Interpreting Field Behaviors of Embankment on Estuarine Clay

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ABSTRACT

In this paper, field behaviors of two trial embankments in Australia are presented. The first case study has a trial embankment which is approximately 90m in length and 40m in width, and has two sections with vertical drain installed, and a no drains section. The second case study gives detail of a trial embankment with stone column, and incorporated 3 separate sections (2 sections with stone columns, and a section without stone column), and is constructed on soft estuarine clay with high sensitivity. The trial embankments were constructed to evaluate the effectiveness of ground improvement techniques on the soft clays in this region. This paper interprets the findings obtained from the field observations during the construction phase.

KEY WORDS: Soft clay; trial embankment; stone columns; vertical drains.

INTRODUCTION

Soft clays are found in many projects in Australia, and they pose difficult problems in the design and construction of roads, expressways and motorways. By definition, soft clays are of low shear strength and high compressibility. Generally, they are sensitive and their strength is readily reduced by disturbance during sampling and testing. Such sub-soil conditions can have considerable implications on the design of embankments and structural foundations. This is due to both low shear strength and a tendency to deform with time. The simplest solution to such unfavorable soil conditions is to find an alternate alignment, although this can be costly and impractical. As an economic alternative to structural foundations, ground improvement techniques are becoming more prevalent. Ground improvement in Australia primarily encompasses the use of stone columns, surcharge with vertical drains, and chemical stabilization.

This paper presents the soil characteristics of a trial site located in Gold Coast (Southeast corner of Queensland). Included in this paper are the in-situ conditions before the embankment was constructed and the subsequent conditions after the embankment was built. The vertical settlement, horizontal settlement profile, and lateral displacement plots, determined from the in-situ field equipment, are provided.

Further, this paper also presents the laboratory results and field behaviors of alluvial soft clay found in Sunshine Coast of Southeast Queensland. The second test embankment presented in this paper was fully instrumented to measure the settlements, lateral movements and the development of excess pore pressures and their dissipation with time under the embankment load. Also, ground improvement technique using prefabricated vertical drains (PVD) was also evaluated for their potential applications.

EMBANKMENTS WITH STONE COLUMN

Site and Soil Condition

Soft estuarine clay in Southeast Queensland has wide varying engineering properties, depending largely on the deposit's depth below the ground surface and the proximity to the water table. Based on the field shear vane tests conducted on the test site, the undrained shear strength of very soft/soft clays is around 5-20 kPa (as shown in Figure 1). Natural moisture contents commonly vary between 60 and 120%. The liquidity indices are generally in the range of 1.5 - 2.5, displaying high sensitivity. Compressibility as high as $C_c/(1 + e_o) = 0.4 - 0.5$ has been observed in the laboratory. At this high compressibility, strain rate effects can be significant.

The trial embankment (approximately 90m in length and 40m in width) was built along the deepest section of the very soft to soft organic clay layer, which extended to a maximum depth of 13.5m. Underlying this layer is a moderately dense to dense sandy sediment strata. On either side of these strata are stiff-hard clay/silty clay. The trial embankment was divided into three sections – section (1) contained no stone column, section (2) had stone columns at 2m spacing, and section (3) had stone columns at 3m spacing. The stone columns were constructed in a square pattern with column diameter of 1m and column length of 16m.

Further, the stone columns were installed with a jetting process (that is, using vibroflotation). The trail embankment was constructed in two stages. Numerous bore holes were drilled along the site where the trial embankment was built. From the bore holes, undisturbed soil samples were taken, at various depths, to determine the nature of the soil stratum. Laboratory tests were used to establish the wet density (γ_{wet}) of the soft organic clay. The undrained shear strength (S_u) of the soft clay was also determined at various depths (as shown in Figure 1(b)). It varied with depth, ranging from 5 to 20 kPa. Clays with undrained shear strength less than 20 kPa are considered very soft. It is also seen that the sensitivity of the soft clay reduced with depth (see Figure 1(c)). Clay with a sensitivity value between 4 and 8 is considered sensitive; therefore the top 4 m of the soft clay stratum is considered extra sensitive.

