$
Fs = $(k_h/k_s - 1) * Ln(d_s/d_e)$
$q_w$ = Discharge capacity of the drain
$k_s$ = Permeability of the smeared zone
$k_h$ = Permeability of the undisturbed zone
$d_s$ = Diameter of smeared zone

## 6.0 PARAMETRIC STUDY OF THE PREBRACATED VERTICAL DRAINS TREATED EMBANKMENT

It has been reported by others [10 - 12] that smear and drain resistance have a significant effect on the time of consolidation. In order to study these effects, a systematic study of the influence of the smear and drain resistance on the time of consolidation were carried out for the prefabricated vertical drains in the treated embankment with the following parameters:-

$q_w$ = 300 $m^3$/year
$k_h/k_s$ = 1, 8, 10, 12, 14 and 16
$C_h$ = 4.5 $m^2$/year
s = $d_s/d_w$ = 2.5
L = Length of drains = 6 meters and 14 meters.

Drain installation disturbed the soil to a certain degree. The degree of disturbance will depend on the sensitivity and macro fabric of the soils.

Because of the extent and condition of the more or less remolded zone around each drain depend very much on the installation method, the value of s = 2.5 was adopted for these analysis. [13]

Figures 5 to 9 illustrate the settlement versus time for values of $k_h/k_s$ = 8, 10, 12, 14 and 16 respectively with s = 2.5. The measured settlement for DS4 which was located at the centre of the treated embankment was also superimposed into these curves for comparison purposes. The degree of consolidation versus time with the above drain and soil parameters were shown in Figure 10. In this figure the degree of consolidation versus time based on field measurement and laboratory consolidation parameters was also plotted. From Figure 10 it can be seen that the best fit to the actual field performance is obtained based on $k_h/k_s = 10$, s = 2.5 and $C_h = 4.5\ m^2$/year.

In addition the hyperbolic equation as mentioned in [14] and is shown below

$$t/s = m^* t + c \tag{5}$$

was also used to predict the settlement versus time curves, where *t* is the time of settlement, *s* is the settlement, *m* and *c*

