Reprinted from Proceedings of In Situ '86
GT Div., ASCE, June 23-25, 1986
Blacksburg, VA

BOREHOLE SHEAR TEST AND SLOPE STABILITY

Richard L. Handy, M., ASCE[1]

ABSTRACT

Three Iowa landslides analyzed on the basis of Borehole Shear Test (BST) data are reported. The soils included CL and ML loess and glacial till, and CH clay shale. Back-calculations from BST friction angles measured in the shear zones consistently indicated that cohesion after sliding is near zero, suggesting that after-slip equilibrium must derive mainly from internal friction in the shear zone. A unique advantage of the BST is that it can measure ϕ that is in effect at the time of sliding.

Stage BST's were performed on a 28-ft (9 m) high lagoon embankment of compacted CH soil over random fill, to determine if the lagoon could be safely put into service. Fourteen BST shear envelopes were obtained in two days and gave sufficient data for probabilistic determinations of the failure risk based both on the range in individual strengths and the range in predicted means. The stage BST separately measures c and ϕ means and variabilities, not possible when tests are performed on individual samples that may or may not share a common shear envelope.

INTRODUCTION

There are two applications where the Borehole Shear Test (BST) may be preeminent: (a) for testing and analysis of active landslides; and (b) for the rapid acquisition of sufficient data for a probablistic risk analysis, based on separate measurement of the c and ϕ variabilities. Both are presented through the example of case histories.

Stage vs. Single Point BST -- The BST is unique among in-situ test methods in giving direct evaluations of soil cohesion (c) and friction angle (ϕ) at any particular depth. The key element of the instrument is the shear head, Figure 1, which opens laterally to apply a controlled constant normal stress acting on the soil through two opposed serrated shear plates.

After a 5-15 minute time for consolidation, the shear head is pulled, causing soil engaged by the shear plates to move with the plates, thus causing adjacent soil to shear. The pulling force is

[1]Professor of Civil Engineering, Spangler Geotechnical Laboratory, Iowa State University, Ames, Iowa 50011, USA

measured and a maximum ascertained, which gives a single point on a Mohr-Coulomb failure envelope.

Figure 1. BST shear head with pore pressure sensor (arrow).

In stage testing, these steps are repeated to obtain additional data points at successively higher normal stresses (Schmertmann, 1975). Alternatively, the device may be removed, cleaned off, and reinserted at a different depth, but this increases point scatter and results in poorer drainage because consolidation times are not cumulative (Demartincourt and Bauer, 1983). The stage test procedure is particularly advantageous for testing shear zones because it gives a complete failure envelope at essentially the same depth position in a single boring.

The BST typically is a consolidated-undrained test in normally consolidated fine-grained soils, and a drained test in overconsolidated or granular soils. The validity of the measured c and ϕ parameters as estimators for c' and ϕ' may be checked by monitoring the pore water pressure during the test. This is done very close to the zone of maximum compression and shear, advantageous for giving a uniquely rapid response time during the test.

SLOPE ANALYSIS

Despite major emphasis on refining and computerizing methods of slope analysis, Huang states (1987, p. 38): "It should be emphasized the greatest uncertainty in stability analysis arises from the determination of strength. The error arising from the selection of stability computations is usually small compared with that arising from the selection of strength parameters . . . "

Inadequacies of the triaxial test to accurately characterize soil strengths mobilized in active landslides are shown by comparisons of test results with the strength parameters back-calculated from actual slides, the triaxial test tending to either overestimate or underestimate ϕ, and overestimate c (Sauer and Christianson, 1985; Skempton, 1977). Skempton (1985) attributes this error to an unrealistic range of normal stresses attained in a conventional triaxial test, to which can be added the lack of realism in having $\sigma_2 = \sigma_3$, a landslide representing essentially a plane strain geometry with the intermediate principal stress a function of K_o normal to the axis of the slide.

A trend therefore has been to rely more on direct-shear or simple-shear tests wherein σ_2 may attain that of a normally consolidated soil.

