DESIGN OF EMBANKMENTS ON SOFT CLAY

CONCEPTION DES LEVEES DE TERRE SUR ARGILE MOLLE

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SYNOPSIS : Several authors have recommended correction factors should be applied to vane shear strengths to allow for overestimation of the undrained strengths of soft clay by the vane shear. A systematic review of case histories of embankments on soft clay shows that much of this correction can be explained by overestimation of the strength of the weathered crust by the vane. Cases requiring larger corrections to the vane shear strength can generally be explained in terms of fissuring of the clay, staged construction and use of data from cut slope/river banks. It is shown that the overconsolidation ratio and liquidity index can be used as a guide as to whether the soils are fissured and the vane will overestimate strength. This uses higher factors of safety for high plasticity index soils than for lower plasticity soils. The "corrections" required are less than required for the Bjerrum method. A recommended procedure is given for design of embankments on soft clay foundations.

INTRODUCTION

The design of embankments constructed on a soft clay foundation (here defined as an undrained shear strength less than 50 kPa, usually less than 20 kPa) continues to involve a degree of uncertainty . This is demonstrated by the continuing occurrence of failures, and by the wide range of predicted failure heights for the Muar test embankment in Malaysia (Brand, 1991, Brand and Premchitt, 1989).

A number of authors have presented methods for design of such embankments, often concentrating on corrections required to field vane strengths (S_{uv}). to allow safe designs. These have included Bjerrum (1972,73); Azzouz, Baligh and Ladd (1983); Larsson (1980), Tavenas and Leroueil (1980); Trak, La Rochelle, Tavenas, Leroueil and Roy (1980); and Aas, Lacasse, Lunne and Hoeg (1986). The need for the correction factor μ ($S_u = \mu S_{uv}$) has been explained in terms of the rate of application of shearing, anisotropy and progressive failure (Bjerrum, 1972, 73).

Others have recommended the use of laboratory testing to determine the strength of the clay, eg. using the SHANSEP method (Ladd and Foott, 1974, Ladd et al, 1977, Ladd, 1991) or recompression method (Bjerrum, 1973, Jamiolkowski et al, 1985).

While the laboratory based methods have the advantage of being more correct theoretically and are apparently more reliable, they rely on relatively sophisticated and expensive testing techniques.

On many projects there remains a place for in-situ testing methods such as the field vane, because of both the simplicity and relatively small cost of the method.

When one attempts to apply the methods proposed by Bjerrum, Azzouz et al (1983) Aas et al (1986), etc (and for that matter the laboratory based methods), one is left with some uncertainty as to what should be done in respect of:

- allowance for the scatter in vane strengths on any site
- allowance for the strength of the embankment
- the strength of the "weathered" upper crust of the foundation soil.

When one reads these papers, and the papers on which they were based, it is apparent that

- most of the papers on which Bjerrum (1972,73) is based used "average" vane strengths, including the average vane strength in the weathered crust. Bjerrum assumed some embankments were cracked, others were not, and recommended that where cracking may occur, this be allowed for in design. As can be seen in Table 1 Bjerrum accepted some factors of safety from the authors' papers on which his work was based, but on others recalculated the factor of safety, usually arriving at a higher factor of safety, implying an assumption of higher strengths (probably in the weathered crust)) than the original authors. Bjerrum recommended that a factor of safety of 1.3 generally be adopted
- Azzouz et al (1983) had some cases in common with Bjerrum and with one exception, used the same factors of safety, again implying that they generally assumed the average vane strength in the weathered crust. However, in case 18, the original author had assumed some reduction in the crust strength to allow for cracking and/or fissuring, and Azzouz et al used that factor of safety. Azzouz et al recommended that the 3 dimensional shape of the failure surfaces be accounted for, resulting in smaller values of μ
- Aas et al (1986) do not make it clear what assumptions they made.

Knowing that these assumptions can make a significant difference to the calculated factor of safety, and that some authors, eg. Tavenas and Leroueil (1980) and Lefebvre et al (1987), questioned the use of vane shear strength in the weathered crust. The authors have set out to reanalyse the case histories in a systematic way with consistent assumptions, to see whether it was possible to remove some of the uncertainty in prediction of factor of safety. The assumptions are clearly set out, so that potential users are not left to make their own assumptions and possibly develop over or under conservative designs. This work follows on some earlier assessment of the problem by Burn, Cemanec and Jackson (1988) under the supervision of the second author.

