

Behaviour of a Highway Embankment on Stone Columns Improved Estuarine Clay

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Abstract: Soft estuarine clays are found in many expressway and motorway projects in Queensland, Australia. The strengths of these clays are very low and as such embankment failures can be encountered during construction. Presently, there are very few case studies presenting projects on estuarine clay in Australia. This paper presents the details of a trial embankment with stone columns in South-east Queensland, Australia. The trial embankment incorporated 3 separate sections i.e. 2 sections with stone columns of 2m and 3m spacing (square pattern), and a section without stone column. It was constructed on estuarine clay with high sensitivity. Undrained shear strength measured from field vane tests varied with depth, ranging from 5 to 20 kPa. Further, the sensitivity of the estuarine clay reduces with depth, ranging from 5 to 12. Analysis of this case study was undertaken using field measurements. The pore pressures responses and deformations below the embankment were investigated. The effectiveness of stone column is also evaluated. This paper gives the details of the ground improvement plan, and presents the findings from measured field values.

1 INTRODUCTION

Soft clays are found in many expressway and motorway projects in Southeast Queensland. 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 consolidate and deform with time. Often the simplest solution to such unfavourable 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 granular stone columns, surcharge with vertical drains, and chemical stabilisation.

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 Fig. 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 $C_c/(1 + e_0) = 0.4 - 0.5$ has been observed in the laboratory. At this high compressibility, strain rate effects can be significant.

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 em-

bankment was built. The vertical settlement, horizontal settlement profile, and lateral displacement plots, determined from the in-situ field equipment, are provided.

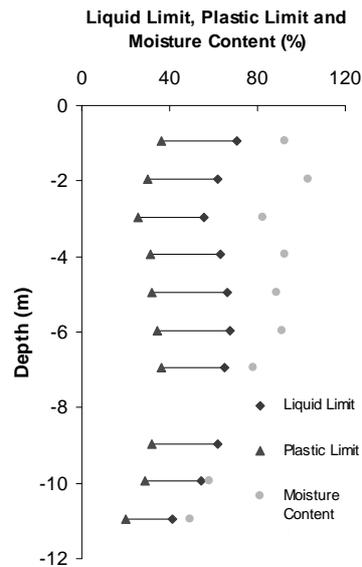


Fig. 1(a) Liquid limit, moisture content and plastic limit profile

2 SOIL CONDITION AND EMBANKMENT GEOMETRY

The trial embankment 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 longitudinal profile of the soil stratum is presented in Fig.2.

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 trial embankment was constructed in two stages. The geometry of the trial embankment and the construction stages are indicated in Fig. 3 illustrated on Fig.3.

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 Fig. 1(b)).

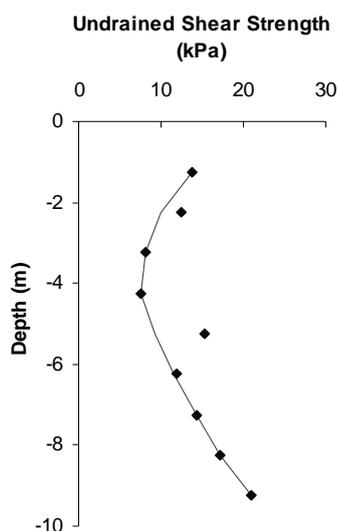


Fig. 1(b) Undrained strength profile

Fig. 1 shows that the undrained shear strength at the test site. 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 Fig. 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.

Oedometer consolidation tests were also undertaken, in the laboratory, to assess the compressibility characteristics of the soft organic clay. Fig.4 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 $m^2/year$.

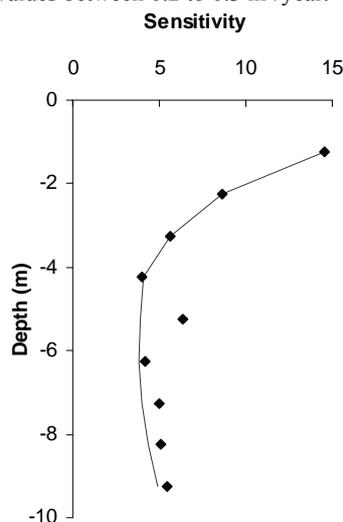


Fig. 1(c) Soil sensitivity profile

3 FIELD INSTRUMENTATION

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. Inclinerometers
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. Inclinerometers 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.

