SOIL MECHANICS AND BITUMINOUS MATERIALS RESEARCH LABORATORY

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REPORT NO. TE-68-2

TO

STATE OF CALIFORNIA DEPARTMENT OF WATER RESOURCES



DEPARTMENT OF CIVIL ENGINEERING
INSTITUTE OF TRANSPORTATION AND TRAFFIC ENGINEERING

University of California · Berkeley

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A Report of an Investigation

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# AN ANALYSIS OF THE SHEFFIELD DAM FAILURE

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H. Bolton Seed,[1] Kenneth L. Lee[2] and I. M. Idriss[3]

## Introduction

The failure of the Sheffield Dam in Santa Barbara, California during an earthquake on June 29, 1925 marks one of the very few recorded cases in which a catastrophic slide failure of an earth dam has resulted from earthquake effects. Accordingly it merits a detailed study of the circumstances involved for the purpose of:

(1) assessing the factors which led to the failure

(2) evaluating the ability of current design methods to predict a failure of this type

and (3) determining whether recent advances in analytical and laboratory test procedures for evaluating embankment stability during earthquakes could throw any light on the observed behavior.

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In view of the fact that it is now some forty years since the failure occurred it might well be questioned whether the circumstances involved could be established with sufficient accuracy to merit a detailed appraisal. Fortunately with the aid of information provided by engineers and seismologists who were in the area at the time, records of the failure made soon after the earthquake, a study of the soil conditions at the site conducted by the U. S. Army Corps of Engineers in 1949[4] and the fact that samples of the same type of soil as that used for construction of the dam can still be obtained at the site, it has been possible to reconstruct reasonably well the conditions existing at the time failure occurred. A description of these conditions and analyses of the embankment stability using pseudo-static and dynamic response approaches are presented in the following pages.

## Description of Dam

The Sheffield Dam was constructed in the winter of 1917 in a ravine north of the city of Santa Barbara. A representative section through the dam at its maximum height is shown in Fig. 1. The embankment, 720 ft long and with a maximum height of about 25 ft, was constructed of soil from the reservoir excavation and compacted by routing the construction equipment over the fill. The body of the dam was composed of silty sand and sandy silt containing some cobbles and boulders but the upstream slope was faced with a 4 ft thick clay blanket which was extended up to 10 ft into the foundation to serve as a cut-off wall. The clay blanket was overlain with a 5-inch concrete facing. No record of the degree of

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[4] U. S. Army Corps of Engineers, "Report on Investigation of Failure of Sheffield Dam, Santa Barbara," June, 1949.

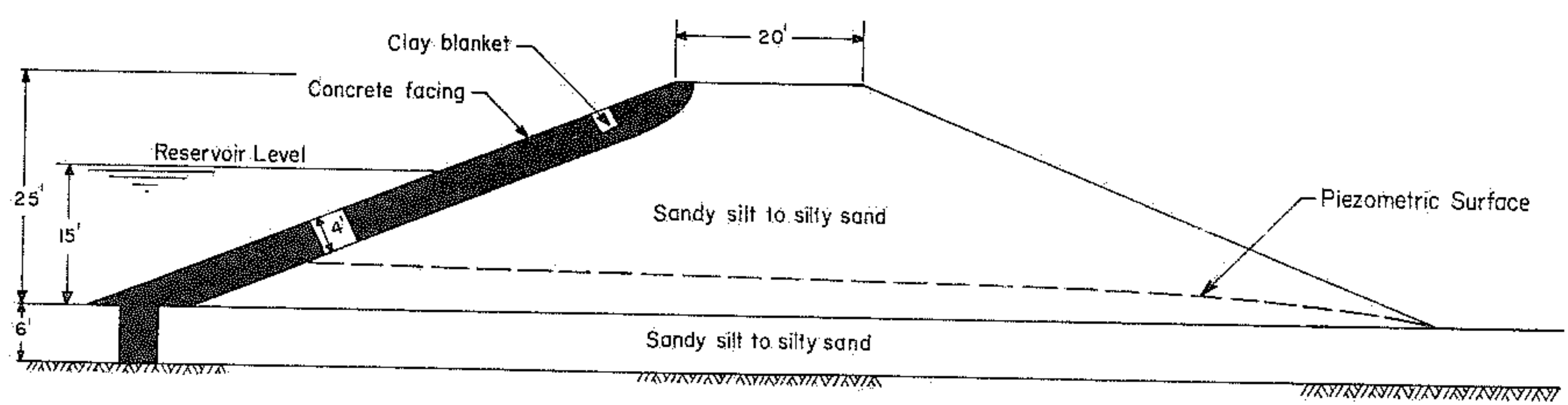


Fig. I - CROSS-SECTION THROUGH EMBANKMENT compaction of the embankment is available. A view of the dam with the reservoir empty is shown in Fig. 2.

The foundation soil consists of a layer of terrace alluvium, four to 10 ft thick, overlying sandstone bedrock. Drill holes made in 1949 by the Corps of Engineers showed the alluvium to consist mainly of silty sand and sandy silt containing cobbles varying from 3 to 6 inches in diameter and with some thin layers of clayey sand and gravelly sandy clay. The upper 1 to 1-1/2 ft of the foundation soil is somewhat looser than the underlying deposits and it has been fairly well established that there was no formal stripping of the upper soil layers prior to construction of the embankment.

It is reported that seepage had been noted near the toe of the downstream slope and in the area beyond the toe before the earthquake occurred. Notes in the files of the Santa Barbara Water Department state that examination after the failure indicated that there was no leakage of water through the upstream core but that seepage around and underneath the cut-off had saturated the lower part of the main structure. Willis[5] also expressed the opinion that "the foundation of the dam had become saturated by percolation." Thus it appears that the water level in the embankment was probably somewhat similar to that shown in Fig. 1. At the time of the earthquake the depth of water in the reservoir was about 15 to 18 ft.

## Santa Barbara Earthquake, 1925

The main shock of the earthquake occurred at 6:42 A.M. in the morning of June 29, 1925. There were no strong motion instruments in existence at

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[5]Willis, Bailey, "A Study of the Santa Barbara Earthquake of June 29, 1925. Bulletin of the Seismological Society of America, Vol. 15, No. 4, Dec. 1925, pp. 255-278.

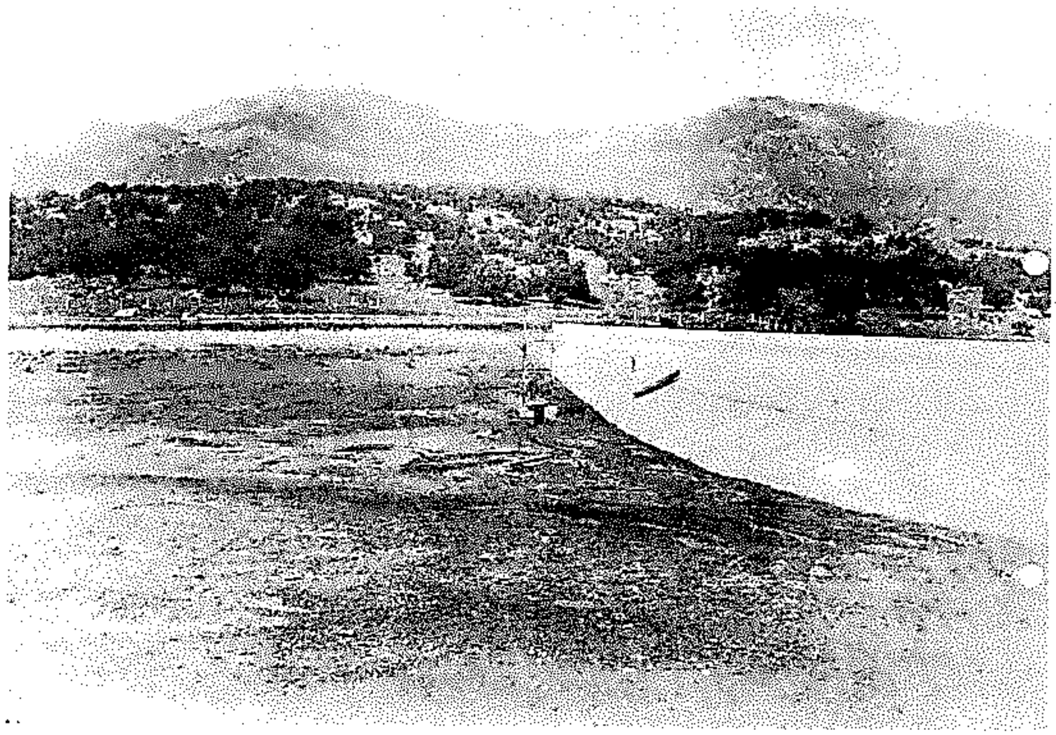

Fig. 2 – SHEFFIELD DAM BEFORE FAILURE

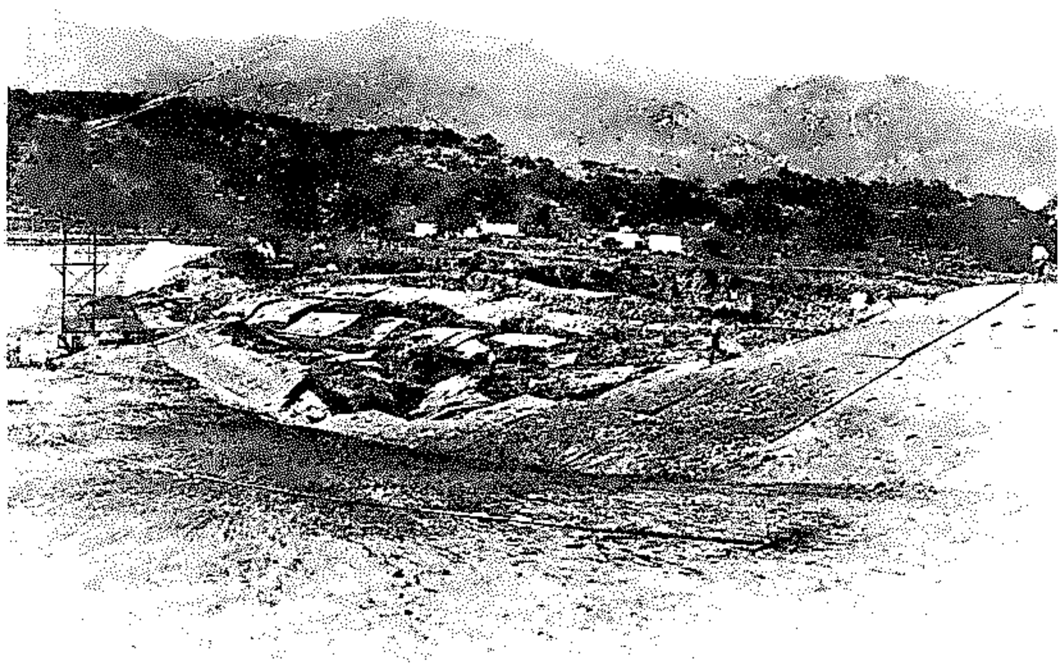

Fig. 3 – SHEFFIELD DAM AFTER FAILURE

the time but on the basis of records obtained at distant stations, the earthquake has been assigned a magnitude rating of 6.3 with an epicenter located some seven miles northwest of the dam-site.[6]

The fact that the earthquake occurred so near to a populated area resulted in considerable loss of life and property. Twelve people were killed and damage to semi-public and public structures was estimated at 6.25 million dollars. The shock was felt over an area of at least 50,000 square miles.

Early reports attributed the earthquake to movement along one of the many faults in the vicinity of Santa Barbara, some of which are quite close to the dam-site. However there was no evidence of horizontal or vertical displacement of the ground surface during the earthquake[5] and more recent studies fail to confirm the existence of a known active fault in the area which could have formed the source of the energy release.

The intensity of ground shaking in and around Santa Barbara was estimated in the usual manner, based on observed damage. Bailey Willis, Stanford University professor and President of the Seismological Society of America was lying awake in the Hotel Miramar, 4-1/2 miles east of Santa Barbara at the time of the earthquake. He observed that the initial jolt came from the west, and he was thrown sideways in that direction. He immediately began to count seconds. By his count the principal vibrations of the earthquake lasted 15 seconds. The initial shock was quickly followed by numerous after-shocks of lesser intensity, a total of 6 being counted within the first 20 minutes following the earthquake. He inspected the city, and assigned a maximum intensity of X on the Rossi-Forell scale.[5]

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[6] Eppley, R. A., "Earthquake History of the United States, Part II," U. S. Government Printing Office, Washington, D. C., No. 41-1, Revised 1960.

It should be noted however that the main business district where the buildings, and hence the damage, were concentrated was on low filled land which probably contributed to the earthquake damage. The Sheffield Dam was situated on firmer and higher ground where a somewhat lower intensity rating would be anticipated. For example, Willis states that the San Ysidro Hotel, which was also on high ground some 4 miles east of the dam suffered only minor cracks, and did not even lose its chimney.

Early in November, some 4 months after the earthquake, Professor Perry Byerley of the University of California made an inspection trip through the entire area affected by the earthquake and assigned a Rossi Forell intensity rating to each town which he visited.[7] From these data the intensity at the dam-site can be interpolated to be between Rossi-Forell VIII and IX.

Accepting these figures as providing a reliable assessment of the intensity of ground shaking, empirical correlations between maximum ground accelerations and Rossi-Forell intensity ratings indicate a maximum ground acceleration in the vicinity of the Sheffield Dam of about 0.15g. Maximum ground accelerations of a similar order of magnitude would also be indicated by computations based on magnitude and epicentral distance, and they are also in accord with values which might be deduced from a comparison with somewhat similar earthquakes, such as the San Francisco earthquake of 1957. These comparisons would also indicate a predominant frequency in the motions at the ground surface of about 3 cycles per second.

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[7] Byerly, Perry, "Notes on the Intensity of the Santa Barbara Earthquake between Santa Barbara and San Luis Obispo," Bulletin of the Seismological Society of America, Vol. 15, No. 4, December 1955, pp. 279-281.

Thus it would appear that the earthquake ground motions at the Sheffield Dam site might be approximated as follows:

| | |
|---|---|
| Maximum ground acceleration | 0.15g |
| Duration of significant shaking | 15 to 18 seconds |
| Predominant frequency of accelerations | 3 cycles per second |

The time history of such a ground motion might be approximated by appropriate scaling of the accelerograph record of the 1940 El Centro earthquake; this record was obtained about 7 miles from the epicenter of an earthquake of magnitude 7.0 at a site underlain by about 100 feet of stiff clay. The ground motions at the Sheffield Dam site might be expected to be somewhat similar in form, but with smaller amplitudes, a slightly higher predominant frequency and a smaller duration of shaking. Thus they might be represented by scaling the ordinates of the El Centro record to a maximum acceleration of 0.15g, scaling the abscissa to a predominant frequency of about 3 cycles per second, and continuing the record for only about 15 seconds. Such a time-history of ground accelerations is shown in Fig. 4. It is believed that a ground motion of this type provides a reasonable basis for analyzing the behavior of the Sheffield Dam.

## Field Data Concerning the Dam Failure

The Sheffield Reservoir formed by the dam was about 800 ft square and was capable of impounding a maximum of about 45 million gallons of water. The reservoir was not essential to the city of Santa Barbara; it could be by-passed by a water-main leading from a larger and more distant storage area behind the Gibralter Dam. At the time of the earthquake, the depth of water in the reservoir was only about 15 to 18 ft, so that the failure released about 30 million gallons of water which temporarily flooded the

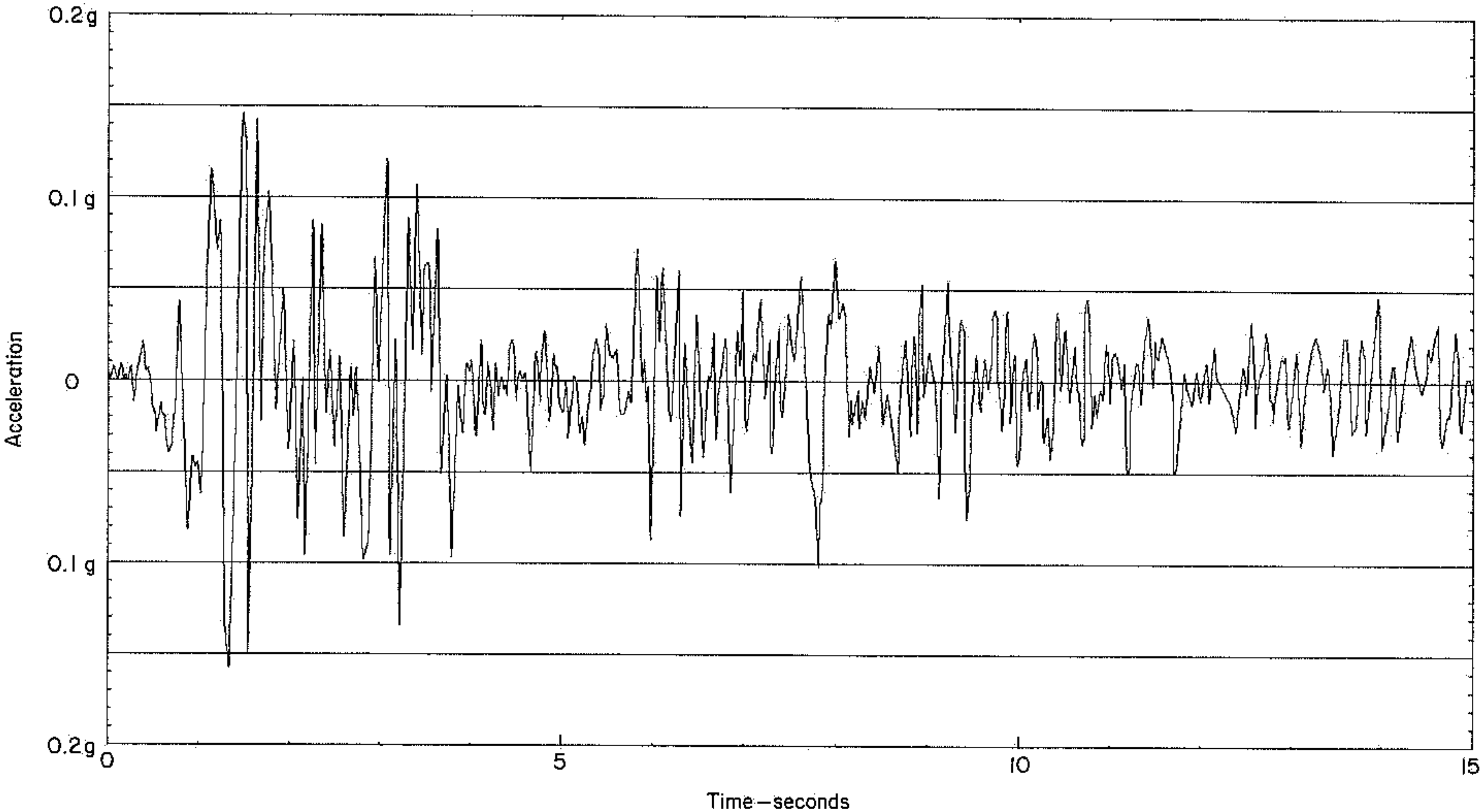


Fig.4 – HYPOTHETICAL GROUND MOTION RECORD FOR SANTA BARBARA EARTHQUAKE (1925)

lower part of the city to a depth of about 1 or 2 ft before discharging into the sea.

Unfortunately there were no eye-witnesses when the failure occurred. However after inspecting the damage M. M. O'Shaughnessy, City Engineer of San Francisco, reported[8] that "a great mass of the center, about 300 ft in length, slid downstream perhaps 100 ft." Herbert Nunn, City Manager of the City of Santa Barbara, wrote:[9] "After examination by several prominent engineers, the conclusion has been reached that the base of the dam had become saturated, and that the shock of the earthquake....had opened vertical fissures from base to top; the water rushing through these fissures simply floated the dam out in sections." Willis reported[5] "The foundations of the dam had become saturated and the rise of the water as the ground was shaken formed a liquid layer of sand under the dam, on which it floated out, swinging about as if on a hinge." A view of the embankment after failure is shown in Fig. 3.

From these accounts it would seem reasonable to conclude that sliding occurred on a surface near the base of the embankment, causing a large portion of the dam to move a considerable distance downstream, breaking up as it did so to give the general appearance shown in Fig. 3. Undoubtedly this sliding was related in some manner to a severe reduction in soil strength resulting from increases in pore-water pressure induced by the earthquake shaking.

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[8] O'Shaughnessy, M. M., Letter to the Editor of the Engineering News Record, July 9, 1925.

[9] Nunn, Herbert, "Municipal Problems of Santa Barbara," Bulletin of the Seismological Society of America, Vol. 15, No. 4, December 1925, pp. 308-319.

Properties of Foundation and Embankment Soils

In 1949, the U. S. Army Corps of Engineers conducted a field investigation to determine the nature of the soils forming the foundation and embankment for the Sheffield Dam.[4] Since it appeared that the major portion of the original dam and its foundation had been removed during reconstruction operations, the exploratory work was conducted in areas immediately adjacent to and downstream of the site to obtain, as far as possible, samples of material similar to that which composed the original foundation and embankment. Five holes, each 16 inches in diameter were drilled and samples were taken at frequent intervals.

The foundation conditions adjacent to the old dam-site consisted mainly of a silty sand and sandy silt containing cobbles varying from 3 to 6 inches in diameter. Grain size analyses of the silty sand and sandy silt samples showed the percent coarser than 0.02 mm varying from about 40 to 60 percent. Samples containing the higher portions of sand sizes were non-plastic; for samples containing the higher proportions of silt sizes, the liquid limit was typically about 24 and the plasticity index about 4. Near the center section of the dam the thickness of the foundation material was only 5 or 6 ft. However the upper 1 to 1.5 ft of this soil was substantially looser than the deeper material. Near the surface the soil had an average dry density of about 90 lb per cu ft whereas below this the dry density was about 101 lb per cu ft. The maximum dry density of the soil in a Standard AASHO Compaction Test is about 118 lb per cu ft. Thus it would appear that prior to construction, the upper layer of foundation material had a degree of compaction of about 76 percent based on the Standard AASHO test procedure. This density also corresponds to a relative density of about 35 to 40 percent for this soil.

(1) A series of isotropically consolidated-undrained (IC-U) tests, with pore-pressure measurements, on samples saturated prior to testing. These tests were performed at confining pressures ranging from 0.2 to 2.0 kg per sq cm. In general the stress-strain relationships showed peak values of the deviator stress and effective principal stress ratio at strains of about 4 to 5 percent with slight reductions in these strength characteristics occurring at higher strains. Values of the pore pressure coefficient $\bar{A}$ at failure varied from about 0.5 for tests conducted using a confining pressure of 0.5 kg per sq cm to about 1.25 for tests with confining pressures between 1 and 2 kg per sq cm.

The effective stress conditions at failure, plotted in the form of half the principal stress difference versus half the sum of the major and minor principal stresses at failure, are shown in Fig. 5. From this plot the strength parameters of the soil in terms of effective stresses are found to be $c' = 0$ and $\phi' = 34.5°$.

The total stress conditions at failure in this test series are plotted in Fig. 6a, from which it may be seen that the envelope of failure is characterized by the parameters $c = 0.1$ kg per sq cm and $\phi = 13°$.

(2) A series of IC-U tests on samples having a degree of saturation of about 85%. The total stress conditions at failure in this test series are shown in Fig. 6b. It is of interest to note that this is essentially the same condition as that of some of the undisturbed samples of foundation soil tested by the Corps

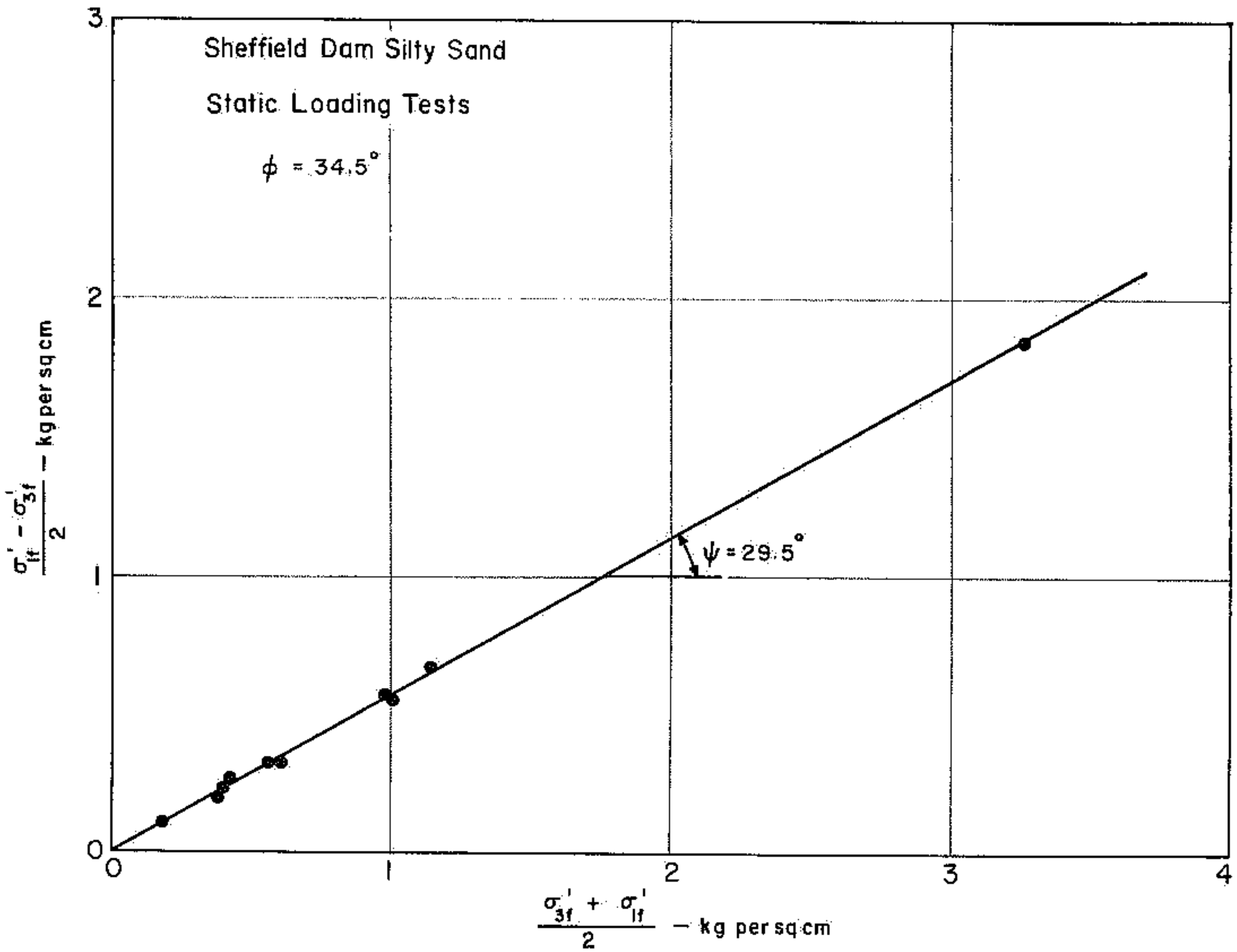


Fig.5 – EFFECTIVE STRESS CONDITIONS AT FAILURE – STATIC LOADING CONDITIONS.