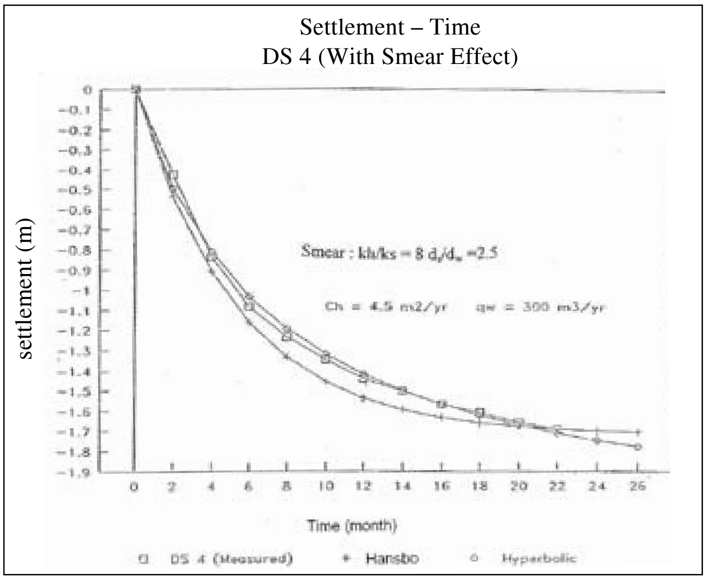


*Figure 5: Instrumentation layout (Control embankment)*

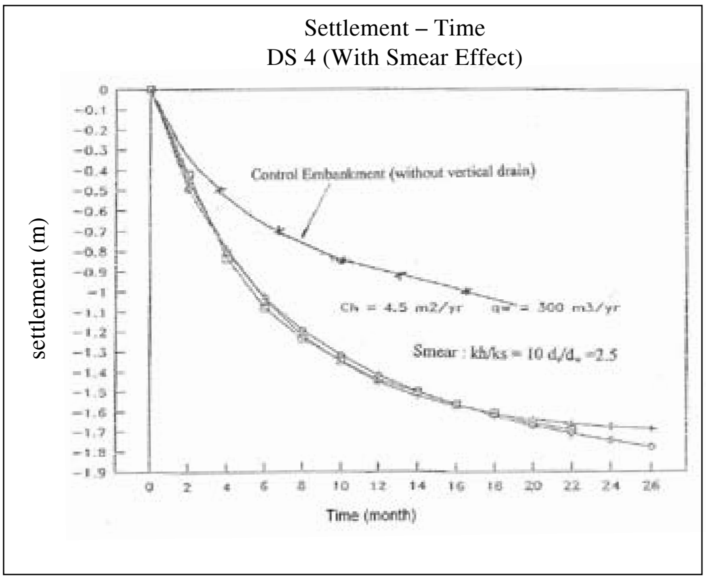


*Figure 6: Instrumentation layout (Control embankment)*

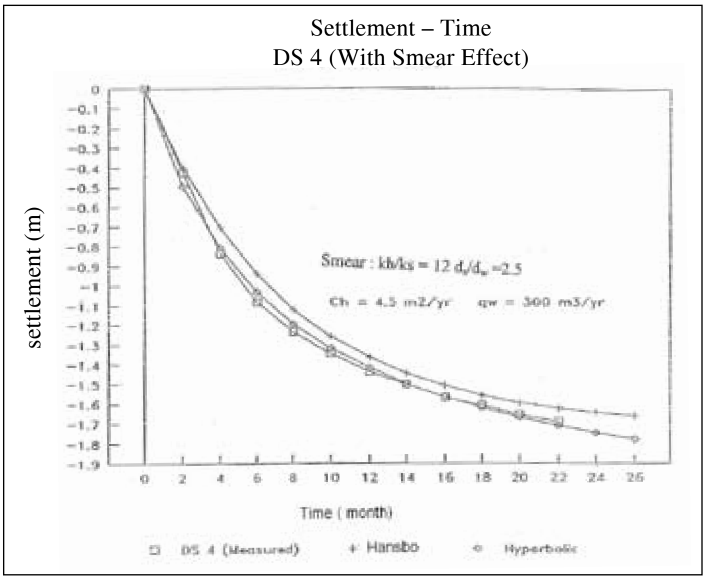


*Figure 7: Settlement versus time ($k_h/k_s$ = 12)*

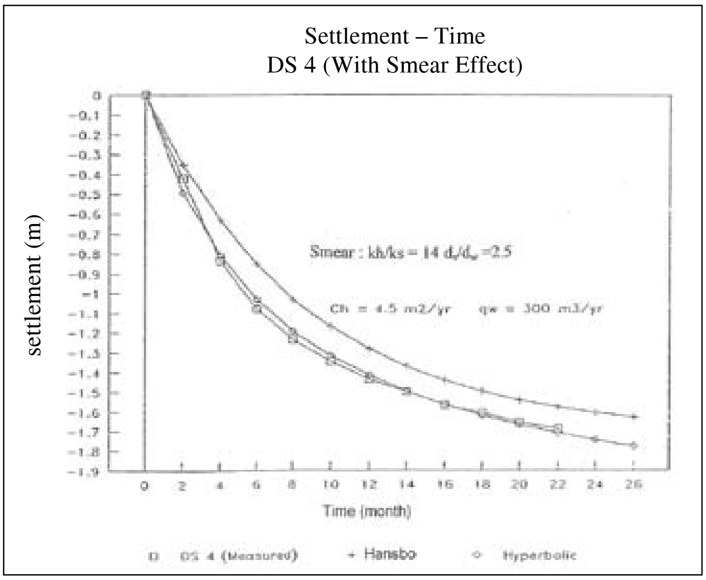


*Figure 8: Settlement versus time ($k_h/k_s$ = 14)*

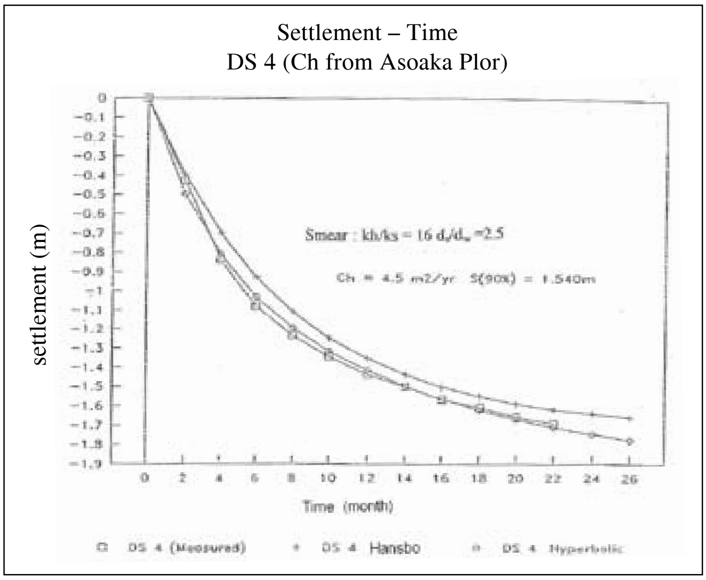


*Figure 9: Settlement versus time ($k_h/k_s$ = 16)*

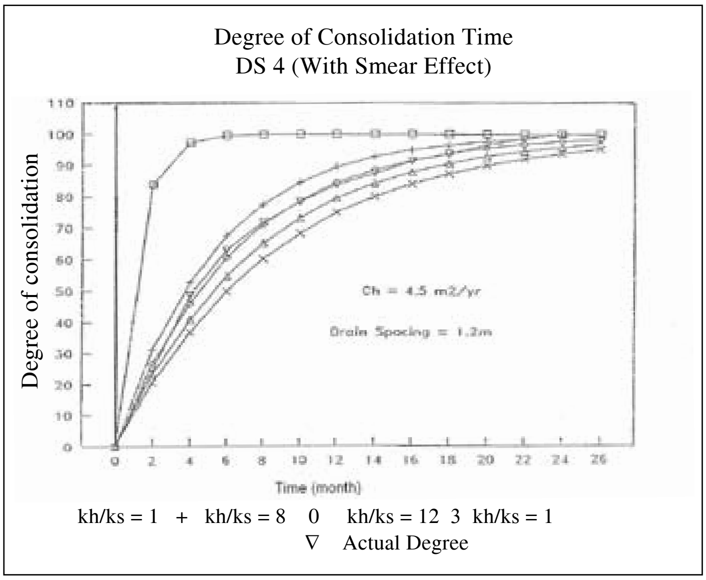


***Figure 10: Degree of consolidation versus time (various $k_h/k_s$)***

are constants. The settlement versus time plots as predicted by the hyperbolic equation and Hansbo's equal strain equation with smear of $k_h/k_s$ = 10 and $d_s/d_e$ = 2.5 and $C_h$ = 4.5 $m^2$/year as shown in Figure 6 which give closed agreement to the actual field performance. In the same plot the settlement versus time curve for the control embankment (without vertical drain) is also included, comparing the observed settlement of the treated embankment with the settlement of the control embankment, it was clear that vertical drains had helped to accelerate the consolidation process.

Tables 4 and 5 show the effect of drain resistance and smear effect on the degree of consolidation. From these tables, it can be seen that there is no or negligible effect on the degree of consolidation for drain length of 6 meters to 14 meters.

