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PREDICTED AND OBSERVED PERFORMANCE OF AN EMBANKMENT BUILT TO FAILURE ON SOFT CLAY

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SYNOPSIS

An important 'prediction' symposium was held in Kuala Lumpur in November 1989 which revolved around 14 trial embankments constructed of compacted sandy clay on soft clay at Muar flats, West Malaysia. One embankment was built rapidly to failure, and 31 individuals made predictions as to the failure height and failure mode before the results were known. This paper compares the predicted and observed performance of the embankment, and reanalyses the failure. The predictions of fill thickness at failure ranged between 2.8 m and 9.5 m, compared with the actual value of 5.4 m. It is demonstrated that the strength of the fill material played a critical part in the embankment stability.

INTRODUCTION

In November 1989, the International Symposium on Trial Embankments on Malaysian Marine Clays was held in Kuala Lumpur. This outstanding event, organized by the Malaysian Highway Authority, was based completely on the predictions and performance of 14 trial embankments constructed on a single site alongside the North-South Expressway on the Muar Flats, about 50 km due east of Malacca on the southwest coast of West Malaysia. Thirteen of the embankments were concerned only with settlement performance, and comprised 11 embankments designed by different consultants/contractors and two 'control' embankments. The fourteenth embankment was built rapidly to failure, and this was used as the subject of a prediction 'competition', much like that staged many years ago at the Massachusetts Institute of Technology (MIT, 1974).

Prior to the construction of the test embankment, four eminent ('major') predictors (A.S. Balasubramaniam, J.-P. Magnan, A. Nakase and H.G. Poulos) were invited to predict pore pressures and displacements during construction, as well as the failure height and failure surface, using extensive amounts of sophisticated soil data. In addition, predictions as to the failure height and failure surface were made by 27 'minor' predictors before or during the Symposium on the basis of a booklet of summarized soil data distributed a few weeks beforehand. Details

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of the measured performance were revealed at the Symposium. The predictions of embankment performance were therefore strictly type 'A' predictions (i.e. made prior to the event) as defined by Lambe (1973).

This paper summarizes the predictions and performance of the test embankment built rapidly to failure. It is a highly condensed version of the Moderator's Report (Brand & Premchitt, 1989) presented to the Symposium, and it concentrates almost exclusively on the predicted and observed failure height and failure mode. The significant lessons learned from this exciting exercise have important implications for the design of embankments on soft clays elsewhere. The full Proceedings of the Symposium (MHA, 1989) contain what is probably the most comprehensive collection of high quality data on embankment performance ever published, and they are an essential addition to any geotechnical library.

THE MUAR CLAY

The formation of soft clays in Southeast Asia is generally closely associated with the past fluctuations in sea level (Cox, 1968, 1970a, 1970b). The clays along the west coast of Peninsular Malaysia constitute a coastal plain of marine or brackish-water clay up to about 20 m thick, with an average lateral extent of about 25 km.

In Fig. 1 is shown the soil profile to a depth of about 26.5 m (-24mRL) at the Muar test site, together with a summary of some of the measured properties. Figure 2 summarizes the test data on water content, Atterberg limits, vane shear strength and Dutch cone (piezocone) penetration resistance for the soft Muar clay to a depth of 15 m (-12.5mRL).

It can be seen that there is a relatively strong 2 m thick crust on the Muar clay, below which the water content is as high as 100% and generally exceeds the liquid limit. The plasticity index ranges between about 40% and 50%. The undrained shear strength, as measured by the field vane, has a minimum value of about 8 kPa at a depth of 3 m and then increases approximately linearly, with an s_u/σ'_v ratio in the range 0.27 to 0.37. The Dutch cone resistance clearly indicates the thickness of the crust and defines precisely the boundary at -5.5mRL between the upper and lower soft clays.

EMBANKMENT CONSTRUCTION AND FAILURE

The test section (Fig. 3) was constructed on a base 55 m wide and 90 m long, initially to a height of 2.5 m, beyond which the test embankment proper was built to leave a 15 m wide berm on three sides. The base dimensions of the test

DEPTH, m +2.5 mRL	SOIL DESCRIPTION		p_c (kPa)	$\frac{C_c}{1+e_o}$	k_h (m/sec)
+0.5	CRUST	Yellowish brown mottled red CLAY with roots, root holes and laterite concretions	110	0.3	-
-5.6	UPPER CLAY	Light greenish grey CLAY with a few shells, very thin discontinuous sand partings, occasional near-vertical roots and some decaying organic matter (<2%)	40	0.5	4×10^{-9}
-15.3	LOWER CLAY	Grey CLAY with some shells, very thin discontinuous sand partings and some decaying organic matter (<2%)	60	0.3	1×10^{-9}
-15.9	PEAT	Dark brown PEAT with no smell			
-19.9	SANDY CLAY	Greyish brown sandy CLAY with a little decaying organic matter	60	0.1	2×10^{-7}
	SAND	Dark grey, very silty medium-to-coarse SAND (SPT >20)	-	-	-