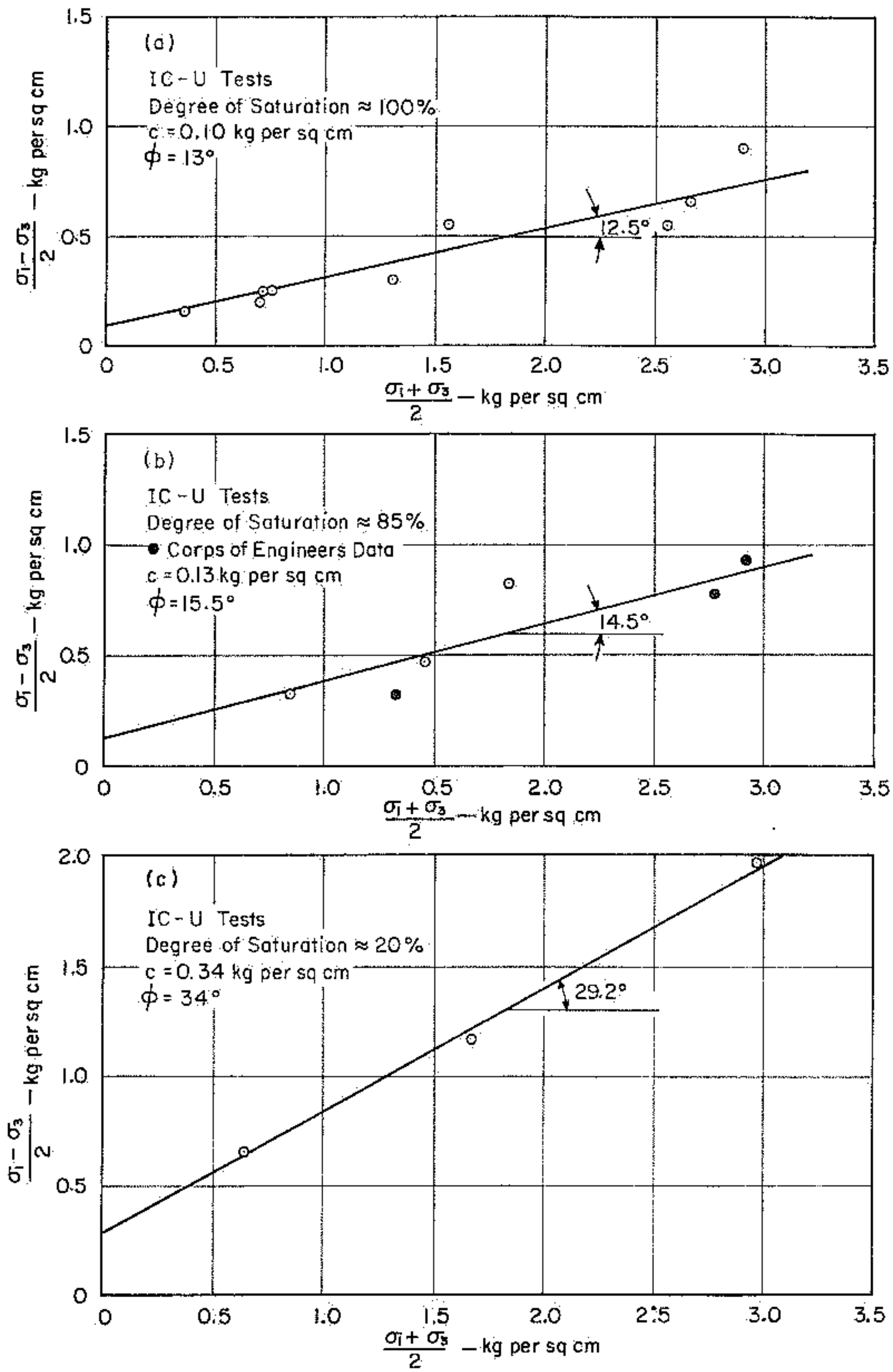


Fig. 6 – STRESS CONDITIONS AT FAILURE IN CONSOLIDATED–UNDRAINED TRIAXIAL COMPRESSION TESTS – STATIC LOADING CONDITIONS.

of Engineers and the data obtained from these tests are plotted in Fig. 6b together with that from the present investigation. The reasonably good agreement between the two sets of data gives added confidence in the data obtained from compacted samples in the present investigation. For samples having a degree of saturation of about 85 percent, the Mohr envelope of failure was found to be represented by the parameters c = 0.13 kg per sq cm and $\phi$ = 15.5°.

(3) A series of IC-U tests on samples having a degree of saturation of about 20 percent. The total stress conditions at failure in this test series are shown in Fig. 6c, giving a Mohr envelope defined by the strength parameters c = 0.34 kg per sq cm and $\phi$ = 34°.

The strength parameters in terms of total stresses for the tests at the various degrees of saturation are summarized in Table 1.

Table 1

Total Stress Strength Parameters Determined by Isotropically Consolidated-Undrained Triaxial Compression Tests Under Static Loading Conditions

| Degree of Saturation (percent) | c (kg per sq cm) | $\phi$ (degrees) |
|---|---|---|
| 20 | 0.34 | 34 |
| 85 | 0.13 | 15.5 |
| 100 | 0.10 | 13 |

The preceding data and the known facts concerning the conditions at failure provide a basis for analyzing the stability of the Sheffield Dam using conventional design procedures. A cross-section through the embankment before the earthquake occurred is shown in Fig. 1. The soil below the ground water level in the dam can be presumed to be saturated with strength characteristics as listed above. If this soil had an initial dry density of about 90 lb per cu ft and some allowance is made for compression under the weight of overlying material, the saturated unit weight would be of the order of 120 lb per cu ft. To simplify the analyses it will be assumed that these soil characteristics are applicable down to the rock surface though in fact the soil is somewhat denser and stronger in the bottom 4 ft.

Measurements in the existing embankment indicate that the soil in the main body of the dam has an average degree of saturation of about 50 percent and it has therefore been considered reasonable to assume that the soil above the water level in the original embankment was in a similar condition. By interpolation in the data listed in Table 1, the total stress strength parameters for a degree of saturation of 50 percent would be $c = 0.25$ kg per sq cm and $\phi = 26°$. The unit weight corresponding to the initial conditions discussed above would be about 107 lb per cu ft.

Stability analyses based on these soil characteristics are presented in the following pages.

## Stability of Downstream Slope Before Earthquake

An analysis of the stability of the downstream slope (the dam failed by a slide in the downstream direction) for the conditions existing before the earthquake would appropriately be made on an effective stress basis.

Such an analysis would typically be made for a slip surface represented by the arc of a circle using the conventional method of slices. Following this procedure for the cross-section shown in Fig. 1 and using effective stress strength parameters $c' = 0$ and $\phi' = 34.5°$ leads to a position for the most critical sliding surface as shown in Fig. 7, and a computed factor of safety of 1.68. On this basis it would appear that the slope was amply stable for the conditions existing before the earthquake occurred.

Conventional Pseudo-Static Procedures for Analyzing the Seismic Stability of Embankments

Past practice and most current practice in the analysis of embankment stability against earthquake forces involves the computation of the minimum factor of safety against sliding when a static horizontal force, intended to represent the disturbing effect of the earthquake, is included in the analysis. The analysis is treated as a static problem and the horizontal force is expressed as the product of a seismic coefficient, k, and the weight, W, of the potential sliding mass. If the factor of safety approaches unity the section is generally considered unsafe, though there is no generally recognized limit for the minimum acceptable factor of safety.

In using a conventional pseudo-static approach of this type, the seismic coefficient adopted is often in the range 0.05 to 0.15. For many earth dams a value of 0.1 has been used. Since the earthquake at Santa Barbara was by no means as strong as the largest recorded or anticipated in the Western United States, its equivalent effects would presumably correspond to some intermediate value in the range of seismic coefficients used for design, such as, say, 0.1. This value might therefore be considered a reasonable value to use for evaluating the stability of the Sheffield Dam during the 1925 Santa Barbara earthquake using conventional analysis procedures.

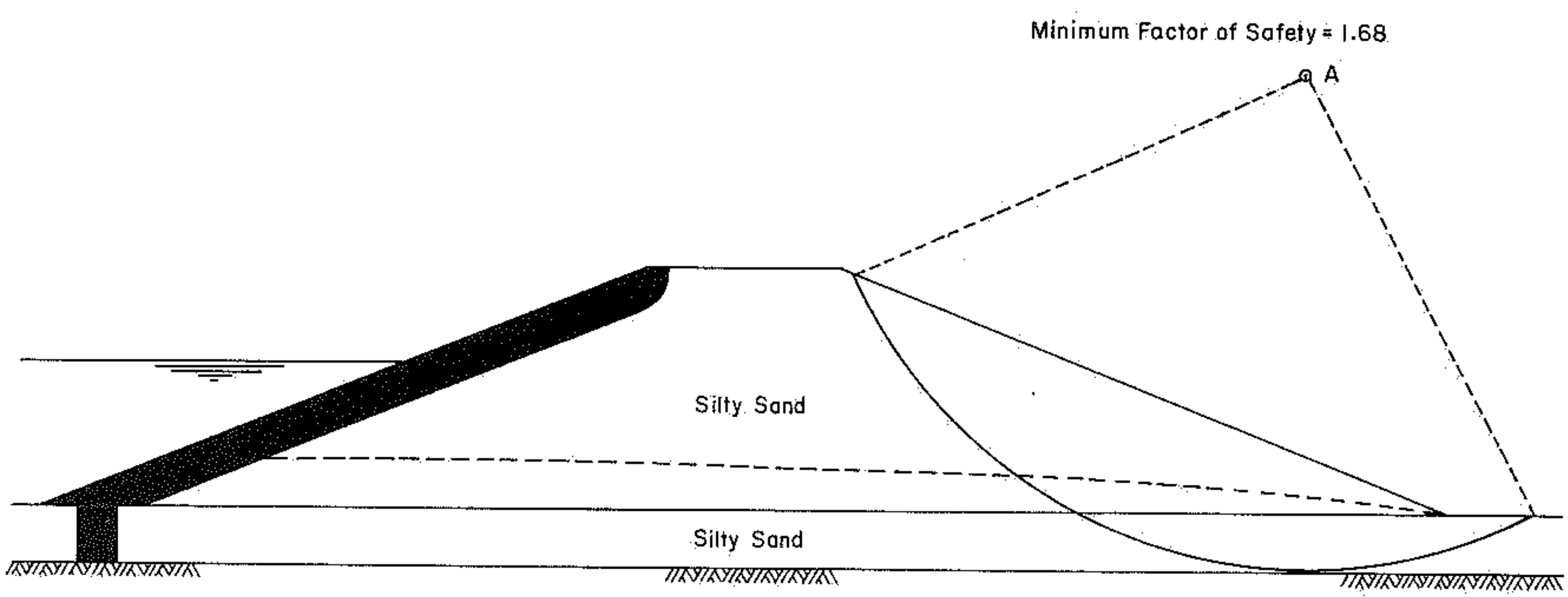


Fig. 7 – ANALYSIS OF EMBANKMENT STABILITY BEFORE EARTHQUAKE

Unfortunately, even in using this conventional procedure with a given value of the seismic coefficient, wide variations exist among design engineers regarding the choice of analytical details, which can have a significant effect on the computed value of the factor of safety. For example in incorporating a given seismic coefficient into an analysis made by the method of slices, it has been found that different engineers make different choices concerning the following:

(1) The use of strength values for the soil determined by drained tests or consolidated-undrained tests. In fact for many dams, there is virtually no possibility of drainage during the application of disturbing forces due to earthquakes and the soil is therefore loaded under essentially undrained conditions. Never-the-less some engineers use drained strengths for analysis even under conditions where no drainage could conceivably occur. Where drainage can readily occur, even in the short loading period of an earthquake, the use of drained strengths is clearly appropriate.

(2) The use of strength characteristics determined by triaxial compression tests or plane strain tests. Most engineers have used triaxial test data for analysis in the past but in recent years plane strain data has been used on several projects.

(3) The use of soil strength parameters determined by Mohr envelopes of failure or a modified form of the consolidated-undrained compression test data expressed by the relationship between the shear stress on the failure plane at failure and the normal stress on the failure plane at the time of consolidation, $\tau_{ff}$ vs

$\sigma_{fc}$, as proposed by Lowe and Karafiath.[10] The difference between the Mohr envelope and the $\tau_{ff}$ vs $\sigma_{fc}$ relationship for the consolidated-undrained tests performed on saturated samples of the Sheffield Dam silty sand is illustrated in Fig. 8. While the difference between the two relationships in this case is small it tends to become larger with increasing steepness of the Mohr envelopes and can often reach very significant properties.

Most designers have used the strength parameters defining the Mohr envelope for analysis purposes. However since present technology does not provide a means for determining the pore water pressures in the embankment during the earthquake and an effective stress analysis is thereby prohibited, the only known values of effective stresses are those existing before the earthquake force was applied. Thus it is more logical to relate the measured strengths ($\tau_{ff}$) to the normal stresses on the failure plane at the time of consolidation for design purposes.

(4) The use of test data obtained by consolidated-undrained tests on samples which have been initially consolidated under isotropic stress conditions (IC-U tests) or under anisotropic stress conditions, (AC-U tests). Most designers have used the more conventional IC-U tests for analysis. However soil elements in the field are invariably subjected to major and minor principal stresses having different magnitudes and the use of anisotropic stress conditions during consolidation provides a more accurate

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[10] Lowe, J. and Karafiath, L., "Stability of Earth Dams Upon Drawdown," Proceedings, 1st Pan-American Conference on Soil Mechanics and Foundation Engineering, Mexico City, Mexico, 1959.

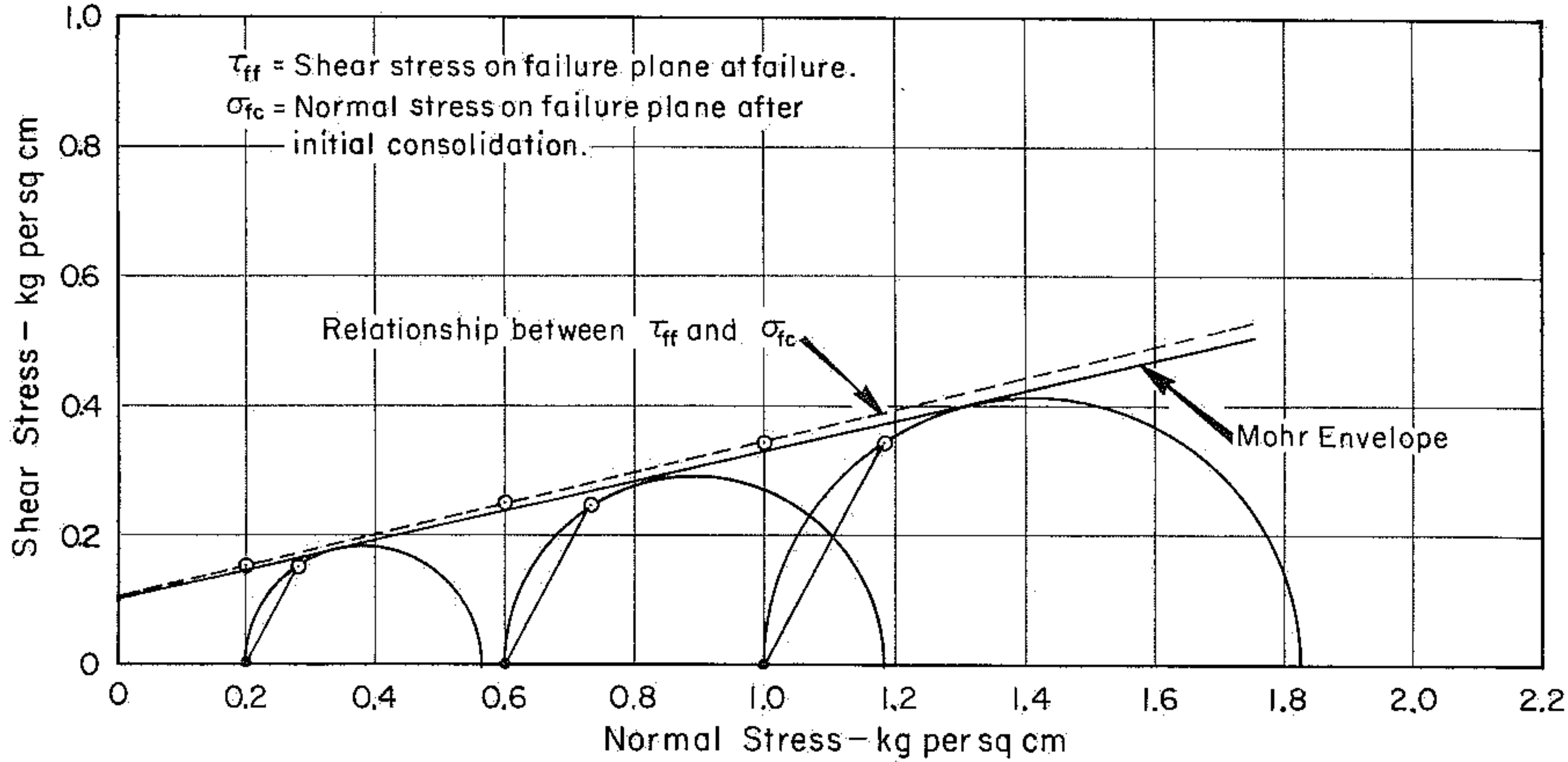


Fig. 8–MOHR ENVELOPE AND $\tau_{ff}$ vs $\sigma_{fc}$ RELATIONSHIP DETERMINED BY CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS ON SATURATED SAMPLES.

representation of field conditions. In consequence some engineers have adopted this procedure in recent years.

(5) The direction of the contact forces between slices in the analytical procedure. Most engineers using a circular arc representation of the sliding surface use the conventional method of slices and ignore the effects of forces on the sides of slices in the computations. However more fundamental analyses have shown that appropriate consideration of these forces can have a significant influence on the computed factor of safety.[11]

(6) The point of application of the seismic forces. In using the method of slices it would be logical to apply the seismic forces at the centroid of each slice. However this introduces some difficulty in resolving the forces at the base of the slice to determine the normal pressure. In consequence it is often assumed, in order to simplify the computations, that the seismic force on each slice also acts at the base of the slice.

(7) The effect of the seismic force on the resisting forces. While the main purpose of the inclusion of a seismic force is to increase the sliding tendency of a potential slide mass, the application of this force also affects the pressures on the potential sliding surface and thus the forces acting to resist sliding. Some engineers prefer to ignore the effect of the seismic force on the resisting forces, while others take this effect into account.

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[11]Bishop, A. W., "The Use of the Slip Circle in the Stability Analyses of Slopes," Geotechnique, Vol. 5, No. 1, March, 1954.

(8) The position of the critical sliding surface. The purpose of a stability analysis is to determine the minimum factor of safety corresponding to the most critical position of the potential sliding surface. This involves a number of trials to determine the critical position of the potential sliding surface when the inertia force is applied. To simplify the computations some engineers assume that the critical sliding surface during the earthquake will be in the same position as the critical sliding surface for the conditions acting before the earthquake. By this means it is only necessary to compute the factor of safety for one potential slide mass when the seismic force is included.

The fact that all of these possibilities exist and are subscribed to by some engineers means that, in fact, there are $2^8$ or 256 possible ways of analyzing the seismic stability of an embankment by a pseudo-static analysis incorporating a given seismic coefficient.

With regard to some of the factors listed above, such as the question of using drained or undrained strengths, the appropriate choice will depend on the permeability of the soils and the geometry of the section. However with regard to many of the factors there is a clear choice of the alternatives involved which leads to improved accuracy in the computational procedure. The preferred choices with regard to each of the computational alternatives, in order to improve the accuracy of the analysis, are shown in Table 2. Also shown in this table, in the column headed 'Most Conventional Choice' are the computational details which appear to be most widely used in engineering practice at the present time. It is interesting to note that for five of the factors listed in Table 2, the conventional

Table 2

| Factor to be Considered | Preferred Choice for Computational Accuracy | Most Conventional Choice | Effect of Using Most Conventional Choice on Analytical Results |
|---|---|---|---|
| Drained or undrained test data | Depends on permeability | Varies | – |
| Triaxial or plane strain tests | Usually plane strain | Triaxial | Conservative |
| Mohr envelope of failure or $\tau_{ff}$ vs $\sigma_{fc}$ | $\tau_{ff}$ vs $\sigma_{fc}$ | Mohr envelope | Conservative |
| Isotropic or anisotropic consolidation | Anisotropic consolidation | Isotropic consolidation | Conservative |
| Influence of forces on sides of slices | Make considered choice of direction | Ignore effects | Conservative |
| Points of application of seismic forces | At centroids of slices | At base of each slice | Conservative |
| Consider seismic force in determining overturning moments and/or resisting moments | Include in both overturning moments and resisting moments | Varies | – |
| Use critical sliding surface with seismic force included or before seismic force included | Critical surface with seismic force included | Varies | Unconservative to use critical surface before seismic force included |

deviations from the preferred procedures all tend to introduce increased conservatism into the analysis.

Thus if the seismic coefficient were correctly evaluated for a pseudo-static analysis, the appropriate type of static strength tests were performed and the critical sliding surface for seismic conditions were determined, a typical conventional analysis would give a conservative result (too low a value for the factor of safety) for one or more of the following reasons:

(1) Strength parameters determined by a Mohr envelope of failure are used for the analysis rather than the more appropriate strength relationship $\tau_{ff}$ vs $\sigma_{fc}$.

(2) The influence of forces on the sides of slices is not considered in the computations.

(3) Soil strengths are based on the results of tests on isotropically-consolidated samples.

(4) Soil strengths are determined by triaxial compression tests rather than plane strain tests.

(5) The seismic force is assumed to act at the base of each slice rather than at the centroid.

Thus if a design were based on a criterion that the computed factor of safety should be 1.1 for a seismic coefficient $k = 0.1$, the actual factor of safety, assuming that other conditions (such as the selection of the value of k and the test conditions for strength evaluation) are correct, would probably be considerably higher.

One of the main difficulties with this situation is that if an engineer uses the same design criterion but incorporates the more computationally accurate choices of analytical details in the study, the resulting embankment could, in fact, be in a more vulnerable condition.

In fact, however, the results of the computations may still shed little light on the probable behavior of the embankment during a major earthquake. Although the analysis would be conservative in some respects, it might well be unconservative in various other respects, for the following reasons:

(1) During a major earthquake the soils are subjected to cyclic loading conditions rather than static loading and the soil strength under these conditions may be substantially lower than that measured under any type of static loading conditions.[12]

(2) The seismic force developed during a major earthquake may well be substantially greater[13] than that corresponding to a seismic coefficient in the range 0.05 to 0.15.

(3) The critical sliding surface during an earthquake may be quite different from that showing the lowest computed factor of safety before the earthquake; the factors of safety against sliding on these surfaces when the seismic force is included in the computations may also be quite different.

Although it is possible to make qualitative evaluations of the effects of these computational details on the results of an analysis, there are invariably too many deviations from realistic conditions in a conventional pseudo-static analysis of seismic stability to enable the engineer to make a realistic assessment of whether the final result is likely to be conservative or not--except by a comparison of the computed factor of safety, using

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[12] Lee, Kenneth L. and Seed, H. Bolton, "Dynamic Strength of Anisotropically Consolidated Sand," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 93, No. SM5, September 1967.

