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Construction of Road Embankments over Very Soft Soil Using Band Drains and Preloading

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Construction of Road Embankments over Very Soft Soils Using Band Drains and Preloading

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SYNOPSIS: As part of a new national road being constructed in South Africa an embankment was built over a deep deposit of very soft soil. To enhance stability the embankment was built in several stages and in order to reduce the time required for consolidation between stages, band drains were installed in the foundation soils. The soils were instrumented to monitor pore water pressure and settlement. The results of the monitoring phase showed that the band drains were effective and operated as designed.

This paper presents the results of the monitoring and discusses the prediction of degree of consolidation from settlement readings as opposed to pore water pressure along.

INTRODUCTION

The National Transport Commission of South Africa is constructing a new national road between the towns of George and Knysna along the southern tip of Africa. The general area of the road is shown in Figure 1.

The road will traverse an alluvial flood plain in the area of the Goukamma River near Buffelsbaai, South Africa. The present alignment of the road was constructed close to a hillside on the edge of a flood plain so as to avoid the necessity for construction over deep soft foundation soils. The present construction calls for realignment of the road and construction of embankments up to five metres high on deep deposits of soft organic clay and silt. Stability analyses indicated that for an embankment constructed to full height within a short period of time, the factor of safety for the most critical section would be less than unity. It was decided, therefore, to construct the embankment in several phases. To facilitate construction so as to reduce construction time, it was decided to utilize band drains for drainage of the foundation materials. During construction it was decided that an allowable factor of safety of 1.3 would be adequate providing that careful monitoring was conducted of excess pore water pressure and deformations.

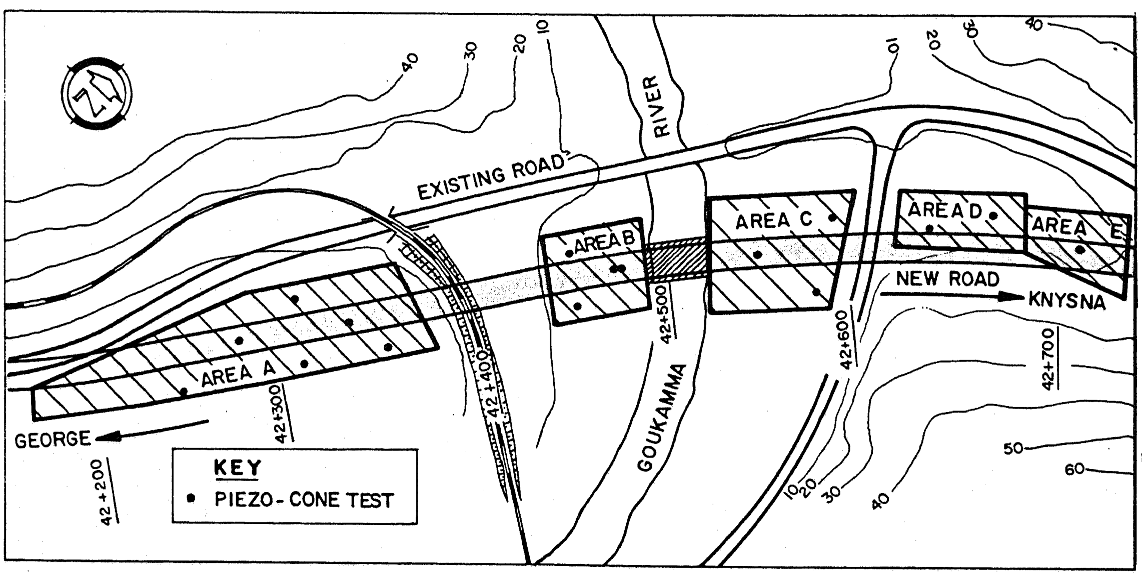

Fig. 1 Embankment Alignment Showing Test Hole Locations

This paper describes the general soil conditions and construction sequences. Results of pore water pressure response and consolidation as influenced by the band drains are presented. It was observed that the band drains operated as designed and were effective in enhancing the consolidation of each phase of construction.

