EMBANKMENT FAILURE ON ORGANIC CLAY
RUPTURE D'UN REMBLAI SUR ARGILE ORGANIQUE

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SYNOPSIS A test embankment of granular fill was constructed in a Portland, Maine tidal area underlain by 25 to 30 ft. of an organic clay containing shells, sand lenses, and wood chips. One slope failed, and cracks developed along two other slopes, at a fill height of 12 ft. The average in situ undrained shear strength, back computed from the failure, equal to 255 psf, was less than one half of the average strength measured by many field vane tests. Unconfined compression tests underestimated the in situ strength, whereas triaxial UU tests were only 10% too high.

Field settlement data were a good indicator of the impending failure of an external slope, but not for an embankment having wide berms. In the latter case, slope indicators and alignment stakes proved most useful. The piezometers were not effective control devices for this particular soil.

INTRODUCTION

The federally assisted highway program calls for construction of Interstate I-295 through Portland and South Portland, Maine, coastal cities in northern New England. To assist in the design of the highway, the Maine State Highway Commission (MSHC) constructed two test sections for the purpose of evaluating three different methods of vertical sand drain installation.

During construction of one test section founded on a soft deposit of organic clay, and before any sand drains had been installed, a portion of the embankment failed unexpectedly. After remedial measures to stabilize the slope were taken and sand drains were installed, additional fill was placed until field observations indicated that another failure was imminent. This paper analyzes the first failure and summarizes settlement and lateral deformation data obtained during final construction of the embankment.

A plan of the test section as originally designed, designated as the Fore River Test (FRT) site, is shown in Figure 1. It is located in a tidal mud flat area of Long Creek, which flows into Fore River, separating Portland and South Portland. Original ground surface elevation averaged -3 ft. MSL and the tide generally varied between ±6 ft. MSL. The soft organic clay, extending to El. -25±8 ft., overlay a silty sand and gravel having an artesian pressure of five feet.

The test section was to be constructed with granular fill to El. +10 over a 240 ft. square area, sand drains installed within the interior 120 ft. square area, and granular fill then placed to El. +20 over the central area. A slide occurred along the North slope, as shown by the cross-hatched area in Figure 1, just after the fill reached El. +9.5. When cracks appeared shortly thereafter parallel to the East and South slopes, these slopes were flattened to 8:1 and the embankment leveled to El. +8.

Field instrumentation existing prior to the failure consisted of 21 settlement platforms (base El. +1.5), four Geonor vibrating wire piezometers (tip El. -16), and 30 alignment stakes installed along the El. +2 berm of the East and North slopes, 100 ft. beyond the East and North slopes, and adjacent to the oil pipelines. Instrumentation installed after the failure included nine hydraulic piezometers and five Wilson slope indicators. Those instruments of particular interest are numbered in Figure 1.

SOIL PROPERTIES

Twenty-one borings were made, mostly prior to construction, within the area shown in Figure 1. Numerous 3.5 in. dia. shelby tube samples were obtained for classification, strength, and consolidation tests.

Some of the results are summarized in Figure 2. The organic clay is quite heterogeneous, with varying amounts of broken shells, organic matter and sand lenses and pockets. The natural water content varied from 30 to 80%; typical Atterberg limits were: liquid limit = 65 ± 10%, plastic limit = 33 ± 2%. The limits generally plotted within ±3% plasticity index of the A-line on Casagrande's plasticity chart.

Values of maximum past pressure determined from standard oedometer test data, plotted at the end of primary consolidation (log time method) and using Casagrande's construction, decreased slightly with depth. The clay is precompressed at the top and almost normally consolidated at the bottom. For virgin compression, $C_c/(1 + e_o)$ averaged 0.205 and ranged from 0.15 to 0.25; the coefficient of consolidation typically equalled 0.05 ± .03 ft.^{2} per day.