METHOD OF STABILITY ANALYSIS

All case histories were analysed using the cross sections shown in the referenced papers. The embankment height was taken as the height above original ground surface, with no allowance for the settlement of the ground surface during construction.

Analysis was by the Bishop simplified method, except for cases 10, 23 and 27 where the critical failure surface was non circular and Morgenstern and Price analysis was adopted.

For each case, four combinations of embankment and foundation strengths were analysed:

Factor of Safety	Foundation Strength	Embankment Strength
F_1	Median	Full
F_2	Minimum	Full
F_3	Median	Cracked
F_4	Minimum	Cracked

Table 1. Factors of safety for case studies.

Number	Factors of Safety Author	Bjerrum	Azzouz		F_1	F_2	F_3	F_4	Data Quality	Site and Reference
1	1.60	1.65(3)	1.65	(5)	1.88	1.79	2.04	1.89	F-P(4)	Scottsdale; Parry and McLeod 1965
2	1.46	1.46(3)	1.46	(5)	1.27	1.22	1.05	1.01	G	Bangkok A) ; Eide and Holmberg
3	1.61	1.61(3)	1.61	(5)	1.40	1.36	1.45	1.40	G	Bangkok B) ;1972
4	1.30	1.52(1)(3)	1.52	(5)	1.25	1.25	1.14	1.14	F-P(4)	Scrapsgate; Golder and Palmer 1955
5	1.50	1.38(1)(3)		(5)	1.09	1.07	1.00	0.97	P-F	Lanester; Pilot 1972
6	1.38	1.38			1.53	1.50	1.50	1.48	P-F(4)	Saint Andre; Pilot 1972
7	1.60	1.53(1)	1.53	(5)	1.44	1.12	1.42	1.15	G	Matagami; Dascal et al 1972
8	1.17	1.17		Not used - bearing capacity failure						Pornic; Pilot 1972
9	0.97-0.87	1.05(1)(3)			1.11	0.83	1.23	0.96	G	New Liskeard; Lo and Stermac 1965
	1.16		1.16							New Liskeard; Lacasse et al 1977
10	1.00	1.02-1.38(3)		(5)	1.04	1.04	1.30	1.30	G	Kings Lynn; Wilkes 1972
11	1.30	1.30		Not used - data too imprecise						Palavas; Pilot 1972
12	0.96	0.99			0.92	0.86	0.90	0.84	P-F	Narbonne ; ilot 1972
13	0.88	0.86-0.92	0.91		0.82	0.73	0.85	0.76	G	Portsmouth; Ladd 1972
14	0.96	0.99	0.99	(5)	0.96	0.96	1.14	1.14	F-P	Fair Haven; Haupt and Olsen 1972
15	1.92		1.92		2.03/	1.97/	2.09/	2.02/	G	Maine; Ladd et al 1969
					1.66	1.63	1.65	1.61		
16	0.87		0.87	Not used - no vane test data						Tjernsmyr; Flaate and Preber 1977
17	0.89		0.89	Not used - no vane test data						Falkenstein; Flaate and Preber 1977
18	0.83		0.83		0.93	0.84	0.97	0.93	F-P	Presterod; Flaate and Preber 1977
19	0.80		0.80	Not used -limited vane test data						As; Flaate and Preber 1977
20	0.88		0.88	Not used - no vane test data						Nessett; Flaate and Preber 1977
21	1.10		1.10		1.47	1.30	1.47	1.35	F-P	Jarlsberg; Flaate and Preber 1977
22	0.92				1.39	1.28	1.40	1.29	F-P	Aulieva; Flaate and Preber 1977
23	0.73			(5)	1.11	1.06	1.11	1.06	P-F	Skjeggerod; Flaate and Preber 1977
24			0.89	Data not available						Ladd et al 1975
25	1.20(2)		1.20		1.15	1.03	1.15	0.96	G	Saint Alban; La Rochelle et al 1974
26	1.17		1.19	(5)	1.15/	0.90/	1.19/	0.98	G	James Bay; Dascal and Tournier 1975
					0.91		0.92	0.92		
27	1.40				1.05	1.03	1.62	1.59	G-F	River Thames; Marsland 1977
28	Not stated			(5)	1.02	0.94	0.91	0.85	G	Rio de Janeiro; Ramalho-Ortigato et al 1983
29	Not stated				2.03/	1.66	2.53/	2.41/	G	Edmonton; Crooks et al 1985
					1.58	1.54	1.87	1.80		
30	Not stated			(5)	1.00	0.85	0.97	0.80	G	Muar; Brand 1992

