

TRIAL EMBANKMENT WITH STAGE LOADING AND VERTICAL DRAINS

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SYNOPSIS

This paper presents the basic design philosophy and performance evaluation of a trial embankment on 16-18m thick soft clay deposit with stage loading and vertical drains. The project is one of the 10 trial embankments with different soil improvement methods carried out under the sponsorship of the Malaysian Highway Authority. Due to practical constraints in carrying out the construction, the embankment was built to only 4.75m high instead of the intended 9.0m. Although definite conclusions regarding the scheme could not be drawn yet at this stage due to the short time available for monitoring, field settlement data do indicate that the vertical drains accelerated the rate of settlement and stage loading could be a feasible way in controlling stability of fills on soft ground.

1. INTRODUCTION

Highway construction in low lands and coastal areas of Malaysia often needs high embankments built on soft clay deposits of low shear strength and high compressibility. There is an urgent need for a cost and performance comparison of various soft clay improvement methods currently being used. In view of this problem, the Malaysian Highway Authority (MHA), with the assistance of many individual professionals and organizations, conducted a major study on construction of embankments on soft clay deposits. A series of trial embankments, 3m and 6m high, using ten different soil improvement methods were constructed and monitored to compare the performance.

Moh and Associates proposed and designed one of the trial embankments using stage construction method with vertical drains (Fig.1). This paper presents the design considerations, field monitoring results and evaluation of the performance of this test embankment.

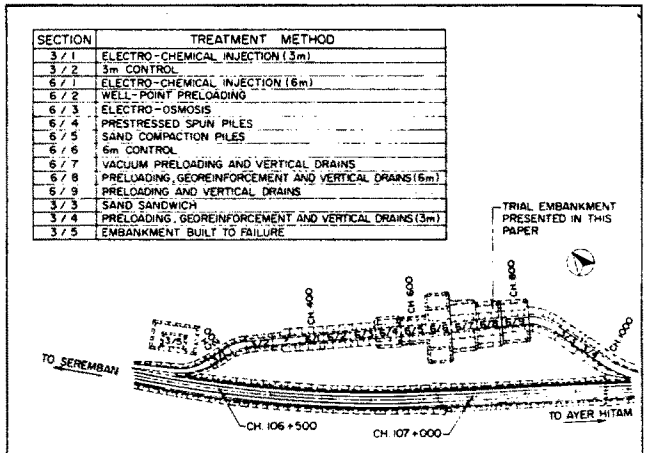


Fig. 1 Layout of Trial Embankments

2. SUBSOIL CONDITION

A series of boreholes was drilled at the test site and undisturbed soil samples were taken for laboratory testing by the MHA project team. Information obtained from the site investigation and laboratory testing, including a continuous sample of the soft clay deposit, were made available to all the design teams of the trial embankments. Additional soil tests were carried out by the Asian Institute of Technology at a later stage. Detailed descriptions of the site investigation and laboratory testing work are presented in Volume 1 of the Proceedings of the Symposium on Trial Embankments on Malaysian Marine Clays.

Figure 2 shows a typical soil profile of the site. The borehole of this soil profile was located under the Trial Embankment 6/8 which is adjacent to the Trial Embankment 6/9 described in this paper.

The subsoil mainly consists of a 16-18m thick soft marine clay layer overlying loose to dense sand layers. The soft clay can be

subdivided into three layers, e.g. the weathered crust, the very soft clay and the soft clay.

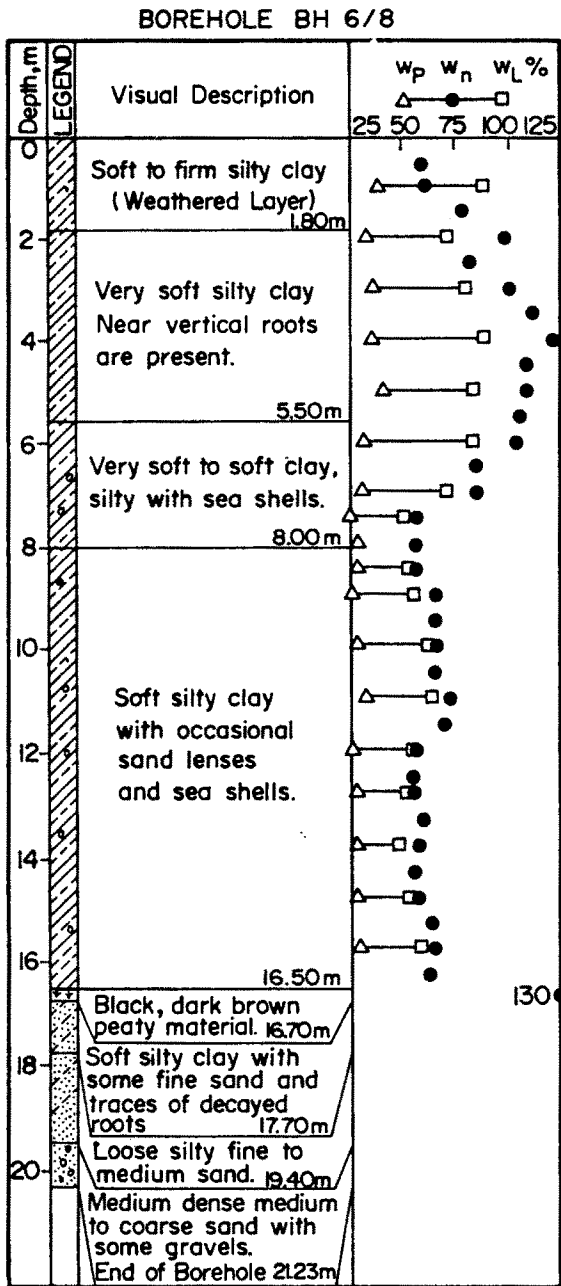


Fig. 2 Typical Subsoil Profile at the Site

Geotechnical properties of the three soft clay layers are quite different. The natural moisture content of very soft clay layer is 100-110% which is significantly higher than its liquid limit (80-85%). Of the soft clay underneath, the natural moisture content is about 60% and very close to its liquid limit. Plasticity index of the very soft clay is about 50% which is about twice the value of the soft clay layer below.

Compressibility characteristics of the subsoils obtained from one-dimensional

oedometer tests are presented in Fig. 3. The very soft clay and the soft clay found below 4m depth appear to be slightly overconsolidated, with overconsolidation ratio varying between 1.0 and 1.5. The overconsolidation ratio of soil at shallow depths (less than 4.0m) is higher, varying from 1.5 to 3.0 for the very soft clay and 7.1 to 18.6 for the weathered soil. The virgin compression index of the very soft clay varies between 1.3 and 2.2 and that of the soft clay underneath varies between 0.7 and 1.2. Variations of the compression indices of the three soil layers with consolidation pressures are shown in Fig. 4. It can be seen from the figure that the compressibility of the very soft clay is more than twice that of the soft clay at most stress levels.