***Table 4 : Effect of smear, drain resistance and drain length on degree of consolidation***

***(Ch = 4.5 $m^2$/year qw = 300 $m^3$/year L = 14 meters )***

| Degree of Consolidation | | | | | | |
|---|---|---|---|---|---|---|
| Month | $K_h/k_s$ = 1 | 8 | 10 | 12 | 14 | 16 |
| 4 | 97.5 | 52.8 | 46.2 | 41.0 | 36.9 | 33.5 |
| 6 | 99.6 | 67.6 | 60.6 | 54.7 | 49.9 | 45.7 |
| 8 | 99.9 | 77.7 | 71.1 | 65.2 | 60.2 | 55.7 |
| 10 | 100 | 84.7 | 78.8 | 73.3 | 68.3 | 63.9 |

***Table 5 : Effect of smear, drain resistance and drain length on degree of consolidation***

***(Ch = 4.5 $m^2$/year qw = 300 $m^3$/year L = 6 meters )***

| Degree of Consolidation | | | | | | |
|---|---|---|---|---|---|---|
| Month | $K_h/k_s$ = 1 | 8 | 10 | 12 | 14 | 16 |
| 4 | 97.5 | 52.8 | 46.2 | 41.0 | 36.9 | 33.5 |
| 6 | 99.6 | 67.6 | 60.6 | 54.7 | 49.9 | 45.7 |
| 8 | 99.9 | 77.7 | 71.1 | 65.2 | 60.2 | 55.7 |
| 10 | 100 | 84.7 | 78.8 | 73.3 | 68.3 | 63.9 |

*Note:*
*6(TE)B1, 6(TE)B3, 6(TE)B4, 6(TE)B6 that was mentioned in the next page were bored holes that were carried out before the embankments were constructed. $m^1$ and $m^2$ were boreholes that were carried out after the embankments were subjected to a rest period of 12 months.*

# 7.0 EARLY PREDICTION OF TOTAL PRIMARY SETTLEMENT USING EMPIRICAL FORMULA

In practice, it is importance to predict the primary consolidation settlement in early stages of surcharging or preloading from field settlement data. With the predicted settlement, the designer will be able to assess the required time for surcharging or preloading removal and also the post construction settlements. Examples of these predictions using empirical methods are Bujang [15] and Mohd [16].

In this study a 360-day settlement data for the treated embankment at locations DS3 DS4 and DS5 were used to predict the total primary settlement by the hyperbolic and the Asoaka's methods [17]. The predicted settlement based on 360-day results were compared to those of the 675-day measurements as shown in Table 6 It can be seen that the average settlement as predicted from the 360-day data by the hyperbolic and the Asoaka's methods are in closed agreement to those predicted from the 675-day data. Table 7 shows similar predictions for the control embankment, but there is a substantial difference between the settlements calculated from laboratory data as compared to those calculated by the empirical methods. It is not clear as to why these empirical methods estimated a lower total primary settlement as compared to those obtained by laboratory e - $\log_{10}$P method. This discrepancy is also reported by other [18].

Tables 8 and 9 show the settlement prediction for both the treated and control embankments using the 360-day data to predict future settlement using the hyperbolic equation. From these two tables, it can be seen that the predicted settlements by the hyperbolic equation and measured settlements are in good agreement.

***Table 6: Settlement comparison (Data based on 675-day and 360-day) treated embankment***

| Predicted Total Primary Settlement by Empirical Methods (m) | | | | | Laboratory Data |
|---|---|---|---|---|---|
| | Hyperbolic | | Asoaka | | |
| Location | 675-day | 360-day | 675-day | 360-day | Average Settlement (m) |
| DS 3 | 1.971 | 1.971 | 1.698 | 1.531 | 6(TE) B1 = 1.748<br>6(TE) B3 = 1.550<br>6(TE) B4 = 1.698<br>6(TE) B6 = 1.724 |
| DS 4 | 2.138 | 2.192 | 1.728 | 1.669 | |
| DS 5 | 2.101 | 2.137 | 1.666 | 1.631 | |
| Average | 2.070 | 2.100 | 1.697 | 1.610 | 1.680 |

***Table 7: Settlement comparison (Data based on 675-day and 360-day) control embankment***

| Predicted Total Primary Settlement by Empirical Methods (m) | | | | | Laboratory Data |
|---|---|---|---|---|---|
| | Hyperbolic | | Asoaka | | |
| Location | 675-day | 360-day | 675-day | 360-day | Average Settlement (m) |
| CS 13 | 1.303 | 1.322 | 1.103 | 1.013 | 6(TE) B1 = 1.748<br>6(TE) B3 = 1.550<br>6(TE) B4 = 1.698<br>6(TE) B6 = 1.724 |
| CS 14 | 1.341 | 1.067 | 1.067 | 0.981 | |
| CS 15 | 1.434 | 1.435 | 1.236 | 1.089 | |
| Average | 1.359 | 1.274 | 1.135 | 1.028 | 1.670 |

*Table 8: 360-day data to predict future settlement by hyperbolic method*

| Treated Embankment DS 3 | | | t/s = 0.507 * t + 89.7 |
|---|---|---|---|
| Day | Measurement | Predicted | Percentage Difference |
| 512 | 1.458 | 1.465 | +0.46 % |
| 615 | 1.530 | 1.531 | +0.07 % |
| 645 | 1.540 | 1.547 | -0.13 % |
| 675 | 1.561 | 1.562 | +0.06% |
| **Treated Embankment DS 4** | | | **t/s = 0.456 * t + 85.9** |
| Day | Measurement | Predicted | Percentage Difference |
| 453 | 1.544 | 1.548 | +0.26 % |
| 512 | 1.596 | 1.602 | +0.38 % |
| 615 | 1.668 | 1.671 | +0.18 % |
| 675 | 1.696 | 1.714 | +1.06% |

