VOL.99 NO.SM9. SEPT. 1973

JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION

PROCEEDINGS OF
THE AMERICAN SOCIETY
OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

CONTENTS

DISCUSSION

Proc. Paper 9963

This Journal is published monthly by the American Society of Civil Engineers. Publications office is at 345 East 47th Street, New York, N.Y. 10017. Address all ASCE correspondence to the Editorial and General Offices at 345 East 47th Street, New York, N.Y. 10017. Allow six weeks for change of address to become effective. Subscription price to members is $8.00. Nonmember subscriptions available; prices obtainable on request. Second-class postage paid at New York, N.Y. and at additional mailing offices. HY, SM.

10019 SEPTEMBER 1973 SM9

JOURNAL OF THE
SOIL MECHANICS AND
FOUNDATIONS DIVISION

Failure of Underwater Slope in San Francisco Bay

By James M. Duncan[1] and Albert L. Buchignani,[2] Members, ASCE

Introduction

In August, 1970, during construction of a new shipping terminal at the Port of San Francisco, a 250-ft long (76-m) portion of an underwater slope about 90 ft (27 m) high failed. Because the failure took place entirely within the deep deposit of San Francisco bay mud at the site, its occurrence has provided a valuable opportunity for assessing the accuracy of conventional laboratory tests and field vane shear tests for measuring the undrained strength of a normally consolidated clay.

Design of LASH Terminal

Requirements for a new terminal for Lighter Abroad Ship (LASH) operations at the Port of San Francisco made it desirable to develop an area in San Francisco Bay that had previously been used as a disposal area for dredged bay mud. The development plan entailed construction of new dock facilities on the outboard side of the dike shown in Fig. 1. The dike had been built of construction debris in 1962 to 1966 to retain the dredged mud spoil from the nearby Army Street Terminal (10).

The low shear strength and high compressibility of the San Francisco bay mud at the site made it necessary to remove the mud from the area where the new dock will be built and replace it with sand. The new dock will be

Note.—Discussion open until February 1, 1974. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 99, No. SM9, September, 1973. Manuscript was submitted for review for possible publication on October 24, 1973.

[1]Prof. of Civ. Engrg., Univ. of California, Berkeley, Calif.

[2]Assoc., Harding-Lawson Assocs., San Rafael, Calif.

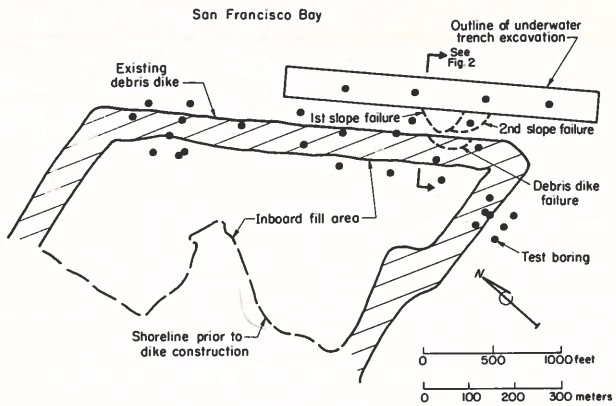

FIG. 1.—Plan of Debris and Trench for Proposed LASH Terminal at Port of San Francisco

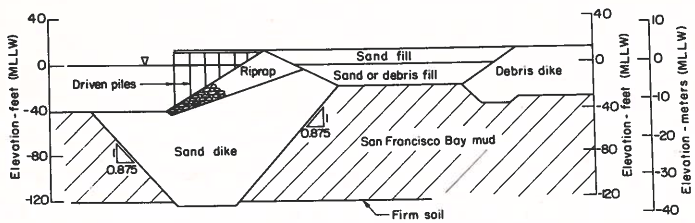

FIG. 2.—Typical Design Section Through Trench and Dike

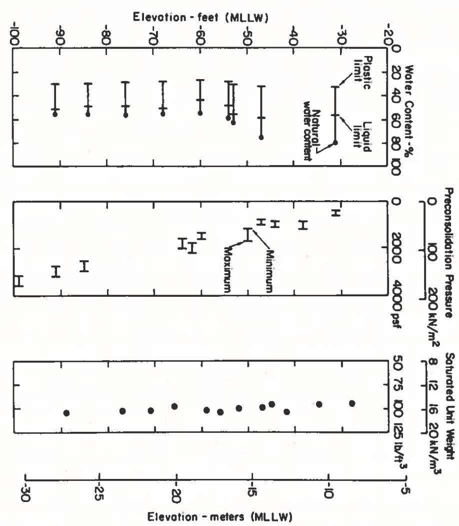

FIG. 3.—Typical Properties of San Francisco Bay Mud—LASH Terminal Site

supported on piles driven into the sand, as shown in Fig. 2. The sand-filled trench serves two purposes: (1) It reduces the settlements of the dock; and (2) it increases the stability of the dock with respect to deep-seated shear failure by forcing the critical shear surface down beneath the soft bay mud. To meet both of these requirements, it was necessary that the sand-filled trench should extend down into the firmer soils beneath the bay mud, requiring excavations from 90 ft to 120 ft (27 m to 37 m) deep.