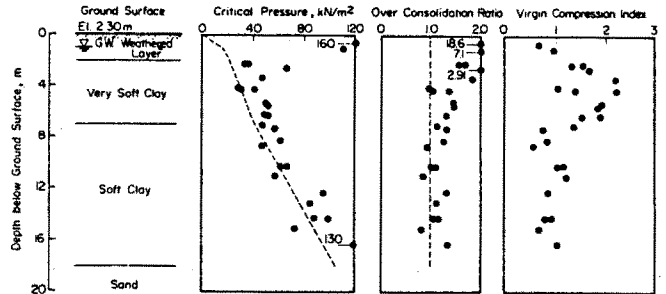


Fig. 3 Compressibility Characteristics of Subsoils

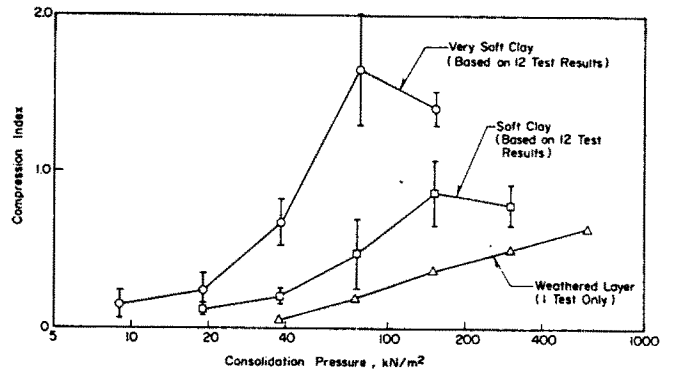


Fig. 4 Variation of Compression Index with Consolidation Pressure

Variations of the vertical coefficient of consolidation (c_v) of the very soft clay and the soft clay with consolidation pressures are shown in Fig. 5. The c_v values of the two soft soil layers are almost the same except for the pressure range where the overconsolidation ratios of the two layers are different. Sufficient test results are not available for the weathered layer to obtain representative c_v values, but indication from the available results of two tests is that c_v values of this layer could be significantly high.

Consolidation tests have been carried out on both vertically and horizontally oriented

specimens and the ratio of c_h/c_v obtained from corresponding pairs of tests at different consolidation pressures for the very soft clay and the soft clay are shown in Fig. 6. It appears that the c_h/c_v ratio varies between 0.5 and 2.0 with an average near 1.0. These test data tend to indicate that as far as vertical and horizontal drainage is concerned, the two clay layers behave quite similarly.

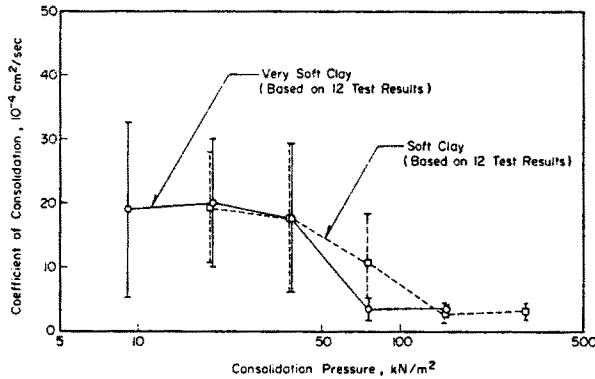


Fig. 5 Variation of Vertical Coefficient of Consolidation with Consolidation Pressure

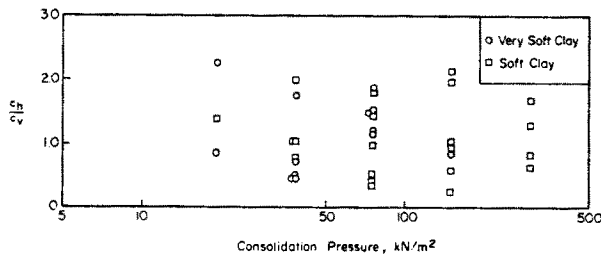


Fig. 6 Variation of c_h/c_v Ratio with Consolidation Pressure

Variation of field vane shear strength with depth is shown in Fig. 7. In the top 1-2m zone (weathered crust), the shear strength is high (25-40 kN/m²), but within the next 2m it drops to about 10kN/m² before it starts to increase with depth. Sensitivity of the soft clay varies between 3 and 6 showing that soft clay is sensitive. However, with this sensitivity, gain in shear strength of disturbed soil with time (thixotropy effect) is expected. The high shear strength of top crust may not be permanent. When water table rises, the soil in this zone may be softened.

Groundwater level observed in this site during the site investigation period varied between 0 and 1m below the ground surface. Some areas were inundated during most of the year.

3. DESIGN PARAMETERS

Consolidation and shear strength characteristics of the soft clay layers are the main

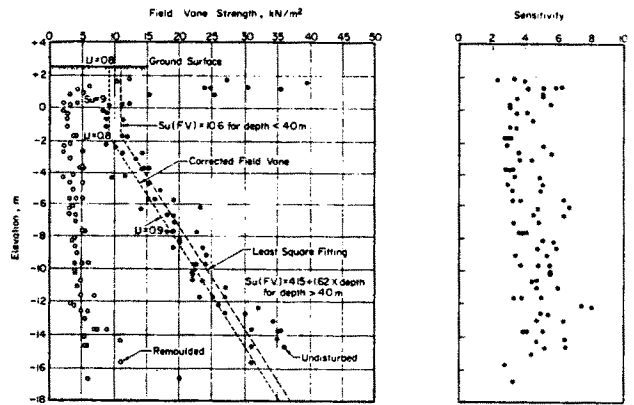


Fig. 7 Field Vane Strength and Sensitivity of Subsoil

geotechnical parameters involved in the design of the trial embankment. Properties of the fill material also play an important role since the total thickness of the final fill is quite large.

3.1 Consolidation Characteristics

For settlement analyses and design, the compressible soil layer was assumed to be 18m thick and it was divided into three sublayers of 2m, 5m and 11m thick according to the simplified compressibility characteristics shown in Fig. 8. The coefficient of consolidation is the most difficult consolidation parameter to be determined from laboratory tests. In this design, the average values obtained from oedometer test results were used. It was further assumed that the values of the horizontal and vertical coefficients of consolidation are equal.