*Table 9: 360-day data to predict future settlement*

| Control Embankment CS 13 | | | t/s = 0.756 * t + 117.5 |
|---|---|---|---|
| Day | Measurement | Predicted | Percentage Difference |
| 519 | 1.050 | 1.018 | -3.05 % |
| 575 | 1.084 | 1.041 | -3.97% |
| 603 | 1.088 | 1.051 | -3.40 % |
| **Control Embankment CS 14** | | | **t/s = 0.744 * t + 129.8** |
| Day | Measurement | Predicted | Percentage Difference |
| 417 | 0.983 | 1.005 | +2.24 % |
| 573 | 1.021 | 1.030 | +0.88 % |
| 612 | 1.071 | 1.042 | -2.71 % |

## 8.0 COMPARISON OF SOIL CHARATERISTICS BEFORE AND AFTER LOADING/TREATMENT

## 8.1 Treated Embankment

### 8.1.1 Compression Ratio

The compression ratio versus depth of the treated embankment before and after about 12 months of rest period (loading) is shown in Figure 11. From this plot it can be seen that there is a reduction of compression ratio (CR = $C_c$ / (1 + eo) after about 12 months of rest period. This reduction in compression ratio is more obvious near the top and bottom drainage layers.

### 8.1.2 Preconsolidation Pressures

The preconsolidation pressures versus depth of the treated embankment before and after about 12 months of rest period was plotted as shown in Figure 12. From this figure it can be seen that there is an increase in preconsolidation pressures Pc' after about 12 months of rest period. This increase in preconsolidation pressures is throughout the thickness of clay layer.

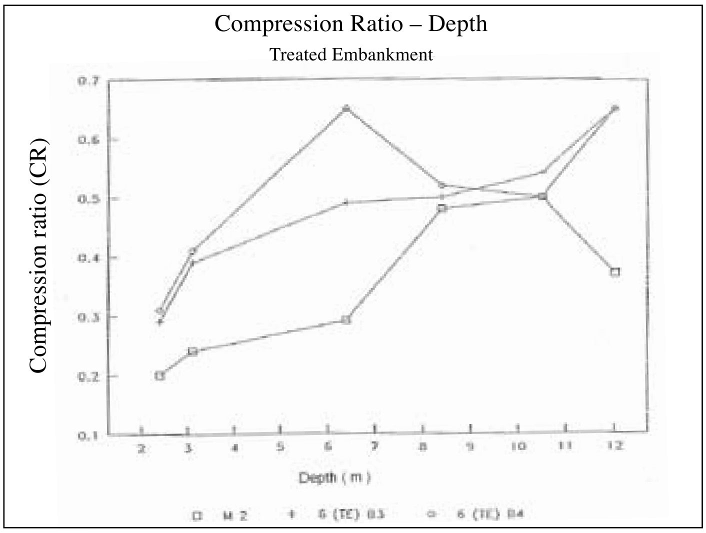


*Figure 11: Compression ratio versus Depth*

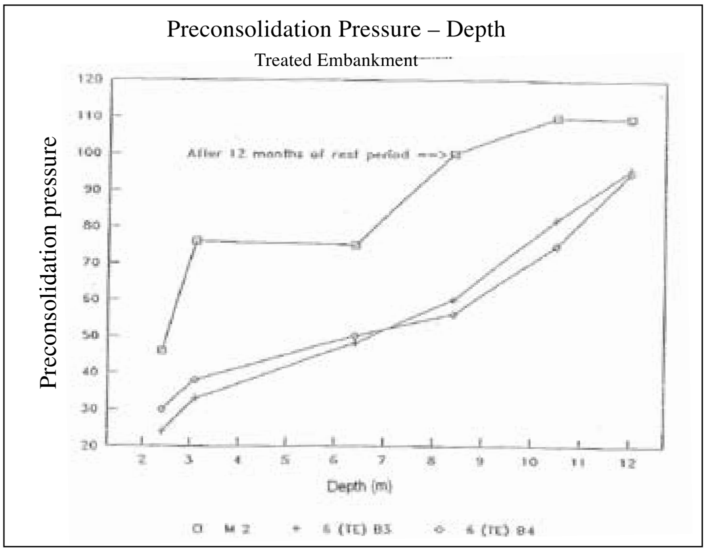


*Figure 12: Preconsolidation presure versus depth (Treated)*

### 8.1.3 Vane Shear Strength

The vane shear strength versus depth of the treated embankment before and after 12 months of rest period is shown in Figure 13. V1 and V7 were the insitu vane shear strengths that were taken after about 12 months of rest period. From this figure it can also be seen that there is an increase in vane shear strength.

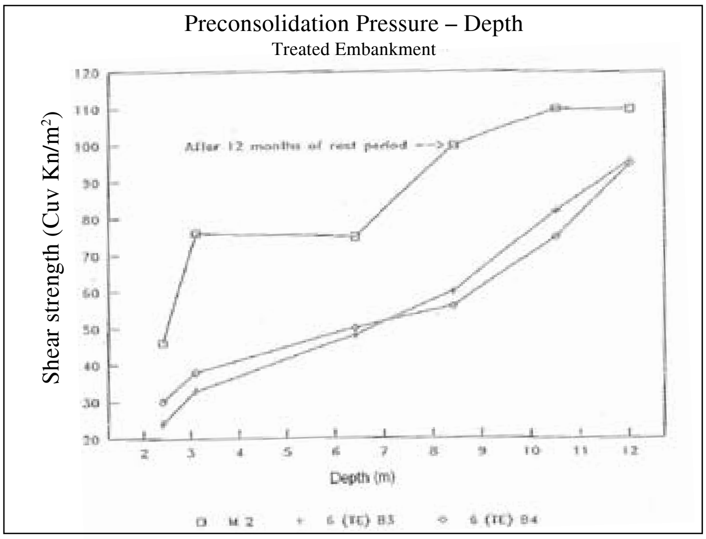


*Figure 13: Vane Shear strength versus depth (Treated)*

## 8.2 Control Embankment

### 8.2.1 Compression Ratio

The compression ratio versus depth of the control embankment before and after about 12 months of rest (loading) is shown Figure 8. From this figure it can be seen that there is practically no change in the compression ratio (CR) about 12 months after the rest period.