FIELD INVESTIGATION

Site Description

The site is located on the alluvial flood plain of the Goukamma River approximately 10 km west of the town of Knysna along the southern coast of South Africa. The flood plain extends for approximately 700 m to the west of the river and 200 m to the east of the river. In general the site is flat and is used for pastureland and dairy cattle.

For purposes of site investigation and design the site was divided into five separate areas designated A through E. Figure 1 shows the general topography of the area and the alignment of the embankments.

Soil Profile

The soil investigation initially comprised 23 piezocone tests at locations as shown on Figure 1. An additional four test holes were bored and samples were taken in the immediate vicinity of the Goukamma River. Figure 2 shows the soil profile through the centerline of the embankment alignment. As can be seen from Figure 1, the north side of the site rests up against a relatively steep hillside. This hill is primarily sand to a depth in excess of about 20 m overlying rock. The silt and clay strata shown on Figure 2 increase in depth from the north towards the south.

The silt and clay deposits were seen to include many thin lenses which exhibited rapid pore water pressure dissipation during the piezocone testing. From the samples taken in test holes near the river, it was seen that these lenses consisted of thin layers of highly organic soil and peat.

The soils were quite variable over the site. In general the soils were classified as being of low plasticity. The clayey silt exhibited values of liquid limit ranging from 31 to 42 with plasticity indices from 6 to 12. The siltey clay had a liquid limit of about 60 with a plasticity index of 16. The soils all exhibited a high organic content.

A typical piezocone record from area C is shown in Figure 3. The existence of organic and/or sandy lenses can be seen from the pore water pressure record. From the point load data the value of undrained strength of the clayey silt was observed to generally be in the range of 10 to 15 kPa. The shear strength typically appeared to be uniform with depth.

The coefficient of consolidation was measured both from laboratory consolidation tests and from piezocone pore water pressure dissipation. The laboratory measured values were about 0.8 m^2/yr. The results from the piezocone testing indicated values of 1.5 m^2/yr to 6.5 m^2/yr for areas A and E and 6 to 20 m^2/yr for areas B, C and D.