[13] Seed, H. Bolton and Martin, G. R., "The Seismic Coefficient in Earth Dam Design," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 92, No. SM3, May, 1966.

a prescribed procedure, with the observed performance of embankments which fail when subjected to major earthquake ground shaking. It would, of course, be necessary to make the analytical computations for the conditions existing in the embankment at the time of the earthquake and also take into account the nature of the ground movements. It is unfortunate that there have not been more studies of this type conducted to provide a basis for evaluating the merits of pseudo-static analytical approaches.

Analysis of the Stability of the Downstream Slope of the Sheffield Dam During the Earthquake Using Pseudo-Static Analysis Procedures

To determine the applicability of pseudo-static approaches for predicting the performance of the Sheffield Dam in 1925, analyses were made using the more conventional choices of analytical details, where such a choice existed, and using the most conservative choice with regard to other details. Thus the analysis was based on the following assumptions:

(1) The sliding surface may be represented by the arc of a circle.

(2) Soil strengths correspond to those determined by isotropically consolidated-undrained triaxial compression tests in the laboratory.

(3) Soil strengths in the embankment may be determined from the strength parameters representing the Mohr envelopes of failure for the laboratory tests.

(4) The factor of safety is determined by a comparison of overturning moments and resisting moments and no consideration need be given to the effects of side forces on slices in making the computations.

(5) The seismic force increases the overturning moment but has no influence on the resisting moment.

(6) The critical sliding surface is that for which the computed factor of safety is a minimum when the seismic force is included in the analysis.

Computations were made both for the seismic force acting at the base of each slice and at the centroid of each slice. Within the framework of conventional procedures, this combination of computational details should lead to about the lowest computed values of the factor of safety for a given value of the seismic coefficient, k.

For convenience, and as an added element of conservatism, it was also assumed that the foundation soil to a depth of 6 ft below the original ground surface in the section shown in Fig. 1, had the same strength as that in the upper 1.5 ft. Thus, in accordance with the data presented in the section on Properties of Foundation and Embankment Soils, the unit weights and strength parameters used in the analyses were as follows:

| | Unit Weight lb/cu. ft. | Strength Parameters c-kg per sq cm | Strength Parameters $\phi$-degrees |
|---|---|---|---|
| All soil above water table | 107 | 0.25 | 26 |
| All soil below water table | 120 | 0.1 | 13 |

The results of analyses made using a seismic coefficient of 0.1 are summarized in Fig. 9. The critical surface of sliding was found to be the arc of a circle having a center at B and tangent to the base of the foundation soil layer. For this circle the computed factor of safety was 1.21 if the seismic force was assumed to act at the base of each slice or 1.32 if the seismic force was assumed to act at the centroid of each slice.

It is interesting to note (see Fig. 10) the difference in computed factor of safety if the analysis is made for the surface of sliding

| Design Assumptions | Factor of Safety |
|---|---|
| Seismic force acting at base of slice | 1.21 |
| Seismic force acting at C.G. of slice | 1.32 |

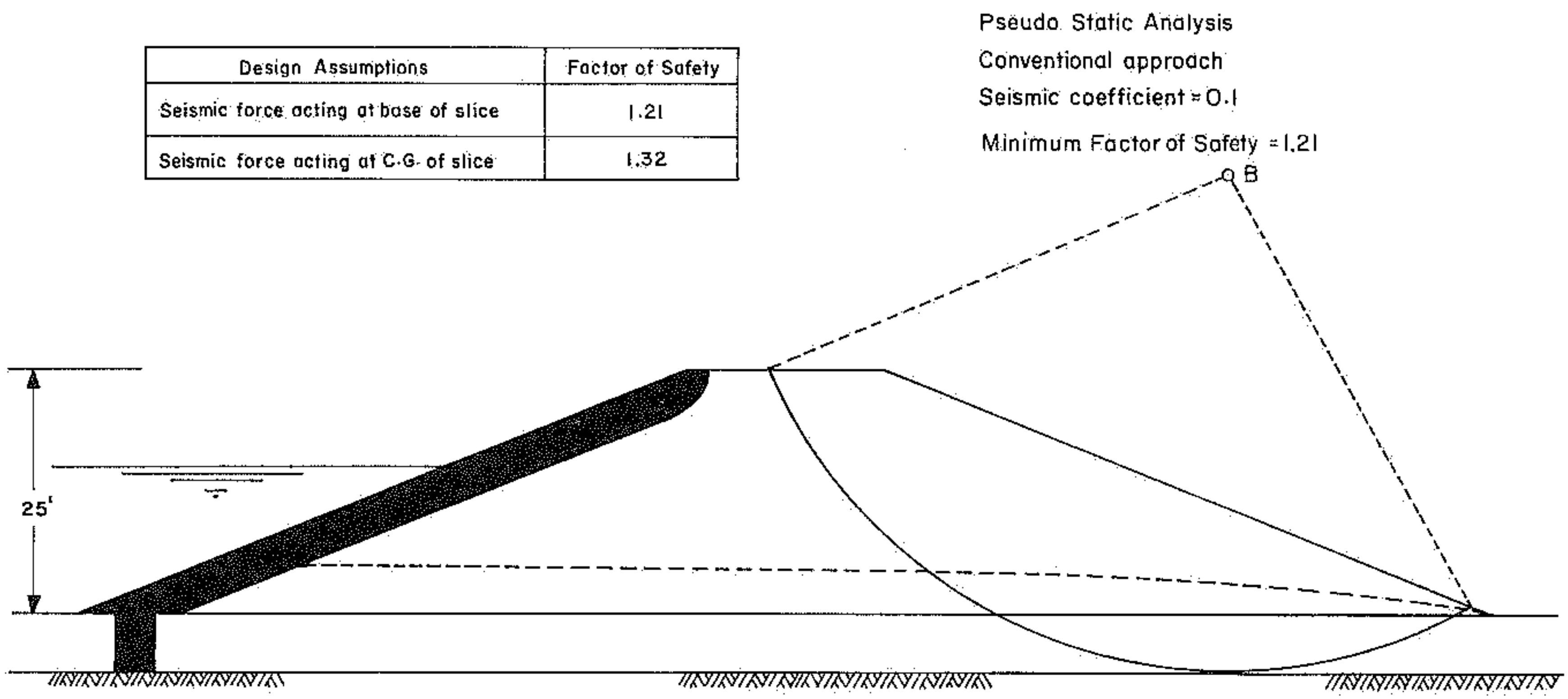


Fig. 9 — PSEUDO STATIC ANALYSIS OF EMBANKMENT STABILITY DURING EARTHQUAKE

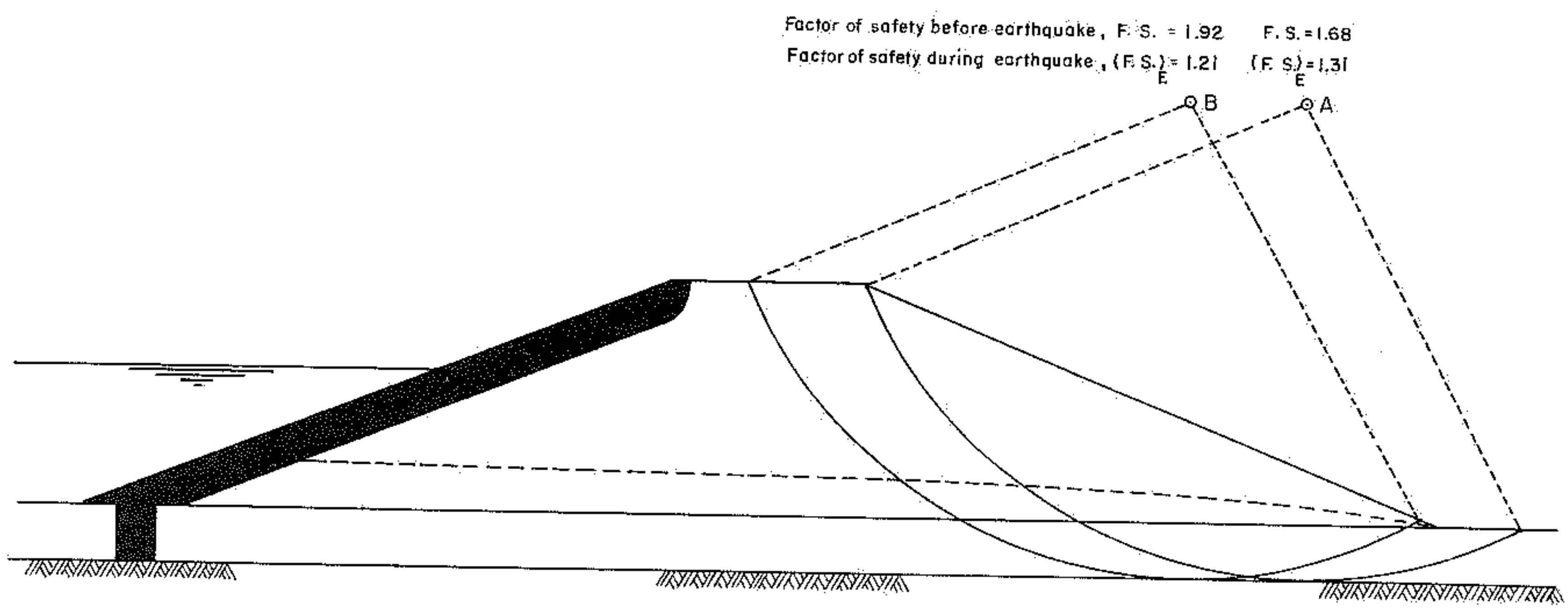


Fig. 10–COMPARISON OF COMPUTED FACTORS OF SAFETY.

exhibiting the lowest factor of safety for the conditions existing before the earthquake (center at point A in Figs. 7 and 10). For this surface, the computed factor of safety before the earthquake was 1.68 and the inclusion of the seismic force, represented by a seismic coefficient of 0.1, reduced the value to 1.31. For the critical circle with seismic forces included (center at B in Figs. 9 and 10) the factor of safety before the earthquake was about 1.9 but this value dropped to 1.21 for a seismic coefficient of 0.1. Thus it would appear to be important, in this case at least, to determine the critical surface of sliding for the assumed seismic conditions.

Clearly by using a somewhat higher value of the seismic coefficient, the computed factor of safety could have been reduced to 1.0. As indicated in Fig. 11, a seismic coefficient of 0.17 would have led to this result.

It should also be recognized however that although the strength parameters and water level conditions for which the preceding analyses were made are believed to represent reasonable average values for the embankment, their evaluation never-the-less involves some degree of judgment, and relatively small changes in these factors could also reduce considerably the computed factors of safety. For example, if the strength test data in Fig. 6 were interpreted somewhat conservatively to give strength parameters $c = 0.075$ kg/cm$^2$ and $\phi = 13.5°$, the computed factor of safety for the critical sliding surface would have been 1.14. If, in addition, the water table elevation in the embankment had been considered to be several feet higher than that shown in Fig. 9, the computed factor of safety could have been only 1.05. The assumption of a rather high but not impossible water table elevation in the embankment would have reduced the factor of safety to unity. In view of these results it would appear

PSEUDO-STATIC ANALYSES - CONVENTIONAL APPROACH

$k_f$ = Seismic coefficient for which factor of safety along sliding surface is unity.

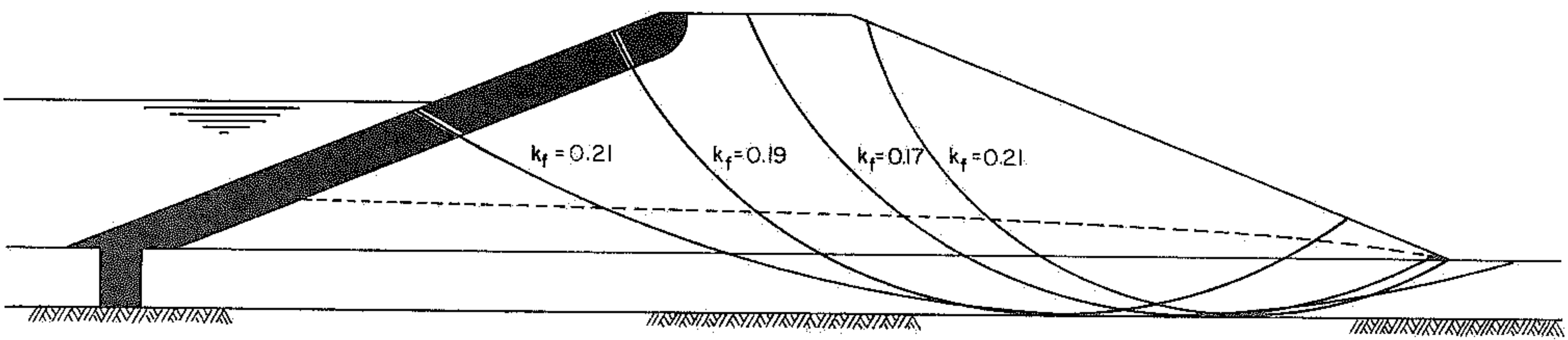


Fig.11 - SEISMIC COEFFICIENTS INDICATING FAILURE IN PSEUDO-STATIC ANALYSES.

reasonable to conclude that the pseudo-static analysis leads to a computed factor of safety for the critical circle in the range of 1.0 to 1.2 with the more probable value near the upper end of this range.

The computed position of the critical sliding surface shown in Fig. 9 bears little resemblance to that on which failure was adjudged to have taken place. In fact, if failure had occurred on the critical sliding surface shown in Fig. 9, the configuration of the embankment after failure would presumably have been similar to that shown in Fig. 12. It is apparent, since there was only about 15 ft of water in the reservoir at the time of the earthquake, that this would not have led to loss of water from the reservoir or even a loss of freeboard. In fact, the entire section of the embankment and foundation to the right of the lower dashed line shown in Fig. 12 could presumably have slid away and still have left a sufficient quantity of embankment standing to form an adequate dam for the water in the reservoir. Thus it is clear that this method of analysis gives an erroneous picture of the position of the critical sliding surface, regardless of the numerical value of the factor of safety associated with it.

Computations can readily be made to investigate the factor of safety for a seismic coefficient of 0.1 or alternatively, the seismic coefficient for which the computed factor of safety would be unity, for a surface of sliding corresponding to the approximate position of that which apparently developed in the Sheffield Dam. In order for water to escape from the reservoir the surface of sliding must presumably have intersected the upstream face below the level of the water in the reservoir. Certainly this was the impression of the engineers who visited the site after the failure occurred. For the water level conditions shown in Fig. 1 and the

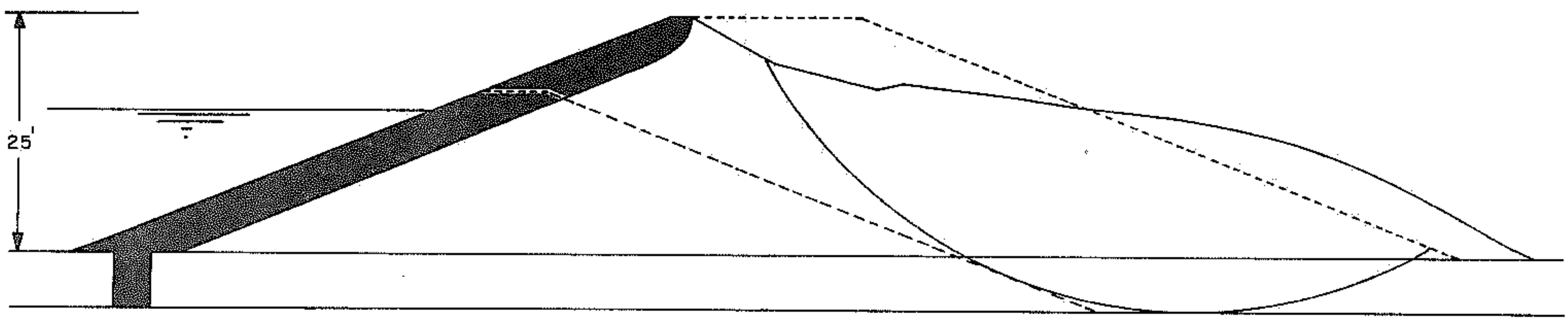


Fig.12 - ANTICIPATED CONFIGURATION IF FAILURE OCCURED AS PREDICTED BY CONVENTIONAL PSEUDO-STATIC ANALYSIS

strength parameters listed on page 21, values of the seismic coefficient, $k_f$, required to indicate a factor of safety of unity for a number of different surfaces are shown in Fig. 11. As may be seen from this figure, the minimum value of the seismic coefficient which would predict failure along a sliding surface in the likely failure zone is about 0.21; the computed factor of safety against sliding on this surface for a seismic coefficient of 0.1 is about 1.45.

For the somewhat lower strength parameters and higher water levels in the embankment discussed above, the value of $k_f$ would be reduced to about 0.15, a value approximately equal to the maximum ground acceleration developed during the earthquake, and the factor of safety for a seismic coefficient of 0.1 would be about 1.14.

In the light of the above discussion it would appear that the use of a pseudo-static approach for analysis of the Sheffield Dam failure leads to the following conclusions:

(1) For the more likely conditions of water table elevation in the embankment, the use of the average measured shear strength characteristics and a seismic coefficient of 0.1 leads to a computed minimum factor of safety of about 1.1; such a value is often accepted as an indication of adequate stability against earthquake effects.

(2) If the strength test data is interpreted conservatively, the water table elevation is taken at a conservatively high elevation in the section and the analysis is made using the conservative choices of analytical details listed on page 20, the use of a seismic coefficient of 0.1 leads to a computed failure condition (factor of safety = 1.0) on a surface of

sliding such as that shown in Fig. 9. However sliding did not occur on such a surface and its development would not appear to lead to a catastrophic slide movement of the embankment section such as that described by engineers who inspected the failed embankment.

(3) If analyses are made for conditions inducing sliding along the approximate failure surface, even the use of conservative strength data, a conservatively high water table elevation in the embankment and conservative analytical details leads to a computed factor of safety of at least 1.1, and a seismic coefficient of 0.15 would be required to give a computed factor of safety of unity; for average strength test data and the probable water level conditions, the seismic coefficient required to give a computed factor of safety of 1.0 would be about 0.2.

Thus, depending on the strength values and water level elevations considered in the computations, the analysis indicates either:

(1) That the embankment would be adequately stable for earthquake effects represented by seismic coefficients in the range of 0.1 to 0.15.

or (2) That the embankment would be brought to a condition of incipient failure by an earthquake force represented by a seismic coefficient of 0.1 to 0.17. However if seismic coefficients in this range are required to indicate failure for earthquake motions of moderate intensity (maximum ground acceleration about 0.15g) it would appear to be necessary to use substantially higher seismic coefficients, of the order of 0.3 or more, in conjunction with conservative choices of computational details to analyze embankment stability against earthquake ground motions with a maximum acceleration of 0.4 to 0.5g such as might be expected in California and other regions adjacent to major active faults.

In the light of these results, the pseudo-static approach does not appear to provide a very satisfactory basis for analyzing the observed failure of the Sheffield Dam.

## Analysis of the Seismic Stability of the Downstream Slope of the Sheffield Dam Using Dynamic Analyses and Cyclic Loading Triaxial Compression Test Data

In view of the limitations of pseudo-static approaches for analyzing the Sheffield Dam failure, it is of interest to determine whether a dynamic analysis, such as that previously proposed by Seed,[14] would adequately predict the observed performance. This type of analysis considers the cyclic nature of the stresses induced in the embankment by an earthquake and involves the following sequence of operations:

(1) A dynamic response analysis of the embankment for the given base motions is conducted to evaluate the time history of the inertia forces developed in the dam by the earthquake.

(2) The time history of the inertia forces is represented by an equivalent number of cycles, N, of an equivalent uniform seismic coefficient.

(3) Analyses are made, for various potential sliding surfaces, to determine the values of the confining pressures and principal stress ratios, before the earthquake, for elements of soil along these surfaces.

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[14] Seed, H. Bolton, "A Method for Earthquake Resistant Design of Earth Dams," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 92, No. SM1, January 1966.

(4) Cyclic loading triaxial compression tests are performed on samples initially consolidated under the range of confining pressures and principal stress ratio conditions to determine the cyclic stresses causing failure in the expected number, N, of stress applications. These results are expressed as relationships, for different values of the initial principal stress ratio, between the peak shear stress on the failure plane causing failure in N cycles and the effective normal stress acting on the failure plane before the earthquake.

(5) From the relationships developed in step (4), the peak shear stresses required to cause failure in N cycles at points along each potential sliding surface are determined and compared with the peak shear stresses induced by the earthquake on these surfaces. The factor by which the shear stresses required to cause failure must be divided in order to bring the slope to an incipient failure condition (factor of safety equal to unity) may be used as an evaluation of the factor of safety against complete failure. Alternatively the analysis can be conducted to determine the factor of safety against development of a given shear strain and this information used as a basis for assessing the probable deformation of the embankment during the earthquake.

Details of the computational procedures involved have been discussed previously.[14]

As a first step in applying this procedure to the Sheffield Dam a response analysis was made to determine the time history of seismic coefficients expressing the inertia forces developed by the Santa Barbara earthquake on potential slide masses extending to the base of the dam. The dam was considered to have a variable shear modulus expressed by the relationship $G = 6.5 \times 10^4 \times \sigma_o^{1/3}$ psf where $\sigma$ is the average overburden pressure along a plane at any depth below the crest, and a damping ratio of 15 percent. These values are typical of those measured for silty sands at strains corresponding to those induced by the earthquake base motions. The results of a computation of the dynamic seismic coefficients developed on a slide mass extending to the base of the dam if it were subjected to the base motions shown in Fig. 4 are shown in Fig. 13. The maximum value of the seismic coefficient developed was 0.21 and for about 6 cycles the seismic coefficient reached values exceeding 0.10. By appropriate weighting of the amplitudes of the various cycles shown in Fig. 13 it was concluded that the effects of such an earthquake would be essentially equivalent to 10 cycles of a uniform cyclic seismic coefficient of about 0.15.

It might be noted that if the dam responded as a rigid body the time-history of inertia forces would be represented by seismic coefficients identical to the base motion accelerations shown in Fig. 13; the slightly higher values shown in the upper part of Fig. 13 are due to a small amplification of the ground motion effects resulting from the deformation characteristics of the embankment soil.

Similar analyses were made for several other base motions having characteristics similar to those likely to have been developed by the Santa Barbara earthquake (see page 5). In general these analyses also showed that the seismic forces induced in the full body of the embankment by the base motions would be equivalent to about 10 cycles of a uniform cycle coefficient in the range 0.13 to 0.15g. Accordingly a seismic effect represented by 10 cycles of a cyclic seismic coefficient of 0.15g was adopted for analysis purposes.

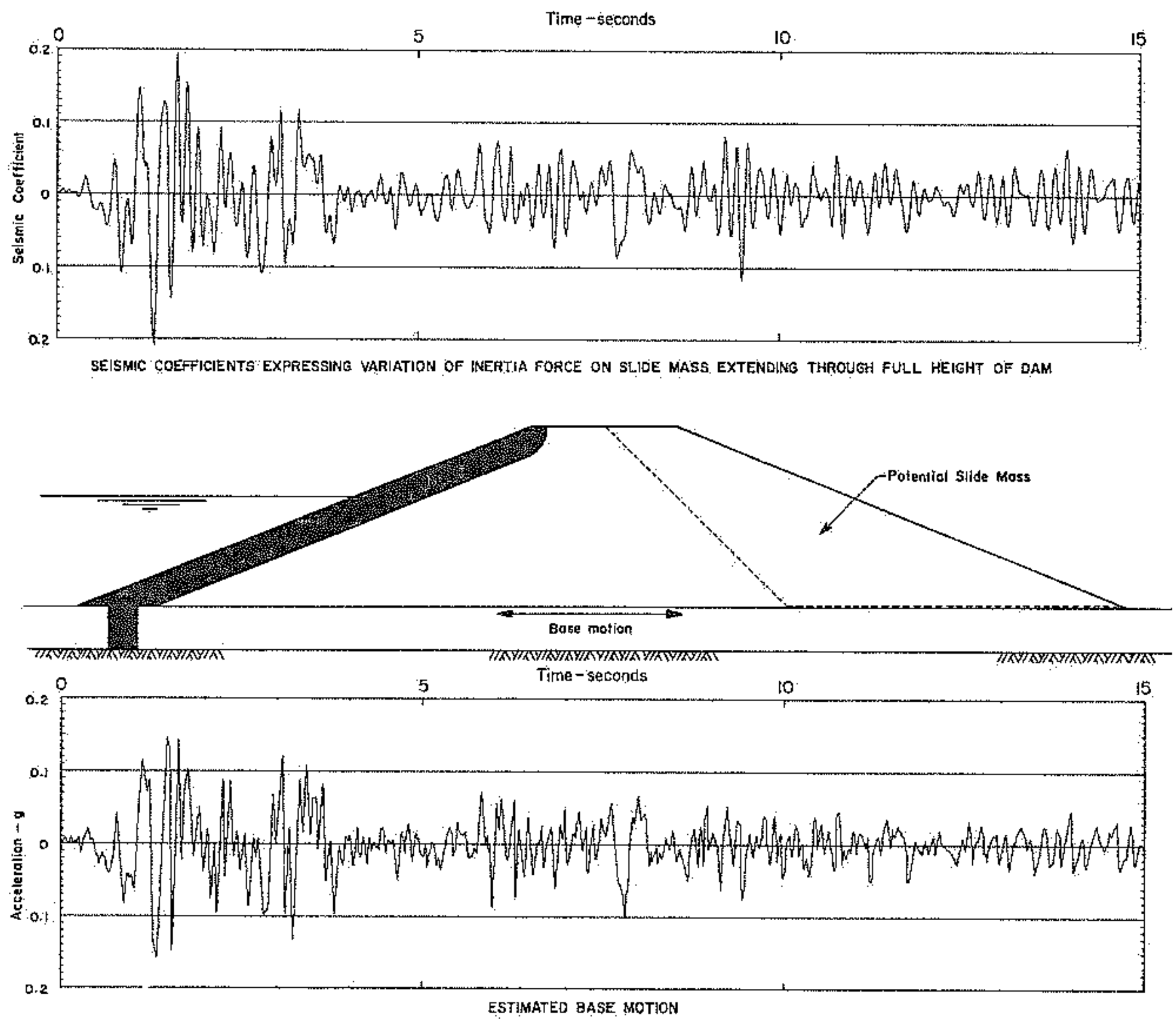



Fig. 13-ANALYSIS OF RESPONSE OF SHEFFIELD DAM TO EARTHQUAKE GROUND MOTIONS

A comprehensive series of triaxial compression tests was conducted to evaluate the strength and deformation characteristics of saturated specimens of the silty sand under cyclic loading conditions.[9] The results of a number of tests conducted on specimens initially consolidated to a principal stress ratio of 1.2 with confining pressures of 0.5, 1.0 and 2.0 kg per sq cm are shown in Fig. 14. By using different values of the cyclic deviator stress and noting the number of cycles required to cause failure (characterized by a sudden liquefaction of the test specimens and the development of large strains), the cyclic deviator stresses required to cause failure in 10 cycles could readily be determined by interpolation.