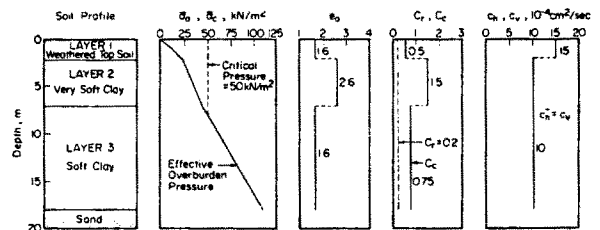


Fig. 8 Consolidation Properties used in the Stage Construction Design

3.2 Undrained Shear Strength

The undrained shear strength profile of the soft clay adopted for the embankment design was taken from the field vane test results shown in Fig. 7 and corrected according to BJERRUM (1972). The adopted correction factors which depend on the plasticity index of the clay, were 0.8 and 0.9 for the very soft clay in the 2 - 4m zone and the clay below 4m depth respectively. The higher shear strength values recorded for the weathered crust were ignored in the design, mainly

because of possible softening of this layer when the subsoil is submerged.

Increase in the undrained shear strength due to consolidation of the soil layers under embankment loading is another important shear strength characteristic which was considered in the design. The increase in undrained shear strength (Δs_u) due to increase in the effective vertical overburden pressure ($\Delta \bar{\sigma}_v$) as a result of soil consolidation was estimated as follows:

$$\Delta s_u = (s_u / \bar{\sigma}_{v0}) \Delta \bar{\sigma}_v \quad \text{----- (1)}$$

where

Δs_u = increase in undrained shear strength

$s_u / \bar{\sigma}_{v0}$ = ratio of undrained shear strength to the effective overburden pressure.

$\Delta \bar{\sigma}_v$ = increase in the effective overburden pressure due to soil consolidation under the imposed load.

According to the field vane shear test results as shown in Fig. 7, the undrained strength ratio $s_u / \bar{\sigma}_{v0}$ is equal to 0.27 for the soft clay below 4.0m depth. A series of anisotropically consolidated undrained triaxial compression tests were carried out in the MAA laboratory on soft clay samples obtained from 8.0m depth. Four specimens were consolidated under estimated anisotropic conditions corresponding to the four proposed stages of embankment loading (to be discussed in a later section). The specimens were then sheared under undrained condition by increasing the vertical axial load by using dead weight. The test results, as presented in Table 1, gave an average value of 0.25 for the ratio of $\Delta s_u / \Delta \bar{\sigma}_1$, in which Δs_u is the increase in undrained shear strength and $\Delta \bar{\sigma}_1$ is the increase in vertical consolidation pressure (equivalent to $\Delta \bar{\sigma}_v$ in the field).

Table 1. Undrained Shear Strength of Soft Clay (from 8.0m Depth) Consolidated Under Different Anisotropic Stress Conditions

Anisotropic Consolidation Press., kN/m ²		Undrained Shear Strength, kN/m ²	Increase Over Insitu Value, Δs_u , kN/m ²	$\frac{\Delta s_u}{\Delta \bar{\sigma}_1}$	Remark
$\bar{\sigma}_1$	$\bar{\sigma}_3$				
51	33	22	—	—	Insitu Condition (At Stage 1 Loading)
83	54	29.5	7.5	0.23	At Stage 2 Loading
93	61	33.5	11.5	0.27	At Stage 3 Loading
110	72	35.5	13.5	0.23	At Stage 4 Loading

3.3 Properties of the Fill Material

In the design of the test embankment, it was assumed that the fill would be lightly compacted and the properties are as follows:

Total Unit Weight = 18 kN/m³
 Cohesion Intercept = 5 kN/m², and
 Angle of Shearing Resistance = 30°

4. DESIGN CONSIDERATIONS

4.1 Design Criteria

The proposed objective is to safely build an embankment with top platform width of 20m and final elevation of 6.0m above existing surrounding ground level after completion of settlements. The proposed time constraint is that the residual settlement of the embankment 450 days after the beginning of construction should not exceed 100mm.

4.2 Design Approach

For the design of the stage loading, the following steps were followed:

- Step 1 - Estimation of the ultimate settlement of the embankment of required elevation and, hence, the total fill thickness needed.
- Step 2 - Estimation of the maximum thickness of the fill that could be placed over the existing ground with a minimum factor of safety of 1.2 against stability failure.
- Step 3 - Estimation of required shear strength gain in subsoil in order to increase the fill thickness beyond the maximum value obtained in Step 2.
- Step 4 - Estimation of the increase in shear strength of soft clay underneath the embankment with time due to consolidation under the loading of the fill that has already been placed.
- Step 5 - Estimation of the time and magnitude of each stage loading based on the results from Steps 3 and 4.

4.3 Method of Analyses

a) Settlement

Primary consolidation settlement of the compressible soil layers was considered and some allowance was made for secondary consolidation. Immediate settlement which does not contribute to the post-construction settlements was neglected. The factor of safety of the embankment was maintained approximately at 1.2 for each stage loading to minimize settlements due to plastic flow.

The ultimate consolidation settlement (ρ_{ult}) was calculated according to Eq. (2) given below.

$$\rho_{ult} = \mu_c \left[\frac{HC_c}{1+e_0} \log \frac{\sigma_{vc}}{\sigma_{v0}} + \frac{HC_c}{1+e_0} \log \frac{\sigma_{v0} + \Delta \sigma_v}{\sigma_{vc}} \right] \quad \text{---- (2)}$$

In the present design, μ_c of 1 was used because most of the compressible layers are only slightly overconsolidated and the available consolidation data are not adequate for more precise evaluation.

The rate of consolidation settlement was calculated by considering the combined vertical and radial flow of porewater. The degree of consolidation U_t at any time "t" was calculated as (BARRON, 1948):

$$U_t = 1 - (1 - U_{ht})(1 - U_{vt}) \quad \text{---- (3)}$$

b) Porewater Pressure

Excess porewater pressure developed in the soft clay under embankment loading can be expressed as follows (SKEMPTON, 1954).

$$\Delta u = B [\Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3)] \quad \text{---- (4)}$$

Dissipation of porewater pressure was calculated by considering both the vertical and horizontal flow of porewater. The porewater pressure at a point at any time "t" is given by:

$$u/u_0 = (u_{vt}/u_0) \times (u_{ht}/u_0) \quad \text{---- (5)}$$

in which u is the excess porewater pressure at time "t", u_0 is the excess porewater pressure at time $t = 0$, $(1 - u_{vt}/u_0)$ is the degree of dissipation of porewater pressure due to vertical flow and $(1 - u_{ht}/u_0)$ is the degree of dissipation due to horizontal flow (BARRON, 1948).