Fig. 1 Soil profile and some soil properties at the site of the Muar test embankment

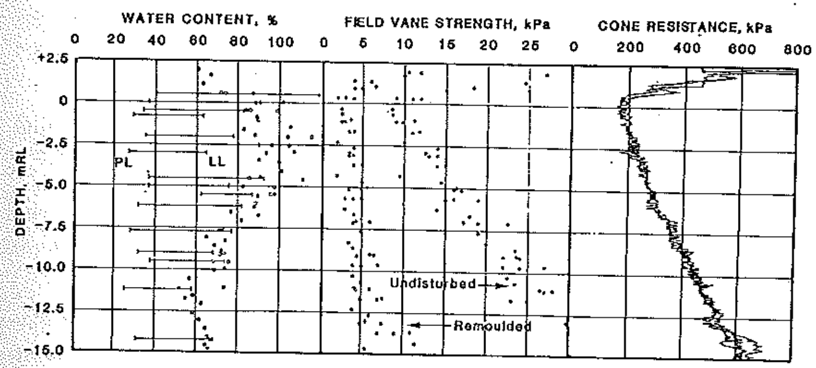

Fig. 2 Water contents, Atterberg Limits, vane strengths and Dutch cone resistances at the test site

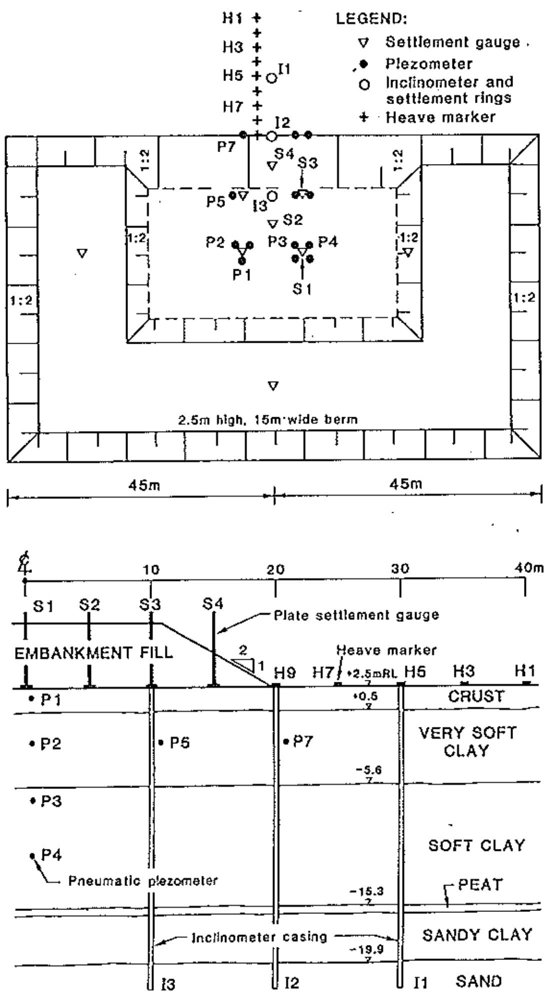

Fig. 3 The test embankment showing the positions of key instrumentation: (a) plan, (b) cross-section through centre

embankment can therefore be taken to be 40 m wide and 60 m long. The instrumentation installed in the ground prior to construction is shown on the plan of the site in Figure 3(a) and on the cross-section through the centre of the embankment in Figure 3(b).

The embankment was constructed directly on the topsoil and grass that remained after the rubber trees had been uprooted. The fill was compacted in 0.2 m layers, at a nominal rate of 0.4 m per week until failure occurred.

The fill material consisted of decomposed granite with the consistency of sandy clay (4% gravel, 50% sand, 8% silt, 38% clay). The average Atterberg limits were LL = 108%, PL = 42%, PI = 66%. The bulk densities of the compacted material were measured by core-cutter and sand-replacement methods. The mean value was about 2.04 Mg/m^3, and the compacted water content ranged between 13.9% and 21.6%, with a mean of 16.2%.

A series of four consolidated drained (CD) triaxial tests was carried out on 100 mm diameter specimens cut from the compacted fill and back-saturated prior to shear. The results are shown in Fig. 4(a) as Mohr's circles at failure, together with the strength envelope interpretation of c' = 14 kPa & $\varnothing'$ = 31° provided by the Symposium organizers. It should be noted that the measured dry densities of the compacted fill specimens used in the CD tests averaged only 1.50 Mg/m^3, compared with an average field value of 1.77 Mg/m^3, which probably meant that the specimens swelled significantly during back-saturation. In addition to the CD tests, four unconsolidated undrained (UU) tests were carried out on 100 mm diameter specimens cut from the compacted fill and sheared at their insitu water contents, the results of which are summarized in Fig. 4(b).