4 FIELD PERFORMANCE OF EMBANKMENT

The vertical settlement profiles at various distances along the three embankments are shown in Figs. 5, 6 and 7. These settlement profiles were obtained from horizontal profile gauges installed beneath the embankment.

Typically vertical settlement gauges are used to measure settlement at the centreline of the trial embankment, and the readings are shown in Fig. 8.

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.

The in-situ settlement time plots at the centre line of each embankment are shown in Fig. 8. These plots illustrate the ground level settlement of the embankment with Fig. 8(a) plotted using Casagrande's log time method, and Fig. 8(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.

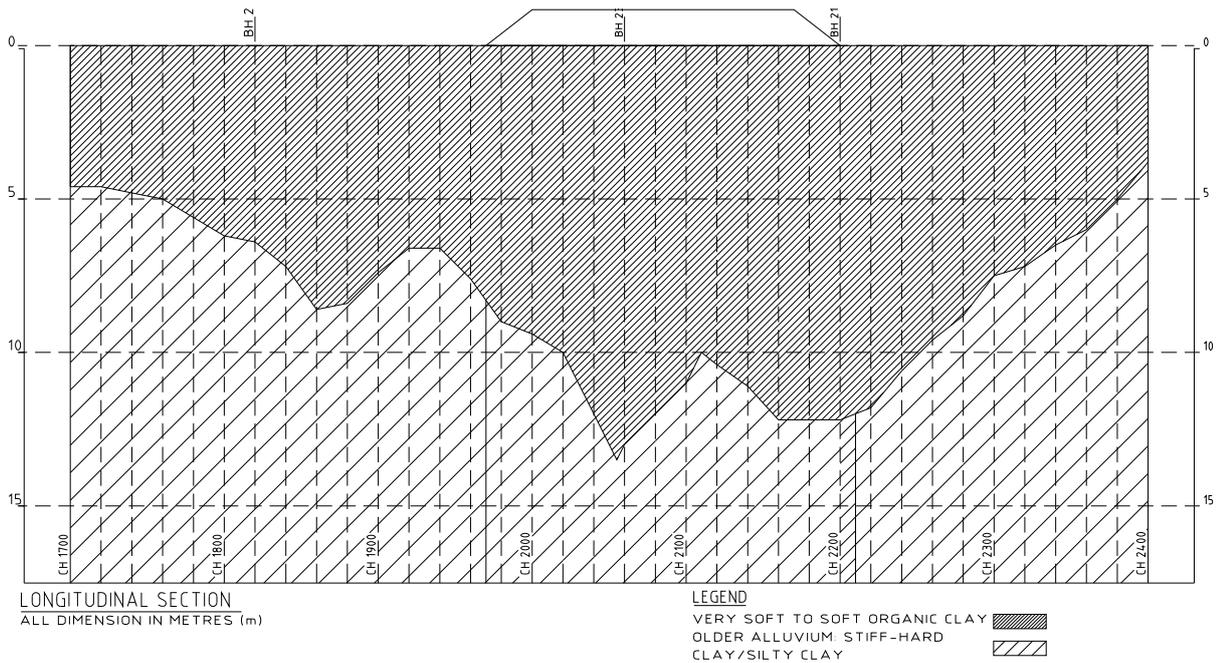


Fig. 2 Longitudinal soil profile of test site

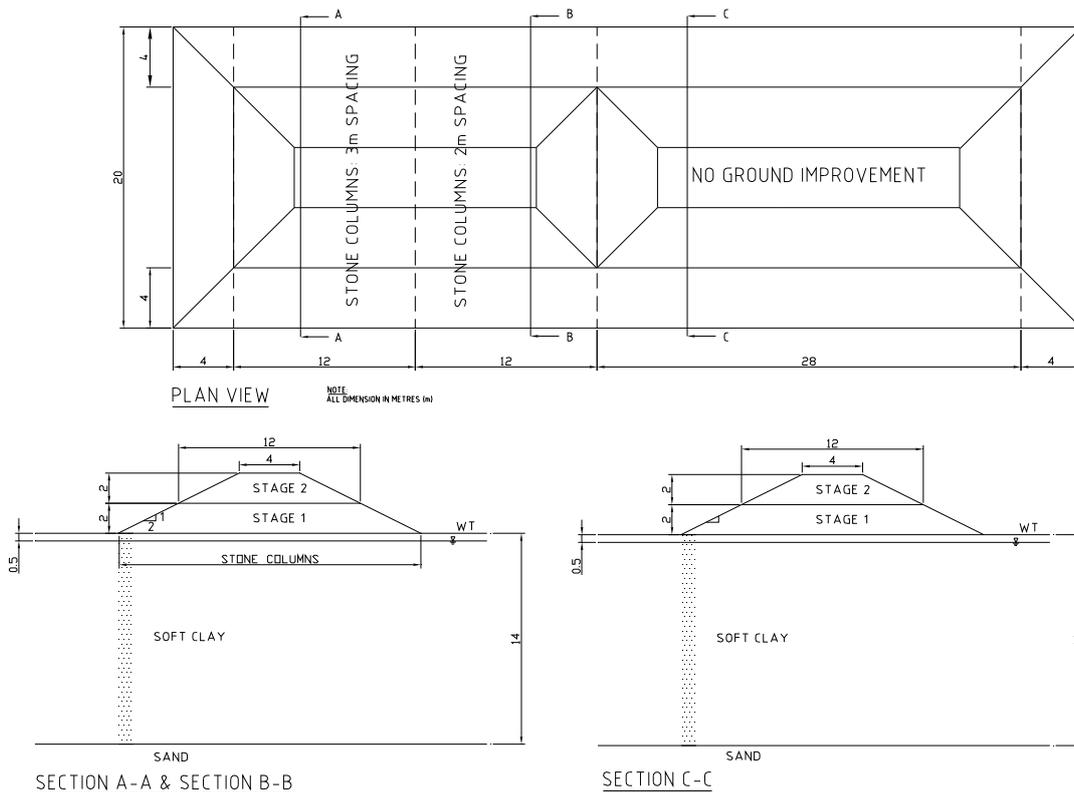


Fig. 3 Trial embankment geometry