Soil Conditions

The soil conditions at the site were found to be quite uniform over the entire area outboard from the dike, with about 80 ft to 100 ft (24 m to 31 m) of

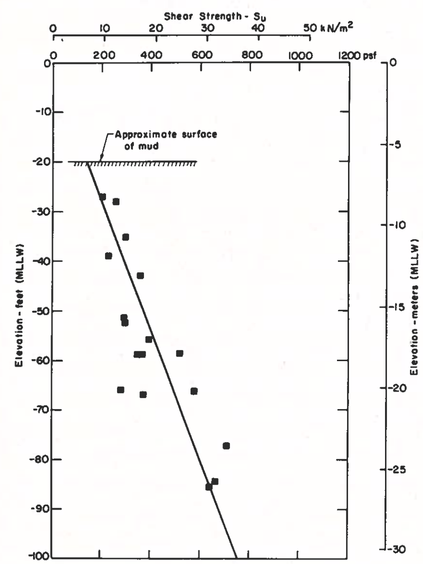

FIG. 4.—Shear Strength Measured in U-U Tests on 2.8 in. Diam Specimens

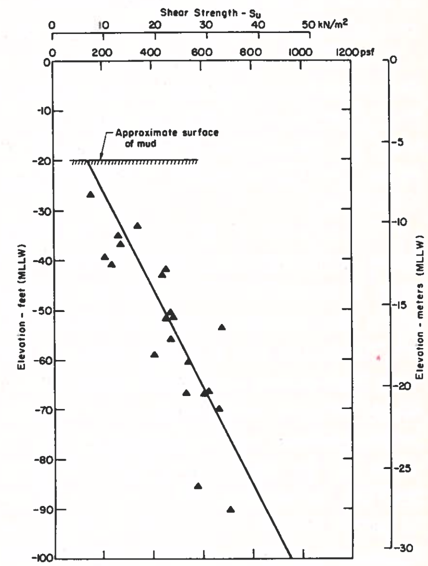

FIG. 5.—Shear Strength Measured in U-U Tests on 1.4 in. Diam Specimens

San Francisco bay mud underlain by firmer clays and sands. The average surface elevation was about −20 ft (−6 m) MLLW and the average elevation of the top of firm soils was about −110 ft (−34 m) MLLW.

The San Francisco bay mud is a normally consolidated, slightly organic clayey silt or silty clay of marine origin. The clay has moderate plasticity, with a liquid limit of about 50% and a plastic limit of about 30%. The values of water content, liquid and plastic limit, preconsolidation pressures, and unit weights for samples from the outboard borings are shown in Fig. 3. It may be seen that the natural water content is greater than the liquid limit throughout the depth of the deposit. As would be expected, the water content decreased with depth.

Determination of Undrained Shear Strength of Bay Mud

As it was anticipated that the trench would be open only a short time before it was filled with sand, and that during this brief period there would not be time for the water content of the mud to change significantly, the stability analyses for the condition during construction were performed using undrained strength values for the mud.

Shelby tube samples, 2.8 in. (71 mm) in diameter, of the bay mud at the site were obtained from a number of test borings at locations shown in Fig. 1. Beneath the debris dike the strength of the upper part of the mud had increased somewhat due to consolidation under the weight of the dike, but the conditions found in all outboard borings were the same, with the strength increasing essentially linearly with depth. The failure occurred entirely within the outboard area, and therefore only the strengths of samples from these borings are considered in the following paragraphs. Because of the similarity of conditions in all these borings, the results for all borings are shown together.

A number of test specimens were prepared by extruding the mud from the sample tubes, trimming the specimens to length, and performing unconsolidated-undrained (U-U) tests on the full 2.8-in. (71-mm) specimens. The results of tests on specimens of this type, shown in Fig. 4, indicate that the undrained shear strength increases approximately linearly with depth beneath the bottom of the bay.

Two aspects of the results of these tests indicated that the test specimens were somewhat disturbed. First, the strains at failure were somewhat larger than is usual for U-U tests on undisturbed specimens of San Francisco bay mud; the average strain at failure for the tests shown in Fig. 4 was about 6%, whereas the average strain at failure is typically about 4% for tests on very high quality undisturbed specimens. Second, the rate of increase of undrained shear strength with depth was somewhat smaller than that determined in research investigations on San Francisco bay mud, where high quality specimens were obtained by hand sampling with large diameter, thin-walled sampling tubes; the rate of increase of strength with depth shown in Fig. 4 is about 7.7 psf/ft of depth (1.2 kN/m^2/m of depth), whereas the rate of increase has been found to be about 10 psf/ft of depth (1.6 kN/m^2/m of depth) for tests on very high quality undisturbed specimens.

In an attempt to minimize the effects of disturbance on the strength test results, a number of specimens were prepared by trimming away the outer 0.7 in. (18 mm) of the 2.8-in. (71-mm) samples after they were extruded. It was considered that the outer portions of the samples probably would be more disturbed than the inner portion, and that by trimming away this material, the effects of disturbance might be reduced.

The results of the tests performed on 1.4-in. (35-mm) diam specimens trimmed from the centers of the 2.8-in. (71-mm) diam samples are shown in Fig. 5, where it may be seen that the rate of increase of shear strength with depth is about 10 psf/ft of depth (1.6 kN/m^2/m of depth). The average value of strain at failure for these tests was found to be about 4%. Thus both the strains at failure and the rate of increase of strength with depth indicate that the 1.4-in. (35-mm) diam specimens were less disturbed than were the 2.8-in. (71-mm) diam specimens.

The undrained shear strength of the bay mud was also measured by means of a number of field vane shear tests performed in the test borings from which undisturbed samples were obtained. The results of the tests performed in outboard borings in the trench area are shown in Fig. 6. The values of vane shear resistance measured after the vane had been rotated several times to remold the material indicated that the sensitivity of the mud was 4 to 8.