The results of field vane tests from nine borings made prior to construction at FRT show considerable scatter (see Figure 2). The average value of undrained shear strength (s_u) equalled 525 ± 15 psf depending on which borings were included in the average or whether or not extreme values were eliminated. The field vane procedures will be discussed later. Undisturbed samples were available from four preconstruction borings made near the center of the site; however, the laboratory testing was not completed until <u>after</u> the failure. The unconfined compression (U) tests yielded values of 0.5 x $(\sigma_1-\sigma_3)$ max. typically in the range of 200 ± 50 psf, as shown in Figure 2, which were generally

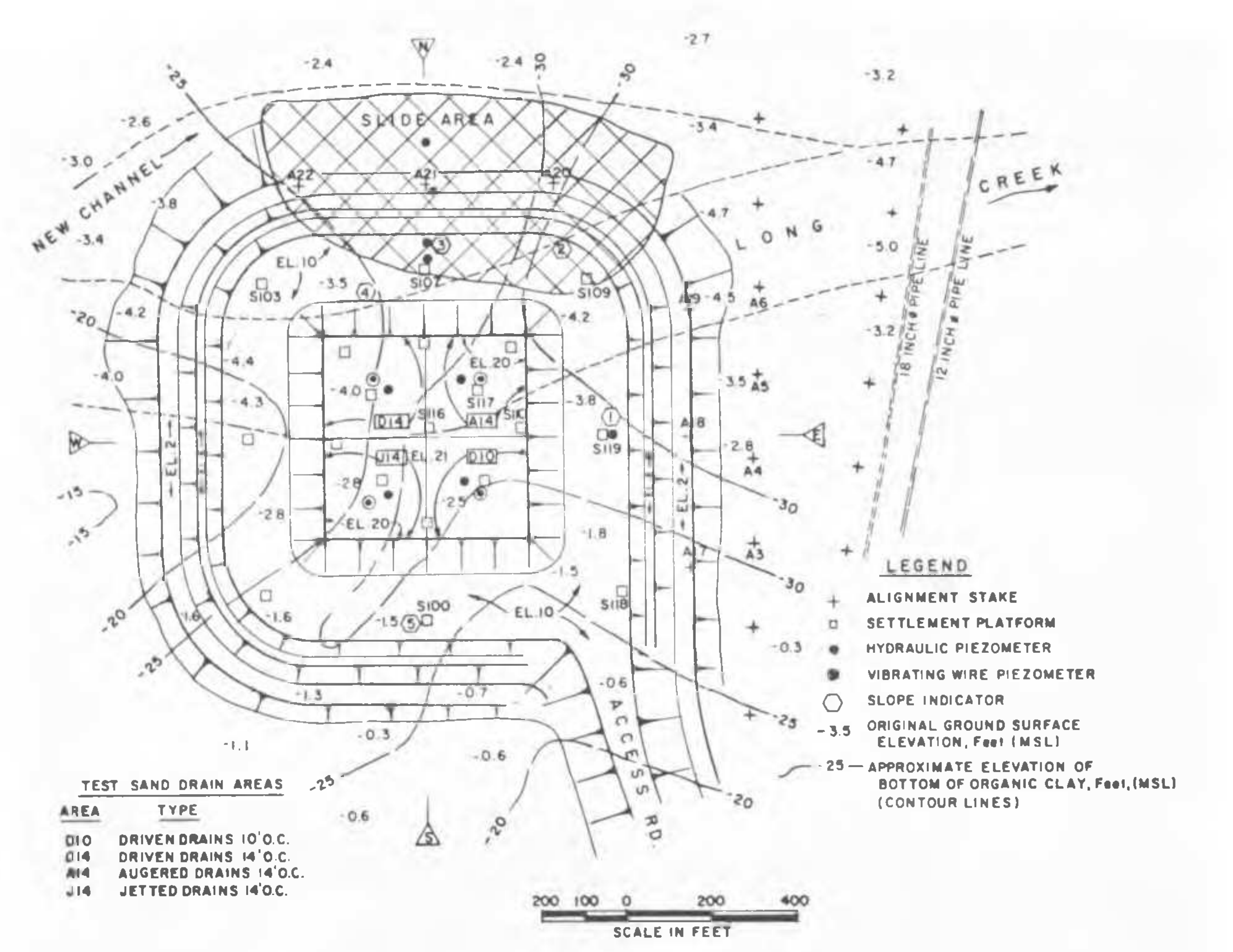

Fig. 1 Plan of Fore River Test Section

less than one half of those from the field vane tests. The 2.8 in. dia. by 6 in. specimens were sheared at 1% per hour. Sixteen lab vane tests run according to Gray (1957) had strengths equal to 380 ± 140 psf or about midway between the unconfined and field vane data.