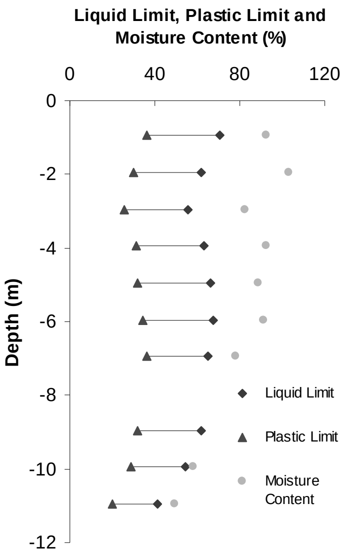

Figure 1(a) Liquid limit, moisture content and plastic limit profile

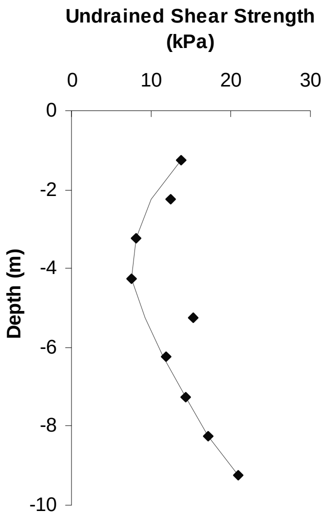

Figure 1(b) Undrained strength profile

Oedometer consolidation tests were also undertaken, in the laboratory, to assess the compressibility characteristics of the soft organic clay. Figure 2 illustrates the compression curves for five different soil samples taken from bore holes located along the test site. From these curves the coefficient of volume decrease (m_v) and coefficient of consolidation (c_v) can be determined. Based on the results, the compressibility of the soft clays ranges from 0.5 to 3.5 m^2/MN. The compressibility profile shows that the soft clay deposit becomes less compressible with depth. Following the results of the oedometer tests, the coefficient of consolidation (c_v) values vary from 0.17 to 2.68 m^2/year, with the majority of values between 0.2 to 0.3 m^2/year.

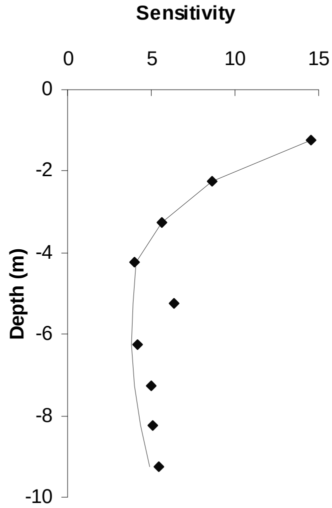

Figure 1(c) Soil sensitivity profile

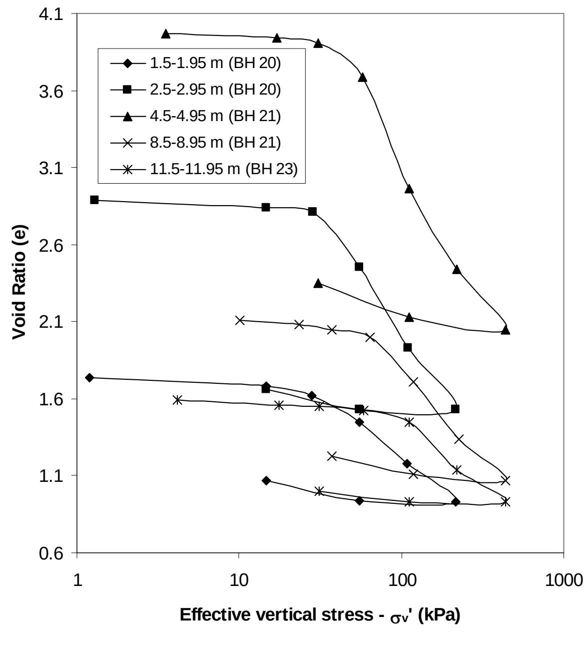

Figure 2 Oedometer consolidation test results

Field Instrumentations and Performances

During construction of the trial embankment, field instrumentation was installed to monitor its performance. The following instrumentation was installed:

1. Settlement gauges
2. Horizontal profile gauges
3. Inclinometers
4. Piezometers

Settlement gauges were installed, at the centre line of the embankment, to monitor vertical settlement. Across the base of the embankment, horizontal profile gauges were installed to record the horizontal settlement profile of the embankment. Inclinometers were installed at the toe of the embankment to monitor lateral displacement. Piezometers were installed at the centre line of the trial embankment to monitor pore pressure dissipation.

The vertical settlement profiles at various distances along the three sections of the embankment are shown in Figures 3, 4 and 5. These settlement profiles were obtained from horizontal profile gauges installed beneath the embankment. Typically vertical settlement gauges are used to measure settlement at the centerline of the trial embankment, and the readings are shown in Figure 6. The final settlement readings obtained after 485 days of monitoring are shown in Table 1. A comparison between the horizontal profile gauge and vertical settlement gauge readings verifies these results. These readings indicate that stone columns had practically no impact on reducing settlement.

Table 1 Summary of settlement results

Trial Embankment (stone column spacing)	Maximum Horizontal Profile Gauge Reading (mm)	Maximum Vertical Settlement Gauge Reading (mm)
3m spacing	490	508
2m spacing	386	450
No treatment	522	508

The in-situ settlement time plots at the centre line of each embankment are shown in Figure 6. These plots illustrate the ground level settlement of the embankment with Figure 6(a) plotted using Casagrande's log time method, and Figure 6(b) plotted using Taylor's square root of time method. The ground level settlement data fits Taylor's method better than Casagrande's method. Both figures illustrate that primary consolidation has not yet been completed. These figures illustrate that installing closely spaced stone columns reduces the amount of ground level settlement. At square root time 22 days the embankment without ground improvement and the embankment with stone columns at 3m spacing had the same ground level settlement.