The variations of strength with depth determined by the laboratory U-U tests and the field vane shear tests are compared in Fig. 7. The rate of increase of strength with depth is somewhat larger for the vane shear tests than for either series of laboratory tests. On the average, the strength values determined by laboratory U-U tests on 1.4-in. (35-mm) diam specimens exceed those on

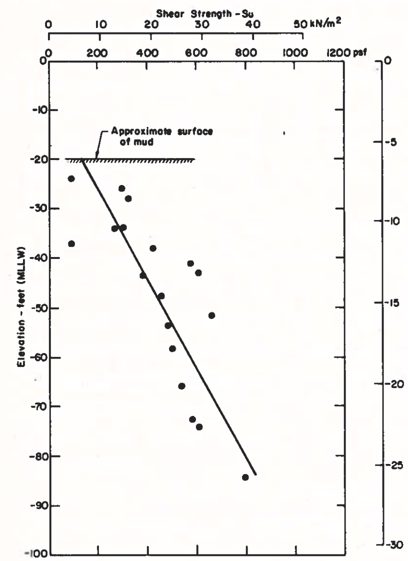

FIG. 6.—Shear Strength Measured in Field Vane Shear Tests

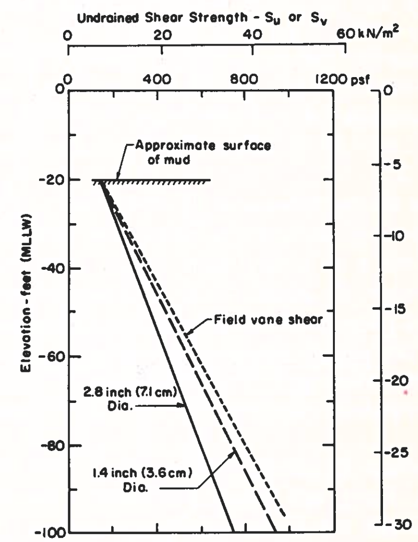

FIG. 7.—Comparisons of Shear Values Measured Using Laboratory U-U Tests and Field Vane Shear Tests

2.8-in. (71-mm) diam specimens by about 20%, and the strength values determined by the field vane shear tests exceed those determined by the tests on 1.4-in (35-mm) specimens by about 8%.

Stability Analyses

A number of stability analyses were performed to determine variations of factor of safety with steepness of the trench slopes and distance from the debris dike. These were total stress analyses, performed using $\phi_u = 0$, as indicated by the strength test results, and values of undrained shear strength increasing with depth. For purposes of the analyses, the continual increase of strength with depth indicated by the test results was represented by a series of layers, each with a constant value of strength over its depth, as shown in Fig. 8.

The shear strength of the debris fill used in the analyses, determined by loading a section of the dike to failure, was about 800 psf (40 kN/m^2). The analyses

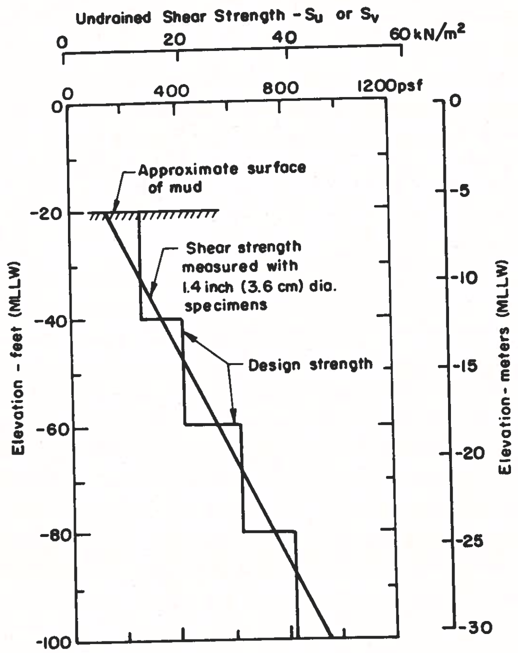

FIG. 8.—Design Strength Based on U-U Tests on 1.4 in. Diam Specimens

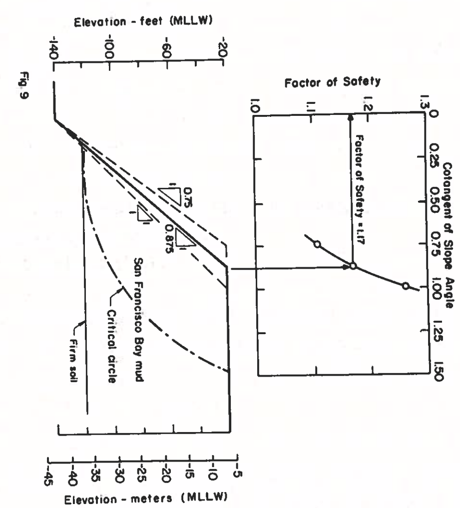

FIG. 9.—Variation of Factor of Safety with Slope Angle for Trench Slopes not Affected by Proximity to Debris Dike

were performed using buoyant unit weights for the materials below the water table to account for the effects of external water pressure on the slopes. The

values of buoyant unit weight were 38 pcf (6.0 kN]^3) for the bay mud and 25 pcf (3.9 kN1hm^3) for the debris fill. The moist unit weight of the debris fill above water was 85 pcf (13.5 kN/m^3).

For the condition during construction, with the trench open, two possible types of failures were investigated. The first was the type that would develop if the trench slope was too steep, wherein the newly excavated slope would fail but the adjacent debris dike would not be affected. The second was the type of failure that would occur if the trench was excavated too near to the debris dike, wherein the dike, or a portion of it, would slip into the trench.