These procedures still do not represent field σ_2 values in overconsolidated soils. By cyclical direct shear or ring-shear, it is possible to measure residual strengths resulting from the development of slickensides (Skempton, 1977).

A troublesome aspect of laboratory shear strength determinations is obtaining enough samples from within an active shear zone for tests at several applied stress levels. In competent soils this can be met by trimming out three or four 35 mm (1.4 in.) diameter specimens from the same depth in a single 100 mm (4 in.) tube sample; however, landslide shear zone soils seldom are that competent. If only single samples are obtained, stage tests may be used (Kenney and Watson, 1961).

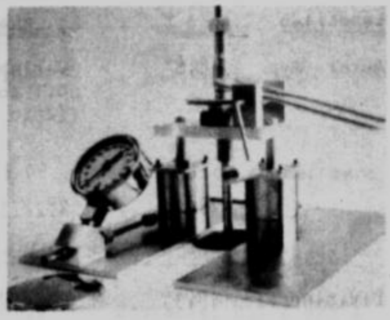


Figure 2. BST pulling device to measure τ.

A criticism of all test methods including the BST is that results should apply only to portions of the landslide with shear plane orientations corresponding to those in the test. This objection becomes less significant for softened, thoroughly remolded shear zone soil found at the base of most soil landslides.

PART I: BST IN ACTIVE LANDSLIDES

Aurora Avenue, Des Moines. In 1959 a landslide developed in loosely compacted mixed MH and CH loess and glacial till fill, damaging and endangering four houses, Figure 3. Sliding was aggravated by restoring soil lost at the top, after which several abortive attempts were made to stabilize the landslide with underground drains, braced walls, and concrete piles. In 1963 two of the lots were successfully stabilized by the first reported use of drilled lime for this purpose (Handy and Williams, 1967), and in 1966 BST's were conducted in stable and in adjacent, untreated areas.

Figure 3. The Aurora Avenue landslide, Des Moines, 1964, and the site of some early Borehole Shear Tests.

Table 1. Analyses of Three Active Landslides, part 1

Table 1, part 2

Landslide	Borehole Shear		Terrain	Assumed Saturation	Assumed c for Analysis, kPa (psf)	Factors of Safety		Remarks
	ϕ	c, kPa (psf)				OMS[1]	Janbu	
Aurora Ave.	18.8°	Variable, 0.1 to 12 (2 to 260)	(a) Before slide	YES	5.9 (124)	1.00	0.99	c back-calculated
			(b) After slide	YES	0.95 (20)	1.00	1.00	Post-slide c near zero
	23.6°	28.2 (194)	(c) Lime stabilized		BST[2]	2.2	----	Stable
Division #1	14.3°	3.8 (75)	(a) Before slide	YES	BST	0.94	0.99	Unstable; BST c and ϕ
			(b) After slide	YES	0	0.89	1.00	Post-slide c = zero
Division #2	14.3°	3.8 (79)	(a) Initial	YES	BST	2.73	----	Stable; BST c and ϕ
			(b) Open trench	YES	BST	1.19	1.19	Stable; BST c and ϕ
			(c) Filled upon	YES	0	0.85	0.89	Unstable; not fully saturated
	14.3°	3.8 (79)	(d) After slide	YES	0	0.70	0.74	Not fully saturated
Foster Ave.	Till:							
	28.2°	13.5 (281)	(a) Before slide	YES	BST	0.82	0.87	Unstable; BST c and ϕ; not fully saturated
	Shale:							
	16.1°	3.4 (70)	(b) After slide	YES	0	0.98	1.01	Post-slide c = 0