Notes:
(1) Recalculated by Bjerrum.
(2) For full crust and fill strength.
(3) Embankment assumed cracked by Bjerrum.
(4) Affected by staged construction.
(5) Embankment assumed cracked by present authors.

The results of the analysis are given in Table 1.

Figure 1(a) shows the median and minimum strengths. It should be noted that below the weathered crust, median and minimum strengths are the same. For cases 15, 26 and 29 lower quartile values have also been used for calculation. This is discussed further below.

For "cracked" embankment strength, the crack was assumed to penetrate the full depth of the embankment regardless of whether this is reasonable. The present authors assessment of whether cracking was likely is also shown.

The analysis correctly modelled the crack. It should be noted that the simplified approach of adoption of $c'=0, \phi'=0$ for the fill significantly underestimates the factor of safety in most cases because the critical circles pass through the fill at a steep angle but not vertical.

SELECTION OF FOUNDATION STRENGTH

All analyses were based only on the vane shear strength test results presented in the papers. Laboratory test results were not used although in practice one should preferably use a combination of vane shear testing, laboratory testing and correlation with known relationships, eg:

$$(S_u/\sigma'_{vo})_{NC} = 0.23 \pm 0.04$$

and $$(S_u/\sigma'_{vo})_{OC} = (S_u/\sigma'_{vo})_{NC} OCR^{0.8}$$

(Jamiolkowski et al, 1985) or similar relationships (eg. Ladd, 1991) to determine the undrained shear strength.

To allow for the natural scatter in vane test data, the median strength was used in the clay below the weathered crust, ie. after discarding abnormally high value, (probably affected by shells or roots in the clay), or abnormally low values (probably due to disturbance on insertion of the vane). The strength profile with half the values above, and half below was adopted. This is considered preferable to the average values as it places less weight on values at the upper and lower extremes of the test data. Figure 1(a) shows an example. For cases 15, 26 and 29 a second strength profile was analysed — the "lower quartile" with one quarter of the results less than the adopted strength, three-quarters above. These cases had an extraordinarily wide scatter of test results similar to that shown in Figure 1(b). Possible explanations of these cases are:

Case 15 — there were many broken shells, organic matter, wood chips and sand lenses in the clay.
Case 26 — the clay was observed to have numerous vertical fissures on exposed slopes and there were large quantities of shells and traces of recent plants.
Case 29 — fissuring, consisting of numerous srandomly oriented lickensided surfaces, were observed throughout the clay (note also that the liquidity index of the unweathered clay is much lower than other case histories, i.e. 0.3 to 0.6 compared to >1.0 for other cases).

The authors believe such conditions could be recognised on a site, and recommend the use of the lower quartile of the vane strengths, coupled with intensive investigation of the strength by laboratory methods. These may show however, that, as for cases 15 and 29, even the lower quartile vane strength overestimates the undrained strength.

It is now well recognised (Lefebvre et al, 1987, Tavenas et al, 1980), that the full vane shear strength of the weathered crust is not available to resist failure because of the presence of the preexisting fissures and subsequent cracking under the settlement of the embankment. To account for this, analyses have been carried out for two versions of the weathered crust strength:

- the so called "median" case
- the minimum strength case.