Analyses were first performed to determine how steeply the trench slope could be excavated, assuming it was far enough away to be unaffected by its proximity to the debris dike. The results of these analyses are shown in Fig. 9, in which the variation of factor of safety with slope angle is shown. Previous experience indicated that quite high underwater slopes in bay mud could be excavated at least as steep as 45° (0.8 rad), or 1 on 1. This is in agreement with the results shown in Fig. 9, which indicate that the calculated factor of safety for a submerged 1 on 1 slope at this site is 1.26. The results in Fig. 9 indicate that even steeper slopes should also be stable; the calculated factor of safety for a 0.875 (horizontal) on 1 (vertical) slope is 1.17, and that for a 0.75 on 1 slope is 1.11.

The difference in estimated construction costs between these alternatives was quite considerable. Including savings in both excavation and backfilling, it was estimated that the use of 0.875 on 1 trench slopes would result in a saving of more than $200,000 as compared to the use of 1 on 1 slopes. In view of these large potential savings and the fact that the trench would be open only during construction, it was decided to use 0.875 on 1 trench slopes and to accept the low value of 1.17 for the factor of safety for the during construction condition, even though this value was lower than is normally considered acceptable.

Having decided the slope angle to be used in the trench, additional analyses were performed to determine the necessary distance between the trench and the debris dike in order that the dike, or a portion of it, should not slip into the trench. The results of these analyses are shown in Fig. 10. It may be seen that the calculated values of factor of safety with respect to this type of failure decrease with decreasing distance between the trench and the dike, and that the calculated factor of safety with respect to this type of failure is equal to 1.17 when the top of the trench slope is located about 160 ft (50 m) outboard from the top of the debris dike. This fact was used to establish 160 ft (50 m) as the minimum distance between the top of the trench slope and the top of the debris dike. With the trench slope inclined at 0.875 on 1 and the top of the trench slope located 160 ft (50 m) from the top of the dike, the calculated factors of safety for both types of failure have the same value, 1.17.

Failure

On August 20,1970, after a section of the trench about 500 ft (150 m) long had been excavated, the dredge operator found that the clamshell bucket could not be lowered to the depth from which mud had been excavated only hours before. Using the side-scanning sonar with which the dredge was equipped,

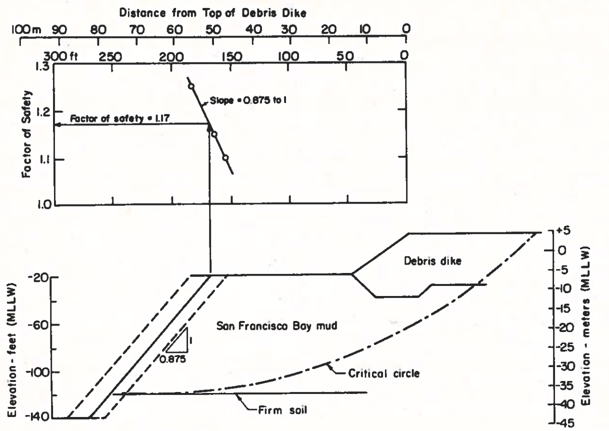

FIG. 10.—Variation of Factor of Safety for 0.875 to 1 Slope with Distance from Debris Dike

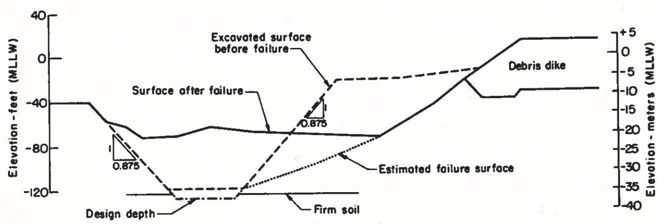

FIG. 11.—Typical Section After Failure

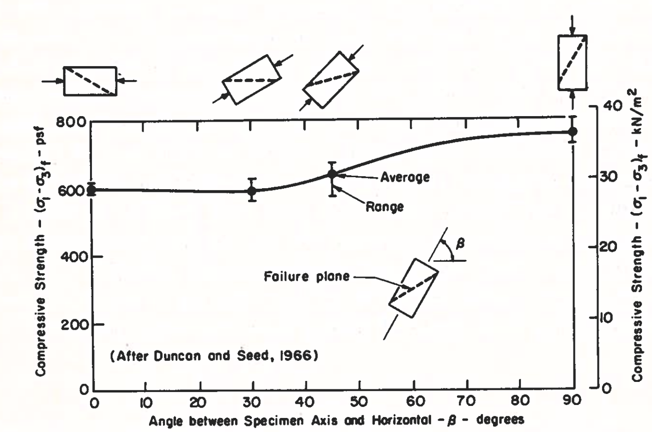

FIG. 12.—Effect of Anisotropy on Undrained Strength of San Francisco Bay Mud (Ref. 11)

four cross sections were made within 2 hr which showed that a failure had occurred that involved a 250-ft (75-m) long section of the trench. The location of the failed section is indicated in Fig. 1. The cross section after failure, and the estimated failure surface, are shown in Fig. 11. The failure surface is somewhat less sharply curved, and is somewhat shallower over most of its length than the calculated critical circle shown in Fig. 9.

Although the shear surface extended back to the toe of the debris dike, the dike itself did not fail at this time. The debris fill had a fairly low unit weight and high shear strength, and fortunately the dike remained stable even after the trench had been widened by the failure. After a period of about 4 months had passed and the failed portion of the trench was still not completely filled with sand, a section of the debris dike about 300 ft (90 m) long did fail (see Fig. 1).

