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Stability of Embankments on Soft Ground – Lessons from Failures

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ABSTRAK

Beberapa benteng yang dibina di atas tanah liat lembut runtuh sewaktu pembinaan sebuah projek lebuh raya utama di Malaysia. Tiga daripada benteng tersebut dianalisis semula berdasarkan geometri benteng sebelum keruntuhan berlaku. Kekuatan ram ricih di situ tanah diguna dalam analisis sewaktu keruntuhan. Prestasi tanah dalam bentuk anjakan tegak dan mengufuk, dan respons piezometer sewaktu keruntuhan diperihalkan.

ABSTRACT

A number of embankments founded on soft clays have become unstable during construction of a major highway project in Malaysia. Three of the embankments were back analysed based on their geometries before the failure. Measured *in situ* vane shear strengths were used in total stress analyses to determine the factor of safety at failure. Performances of the soft clay foundation with regards to vertical and horizontal displacements, and piezometric response, when the failure was imminent were also described.

Keywords: embankment, failure, soft clay

INTRODUCTION

Extensive deposits of low strength, compressible soils are found worldwide, and the difficulties of supporting loads on such foundations have been widely reported. In Malaysia, Quaternary erosion accentuated by climatic and sea level changes has produced widespread, thick deposits of soft clays in the coastal areas and major river valleys, varying from 5 m to 30 m in thickness. Reviews of the basic and engineering properties of some of these deposits have been published by Ting *et al.* (1987) and Abdullah & Chandra (1987). Roads founded on the soft deposits often have to be raised on high embankments, giving rise to problems of instability during construction, and long-term, persistent settlement subsequently.

This paper presents the findings of back analysis of three embankment failures on soft ground. The aim is to establish whether:

a] conventional total stress stability analysis is sufficient for routine calculations, and b] field instrumentation such as settlement gauges, inclinometer and piezometer may be used for control of field construction

SITE AND HISTORY OF THE FAILED EMBANKMENTS

All three embankments (designated Embankment 1, 2 and 3) were part of a recently constructed highway in northern Peninsular Malaysia

Fig. 1(a), 1(b) and 1(c) show plan, profile and instrumentation of the failed embankment sections and their vicinity. All these embankments were instrumented with settlement markers. Additional instrumentation in the form of piezometers and inclinometers was also incorporated in one of the embankments (embankment 3).

Fig. 1(a) shows location of Embankment 1. It lies north of an expressway bridge, located in the area of an oil palm plantation. The embankment was to be built on untreated soft clay foundation to a height of about 5 m with a side slope of 2 horizontal to 1 vertical. Tension crack was discovered at the left-hand side of the embankment by mid-July 1992, i.e. on Day 155 from start of filling. This was discovered as the final fill layer was being placed. The height of fill above original ground level was then about 4.6 m. On the following day the crack opened wider with more new cracks developing. On Day 158 about 1 m fill was removed and counterberm was constructed on the left-hand side to prevent complete collapse of the embankment.

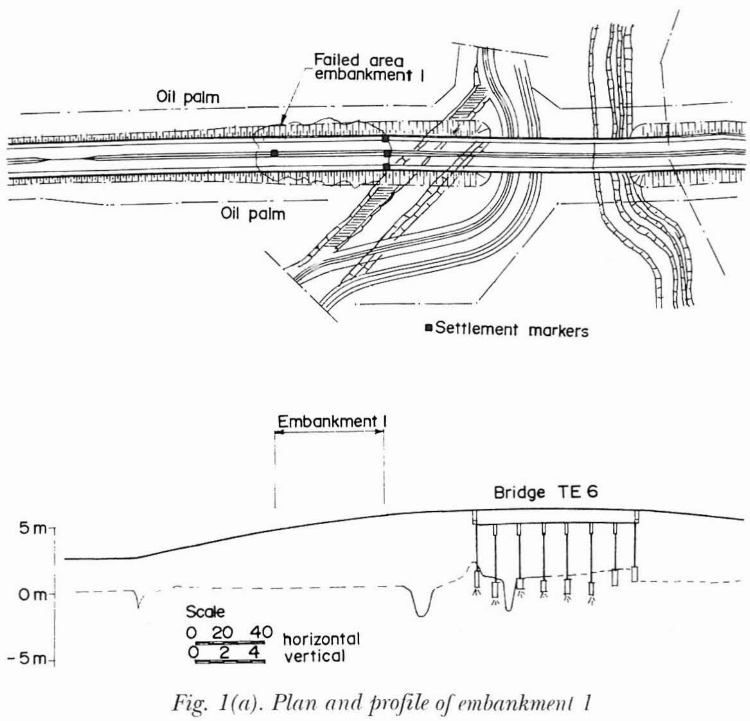


Fig. 1(a). Plan and profile of embankment 1

Fig. 1(b) shows location of Embankment 2 and its vicinity. The failed embankment lies just north of an existing water canal. Beneath the embankment vertical drains were installed to the full depth of the clay foundation (13 m) at a triangular grid spacing of 1.2 m centre to centre. Construction of the embankment commenced with placement of geotextile separator layer and 400 mm thick sand blanket in February 1992, followed by installation of the drain in May 1992. The earth filling was commenced by mid-June 1992 and was almost to the final level when the failure occurred at the end of October 1992, i.e. on Day 220 (Day 0 being placement of sand blanket). The fill thickness was then about 3.6 m above original ground level.

