Effect of Soil Cement Replacement Ratio on Settlement Reduction

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ABSTRACT
This study investigates the effect of soil cement column diameter and spacing on overall settlement beneath a road embankment situated on normally consolidated estuarine deposits in the South East Queensland region. The effect of Area Replacement Ratio (a_s) on settlement with consideration to column diameter and spacing variation is also compared to actual site settlement gauge data. The modelling was undertaken using Plaxis 2D and the results are discussed with respect to the variation in parameters as described. The sub surface profile was established during the investigation for the embankment and a series of oedometer tests were undertaken on samples obtained from the boreholes. The Plaxis inputs were calculated from the results of the oedometer tests and by several recognised methods as discussed in the paper. The aim of the research is to establish a graph suitable for initial settlement estimations on cement improved compressible soil.

KEYWORDS: Soil Columns; Embankment, Settlement, Numerical Modelling, Queensland, Australia, Case Study.

INTRODUCTION
Deep, unconsolidated compressible clay soils form the sub-surface profile of many coastal Holocene estuarine deposits in Australia. These soil profiles are particularly prevalent in the south east region of Queensland from Brisbane to the northern Gold Coast, areas, which are in high demand for large scale infrastructure development supporting the growing domestic construction industry.

Previously, conventional construction techniques on roadway embankments in these areas have employed pore pressure release mechanisms, such as vertical drains, to accelerate the increase in shear strength of these unconsolidated soils through primary consolidation either due to the embankment load or a load resulting from the embankment and an engineered pre-load.
prior to pavement construction and/or service installation. In some instances, the construction program can also be delayed, allowing an increase in the factor of safety of embankment stability as the embankment load increases.

Internationally, the use of soil/cement columns has been used extensively in similar soil profiles to great effect. Deep Mixing is one alternative that offers a wide variety of benefits and possibilities (Holm 2000). It improves the engineering properties of the treated soil (Porbaha et al. 1998a, Holm 2005, Bruce 2001) and is a very cost-effective foundation solution (Porbaha 1998, Holm 2000).

This paper presents the results of a comparative numerical study performed on a typical south-east Queensland soil profile subjected to typical embankment geometry. The study compares the performance of the embankment when constructed with soil cement columns at differing diameters and spacings. The numerical modelling has been performed using Plaxis 2D. This method has been on soil profiles in countries such as Thailand and Japan, however, this research fills a gap in undertaking a case study in Australian conditions.

**SUBSURFACE PROFILE**

The soil profile employed for the study is typical of the estuarine deposits in the South East Queensland region. The subsurface profile consisted of a surficial layer of topsoil and rock to a depth of 0.7 metres overlying a deep soft estuarine clay deposit to a depth of nearly 20 metres. The profile is summarised in Table 1 below:

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>0 – 3.5</th>
<th>3.5 – 5.0</th>
<th>5.0 – 19.1</th>
<th>19.1 – 23.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Type</td>
<td>Estuarine Silty Clay</td>
<td>Estuarine Sandy Silty Clay</td>
<td>Estuarine Silty Clay</td>
<td>Sandy Gravel See note below</td>
</tr>
<tr>
<td>E (kN/m$^2$)</td>
<td>1650</td>
<td>2400</td>
<td>1260</td>
<td></td>
</tr>
<tr>
<td>$\rho_d$ (kN/m$^3$)</td>
<td>10.02</td>
<td>11.40</td>
<td>11.00</td>
<td></td>
</tr>
<tr>
<td>$\rho_s$ (kN/m$^3$)</td>
<td>10.04</td>
<td>17.00</td>
<td>17.02</td>
<td></td>
</tr>
<tr>
<td>c (kN/m$^2$)</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>$\theta$ (degrees)</td>
<td>27°</td>
<td>27°</td>
<td>27°</td>
<td></td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>k (m/day)</td>
<td>8.64 x 10$^{-3}$</td>
<td>8.21 x 10$^{-3}$</td>
<td>3.88 x 10$^{-3}$</td>
<td></td>
</tr>
</tbody>
</table>

Note that the Sandy Gravel encountered at 19.1 metres has been assumed to be non-compressible and is represented as a permeable boundary in the model. The groundwater level is situated 1.0 metres below the existing surface level.

Plaxis 8 provides six (6) different soil models to simulate soil behaviour. The Mohr Coulomb model was adopted as recommended for situations where soil parameters are approximated. This model requires: Young’s Modulus ($E$) and Poisson’s Ratio ($\nu$) for elasticity, Dilatancy Angle ($\psi$), Permeability ($k$), Dry Unit Weight ($\gamma_d$) and Saturated Unit Weight ($\gamma_s$).