As soon as possible after the failure occurred, the debris dike adjacent to the failed section was cut down to just above water level to minimize the possibility that a portion of the debris dike might slip into the trench later. It was also considered desirable to fill the already excavated portion of the trench with sand as soon as possible to prevent additional failures of the type that had occurred. However, because of the contractors' requirements for scheduling, some portions of the trench had to remain open for about 4 months after excavation. During this period, one additional failure occurred, of the same type as the first, involving an additional 200 ft (60 m) of length along the trench. The location of this second trench slope failure is also indicated in Fig. 1. The fact that two failures occurred in one section of the slope, while the rest remained stable, is probably due to minor local variations in soil strength or slope geometry. Even the stable sections of the slope must have been very close to failure.

Because the factors of safety for the condition after the end of construction, with the shipping terminal in operation, were higher than for the condition during construction, it was decided not to change the design. The excavation of the remainder of the trench was completed and the trench was backfilled, without further incident of failure.

Sources of Error in Design Analyses

After the failure, detailed consideration was given to a number of possible sources of error in the analyses and data on which the design of the trench slope was based. These included possible inaccuracies resulting from the slope stability analysis procedure, the assumption that the slope would remain undrained throughout the construction period, and errors in determining the unit weight and the shear strength of the San Francisco bay mud. Because the failure involved only the bay mud, and because the underwater deposit at the site had no desiccated upper crust, the investigation of this failure provided a good basis for evaluating the importance of various factors which affect the shear strength of this normally consolidated, highly plastic clay.

Mechanics of Stability Analyses.—It is believed that the method of stability analysis did not involve appreciable computational error. The analyses were performed using assumed circular shear surfaces, but, judging by the fact that the actual failure surface was approximately circular, this could not have involved significant error. Furthermore, because the analyses were performed using $\phi_u = 0$ and because the method employed satisfied moment equilibrium, there were no errors or approximations in the mechanics of the analyses. To check the effect of using the stepwise variation of strength with depth shown in Fig. 8, the analysis of one section was repeated after the failure using a continuous variation of strength with depth. The minimum factor of safety determined by the new analysis differed by less than 1% from the earlier analyses, and the location of the critical circle was also nearly the same, thus showing that use of the stepwise strength variation did not induce appreciable inaccuracy.

Drainage.—One of the most basic assumptions employed in the design analyses was that the trench slope would remain in an undrained condition throughout the construction period. The importance of this assumption stems from the fact that, over a long period of time, the water content of the clay would increase as the clay swelled under the reduced stresses, and the swelling would be accompanied by a reduction in shear strength. Thus an increase in water content would result in a reduction in shear strength which could explain the failure. It is considered highly unlikely, however, that any significant increase in water content occurred during the time between the beginning of excavation and the occurrence of the failure, which was about 2 months. Approximate calculations based on consolidation theory indicate that there would be no swelling whatsoever at depths more than about 5 ft to 10 ft (1.5 m to 3 m) beneath the excavated surface. Furthermore, most of the trench slope remained open from 4 months to 6 months without failing. If increasing water content was responsible for the failure, it would be expected that failures would have occurred with greater frequency later, when the water content of a greater portion of the slope had been affected.

Unit Weight.—Because the slope was submerged and the buoyant unit weight of the bay mud is only 38 pcf (6.0 kN/m^3), an error of only 4 pcf (0.6 kN/m^3) in evaluating the unit weight would change the factor of safety by 10%. Although there was some scatter in the measured values of unit weight as shown in Fig. 3, it does not seem possible that use of the average value could have resulted in inaccuracy of significant magnitude to introduce a significant error.

Shear Strength.—There are several possible sources of error in the shear strength measurements. Although the specimens tested were trimmed from Shelby tube samples and were of good quality, sampling operations inevitably involve some disturbance, and the specimens tested were undoubtedly not completely free of disturbance effects. In accordance with conventional practice the U-U triaxial tests were conducted in 10 min to 20 min on vertical specimens using rough caps and bases. Although similar sampling and testing procedures have been used successfully many times in the past, they involve several systematic errors, the magnitudes of which are reviewed in the following paragraphs.

All the strength tests conducted on bay mud from the LASH terminal site were performed on vertical specimens. Studies have shown, however, that the San Francisco bay mud is anisotropic, and that the shear strengths of specimens trimmed in other orientations are smaller than for vertical specimens. The results of such a study, conducted on specimens from another site, are shown in Fig. 12. About the same results would be expected for the LASH terminal site, since the other properties of the mud from the two sites are similar. Considering the shape of the actual failure surface shown in Fig. 11 and the strength variation

shown in Fig. 12, it is estimated that the effect of anisotropy may reduce the field undrained strength about 10% as compared with the strength of vertical specimens.

Because the tests used to measure the strength of the mud at the LASH site were conducted using rough caps and bases, there was enough end restraint to prevent lateral deformation at the ends of the specimens. The results of previous studies by Duncan and Dunlop (12) on San Francisco bay mud indicate that the strength increase due to end restraint is about 5%. Therefore, it would be expected that the field undrained strength would be about 5% less than the laboratory value as a result of end restraint in the triaxial tests.