Thus for example, from the data presented in Fig. 14 the following combinations of initial stress conditions and failure conditions were determined:

| Test Condition No. | Initial Stress Conditions | | | Failure Conditions | |
|---|---|---|---|---|---|
| | $\sigma_{3c}$ | $\sigma_{1c}$ | $\sigma_{1c}/\sigma_{3c}$ | $\sigma_3$ | Peak deviator stress causing failure in 10 cycles |
| 1 | 0.5 kg/cm$^2$ | 0.6 kg/cm$^2$ | 1.2 | 0.5 kg/cm$^2$ | 0.41 kg/cm$^2$ |
| 2 | 1.0 kg/cm$^2$ | 1.2 kg/cm$^2$ | 1.2 | 1.0 kg/cm$^2$ | 0.64 kg/cm$^2$ |
| 3 | 2.0 kg/cm$^2$ | 2.4 kg/cm$^2$ | 1.2 | 2.0 kg/cm$^2$ | 1.35 kg/cm$^2$ |

If it is assumed that the failure plane is inclined at an angle of $45 + \frac{\phi'}{2}$ to the direction of the minor principal stress, the values of the normal stress on the failure plane during consolidation, $\sigma_{fc}$, and the peak shear stress on the failure plane causing failure in 10 cycles, $\tau_{ff}$, can

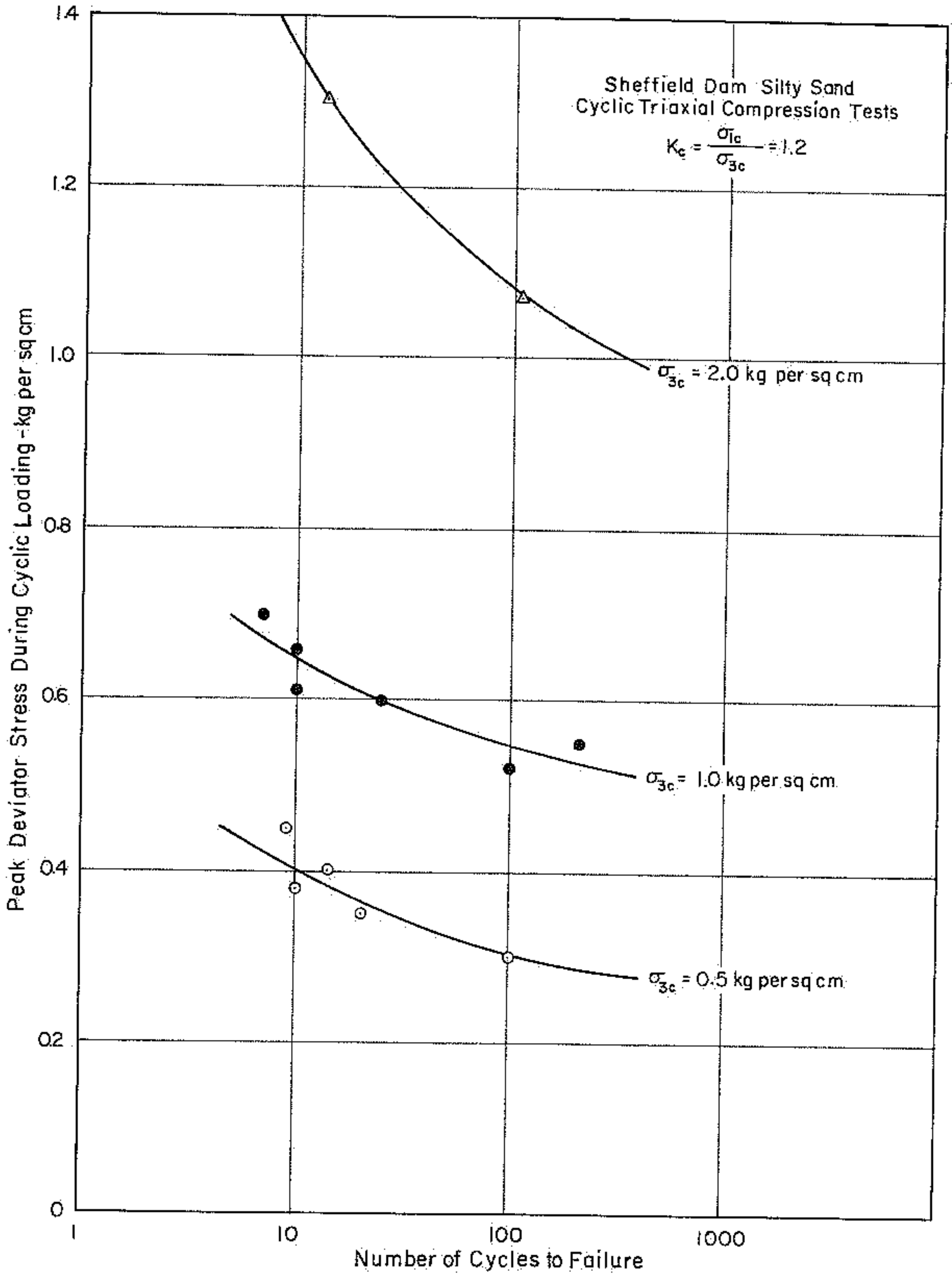



Fig.14-RESULTS OF CYCLIC LOADING TRIAXIAL COMPRESSION TESTS.

readily be determined from these data for each of the three test conditions. These determinations are shown graphically in Fig. 15, thereby establishing, for samples initially consolidated to a principal stress ratio of 1.2, the relationship between $\tau_{ff}$ and $\sigma_{fc}$ for use in stability analyses.

Similar series of tests were conducted for samples initially consolidated under conditions producing principal stress ratios of 1.0, 1.5 and 2.0. A summary of the test results for these values of the principal stress ratio during consolidation is presented in Fig. 16 and determinations of the corresponding relationships between $\tau_{ff}$ and $\sigma_{fc}$ in Fig. 17. A summary of the experimentally-determined relationship between the normal stress on the failure plane of a sample at the time of consolidation, $\sigma_{fc}$, and the peak shear stress on the failure plane causing failure to occur in 10 cycles, $\tau_{ff}$, for saturated soil samples initially consolidated to principal stress ratios of 1.0, 1.2, 1.5 and 2.0 is presented in Fig. 18. It is of interest to note that for any value of $\sigma_{fc}$, the cyclic loading strength increases considerably with increasing value of the principal stress ratio during consolidation. Furthermore for tests at principal stress ratios of 1.0 and 1.2, failure was characterized by sudden liquefaction of the soil and the accompanying development of large strains whereas for tests at principal stress ratios of 1.5 and 2.0, failure and liquefaction developed more gradually with increasing numbers of stress cycles.

It is also interesting to note that for the same initial isotropic stress conditions during consolidation, the strengths of samples subjected to cyclic loading were somewhat less than those of samples subjected to static loading. A comparison of the measured strengths for samples

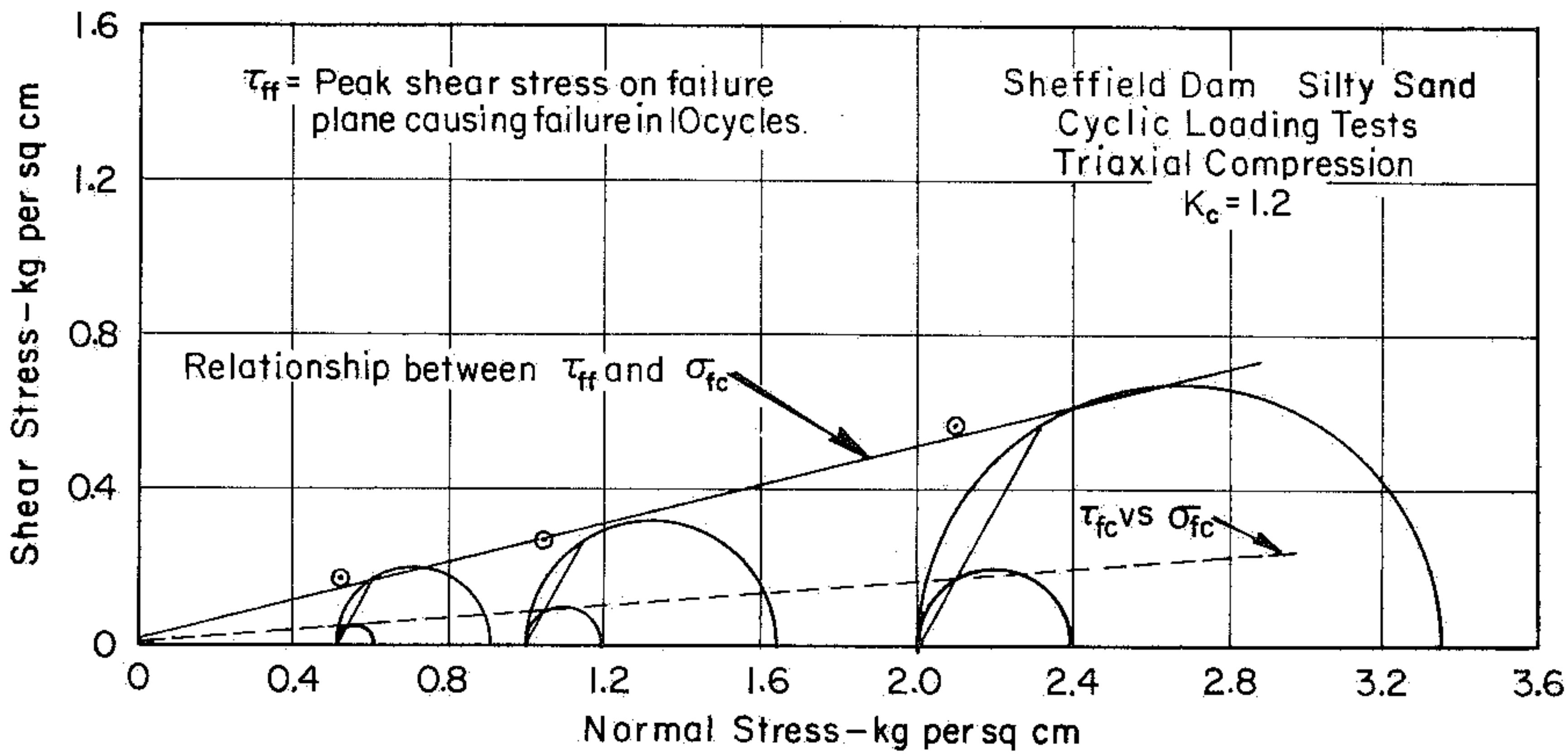



Fig. 15 – DETERMINATION OF RELATIONSHIP BETWEEN $\tau_{ff}$ AND $\sigma_{fc}$ FOR CYCLIC LOADING TRIAXIAL COMPRESSION TESTS WITH $K_c = 1.2$

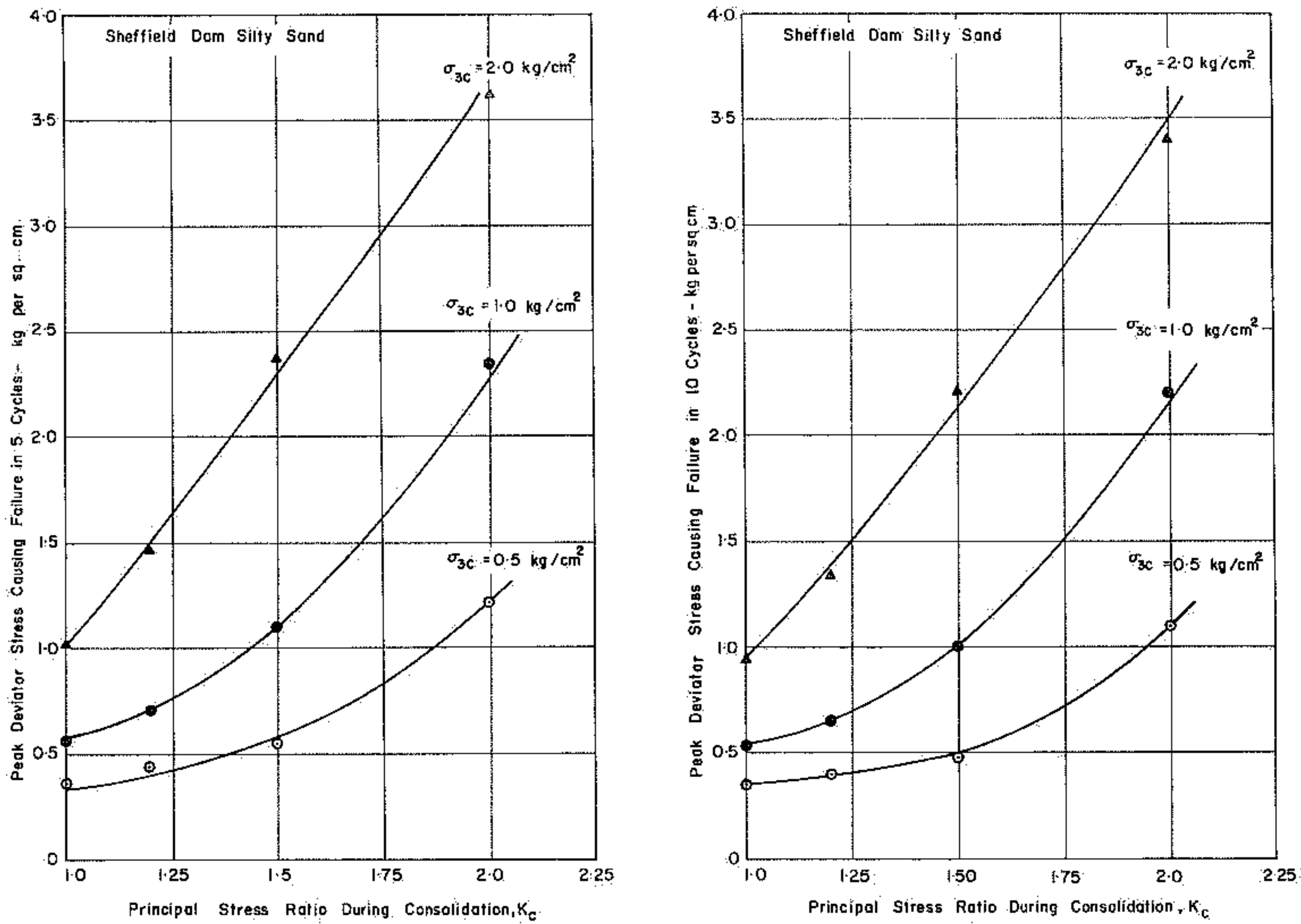



Fig.16 – INFLUENCE OF INITIAL PRINCIPAL STRESS RATIO ON PEAK DEVIATOR STRESSES CAUSING FAILURE IN CYCLIC LOADING TRIAXIAL COMPRESSION TESTS.

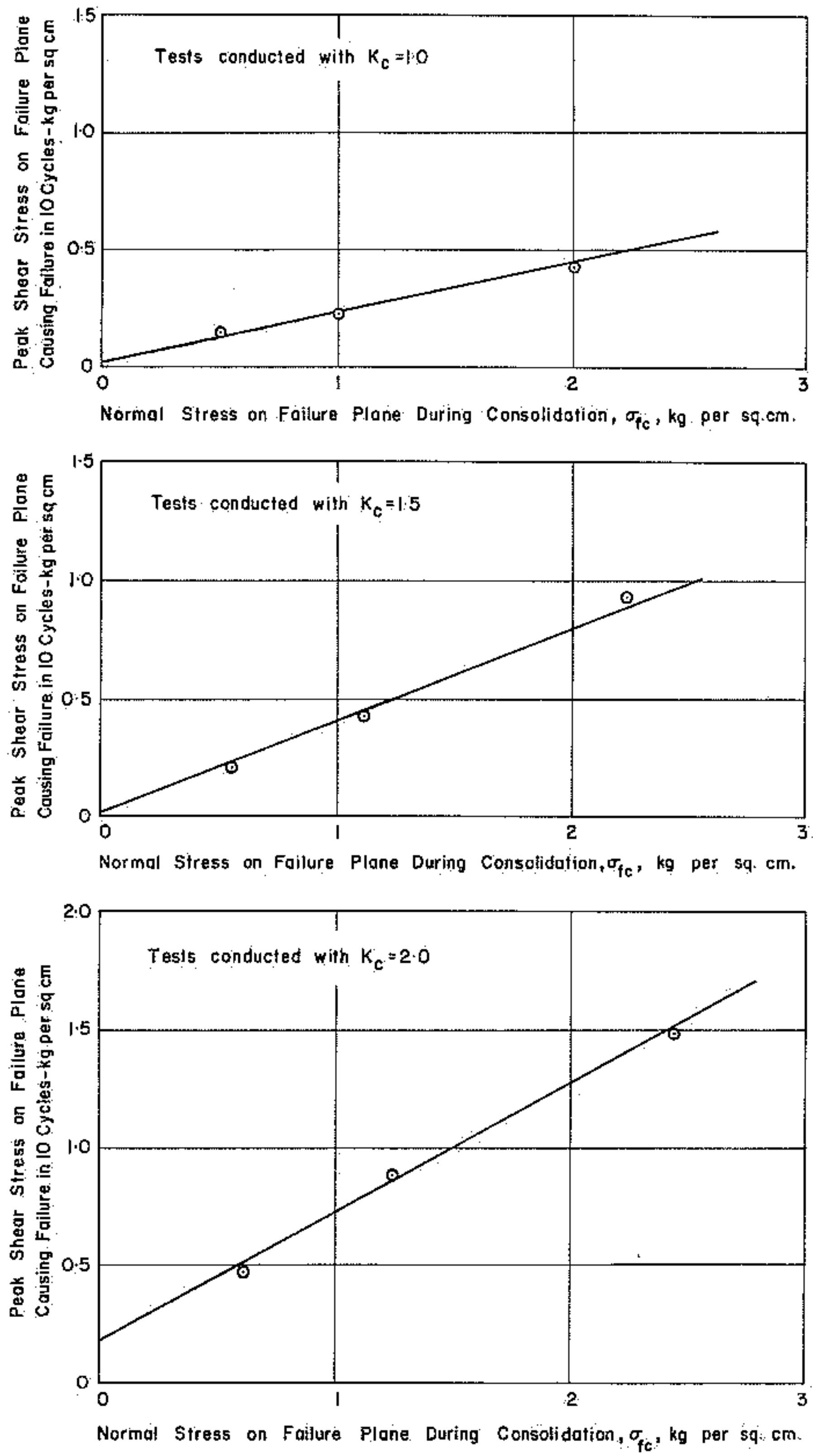



Fig. 17 – RELATIONSHIPS BETWEEN $\tau_{ff}$ AND $\sigma_{fc}$ FOR CYCLIC LOADING TRIAXIAL COMPRESSION TESTS.

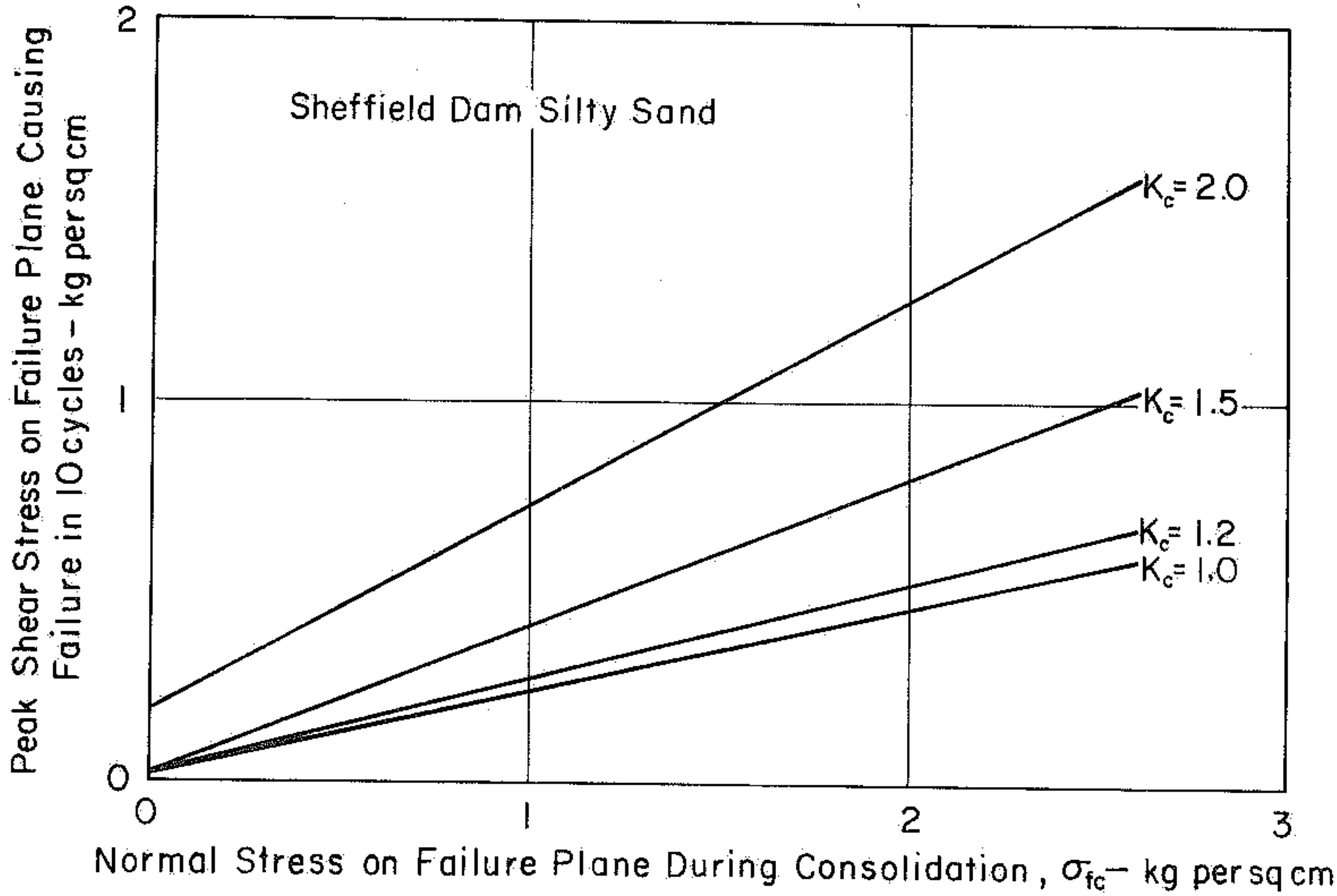



Fig. 18 – $\tau_{ff}$ vs $\sigma_{fc}$ RELATIONSHIPS FOR CYCLIC LOADING TRIAXIAL COMPRESSION TESTS.

initially consolidated under these conditions is shown in Fig. 19. Unfortunately the static test data presented previously does not permit a comparison for other values of the principal stress ratio during consolidation.

Using the cyclic loading triaxial test data for soil below the water table in the dam and static strength data for soil above the water table, analyses were made of the seismic stability of the Sheffield Dam using the dynamic analysis procedure previously described. The use of static test data was considered justifiable for soil above the water table since (1) there would be no significant change in pore water pressures, and therefore of strength, due to cyclic loading in a soil with a relatively low degree of saturation and (2) the main zone of interest from a failure surface point of view was almost entirely within the saturated portion of the embankment.

The results of these analyses are summarized in Figs. 20 and 21. The most critical surface was found to be close to that shown in Fig. 20, and the corresponding factor of safety against failure, accompanied by soil liquefaction, was 1.15. This position of the critical sliding surface and mode of failure are in good accord with the observed performance of the Sheffield Dam though the computed factor of safety is somewhat higher than would be desired.