The embankment failed dramatically on day 100 at a fill thickness of 5.4 m. The failure height above the original ground surface was 4.7 m, so the embankment had settled 0.7 m at failure. Failure was preceded by a longitudinal crack approximately along the embankment centre-line. Once differential vertical movement commenced, the failure took less than a minute, with the embankment coming to rest as shown in Fig. 5.

The shape of the failure surface through the soft clay was determined, with reasonable accuracy, from measurements taken immediately after failure on the surface settlement gauges, surface heave markers and inclinometers. As can be seen from Fig. 5, the failure surface through the embankment itself was very close to vertical.

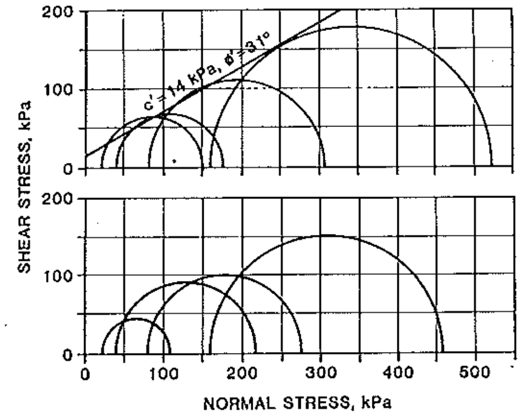

Fig. 4 Results of triaxial tests on 100 mm specimens of compacted fill: (a) CD tests after back saturation, (b) UU tests without saturation

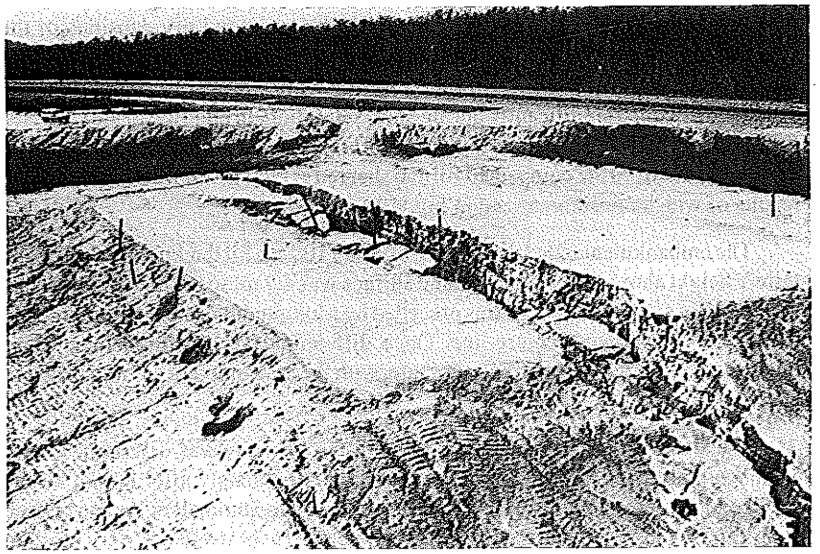

Fig. 5 General view of the embankment just after failure

PREDICTED AND OBSERVED FAILURE CONDITIONS

Summary

The predictions of failure height and failure mode made by the four 'major' and 27 'minor' predictors are summarized in Table 1, which also shows the observed (actual) failure conditions. It can be seen that the predicted failure thickness for the embankment varied between the wide limits of 2.8 m and 9.5 m. The vast majority of the individuals concerned used the Fellenius (Swedish) or Bishop (1954) undrained analysis based on vane strengths, although several used published stability charts. However, there were considerable differences in the vane correction factors μ (Bjerrum, 1972, 1973) and in the strengths adopted for the compacted fill. These two factors alone generally determined the value of the predicted failure thickness. A few predictors made allowance for clay strength increases caused by partial consolidation during construction. It should be noted that the 'minor' predictors were only given the parameters c' = 14 kPa & $\varnothing'$ = 31° for the fill strength; they were not provided with the results shown in Fig. 4.

The failure conditions predicted by the four 'major' predictors are summarized in Table 2 for comparison with the actual conditions.

All the predicted failure surfaces are plotted in Fig. 6 for comparison with the observed failure surface. It can be seen that the predicted surfaces varied enormously, due partly to the different vane shear strengths adopted. In many cases, however, the stability analyses did not establish the critical surface (and therefore the minimum factor of safety), because the principle of the mid-point circle was not complied with.

The principle of the mid-point circle (Brand & Shen, 1984) states that the centre of the critical circle for an undrained analysis must lie vertically above the mid-point of the slope, provided the shear strength of the soft clay does not vary laterally. The application of this simple principle greatly aids the $\varnothing = 0$ analysis of base failures of embankments and other structures by facilitating the rapid determination of the location of the critical circular slip surface.