Within 8 to 14 days after the failure, three pairs of borings were made along the North axis of FRT at 107, 147 and 201 ft. from the center of the site. Almost continuous field vane tests in one set of the borings did not show any consistent change in strength with depth, nor was the average significantly different from the preconstruction data (see Figure 2). Unconfined compression and unconsolidated-undrained (UU) triaxial compression tests were run on undisturbed samples taken in the second set of borings. The UU tests had a cell pressure of 2050 psf and used a strain rate of 0.8% per min. with 2.8 in. dia. by 6.1 in. specimens. As with the field vane tests, the U strengths were essentially the same as those obtained with the preconstruction samples. The confinement in the UU tests generally gave higher strengths, as would be expected with a soil containing shells, fibers, and sand pockets and lenses. The axial strains at failure varied from 5 to 15% in the U and UU tests.

Ten consolidated-undrained direct-simple shear ($\overline{CU}$ DSS) tests employing the Geonor apparatus (Bjerrum and Landva, 1966) were run on undisturbed samples of the organic clay at various vertical consolidation stresses ($\bar{\sigma}_{vc}$) and overconsolidation ratios (OCR). The data were used to develop a relationship between the strength ratio $s_u/\bar{\sigma}_{vc}$ and the OCR. This relationship, when combined with the values of $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$ plotted in Figure 2, yielded the following estimates of undrained shear strength (s_u):

El. (ft.)	= -10	-15	-20	-25	-30
OCR	= 5.4	2.55	1.65	1.25	1.05
$s_u/\bar{\sigma}_{vc}$	= 1.14	0.65	0.45	0.35	0.30
s_u (psf)	= 210	205	200	205	215

Thus the use of strength ratios from CU tests and a knowledge of the stress history of the clay predicted an essentially constant strength with depth and one that equalled the average s_u from the unconfined compression tests.