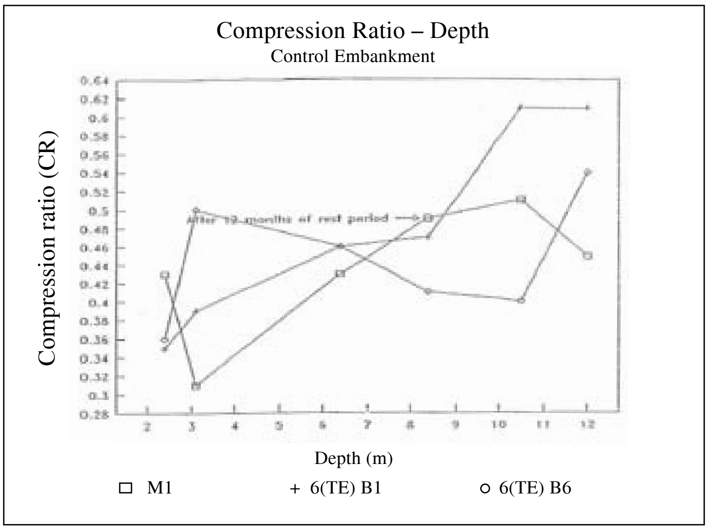


*Figure 14: Compression ratio versus depth (Control)*

### 8.2.2 Preconsolidation Pressures

The preconsolidation pressure versus depth of the control embankment is shown in Figure 15. Based on this plot it can be seen that there is a slight increase in preconsolidation pressure (Pc') about 12 months after the rest period except at the location near R.L. 10.5 meters.

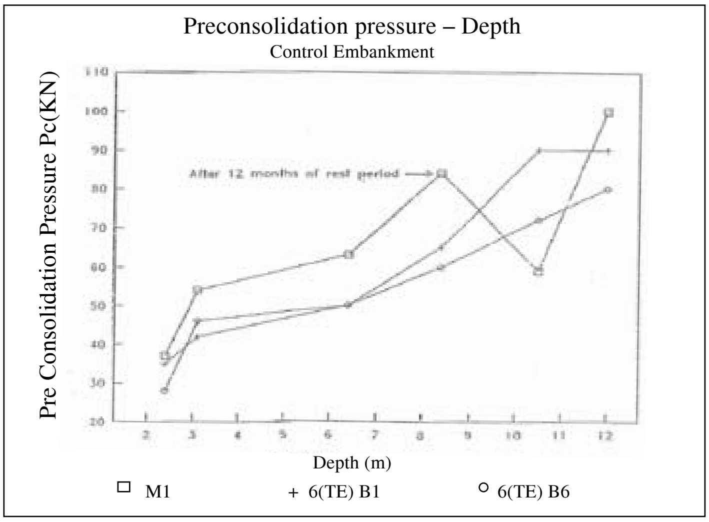


*Figure 15: Preconsolidation pressure versus depth (Control)*

### 8.2.3 Vane Shear Strength

The graph of vane shear strength versus depth of the control embankment is shown in Figure 16 V6 and V8 were the insitu vane shear strength that was taken about 12 months of rest period. From this figure it can seen that there is some increase in the vane shear strength of the control embankment especially near the top drainage layer.

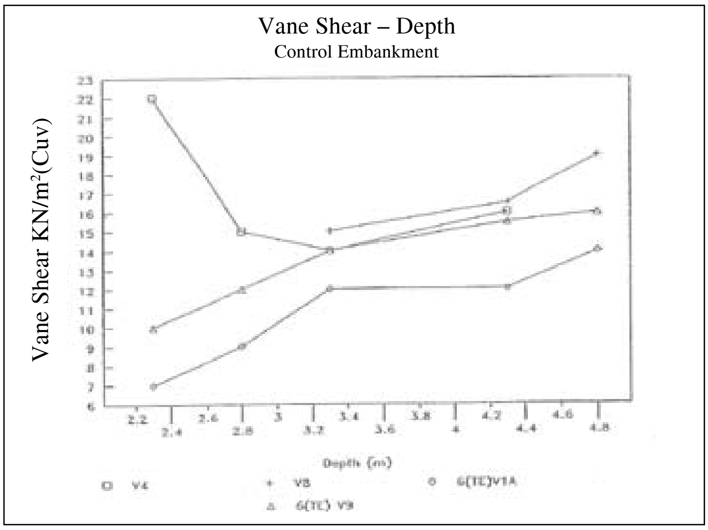


*Figure 16: Vane Shear strength versus depth (Control)*

### 8.2.4 Treated and Control Embankments Comparisons

The compression ratio, preconsolidation pressures and vane shear strength versus depth for both the treated and control embankments after about 12 months of rest period are shown in Figures 17 to 19. From these figures it can be seen that there are much better improvement in the compression ratio, preconsolidation pressures and vane shear strength in the treated embankment than those in the control embankment.