Shen's Prediction

Of all the 31 predictions made as to the failure thickness and failure surface of the embankment, that of Mr J.M. Shen came closest to the actual failure conditions (Table 1). For all practical purposes, his predicted failurev thickness of 5.3 m was the same as the actual failure thickness of 5.4 m. Perhaps more remarkable, however, was that his predicted failure surface (Fig. 7) was almost identical to that

Table 1 Summary of the 31 predictions made for failure thickness and failure surface of the test embankment (see Fig. 6)

Predictor		Method of Analysis	Strengths Adopted		Predictions for Failure, m		
					Failure Surface		Fill Thickness
No.	Name and Organization		Soft Clay	Compacted Fill	Depth	Radius	
1	J.S. Younger, J. Riyanto & C. Setjadiningrat, Bandung Institute of Technology, Indonesia	Fellenius	0.68 – 0.86 × vane (μ varied with PI)	s_u = 160 kPa (from cone data)	–	–	9.5
2	K.L. Siu, Special Projects Division, GCO, Hong Kong	Bearing capacity	s_u = 20 kPa	Zero	7.0*	14*	8.0*
3	R.A. Fraser, Maunsell Geotechnical Services Ltd, Hong Kong	Empirical chart (Kurihara, 1977)	Kurihara Fig. 6.5, with w = 80%	Zero	13.0*	~25*	6.5
4	'Trainees', Design Division, GCO, Hong Kong	Fellenius	1.2 × vane (consolidation)	c = 14 kPa, Ø = 31° (partially cracked)	8.0*	20*	6.4*
5	'Design Team', Design Division, GCO, Hong Kong	Bishop	1.2 × vane (consolidation)	c = 14 kPa, Ø = 31°	8.0*	18*	6.0*
6	C.O. Lo & Y.C. Lo, Port Works Division Civil Engineering Department, Hong Kong	Bishop	0.8 × vane	c = 14 kPa, Ø = 31° (partially cracked)	5.2*	9*	6.0
7	'Engineer', Island East Division, GCO, Hong Kong	Stability chart (NHRI, 1982)	0.8 × vane	Zero	4.0	14	6.0
–	ACTUAL FAILURE				~8.0	~26	5.4
8	J.M. Shen, Materials Division, GCO, Hong Kong	Fellenius	0.8 × vane	c = 50 kPa, Ø = 0	8.0	19	5.3
9	A.S. BALASUBRAMANIAM et al, Asian Institute of Technology, Bangkok, Thailand	Fellenius	0.9 × vane	c = 19 kPa, Ø = 26°	5.0	13	5.0
10	T.S.K. Lam, Special Projects Division, GCO, Hong Kong	Bishop	1.2 × vane (consolidation)	c = 14 kPa, Ø = 31°	6.0*	16*	5.0*
11	P. To, Special Projects Division, GCO, Hong Kong	Stability chart (Low, 1989)	s_u = 12 kPa (consolidation)	c = 14 kPa, Ø = 31°	8.0	19	5.0
12	'E & N Sections' Island East Division, GCO, Hong Kong	Stability chart (Taylor, 1948)	s_u = 15 kPa	Zero	–	–	5.0
13	S.H. Mak, Island East Division, GCO, Hong Kong	Stability chart (Low, 1989)	0.8 × vane	c = 14 kPa, Ø = 37° (Ø from cone data)	5.0	14	5.0
14	M.P. Chang, Nanyang Technological [illegible]	Bearing capacity	s_u = 9 kPa	c = 0, Ø = 40°	3.5	~9.5	4.7
15	C.P. Wroth, Oxford University, UK	'Judgement' based on Invited Predictors	'Corrected vane'	'Some contribution'	–	–	4.6
16	G. Wong, Greg Wong & Associates, Hong Kong	'Experience' checked by bearing capacity	Average vane, with N_c = 6	Zero (cracked)	–	–	4.4
17	P.K. Chen, Ove Arup & Partners, Hong Kong	Janbu	0.8 × vane	c = 14 kPa, Ø = 31°	5.0*	10*	4.0*
18	S. Buttling, Scott Wilson Kirkpatrick & Partners, Hong Kong	Janbu	0.8 × vane	Zero (cracked)	4.5	14	4.0
19	H. Aboshi, Fukken Co. Ltd, Hiroshima, Japan	Fellenius	0.85 × vane (+ 2.5 kPa consolidation)	Zero	4.5*	10*	3.9*
20	J.P. MAGNAN, Laboratoire Central des Ponts et Chaussées, Paris, France	Bishop	0.85 × vane	c = 20 kPa, Ø = 0	7.2–11.0	13–21	3.8
21	H.G. POULOS, C.Y. Lee & J.C. Small, Sydney University, Australia	Fellenius	Vane, lower bound	Zero (cracked)	5.9	13	3.8
22	M.H. Goldsworthy, Howard Humphries & Partners, Johore, Malaysia	Jakobson (1948)	s_u = 13.4 kPa (average vane to 8 m)	Zero (Cracked)	8.0	13	3.8
23	H.S. Tan, H. Ernst & C.C. Ladd, Massachusetts Institute of Technology, USA	Bishop	Vane/SHANSEP (Ladd & Foott, 1974)	c = 0, Ø = 31°	5.8	12	3.8
24	B.N. Leung, S.H. Tse & Y.C. Chan, Mainland West Division, GCO, Hong Kong	Bishop	0.85 × vane	c = 14 kPa, Ø = 31° (partially cracked)	6.0	11	3.7
25	'S Section', Island East Division, GCO, Hong Kong	Bishop	0.8 × vane	Zero (cracked)	8.0	13	3.6
26	A. NAKASE & J.Takemura, Tokyo Institute of Technology, Japan	Fellenius	$s_u = 0.27\sigma'_v$	c = 0, Ø = 30°	4.6	9.5	3.5
27	D.T. Bergado, Asian Institute of Technology, Bangkok, Thailand	Bishop	0.8 × vane	c = 19 kPa, Ø = 26°	8.0*	17*	3.5*
28	G. Aas & O. Eide, Norwegian Geotechnical Institute, Oslo, Norway	Bishop (effective in crust)	~0.63 - 0.7 × vane (μ varied with PI)	Zero (cracked)	5.0	12	2.9
29	'Student', Development & Airport Division, Civil Engineering Department, Hong Kong	Fellenius	Vane	c = 14 kPa, Ø = 0	5.0*	11.5*	2.8*
30	T.W. Lo, Nanyang Technological Institute, Singapore	Fellenius	0.75 × vane	Zero	3.5*	16*	2.8*
31	K.S. Wong, Nanyang Technological Institute, Singapore	Finite elements	0.8 × vane, E_u = 200 s_u	c = 14 kPa, Ø = 31°	4.0*	16*	2.8*