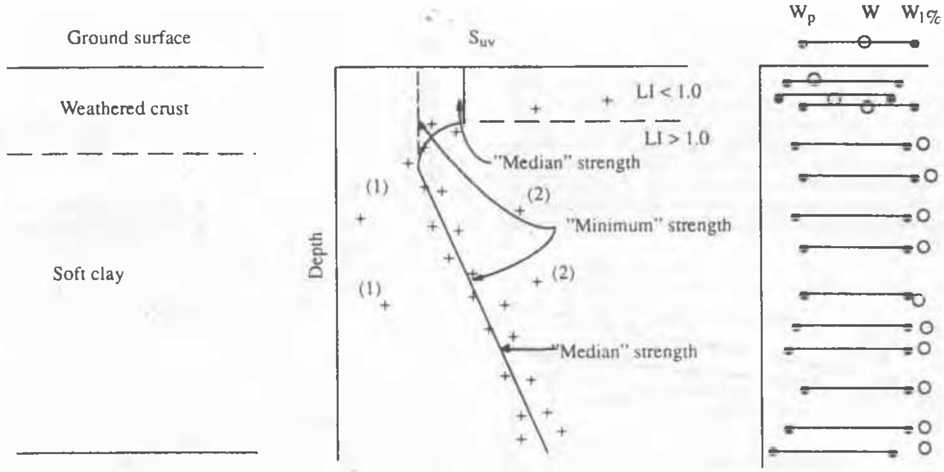

Figure 1(a). Selection of foundation strength – Normal case.
Notes: (1) Discard, probably disturbed.
(2) Discard, probably affected by shells, roots etc.

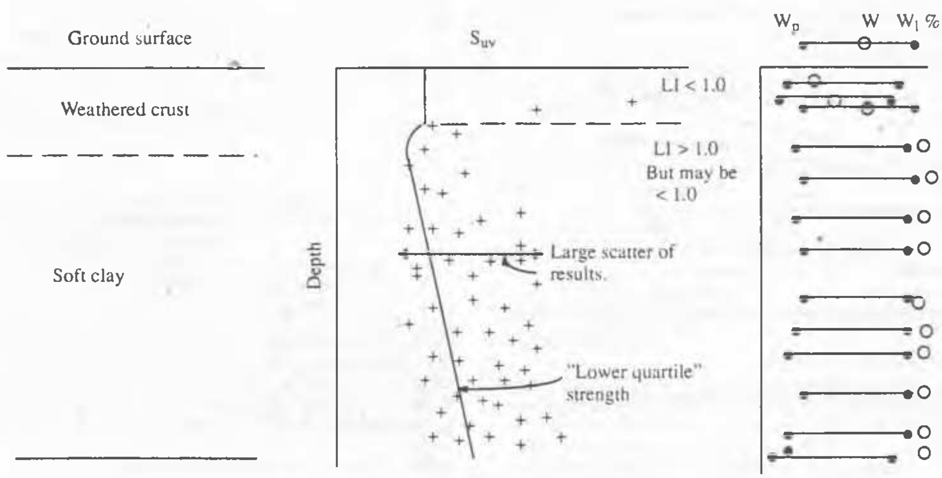

Figure 1(b). Selection of foundation strength – influenced by shells, roots etc. and large scatter of data.

For the median strength analysis, the median of the measured vane shear strengths have been used where the liquidity index is greater than 1.0 (ie. the water content is greater than the liquid limit). Above that level, where the liquidity index is less than 1 (typically less than 0.5), a strength equal to the median vane shear strength at the base of the layer is used (as shown in Figure 1(a). This this is broadly consistent with the findings of Lefebvre et al (1987) who carried out plate bearing and direct shear testing, and found that at in-situ stresses the available undrained shear strength of the crust was about equal to the vane shear strength in the intact clay immediately below the crust. They also suggested that under the embankment the confining load of the embankment should result in an increase in the undrained shear strength, and suggested using a strength equal to one quarter of the weight of the embankment, at the surface, decreasing linearly to the vane shear strength at the base of the weathered layer. The present authors are reluctant to allow for such a strength increase, in view of the unpredictable effects of settlement and lateral spreading on the relatively stiff crust.

The second assumption, that the weathered crust strength equals the minimum strength (see Figure 1(a)) is considered too conservative by Tavenas et al (1980), but has been used to gauge the effect of what might be regarded as a lower bound estimate of the strength.

SELECTION OF EMBANKMENT STRENGTH

For most cases the strength recorded in the paper describing the case history has been used. The exceptions were case 12, where $c'=0, \phi'=35°$ was used instead of $c'=53$ kPa, $\phi'=26°$ for the run of quarry gravelly fill because it was considered that the high effective cohesion was unrealistic, and case 29, where no strength was given for the compacted clay shale, and $c'=0, \phi'=35°$ was adopted.