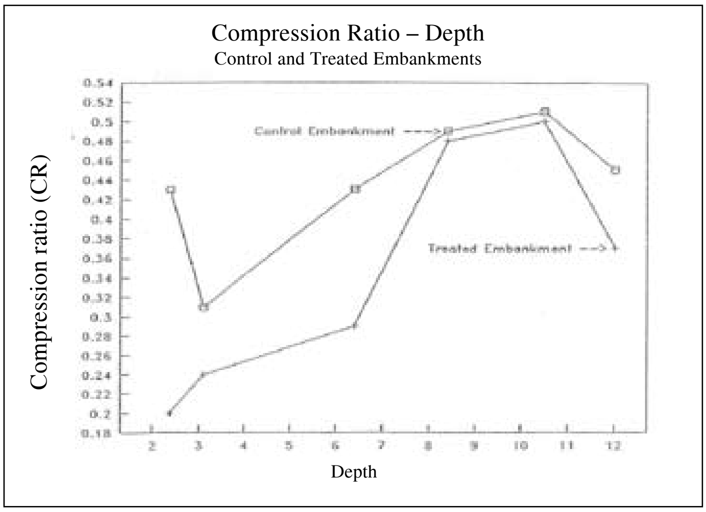


*Figure 17: Compression ratio versus depth (Treated and control)*

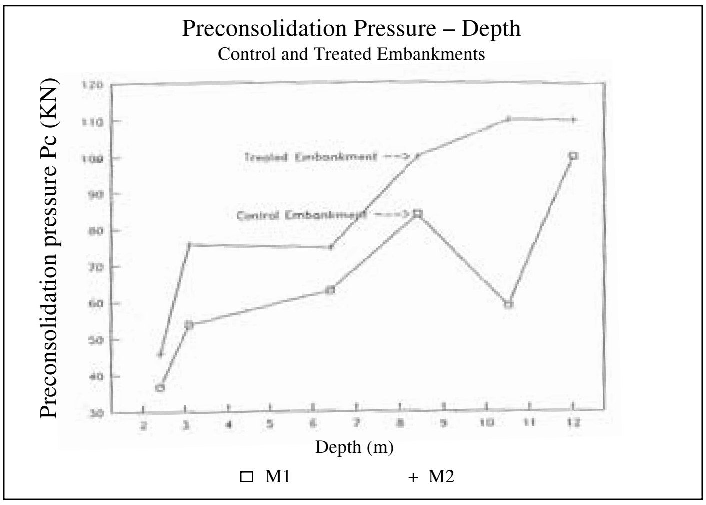


*Figure 18: Preconsolidation pressure versus depth (Treated and control)*

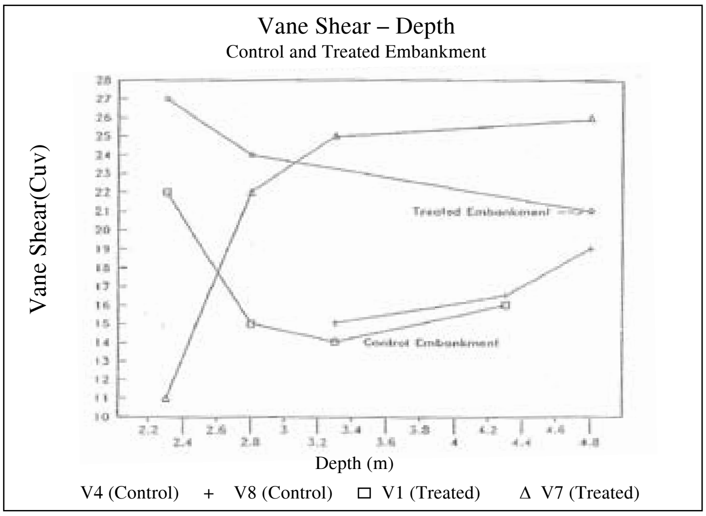


*Figure 19: Vane Shear strength versus depth (Treated and control)*

## 9.0 LATERAL DEFORMATION

The lateral deformations versus time and fill thickness is shown in Figure 20. In Figure 20 it can be seen that lateral deformation increases as the fill thickness increased and it continued to deform even under constant load. The lateral deformation versus time at various stages of loadings for the treated embankment is shown in Figure 21.