As in the case of the pseudo-static analyses, computations were also made to determine the values of the seismic coefficient, $k_f$, which would have led to a computed factor of safety of 1.0 against complete failure, for various potential sliding surfaces. The results of these computations are shown in Fig. 21. The principal stress ratios shown in the upper part of the figure represent uniform average values determined by the analytical procedure for the various potential sliding surfaces. Soil strength data

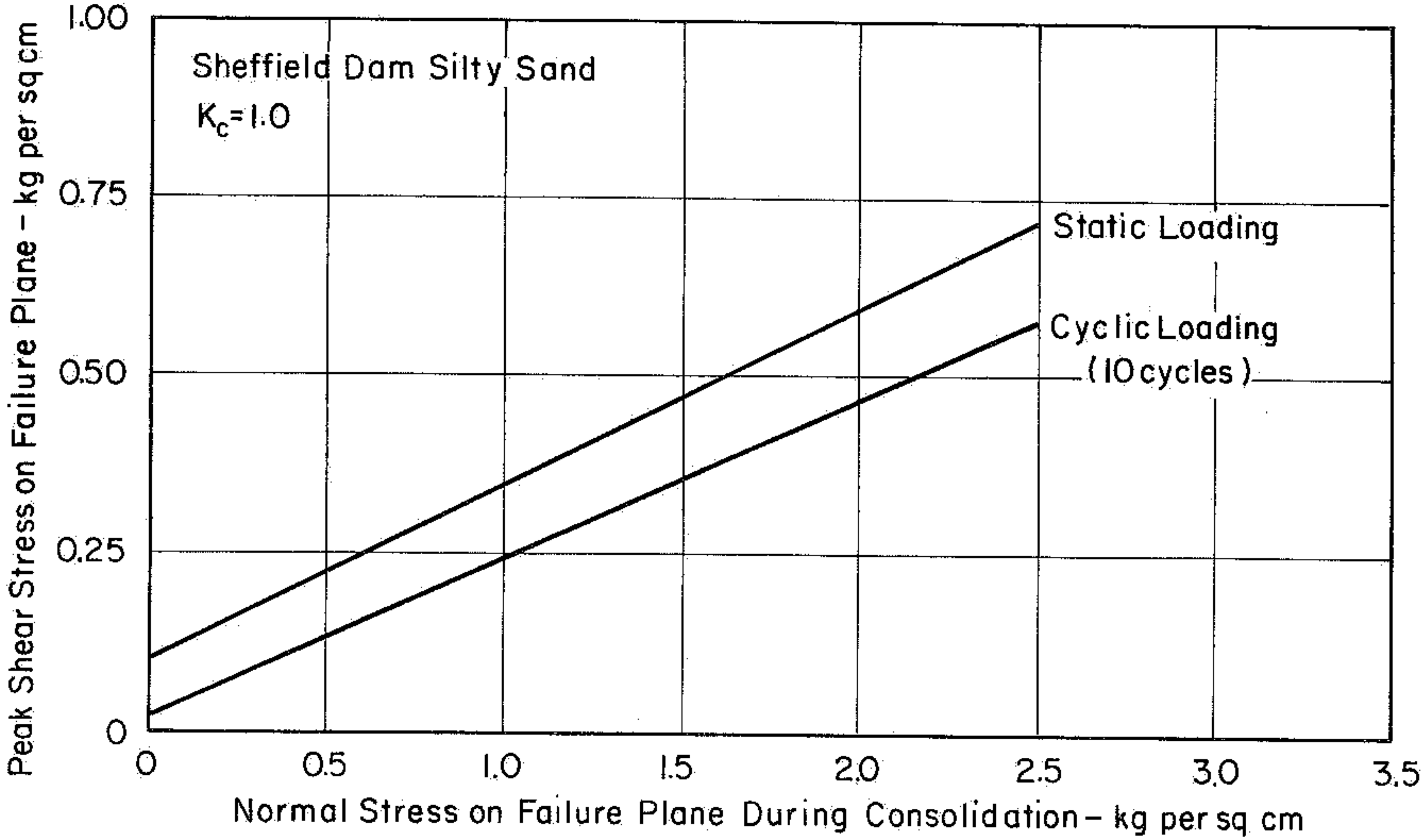



Fig. 19- COMPARISON OF SOIL STRENGTH UNDER STATIC AND CYCLIC LOADING CONDITIONS.

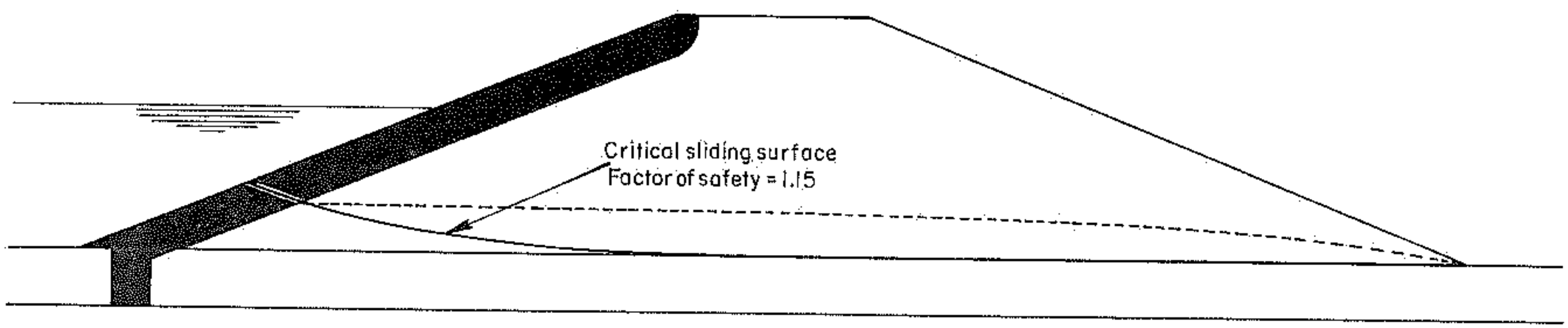



Fig. 20 – CRITICAL SLIDING SURFACE DETERMINED BY DYNAMIC ANALYSIS USING CYCLIC LOADING TRIAXIAL COMPRESSION TEST DATA.

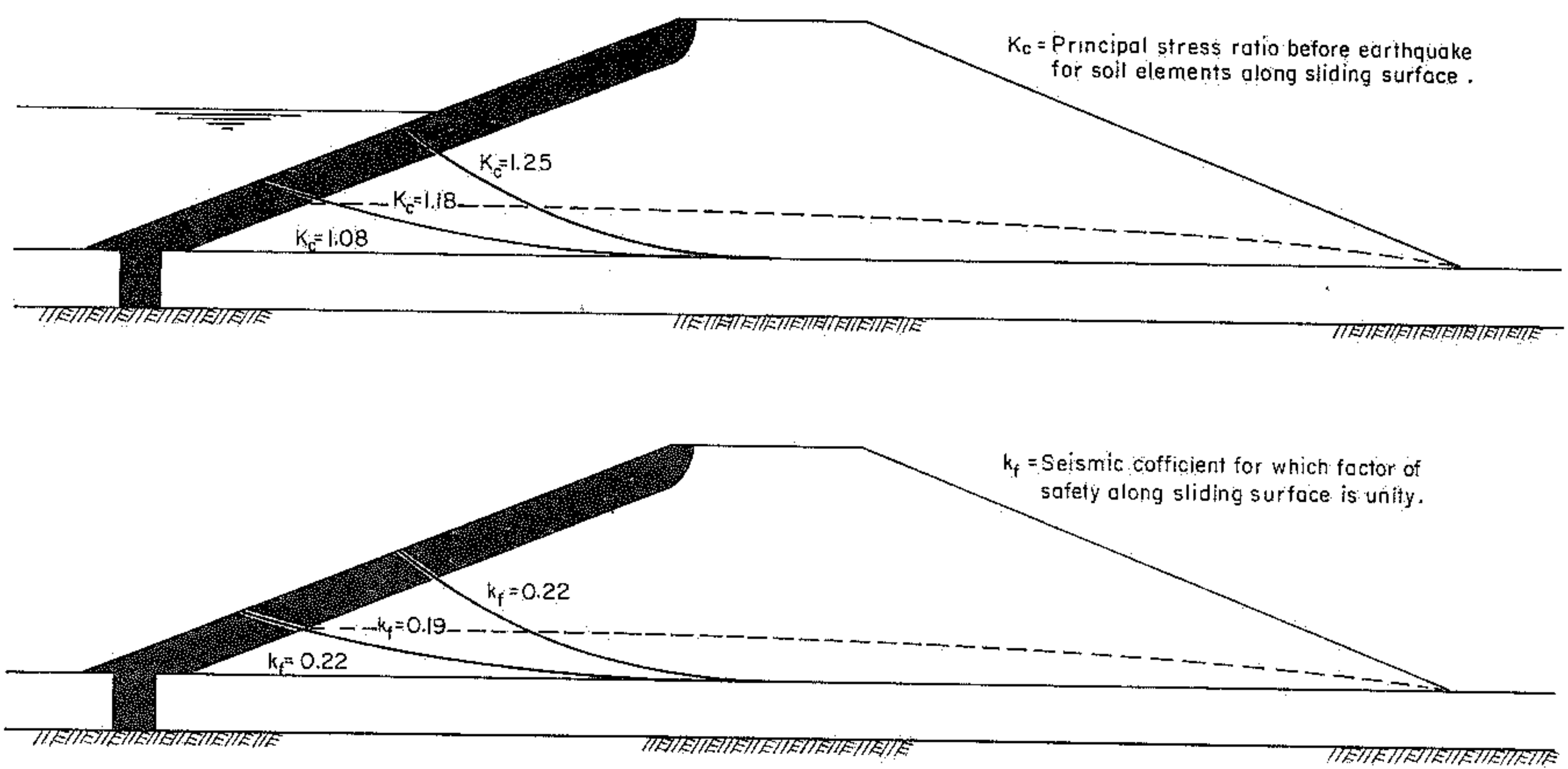



Fig.21 – DYNAMIC ANALYSIS USING CYCLIC LOADING TRIAXIAL COMPRESSION TEST DATA

corresponding to these values were used in computing the values of $k_f$ shown in the lower part of the figure. For slip surfaces intersecting the upstream slope below the water level the seismic coefficient values which lead to a computed failure condition range only from 0.19 to 0.22, compared with values greater than 0.21 for analyses of similar conditions using the pseudo-static approach. The seismic coefficient indicating failure might have been reduced to about 0.17 by considering the water level to be several feet higher in the embankment.

In view of the apparently lower strengths of the soil under cyclic loading conditions it might at first sight seem surprising that the seismic coefficients, $k_f$, leading to a factor of safety of unity against complete failure are not substantially lower than those indicated above. However in comparison with the pseudo-static analysis it should be noted that the dynamic analysis attempts to incorporate some of the more logical choices of computational details, which tend to increase the computed factors of safety. Included in this category are the following:

(1) The use of strength characteristics determined by tests on samples initially consolidated under anisotropic stress conditions.

(2) Consideration of the effects of forces on the sides of slices in the stability analysis.

(3) Application of the seismic forces at the centroids of slices.

(4) Use of the relationship between $\tau_{ff}$ and $\sigma_{fc}$ to determine soil strengths mobilized in the embankment.

Incorporation of these same features in the pseudo-static analysis would have greatly increased the discrepancy between the computed results and the observed performance.

At the same time the dynamic analysis utilizes lower values of soil strength and a more generalized form for the shape of the failure surface than those used in the pseudo-static approach. The influence of these two features is apparently just sufficient to offset the effects of the factors tending to increase the computed factor of safety for the most critical positions of the sliding surfaces in the two analyses. Similarly for slip surfaces near the actual zone of sliding, the dynamic analysis indicates somewhat lower values of the seismic coefficient required to produce a failure condition.

It would thus appear that the merits of the dynamic analysis procedure for this case may be summarized as follows:

(1) The analysis predicts a reasonably correct position for the failure surface.

(2) The analysis predicts a factor of safety of about 1.15 against catastrophic failure due to soil liquefaction.

At the same time the computed factor of safety against catastrophic failure, though close to unity, is perhaps somewhat higher than might have been anticipated for a dam which failed so completely. Thus an essentially similar type of analysis was made using test data obtained from cyclic loading simple shear tests rather than triaxial compression tests in an effort to explore possible improvements in the analytical procedures. The results of these studies are described in the following section.

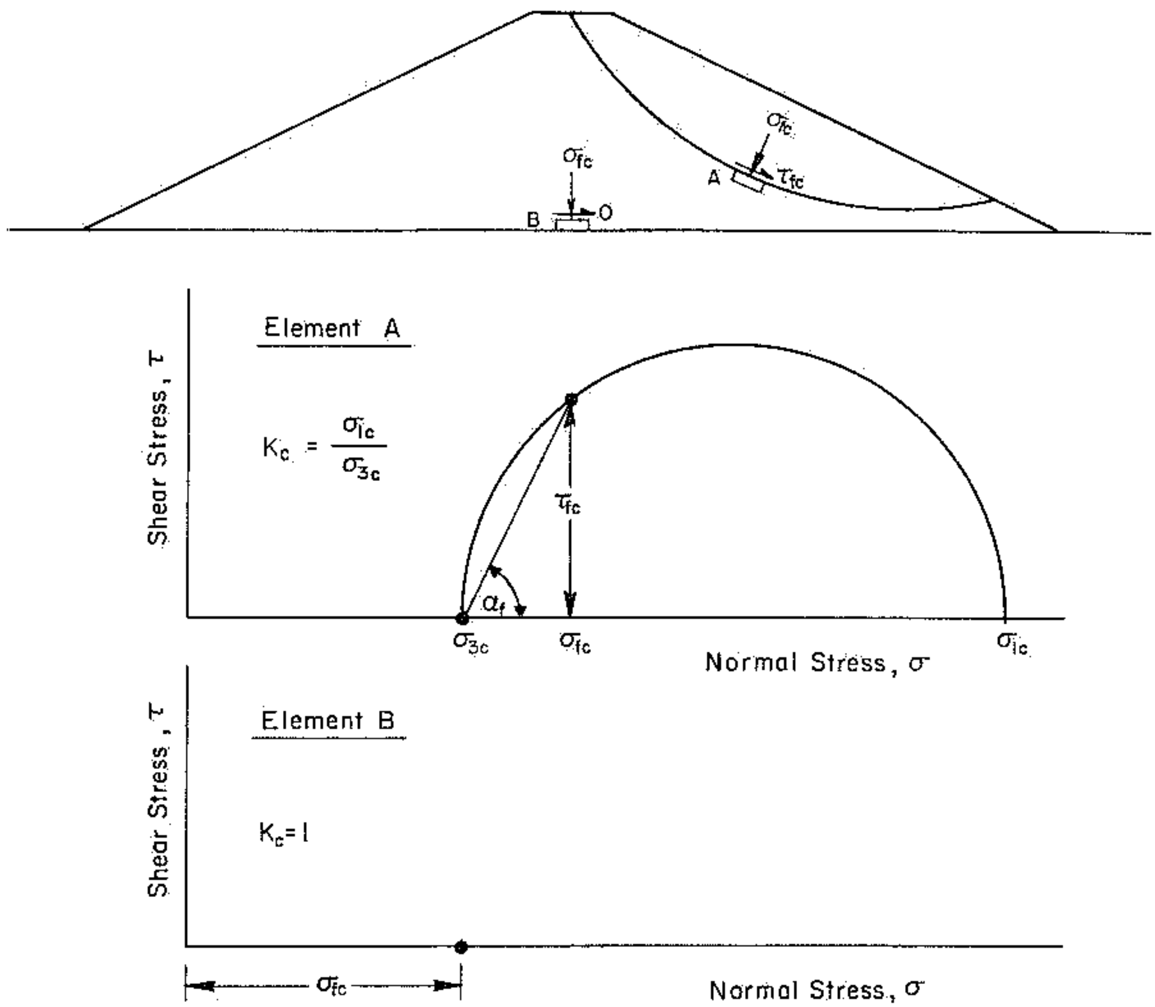
## Analyses of the Seismic Stability of the Downstream Slope of the Sheffield Dam Using Dynamic Analyses and Cyclic Loading Simple Shear Test Data

A critical appraisal of the results presented in the previous section indicates that logical difficulties are encountered in attempting to use the triaxial compression test procedure to simulate the stress conditions induced by an earthquake on relatively flat potential failure surfaces having high static factors of safety. Major difficulties in this regard are:

(1) For elements of soil on relatively flat surfaces the shear stresses on the potential failure planes are very low before the earthquake yet the normal stresses on these surfaces are high and the principal stress ratios, $K_c$, may well be of the order of 2. The only manner in which a combination of high normal stress and low shear stress may be induced on a potential failure surface in a conventional triaxial test is to consolidate a sample under approximately equal values of major and minor principal stress; that is, with a very low value of the principal stress ratio rather than the much higher field value. For example, for the soil element at point B in Fig. 22, the initial shear stress on the horizontal plane, which is a potential surface of sliding during an earthquake, is zero and the triaxial test representation of this condition would involve consolidation of a sample under isotropic stress conditions, as shown in Fig. 22. Yet in the embankment the soil element is consolidated under stresses corresponding closely to $K_o$ - conditions.

This discrepancy in consolidation conditions does not arise to the same extent for soil elements on surfaces having initially high shear stresses; that is, having relatively lower values of the factor of safety under static loading conditions. Thus the triaxial test can reasonably represent the pre-earthquake stresses on an element such as A in Fig. 22 but not those on an element such as B.

(2) During an earthquake the directions of the induced principal stresses do not coincide with those existing before the earthquake.




Soil element A : $K_c$ reasonably correct; little principal stress re-orientation during earthquake – AC-U triaxial compression test adequate.

Soil element B : $K_c \neq 1$; major principal stress re-orientation during earthquake — triaxial compression test inadequate.

Fig. 22–SIMULATION OF STRESS CONDITIONS ON SOIL ELEMENTS IN EMBANKMENT.

Thus for all elements of soil in an embankment there is some degree of re-orientation of principal stress directions during the shaking. For an element such as A in Fig. 22, the degree of principal stress re-orientation when the factor of safety is reduced is relatively small but for an element such as B, the degree of principal stress re-orientation is substantially larger.

In a triaxial compression test it is not possible to effect any re-orientation of principal stress directions other than 90 degrees. In most tests there is no re-orientation of principal stress directions and while this provides a reasonable simulation of the deforming stress conditions on elements such as A, it does not correspond at all well with the cyclic re-orientation of principal stress directions which occurs on elements such as B as the shaking continues during an earthquake.

Thus while triaxial test procedures may be entirely adequate for investigating the seismic stability of an embankment along potential slip surfaces which are subjected to high initial shear stresses due to static loads, they do not provide a good simulation of field loading conditions for soil elements located along relatively flat potential slip surfaces where static shear stresses are low. For this reason it would appear that cyclic loading simple shear tests might provide a better basis for analysis.

The test equipment and the mechanics of sample deformations in a cyclic loading simple shear test have been described elsewhere.[15] Essentially a

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[15]Peacock, W. H. and Seed, H. Bolton, "Liquefaction of Saturated Sand Under Cyclic Loading Simple Shear Conditions," Research Report, Soil Mechanics and Bituminous Materials Laboratory, University of California, July, 1967.

sample is maintained in a condition of plane strain in one horizontal direction but it can be loaded vertically and can undergo shear strains in the other horizontal direction. Thus it can be deformed to simulate the strains developed in soil elements subjected to plane strain loading conditions in the field. A schematic diagram of the test equipment is shown in Fig. 23.

In simple shear tests a soil sample can be brought to initial equilibrium under any given normal stress $\sigma_{fc}$ and any given shear stress, $\tau_{fc}$, representing the initial stresses on a soil element on a potential failure surface in the field. Cyclic shear stresses can then be superimposed to represent the effects of the earthquake. By determining the cyclic shear stresses required to cause failure, the test data leads directly to a relationship between the peak cyclic shear stress causing failure in a given number of cycles, $\tau_{ff}$, and the normal stress on a soil element before the earthquake effects were applied, $\sigma_{fc}$.

Tests can be conducted for any desired ratio of $\tau_{fc}/\sigma_{fc}$ but with the soil sample still being subjected to an effective principal stress ratio approaching that in the field, and the major principal stress can be caused to undergo a cyclic re-orientation during simulated earthquake loading. Thus the test eliminates some of the disadvantages of the triaxial compression test procedure with regard to the simulation of field loading conditions. Its main limitation is that for soils showing a substantial loss in strength after failure, stress concentrations near the edges of samples tend to induce failure at average stresses somewhat lower than the peak strength. Nevertheless if due allowance is made for this effect, the test offers considerable advantages over other testing techniques.

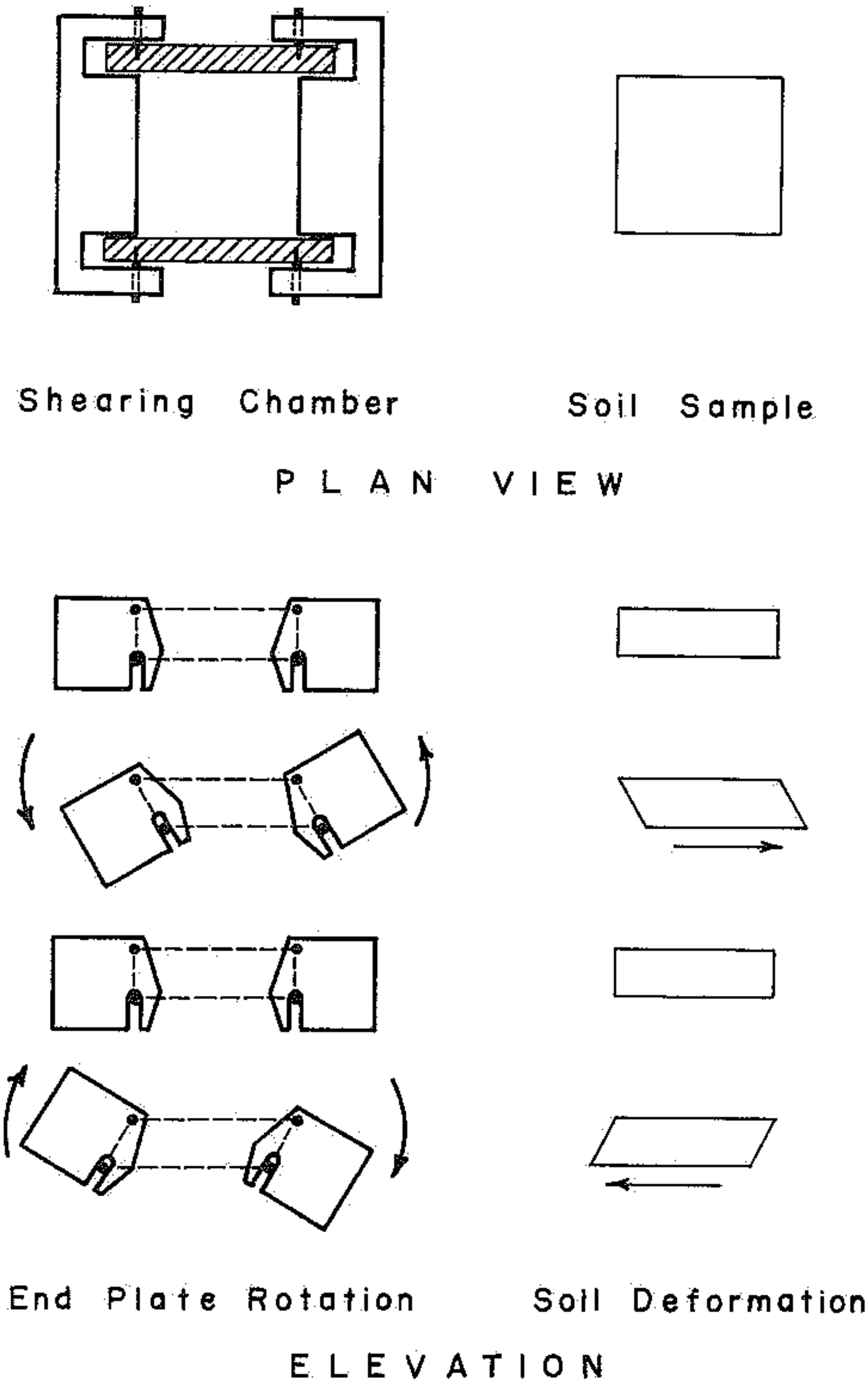



Fig. 23-SCHEMATIC DIAGRAM OF SIMPLE SHEAR TEST EQUIPMENT.

The results of a series of cyclic loading simple shear tests performed on samples of silty sand from the Sheffield dam-site are shown in Fig. 24. For this test series, samples were initially consolidated under $K_o$-conditions to a normal stress of 0.70 kg per sq cm. The samples were then subjected to cyclic shear stresses of different magnitudes under undrained conditions to determine the relationship between the magnitude of the cyclic stress and the number of stress cycles required to cause failure. In all tests, failure was accompanied by sudden liquefaction of the soil.

Fig. 25 shows similar data for samples consolidated initially under normal stresses, $\sigma_{fc}$, of 1.08 and 2.16 kg per sq cm but with initial shear stresses, $\tau_{fc}$, to maintain the ratio of $\tau_{fc}/\sigma_{fc}$ equal to 0.09.

From data of the type presented in Figs. 24 and 25, it is possible to establish the relationship between the peak cyclic shear stress required to cause failure in 10 cycles and the normal stress on the failure surface at the end of the consolidation stage of the test. These relationships, for initial conditions of $\tau_{fc}/\sigma_{fc} = 0$ and $\tau_{fc}/\sigma_{fc} = 0.09$ are shown in Fig. 26.