RESULTS OF ANALYSIS

Table 1 summarizes the results of the analysis. It will be apparent that the best estimate factor of safety F_b is usually less than adopted by Bjerrum (cases 1 to 14), reflecting the lower strengths adopted for the weathered crust. This implies generally smaller correction factors (μ) would be needed than suggested by Bjerrum (1972,73). Table 1 includes an assessment of data quality. This is a qualitative assessment on the likely accuracy of the analysis, taking into account the accuracy of the cross section data (some diagrams are small and difficult to scale accurately), the amount of information on shear strength, and the influence of such factors as staged construction. In the table P=poor, F=fair, G=good. Some cases included by Bjerrum (1972,73) and Azzouz et al (1983) are considered unusable for the reasons shown in the table. It will be apparent that in most cases the assumption on whether the embankment is cracked or not does not have much influence on the factor of safety, provided the crack is modelled correctly. It will also be apparent that assuming the embankment is cracked does not always result in lower factors of safety.

It will be seen that despite the attempts to analyse the data more consistently than previous authors, a wide range of factors of safety have been calculated. To ascertain whether these can be related to other measured properties F_1, (or F_3) and F_2 (or F_4) have been plotted against plasticity index, liquidity index, sensitivity S_u/σ_{vo} and overconsolidation ratio (OCR) of the soft clay (not the weathered crust). The results are given in Figures 2 to 6 and on each graph, the good quality data is differentiated from the lesser quality. For cases 15, 26 and 29 the lower quartile information has been used. It should be noted that data is not available for all cases, eg. there is no OCR data for case 23, and no sensitivity data for case 29.

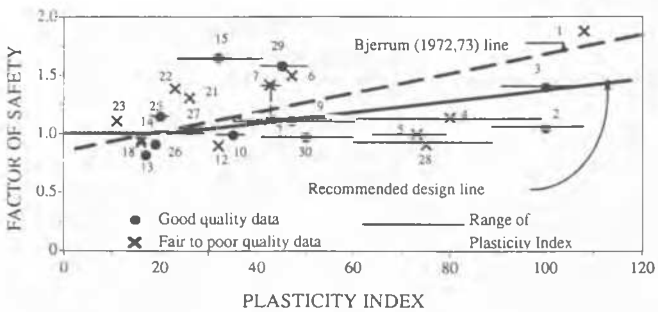

Figure 2(a). Factor of safety (F1 or F3) vs Plasticity Index of soft clay.