Notes : (1) All stability analyses were carried out in terms of total stress unless otherwise stated.
(2) * Signifies that the predicted failure surface is not the critical one for the assumptions adopted, usually because the mid-point circle principle is violated.

Table 2 Actual failure conditions for the test embankment compared with predictions of 'major' predictors (all values in metres)

Predictor	Fill Thickness	Fill Height	Slip Surface Depth	Maximum Embankment Settlement	Maximum Surface Heave	Maximum Lateral Movement	Excess Pore Pressure in Piezometer P2
Bala	5.0	4.35	5.0	0.65	0.18	0.35	1.0
Nakase	3.5	3.20	4.6	0.30	0.25	0.50	6.2
Poulos	3.8	3.45	5.9	0.35	0.05	0.16	5.7
Magnan	3.8	3.25	7.2–11.0	0.55	0.09	0.12	7.0 ?
Average	4.0	3.55	6.1	0.50	0.14	0.28	5.0
ACTUAL	5.4	4.70	8.2	0.70	0.15	0.37	9.3

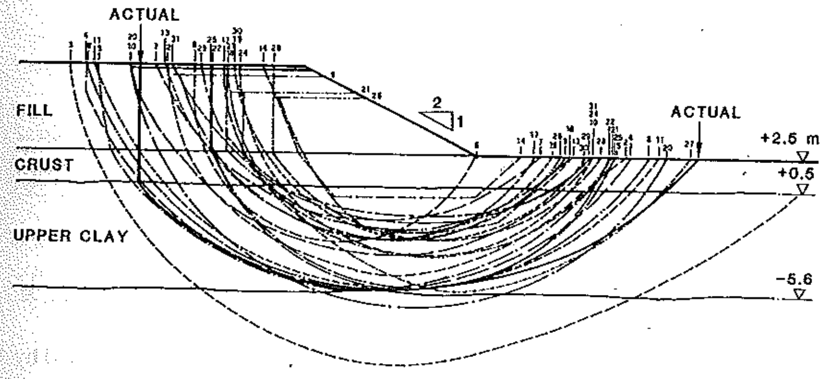

Fig. 6 The failure surfaces predicted by all 31 predictors (see Table 1 for surface numbers)

deduced from the field measurements. It is therefore of considerable interest to examine how Shen was able to make such an accurate prediction of the failure conditions.

Shen used a Fellenius total stress analysis, together with the average vane shear strengths corrected by a factor of 0.8. Of course, his analysis followed the mid-point circle principle, so that he was able to search economically through a large number of potential critical failure surfaces.

The initial selection of the fill strength was the first major step in Shen's stability prediction. On the basis of the limited information available to him (the CD parameters and Dutch cone data), he decided that an undrained strength between 25 and 50 kPa would be appropriate. His extensive experience with laboratory test results on sandy clay formed from the decomposition of granites in Hong Kong provided support for this strength range.

Central to Shen's prediction was his judgement of the depth to which the failure surface would penetrate the soft clay. He was confident that this surface would touch the interface between the upper and lower soft clays, since high pore pressures were likely to develop there because of the significant permeability difference.

Shen's stability analysis then consisted of adjusting the fill thickness and fill strength to give a factor of safety of unity on a critical surface which touched the boundary between the upper and lower clay strata. He found this condition to be satisfied for a fill thickness of 5.3 m and an undrained fill strength of 50 kPa.