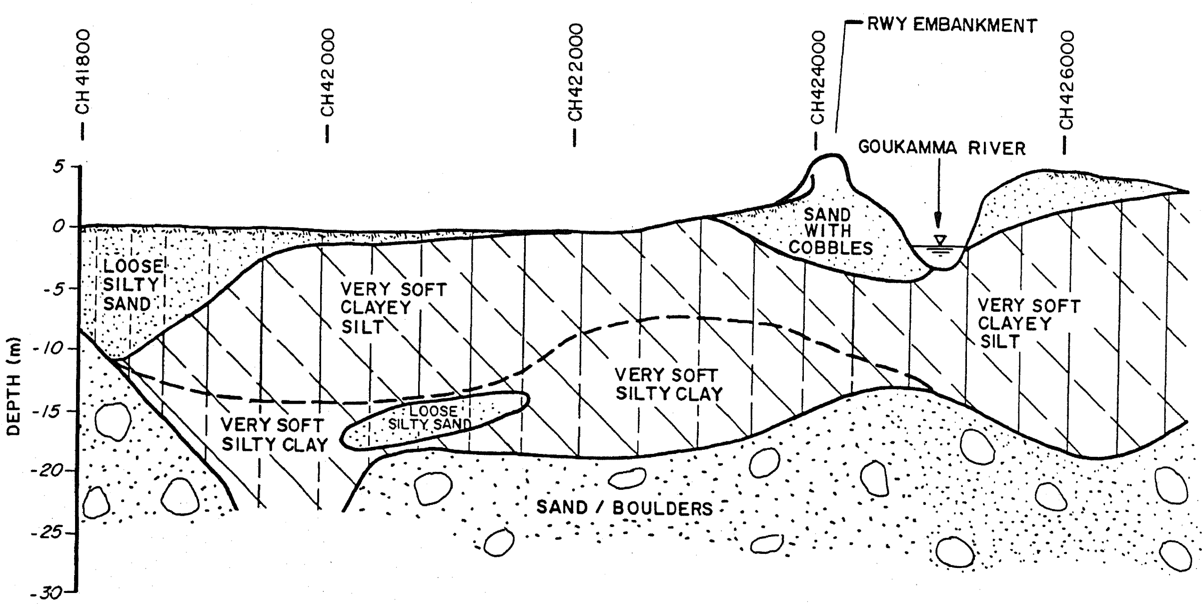

Fig. 2 Soil Profile Along Embankment Centerline

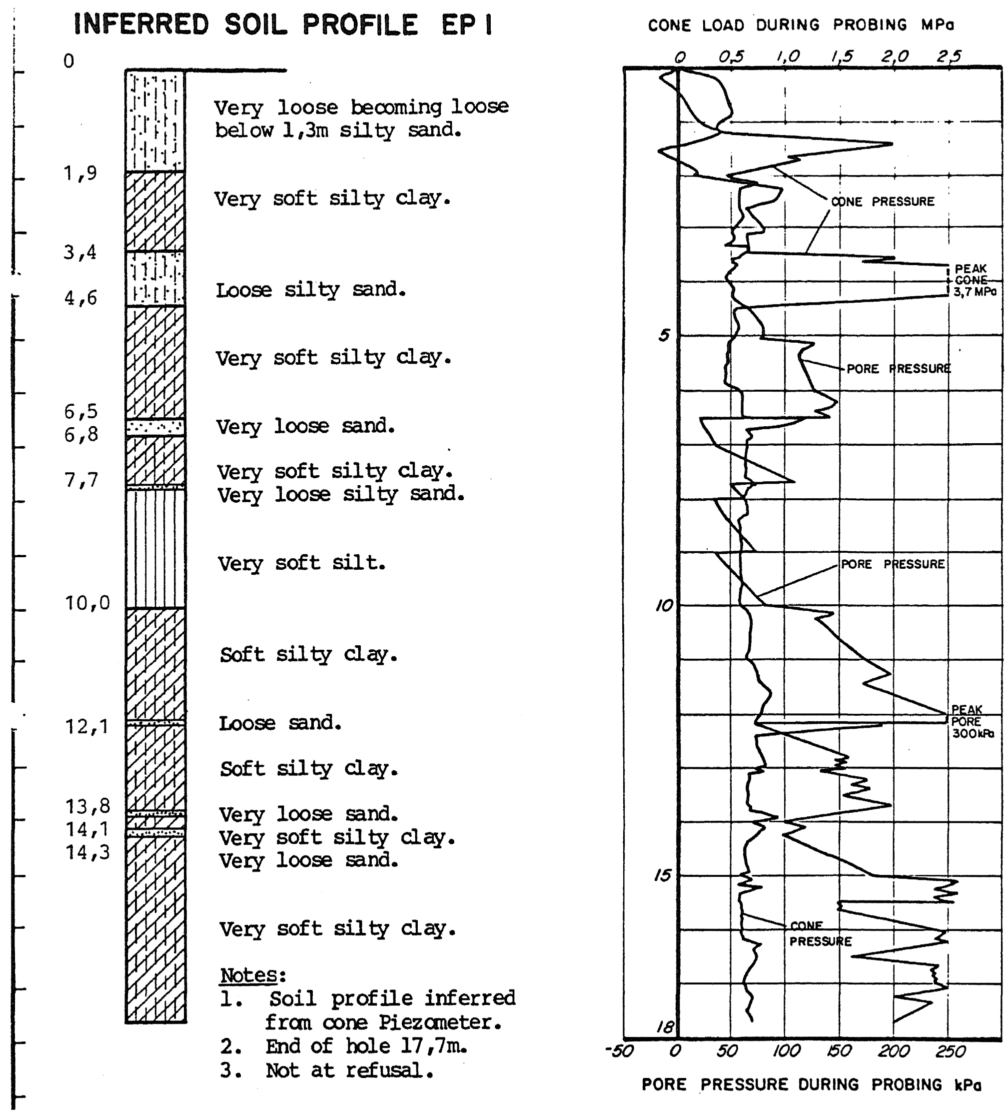

Fig. 3 Typical Piezocone Record - Area C

EMBANKMENT STABILITY AND DESIGN

General Considerations

The most critical time for stability of an embankment on soft soils of the nature encountered at this site is immediately after construction. Stability analyses utilizing undrained shear strength indicated that it would not be possible to construct the embankment to full height in one operation if time was not allowed for pore water pressure dissipation. It was decided therefore, to utilize staged construction allowing the pore water pressures to dissipate to 75 percent consolidation between lifts.

Even using the highest values of coefficient of consolidation (i.e., 6 and 20 m^2/yr) the times required between lifts to achieve 75 percent consolidation would have been from about three to seven years. In order to shorten these times to values more consistent with allowable construction periods, it was decided to install band drains to enhance consolidation.

Drain Design

It was desired to allow no more than six months between construction of each lift. The spacing of the drains was designed using the procedure developed by Hansbo (1979). The times required to achieve 75 percent dissipation as a function of wick or band drain spacing for various values of coefficient of consolidation are shown in Figure 4. The design drain spacings from different areas are shown in Table 1.

TABLE I. Design Drain Spacing

Area	Drain Spacing	Chainage From	Chainage To	Estimated Depth (m)
A	2,3	42165	42380	15
B	1,5	42450	42490	14
C	1,5	42520	42555	25
C	2,0	42555	42595	20
D	2,3	42615	42665	13

Drains were driven on a square grid. A sand blanket 700 mm thick was placed on the surface to provide drainage. Finger drains were placed transverse to the centerline of the embankment at every fourth row of wick drains. The finger drains connected to a main collector drain along the edge of the embankment. The finger drains and collector drains consisted of 19 mm washed stone wrapped in geotextile filter fabric. The collector drain also contained a 100 mm diameter clay pipe.

Loading Procedure

The first lift of the embankment consisted of the drainage blanket. After 120 days a second lift was placed and a third lift was placed at about 300 days. The loading sequence is shown in Figures 6 and 7.