[1] OMS = Ordinary Method of Slices

[2] ϕ and c used in the analysis = ϕ and c from the BST

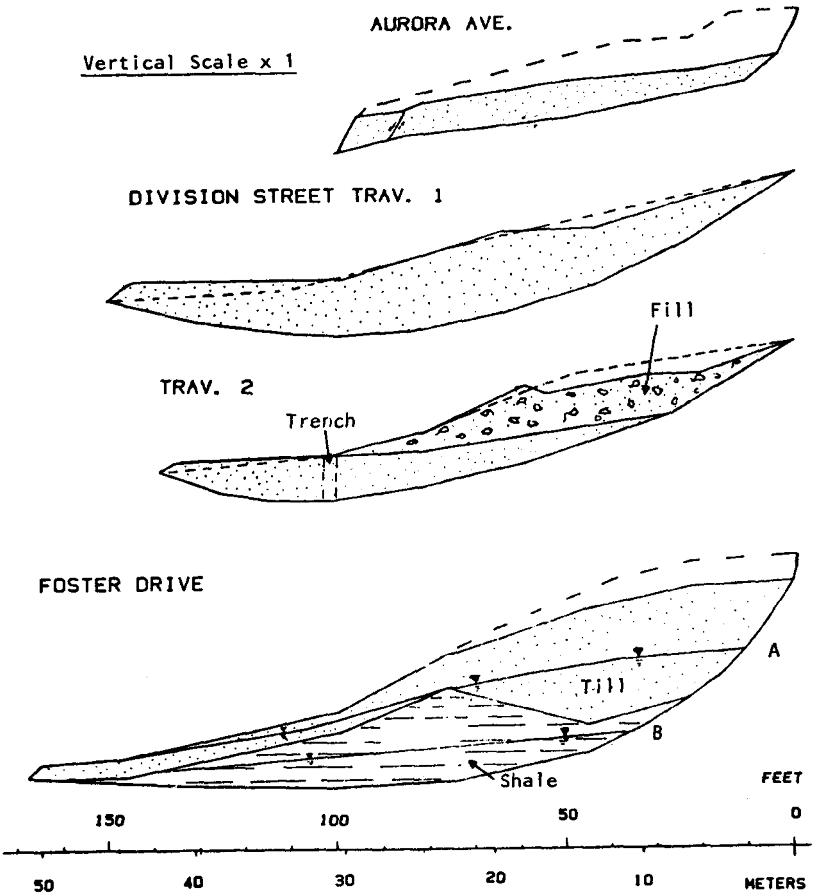


Figure 4. Cross-sections of three landslides.

Three BST's performed in untreated areas gave friction angles in a narrow range from 17.6° to 19.6° while cohesions were exceedingly variable, 0.1 to 12 kPa (2 to 260 psf). Vane shear strengths also were highly variable, ranging from 0 to 11 kPa (0 to 220 psf) in the shear zones (Biggs and Sendlein, 1964).

A cross-section of the Aurora Avenue landslide is shown in the upper diagram of Figure 4. The slip was essentially a block slide with a rotational component only near the active scarp, indicating that relatively little error would result from application of the ordinary method of slices (Skempton, 1977). Analyses also were performed by the simplified Janbu method (Janbu, 1973) with results shown in Table 1.

The average BST friction angle (18.8°) was used in back-calculation to obtain c = 5.9 kPa (124 psf) prior to sliding; this reduced to 0.95 kPa (20 psf) after sliding. These results agree with Skempton's (1971) findings that the remolded or fully softened cohesion component of shear strength approaches zero.

After the landslide was stabilized by quicklime, BST's showed substantial increases in c, ϕ, and the factor of safety (Handy and Williams, 1967); after 20 years the slide remains stable.

Division Street, Burlington. The Division Street landslide presented an array of contributing factors that included filling at the top, cutting away at the toe, and water overflow from the street onto the top of the slide area. During 1970 the slide carried away part of the street and mobilized some mobile homes parked below.

Prior to the 1970 slide, a utility trench had been cut across part of the natural hillside, but the slide did not activate.

BST's were conducted at several depths in two borings and gave ϕ = 22.4° to 40° above the slip zone and 14.3° within the zone, Table 1. Two elevation and boring traverses were made, one outside and one inside the filled area, and yielded the calculated factors of safety (FS) listed in Table 1.

The investigation was conducted after a period of dry weather such that the positions of the ground water table were not known; hence, the worst condition of full saturation was assumed. Along Traverse 1 this assumption appears satisfactory, as the calculated FS prior to sliding is 1.0, so it was acknowledged that water overflow from the street probably did contribute to the landslide in this area.