The subsurface settlement beneath all three embankments decreased with depth. This was expected because the upper weathered crust of the soft clay layer has low strength and is highly compressible. At square root time 22 days the embankment with no ground improvement had the greatest subsurface settlement. At the same time interval, the embankment with stone columns at 2m spacing had reduced settlement compared to the embankment with stone columns at 3m spacing. The 3m and 2m spaced stone column performance are comparable. Figures 6(a) and 6(b) showed the ongoing settlements. Thus, the use of stone columns at the test site does not reduce settlements or consolidation time.

Typically, inclinometers are installed to measure horizontal sub-ground level movements. At the test site, lateral displacement was monitored at the trial embankment toe and is shown in Figure 7. The maximum lateral displacement at the toe was 76.84 mm, and is seen in the sensitive upper layers.

The embankment which has the greatest lateral displacement is the section with no ground improvement. Installing stone columns at 2m spacing reduces the amount of lateral displacement, approximately by half, when compared to the embankment with no ground improvement. The lateral displacement of the embankment with stone columns at 3m spacing had slightly higher displacements when compared to the embankment with stone columns at 2m spacing. Thus, installing stone columns beneath the trial embankment reduces lateral displacement.

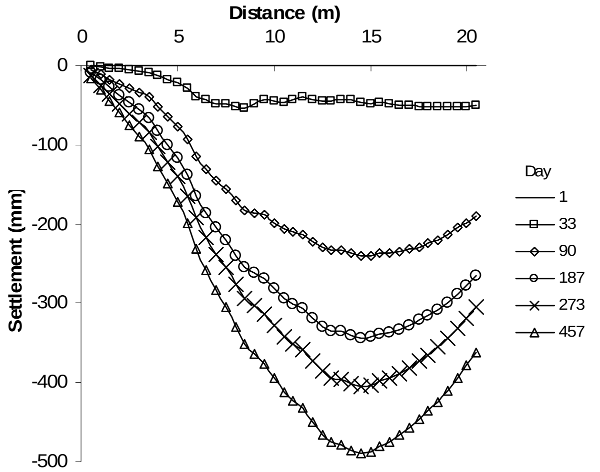

Figure 3 Measured settlements for embankment treated with 3m spaced stone column.

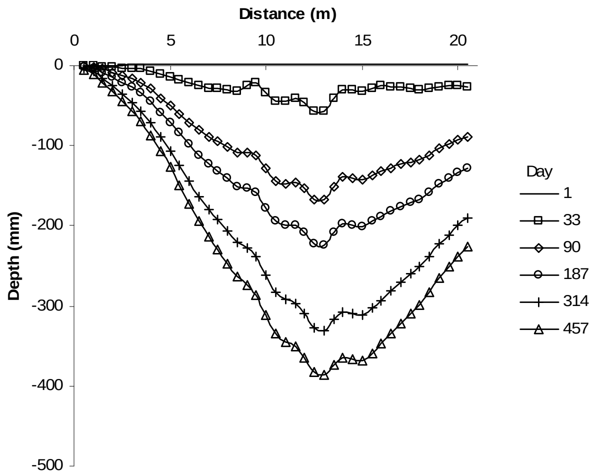

Figure 4 Measured settlements for embankment treated with 2m spaced stone column.

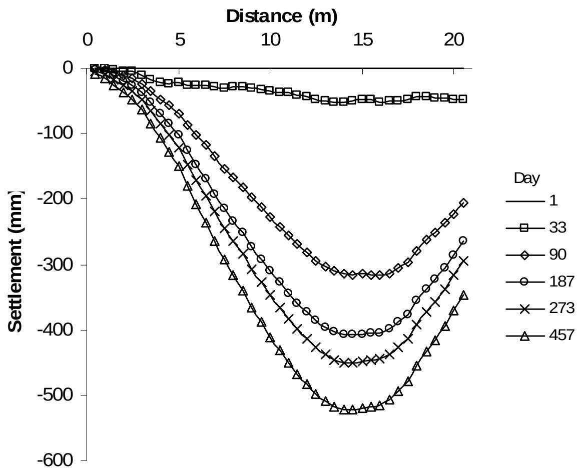

Figure 5 Measured settlements for embankment without stone column (no treatment)

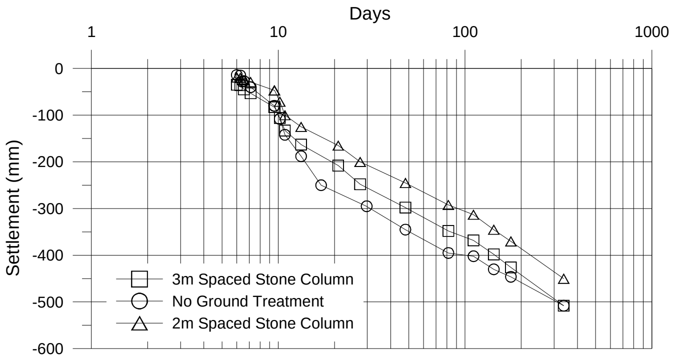

Figure 6(a) Measured settlements at centreline of embankment (Casagrande's method)