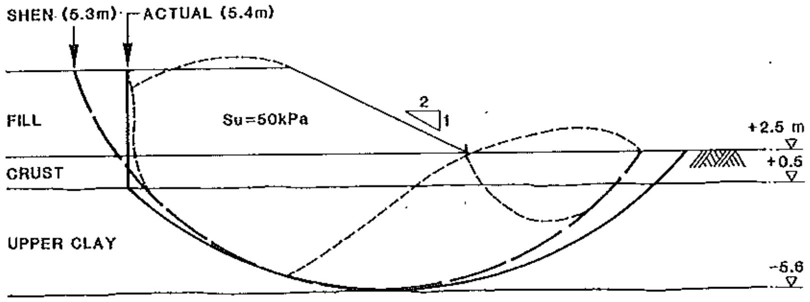

Fig. 7 Embankment failure thickness and failure mechanism predicted by J.M. Shen (see Table 1)

REANALYSIS OF THE EMBANKMENT FAILURE

In the Moderator's Report (Brand & Premchitt, 1989), the results are given of a limited amount of reanalysis of the failure of the test embankment to try to explain why it failed at a thickness of 5.4 m along the surface that it did.

Limit equilibrium stability analyses were carried out for various thicknesses of the embankment. For this purpose, a Bishop total stress analysis was adopted based on average vane shear strengths, and the fill strength was varied along the circular arc assumed to continue through the fill.

When the fill strength was assumed to be zero, a failure thickness of 3.4 m was obtained using uncorrected vane strengths. For fill with c = 14 kPa & Ø = 30°, the failure thickness was found to be 4.1 m. Where corrected vane strengths were used, the corresponding fill thicknesses were 2.7 m and 3.3 m.

On the assumption that Ø = 30° for the compacted fill, a back-analysis was undertaken for the 5.4 m embankment, to give c = 37 kPa and c = 51 kPa for the fill on the basis of uncorrected and corrected vane strengths respectively.

A series of stability analyses using uncorrected vane strengths was also carried out to examine the effect of fill strength on the theoretical failure thickness of the embankment. The results are illustrated in Fig. 8, which is based on a 'reference' fill strength of c = 37 kPa & Ø = 30°. It can be seen that the angle Ø has very little effect on the stability, because the normal stress on the assumed failure surface is taken in the method of slices to be the normal component of the vertical weight of the slice, and this component approaches zero as the failure surface approaches the vertical.

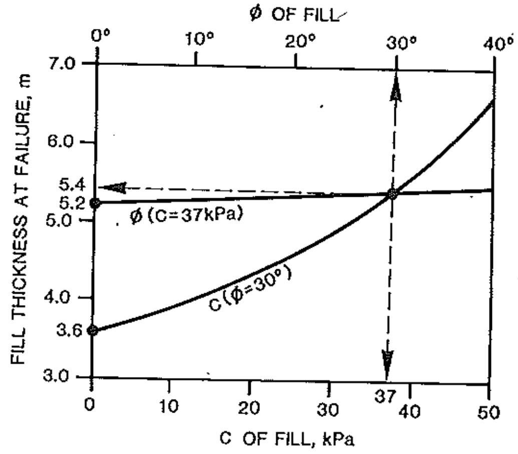

Fig. 8 Theoretical variation in fill thickness at failure with fill cohesion c and fill friction angle Ø, based on uncorrected vane shear strengths of the clay

On the other hand, the predicted failure thickness is appreciably affected by changes in the value of the cohesion c.

From Fig. 8, it can be seen that a Ø-value of 30° is equivalent to only about c = 5 kPa in the stability analysis, in that the failure thickness increases by about 0.2 m if Ø is increased from 0 to 30° or if c is increased from 0 to 5 kPa. For the embankment to fail at a thickness of 5.4 m, therefore, the undrained strength of the fill needs to be 42 kPa if uncorrected vane strengths are adopted for the soft clay. If corrected (μ = 0.8) vane strengths are used, the fill strength needs to be 56 kPa. It is of interest to note that, on the basis of slip circle analysis, the embankment can theoretically be built to an infinite height if the undrained strength of the fill exceeds about 120 kPa, irrespective of whether corrected or uncorrected vane strengths are used for the soft clay.

The strength testing carried out on the compacted fill samples taken from the embankment was unfortunately not conclusive, but the UU triaxial test data in Fig. 4(b) suggests that the undrained strength was in excess of 40 kPa.

On the basis of the $s_u/\varnothing_v'$ values of 0.27 and 0.37 for the upper and lower soft clay respectively, undrained strength increases could be related directly to degree of excess pore pressure dissipation (or degree of consolidation). Figure 9 illustrates the effect of the degree of consolidation, and its associated clay strength increase (uncorrected), on the theoretical fill thickness at failure for three assumed fill strengths. If the fill strength is neglected, a clay strength increase of about 7 kPa ($\Delta u = 24\%$) is needed to increase the theoretical fill thickness at failure from 3.4 m to 5.4 m. For a fill strength of c = 14 kPa & $\varnothing = 30°$ (equivalent to $s_u = 19$ kPa), the clay strength increase needed for a theoretical failure thickness of 5.4 m is about 5 kPa ($\Delta u = 15\%$). These results are summarized in Table 3, which also shows the higher necessary strength increases if corrected vane strengths are used for the analysis.