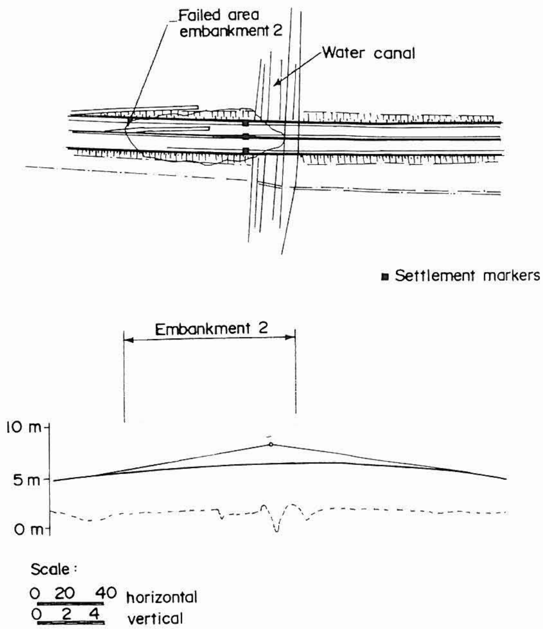


Fig. 1(b). Plan and profile of embankment 2

The location of Embankment 3 is shown in *Fig. 1(c)*. This embankment transversed an area of rubber and oil palm estate. It was to be built to about 3.2 m height above original ground level, inclusive of a 1.5 m surcharge. Site clearing was commenced in early January 1992 followed by placement of geotextile separator layer with 400 mm thick sand blanket. Earth filling was commenced in early March 1992 and was completed to final level by mid-July 1992 (i.e. on Day 140). Five days later (Day 145) the embankment collapsed without prior sighting of any tension crack. It was reported that during construction the embankment was used as part of a haulage road for transportation of earth, piles and other construction materials from an existing

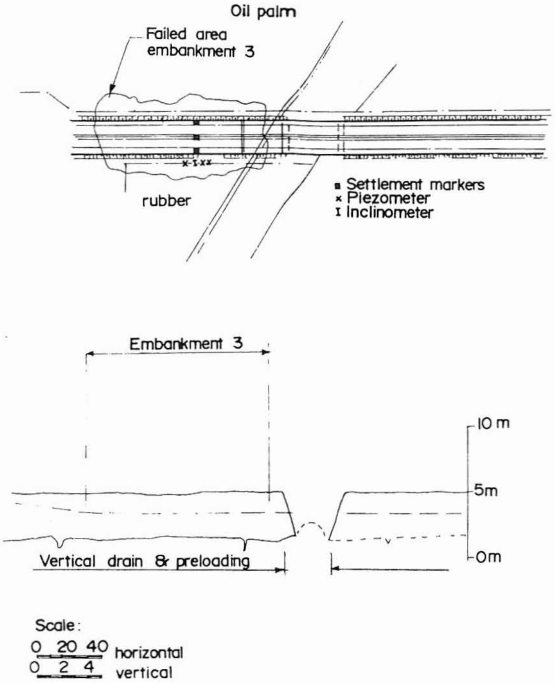


Fig. 1(c). Plan and profile of embankment 3

main road to nearby sections. It is also of interest to note that in all the cases cited, no period of exceptional weather conditions, such as heavy rainfall, was reported prior to the embankment failure.

SOIL PROFILES AND PROPERTIES

Fig. 2(a), 2(b) & 2(c) show subsoil profiles of each of the embankments as obtained from the site investigation carried out during the design stage. In general they comprise soft silty clays of 7 m – 13 m thickness, underlain by a layer of loose to medium dense sand. The liquid limits of the soft clays vary from 50% – 120% with natural water content clsoe to the liquid limit, and plasticity indices in the range of 30% – 80%. Undrained strengths obtained from the vane test show a general trend of strength(S_u) increase below a weathered upper crust which varies from 1 m – 1.5 m in thickness *(Fig. 3)*, with Su/σ c ratio in the range of 0.3 – 0.4. The clays are shown to be moderately sensitive, with sensitivity ratios in the range of 3 – 12. Results obtained from the oedometer tests indicated that the clays are slightly over-consolidated but highly compressible. This apparent over-consolidation of the clay is believed to be due to that of the weathered crust. The values of Cv are typically low, ranging from 1 – 10m^2/yr.