Along Traverse 2, prior to filling the calculated FS was 2.7, assuming full saturation of the natural soil; with the trench opened, FS was 1.2 and the slope was stable. With the trench closed, filling reduced the fully saturated FS to 0.9, indicating that the fill was a substantial causal factor and may not have attained full saturation prior to sliding.

After sliding had occurred, back-calculations using the BST friction angle of 14.3° for Traverse 1 indicated c = 0, in agreement with the observation made for the Aurora Avenue slide. Along Traverse 2, the FS with full saturation and c = 0 is only 0.7, a further indication that the fill probably was not fully saturated at the time of sliding. A recalculation assuming a 50 percent level of saturation gave a FS = 1.1.

By use of the BST data and analyses wherein each of the several contributing factors was omitted, a relative contribution from each could be assessed and was used as the basis for an out-of-court settlement.

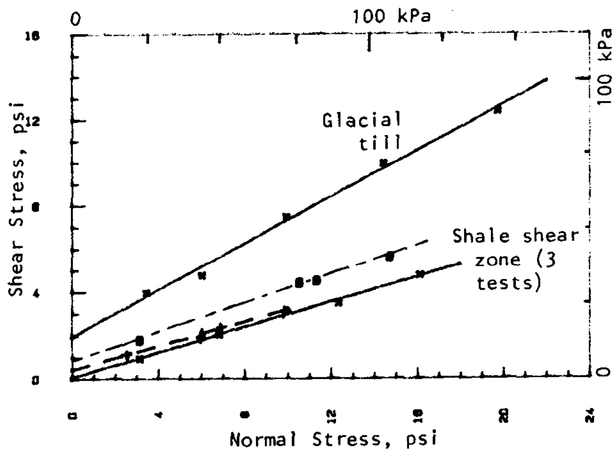
Foster Drive, Des Moines. A large home built in a landslide-prone area had been stable for 40 years when additions were made in 1979 without a geotechnical assessment of the area, and a landslide developed that carried away the additions and threatened the house.

Inclinometers were installed, and the slip zone was found to extend down through glacial till and across through shale, Figure 4. Piezometers indicated that the soil was not fully saturated, and some movements continued as the water table lowered almost to level B, indicated in Figure 4.

The uppermost phreatic level A was estimated from the subsurface stratigraphy by assuming a perched condition on the shale. The composite slip surface is about 20 percent in pre-Illinoian CL and ML glacial till and 80 percent in Pennsylvanian CH shale, with about 50 percent parallel to bedding planes in the shale.

Four BST's were conducted at three locations in the shale slip zone and one in the glacial till. Results, Figure 5, revealed a relatively narrow range of ϕ values for the shale, so the tangent average $\phi = 16.1°$ was used in the analysis. As in the other studies, c was more variable, and an average value was used for the analysis prior to sliding.

Figure 5. BST data from the Foster Avenue landslide.



From the BST data of Figure 5 and the assumed highest position of the ground water table at A one obtains a FS = 0.9, Table 1, suggesting either an underestimation of the average ϕ or an overestimation of the maximum ground water level.

When sliding stopped and the water table was in position B in Figure 4, the BST ϕ values with c = 0 give a FS = 1.0. Thus, while excess ground water appears to have been an important factor contributing to the initiation of this particular slide, drainage would not have been an effective means of stopping it because of the loss of soil cohesion from sliding.

Summary: Role of the BST in analysis and repair of active landslides. From these and other case histories, we conclude that the BST is particularly useful for quickly and accurately calculating soil friction angle(s) acting within the slip zone of an active landslide, and for assigning responsibilities and designing appropriate repairs. After the slide activates, c appears to become essentially zero, in effect a viscosity that with slow movements is very low.

Some practicing engineers have suggested that detailed analysis of a landslide is superfluous because all that is necessary to stop it is to drain it: the last case history indicates that this may not always be the case. Certainly a knowledge of the soil friction angle(s) readily attainable with the BST is indispensible for an intelligent consideration of repair options. For example, a 5° error in friction angle, say from 10° to 15°, could change tan ϕ and hence the apparent effectiveness of a toe surcharge by 50%.