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. Figs. 8(a) and 8(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 Fig. 9. The maximum lateral displacement at the toe was 76.84 mm, and is seen in the sensitive upper layers.

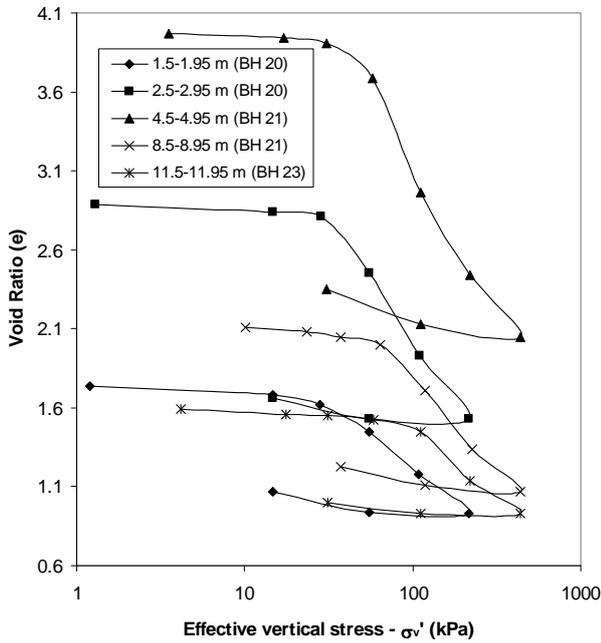


Fig.4 Oedometer consolidation test results

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

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

5 IN-SITU COEFFICIENT OF CONSOLIDATION (C_v)

In 1978, Asaoka presented an approach to estimate the final consolidation settlement (ρ_{oc}) and in-situ coefficient of consolidation (c_v) from settlement time data for a certain time period. The in-situ coefficient of consolidation (c_v) is given by the following expression

$$c_v = -\frac{5}{12} H^2 \frac{\ln \beta_1}{\Delta t} \quad (1)$$

The plots of the settlement values and best-fit straight lines, for the three embankments, are shown in Figs. 10(a), 10(b) and 10(c).

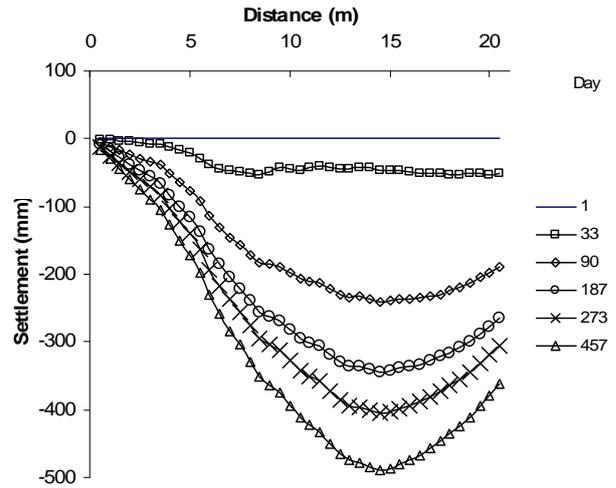


Fig. 5 Measured settlements for embankment treated with stone column at 3m spacing