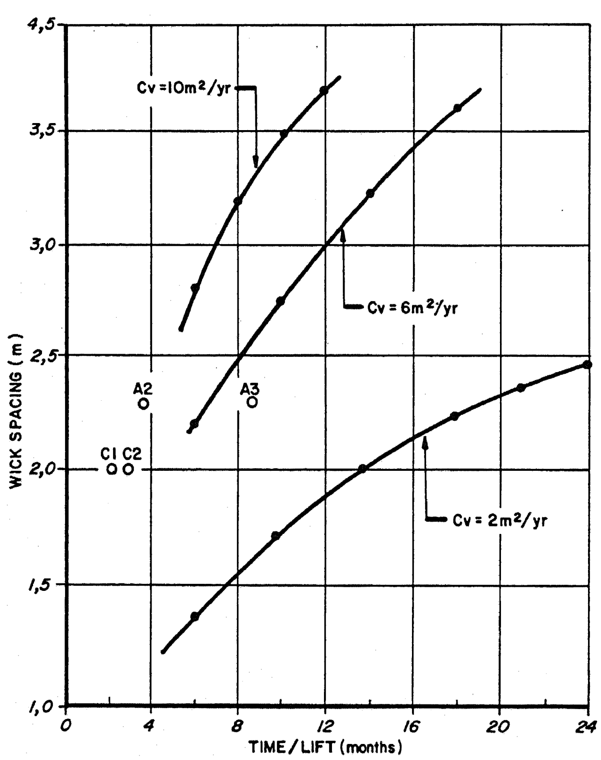

Fig. 4 Effect of Band Spacing on Times for Pore Pressure Dissipation

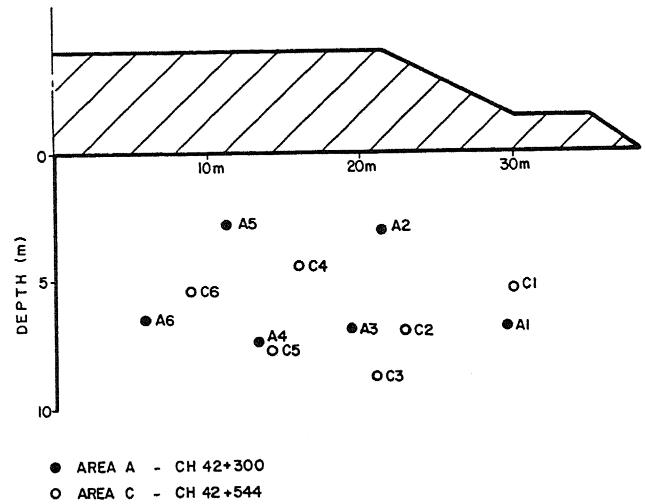

Fig. 5 Piezometer Locations

INSTRUMENTATION

Settlement plates and pneumatic piezometers were placed in areas A and C of the embankment. The settlement plates were installed at the centerline, at the shoulder of the embankment, and at the toe. The locations of the piezometers are shown on Figure 5. The piezometers were installed after placement of the drainage blanket (the first lift).

FIELD MEASUREMENTS

Piezometers

The loading history and the piezometer responses are shown on Figures 6 and 7. In area A the piezometer showed an almost immediate response of approximately 70 to 80 percent of the applied surface load. In area C the response was very close to 100 percent of the applied load.

Of particular interest is the fact the pore water pressure remained at a nearly constant level or even increased for several of the piezometers before it began to dissipate. Furthermore, the pore water pressure remained at a constant level higher than the hydrostatic level after the excess pore water pressure appeared to have dissipated. This phenomenon of delayed pore water pressure dissipation and the existence of a "trapped" pore water pressure after dissipation has also been observed by Hansbo, Jamiolkowski, and Kok (1981). As they noted, it points out the difficulty of determining the degree of consolidation solely from piezometer data.