The alternative of back-calculation assumes that only one value of friction angle is acting, and requires that the ground water condition be known, which may not be possible if the borings are made during a dry season when the slide is temporarily inactive. This suggests that the best way to obtain the relevant soil friction angles is to measure them directly, in situ, with the BST that does not depend on empirical correlations.

PART II: RISK ANALYSIS OF AN UNFAILED SLOPE

As previously stated, a second advantage of BST is for rapid and inexpensive acquisition of sufficient c and ϕ data to afford probabilistic evaluations of factors of safety. Ideally there is a 50 percent probability that a conventionally calculated FS based on means will exceed the actual or "true" factor of safety, here designated FS*, as it exists in the field. We may then ask, what is the probability that the true FS* $\leq$ 1.0, i.e., that the structure will fail? This inquiry takes into account the data variability as well as data averages. It is a simple matter to illustrate how a structure with a design FS = 1.5 may exhibit a much higher probability of failure than a similar structure with a design FS of, say, 1.2, where the data are less variable.

An excellent opportunity to test this BST capability came after construction and before filling of a 100 x 150 x 10 m deep (350 x 500 x

30 ft) sewage lagoon that was said to be the largest of its kind in the world. The lagoon embankment foundation soil included areas of random clay and rubble fill, and the embankment itself was CH clay compacted to 95 percent standard density, with an allowable maximum moisture content that did not preclude the possibility for overcompaction. A recommendation therefore was made that the soils be tested and the stability of the embankment slopes calculated prior to putting the lagoon into service. Furthermore the testing program obviously had to provide a measure of the reliability of the FS, even if its value were high enough to be presumably adequate. Another advantage of this approach was that the owner was automatically apprised that regardless of the calculated FS, there would be some probability, however small, that the lagoon embankment still might fail.

Two options were considered: thin-walled tube sampling with triaxial tests that would produce a minimum of three failure envelopes in 2-3 weeks time, or two days BST's for as many failure envelopes as could be obtained on-the-spot. The costs were approximately equal, and the BST was selected. Tube samples obtained for back-up failed upon extrusion, causing full reliance on the BST data.

In two days that included mobilization, drilling, and testing, 13 BST's were completed with 62 test points, of which 44 were suitable to include in failure envelopes. Invalid points are attributed to initial seating of the shear plates at the lowest applied normal stress, and to excess pore pressures at the highest. The latter condition can be checked by pore pressure measurements, but this was not done because of limitations of time and availability of the appropriate BST equipment.

BST Results. The three highest values of BST friction angles were in a discreet range of 28° to 37° in the same test boring, where the compacted soil already had been removed and replaced. This dike section was deemed to be safe and these data sets were excluded from further analysis.

Of the 10 BST failure envelopes obtained in the remaining dike sections, four were in the compacted embankment and six were in the underlying random fill soil. Friction angles are relatively uniform in the compacted fill (Figure 6) and variable in the random fill (Figure 7), whereas the reverse is true for cohesion. Results are summarized in Table 2.

Taylor Number. A preliminary estimate of the embankment stability was made by use of Huang's charts (Huang, 1983, p. 10), based on cylindrical failure, Bishop's simplified method, and the Taylor stability number, $c/\gamma H$. Mean values for c and ϕ (Table 2), $\gamma = 1.79$ g/cm^3 (112 pcf), height H = 6.7 m (22 ft), 1:3 side slopes, and pore pressure u = 0 were used, the latter because of the permeable nature of the supporting fill and the plan to use an impermeable plastic liner. The resulting FS = 1.46, a value that under most circumstances would be considered safe.

Method of Slices. The above approximation does not take into account the soil layers that in this case emphasize the role of the

Table 2. Borehole Shear Data from Lagoon Embankment Study.