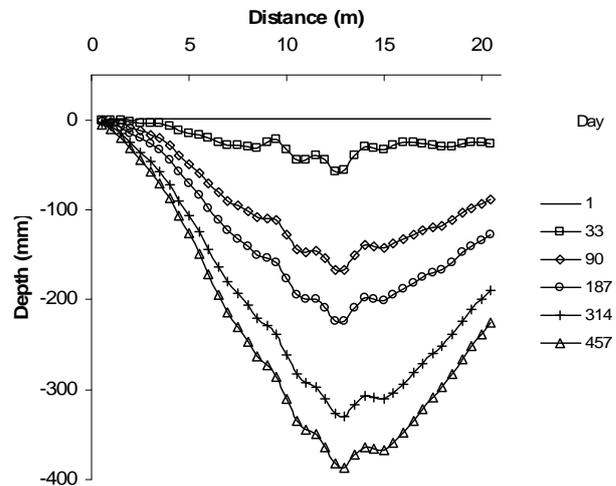


Fig. 6 Measured settlements for embankment treated with stone column at 2m spacing

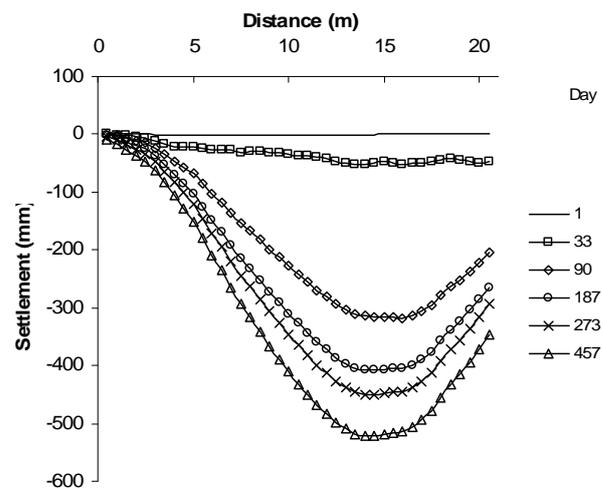


Fig. 7 Measured settlements for embankment without stone column (no treatment)

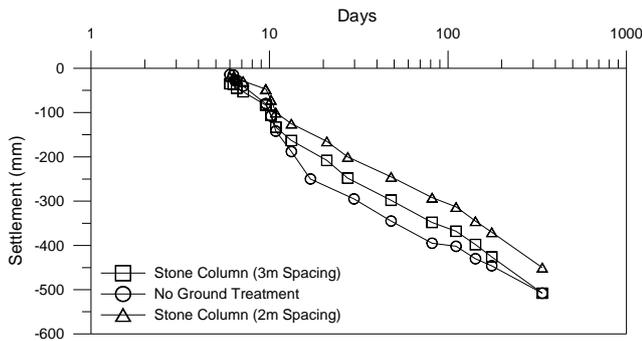


Fig. 8(a) Measured settlements at centreline of embankment (Casagrande's method)

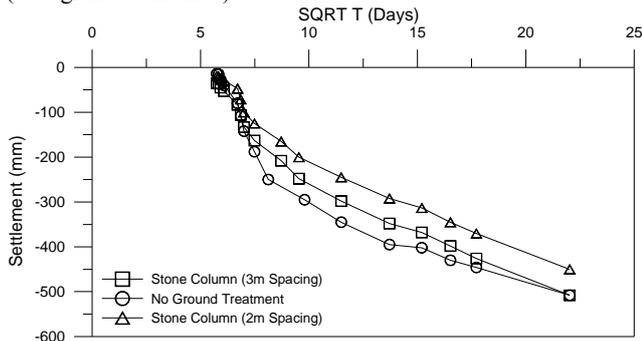


Fig. 8(b) Measured settlements at centreline of embankment (Taylor's method)