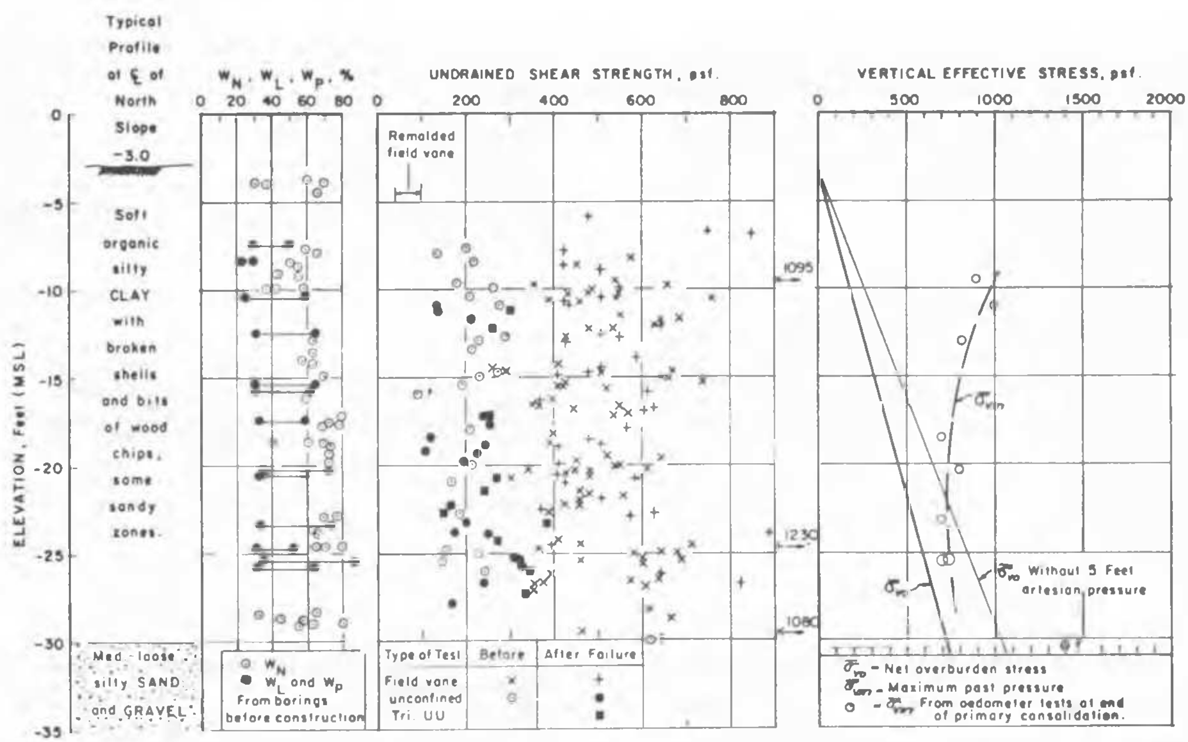

Fig. 2 Soil Properties

Consolidated-undrained triaxial compression and direct-simple shear tests on K_o consolidated samples of the organic clay in the normally consolidated range showed zero cohesion intercept and a friction angle of 32 ± 2 degrees at maximum obliquity in terms of effective stresses.

The granular fill placed in the embankment was a clean gravelly sand with 75 to 100% passing the No. 4 sieve and about 5% passing the No. 100 sieve.

CONSTRUCTION HISTORY AND DESCRIPTION OF FAILURE

After three weeks of construction, the fill reached El. +2 on 24 May 1967. The settlement platforms were installed and filling resumed on 6 July at an almost constant rate until El. +9.5 was reached on Saturday 29 July 1967. The failure of the North slope was discovered the following Monday morning. It had probably occurred at one of the low tides during the preceding day, which had a heavy rain. Settlement platforms within the slide area (S107 and S109 in Figure 1) dropped 1.5 to 3.5 ft. and continued to settle an additional two feet during the next two weeks. These platforms also moved outward several feet. The slide displaced the alignment stakes A20 and A21 by 3.0 to 5.5 ft., these moved outward an additional 1.5 to 2 ft. with time. The toe of the slope heaved several feet.

During the week following the failure, cracks appeared along the East and South slopes. These slopes were than flattened to 8 to 1 and the embankment was cut to El. +8 on 11 August. No signs of distress were observed along the West slope where the depth of clay was much less, as shown by the contours in Figure 1.

STABILITY ANALYSES

Total stress stability analyses employed both sliding wedges and circular arcs. The latter used M.I.T.'s Project ICES computer programs (Whitman and Bailey, 1967) based on the simplified Bishop (Bishop, 1955) and bouyant unit weight Fellenius (Turnbull and Hvorslev, 1967) methods of analysis. The cross section and soil properties used in the analyses, and the location of the most critical wedge and circular arc, are shown in Figure 3. The backcomputed undrained shear strengths (s_u) of the organic clay required for a factor of safety (F) equal to unity from these analyses were:

Sliding wedge	s_u = 255 psf
Bishop circular arc	s_u = 250 psf
Fellenius circular arc	s_u = 265 psf

Decreasing the friction angle in the top six feet of fill from 40 to 30 degrees or raising the water table in the fill by two feet decreased the required s_u by less than 5 psf; decreasing the depth of clay reduced s_u by 3 psf per foot change in thickness.

It is concluded that the average in situ undrained strength at failure equalled 255 psf. This value is thought to be realistic since: (1) horizontal force and moment equilibrium yielded the same results; (2) the critical wedge and circle locations agreed with the observed deformations, which were not too different from plane strain; (3) the fill properties were relatively unimportant; (4) the depth of clay and fill geometry were well defined; (5) laboratory and field strength data indicated an almost constant strength with depth; and (6) the clay was not particularly strain sensitive.