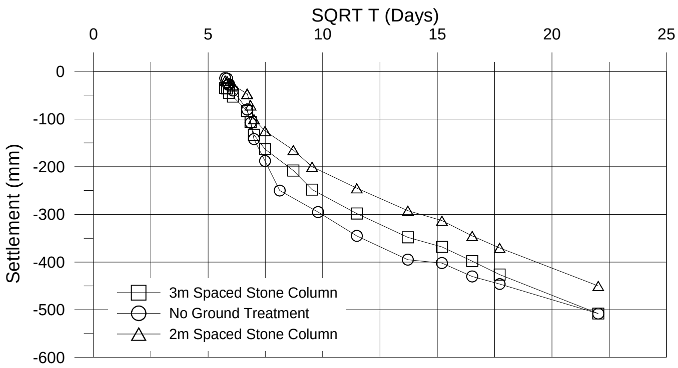

Figure 6(b) Measured settlements at centreline of embankment (Taylor's method)

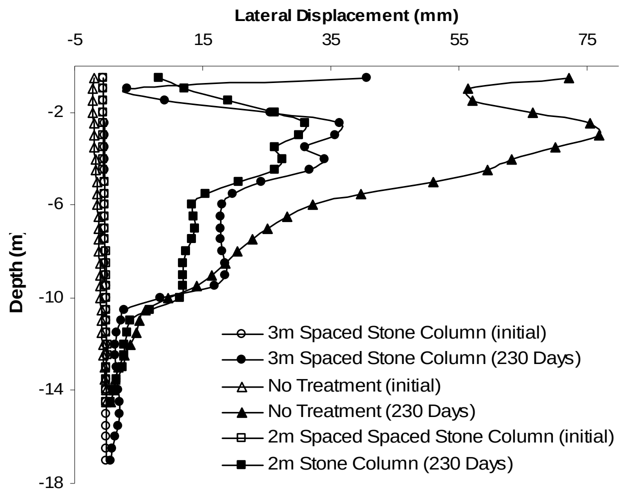

Figure 7 Inclinometer movements at toe

Stone columns are inherently non-homogeneous as their modulus of deformation increases with the confining stress, which is depth dependent. When an embankment is constructed over the soft ground, lateral spreading occurs beneath the embankment, and would reduce the confinement of the stone column. It is postulated that, bulging of stone columns is due to lack of lateral confining resistance and this is observed in the lateral displacement between depths 2 to 4m (see Figure 7). Further, the stone column installation had disturbed the sensitive soft clay, which had reduced the soil strength. Therefore, the variations of displacements with depth of the stone column are affected by the stone column non-homogeneity and sensitivity of the soft clay.

EMBANKMENTS WITH PVD AND PRELOADING

Site Description and Soil Conditions

The soil strata can be classified into several layers. Field testing has indicated a substantial deposit of very soft compressible organic silty clay between 4m and 10m thick. This material is underlain by a layer of very loose silty sand of approximately 2 m thick. This in turn is underlain by moderately dense to dense sand (coffee rock) strata of 4 m to 6 m thick.

Figure 8 indicates typical sub-soil layers in the trial embankment area. In this figure, silty clay (CH) is found up to about 8m depth followed by clayey silt (MH), silty clay (CH) and clayey sand up to 12m depth. The natural water content of these layers were substantially higher than the liquid limit and the highest water content of 120% is found at 2m - 6.5m depth, followed by lower water content of about 80% from 6.5 m to 12 m depth. As such the weakest soft clay is encountered at 2 m - 6.5 m depth and this layer is bound to be of low shear strength as revealed from the natural water content. The liquidity index of the clay is higher than 1.0 as the natural water content is higher than the liquid limit. The plasticity index of the clay is uniform with depth and it is about 40%.

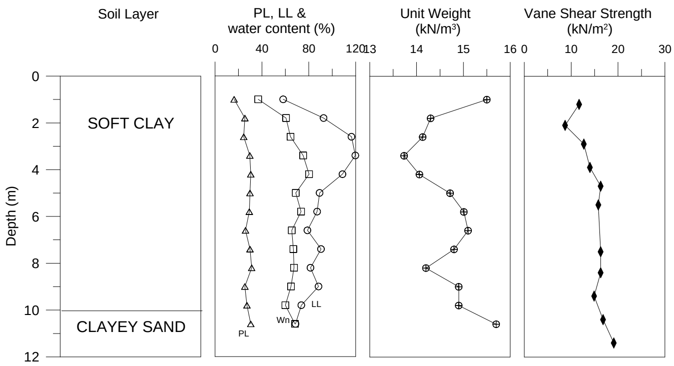

Figure 8 Index properties, unit weight and vane shear strength

The voids ratio-effective vertical stress relationships are presented in Figure 9. The compression indexes for all samples were found to increase with effective vertical stress initially and thereafter, it remains constant. Depending on the initial water content of the samples, the C_c vales generally ranged from 0.5 to 1.0 at stresses in the normally consolidated range. The exception was the sample taken at depth of 8.2 m to 9.0m, which indicated a C_c value of 1.5 in the normally consolidated range of stress levels.