It is of interest to compare the cyclic loading simple shear test data in Fig. 26 with the results obtained from the corresponding cyclic loading triaxial compression tests in Fig. 18. For ease of comparison the results are reproduced in Fig. 27. It is readily apparent that for samples with no initial shear stress, the cyclic shear stresses causing failure in simple shear tests are very much lower than those causing failure in the corresponding type of triaxial compression test. However as the magnitude of the initial shear stress on the failure plane increases, the difference between simple shear and triaxial compression test data becomes much smaller. It appears likely that if simple shear

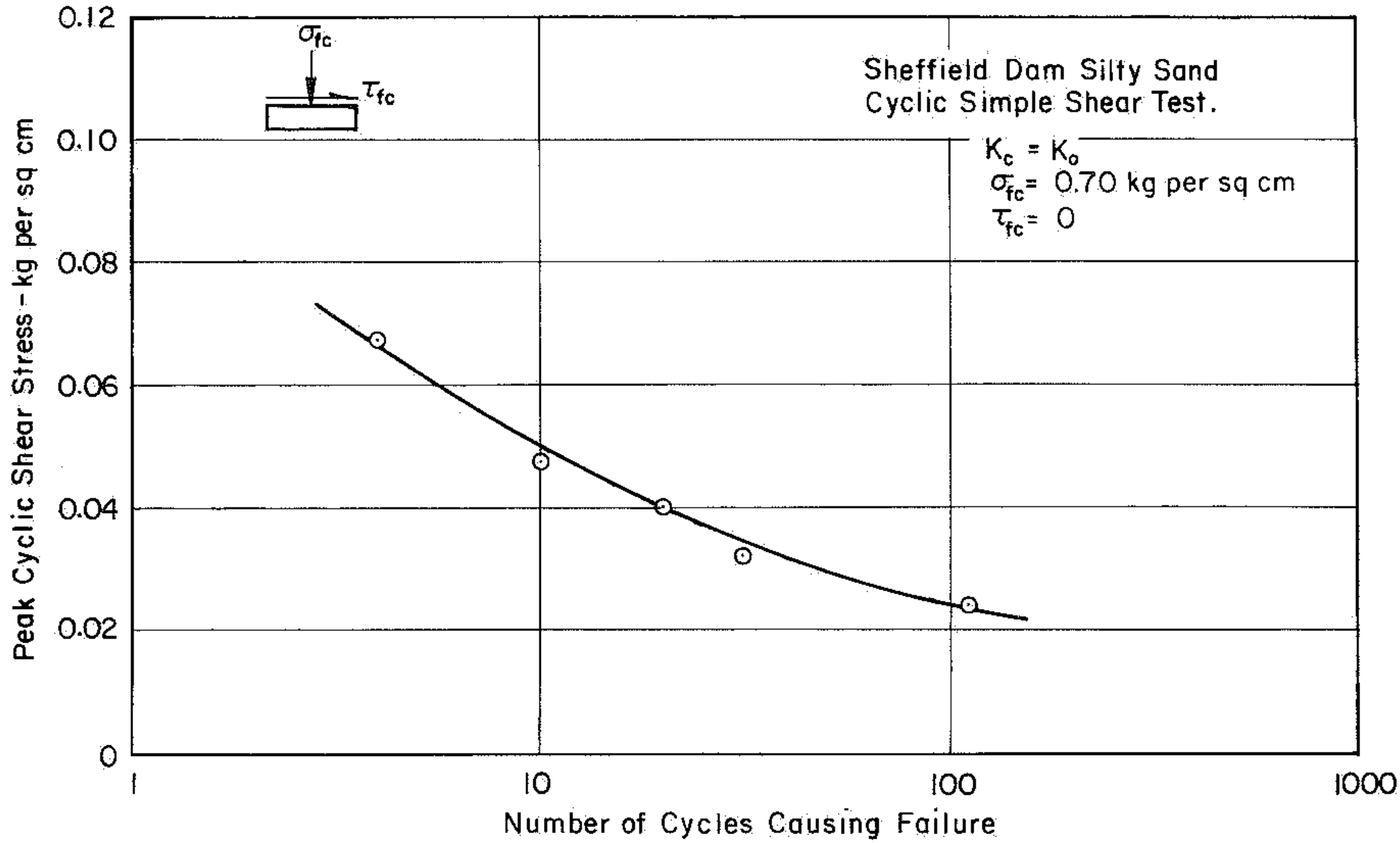



Fig.24 - RESULTS OF CYCLIC LOADING SIMPLE SHEAR TESTS WITH $\tau_{fc} = 0$

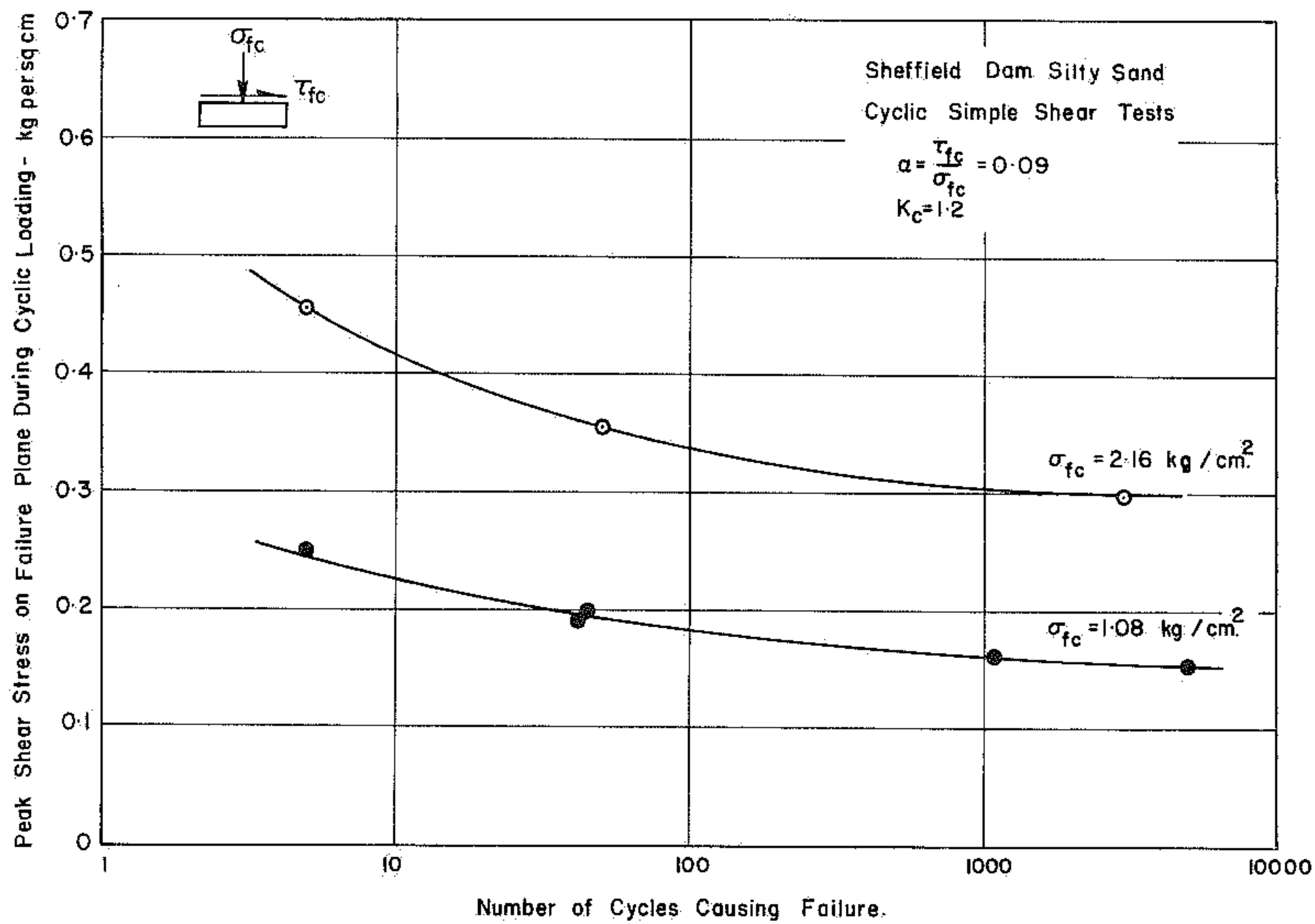



Fig.25–RESULTS OF CYCLIC LOADING SIMPLE SHEAR TESTS WITH $\tau_{fc}/\sigma_{fc}$=0.09

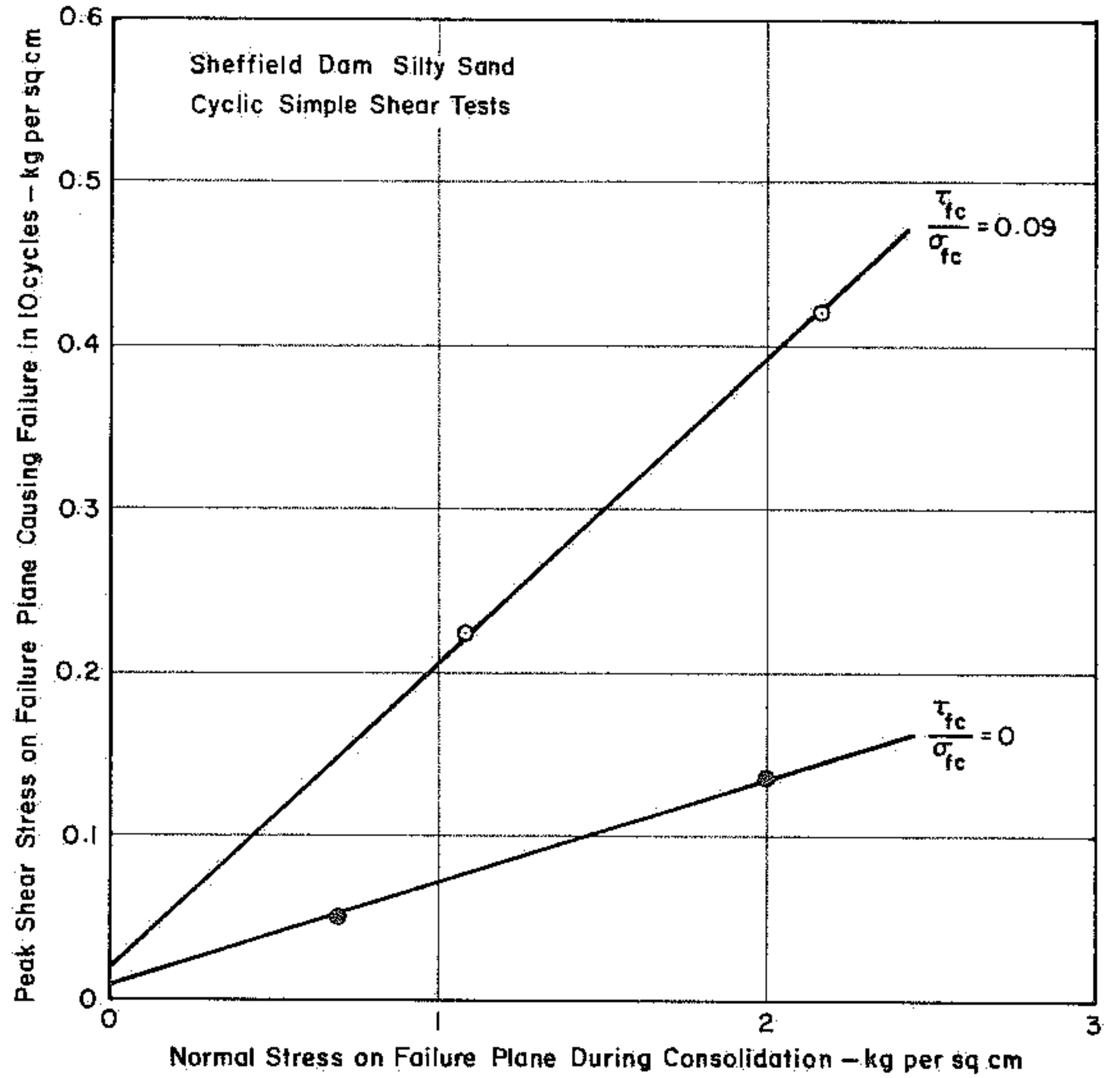



Fig.26–SUMMARY OF RESULTS OF CYCLIC LOADING SIMPLE SHEAR TESTS.

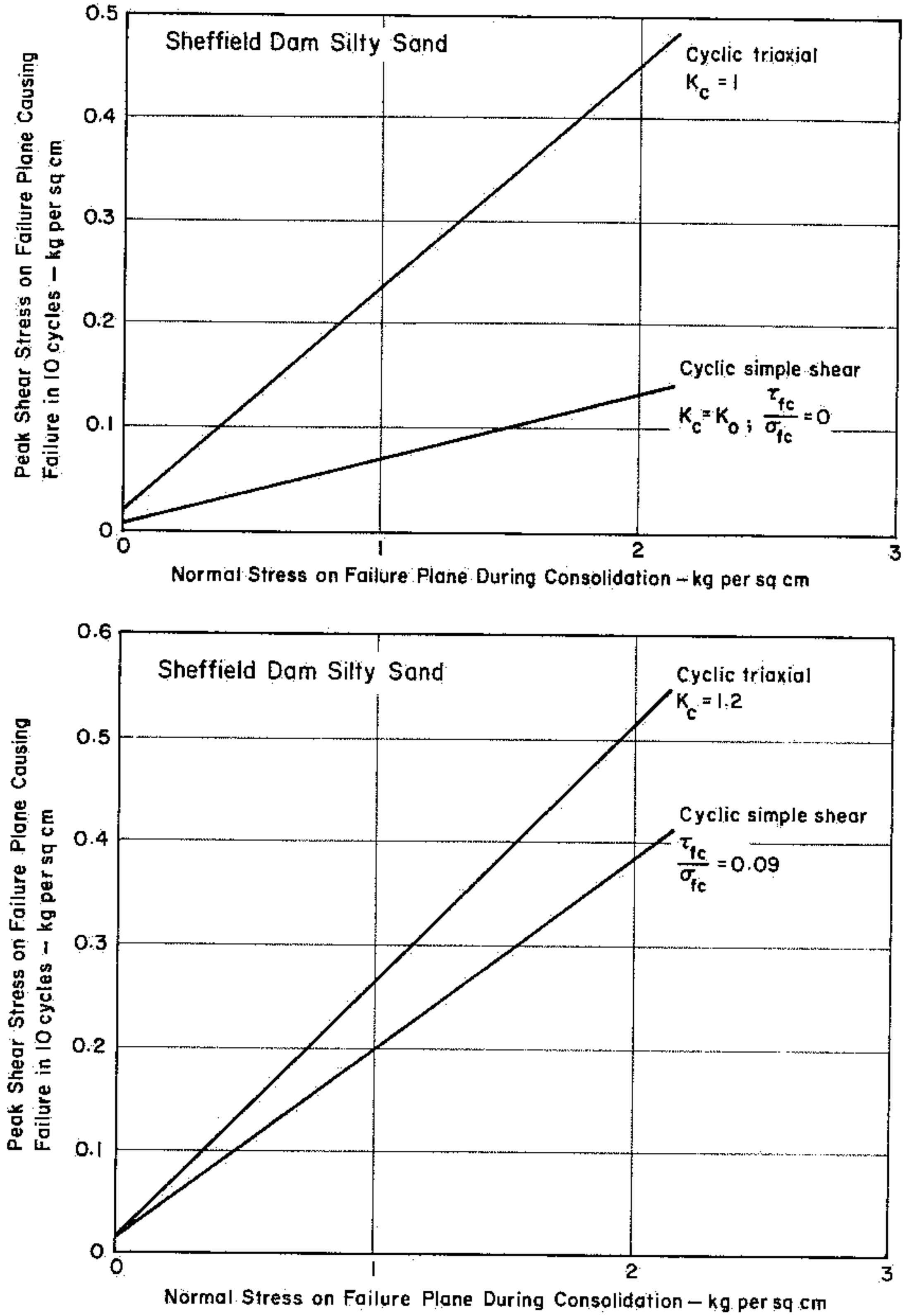



Fig.27–COMPARISON OF SOIL STRENGTHS IN CYCLIC TRIAXIAL AND CYCLIC SIMPLE SHEAR TESTS.

tests had been conducted using higher values of $\tau_{fc}/\sigma_{fc}$, the differences between the test data and that obtained from corresponding triaxial compression tests would have been insignificant. The results thus lend support to the argument that triaxial compression test data is adequate for analyzing potential sliding surfaces having high initial shear stresses but is somewhat deficient for analysis of surfaces having low initial shear stresses.

Previous studies of the cyclic shear stresses causing liquefaction and failure of saturated sands in simple shear tests in which $\tau_{fc}/\sigma_{fc} = 0$ have led to the conclusion that the recorded strengths are possibly 40 to 50 percent too low as a result of stress concentrations and difficulties in testing techniques.[12] Thus it is appropriate to make a correction of this amount to the simple shear test data shown in Fig. 26 before using it for analysis purposes.

Similarly a smaller correction would be appropriate for the test data obtained for $\tau_{fc}/\sigma_{fc} = 0.09$. From the data shown in Fig. 27 it would appear that the necessary correction would be of the order of 15 percent for $\tau_{fc}/\sigma_{fc} = 0.09$ or $K_c = 1.2$, decreasing to zero when $\tau_{fc}/\sigma_{fc} = 0.15$ or $K_c = 1.4$.

Applying corrections of the above magnitudes to the simple shear test data leads to the corrected values for analysis purposes shown in Fig. 28.

Using the corrected cyclic simple shear test data for the strength of the soils in the Sheffield Dam and its foundation and the dynamic analysis procedure previously described, analyses were made of the seismic stability of the dam during the Santa Barbara earthquake. The results of these analyses are shown in Figs. 29 and 30. The most critical surface of sliding was found to be along the base of the dam where the computed factor of safety

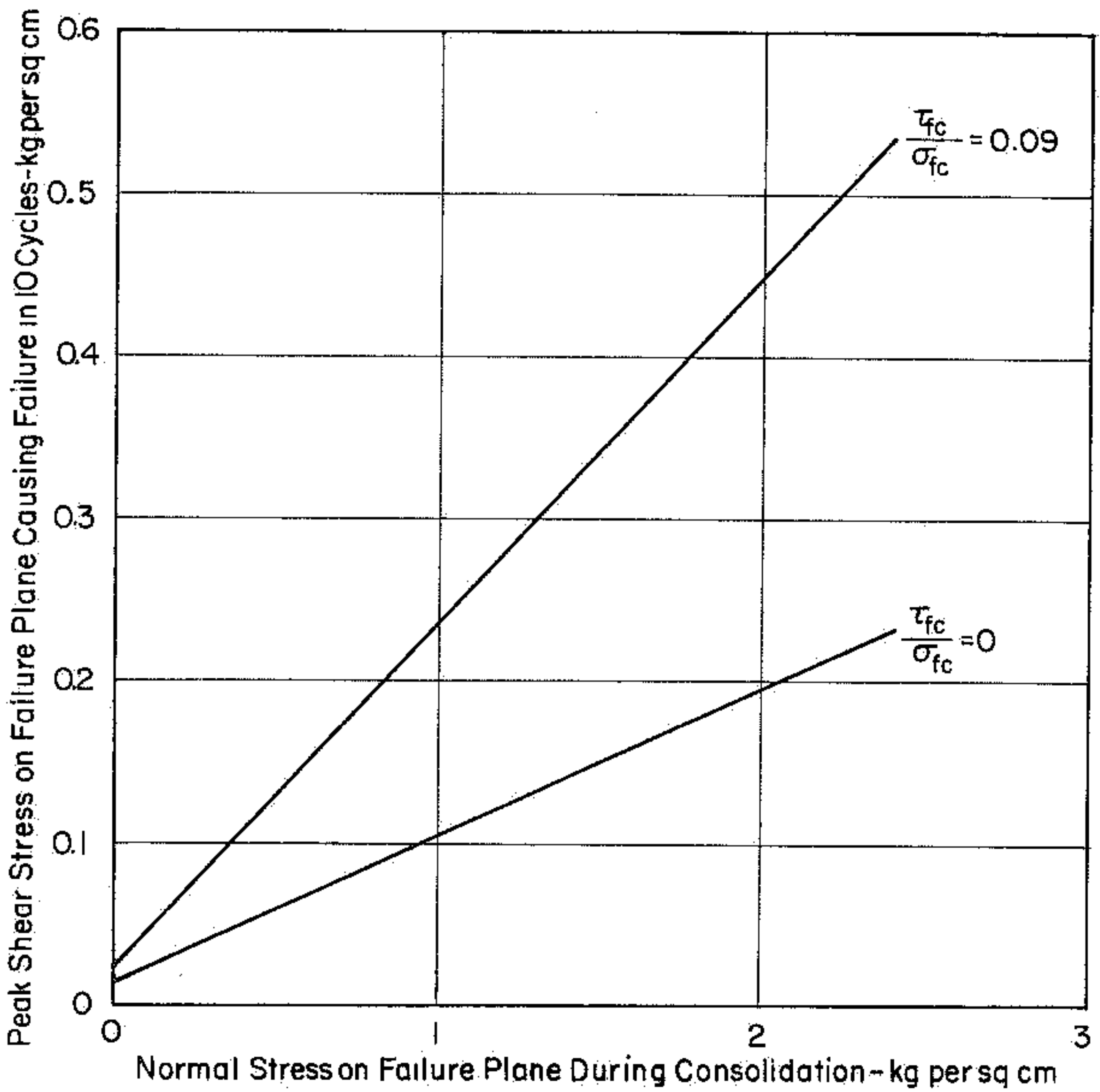



Fig. 28-CORRECTED DATA FROM SIMPLE SHEAR TESTS FOR USE IN ANALYSIS.

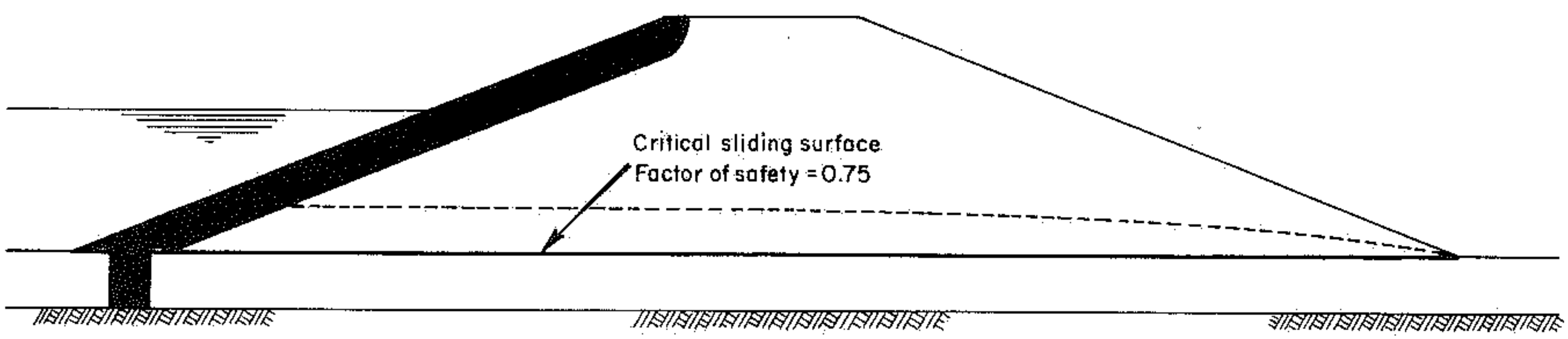



Fig. 29- CRITICAL SLIDING SURFACE DETERMINED BY DYNAMIC ANALYSIS USING CYCLIC LOADING SIMPLE SHEAR TEST DATA.

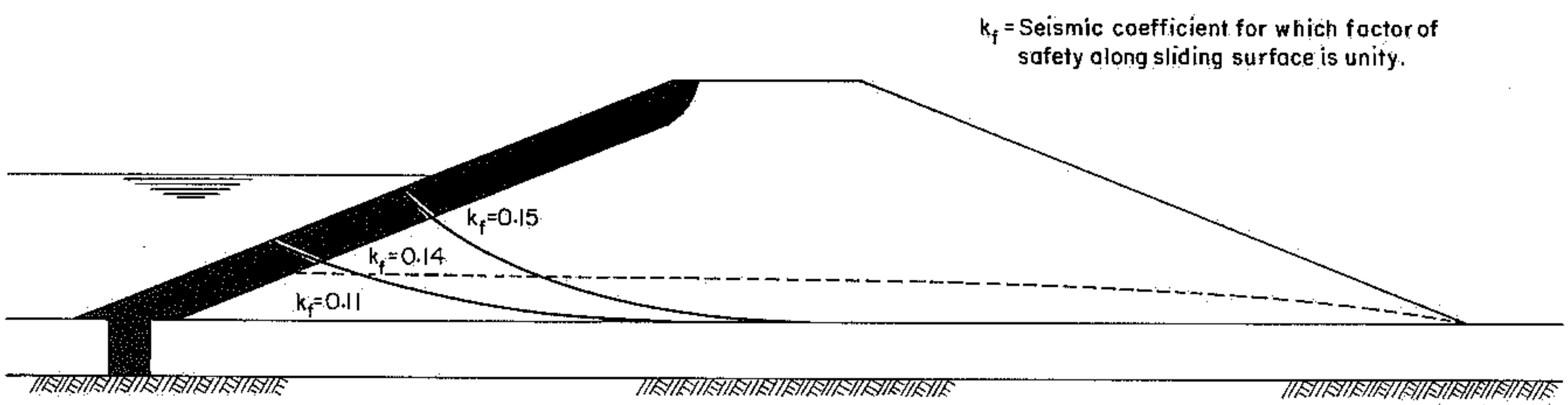



Fig. 30 – DYNAMIC ANALYSES USING CYCLIC LOADING SIMPLE SHEAR TEST DATA.

was only 0.75. It should be noted however that in analyzing an embankment failure during an earthquake there is no reason why the actual factor of safety should not be less than unity since the embankment might have failed before the surface ground motions had ceased - a situation comparable to the existence of a factor of safety of unity under a smaller earthquake or a factor of safety less than unity for the full earthquake. Judging from the extent of sliding of the Sheffield Dam it would appear that failure occurred before the ground motions had subsided.