Circular arc effective stress analyses using the forementioned computer program studied stability as a function of the pore pressure ratio R_u = (pore pressure u) / (vertical total stress σ_v) with a clay friction angle of thirty degrees. For the cross section in Figure 3, the back computed required average R_u at failure equalled 0.76 and 0.73 from the Bishop and Fellenius methods respectively. Since R_u equals 0.73 without <u>any</u> fill, and since the average degree of consolidation under the embankment at the time of failure was probably less than 15 ± 5%, these effective stress analyses did not yield realistic results. Analyses of the slope that existed after failure based on pore pressure measurements from the piezometers installed shortly after failure (see Figure 3) gave safety factors less than 0.8. Because values of F were very sensitive to relatively small changes in friction angle and R_u, further analyses are required before reaching definite conclusions regarding the reliability of effective stress analyses with this soil.

MEASURED VS. IN SITU UNDRAINED STRENGTH

Table 1 compares the backcomputed average in situ undrained shear strength of 255 psf with values measured by six different methods. The stability analysis and the field vane and direct-simple shear tests presumedly measure the shear stress on the failure plane at failure, τ_f. U and UU tests directly measure the maximum shear stress, $q_f = 0.5\ (\sigma_1-\sigma_3)_f$, but equate this to s_u to partially account for the effects of sample disturbance.

The field vane overestimated the in situ s_u by a factor of two. Gray (1957) describes the procedures used by the Maine State Highway Commission (MSHC). A 3.5 in. dia. by 7 in. long vane is inserted into the soil until the top of the vane is level with the bottom of the bore hole. A hand operated torque wrench rotates the vane such that failure is usually reached in about five seconds. The resulting rate of rotation is about 40 times faster than that recommended by ASTM D2573-67T. Recent vane tests by the MSHC showed an average reduction of 10% in strength when failure was reached in 25 to 30 seconds. Tests where the vane is inserted one to two feet below the bottom of the bore hole do not show significantly different strengths. Thus the procedures used by the MSHC can not, by themselves, explain the large discrepancy between measured and in situ strengths. The presence of shells, sand lenses, and wood chips in the organic clay undoubtedly contribute to the greater measured strength. However, there is no reason a priori why field vane tests should necessarily yield the appropriate in situ strength (Ladd, 1967).

The unconfined compression tests underestimated the strength although the difference was only 15 to 20%, based on the usual definition of s_u. When a confining pressure was applied, the strengths were substantially increased and they agreed quite well with the in situ value. The strength ratio-stress history method of prediction from direct-simple shear tests yielded an average strength that was 18% too low based on the correct definition of s_u.

As previously pointed out, the average degree of consolidation $\bar{U}$ under the embankment at the time of failure was probably less than 15 ± 5%. For this value of $\bar{U}$, and since most, if not all, of the clay is precompressed, the resulting average increase in s_u due to consolidation should have been less than 10 psf. It is therefore valid to compare strengths from Methods 1, 2 and 6 in Table 1 with the backcomputed value at failure. The fact that field vane and U data obtained before and after failure agreed so well may be fortuitous. The substantial movements must have caused some remolding and loss of strength; this may have been counterbalanced by strength increases due to consolidation that occurred prior to the post-failure borings.

DEFORMATIONS DURING CONSTRUCTION

Settlements measured during construction of the El. 10 berm (see Figure 1) are summarized in Figure 4. At the time of the failure of the North slope, the center of the test section (S116) was settling at a rate of 0.015 ft. per day, which is about double the rate of one-dimensional consolidation measured shortly thereafter. The difference is due to strains induced primarily by undrained shear of the foundation clay. Such shear induced settlements were much greater along the crest of the embankment. Had these data been plotted and interpreted on a daily basis*, they would have clearly warned of imminent distress.

*Since the field vane strengths used in design indicated a very stable embankment, there was no pressing reason to immediately plot and study the field data.