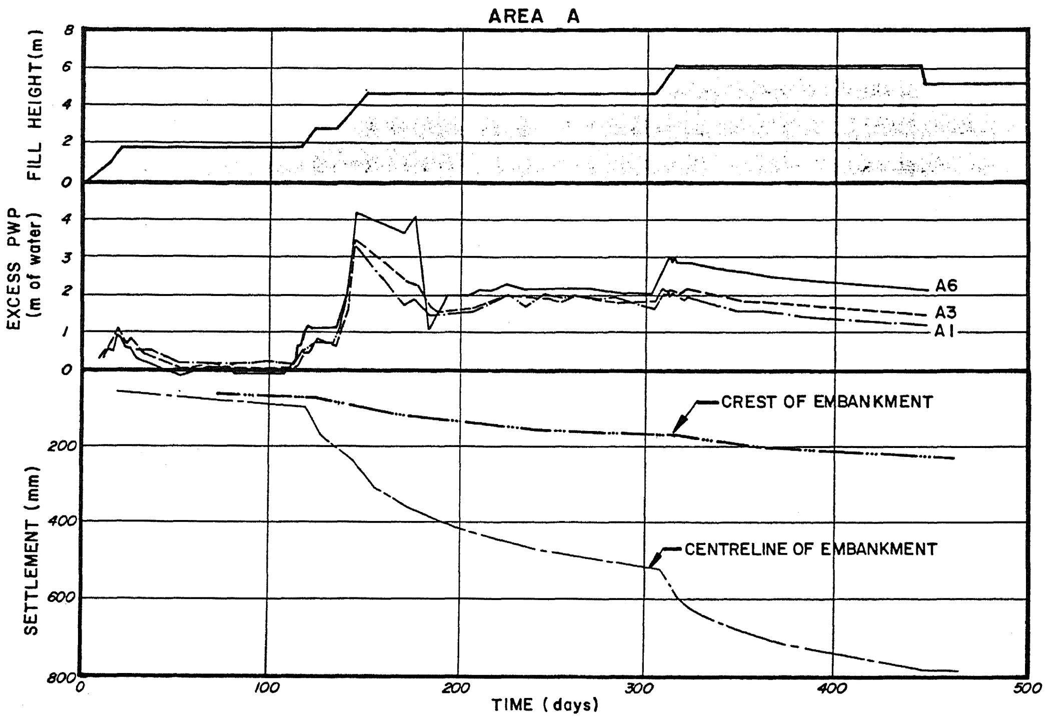

Fig. 6 Loading History, Piezometer Response and Settlement - Area A

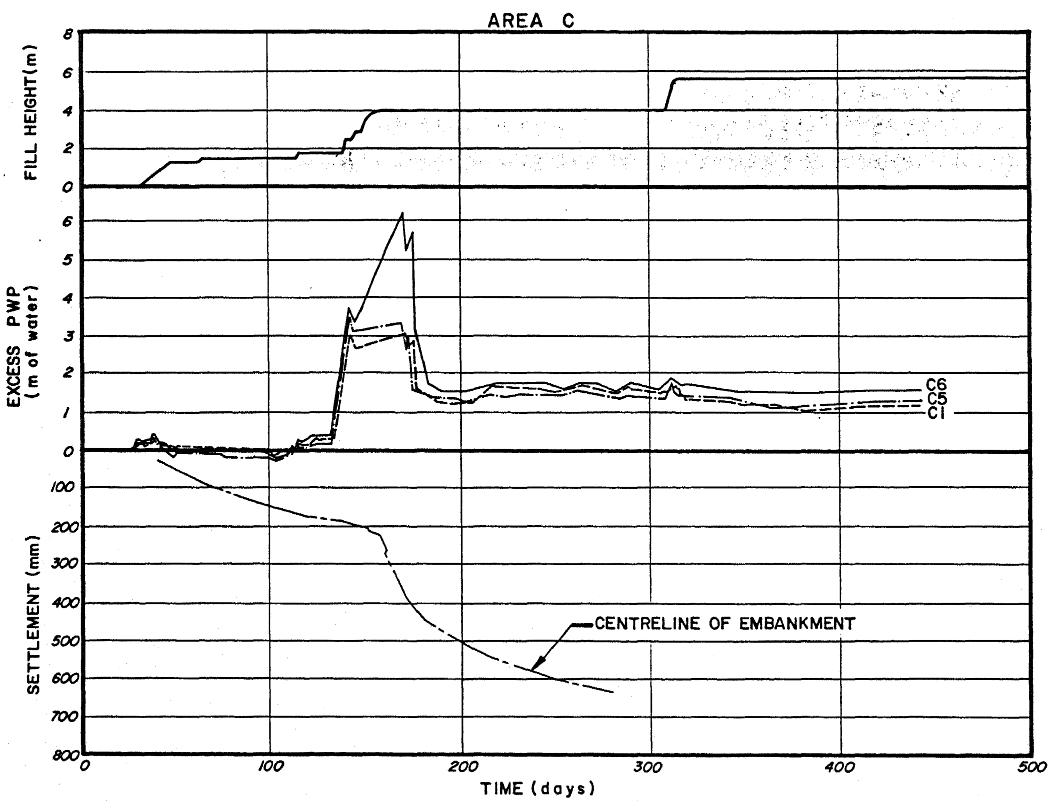

Fig. 7 Loading History, Piezometer Response and Settlement - Area C

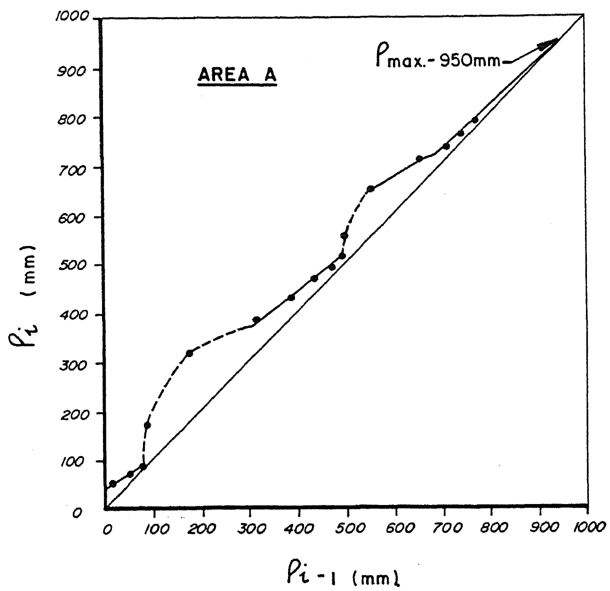

Fig. 8 Plot of Consolidation Data

It is of interest to note that piezometers A5, A6, C4, and C6 were located in areas of relatively high stress increase, which would have increased the breakdown of the soil structure. Also, the piezometers which exhibited the greatest delay in pore water pressure dissipation appeared to be in locations of relatively higher stress increase than those in which the pore water pressure dissipated more rapidly. It is believed that in this case the delayed pore water pressure dissipation and the increase in pore water pressure in the case of piezometers A5, A6, C4, and C6 is due to the collapse of the structure of the very soft soil. Until the soil structure can gain sufficient stiffness to begin to carry the effective stress, the pore water pressure carries the major part of the load. Because of the nonlinearity of the stress strain characteristics of the soft soil, a significant amount of water must be drained before the pore water pressure begins to dissipate. In the areas of lower applied stress level the soil structure was not broken down and it retained a greater initial stiffness.

Settlement

The settlement records are shown in Figures 6 and 7. These data were plotted in accordance with the procedures suggested by Asoaka (1978). This procedure consists of dividing the settlement-time curve into a series of time increments and plotting ρ_i versus ρ_{i-1} where ρ_i is the increment of settlement in the time increment t_i. The point where this plot intersects a 45^o line represents the point where ρ_i equals ρ_{i-1} or the point of 100 percent consolidation. The plots of the data for area A are shown in Figure 8.

The time required for 75 percent consolidation of the clay layer can then be determined as the time required for 75 percent of the projected 100 percent settlement to occur from Figures 6 and 7. These times are plotted on Figure 4. It can be seen that the times required for 75 percent consolidated are well within the general magnitudes predicted for the design wick spacings. The points fall within a range of values for coefficient of consolidation of about 6 to 10^2/yr, which are reasonable values determined on the basis of the piezocone data.

DISCUSSION AND CONCLUSIONS

It has been shown that the band drains were effective in increasing the rate of consolidation of the silty clay and clayey silt foundation soils under the embankments. It is of interest to note that for the points shown for both areas A and C on Figure 4, the time required for consolidation of subsequent loadings was greater than for the earlier loadings. One possible explanation for this is a decrease in the coefficient of consolidation for the foundation soils as consolidation takes place. This is very feasible at this site because of the existence of the lenses of inorganic material which would compress and also could become clogged by silt and clay being forced into the peaty material.

Another possibility for the increase in time of consolidation is the potential clogging of the wick drains by the piping of silt and clay through the outer geofabric covering. Either or both of these factors are a possibility.

It is evident that the band drains have been effective in increasing the rate of consolidation and that they have operated as designed.

ACKNOWLEDGMENTS

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