The ratio u_{vt}/u_0 was calculated using TERZAGHI's theory and the ratio u_{ht}/u_0 was estimated on the basis of BARRON's solution for equal strain consolidation with no smear effect.

c) Stability

Simplified Bishop's Method of slices was adopted for the stability analyses of the embankment and a computer program with the capacity for searching the critical sliding circle was used. In this searching process for the critical sliding circle, a series of sliding circles corresponding to a network of centers are analyzed and that surface which gives the minimum factor of safety is chosen as the critical sliding surface.

Lateral variation of the undrained shear strength of the soft clay could be considered to be very minor during the first stage of loading. However, at subsequent loading stages the undrained shear strength of soils under the embankment area has increased due to consolidation, but that of soils outside the embankment area has not thus creating a considerable lateral variation of the undrained shear strength. The critical sliding surfaces pass through these zones of varying undrained shear strength, and therefore in the stability analyses, these variations of undrained shear strength should be considered.

In the design of this trial embankment, weighted average of shear strength was used considering the lengths of critical sliding surface in different zones of varying shear strength. It should be pointed out that the

effect of the vertical drains was not considered in the stability analyses.

d) Other Considerations

In order to increase the thickness of the fill subsequent to the first stage filling, stabilizing berms (15m wide) were found to be necessary. Vertical drains were also proposed under these stabilizing berms so that the improved soft clay area is extended.

5. DESIGN OF THE TRIAL EMBANKMENT

The estimated ultimate consolidation settlement of the proposed embankment with the top elevation 6m above the adjacent ground level was 2.86m. It was therefore decided that the total thickness of the fill should be 9.00m allowing an extra 14cm to account for immediate and secondary consolidation settlements. Vertical drains were proposed to be installed in a triangular pattern at 1.3m spacing over the area including the stabilizing berms. A sand blanket, 0.5m thick, was proposed to be placed over the site as a working platform.

The fill was to be placed in four stages and the time for the final stage loading was 225 days after the first loading (Fig. 9). In the first stage, the embankment including the stabilizing berms was proposed to be 2.5m thick and in the second stage, 87 days after the first stage loading, the stabilizing berms were to be raised by one more meter bringing the total thickness to 3.5m and the main embankment to be raised by 3m. In the third and fourth stages, the main embankment was to be raised by 2.0m and 1.5m, 108 days and 225 days after the first stage of loading, respectively.

Results of the embankment stability analyses at different stages of proposed loading are shown in Fig. 10 together with the undrained shear strength profile considered. This undrained shear strength profile was obtained by approximately averaging the shear strength increases along the sliding surfaces due to the soil consolidation.

The estimated primary consolidation settlements with time due to the proposed embankment loading are shown in Fig. 11.

6. INSTRUMENTATIONS

Instrumentations used for monitoring of the embankment performance are shown in Fig. 12. They include 11 surface settlement points, 7 pneumatic piezometers, 2 Sondex settlement systems each fitted with 17 sensing rings and 1 inclinometer casing installed together with one of the Sondex settlement systems.

Piezometers, Sondex settlement gauges and the inclinometer casing were installed during the period of 30 March to 5 April, 1988, and the surface settlement points were installed on 4 June, 1988.

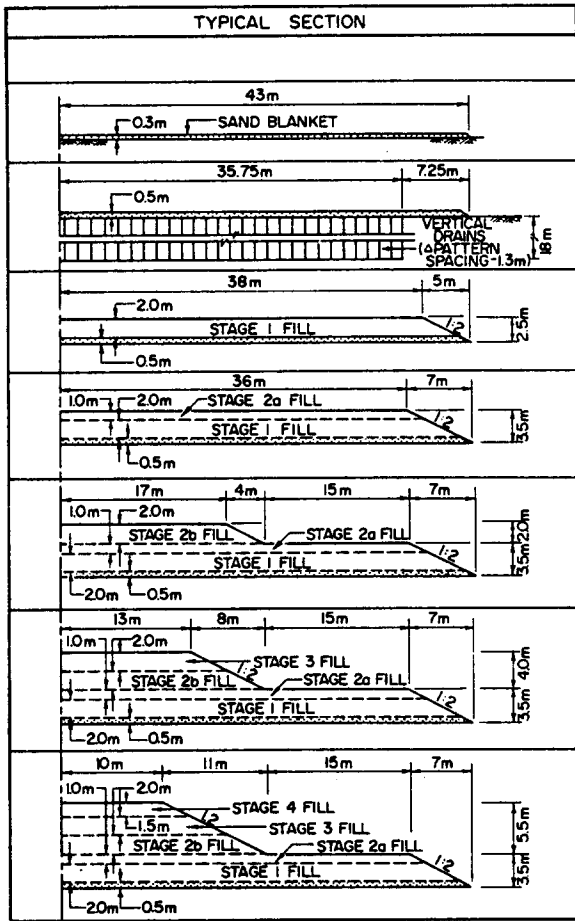


Fig. 9 Proposed Sequence of Stage Construction

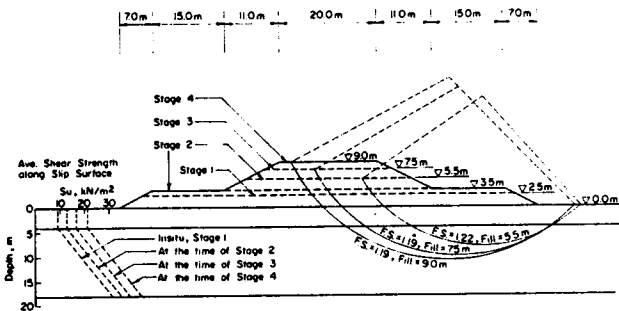


Fig. 10 Stability of Embankment during Stage Construction

In addition there were reference bench marks, 4 pneumatic piezometers and 3 standpipe piezometers installed outside the influence zone of the embankment as dummy instruments for the entire project.

All the instruments were installed and regularly monitored by the MHA Project Team. Monitoring frequency was every 3-4 days during embankment loading and once every week in other times.