It has been shown that a reasonable estimation of Young’s Modulus with depth can be made from empirical relationships (USACE, 1990). Young’s Modulus has been estimated from the following:
\[ E_s = K_c C_u \]  \hspace{1cm} (1)

where

- \( E_s \) = Young's Modulus (kPa)
- \( K_c \) = Correlation Factor
- \( C_u \) = Undrained Shear Strength

The correlation factor \( K_c \) is determined from the Overconsolidation Ratio (OCR) and the soil Pasticity Index (PI). The primary benefit of employing this technique is to avoid the inherent disturbance found in the undisturbed sampling procedure; particularly in soft estuarine soils that this study deals with. \( K_c \) was estimated from the chart presented by the U.S. Army Corps of Engineers in the Engineering Manual 1110-1-1904 (USACE 1990).

The Young’s Moduli estimated for this study are presented in Table 1 below:

<table>
<thead>
<tr>
<th>Layer Thickness</th>
<th>OCR</th>
<th>PI</th>
<th>( K_c )</th>
<th>( C_u ) (kPa)</th>
<th>( E_s ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5 m.</td>
<td>1.9</td>
<td>28</td>
<td>61</td>
<td>27</td>
<td>1650</td>
</tr>
<tr>
<td>1.5 m.</td>
<td>1.6</td>
<td>29</td>
<td>60</td>
<td>40</td>
<td>2400</td>
</tr>
<tr>
<td>14.1 m.</td>
<td>1.3</td>
<td>31</td>
<td>60</td>
<td>21</td>
<td>1260</td>
</tr>
</tbody>
</table>

Permeability was determined from Terzaghi’s theory of consolidation from the equation:

\[ k = C_v M_s \gamma_w \]  \hspace{1cm} (2)

where \( \gamma_w \) is the unit weight of water.

A series of Oedometer were performed on samples from the sub surface profile. Representative results from layer midpoints are presented in Table 3 below.

Permeability was calculated by determining the effective overburden stress at the centre of each layer and interpolating the oedometer results. The calculated permeability values are presented in Table 3 below:

<table>
<thead>
<tr>
<th>Layer Midpoint</th>
<th>( \sigma' ) (kPa)</th>
<th>( m_v ) (m²/MN)</th>
<th>( c_v ) (m²/year)</th>
<th>( k ) (m/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.75 m.</td>
<td>50.95</td>
<td>0.85</td>
<td>5.01</td>
<td>8.64 x 10⁻⁵</td>
</tr>
<tr>
<td>6.25 m.</td>
<td>67.9</td>
<td>0.88</td>
<td>3.48</td>
<td>8.21 x 10⁻⁵</td>
</tr>
<tr>
<td>14.05 m.</td>
<td>125.47</td>
<td>0.975</td>
<td>1.63</td>
<td>3.88 x 10⁻⁵</td>
</tr>
</tbody>
</table>

Given the normally consolidated state of the estuarine clays, the Dilatancy Angle (\( \psi \)) has been assumed to equal zero for all layers in the subsurface profile in accordance with Abusharar et al. (2000).
Poisson’s ratio for the compressible estuarine clays has been assumed to equal 0.3 (Abusharar et al. 2000).

In summary, the input parameters for the Plaxis analysis are presented below in Table 4.

**Table 4: Plaxis Input Soil Parameters (in situ Profile)**

<table>
<thead>
<tr>
<th></th>
<th>3.5 m layer</th>
<th>1.5 m layer</th>
<th>14.1 m layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c'$ (kPa)</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>$\Phi$ (degrees)</td>
<td>23</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>$E_s$ (kPa)</td>
<td>1650</td>
<td>2400</td>
<td>1260</td>
</tr>
<tr>
<td>$K$ (m/day)</td>
<td>$8.64 \times 10^{-5}$</td>
<td>$8.21 \times 10^{-5}$</td>
<td>$3.88 \times 10^{-5}$</td>
</tr>
<tr>
<td>$\Psi$ (degrees)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>$\gamma_d$ (kN/m$^2$)</td>
<td>10.2</td>
<td>11.4</td>
<td>11.0</td>
</tr>
<tr>
<td>$\gamma_s$ (kN/m$^2$)</td>
<td>16.4</td>
<td>17</td>
<td>17.2</td>
</tr>
</tbody>
</table>

The input parameters for the soil / cement columns have been adopted from the work by Han et al. (2007), Yi et al. (2006) and Abusharar et al. (2009). The parameters for both the embankment fill material and the soil / cement columns are presented below in Table 5.