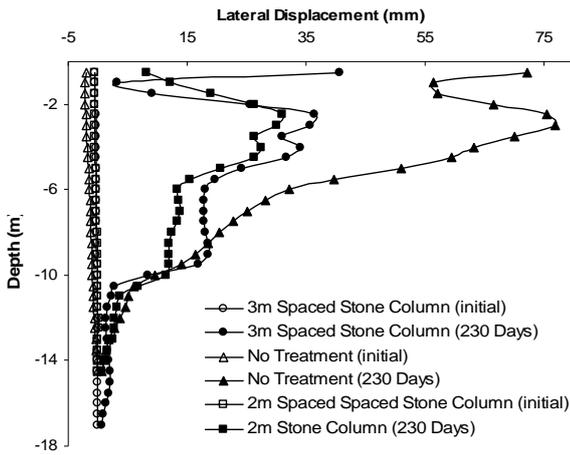


Fig. 9 Incliner movements at toe

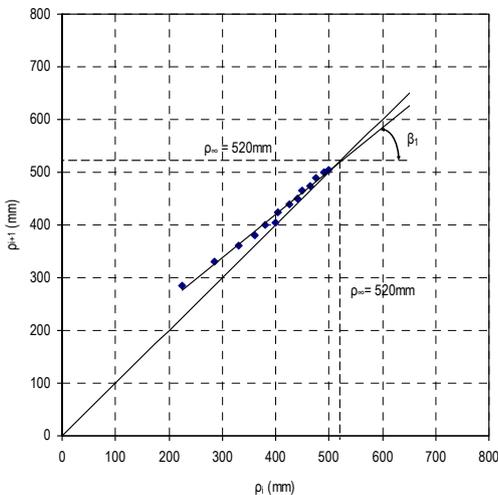


Fig. 10(a) Asaoka's method for graphical evaluation of settlement records for embankment on stone columns at 3m spacing

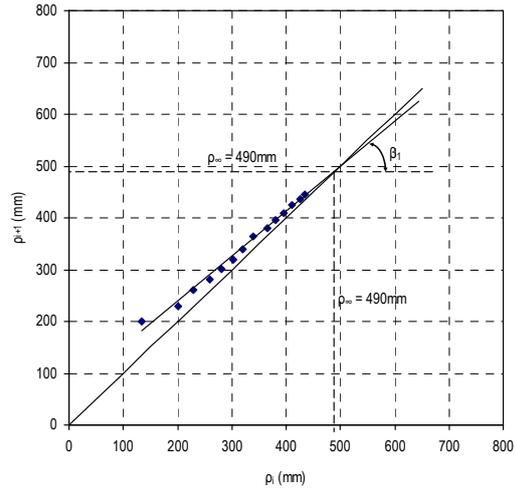


Fig. 10(b) Asaoka's method for graphical evaluation of settlement records for embankment on stone columns at 2m spacing

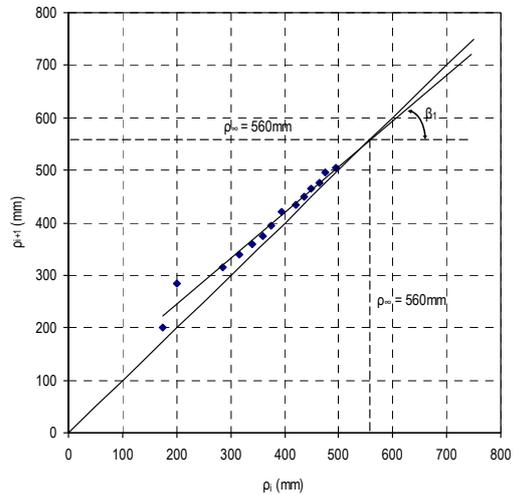


Fig. 10(c) Asaoka's method for graphical evaluation of settlement records for embankment without stone column (no treatment)

6 CONCLUSIONS

This paper provides valuable insight into the laboratory and field behaviour 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 a trial embankment constructed with and without stone column.

Settlement gauges and horizontal profile gauges were installed in the trial embankment to record the variation of vertical settlement with time. 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.

Inclinometers were installed at the toe of the embankments to monitor lateral displacement. The peak lateral displacement oc-

curred for all three sections between depths 2 to 4m. The soil within this region has extremely low strength and low resistance to lateral movement.

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 Fig. 9). 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.

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. Also, reduced column spacing had negligible impact on further reducing settlement, again due to installation disturbance. 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.

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

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REFEERNCES

Asaoka, A., (1978). Observation procedure of settlement prediction. *Soils and Foundations*, Vol. 18, No. 4, pp. 87-101.