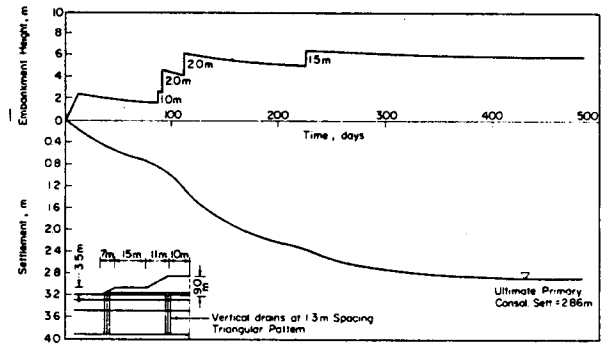


Fig. 11 Estimated Time - Settlement Relationship of Embankment with Stage Loading

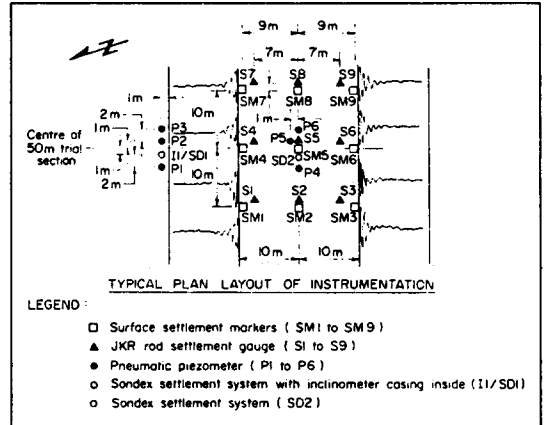
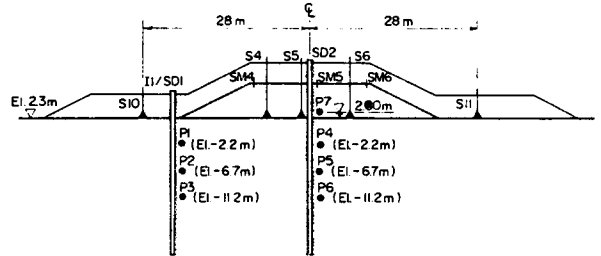


Fig. 12 Instrumentations of Trial Embankment 6/9



7. EMBANKMENT CONSTRUCTION

The actual progress of the embankment construction is shown in Fig. 13. Due to various problems encountered in the field, the embankment construction was not carried out as scheduled. It took about 7 months since the start of the placement of the sand blanket to the Stage 1 of the embankment filling. Ninety days after the completion of the first stage, the second stage loading was carried out upto 4.74m within a period of about 25 days. The embankment was not built upto 5.5m as proposed because of the significant lateral soil displacement underneath the embankment. It was decided to delay filling of the embankment until a later date. Unfortunately, due to

construction constraints, further filling could not be carried out.

The average unit weight of the fill as determined by field density tests was 19.6 kN/m³ which is higher than the 18 kN/m³ assumed in the original design.

The vertical prefabricated drains used for the trial embankment was the Desol Drain manufactured in France. The drain, supplied by Pilecon Sdn. Bhd., is a single component polyolefine plastic drain. According to the manufacturer, both sides of the drain are uniformly perforated with holes usually 0.2mm in diameter at 2mm centers and along 24 running lines. The drains were installed in a triangular pattern at 1.3m spacing.

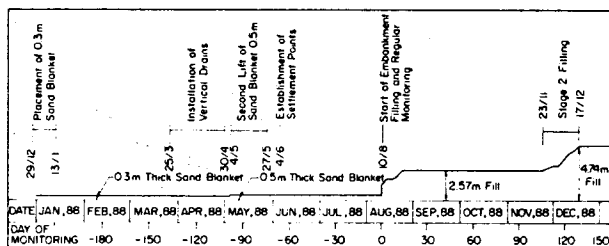


Fig. 13 Stages of Actual Embankment Construction

8. EVALUATION OF PERFORMANCE OF TRIAL EMBANKMENT

Regular monitoring of the instruments in this trial embankment was started on 10 August 1988. This day was therefore taken as the Day Zero of monitoring. The initial readings of the inclinometer and Sondex settlement gauges were taken on this day and therefore measurements of lateral soil movement and settlements at various depths (from Sondex rings) are those taken place since this day. Initial readings of all surface settlement points were, however, taken on 4 June 1988 and therefore the monitoring records of these settlement points show the total settlement since this day. However, no monitoring of the settlement points was carried out until 10 August 1988, which was the starting day of the embankment filling.

8.1 Lateral Soil Displacement

Lateral displacement of the subsoils underneath the embankment was regularly measured at Inclinometer I1, since 10 August 1988. Displacement (displacement in the direction perpendicular to the longitudinal axis of the embankment) profiles at six different times are shown in Fig. 14 and the progress of the maximum lateral displacement and displacement at five other different depths are shown in Fig. 15.

During the first 105 days of monitoring, the lateral displacement of soil was very small