TABLE 1.—Comparison of Undrained Strength of San Francisco Bay Mud Measured in Triaxial and Plane Strain Tests

Type of test (1)	Strain condition (2)	Result (3)
Unconsolidated-undrained tests (12)	Triaxial	s_u = 360 psf (17 kN/m^2)
	Plane strain	s_u = 380 psf (18 kN/m^2)
Anisotropically consolidated-undrained tests (11)	Triaxial	s_u/p = 0.35
	Plane strain	s_u/p = 0.37

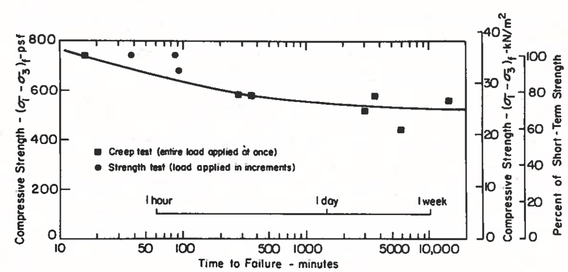

FIG. 13.—Effect of Sustained Loading on Undrained Strength of San Francisco Bay Mud

As a matter of convenience, triaxial tests were employed to measure the undrained strength of the mud, even though the slope was quite long and would be expected to undergo deformations corresponding closely to plane strain. It is known that the undrained strength of San Francisco bay mud measured in plane strain tests is slightly higher than that measured in triaxial tests. The results of two separate investigations of this effect, summarized in Table 1, indicate that the difference amounts to about 5%. Thus, as a result of the plane strain effect, the field undrained strength would be expected to be about 5% greater than that measured in triaxial tests.

As explained previously, efforts were made to reduce the amount of disturbance in the specimens tested by trimming away the outer 0.7 in. (18 mm) of the samples. Judging by the fact that the 1.4-in. (35-mm) diam specimens were about 20% stronger on the average than the 2.8-in. (71-mm) diam specimens, it is apparent that the amount of disturbance was reduced by trimming away the outer portions of the samples. Even so, it is not expected that the 1.4-in. (35-mm) diam specimens were completely free of disturbance. Although the effect of this remaining disturbance on the measured strength value is not known, it is estimated that it may amount to 10% to 20%. If so, the field undrained strength would exceed the laboratory undrained strength by 10% to 20% as a result of disturbance.

This review shows that the systematic errors involved in the laboratory tests used to measure the undrained shear strength of the bay mud at the LASH terminal site were largely self-compensating. As a result of anisotropy and end restraint, the field undrained strength would be expected to be reduced by about 15% in comparison with the laboratory strength. On the other hand, owing to the effects of plane strain deformation and disturbance, the field undrained strength would be expected to be 15% to 25% higher than the laboratory value. As the net effect of both the positive and the negative systematic errors, it would be expected that the field undrained strength would be about equal to, or perhaps 10% greater than, the value measured in laboratory tests. It is thus apparent that the failure cannot be explained in terms of these systematic errors, because the failure shows that the field strength was about 15% less than that measured in the laboratory tests.

Effect of Strength Loss During Creep

To explain the failure of the trench slope at the LASH terminal, it is necessary to consider the reduction in shearing resistance of the bay mud caused by sustained loading and creep. The results of a study of the effect of sustained loading of San Francisco bay mud are shown in Fig. 13. The circles shown in this figure indicate values of undrained strength measured in conventional triaxial tests, where the load was built up gradually to a value that caused failure. The times to failure in these tests varied from about 40 min to 90 min. Other specimens were loaded to various percentages of the compressive strength measured in the conventional tests, and the loads were maintained constant while the deformations were measured. The specimens continued to strain or creep under the constant loads, and eventually the deformation rates increased and the specimens failed. The axial compressive stresses causing failure in these tests have been plotted against the time between application of the load and failure. It may be seen that the compressive strength of the bay mud measured in this way decreases as the time to failure increases. For loads maintained a week or longer, the shearing resistance is only about 70% of the value measured in conventional triaxial tests.

These results show that the shearing resistance of San Francisco bay mud is reduced significantly by sustained loading. Furthermore, the magnitude of this effect is large enough to explain the discrepancy between the strength of the mud mobilized at failure of the trench slope and the strength measured in laboratory tests after correction for all systematic errors.

Loss of strength due to sustained loading has been recognized for several years as a phenomena common to many plastic clays. A considerable amount of research work has been done to investigate this phenomenon in the laboratory and to relate it to the behavior of clays in the field [see Burmister (5), Casagrande and Wilson (6), Casagrande and Rivard (7), Casagrande (8), Singh and Mitchell (15), Skempton and Hutchinson (16), Walker (17), and Wu and Douglas (18)]. Despite this excellent work, it appears to have been the practice in many cases to make no allowance in design for the loss of shear resistance caused by sustained loading. The fact that failures did not occur in some cases where no such allowance was made, of course, does not prove that it is correct to neglect creep strength loss. Designs made with no allowance for creep strength loss may be successful for one or both of the following reasons: (1) The design factor of safety may have been large enough to provide a margin covering the creep strength loss effect; and (2) if the effects of disturbance are large, the resulting reduction in the measured strength would help to compensate for the fact that no allowance was made for creep strength loss.

For the LASH terminal trench slope, the design factor of safety was unusually low, and the samples were of quite high quality. As a result, the margin of safety was not large enough to cover the effects of creep strength loss.

Empirical Corrections for Stability Analyses

As a result of systematic errors in the strength tests and the loss of shear resistance under sustained loading in the field, it would not be expected that the value of undrained strength measured in laboratory tests would be equal to the shear resistance which can be mobilized in the field. Therefore, values of shear strength measured in laboratory tests should be considered as arbitrary indices of soil strength, which require correction before they can be applied to the field. The present case indicates, for example, that it would have been appropriate to reduce the values of undrained strength measured in the laboratory tests by about 15%.