Boring and Test	No. Points	Corr. Coeff.	Upper, compacted fill		Lower, random fill	
			ϕ, degrees	c, kPa (psf)	ϕ, degrees	c, kPa (psf)
3-1	2	---	11.6	2.41 (50)		
3-2	4	0.981			23.8	8.62 (180)
3-3	4	0.807			6.2	12.1 (250)
4-1	4	0.990	10.5	7.17 (150)		
4-2	3	0.991			7.2	9.86 (206)
4-3	3	0.991			7.1	3.79 (79)
5-1	3	0.941	12.2	19.3 (403)		
5-2	3	0.941	9.7	11.7 (243)		
5-3	2	---			15.6	9.65 (202)
6-1	5	0.991			11.9	8.76 (183)
Mean			11.0	10.1 (211)	11.9	8.8 (184)
Standard Deviation			±1.1	±7.2 (±150)	±6.8	±2.7 (±56)
Means and Standard Deviations of all above tests	$\overline{\phi} = 11.56 \pm 5.17°$; $\overline{c} = 9.33 \pm 4.67$ kPa (195 ± 97 psf)					

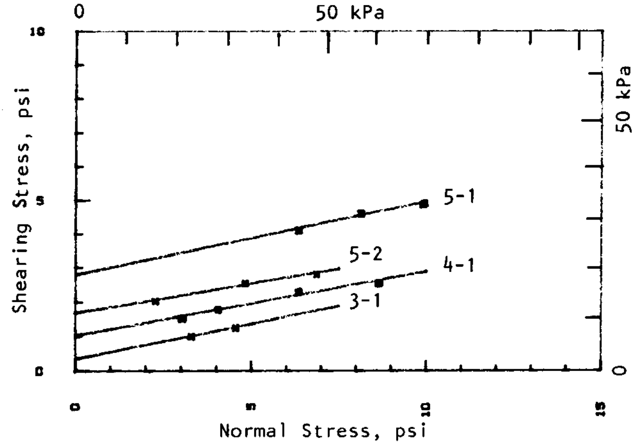


Figure 6. BST's for compacted CH fill.

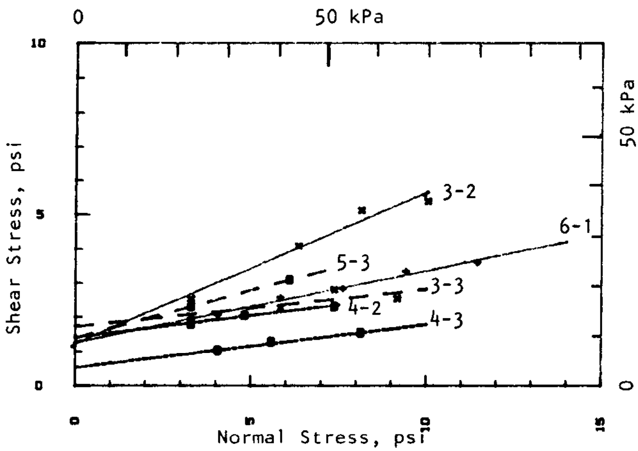


Figure 7. BST's for random fill.

random fill, as shown in Figure 8. A simplified Bishop analysis from this figure gives FS = 1.28, still within marginally acceptable limits for safety.

In order to calculate probabilities and risk, it is convenient to return to the ordinary method of slices, wherein:

$$S = \Sigma S_i = \Sigma[cL_i + (N_i - U_i)\tan\phi \quad (1)$$

where S is the shearing resistance at the base of the slice along arc length L, U is the pore water uplift force opposing normal force N at the base of the slice, and i signifies individual slices.