As for the analysis procedures previously described, computations were also made for various potential sliding surfaces to determine the values of the seismic coefficient, $k_f$, which would have been required to produce a computed factor of safety of unity. The results of these computations are shown in Fig. 30.

For slip surfaces intersecting the upstream slope within about 5 ft of the reservoir level the seismic coefficients which would have led to a computed failure condition were all about 0.15 compared with the induced value of about 0.15 indicated by the dynamic response analysis. Thus the analyses would indicate that failure could well have developed in these zones during the earthquake if it had not previously developed elsewhere. Since, however, the computed seismic coefficient required to cause failure by sliding along the base of the dam was only 0.11, the analyses would indicate that it is along this surface that failure would first occur. Furthermore, as shown in the cyclic loading tests, the failure would be accompanied by liquefaction of the soil.

This mode of failure is in excellent agreement with the apparent behavior of the dam. Thus the dynamic analysis procedure incorporating cyclic simple shear test data appears to provide a reasonable and adequate basis for understanding and evaluating the mechanics of the Sheffield Dam failure.

Direct Analysis of Extent of Soil Liquefaction Along Base of Embankment

While the procedures previously described provide means for analyzing the stability of dams during earthquakes it should be recognized that they necessarily involve different degrees of simplification of the actual conditions in a dam both before and during an earthquake. Such procedures may be eminently suitable for some design purposes but it is of interest to develop a closer insight into the probable behavior of embankments during earthquakes in order to recognize the extent of the simplifications involved. New analytical tools provide a means for conducting more rigorous studies of the probable behavior of the Sheffield Dam during the Santa Barbara earthquake, as described below.

It has been shown that the effects of cyclic loading on the behavior of saturated sands are influenced to a considerable extent by the stresses existing in the sand before the cyclic stresses are applied (see, for example, Fig. 26). Of particular importance, for elements along a potential failure surface are the initial normal stresses $\sigma_{fc}$ and the ratio $\tau_{fc}/\sigma_{fc}$, where $\tau_{fc}$ is the initial shear stress. Values of these initial stress conditions may readily be determined by finite element analyses. The stress conditions along the base of the Sheffield Dam, as determined by this method, are shown in Figs. 31 and 32. In computing these stresses the embankment was considered to be constructed of a silty sand with the following characteristics:

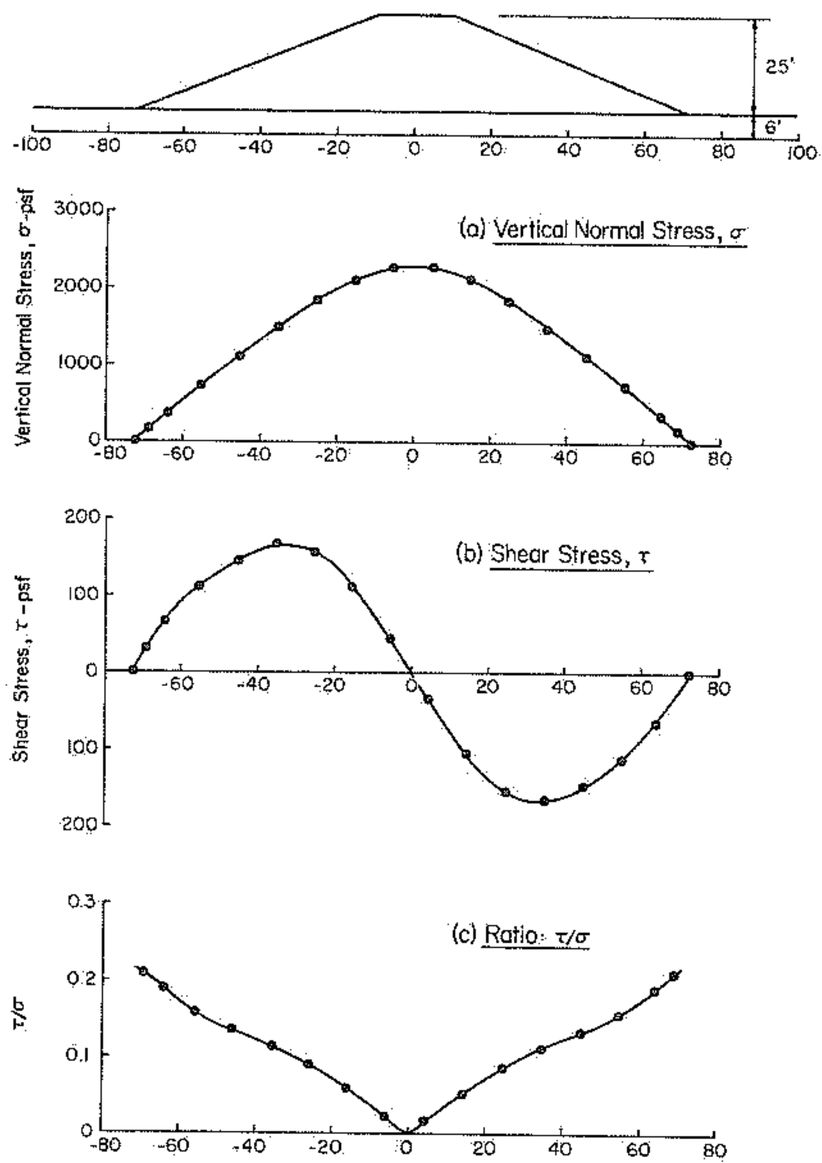



FIG. 31 STRESS DISTRIBUTION ALONG BASE OF DAM WITH RESERVOIR EMPTY.

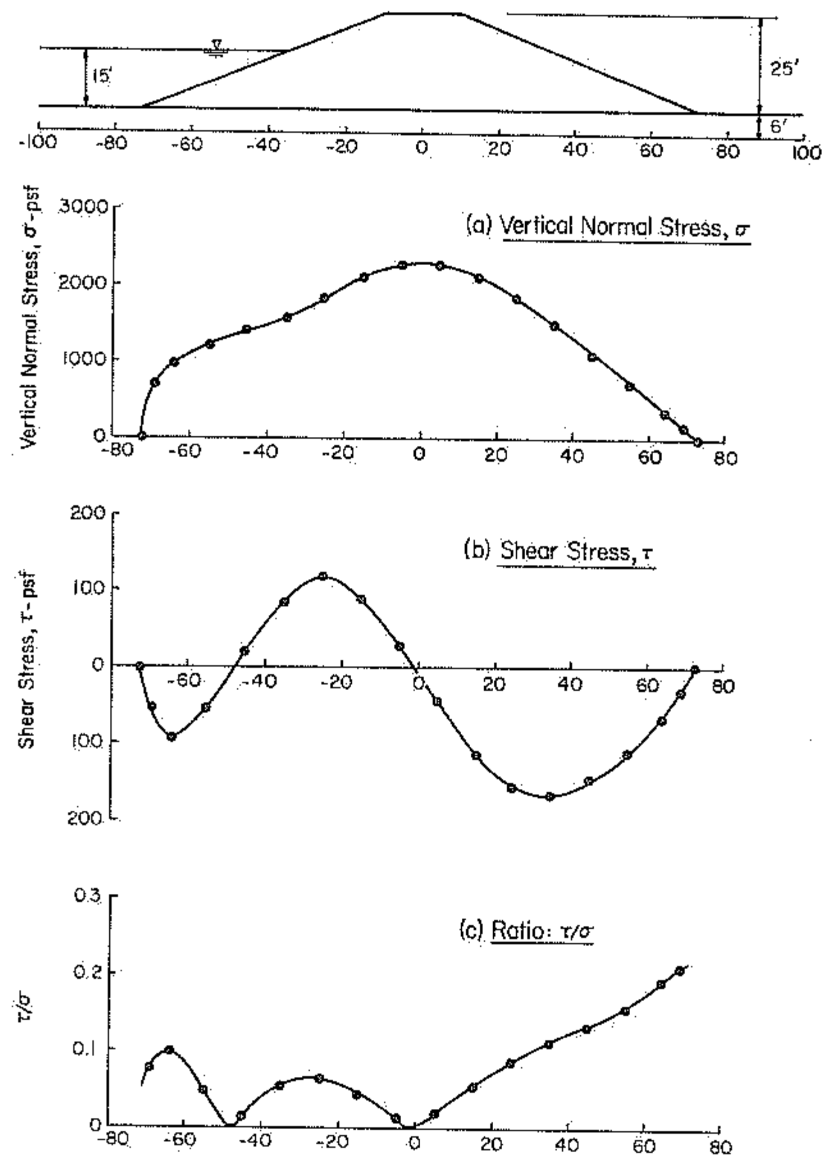



FIG. 32 STRESS DISTRIBUTION ALONG BASE OF DAM WITH 15 FT. OF WATER IN RESERVOIR.

Unit weight, $\gamma = 95$ pcf

Elastic modulus, $E = 800\ \sigma_3^{0.55}$ psf

Poisson's ratio, $\mu = 0.25$

Taking into account the water table elevation, these characteristics represent reasonable average values for the silty sand in the embankment and its foundation.

Fig. 31 shows values of the vertical normal stress, $\sigma$, the horizontal shear stress, $\tau$, and the ratio of these stresses, $\tau/\sigma$, along the base of the embankment with the reservoir empty. At the time of the Santa Barbara earthquake the depth of water in the reservoir was about 15 to 18 ft, and Fig. 32 shows the stress conditions along the base with the pressures caused by 15 ft of water in the reservoir acting on the impervious upstream face. It may be seen that the water load causes a substantial change in the shear stress distribution along the base under the upstream slope and a corresponding change in the ratio of $\tau/\sigma$. The reduced values of $\tau/\sigma$ in this zone of the embankment would tend to increase the susceptibility of the soil to liquefaction during an earthquake, though the effects of this factor will be offset to some extent by the increase in vertical normal stress on soil elements in this zone.

From a knowledge of the initial stress conditions on soil elements along the base of the embankment, the superimposed cyclic stresses which will cause failure or liquefaction of these elements can be determined from the test data presented in Fig. 28. In this figure the data are presented in terms of the peak cyclic shear stresses (initial static stresses plus superimposed cyclic stresses) causing failure of a soil element in 10 cycles. For present purposes it is convenient to replot the data to show only the superimposed cyclic shear stresses causing failure in 10 cycles; the data in Fig. 28, replotted in this form, are presented in

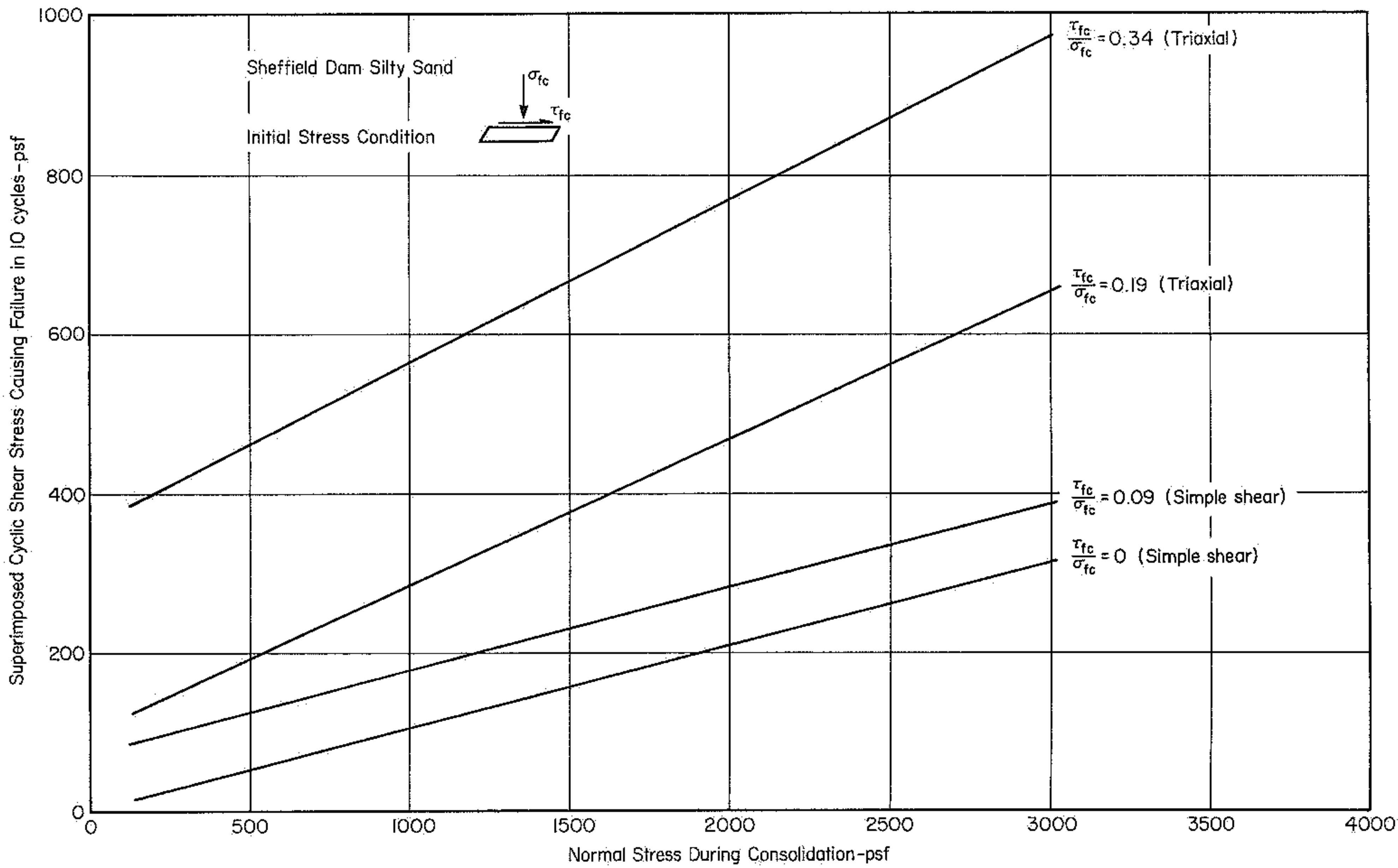



FIG. 33 EFFECT OF INITIAL STRESS CONDITIONS ON CYCLIC SHEAR STRESSES CAUSING FAILURE IN 10 CYCLES.

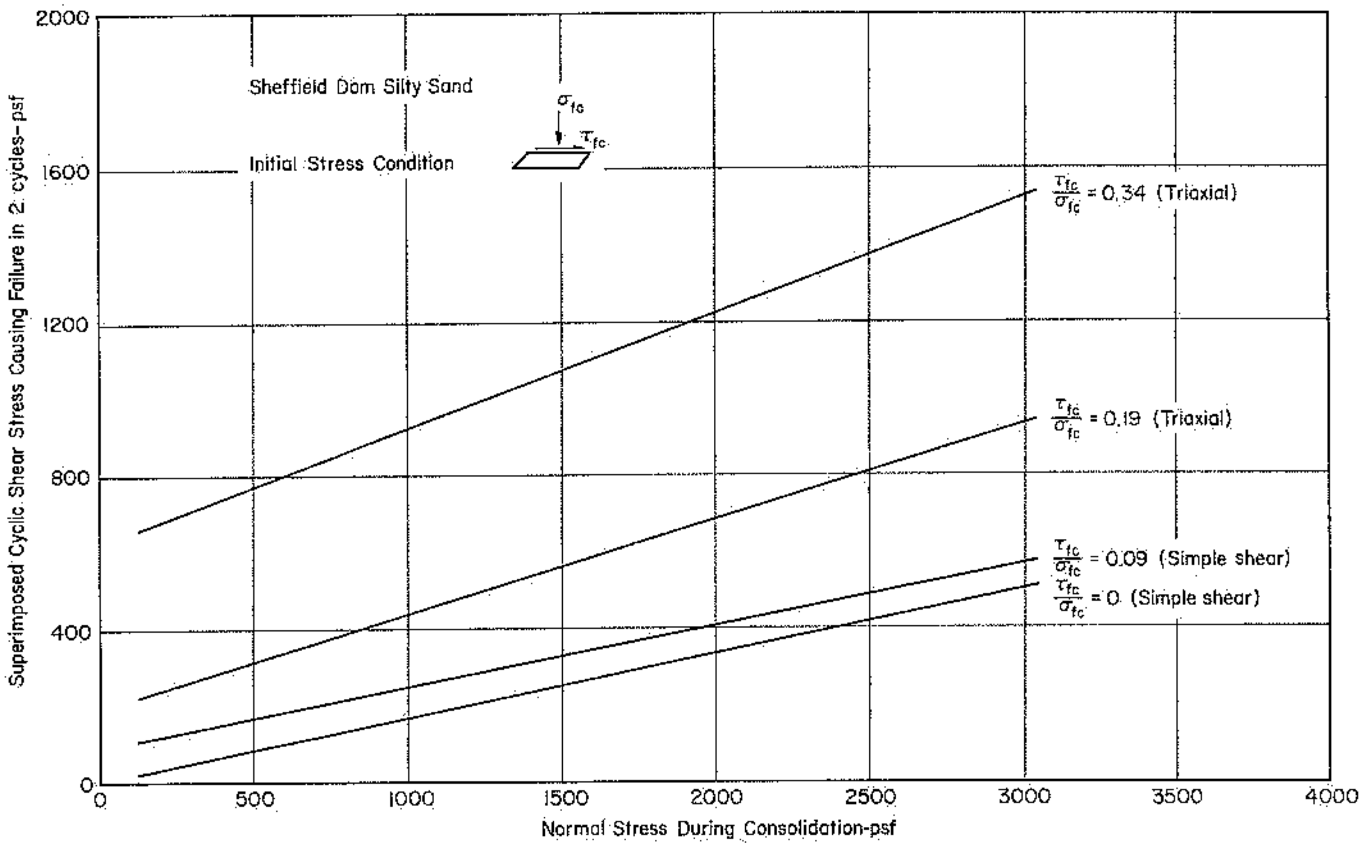



FIG. 34 EFFECT OF INITIAL STRESS CONDITIONS ON CYCLIC SHEAR STRESSES CAUSING FAILURE IN 2 CYCLES.

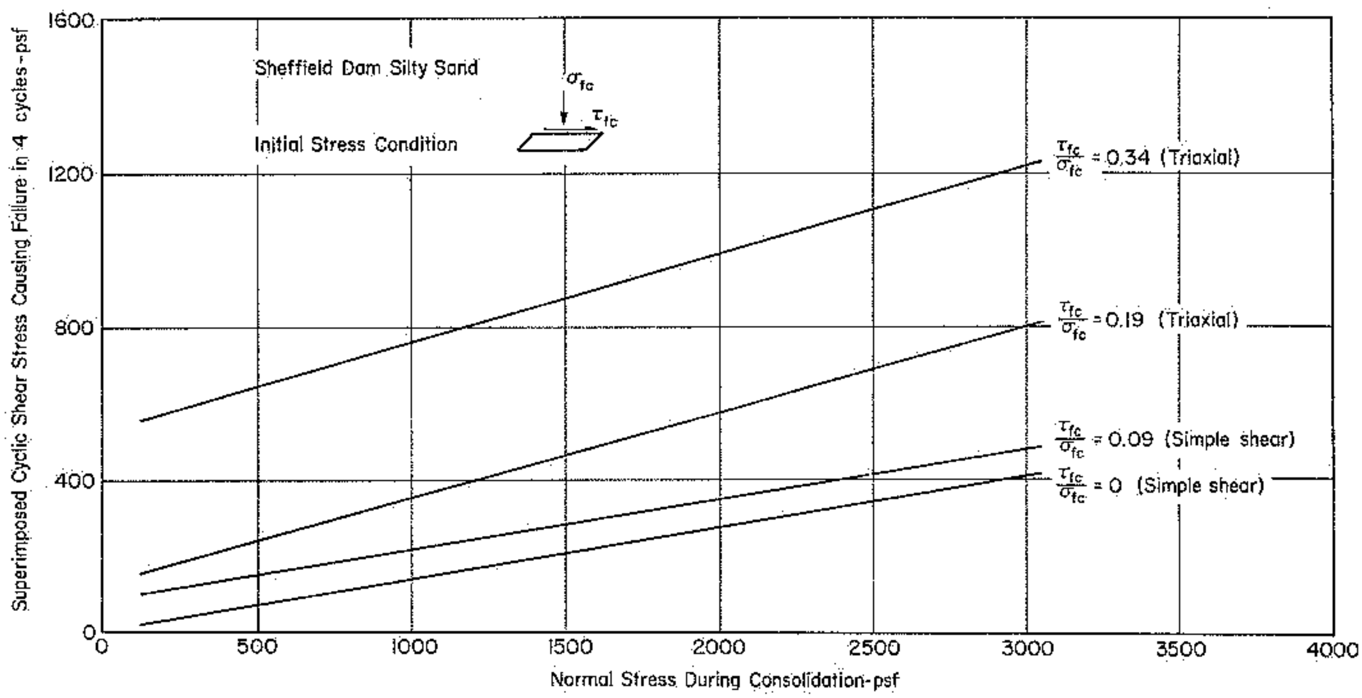
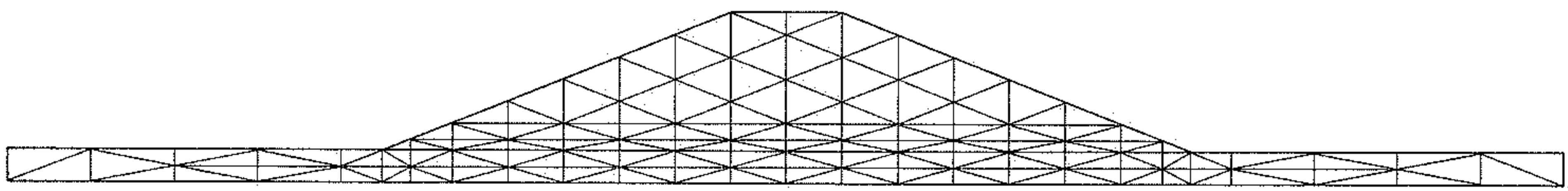



FIG. 35 EFFECT OF INITIAL STRESS CONDITIONS ON CYCLIC SHEAR STRESSES CAUSING FAILURE IN 4 CYCLES.

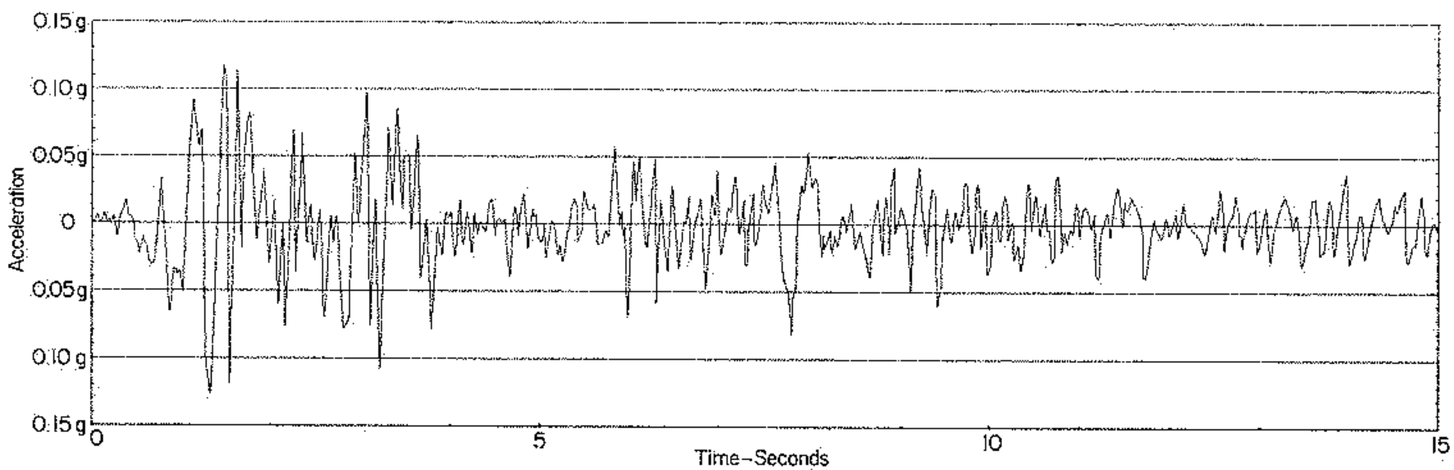
Fig. 33, together with additional data from the cyclic loading triaxial compression tests. Similar plots, showing the cyclic shear stresses causing failure of the Sheffield Dam silty sand after 2 and 4 stress cycles, for various values of normal stress and $\tau/\sigma$ ratios are shown in Figs. 34 and 35.