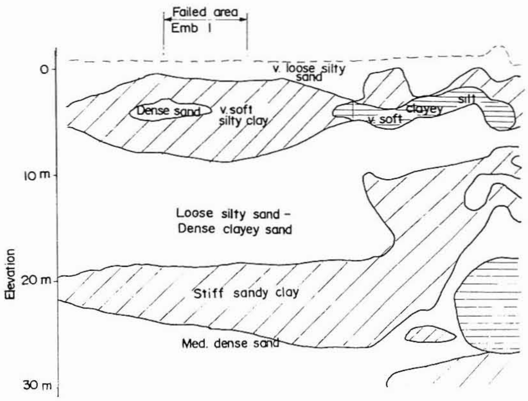


Fig. 2(a). Subsoil profile of embankment 1

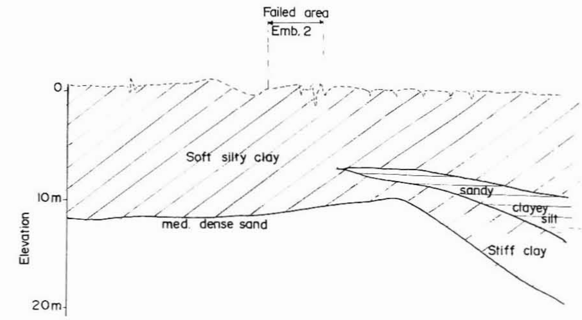


Fig. 2(b). Subsoil profile of embankment 2

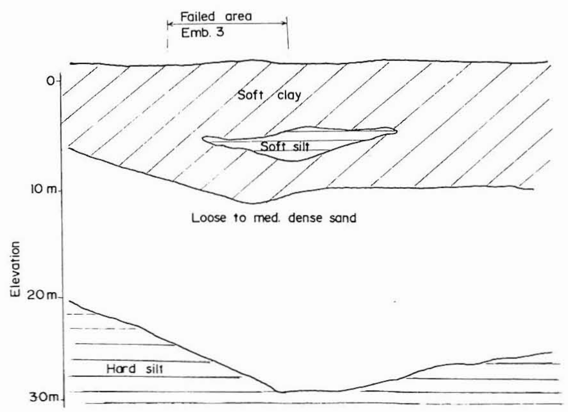


Fig. 2(c). Subsoil profile of embankment 3

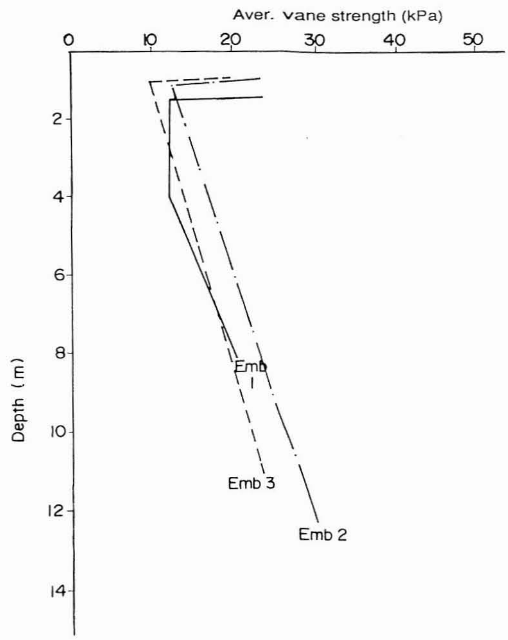


Fig. 3. Average vane strengths

INVESTIGATION OF FAILURE

Soon after the failure, attempts were made to obtain sufficient geotechnical data at each of the distressed embankments to enable analyses of the failure to be carried out.

For this purpose, the following were done:

a) Survey was carried out to map the collapsed section of the embankment to determine the probable mode of failure.
b) Additional soil strength measurements were made. These were in the form of consolidated isotropic undrained triaxial tests on undisturbed samples recovered, using Mazier sampler, from the embankment fill. For the clay foundation, additional soil strengths were measured *in situ* by means of field vanes at locations beneath the embankment centre, at toe and at some distance forward of the toe. According to Brand and Krasaesin (1971), the field vane is the most reliable method for determining undrained shear strength of a soft, reasonably sensitive clay.