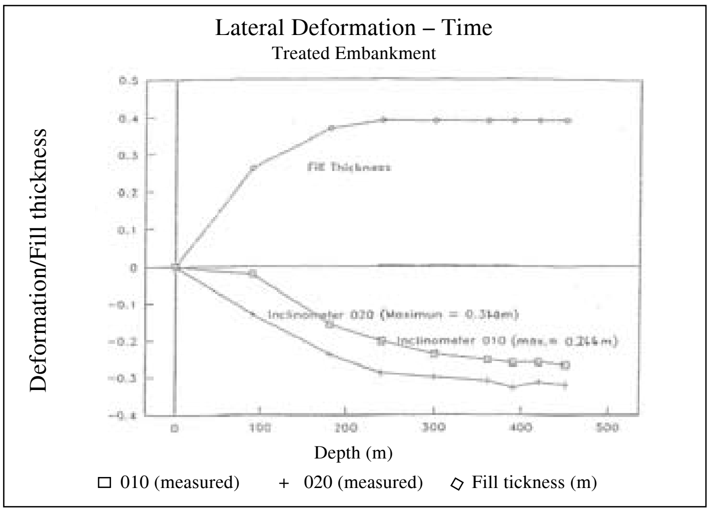


*Figure 20: Lateral deformation versus time (Treated)*

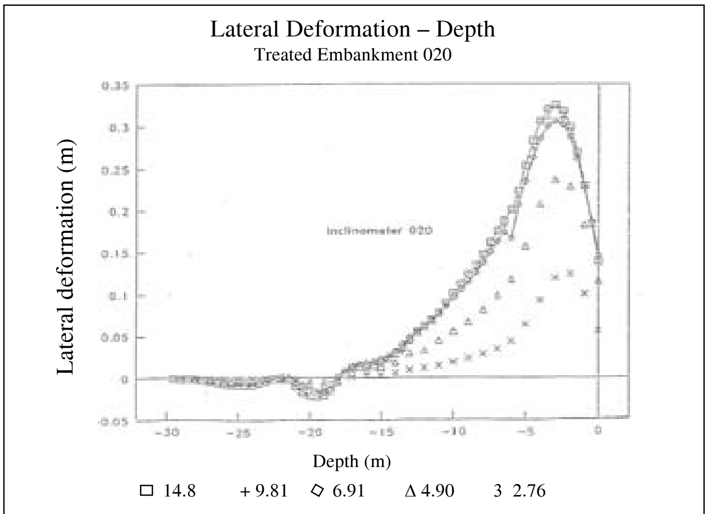


*Figure 21: Lateral deformation versus depth (Treated)*

The maximum lateral deformation for inclinometer 020 is about 0.32 meter after a period of 450 days. From inclinometer 020 it was noticed that the maximum deformation for the treated embankment was about 3 meters below the existing ground level.

The prediction of lateral deformation profiles for a loaded embankment is always a difficult task [19] and the prediction of ultimate lateral deformation is seldom carried out. The author made use of the hyperbolic equation and the measured lateral deformation data from inclinometers No. 010 and 020 to predict the ultimate lateral deformation for the treated embankment. It can be seen from Table 12, the "$R^2$" values of the linear regression for the hyperbolic equation based on the measured data for inclinometers 010 and 020 is 0.9432 and 0.9536 respectively showing a reasonable linearity between t/d and *t*, where *t* is the time and *d* is the lateral deformation. The hyperbola of these lateral deformations together with the actual measurements are shown in Figures 22 and 23.

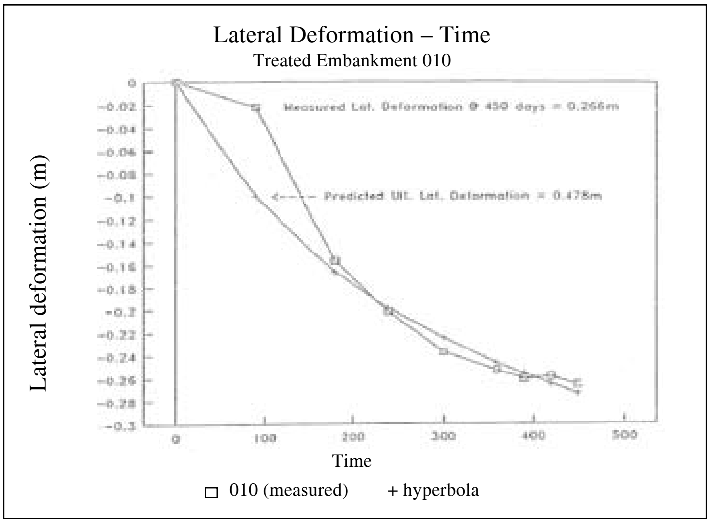


*Figure 22: Lateral deformation versus time (010 Treated)(Hyperbolic)*

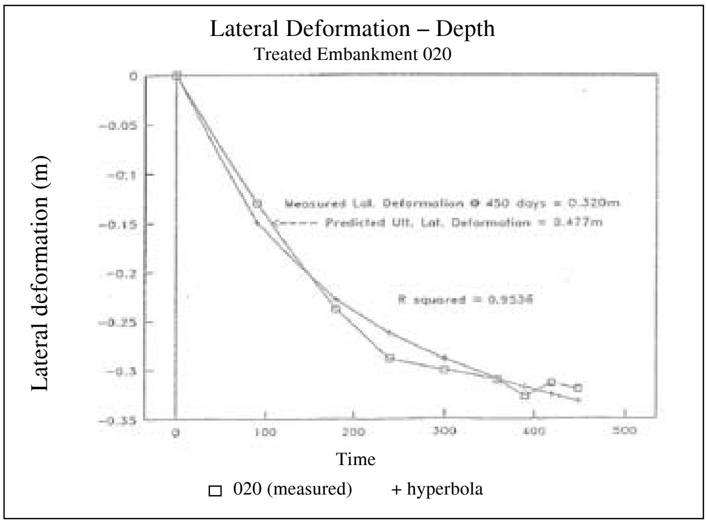


*Figure 23: Lateral deformation versus time (020 Treated)(Hyperbolic)*

The lateral deformation versus time and fill thickness for the control embankment for inclinometers 040 and 050 were plotted and shown in Figure 24. The behaviour of this plot was similar to those of the treated embankment. Figure 25 shows the lateral deformation versus depth of the control embankment, it can be seen that the maximum deformation is around 3.2 meters below existing ground level. Similarly the hyperbolic equation was used for the prediction of the ultimate lateral deformation of the control embankment. The predicted ultimate lateral deformation from the measured data for inclinometers 040 and 050 are shown in Figures 26 and 27 The "$R^2$" values of the linear regression for the hyperbolic equation based on the measured data for inclinometers 040 and 050 are 0.9824 and 0.9903 respectively, (see Table 12).