Similarly, as analyzed by Bjerrum (2) it would be expected that values of undrained shear strength measured by means of field vane shear tests would also require correction before they were applied to field problems. If the vane shear strength values shown in Fig. 6 had been used for the design analyses, the calculated factor of safety for the trench slope would have been about 1.26. This indicates that it would have been appropriate to reduce the measured vane shear strength values by about 20% to arrive at the field strength. The magnitude of the correction appropriate for the vane shear tests differs from that for the laboratory tests because anisotropy plays a different role in the two tests, because the strain conditions and times to failure are not the same in the two tests, and because the amount of disturbance induced by insertng the vane is not the same as the disturbance induced by boring, sampling, and handling. Nevertheless, the magnitude of the appropriate correction for strength values from either type of test can be determined by analyses of failures.

In Norway, where the vane shear tests are used more frequently than any other to measure the undrained shear strength of saturated clays for practical purposes, much work has already been done by Bjerrum, et al. (3) to determine the magnitude of the corrections required for vane shear strength values. Aas (1) has compiled a number of instances of short-term failures of excavations in normally consolidated and lightly overconsolidated clays in Norway and Finland. The results of this investigation, shown in Fig. 14, indicate that the magnitude of the calculated factor of safety increases with increasing values of s_u/p for the clays. The results for the failure of the LASH terminal trench slope have also been plotted in Fig. 14, and it may be seen that this case is in good agreement with the data compiled by Aas.

Bjerrum (4) has collected data for failures in both embankments and excavations. These data are shown in Fig. 15, where the factors of safety calculated using vane shear strength have been plotted against the plasticity indices of the clays. The factors of safety for the embankments were calculated assuming a tension crack through the full height of the embankments, thus in effect ignoring the shear strengths of the fill materials. The results for the LASH terminal trench slope are shown in Fig. 15, and it may be seen that it is in good agreement with Bjerrum's (4) data for excavations.

It is important to note that all of the vane shear tests for which data are shown in Figs. 14 and 15 were performed with vanes of the same shape, and using about the same rates of rotation. As a result, it would be expected that

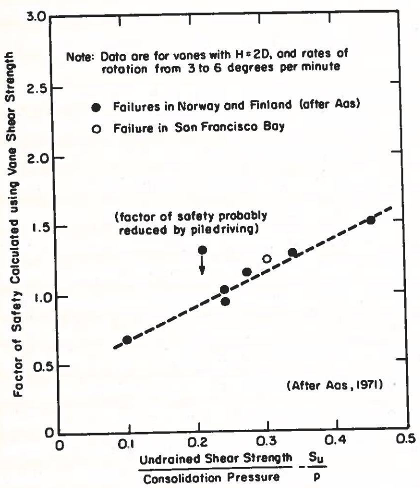

FIG. 14.—Relationship between Calculated Factors of Safety and s_u/p for Excavation Failures in Clay (Ref. 7)

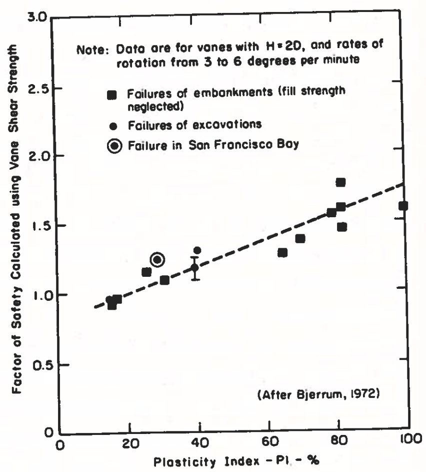

FIG. 15.—Relationship Between Calculated Factor of Safety and Plasticity Index for Excavation and Embankment Failures in Clay (Ref. 4)

the effects of disturbance and rate effect would be about the same in all the tests.

It may be seen that the magnitude of the required correction is largest for clays with large values of s_u/p and plasticity index. This seems entirely reasonable, since these clays are the ones that are subject to the largest strength loss under sustained loading. For clays with small values of s_u/p and plasticity index, the correction factor is even smaller than unity, indicating that the strength measured by vane shear tests is smaller than the strength that can be mobilized in the field. This is undoubtedly because such clays are easily disturbed, and the disturbance due to insertion of the vane, therefore, dominated over the effect of sustained loading.

The correlations shown in Figs. 14 and 15 indicate that it is possible to determine the magnitude of corrections which should be applied to vane shear test results

by studying failures. It would be expected that the same could be done for laboratory U-U triaxial tests as well, but the magnitude of the required correction would not necessarily be the same, as noted previously. Furthermore, the correction required for laboratory triaxial tests would depend on the amount of disturbance of the samples, which in turn depends on the area ratio of the sampler, the sampling procedure, and the amount of the sample which is trimmed away and discarded.