Instrumentations and Performances

The test embankment is approximately 90m in length and 36m in width. Vertical drains were installed with a crawler-mounted machine from the working platform. Section A and C of the test embankment are the prime sections representing the design vertical drain spacing (1m of triangular pattern) and no drains respectively. Section C representing an intermediate case with 2m triangular pattern drain spacing and less instruments

Horizontal profile gauges were installed in Section A and Section B to monitor the settlements across the sections with varying time period ranging from one day to 724 days. These hydraulic profile gauge data are presented in Figure 10 for Section A and Section B, and they were interpreted extensively. In the interpretation of these hydraulic profile gauges, there are 2 observations.

Firstly, Section A with PVD, showed higher settlement at any time when compared with Section B without PVD. This is illustrated in Figure 10 for time period of approximately, 1 day, 62 days, 93 days and 533 days. At all times, Section A experienced higher settlement than Section B. Also individual settlement–log time plots are plotted for all sections show that Section A experienced greater settlement than Section B.

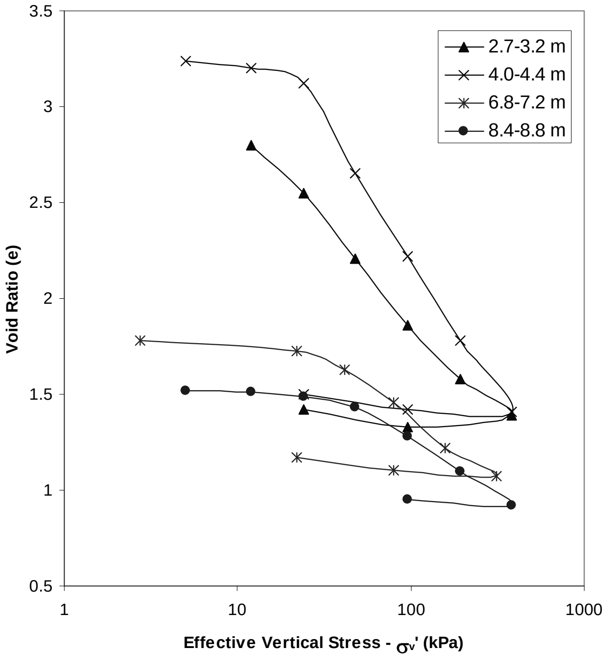

Figure 9 Voids ratio – effective vertical stress

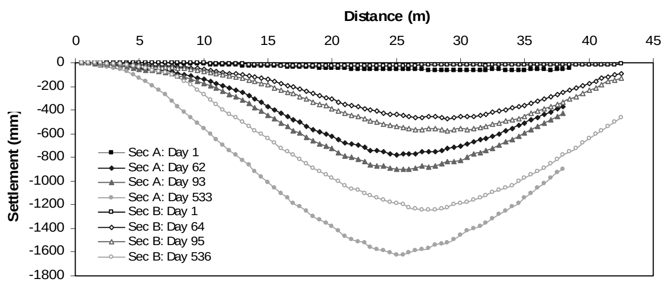

Figure 10 Surface settlements from horizontal profile gauge in Section A and Section B

Secondly, Section A with the PVD, the settlement-log time plots indicated that 100% primary consolidation is over virtually for all sections, to the left of the centre line. But these plots indicate that the section to the right of the centre line were unable to achieve 100% primary consolidation even with the use of PVD in closer spacing of 1.0m. This observation is rather difficult to comprehend.

The above observation was established systematically, by plotting the settlement-log time plot of each location separately to the left and to the right of the centre line of Section A. The settlements for the centre line and the locations to the left are shown in Figure 11; those corresponding to the right are shown in Figure 12. The location marked EE corresponds to the centreline of the embankment. The locations DD, CC, BB and AA were taken from the centre line to the left at distances of 5m, 10m, 15m and 20m respectively. Similarly the locations FF, GG and HH were on the right hand side and at distances of 5m, 10m and 15m respectively away from the centreline. The settlement-log time plot of each separate location to the left and to the right of the centre line of Section B is given in Figure 13. When the settlement-log time plot do not show an *S* curve, and where the Casagrande method cannot be applied to estimate the 100% primary consolidation, the Aasoka (1978) method was used to estimate the 100% primary consolidation. It was observed later that the Asaoka Method at times give lower values for the ultimate settlement.