ANALYSES OF THE FAILURE

Mode of failure

Cross-sections of the embankment surveyed after the failure are shown in

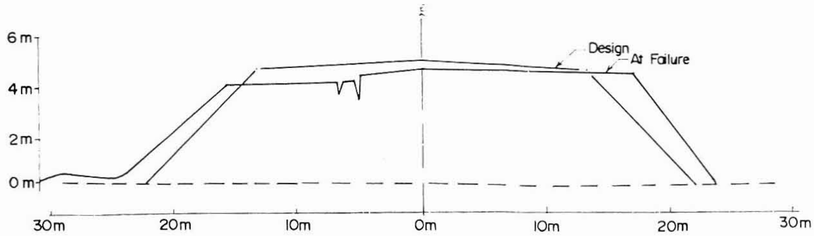


Fig. 4(a). Cross-section of embankment 1

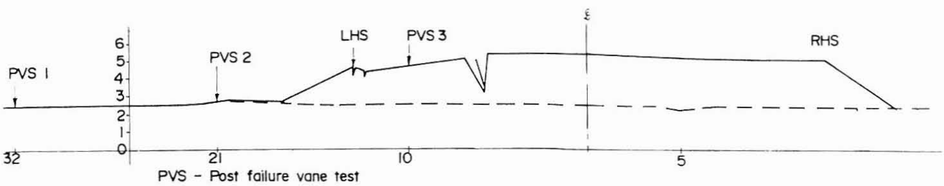


Fig. 4(b). Cross-section of embankment 2

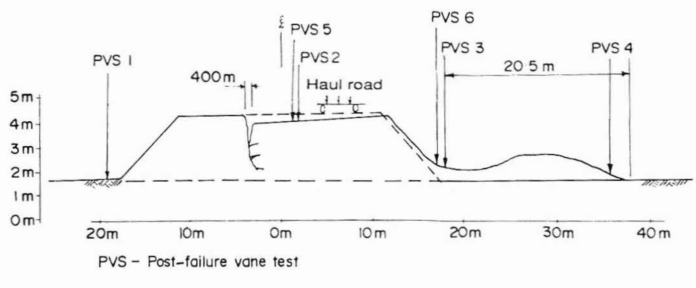


Fig. 4(a). Cross-section of embankment 3

Fig. 4 (a), 4(b) and 4(c). For comparison purposes, designed cross-sections of the embankment are superimposed. In all cases, failure of the embankment was characterized by wide open cracks through embankment fill with a significant heave at the embankment toe. This indicated that the mode of the embankment failure has been that of a rotational slip.

In the case of embankment 1, the cracks extend over a longitudinal distance of 80 m. A 0.25 m differential settlement was measured between the failed and intact portions of the embankment. The toe heave extends over a distance of 7 m from the embankment toe.

In the case of embankment 2, the cracks extend over a longitudinal distance of 110 m. Movement of the ground beyond the embankment toe was in evidence over a distance of about 5 m.

In embankment 3, the failure stretched over a distance of 140 m. A crack about 400 mm wide (at surface) appeared about 4.0 m away from the centreline, with about 1 m differential settlement across the crack. A 1.26 m high heave was observed at the embankment toe, and the width of this upheaval was about 20.5 m.

SOIL STRENGTH PARAMETERS

Additional soil strength measurements were made after the failures. These were in the form of isotropic undrained trial (CIU) test on undisturbed samples of the fill material of each of the three embankment sections. The results of the CIU tests, general descriptions and index properties of the embankment fill are summarized in Table 1. The fills are of sandy, silty clay with greater than 60% of the material in the silt-clay range. The plasticity indices are low, in the range of 17% to 23%. Shear strength parameters are C'= 17 - 42 KPa, and Ø' = 23 - 27°.

TABLE 1
Description and properties of embankment fill

Embankment 1		
Soil Description	:	Dark grey sandy silty clay
Atterberg's Limit	:	W_L = 35% W_p = 17% PI = 18%
Sieve Analysis	:	Gravel 25% Sand 7% Silt and Clay 68%
Max. Dry Density	:	1.95 Mg/m^3 OMC = 12%
Strength Parameters (CIU)	:	C' = 17 kPa Ø' = 27
Embankment 2		
Soil Description	:	Brownish yellow silty clay
Atterberg's Limit	:	W_L = 41% W_p = 18% PI = 23%
Max. Dry Density	:	2.13 Mg/m^3 OMC = 13.5%
Strength Parameters (CIU)	:	C' = 42 kPa Ø' = 26
Embankment 3		
Soil Description	:	Reddish sandy clay
Atterberg's Limit	:	W_L = 42% W_p = 25% PI = 17%
Sieve Analysis	:	Gravel 8% Sand 23% Silt and Clay 69%
Max. Dry Density	:	2.07 Mg/m^3 OMC = 10.2%
Strength Parameters (CIU)	:	C' = 25 kPa Ø' = 23

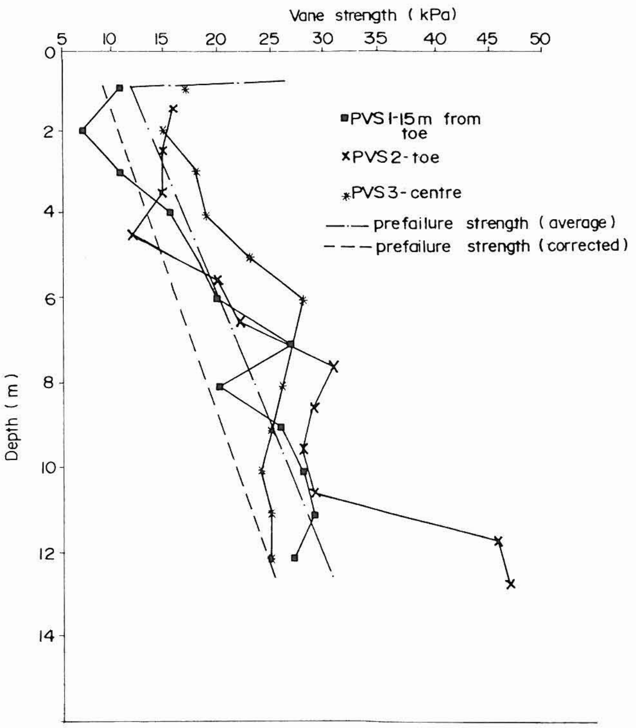


Fig. 5. Post-failure vane strengths of embankment 2

Post failure *in situ* vane shear tests were also carried out for embankments 2 and 3. Positions of the vane tests are shown in *Fig. 4(b) and 4(c)*. For embankment 2, the vane measurements were carried out at a location close to the embankment centre, at toe and at a location about 15 m beyond the toe, about 1 month after the failure. The data obtained are presented in *Fig. 5*. Superimposed also on the figure are the average vane strengths of the initial (pre-failure) investigation (see also *Fig. 3*).