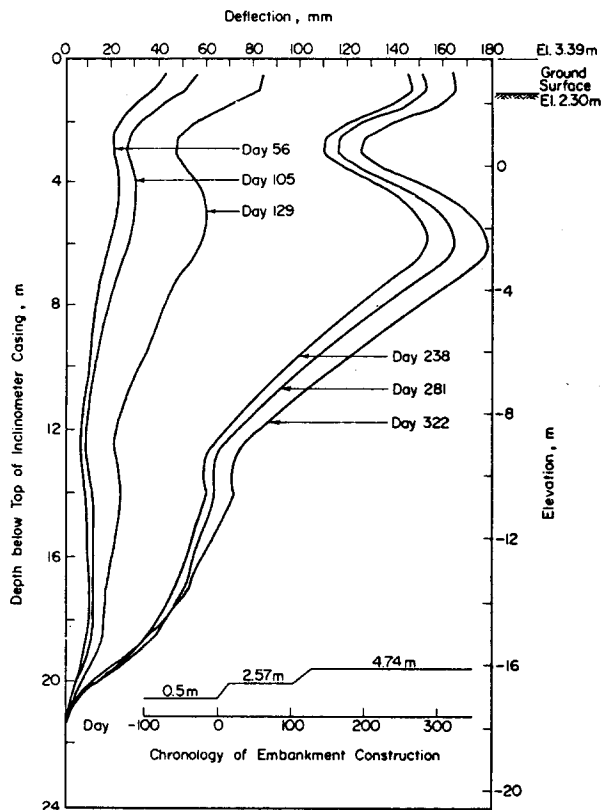


Fig. 14 Lateral Soil Displacement Profile at Different Times

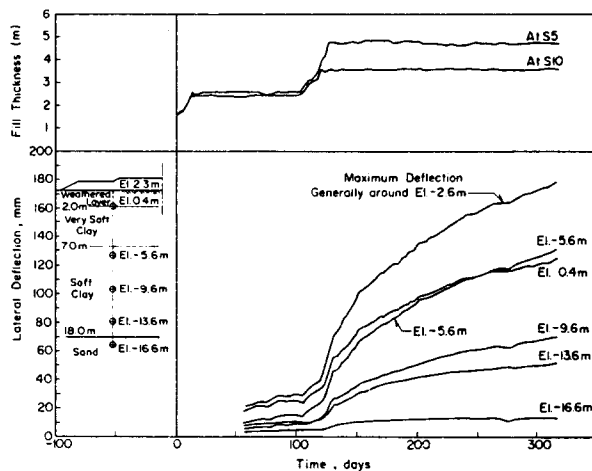


Fig. 15 Lateral Soil Displacement at Different Depths

with a maximum value of about 31mm at El. -1.11m and progressing at a very slow rate until the second stage filling of the embankment was placed at Day 129 (Fig. 15). After the second stage of loading (maximum 4.74m), the lateral displacement increased rapidly to higher values with maximum displacement reaching 71mm at El. -1.61m and then continued at a diminishing rate. Fig. 16 shows the rate of lateral displacement of soil with time. The general trend shows that during the embankment loading, the rate of lateral movement increased sharply.

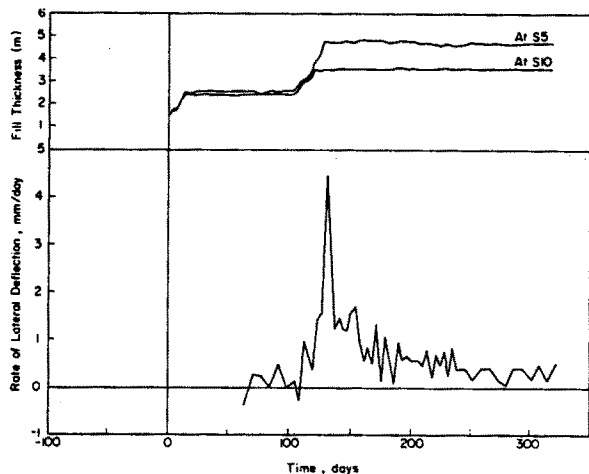


Fig. 16 Rate of Lateral Soil Displacement at Depth of Maximum Deflection (at about El. - 2.6 m)

After the completion of loading, the rate slowly reduced to a more stable smaller value. It must be noted, however, that the lateral displacement has not ceased at the time of preparation of this paper.

The very soft clay layer (Layer 2) found in the 2-7m depth range has the largest lateral displacement. The maximum displacement on the last day of reporting here was 179mm. At about 10m below the ground surface, a change of the displacement profile can be observed. There is a higher tendency for the soil to be displaced. The soil profile at the Borehole BH6/8 shows a layer of higher moisture content around this depth zone.

8.2 Settlement

Vertical settlements across the embankment recorded at surface settlement points S10, S4, S5, S6 and S11 are shown in Fig. 17. Two of these points (S10 and S11) are under the stabilizing berms and the other three are under the main embankment.

At the center of the embankment where the consolidation pressures on the subsoil layers are highest, a total settlement of 1.241m was observed on Day 322. At the other two points S4 and S6 under the main embankment, the observed settlements were 1.243m and 1.183m respectively, although the two points are geometrically symmetrical. At the other two symmetrical settlement points S10 and S11, the observed settlements were also different, being 1.019m and 0.959m respectively. These data show that the settlement across the embankment is larger towards the north side of the site (the settlement point S10) than towards the south side (settlement point S11).

Settlements underneath the embankment at different depths below the ground surface were recorded at the two Sondex settlement gauge locations, one at the center of the embankment (SD2) and the other under the north side

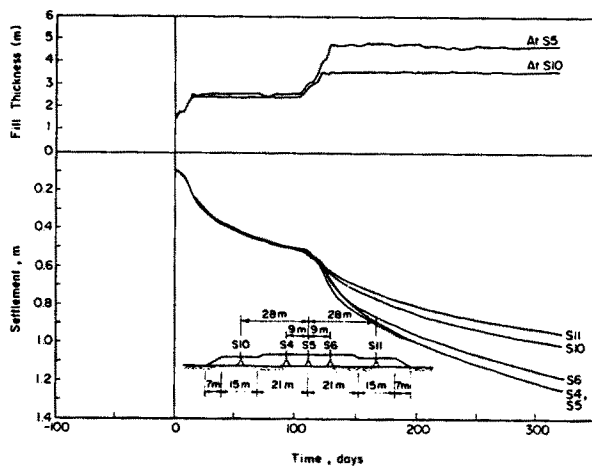


Fig. 17 Observed Settlement at Different Points across the Embankment

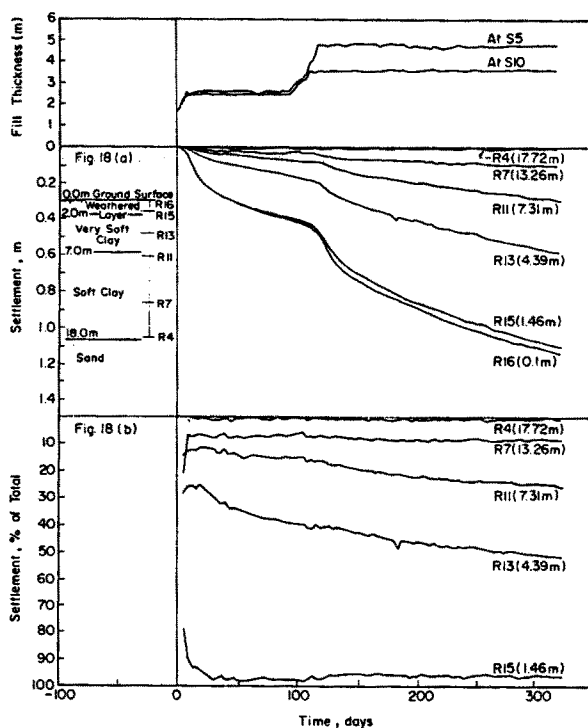


Fig. 18 (a) Observed Settlements at Different Depths below the Center of the Embankment (SD2)

(b) Settlement at Different Depths as Percentage of Total Settlement below the Center of the Embankment (SD2)

stabilizing berm (SD1). The observed settlements at six different depths at these two locations are given in Fig. 18 and Fig. 19 together with the settlements expressed as a percentage of the total settlement. It can be seen that about 80% of the settlement observed at ground level is due to compression of the soil layer in the 1.5m to 8.0m depth zone.