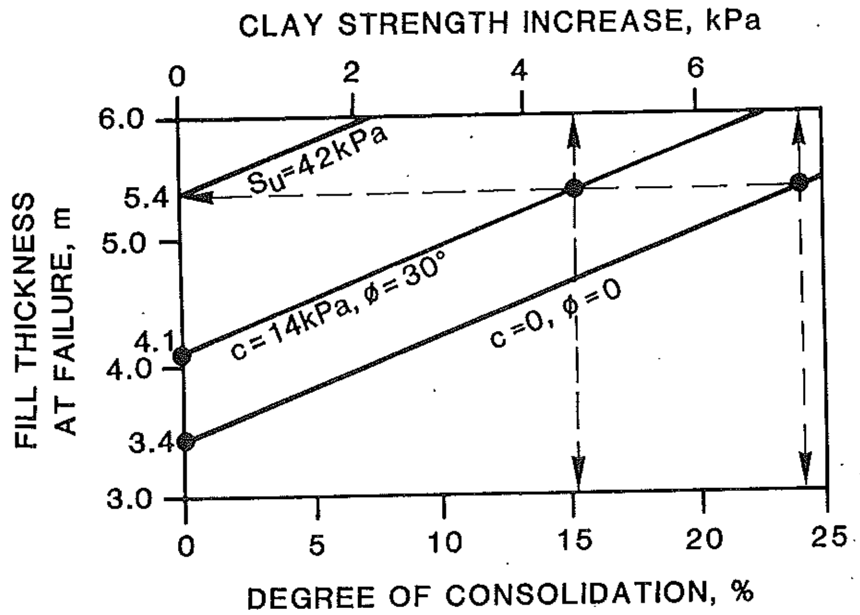

Fig. 9 Theoretical variation in fill thickness at failure with degree of consolidation of the soft clay, for three fill strengths, based on uncorrected vane shear strengths of the clay

Table 3 Theoretical increases in clay strength needed for the embankment to fail at the observed fill thickness of 5.4 m

Assumed Fill Strength, s_u, kPa	Failure Thickness, m		Clay Strength Increase for 5.4 m, kPa	
	Vane	0.8 × Vane	Vane	0.8 × Vane
0	3.4	2.7	7 ($\Delta u = 24\%$)	9.5 ($\Delta u = 32\%$)
19	4.1	3.3	5 ($\Delta u = 15\%$)	7.5 ($\Delta u = 26\%$)
42	5.4	4.3	0	2.5 ($\Delta u = 8\%$)
56	6.7	5.4	–	0

The embankment at failure was analysed to establish the theoretical failure surface as accurately as possible. Uncorrected vane strengths were used for the soft clay. It was assumed that the surface continued as a circular arc through the fill, and that the undrained strength of the fill was 42 kPa. It should be noted that the critical surface would have been the same if a fill strength of 56 kPa had been used with corrected vane strengths for the soft clay.

The results of the analysis are shown in Fig. 10 for comparison with the observed failure surface. This illustrates the extreme difficulty of predicting the failure surface that actually occurred, since there is a broad 'failure zone' in which the theoretical factor of safety varies between only 0.98 and 1.02.

PREDICTED AND OBSERVED PORE PRESSURES AND DISPLACEMENTS

Only the four 'major' predictors were asked to predict pore pressures and displacements. They were required to predict pore pressures at piezometers P1, P2, P3, P4, P5 & P7 (Fig. 3) throughout the loading, horizontal movement profiles at inclinometers I1, I2 & I3, and changes in the ground surface profile. Full details of their predictions (made by means of Finite Element Analyses) and the measured