Tables 2 and 3 summarise the 100% settlement estimated from the Casagrande and Asaoka methods for Section A and Section B respectively. These values indicate that even Section B is having proportionately higher settlements, even though no PVD were used. The major reason for this was perhaps due to the fact that the PVD Section A and Section C are on either side of the section B which have no PVD. Earlier work carried out at other sites in Southeast Asia and elsewhere indicated a better arrangement would have been to separately locate the PVD Sections and the no-PVD Section so that there is no interference effect. By not doing so at the test embankment presented here, the lateral drainage from the Section with no PVD (Section B) and through silt and sand lenses to the PVD in the drained sections have possibly occurred.

Table 2 Ultimate settlement (100 percent consolidation settlement) in Section A

Location	Casagrande's Method 100 % Settlement (mm)	Asaoka's Method 100 % Settlement (mm)
CL	1570	-
5m Left from CL	1320	-
10m Left from CL	885	-
15m Left from CL	380	-
20m Left from CL	-	-
5m Right from CL	-	1506
10m Right from CL	-	1316
15m Right from CL	-	970

Table 3 Ultimate settlement (100 percent consolidation settlement) in Section B

Location	Casagrande's Method 100 pc Settlement (mm)	Asaoka's Method 100 pc Settlement (mm)
CL	-	1200
5m Left from CL	-	1050
10m Left from CL	-	480
15m Left from CL	-	290
20m Left from CL	-	-
5m Right from CL	-	1200
10m Right from CL	-	1060
15m Right from CL	-	830

Lateral deformation profiles were determined at locations toe of embankment. However, only in Section A and Section B inclinometer casings were installed at the embankment toe. Figure 14 illustrates the lateral deformation profiles for Section A and Section B after one day, 62 days, 93 days and 526 days. Initially, Section A with PVD was found to develop more lateral deformation. Perhaps this may be due to the disturbance created by the installation of PVD. However at 526 days time both Section A and Section B have similar lateral deformation profiles.

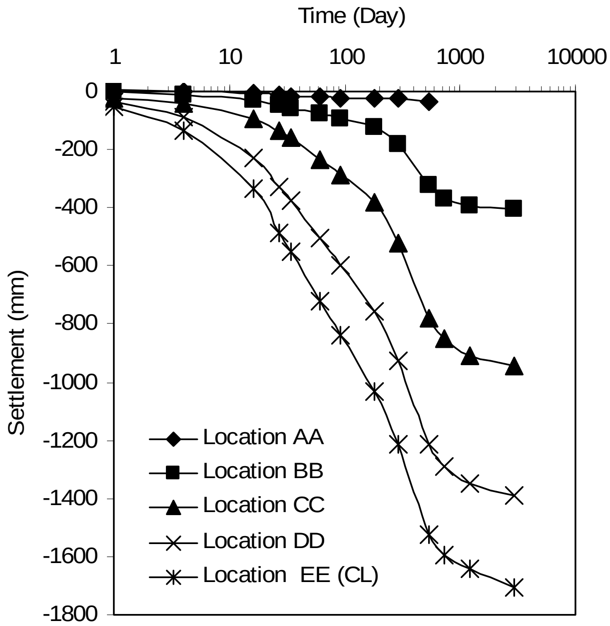

Figure 11 Variation of settlement with time along the centerline and the locations to the left in Section A (PVD at 1m spacing, 100% Consolidation Completed)

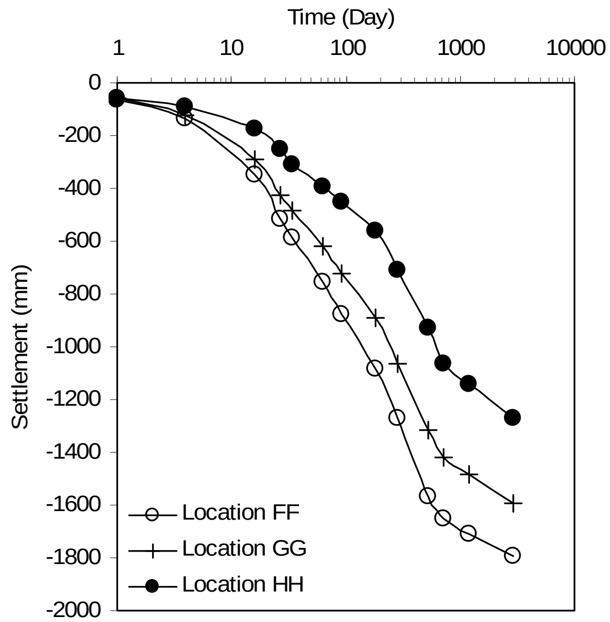

Figure 12 Variation of settlement with time at the locations to the right of centerline in Section A (PVD at 1m spacing, 100% Consolidation Not Completed)

The excess pore pressures were determined along three alignments in Section A. The construction sequence adopted is shown in Figure 15. The excess pore pressures as indicated by piezometers PVA4, PVA10 and PVA 14 are shown in Figure 16. These piezometers are located around 5.5m depth and PVA4 is along the centre line of the Section, while PVA10 is along the location XX and PVA 14 is between locations XX and YY. It is noted that the piezometer PVA4 along the centre line indicates maximum excess pore pressures and this is followed by piezometer PVA 10 and the least excess pore pressure was indicated by piezometer PVA 14. These measurements are in accordance with the excess stress at these points due to the embankment loading.