Let us denote the standard deviations by a tilde (~) such that $\tilde{c}$ is the standard deviation of cohesion c, and $\tilde{c}^2$ is its variance, a mathematical convenience because variances are directly additive. If L is not a variable, we may write (Vanmarke, 1980):

$$\tilde{S}_i^2 = (\tilde{c}L_i)^2 + (\tilde{N}_i^2 + \tilde{U}_i^2)\overline{\tan}^2\phi + (\overline{N}_i^2 - \overline{U}_i^2)\tilde{\tan}^2\phi \tag{2}$$

where a bar (—) signifies a mean value. Variability of $\tilde{N}_i$ is small, and variability of $\tilde{U}_i$ either will be small or impossible to evaluate without piezometer measurements over extended periods. It is therefore convenient to arbitrarily assign maximum anticipated values for U_i and let $\tilde{N}_i = \tilde{U}_i = 0$, whereupon

$$\tilde{S}_i^2 = (\tilde{c}L_i)^2 + (N_i^2 - U_{imax}^2)\tilde{\tan}^2\phi \tag{3}$$

This is a simple and rapid calculation via computer program or spreadsheet. The individual variances are summed, giving a standard deviation for total resisting force

$$\tilde{S} = \sqrt{\Sigma\tilde{S}_i^2} \tag{4}$$

The use of the ordinary method of slices will tend to overestimate N_i and therefore $\tilde{S}$, which is on the safe side.

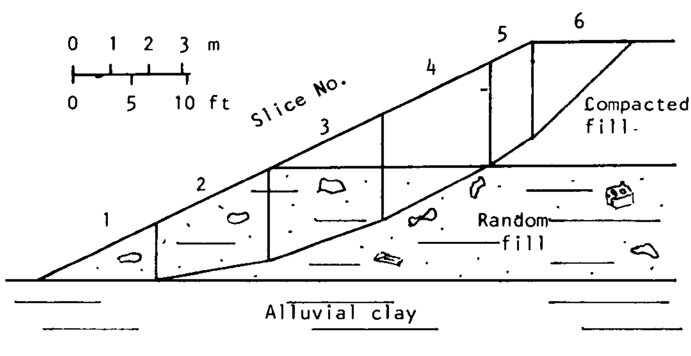


Figure 8. Lagoon embankment cross-section.

Risk Analysis Based on Normal Curve Area. In the present problem $U_i = 0$, and from equations (3) and (4) it can be shown that $S = \overline{S} \pm \tilde{S}$

= 18,670 ± 3,063 lb. By the ordinary method of slices FS = 1.25, so the critical value of S* = 18,670/1.25 = 14,940 for failure. This value is (18,670 - 14,940)/3,063 = 1.21 standard deviations from the mean. Comparison with a table of areas under a normal curve indicates a probability of a lower value of 0.5 - 0.387 = 0.11, or 11 percent probability that the shear strength as indicated by the individual BST's is less than that required for stability. Except in a highly sensitive soil this will not be a probability of failure because the BST measures shearing strength over an area only 10 sq in. (0.0645 m^2) whereas a slope failure involves simultaneous shearing of an area that is many orders of magnitude larger.

Shear Averaging and the Use of Student's t. More relevant than the range of individual strength values may be the lower confidence limit for the mean, which for a small number of tests is appropriately evaluated by use of "student's t" (Snedecor and Cochran, 1967):

$$t_{2p} = \frac{(\overline{S} - S^*)\sqrt{n}}{\tilde{S}} = \frac{\overline{S}(1 - 1/FS)\sqrt{n}}{\tilde{S}} \tag{5}$$

where S* is the minimum value of $\overline{S}$ to prevent failure, and p is a subscript denoting the probability of a higher value of t and hence a higher or lower value of S*. (See also Harr, 1977.) The multiplier 2p is used because this is a "one-tailed" tests for lower S* values only. In the problem under consideration

$$t_{2p} = \frac{(18{,}670 - 14{,}940)\sqrt{10}}{3{,}063} = 3.85$$

The degrees of freedom being n - 1 = 9, from a table of t the probability p is less than 0.5 percent that the FS < 1, i.e., that the embankment must fail because the mean strength is less than that required for stability.

Precision of Estimates. The calculated FS and the failure probabilities depend on precision of estimates of the means and standard deviations, which in turn depend on the number of tests. A convenient formula based on the area under a normal curve is (ASTM Designation: E122-72):

$$n = \left[\frac{1.96\tilde{S}}{\overline{S}\ (e/100)}\right]^2 \tag{6}$$

when n is the number of tests required to define a mean $\overline{S}$ within an acceptable percent error e, the value 1.96 is selected to correspond to a 95 percent confidence that error will be less than e. In the case at hand for e = 10 percent, n = 9 tests.