It is of interest to note that there appears to be no significant gain in shear strengths at locations beneath the embankment centre after construction. Note that at this particular embankment section prefabricated vertical drains were installed. There also appears to be no significant difference in undrained strength beneath the embankment centre compared with that at locations at the embankment toe, and away from the toe.

Fig. 6 shows the post-failure vane strengths of embankment 3. These measurements were carried out in November 1992, three months after the failure. As in the case of embankment 2, there appears to be minimal strength gain when the post and prefailure vane strengths are compared. It may therefore be concluded that the failure of both embankments 2 and 3 (and embankments 1) occurred essentially under undrained conditions. A

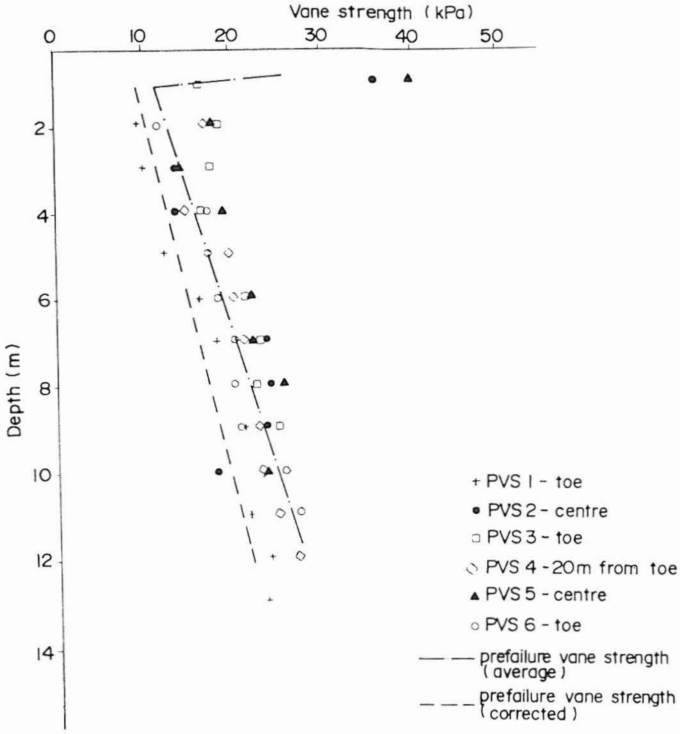


Fig. 6. Post-failure vane strength of embankment 3

total stress ($\varnothing_u = 0$) analysis can therefore be applied. Wolski *et al.* (1989) have suggested that for a single-stage embankment construction on soft clay, the formation can be considered as practically undrained. All these embankments were essentially constructed in a single stage.

Rotational Slip

In the total stress analysis the proportion of undrained shear strength, S_u, mobilized at a point on a given surface of rotation, where the safety factor is F, is:

$$\tau m = \frac{S_u}{F}$$

and, for the whole surface :

$$F = \frac{\sum S_u L}{\sum \tau m\, L}$$

When Bishop simplified method of slices is used for the integration around the circuluar arc, this becomes:

$$F = \frac{\sum S_u L}{\sum W \operatorname{Sin} \beta}$$

where L is the length of the arc of the slice, W is the weight of the slice and β is the angle the normal surface makes to the vertical of rotation.

The above analysis is used for calculating factor of safety of the embankment at failure. The properties of the fill are accounted for with values obtained from the CIU tests summarized in Table 1. Neglect of the embankment strength as suggested by Bjerrum (1973) would be too conservative. It can also be argued that tension crack is a consequence, not the cause of failure. There is evidence from good case histories of embankment failures on soft clays that compacted cohesive fill tends to play a part in resisting failure (Balasubramaniam *et al.* 1989; Brand & Premchitt, 1989).

For the clay foundation, the average vane strengths shown in *Fig. 3* are used as input parameters. These were corrected with Bjerrum's (1973) correction factors for the effect of anisotropy and shear rate. As shown in *Fig. 7,*