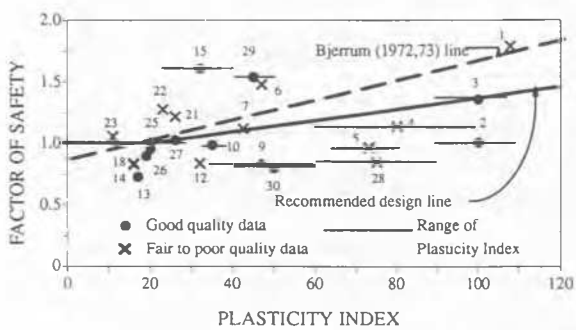

Figure 2(b). Factor of safety (F2 or F4) vs Plasticity Index of soft clay

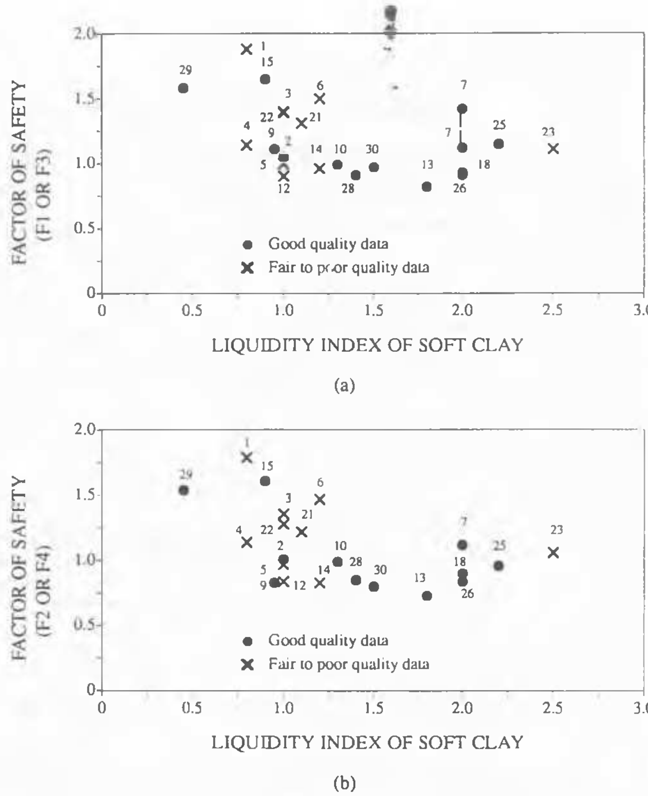

Figure 3. Factor of Safety versus Liquidity Index of Soft Clay.
(a) Median strengths
(b) Minimum strengths

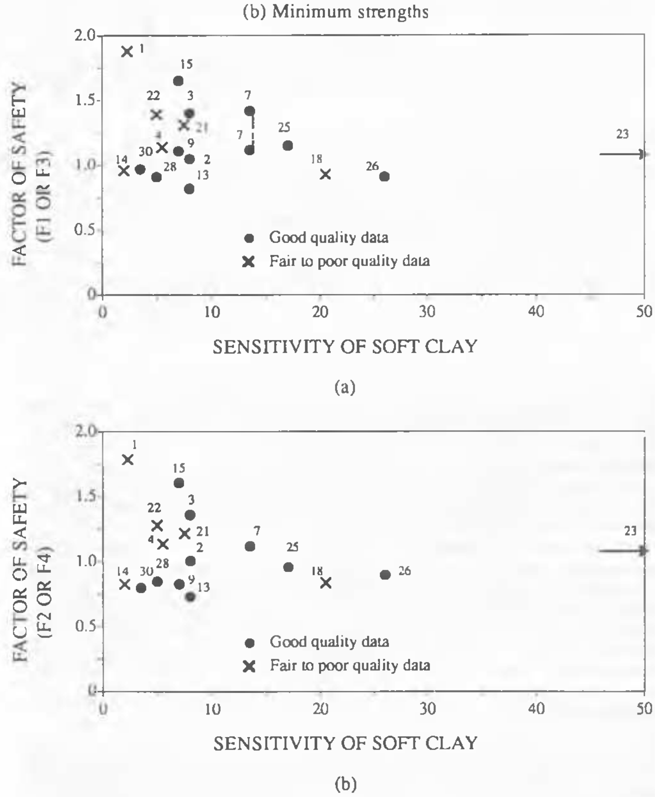

Figure 4. Factor of Safety versus Sensitivity of Soft Clay.
(a) Median strengths
(b) Minimum strengths

The following comments are made:

(a) there is apparently a wide scatter in the plots of factor of safety versus plasticity index, particularly at lower plasticity index. However, one can reasonably explain the high factors of safety in most of these cases, and hence reasonably ignore them in any design method provided that such conditions are not present
- cases 15 and 29 were affected by shells, wood chips, sand layers, and numerous fissures in the soft clay respectively. The authors' proposed adoption of lower quartile strengths has not been sufficient to correct for the overestimation of strength by the vane shear in such circumstances
- syneresis fissuring was observed in cases 2 and 3. Whether this could be detected in a project is not clear
- cases 1, 4 and 6 were affected by staged construction and can reasonably be excluded
- cases 22 and 23 involved stability of canal and cut slopes. They were the only such cases, and again may reasonably be excluded. However, they serve as a warning that cut slopes may behave worse than embankments, possibly because of progressive failure effects.

This leaves cases 21 and 7. Case 21 was reasonably poor quality data. It is notable that the authors' calculated factor of safety for this case was 1.10. The present authors are of the view that it can be ignored for decision making purposes. Case 7 remains somewhat of an outlier in the median strength case. Work by Lefebvre et al (1987) showed that on this site the crust strength was less than that assumed for the median strength analysis, and the minimum strength case is more appropriate. However, in any systematic analysis case 7 needs to be considered.