The above evaluations of failure risk do not incorporate a variability in acting force that, as it depends mainly on unit weight, is very small compared to the large variability in the shear strength.

Solution. Consideration had been given to adding a toe berm to the embankments, but because of the cost and the costs of delay it was decided that the level of risk was acceptable for water testing. The lagoon is now functioning satisfactorily and has been in service about one year.

CONCLUSIONS

1. The Borehole Shear Test (BST) provides a rapid and accurate method for determining relevant Mohr-Coulomb failure parameters in homogeneous slip zones of active landslides.

2. Based on back-calculations and BST ϕ data, sliding tends to reduce the cohesion of saturated, fine-grained soils to near zero.

3. The BST yields large amounts of c and ϕ data in a short time, and thus is adaptable for probabilistic risk analysis based either on strength variability or on indeterminacy of the means.

4. The BST stage test method is particularly useful for defining c and ϕ values for each sample, these values being more appropriate for averaging than are the analogous values derived from separate tests of individual samples.

ACKNOWLEDGEMENTS

BST's for the Division Street landslide and for the lagoon study were performed by Dr. Glen Ferguson and Terracon Consultants, Inc.; and for the Foster Drive landslide by Brian Linnan and Patzig Laboratories, Inc. The author wishes to thank the several owners involved for making these data available. A special thanks to Dave Robson for editing and layout.

APPENDIX I -- REFERENCES CITED

1. Biggs, Donald L., and Sendlein, Lyle L. "Site Investigation, Des Moines." Unpublished report, 1964.
2. DeMartincourt, J. P., and Bauer, G. E. "The Modified Borehole Shear Device." Geotechnical Testing Journal, ASTM, 1983, Vol. 6, No. 1, pp. 24-29.
3. Handy, Richard L., and Fox, N. S. "A Soil Bore-hole Direct Shear Test Device." Highway Research News, Highway Research Board, No. 27, 1967, pp. 42-51.
4. Handy, Richard L., and Williams, W. W. "Chemical Stabilization of an Active Landslide." Civil Engineering, ASCE, Vol. 37, 1967, No. 8, pp. 62-65.
5. Harr, Milton E. Mechanics of Particulate Media: A Probabilistic Approach. McGraw-Hill Int. Book Co., New York, 1977.
6. Huang, Yang H. Stability Analysis of Earth Slopes. Van Nostrand Reinhold Co., New York, 1983.
7. Kenney, T. C., and Watson, G. H. "Multiple-Stage Triaxial Test for Determining c′ and ϕ′ of Saturated Soils." Proceedings of the V ICSMFE, Paris, 1961, Vol.1 pp. 191-195.
8. Janbu, Nilmar. "Slope Stability Computations," in Embankment-Dam Engineering, Casagrande Volume. John Wiley and Sons, N.Y., 1973, pp. 47-86.
9. Sauer, E. Karl, and Christiansen, E. A. "A Landslide in Till near Warman, Saskatchewan, Canada." Canadian Geotechnical Journal, 1985, Vol. 22, No. 2, pp. 195-204.
10. Schmertmann, John H. "Measurement of In Situ Shear Strength." Proceedings of the Conference on In Situ Measurement of Soil Properties. ASCE, Vol. II, pp. 47-138.
11. Skempton, A. W. "Slope Stability of Cuttings in Brown London Clay." Proceedings of the IX ICSMFE, Tokyo, 1977, Vol. 3, pp. 261-270.
12. ________, discussion, Proceedings of the XI ICSMFE, San Francisco, 1985, Vol. 5 (in press).
13. Snedecor, George W., and Cochran, William G. Statistical Methods, 6th Ed., Iowa State University Press, Ames, 1967.
14. Vanmarke, E. H. "Probabilistic Stability Analysis of Earth Slopes." Engineering Geology, 1980, Vol. 16, pp. 29-50.