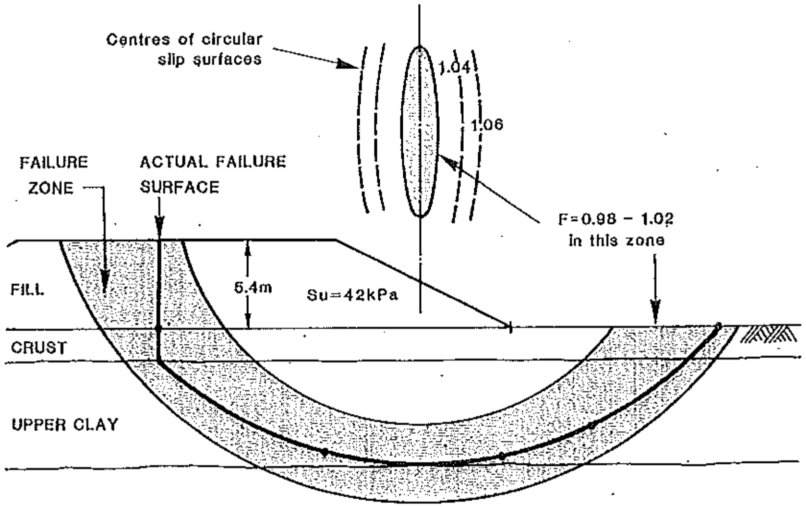

Fig. 10 Reanalysis of the embankment failure, showing the possible failure zone

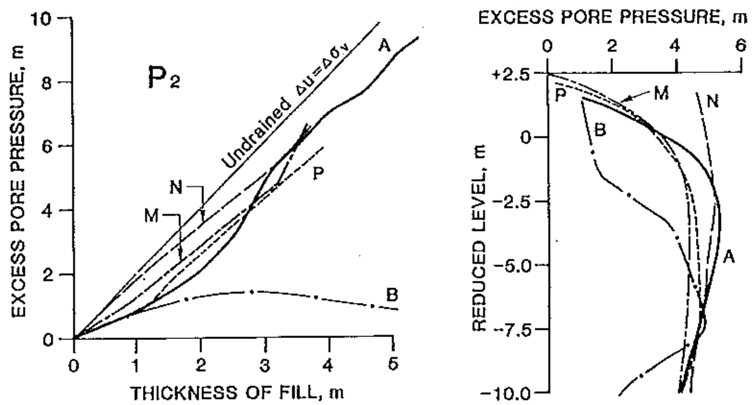

Fig. 11 Predicted and actual pore pressures: (a) in piezometer P2, (b) under embankment centre at 3 m fill thickness (A = Actual, B = Balasubramaniam, M = Magnan, N = Nakase, P = Poulos)

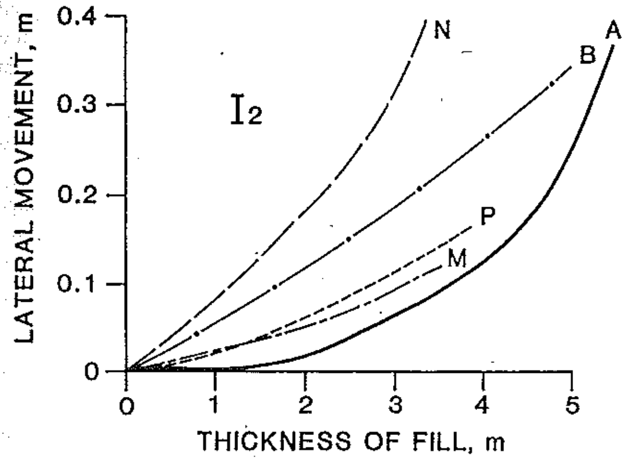

Fig. 12 Predicted and actual horizontal displacements in inclinometer I2 (A = Actual, B = Balasubramaniam, M = Magnan, N = Nakase, P = Poulos)

values are given in the Moderator's Report (Brand & Premchitt, 1989), but typical results for pore pressures and horizontal displacements are shown in Figs. 11 & 12. It can be seen that the predictions of pore pressure were fair, but displacement predictions were universally very poor.

The vertical settlements of the ground surface were predicted tolerably well by some predictors, but surface heaves were universally overestimated. There was in fact almost immeasurable surface heaving until the embankment approached failure.

CONCLUSIONS

The Kuala Lumpur Prediction Symposium proved to be a valuable exercise for all concerned. The test embankment was designed and constructed to very high standards, the data obtained being of the highest quality. When taken together, the 14 trial embankments probably comprise the most comprehensive series of trial embankments ever undertaken at one time. The vast amount of data collected (MHA, 1989) will be of lasting value to all those concerned with the design and performance of embankments on soft clay.

Although only one of the 'major' predictors and a few of the 'minor' predictors closely predicted the observed failure thickness of the embankment, it is noteworthy that the vast majority underpredicted the thickness. This perhaps demonstrates the natural design conservatism of the average geotechnical engineer.

The strength of the compacted fill material appears to have been the key to the embankment's stability. Back-analyses gave an estimated value of between 42 kPa and 56 kPa for the undrained strength of the compacted fill, and this appears to accord reasonably well with the limited amount of laboratory test data. It is also possible that some increase occurred in the undrained strength of the soft clay due to partial consolidation during the construction period of the embankment.

The excess pore pressures under the embankment were predicted reasonably well by some 'major' predictors, but these predictions would not have been adequate for the purpose of an accurate effective stress stability analysis. Total stress stability analyses are clearly much more reliable, and all the predictors adopted this simple approach to predict the failure conditions.

Despite the availability of highly sophisticated methods of analysis, our ability to predict horizontal displacements is very poor indeed. With the present state-of-the-art, these displacements are probably best estimated empirically purely on the basis of measurements from other embankments.

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