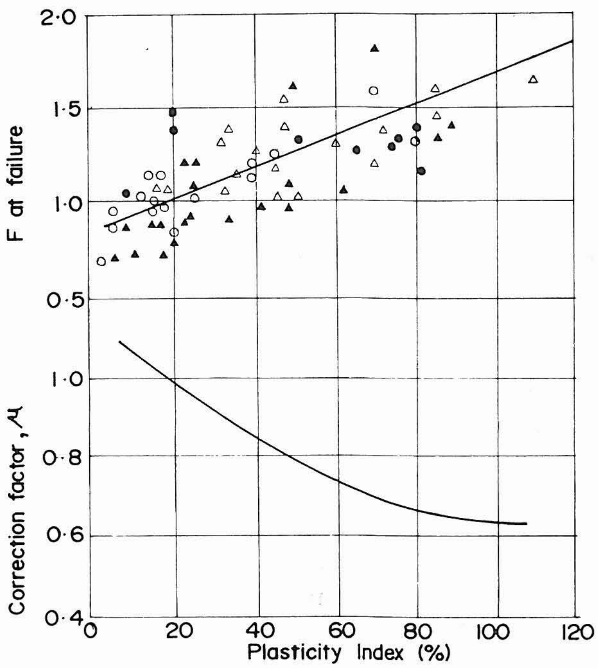


Fig. 7. Relationship between calculated factor of safety and plasticity index for embankment failures, and deduced correction factors for measured vane strengths (Bjerrum 1973)

for plasticity index of 30 - 80%, the correction factor applied is between 0.7 - 0.9. It is of interest to note that the corrected average vane strength of the clay lies close to the lower bound of the field data (See *Fig. 5, 6)*. Note also that for embankment 3 surcharge loading of the traffic is accounted for, estimated in the order of 10 kN/m^2. The results of the calculation are summarized in Table 2.

TABLE 2
Factor of safety of embankment at failure

Embankment	Fill height above OGL (m)	Fill strength C' (kPa) Ø' (deg)	Factor of safety
1	4.6	17 , 27	0.91
2.	3.6	42 , 26	1.04
3	3.2	25 , 23	1.03

The factors of safety of the embankment at failure are essentially equal to unity. This indicated that given representative strength parameters have been obtained for both fill and foundation, the total stress analysis is sufficient for short-term failures of embankment on soft clays, i. e. till end of construction time. It has been suggested that the total stress analysis offers an unambiguous way of estimating stability (Pilot 1972; Brand, 1983; Wolski *et al.* 1989).

OBSERVED BEHAVIOUR OF EMBANKMENT AT THE ONSET OF FAILURE

Rate of Settlement

Fig. 8 and *Fig. 9* show construction histories and settlements of embankment 2 and embankment 3 respectively. In both cases there was a dramatic increase in settlement rate just prior to the failure.

Lateral Deformation

Only embankment 3 was installed with an inclinometer and three piezometers in addition to the settlement markers. *Fig. 10* shows plot of maximum lateral displacement (Δ y) versus centreline settlement (Δ S) of the embankment. It is of interest to note that when the fill was low, i.e. for fill thickness of less than 1 m, the Δ y/ Δ S relation is small with $\Delta y \cong 0.29 \Delta S$. But at the onset of the failure, Δ y increases significantly; from approximately equal to Δ S to almost 3 times Δ S.

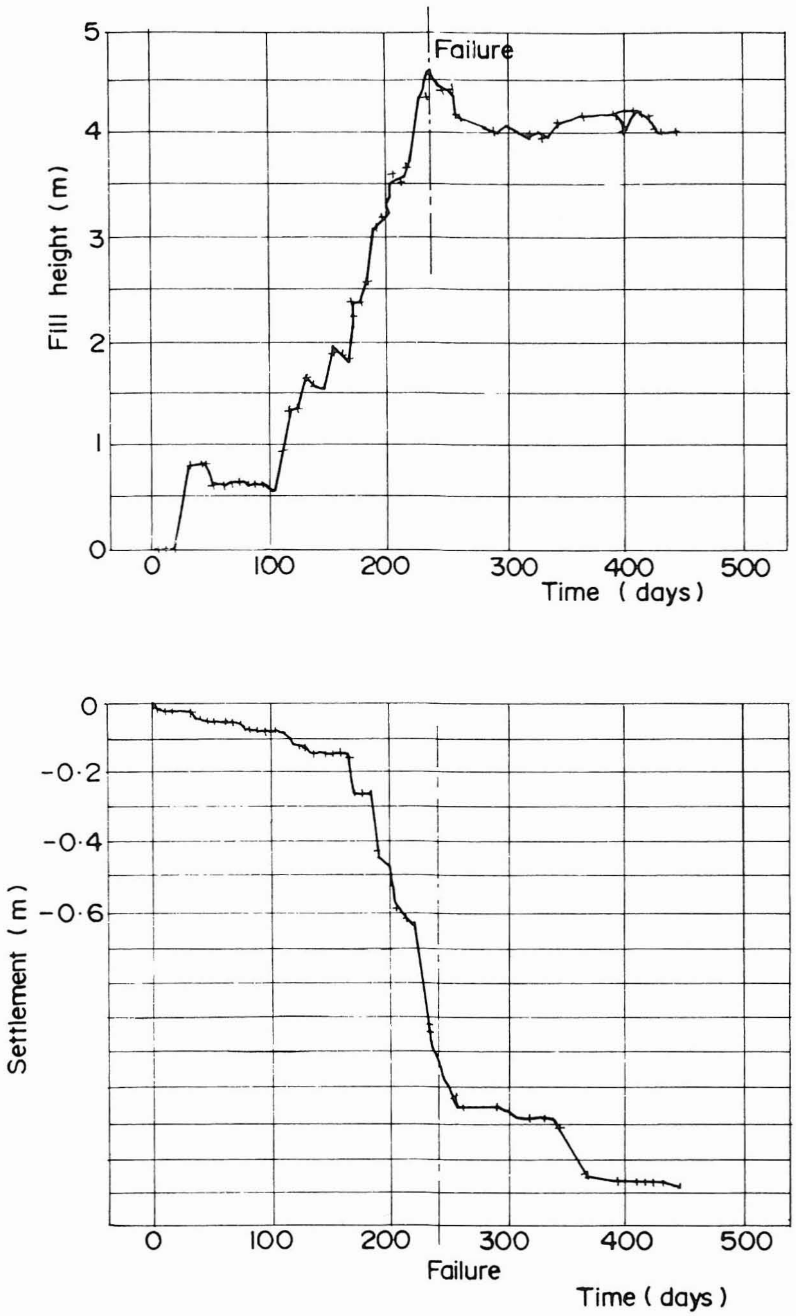
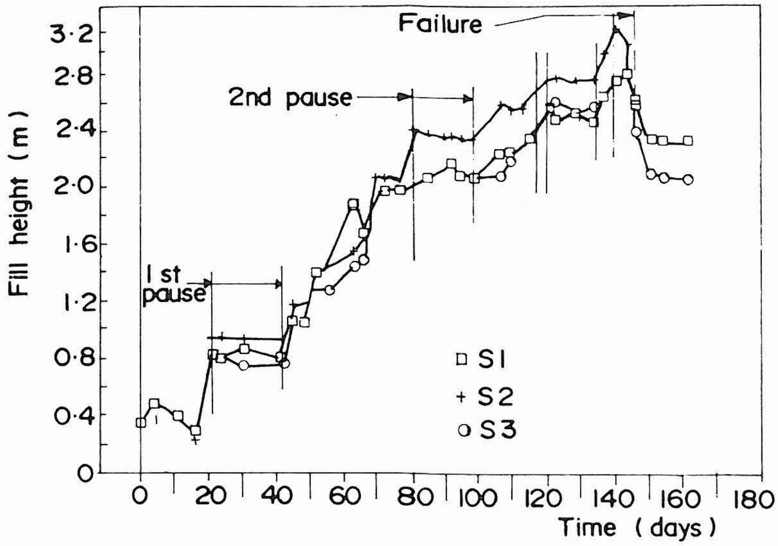