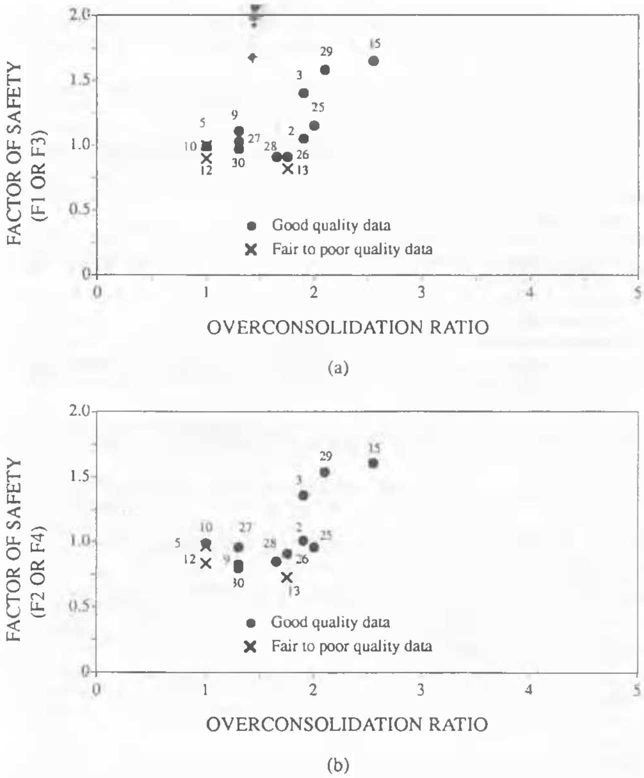

Figure 5. Factor of Safety versus Overconsolidation Ratio of Soft Clay.
(a) Median strengths
(b) Minimum strengths

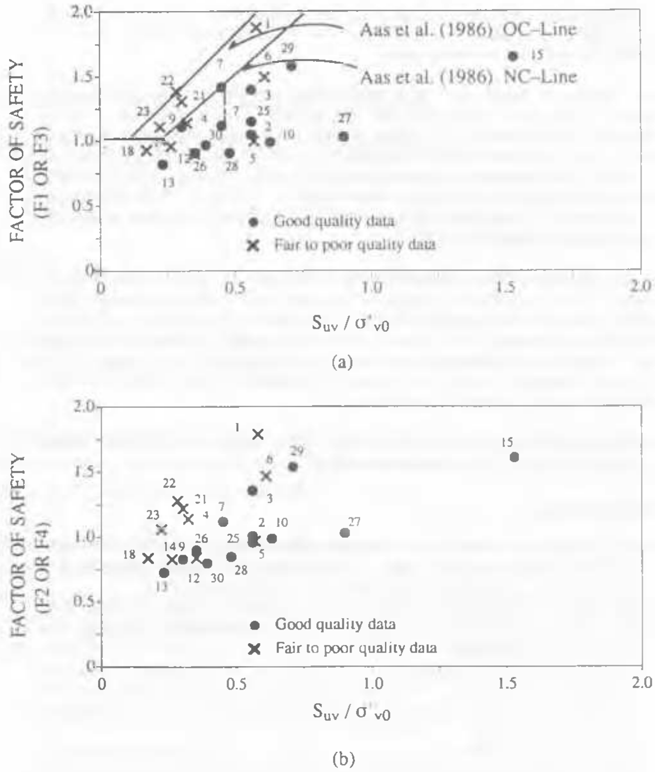

Figure 6. Factor of Safety versus S_{uv} / σ'_{v0} of Soft Clay.
(a) Median strengths
(b) Minimum strengths

(b) when these cases are excluded, the factors of safety are between 0.8 and 1.15 for the median strength case, with little apparent dependence on plasticity index. If cases 2 and 3 are left in the set, there is a case to be made for an increase in factor of safety with plasticity index. Without additional information the authors are reluctant to ignore these cases.