**Table 5: Plaxis Input Soil Parameters (Embankment and Columns)**

<table>
<thead>
<tr>
<th></th>
<th>Embankment Fill</th>
<th>Soil / Cement Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c'$ (kPa)</td>
<td>12.5</td>
<td>150</td>
</tr>
<tr>
<td>$\Phi$ (degrees)</td>
<td>30</td>
<td>35</td>
</tr>
<tr>
<td>$E_s$ (kPa)</td>
<td>7000</td>
<td>30000</td>
</tr>
<tr>
<td>$K$ (m/day)</td>
<td>1</td>
<td>$3.88 \times 10^{-5}$</td>
</tr>
<tr>
<td>$\Psi$ (degrees)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>$\gamma_d$ (kN/m$^2$)</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>$\gamma_s$ (kN/m$^2$)</td>
<td>22</td>
<td>21</td>
</tr>
</tbody>
</table>

The results of the Plaxis analysis are to be compared to actual settlement measurements taken on an embankment on the Port of Brisbane Motorway, where the assumed subsurface profile closely matches the in situ soil conditions. The embankment was constructed in one stage to a height of 3.1 metres in a linear operation over a period of 37 days. The results from a Settlement Gauge installed during construction (SG1) are employed for comparison purposes.

**PLAXIS MODELLING**

The insitu soils layers were modelling employing 15 node triangular elements. The boundary conditions were modelled employing standard fixities, thus the base line of the geometry is fully fixed in both the horizontal and vertical directions. This will simulate the nearly incompressible sandy gravel layer that is not present in the FEM. However the left and right boundaries of the geometry have a roller condition and are free to move vertically but not horizontally. The far right boundary was extended to approximately 5 times the width of the embankment to avoid any
effect the horizontal fixity will have on settlement. The Mohr-Coulomb’s elastic perfectly plastic model was used to simulate the soil and soil cement column behaviour. After the soil parameters were input, a finite element mesh was generated using the coarse setting.

Analysis Procedural Summary:

- **Boundary Conditions** – Due to symmetrical left hand boundary which allows no flow, the boundary has been set to a closed consolidation boundary condition, as is the far right boundary. The bottom boundary has been left open to allow dissipation into the gravel layer.

- **Stresses** are generated by applying soil self-weight in the first calculation stage. Plastic calculation is employed with the loading input set to Total Multipliers and $\Sigma$Mweight set to 1.0. Once the initial stresses had been generated, the displacements were reset to zero at the start of the next calculation phase.

- **After gravity loading**, the construction phase is instigated. The loading input is set to staged construction and a time interval of 37 days was input to simulate the construction period. The embankment cluster was activated.

- **On completion of the construction phase**, the consolidation phase is begun. The Loading input is set to minimum pore pressure and the default value of 1 kN/m² was unchanged, as values less than this can cause problems with the analysis. As large settlements were expected to occur, the updated mesh and updated water pressures options were selected in the untreated model. Selecting these options helps simulate the soil that was originally above the phreatic level, settling below the phreatic level.

**PARAMETRIC STUDY**

An area replacement ratio was adopted in accordance with Bergado (1996). As the column layout was square, the area replacement ratio was calculated from:

$$a_s = \frac{\pi}{4} \left( \frac{D}{S} \right)^2$$

where $D =$ the column diameter and $S =$ the centre to centre column spacing.

To determine the effect of centre to centre spacing has on settlement in South East Queensland Estuarine Clays, models have been run with 2, 3 and 4 metre spacings with column diameters from 500 to 1000 mm. The models were terminated when pore pressure reached the previously input 1 kN/m² limit indicating primary consolidation had been reached (or $t_{90}$ had been achieved). Table 7 below presents actual area replacement ratios for each spacing and diameter.
Table 6: Area Replacement Ratios for varied Centre to Centre Spacing’s

<table>
<thead>
<tr>
<th>Diameter (m)</th>
<th>Spacing = 2m</th>
<th>Spacing = 3m</th>
<th>Spacing = 4m</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.196</td>
<td>0.087</td>
<td>0.049</td>
</tr>
<tr>
<td>0.9</td>
<td>0.159</td>
<td>0.070</td>
<td>0.040</td>
</tr>
<tr>
<td>0.8</td>
<td>0.127</td>
<td>0.056</td>
<td>0.031</td>
</tr>
<tr>
<td>0.7</td>
<td>0.096</td>
<td>0.043</td>
<td>0.024</td>
</tr>
<tr>
<td>0.6</td>
<td>0.071</td>
<td>0.031</td>
<td>0.018</td>
</tr>
<tr>
<td>0.5</td>
<td>0.049</td>
<td>0.022</td>
<td>0.012</td>
</tr>
</tbody>
</table>

The results of the settlement modelling are presented in Figures 1, 2 and 3 below.