Fig. 8. Construction history and settlement of embankment 2



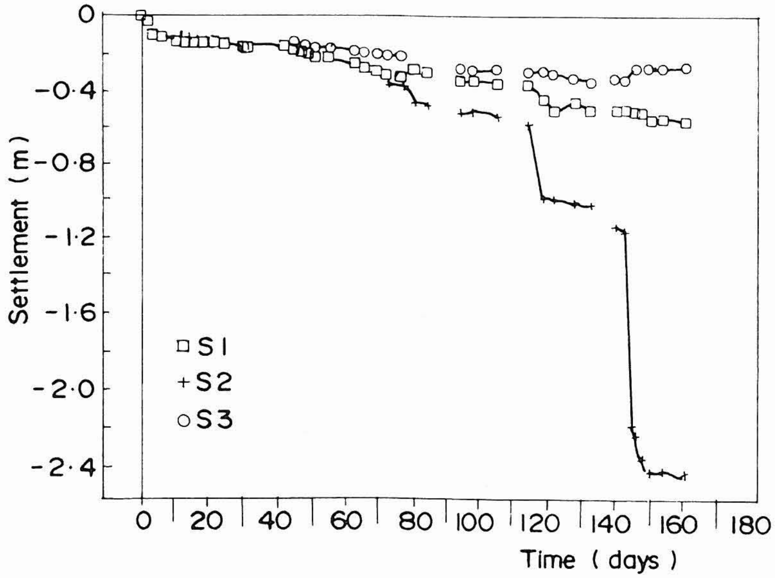


Fig. 9. Construction history and settlement of embankment 3

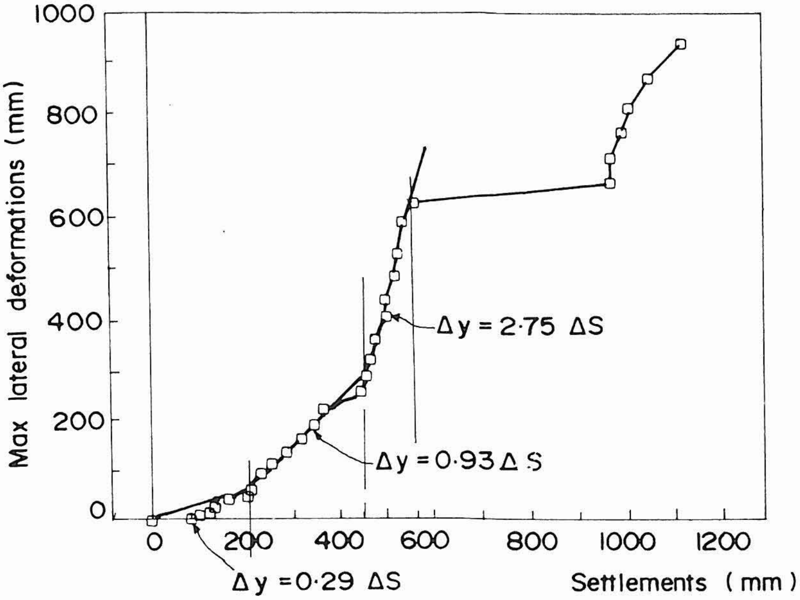


Fig. 10. Max lateral deformations – centreline settlements of embankment 3

Piezometric Response

The variation of excess pore water pressure (Δ u) with time of embankment 3 is shown in *Fig. 11*, while *Fig. 12* shows plot of Δ u with applied vertical stress, $\Delta \sigma_V$.