(c) Figures 3 and 5 together show some interesting features, when considered in conjunction with the observation of fissuring. Fissuring was noted in the weathered crust in cases 2, 3, 9?, 21? and 22?, 26, 29 and in the soft clay in cases 2, 3, 15, 26 and 29 (the ? indicates the description is limited). It is notable that these cases consistently group in the area where liquidity index is less than about 1 or 1.2 and the overconsolidation ratio is greater than about 1.9. This is not an unexpected relationship since fissuring is more likely to occur in lower water content overconsolidated soils. Hence, it appears that one could use this as a signal to be on the lookout for fissuring which can reduce the undrained strength of the clay below that measured by the vane shear. It might be argued that there is a correlation between OCR and factor of safety. However, the authors prefer the interpretation that factor of safety is independent of OCR for OCR $\leq$ 1.8.

d) as shown in Figure 4, there seems little correlation between factor of safety and sensitivity. This is contrary to what one might have expected, given that one could expect strain weakening ("progressive failure") to be more likely in highly sensitive clays. If anything there is an opposite trend, possibly due to disturbance of the more sensitive clay on insertion of the vane

(e) as shown in Figure 6, there is a poor correlation between S_u/σ'_{v0} and factor of safety, particularly when cases 15, 26 and 29 are excluded. This is in contrast to the findings of Aas et al (1986). Shown are the Aas et al (1986) boundaries for normally consolidated (NC) and overconsolidated (OC) soil. It can be seen that the Aas et al boundaries are apparently conservative, although this reflects the different assumptions on weathered crust strength. However, it is apparent that if the Aas et al (1986) approach was adopted, it would be very conservative in most cases.

CONCLUSIONS AND RECOMMENDED DESIGN PROCEDURE

It is concluded that a significant proportion of the correction factor μ proposed by Bjerrum (1972,73), Azzouz et al (1983) and Aas et al (1986) is correcting for an overestimation of strength of the weathered crust by vane shear testing. This overestimation occurs because of the presence of the fissures in the weathered crust, and cracking under settlement. It is considered that the proposed median strength method gives a reasonable approximation of the strength, although the minimum strength approach may be applicable in some cases. It is not possible to determine which is appropriate in advance, unless for example plate load and direct shear testing as done by Lefebvre et al (1987) is carried out. Other cases where high factors of safety are calculated can be explained by the presence of fissuring, an abundance of shells or other obstructions which influence the vane strength, by staged construction effects not properly accounted for in the analysis or by the case being a cut slope.

It is recommended that the design of embankments be carried out by:

(a) plotting the data as shown in Figure 1 and obtaining the median strength

(b) inspecting undisturbed tube samples of the soft clay for the presence of fissuring, extensive roots or shells which can affect the strength. These will be more apparent if the samples are allowed to dry after extrusion from the tube. The presence of clays with

Liquidity index	$\leq$	1.2
Overconsolidation	$\geq$	1.8

is a guide to the likely pressure of fissuring. In the event that such features are present, other means of assessing the undrained shear strength must be adopted. For preliminary design only, a large factor of safety (probably >2 for lower quartile strengths) could be adopted

(c) a factor of safety at least 0.3 higher than the "recommended design line" in Figure 2(a) should be adopted for a design based on the median strengths. Analysis should be carried out for full embankment strength. If the embankment is likely to crack, the analysis should be checked for the cracked case, and the lower value adopted

(d) the analysis should be repeated for the minimum strengths, and a factor of safety at least 0.2 higher than the "recommended design line " in Figure 2(b) should be adopted.

These factors of safety are lower limits, and it would be desirable to adopt values of 0.5 and 0.4 higher than the design lines respectively where practicable, and certainly for larger projects, the undrained shear strength should also be determined by other methods, eg. recompression or SHANSEP type laboratory testing, and/or indirectly through effective stress parameters and/or by relations between S_u and σ'_{vo}, OCR. This redundancy in assessment of strength is an important way of avoiding over and underestimation of the strength.

Site investigations should be sufficiently intensive to locate areas of lower strength. It is the second authors' experience that failure to locate lower strength areas within a project has led to failures with significant cost effects. It should be recognised that because much of the available strength of soft clay is lost when an embankment fails (how much depends on the sensitivity of the clay) remedial designs are usually expensive, and usually involve significant delays to project completion.

There is some evidence that for the design of cut slopes or river banks in soft clay, larger factors of safety may be needed.

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