Contribution from the top 1.5m soil to the total settlement is only 3.4%. At SD2, settlement records at 17.7m below ground level (R4) show a negligible amount, but at SD1 compression of soil below 17.75m (R4) contributes about 5% to the total settlement. This tends to indicate that the thickness of the compressible soil is larger on the north side of the site. Another observation that could be made is that percentage contribution to the total settlement from the compression of soil layers below 4.4m depth (R13) increases with time whereas the contribution from the layer above this level decreases with time. The total settlement measured by the two monitoring systems, i.e. Sondex and surface settlement points, were almost the same when adjustment was made for the initial zero reading.

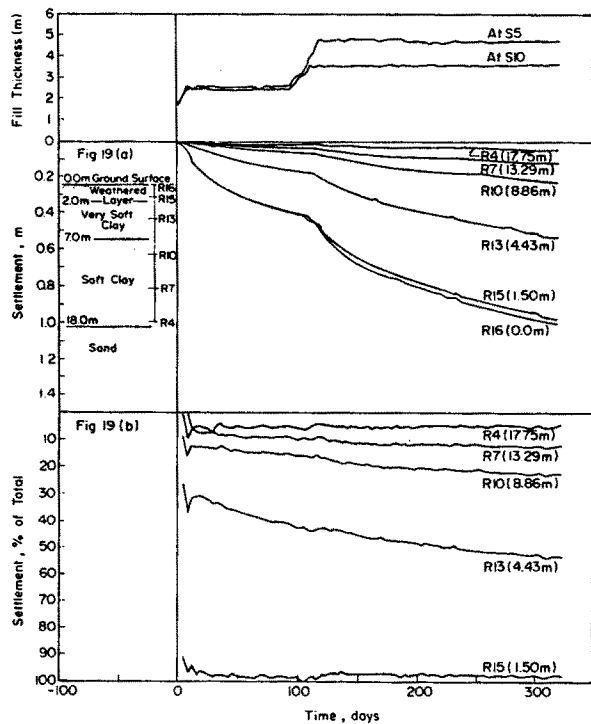


Fig. 19 (a) Observed Settlements at Different Depths below the stabilizing Berm (SD1)
 (b) Settlement at Different Depths as Percentage of Total Settlement below the Stabilizing Berm (SD1)

The measured settlements include both the consolidation settlement and settlement due to elastic and plastic deformations. The consolidation settlement is due to reduction in void ratio of the soil whereas the elastic and plastic deformations take place without any significant volume change of the soil. Therefore any settlement due to elastic and plastic deformations taken place underneath the embankment should appear as lateral deformation of the soil. Lateral soil displacements were regularly monitored at the location of Inclinator Casing I1 (Fig. 12).

Based on the lateral deformation profiles, the volume of soil displaced since the beginning of monitoring was calculated and hence that part of the settlement due to elastic and plastic deformation of soil could be estimated. It was assumed in this calculation that lateral soil movement occurs only in the direction perpendicular to the longitudinal axis of the embankment and it is proportional to the load intensity acting on the ground surface.

It was pointed out that, the lateral soil displacement was small until Day 105 on which day the second stage of loading was started.

Therefore any lateral displacement of the soil due to loading upto Day 0 on which day the initial reading of the lateral soil movements were taken should be small enough to be disregarded. Until about Day 115, the fill thickness over the embankment area was uniform and the estimated settlement due to elastic and plastic deformation of the soil at S5 (center of embankment) and S10 (stabilizing berm) was similar and equal to 1.5cm. However, after the second stage of loading, the lateral soil displacement increased significantly and on the last day of monitoring reported here (Day 322), the estimated settlement due to lateral displacement was 11cm at S5 and was 5.6cm at S10. The amount of consolidation settlements at the various settlement points were estimated by subtracting that part of the settlement due to elastic and plastic deformation of soil from the total settlement. More than 90% of the total settlement observed was due to soil consolidation and the remaining, less than 10% was due to elastic and plastic deformation. Of the consolidation settlement, about 70% was contributed by the very soft clay layer (Layer 2) in the depth range of 2-7m, according to the settlement records at different depths. Contribution from the soft clay layer below 7m (Layer 3) was about 25%.

The amount of consolidation settlement under the actual embankment loading (i.e. 4.74m maximum) was estimated by using the consolidation properties used in the embankment design. These calculated settlements at settlement points S5 and S10 are shown in Fig. 20 and Fig. 21 respectively. Also shown in these two figures are the calculated settlements by considering variations of the lower bound values of compression index of the soil layers with consolidation pressure (Fig. 4) and settlements without using vertical drains. It is evident that the differences between the calculated settlements by using parameters adopted in the original design and the measured settlements are quite large. However, when the lower bound values of compression index were used in the calculations, the calculated settlement became quite close to those actually observed values. The high mean values of compression index of soils obtained from laboratory oedometer tests could be due to disturbance of the soil samples being tested.

Comparing the observed settlements with no-

drain case, it is clear that vertical drains have significantly accelerated the consolidation settlement. On the last day of monitoring, the observed settlements at S5 and S10 are 56% and 62% of respective estimated ultimate settlements compared to about only 20% that would have taken place without the vertical drains. On the other hand, settlement performance of the embankment with the vertical drains in this trial embankment is not as good as that expected. The expected settlement on the last day of monitoring is nearly 100% completion as compared to 50-60% which actually took place. This is of course dependent upon the accuracy of the soil parameters used in the calculations.

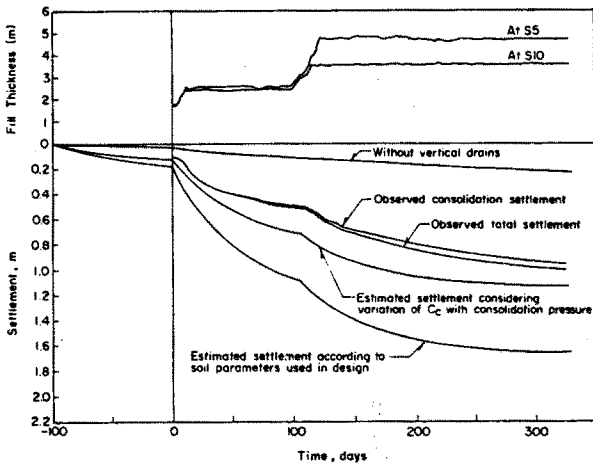


Fig. 20 Comparison of Observed and Calculated Settlement at Settlement Point S5 under Center of Embankment