For low fill (less than 1 m), the piezometric response of the clay foundation is low; Δ u = 0.44 $\Delta \sigma_V$. The increase in the pore water pressure then becomes approximately equal to the applied vertical stress. However, on the onset of failure, the piezometric response is large; $\Delta u >> \Delta \sigma_V$. This is probably due to the following scenario. Prior to collapse of the embankment, local failures may have been initiated in the subsoil beneath the embankment. This results in strain softening, hence an increase in total horizontal stresses and pore water pressures. Therefore, prior to the failure one should expect $\Delta u > \Delta \sigma_V$, and this is shown in the pore pressure plot in *Fig. 12.* A similar observation of $\Delta u > \Delta \sigma_V$, at failure has also been made by D'Appolonia *et al.* (1971), and Davies and Parry (1985).

CONCLUSION

Provided adequate information has been obtained on the strengths of both fill and soft clays, the total stress approach can be used for design of embankment on soft ground. Failure of the embankment occurred essentially at factor of safety equals unity, based on stability analysis with vane readings corrected with Bjerrum's factor.

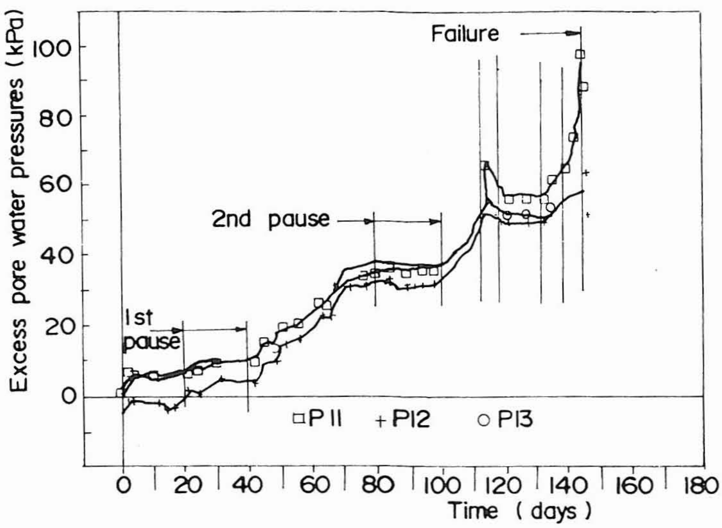


Fig. 11.Excess pore water pressure of embankment 3

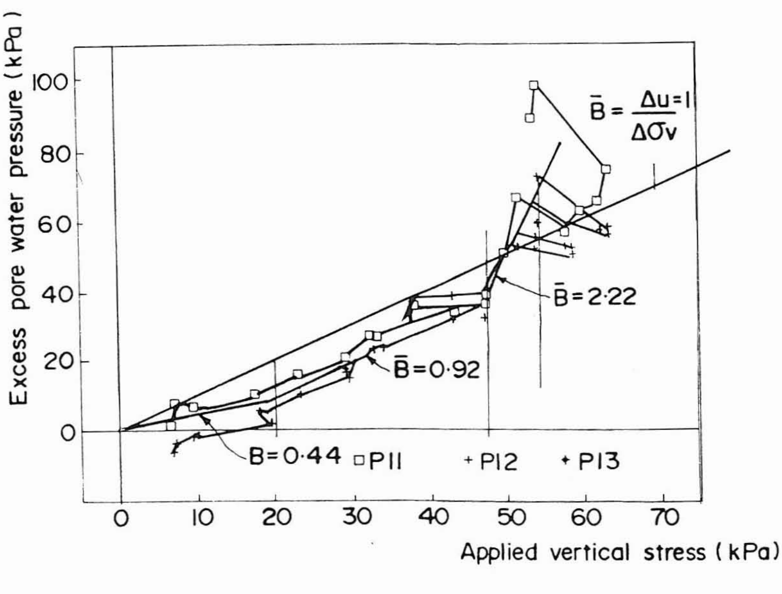


Fig. 12. Pore pressure ratio ($\bar{B} = \triangle u/d\sigma v$) of embankment 3

For cases of compacted cohesive fill, the strength properties of the fill should be accounted for in the stability analysis. The corrected average vane strengths of the soft clay generally lie close to the lower bound any of the field data.

Failure of the embankment is preceded by soft response of the foundation with regards to generation of excess pore water pressures, settlement and lateral deformations.

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