Figure 1: Settlement - 2m Centre to Centre Soil Cement Columns
Figure 2: Settlement - 3m Centre to Centre Soil Cement Columns

Figure 3: Settlement - 4m Centre to Centre Soil Cement Columns
For comparative purposes, the settlement curves for 2 metres spacing, 500 mm diameter, 3 metres spacing 700 mm diameter and 4 metre spacing 1000 mm diameter, with area replacement ratios of 0.49, 0.43 and 0.49 respectively, are presented in the following graph.

**Figure 4: Settlement – Comparison of Similar Area Replacement Ratios (a_s)**

From the above Figure it can be seen at that for a given a_s value, a considerable variation can be expected in the application of soil cement columns in a typical Queensland estuarine deposit. The table below presents interpolation of the three settlement curves after 5000 days.

**Table 7: a_s Comparison - Settlement at 5000 days**

<table>
<thead>
<tr>
<th>Column Spacing</th>
<th>Diameter</th>
<th>a_s</th>
<th>Settlement at 5000 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 m</td>
<td>500 mm</td>
<td>0.049</td>
<td>-129.6 mm</td>
</tr>
<tr>
<td>3 m</td>
<td>700 mm</td>
<td>0.043</td>
<td>-147.5 mm</td>
</tr>
<tr>
<td>4 m</td>
<td>1000 m</td>
<td>0.049</td>
<td>-162.7 mm</td>
</tr>
</tbody>
</table>

From the above table it can be seen at 5000 days, an increase in settlement of approximately 25% can be expected for the given a_s of 0.049 when using 1000mm diameter columns at 4 metre spacing when compared to 500 mm columns at 2 metre spacing.
Settlement Gauge 1 was installed on a section of the constructed embankment, in the region that the sub-surface profile adopted for this study was taken from. The embankment was constructed employing vertical drains at 3 metre centres as pore pressure release mechanisms. The settlement gauge was monitored for a period of 889 days and hence the modelled data presented above has been limited to this time period for the purpose of comparison. By reference to Figure 5 above, it can be seen that there is a considerable reduction in settlement, as would be expected, when comparing the actual embankment settlement to the modelled data. The modelled data shows settlement to be of the order of 18% when compared to the use of vertical drains.

For the purpose of the analysis being performed the term $U_r$ has been adopted to describe the Consolidation Settlement Ratio. Where:

$$U_r = \frac{U_t}{U_u}$$  \hspace{1cm} (4)

where $U_t$ is the settlement of the treated soil profile
$U_u$ is the settlement of the untreated soil profile

Figure 6 presents the Consolidation Settlement Ratio ($U_r$) in terms of Area Replacement Ratio for soil cement columns treated 10% cement (50750 kPa) and 15% cement (65000 kPa) cured for 2 months. From the figure it can be seen that there is very little difference in the
Consolidation Settlement Ratio with regard to undrained elastic modulus, especially at the higher values and the data appears that these values can be conjoined for practical purposes.

**Figure 6: Comparison of Cement Columns with 2 Months Curing**

Based on the observed behaviour it appears that the principal property of the constructed profile controlling the overall consolidation settlement is the Elastic Undrained Modulus of the treated columns. This reflects work done by others showing the increase in strength in a soil profile resulting from the installation of soil columns (Porbaha *et al.* 1998a, Holm 2005, Bruce 2001). Based on this, the following nomograph has been prepared.
Figure 7: Nomograph for Primary Consolidation Estimation

It should be noted that reduction in primary consolidation shows only a small improvement for an Area Replacement Ratio in excess of 0.1 for all cases. Increasing column diameter or decreasing column spacings over this value may not be economically desirable.

CONCLUSIONS

This paper aims to examine the effect of cement replacement in compressible soil. The results of the parametric study revealed that the use of soil cement columns would be an effective settlement control solution in the deep estuarine deposits of South East Queensland. By varying the column diameter and spacing the effect of the Area Replacement Ratio \( (a_s) \) was evaluated on the overall effect of the column grid with regard to settlement control.

It was found that a considerable decrease in settlement occurred at equivalent Area Replacement Ratios when using smaller diameter columns at closer centre to centre spacings when compared to larger diameter columns at higher spacings.

The study also shows that the use of soil cement columns results in overall settlements of below 20% of the actual measured embankment settlement. The resulting reduction in settlement could result in a significantly reduced delay in the construction sequence.
The primary consolidation can be estimated in South East Queensland typical compressible clay profile subjected to typical roadway embankment loads if the modulus of soil / additive column and the Area Replacement Ratio is known.

REFERENCES