Conclusions

The conclusions drawn from study of the failure of the LASH terminal trench slope in San Francisco Bay may be summarized as follows:

1. The undrained shear strength measured in the laboratory tests was larger than the shear strength mobilized in the field when the trench slope failed. The systematic testing errors resulting from the effects of anisotropy, end restraint, plane strain, and sample disturbance cannot explain this discrepancy. An analysis indicates that the effects of these factors compensated each other to a large degree, and their combined effect was quite small.
2. The values of shear strength measured in field vane shear tests are larger than the values measured in the laboratory tests, and they, therefore, exceed the strength mobilized when the trench slope failed by a wider margin.
3. To reconcile the differences between the measured strength values and the strength mobilized in the field where the trench slope failed, it is necessary to consider the reduction of shearing resistance of the San Francisco bay mud caused by sustained loading. Laboratory tests indicate that for loads maintained 1 week or longer, the shearing resistance is only about 70% of the value measured in conventional short-term triaxial tests.
4. Whether laboratory tests or field vane shear tests are used to measure the undrained shear strengths of clays, the values measured should be corrected for the effects of systematic errors and the loss of shearing resistance under sustained loading. As shown by recent studies of the Norwegian Geotechnical Institute, summarized in Figs. 14 and 15, the values of the appropriate corrections can be determined by analysis of slope failures. The magnitude of the required correction for values of strength measured by field vane shear tests is largest for highly plastic clays, which are subject to the largest strength loss under sustained loading. For clays of low plasticity, the correction factor is smaller than unity, indicating that for these clays the amount of disturbance due to insertion of the vane dominates over the effects of sustained loading.
5. In principle, it should be possible to develop correction factors similar to those shown in Figs. 14 and 15 for use in cases where the shear strength is measured by laboratory triaxial tests. This would, however, require consideration of the sampling and testing procedures employed in the tests, because these control the amount of disturbance and the magnitude of the systematic errors. In practice, it is easier to determine the required correction factors for field vane shear tests, because the procedures employed do not vary as widely as for laboratory tests. Therefore, vane shear test results, approximately corrected for systematic errors and loss of strength under sustained loading,

probably provide a more reliable estimate of the undrained strength which can be mobilized in the field over a long period of time.

Acknowledgment

The writers wish to acknowledge the assistance of Ove Eide, Gunnar Aas, Kunt Andersen, and the late Laurits Bjerrum of the Norwegian Geotechnical Institute, Henry Taylor and Robert Lawson of Harding-Lawson Associates, Eugene Sembler and Charles Vickers of the San Francisco Port Commission, and H. B. Seed of the University of California who made many helpful suggestions and provided much useful information. Camden McConnel performed the sustained loading tests on San Francisco bay mud as a student project at the University of California and Lester Abney of Harding-Lawson Associates drafted the figures.

Appendix.—References

1. Aas, G., "Stabilitet og deformasjoner ved uavstivede byggegroper i leire," Oslo, Norway, Den norske igeniørforening. Kurs i utgraving og sikring av byggegroper, Kongsberg, 1971.
2. Bjerrum, L., "Effect of Rate of Strain on Undrained Shear Strength of Soft Clays," *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico City, Mexico, Aug., 1969.
3. Bjerrum, L., Clausen, C. J. F., and Duncan, J. M. "General Report on Stability of Flexible Structures," *Proceedings of the 5th European Conference on Soil Mechanics and Foundation Engineering*, Madrid, Spain, Apr., 1972.
4. Bjerrum, L., "Report on Embankments on Soft Ground," *Proceedings of the ASCE Conference on Performance of Earth and Earth-Supported Structures*, Purdue University, Lafayette, Ind., June, 1972.
5. Burmister, D. M., "Strain-Rate Behavior of Clay and Organic Soils," *Special Technical Publication No. 254*, American Society for Testing and Materials, Philadelphia, Pa., 1959.
6. Casagrande, A., and Wilson, S. D., "Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content," *Geotechnique*, London, England, June, 1951.
7. Casgrande, A., and Rivard, P. J., "Strength of Highly Plastic Clays," *Publication No. 31*, Norwegian Geotechnical Institute, 1959.
8. Casgrande, A., "An Unsolved Problem of Embankmant Stability on Soft Ground," *Proceedings of the 1st Panamerican Conference on Soil Mechanics and Foundation Engineering*, Mexico City, Mexico, Sept., 1959.
9. Coates, D. F., Burn, K. N., and McRostie, G. C., "Strain-Time-Strength Relationship in Marine Clay," *Transactions*, Engineering Institute of Canada, Vol. 6, No. A-11, Oct., 1963.
10. "Construction Debris Substitutes for Rockfill in Dike," *Engineering News-Record*, Feb. 17, 1966.
11. Duncan, J. M., and Seed, H. B., "Strength Variation Along Failure Surfaces in Clay," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 92, No. SM6 Proc. Paper 4971, Nov., 1966, pp. 81-104.
12. Duncan, J. M., and Dunlop, P., "The Significance of Cap and Base Restraint," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 94, No. SM1, Proc. Paper 5768, Jan., 1968, pp. 271-290.
13. Scott, R. F., and Ko, H-Y., "Stress Deformation and Strength Characteristics," State-of-the-Art Report, *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico City, Mexico, Aug., 1969.
14. Shibata, T., and Karube, D., "Creep Rate and Creep Strength of Clays," *Proceedings

of the 7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico City, Mexico, Aug. 1969.
15. Singh, A., and Mitchell, J. K., "Creep Potential and Creep Rupture of Soils," *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico City, Mexico, 1969.
16. Skempton, A. W., and Hutchinson, J., "Stability of Natural Slopes and Embankment Foundations," State-of-the-Art Report, *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico City, Mexico, Aug., 1969.
17. Walker, L. K., "Undrained Creep in Sensitive Clay," *Geotechnique*, London, England, Vol. 19, No. 4, 1969.
18. Wu, T. H., and Douglas, A. G., "Friction and Cohesion of Saturated Clays," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 88, No. SM3, Proc. Paper 3158, June, 1962, pp. 1-32.