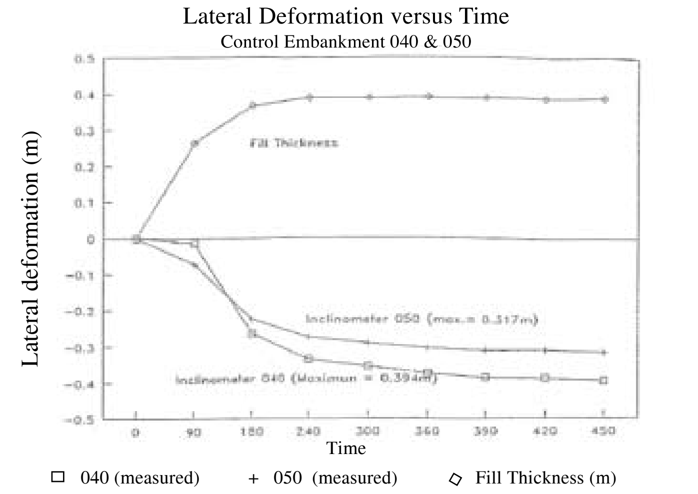


*Figure 24: Lateral Deformation versus time (Control)*

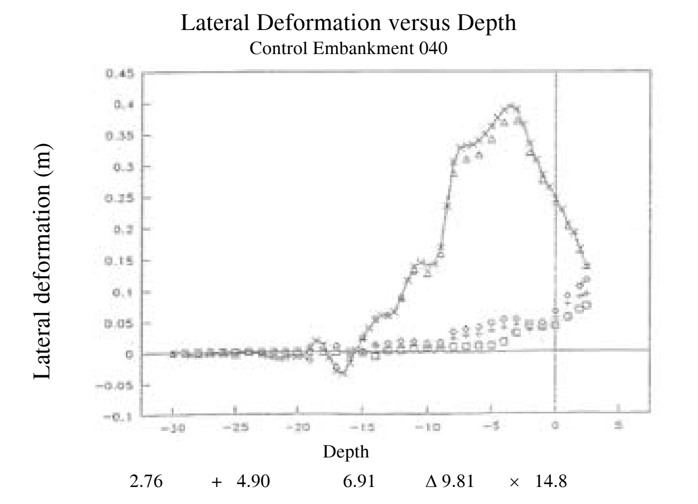


*Figure 25: Lateral Deformation versus depth (Control)*

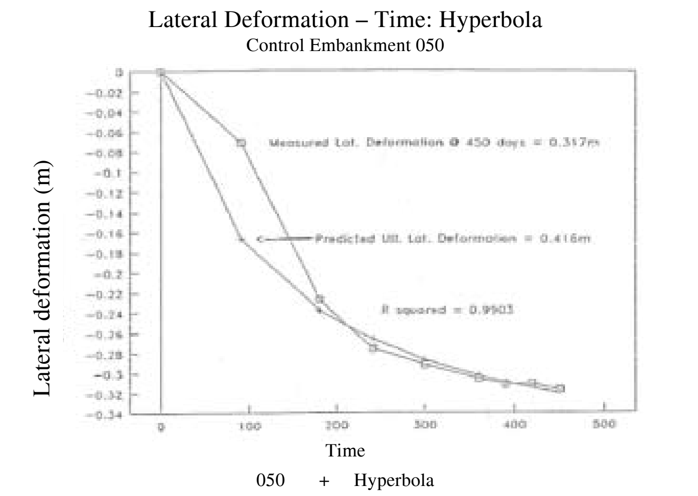


*Figure 26: Lateral Deformation versus time (050 Control)(Hyperbolic)*

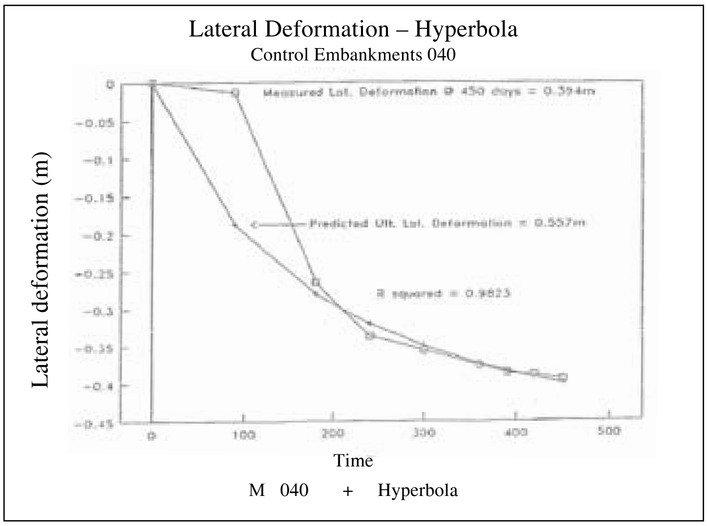


*Figure 27: Lateral Deformation versus time (040 Control)(Hyperbolic)*

*Table 12: Predicted ultimate lateral deformation by hyperbolic method*

| Treated Embankment | | | |
|---|---|---|---|
| Inclinometer | R Squared | Predicted Ultimate Lateral Deformation | Measured (450 days) Lateral Deformation[3] |
| 010 | 0.9432 | 0.478 m | 0.266 m |
| 020 | 0.9536 | 0.477 m | 0.320 m |
| **Control Embankment** | | | |
| Inclinometer | R Squared | Predicted Ultimate Lateral Deformation | Measured (450 days) Lateral Deformation |
| 040 | 0.9824 | 0.557 m | 0.394 m |
| 050 | 0.9903 | 0.416 m | 0.317 m |

## 10.0 CONCLUSIONS

Based on the results and discussion as mentioned above, the following conclusion can be drawn:-

- Prefabricated vertical drains had helped to accelerate the consolidation process.
- The 360–day measured settlement data can be used both in the hyperbolic equation and the Asaoka's method to predict the total primary consolidation settlement with good accuracy.
- $C_h$ value of 4.5 $m^2$/year (back calculated from Asaoka's plot) with $k_h/k_s$ = 10 and s = 2.5 seems to give the best fit to the measured settlement for the treated embankment
- There is generally a significant reduction in the compression ratio in the treated embankment as compared to the control embankment.
- There are substantial increase in preconsolidation pressures and vane shear strength in the treated embankment and negligible increased in the control embankment except near the drainage layers.
- The ultimate lateral deformations of the embankments that were predicted by the hyperbolic equation are 0.478 meter (inclinometer 010) and 0.557 meter (inclinometer 040) for the treated and the control embankments respectively.

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## PROFILE

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