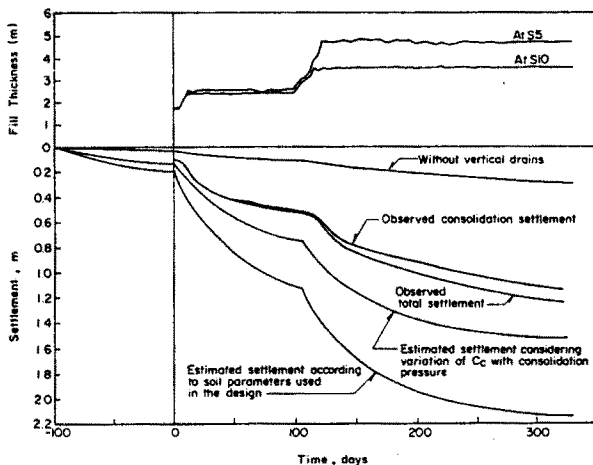


Fig. 21 Comparison of Observed and Calculated Settlement at Settlement Point S10 Under Stabilizing Berm

A further examination of Fig. 20 and 21 indicates that immediately after the Stage 1 loading, the observed settlement rates were

fast, but after that the settlements slowed down rapidly to very low rates which were only slightly higher than that corresponding to the no-drain case. After the Stage 2 loading, similar behavior was observed. There is some indication that the effectiveness of the drains might have diminished with time. It thus appears desirable to extract some of the drains from the field to examine the performance after sometime in the ground.

8.3 Porewater Pressure

Monitoring results of the 6 piezometers installed in the soft clay underneath the embankment are given in Fig. 22. Piezometer P4, installed at about 4.5m below the ground surface under the main embankment malfunctioned after giving very high reading of 12.5m excess porewater pressure on the second day of monitoring, i.e. during the embankment filling. Piezometers P1 (4.5m below ground surface under the stabilizing berm) and P5 (9m below ground surface under the main embankment) gave high excess porewater pressures and did not drop except for random variations. A replacement piezometer P5A installed near P5 gave a very low excess porewater pressure than that shown by piezometer P5. Similarly the replacement piezometer P4A installed near P4 also gave very low readings.

Piezometers P2 (about 9m below the ground surface under the stabilizing berm), P3 and P6 (about 13.5m below the ground surface) gave gradual variations of the excess porewater pressures, but P2 malfunctioned about 200 days after start of the monitoring.

In general, the piezometer monitoring results appear to be erratic and not reliable, particularly for those piezometers installed at shallow depths.

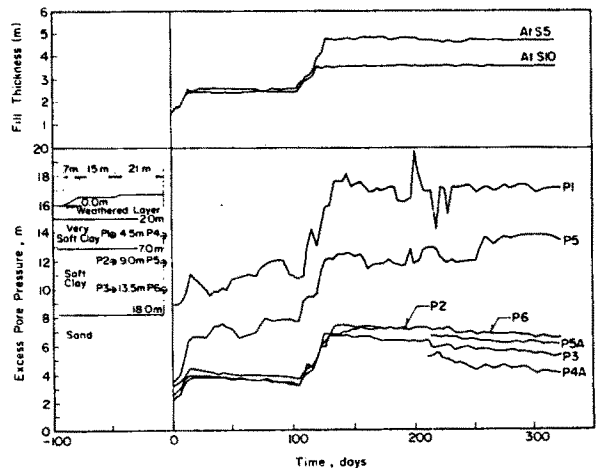


Fig. 22 Piezometer Monitoring Results

8.4 Stability

Stability of the embankment was estimated at the two stages of the construction and the

results are presented in Fig. 23. At the time of Stage 1 loading, the clay layers have consolidated under the weight of the sand blanket (about 0.5m thick) and the shear strength improvement was very small, the estimated average improvement is about 0.7 kN/m². The estimated factor of safety of the embankment at the Stage 1 loading (2.57m thick fill) was 1.10 which was smaller than that estimated at the design stage mainly because of the difference in loading due to difference in the unit weight of the fill material (unit weight of the fill used in the design was 18.0 kN/m³ and the actual unit weight measured was about 19.6 kN/m³).

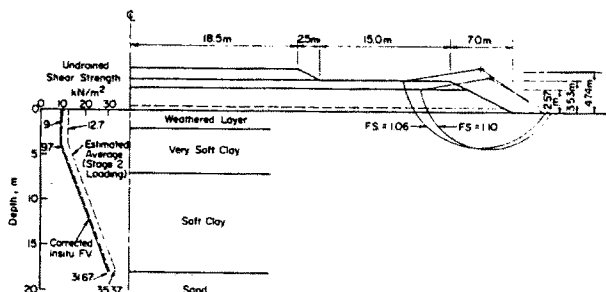


Fig. 23 Stability of the Embankment Under the Stage Loading

At the second stage of loading in which the thickness of the stabilizing berms was increased to 3.53m and that of the main embankment was increased to 4.74m, the estimated increase in shear strength of the subsoil was about 7.3 kN/m² under the embankment area. This increase in shear strength was reduced by 50% to obtain the average shear strength along the sliding surface which passes through the soil outside the embankment area where the shear strength of the soil has not been improved.

The estimated factor of safety of the embankment at this stage was 1.06 and the critical sliding surface was located at the edge of the stabilizing berm area. However, in the field, there was no sign of such a distress condition of the embankment at the time of second stage loading which indicates that the embankment was at a better stable state than estimated. This could be due to better improvement of the shear strength of subsoil than estimated, better strength parameters of the fill material than assumed or the actual unit weight of the fill material being lower.

9. CONCLUSIONS

Results of the study show that stage construction of embankments on the soft Malaysian marine clay is feasible. Due to the soil consolidation, the undrained shear strength of the soft soil could improve with time. Vertical drains have helped to accelerate the consolidation process of the

soft clay, but their effectiveness in this particular case did not seem to be as good as expected. There are indications from monitoring results that the effectiveness of the drains might have deteriorated with time. This can be verified if some of the drains could be extracted and checks are carried out. The drains used in this study ("DESOL") are different from other types of drains available in the market. This drain does not have a filter sleeve, it consists of a single piece with perforations.

Efforts made in this study by the Malaysian Highway Authority, different organizations and individuals must be well commended. Only through this kind of studies, a good comparison of various methods of soil improvements can be made. Results of this work when properly studied could give the design engineers a good insight into the soil improvement methods and embankment construction on soft soils. Some of the important points which should be noted in planning similar type of work in future are as follows.

Individual embankments, rather than interconnected embankments are preferable. Interference from adjoining embankments could mask valuable results, if not having premature failure together with the neighbor.

Geotechnical Investigation should include field testing of coefficient of consolidation (i.e. dissipation test in piezocone testing) so that estimation of settlement rates could have been made more accurately.

Performance of piezometers was very poor in this study. The actual reasons responsible for this poor performance was not known, it could be partly due to improper installation procedures.

10. ACKNOWLEDGEMENTS

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