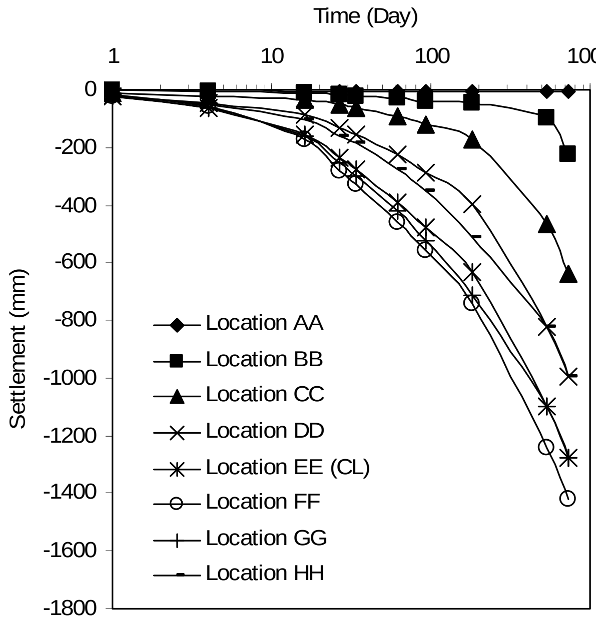

Figure 13 Variation of settlement with time in Section B (no treatment)

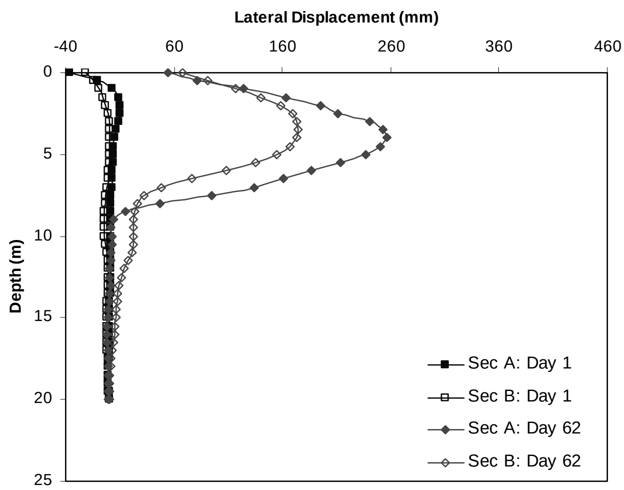

Figure 14(a) Variation of lateral displacements in Section A (PVD at 1m spacing) and Section B (no treatment) before end of construction

For Section B, excess pore pressures are plotted in Figure 17 along the centre line as indicated by piezometers PPB21 and PPB23 located at 4.5m and 9.5m depth respectively. Both piezometers indicate similar development of excess pore pressures and also dissipation pattern. Unlike the piezometers in Section A with PVD, the piezometers PPB21 and PPB 23 in Section B with no PVD did not indicate faster dissipation of excess pore pressures.

The pore pressure dissipation in Section A with PVD is faster than the corresponding dissipation Section B with no PVD. Due to space limitations, Section C of the embankment is not reported here.

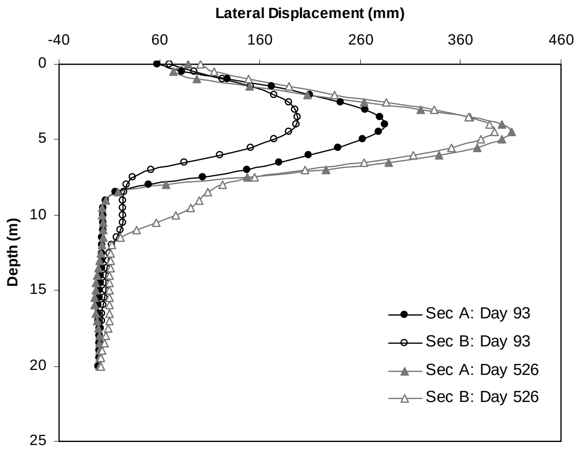

Figure 14(b) Variation of lateral displacements in Section A (PVD at 1m spacing) and Section B (no treatment) after end of construction