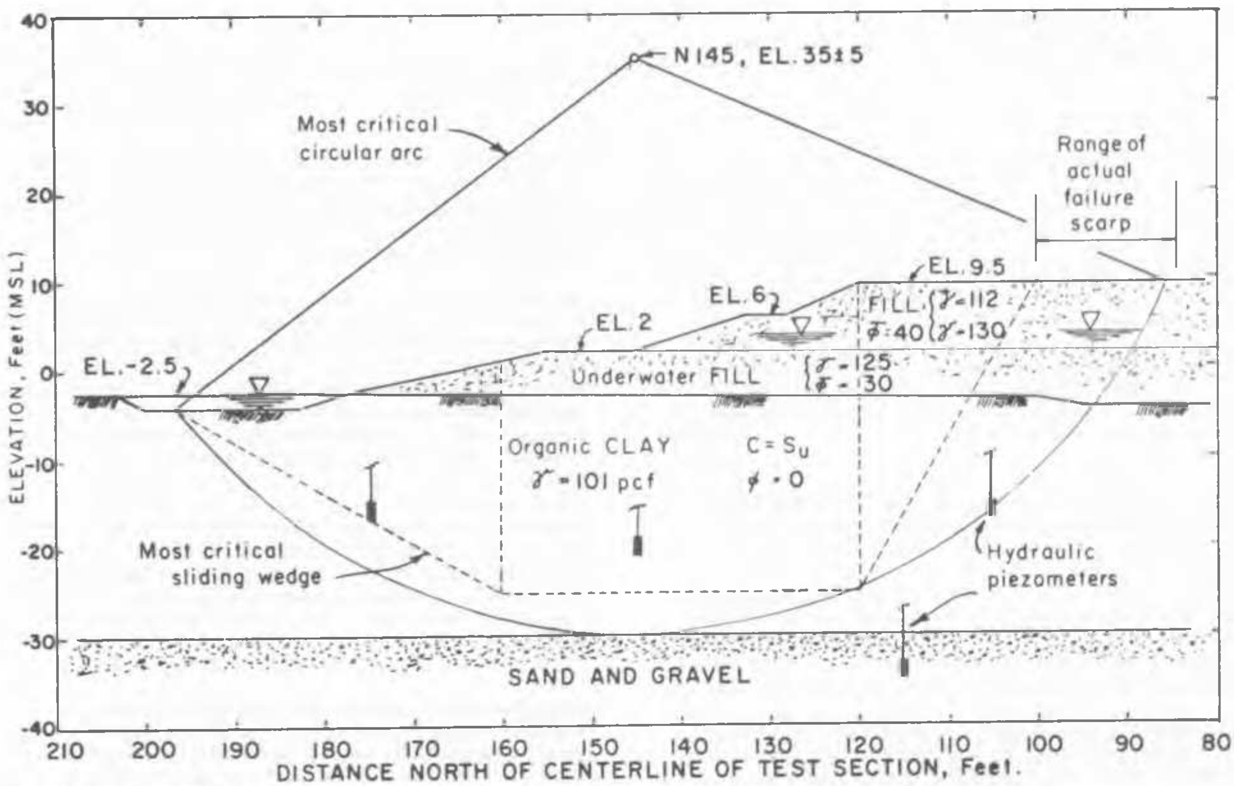

Fig. 3 Stability Analyses

Table 1 Measured vs. In Situ Undrained Strength

Test Method	No. of Tests	Average Undrained Shear Strength, s_u, psf: $\tau_f^{(1)} = q_f \cos\bar{\phi}$	Average Undrained Shear Strength, s_u, psf: $q_f = \frac{1}{2}(\sigma_1-\sigma_3)_f$	Standard Deviation / Average Strength	Remarks
STABILITY ANALYSES	-	255	-	-	Believed reliable within ± 15-25 psf.
BEFORE FAILURE					
1. Field Vane	70	525 ± 15	-	-	El. -5 to -25 from 9 borings.
2. Unconfined Compression	21	180$^{(2)}$	210	20%	El.-8 to -26 from 4 borings, highest and lowest value excluded from average.
AFTER FAILURE					
3. Field Vane	50	520	-	-	El.-5 to -25 from 3 borings; 2 highest and lowest values excluded from average.
4. Unconfined Compression	19	180$^{(2)}$	210$^{(3)}$	31%	El. -11 to -28 from 3 borings.
5. Triaxial UU	10	240$^{(2)}$	280$^{(3)}$	23%	Same as above.
6. $\overline{CU}$ Direct-Simple Shear Tests	10	210	-	-	From strength ratio-stress history method.

(1) τ_f = shear stress on failure plane at failure

(2) $\bar{\phi}$ = 30° for U and UU tests

(3) If samples from below El. -25 are excluded, average s_u = 190 and 265 psf from U and UU tests respectively

Figure 4 shows several interesting facts: (1) the rate of settlement along the East and South slopes increased several days prior to the development of noticeable cracks. Undetected cracks probably developed along the North slope prior to its failure. (2) The settlements were largest along those slopes first showing signs of distress, i.e., North slope before East slope before South slope. (3) The removal of two feet of fill and flattening of the slopes to 8 to 1 caused an immediate and substantial reduction in the rate of settlement.

Analyses based on strengths backcomputed from the North slope failure showed that completion of the test section to El. 20 within the central 120 ft. might cause another much larger failure. Consequently, new instrumentation was installed and the rate of construction was carefully controlled via field measurements. Some of these data are summarized in Figure 5.

The jetted and augered sand drains were installed during 16-22 August 1967, followed by the driven drains during 11-13 September. Fill placement commenced 20 September; when large cracks developed along the easterly portion of FRT at fill El. 18.5, some fill was removed and construction was stopped.

As shown in Figure 5, a comparison of settlements did not serve as good predictors of distress since the entire interior section settled more or less uniformly. In contrast, the slope indicators clearly showed substantial lateral deformation within the organic clay after the fill exceeded El.12 to 14. Note that installation of the driven drains in the Northwest and Southwest quadrants (see Figure 1) caused a lateral movement of one inch at a distance of 50 ft. from the closest drain. The alignment stakes did not undergo consistantly large displacements until the fill exceeded El. 17. However, they clearly showed imminent distress two weeks prior to the occurrence of noticeable cracks.

Piezometers within the sand drain area exhibited excess pore pressures approximately equal to the increased vertical stress whereas those installed in the North and South slopes showed little

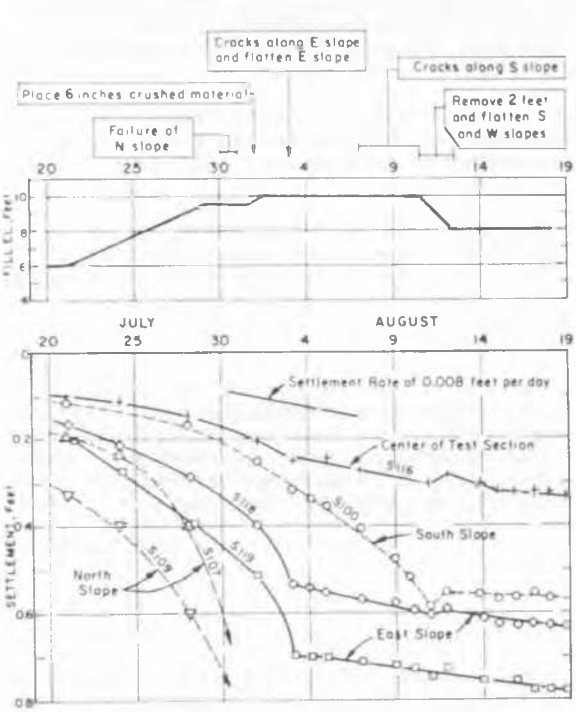

Fig. 4 Settlement Vs. Time

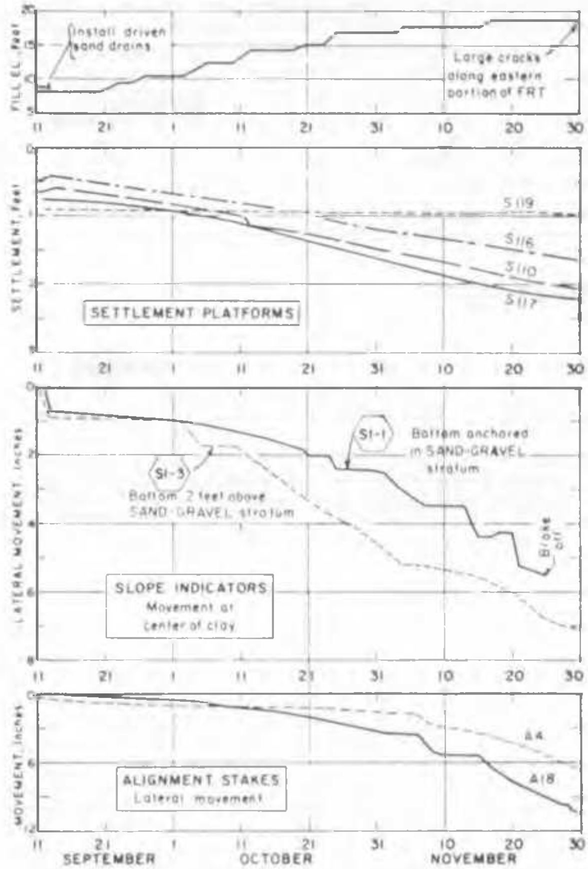

Fig. 5 Deformations During Subsequent Construction

change in pressure during fill placement. Thus the piezometers did not act as predictors of distress.

SUMMARY AND CONCLUSIONS

An embankment rapidly constructed with granular fill on a 25 to 30 ft. tidal deposit of soft organic clay failed along one slope when it reached a height of 12 ft. Analyses of the failure and of field data obtained prior to failure and during subsequent construction lead to the following conclusions:

(1) The average in situ undrained shear strength s_u at failure could be accurately determined as 255 ± 20 psf.

(2) Field vane tests had an average s_u more than double this value, partly due to a fast rate of vane rotation and the presence of shells, sand, etc. in the clay. However, vane tests do not a priori give correct in situ undrained strengths.

(3) The average unconfined shear strength was 15 to 20% too low; confinement in triaxial UU tests increased s_u by one third.

(4) Settlement behavior was a good predictor of the impending failure of an exterior slope, but not for an embankment having wide berms.

(5) In the latter case, slope indicators, and alignment stakes to a lesser degree, were most useful in predicting an imminent failure.

(6) Stability analyses in terms of effective stresses proved inconclusive and piezometers were relatively ineffectual control devices for this particular soil.

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REFERENCES

Bishop, A. W. (1955) "The Use of the Slip Circle in the Stability Analysis of Slopes", Geotechnique, Vol, 5, pp. 7-17.

Bjerrum, L. and Landva, A. (1966) "Direct Simple-Shear Tests on a Norwegian Quick Clay", Geotechnique, Vol. 16, pp. 1-20.

Gray, H. (1957) "Field Vane Shear Tests of Sensitive Cohesive Soils", Trans. American Society of Civil Engineers, Vol. 122, pp. 844-863.

Ladd, C. C. (1967) "In Situ Undrained Strength of Soft Saturated Clay", Proc. Geotechnical Conference, Oslo, Vol. 2, pp. 112-115.

Turnbull, W. J. and Hvorslev, M. J. (1967) "Special Problems in Slope Stability", Proc. American Society of Civil Engineers, Jour. of the Soil Mechanics and Foundations Division, Vol. 93, No. SM4, pp. 499-528.

Whitman, R. V. and Bailey, W. A. (1967) "Use of Computers for Slope Stability Analysis", Proc. American Society of Civil Engineers, Jour. of the Soil Mechanics and Foundations Division, Vol. 93, No. SM4, pp. 475-498.