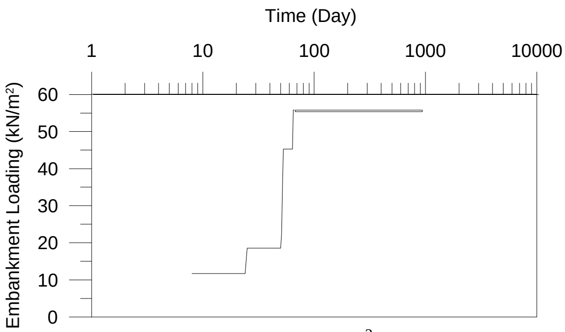

Figure 15 Embankment loading (kN/m^2) with time (days) in Section A

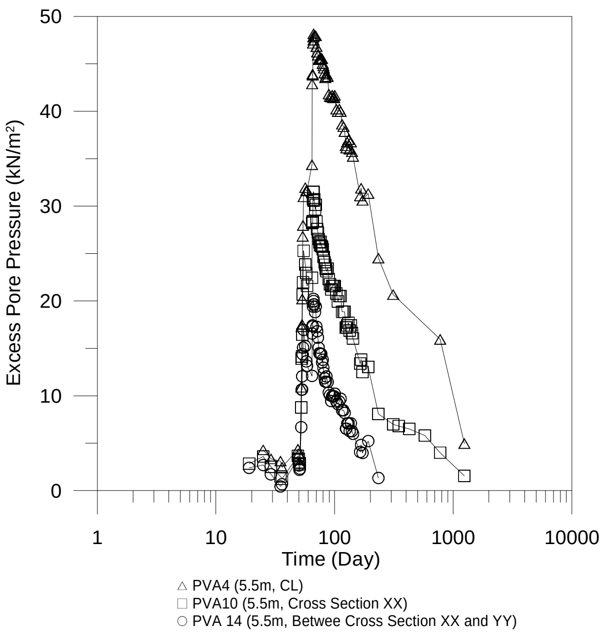

Figure 16 Variation of excess pore pressure with time in Section A (PVD at 1m spacing)

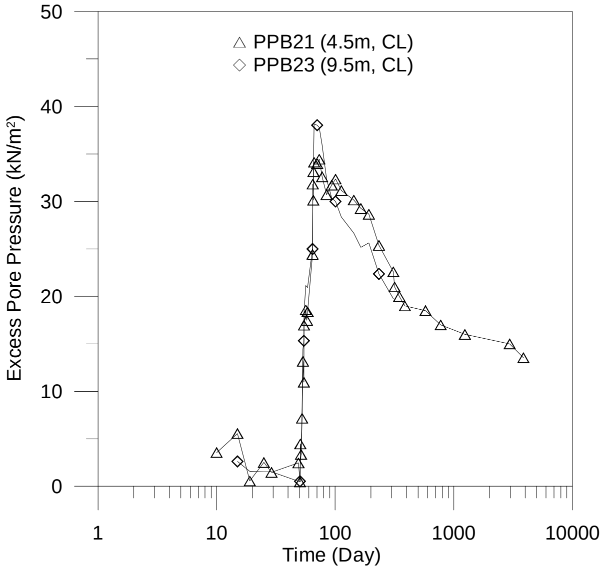

Figure 17 Variation of excess pore pressure with time in Section B (no treatment)

CONCLUSIONS

This paper provides valuable insight into the laboratory and field behavior of soft estuarine clay found in Southeast Queensland when subject to embankment loading. The major emphasis was in the presentation of data collected from the extensive monitoring of two trial embankments constructed with and without ground improvements.

The following concluding remarks can be drawn from the trial embankment with and without stone column:

1. The embankment with stone columns at 2m spacing had the least settlement. The embankment with no ground improvement and stone columns at 3m spacing had comparable settlement. The greatest settlement of the trial embankment occurred at the embankment centre line. Since the width of the embankment is only 20m at the base, the in-situ settlement profile at the centre line of the embankment can not be purely classified as consolidation settlement with respect to the thickness of compressible layer.
2. Inclinometers were installed at the toe of the embankments to monitor lateral displacement. The peak lateral displacement occurred for all three sections between depths 2 to 4m. The soil within this region has extremely low strength and low resistance to lateral movement.
3. Stone column treatment at the test site was ineffective in reducing settlement. Further, it is considered that installation disturbance caused to the sensitive clay diminishes the effect of the columns. In this sensitive soft estuarine clay, the use of stone column has not proven to be effective and the use of such method in similar soil conditions will require careful consideration.

The following remarks can be concluded from the trial embankment with and without vertical drain:

1. The laboratory results indicated that soft clays deposit in the studied areas is very soft and highly compressible. The under lying soils below the trial embankment can be considered as normally to slightly overconsolidated soil.

2. Maximum lateral displacement of the order of 400mm was observed and the lateral displacement is contained in the upper 8 – 10m of soft silty clay, which is susceptible to shear failure
3. The pore pressure dissipation indicated that the settlement measured is largely of the consolidation type.

Due to space limitations, other issues such as review of design methods for stone column and vertical in the light of the trial embankments have not been discussed and will be reported elsewhere

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