It should be noted, in connection with the test data shown in Figs. 33, 34 and 35, that for tests in which the initial ratio of $\tau_{fc}/\sigma_{fc}$ was 0 or 0.09, failure occurred as a result of a sudden liquefaction of the soil whereas for tests in which this initial stress ratio was greater than 0.19, failure and liquefaction developed gradually with increasing numbers of stress cycles. Thus it seems reasonable to assume that failure by sudden liquefaction would only develop in soil elements where the initial stress ratio $\tau_{fc}/\sigma_{fc}$ is below some limiting value, of the order of say 0.14. For the initial stress conditions along the base of the embankment shown in Fig. 32, this would mean that sudden liquefaction could be induced by sufficiently large cyclic stress applications anywhere along the base except within about 20 ft of the downstream toe.

The next step in an analysis of the embankment performance is to evaluate the cyclic shear stresses which are likely to be induced in soil elements along the base of the dam during the earthquake. These may also be determined by finite element procedures from a knowledge of the base motions affecting the dam and the material properties of the dam and its foundation. For the present study the embankment and foundation soils were represented by the series of elements shown in Fig. 36 and the response of the system to the hypothesized base rock motions shown in Fig. 36 was evaluated. It may be noted that the base motions shown in this figure are somewhat smaller in amplitude than those shown in Fig. 13



Finite Element Representation of Dam Cross Section




Estimated Base Motion

FIG. 36 FINITE ELEMENT REPRESENTATION AND BASE MOTION USED TO EVALUATE DYNAMIC RESPONSE OF EMBANKMENT.

since there is a small amplification of the rock motions by the time they reach the base of the embankment.

For the dynamic response analyses the following soil properties were used:

Unit weight of soil above water table = 107 lb per cu ft.

Unit weight of soil below water table = 120 lb per cu ft.

Modulus of embankment material, $E = 1.6 \times 10^5 \times \sigma_o^{1/3}$ psf

Modulus of foundation material, $E = 2.0 \times 10^5 \times \sigma_o^{1/3}$ psf

Poisson's ratio for all soils, $\mu = 0.25$

Damping factor for all soils, $\lambda = 0.15$

where $\sigma_o$ is the overburden pressure at any depth. These characteristics represent typical values determined for silty sands at the strain levels developed in the present study.

The dynamic response analysis leads to a variety of response parameters including the time history of horizontal shear stresses developed in the soil elements along the base of the embankment. The computed values of these stresses, for the embankment and base motion conditions shown in Fig. 36 are shown in Fig. 37. It may be seen that the shear stresses induced by the earthquake at different points along the base of the embankment vary considerably in magnitude, decreasing progressively from the largest values near the center-line to relatively small values near the toes of the embankment.

Furthermore, at any one point, the magnitude of the induced cyclic stress also varies considerably throughout the period of ground shaking. The plots in Fig. 37 indicate two large stress cycles occurring after about 1.5 seconds with substantially smaller stress cycles occurring

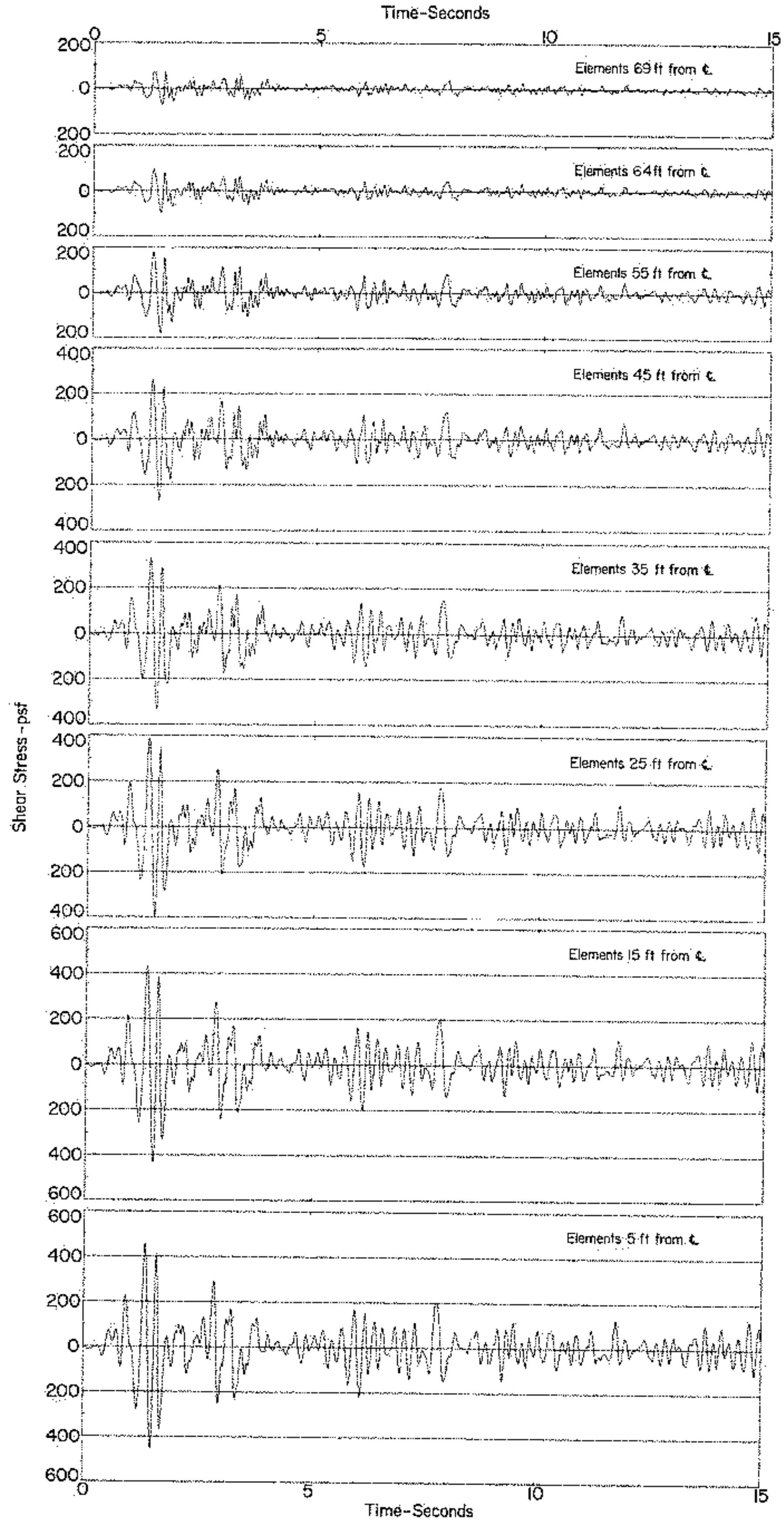



FIG.37 TIME HISTORIES OF DYNAMIC SHEAR STRESSES DEVELOPED ALONG BASE OF DAM IF ANALYSIS CONDUCTED DIRECTLY FOR FULL 15 SECONDS DURATION OF GROUND MOTION.

during the remainder of the earthquake. By appropriate weighting the various stress cycles, based on the laboratory test data, it is possible to determine, for each of the stress history plots shown in Fig. 37, an equivalent series of uniform cyclic stress applications. Thus, for example, it may be determined from the test data that the various stress history plots shown in Fig. 37 are for practical purposes equivalent to the following sequences of uniform cyclic stress applications.

| Distance of soil element from center-line of embankment | Equivalent uniform cyclic stress sequence |
|---|---|
| 5 ft | 10 cycles at 290 psf |
| 15 | 10 cycles at 270 psf |
| 25 | 10 cycles at 250 psf |
| 35 | 10 cycles at 210 psf |
| 45 | 10 cycles at 170 psf |
| 55 | 10 cycles at 125 psf |
| 64 | 10 cycles at 70 psf |
| 69 | 10 cycles at 50 psf |

It should be noted that although the equivalent uniform cyclic stress sequences are all expressed in terms of 10 equivalent stress cycles in the above table, there are in fact a number of equivalent uniform cyclic stress sequences for any one stress history plot.

With the aid of the information shown in Figs. 32, 33 and the above table it is possible to make a simple evaluation of the extent of soil liquefaction along the base of the embankment during the 15 second period of earthquake ground motions. The steps in this procedure are shown in Fig. 38. The initial stress conditions for soil elements along the base

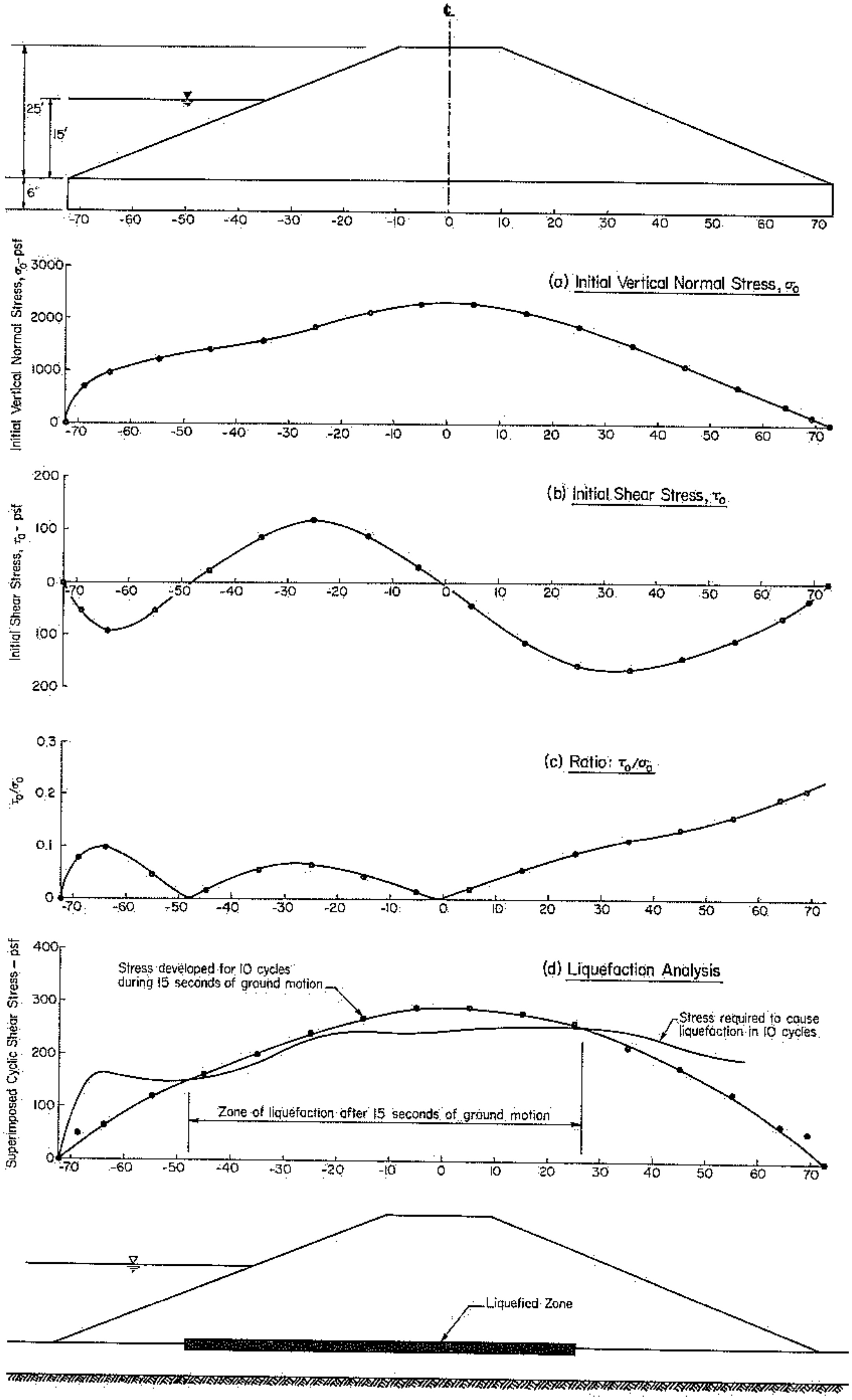



FIG. 38 DETERMINATION OF ZONE OF LIQUEFACTION IF ANALYSIS IS CONDUCTED DIRECTLY FOR FULL 15 SECONDS OF GROUND MOTION.

of the embankment are shown in parts (a), (b) and (c) of this figure. For any point along the base, it is readily possible, from a knowledge of the vertical normal stress shown in Fig. 38(a) and the initial ratio of $\tau/\sigma$ shown in Fig. 38(c), to read off from the test data in Fig. 33, the cyclic shear stress which would be required to cause liquefaction or failure of this element in 10 cyclic applications. By determining such values for a number of points along the base, the distribution of stresses required to cause liquefaction in 10 cycles for all elements along the base of the embankment can be plotted as shown in Fig. 38(d). By superimposing on this same plot the equivalent uniform cyclic stresses, developed for 10 cycles during the 15 second period of ground motions, as listed in the table on page 43, the extent of the zone in which liquefaction would be expected to occur can be evaluated. Thus it may be seen that over a zone extending from about 26 ft downstream from the center-line to about 48 upstream from the center-line, the induced equivalent stresses determined by the response analysis exceed those required to cause liquefaction. Presumably, therefore, liquefaction would occur in this zone as indicated in the bottom part of Fig. 38. However, in other sections of the base of the embankment, liquefaction would apparently not occur.

It may be noted that due to the asymmetry of the initial stress conditions, the liquefied section along the base would extend somewhat further upstream than downstream. However in spite of the fact that liquefaction would apparently extend over a substantial portion of the base, the analysis does not indicate that failure would occur. If liquefaction developed as shown in Fig. 38, there would be a tendency for small upstream and downstream movements of the embankment. However any significant upstream movement would be resisted by the water pressure on the impervious upstream face, and the shearing resistance developed along the non-liquefied portion of the base near the downstream toe would be more than sufficient to withstand the lateral forces tending to cause downstream sliding. Thus the analysis presented in Fig. 38 does not provide a full explanation of the observed mode of failure of the dam. This deficiency of the analytical procedure can readily be overcome, however, if consideration is given to the progressive nature of liquefaction development as outlined in the following section.

## Progressive Development of Liquefaction Along Base of Embankment

In the preceding section of this report the extent of the zone of soil liquefaction along the base of the embankment was determined by considering, in effect, that all liquefaction developed simultaneously after 10 cycles of stress application induced during 15 seconds of ground motions. In fact, as is apparent from Fig. 38(d), some sections liquefy more easily, and thus more quickly, than others, resulting in a significant redistribution of both static and dynamic stresses in the non-liquefied elements during the remainder of the earthquake.

This is readily illustrated by considering the conditions in the embankment after, say, only 2 seconds of earthquake ground motions. The stress history plots in Fig. 37 would be applicable up to this time and may be used to analyze the extent of the liquefied zone after 2 seconds of ground shaking, by a procedure similar to that described in Fig. 38. In this case the soil elements would be subjected to only 2 significant stress cycles and it would be necessary to use the test data shown in Fig. 34 for analysis purposes.

The results of a study of conditions after 2 seconds of earthquake ground motions are shown in Fig. 39. Fig. 39(d) shows a comparison of the stresses developed for two cycles along the base of the embankment by the earthquake and the stresses required to cause liquefaction in 2 cycles. It is readily apparent that even after this short period of time, the analysis indicates a zone of liquefaction extending from about 20 ft downstream of the center-line to about 40 ft upstream of the centerline. As a consequence there would be a series of changes in stress conditions as follows:

(1) There would be a tendency for some spreading of the base of the embankment with the result that the lateral stress transmitted across the center-line of the embankment would drop to a level approaching a minimum active pressure value.

(2) There would be a redistribution of the static shear stresses along the base of the embankment; in the liquefied section the shear stress would drop to zero and in the non-liquefied sections, the stress distributions would accommodate themselves to the new stress conditions in the embankment. Thus on the downstream side, for example, the shear stresses would only have to resist the reduced lateral force resulting from the tendency for base-spreading. On the upstream side the shear stresses would have to maintain a balance between the lateral pressure of the water in the reservoir and the internal stresses in the embankment. Taking into account these considerations it is considered that the new distribution of shear stresses would be something similar to that shown in Fig. 40(b). Since the vertical normal stresses would be essentially unchanged, the ratio of $\tau/\sigma$ for

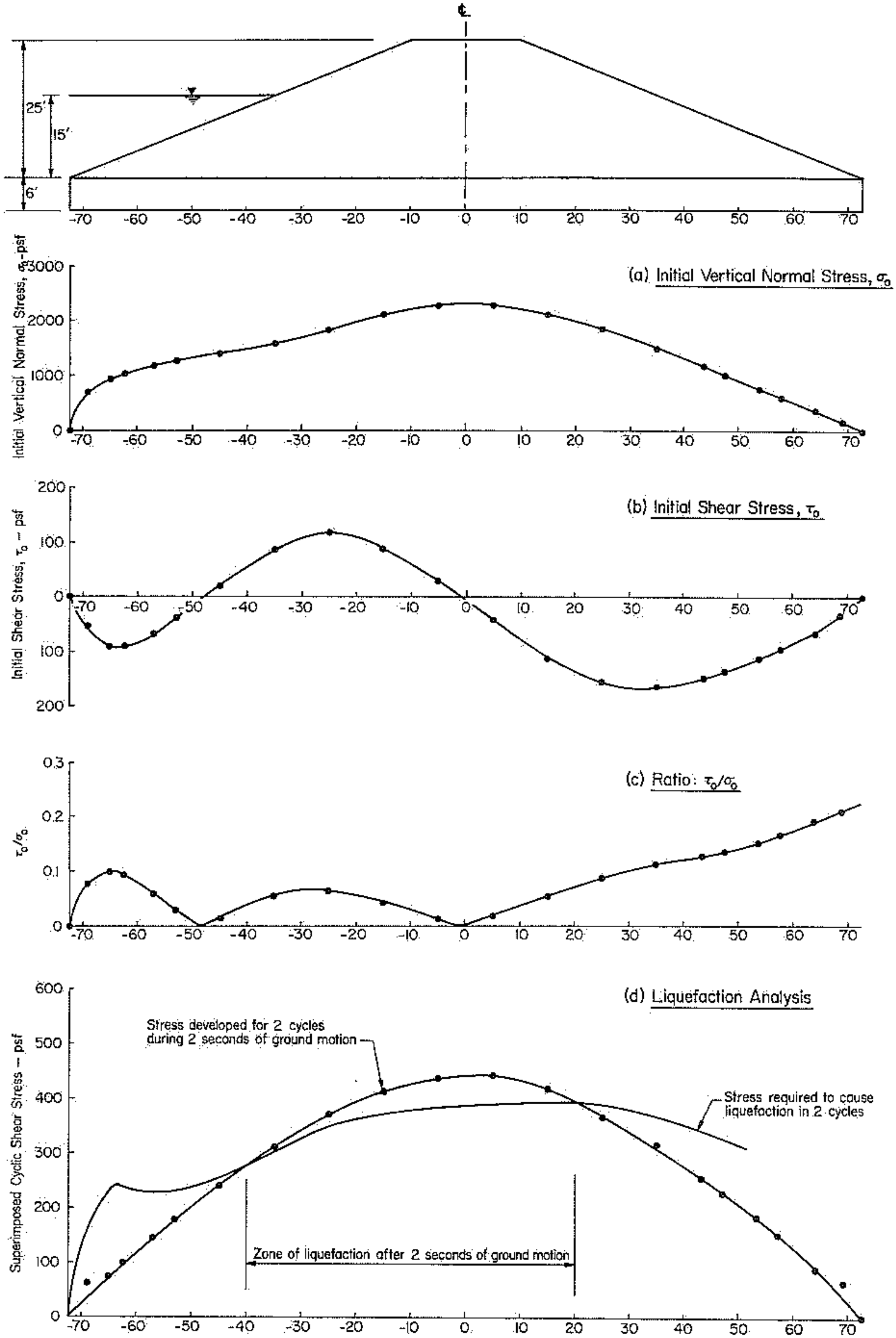



FIG. 39 PROGRESSIVE LIQUEFACTION ANALYSIS: DETERMINATION OF ZONE OF LIQUEFACTION AFTER 2 SECONDS OF GROUND MOTION.

dead load stresses along the base of the embankment would have values similar to those shown in Fig. 40(c).

(3) The development of liquefaction over a substantial portion of the base of the embankment would lead also to a re-distribution of dynamic stresses. This can be studied by dropping the shear moduli close to zero in the liquefied zone for the ensuing period of the dynamic response analysis. Under these conditions, the shear stresses developed in the various elements along the base of the embankment have the values shown in Fig. 41. It may be seen that in the liquefied zone, the shear stress remains essentially at zero for the remainder of the earthquake. However in elements adjacent to the liquefied zone, very large shear stresses are developed in the period from 2 to 4 seconds of the earthquake motions.

In view of these high stresses it becomes pertinent to investigate the extent of liquefaction after 4 seconds of ground shaking. This may readily be accomplished following the procedures previously described. The static stress conditions in Figs. 40(a) and 40(c) together with the test data in Fig. 35 provide a means for evaluating the cyclic shear stresses required to cause liquefaction for elements along the base of the embankment in 4 cycles of stress application. The computed stresses in Fig. 41 permit an evaluation of the equivalent uniform stresses induced for 4 cycles in the various elements. Comparison of these values in Fig. 40(d) indicates that a substantial additional zone of soil will liquefy in the 2 to 4 second period of the ground shaking.

This procedure can now be repeated, progressively for other portions of the total period of earthquake ground motions. Fig. 42 shows an

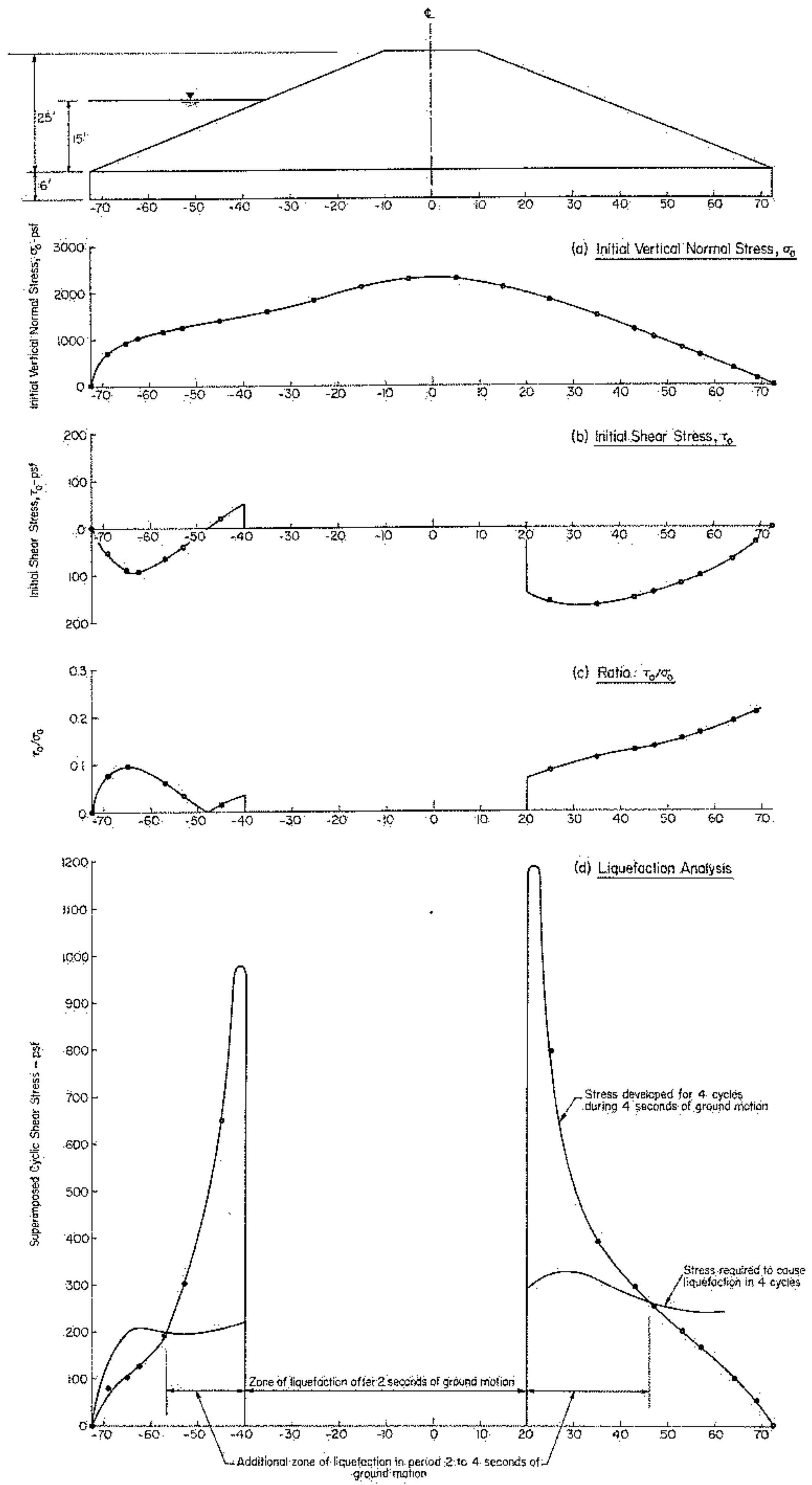



FIG. 40 PROGRESSIVE LIQUEFACTION ANALYSIS: DETERMINATION OF ZONE OF LIQUEFACTION AFTER 4 SECONDS OF GROUND MOTION.

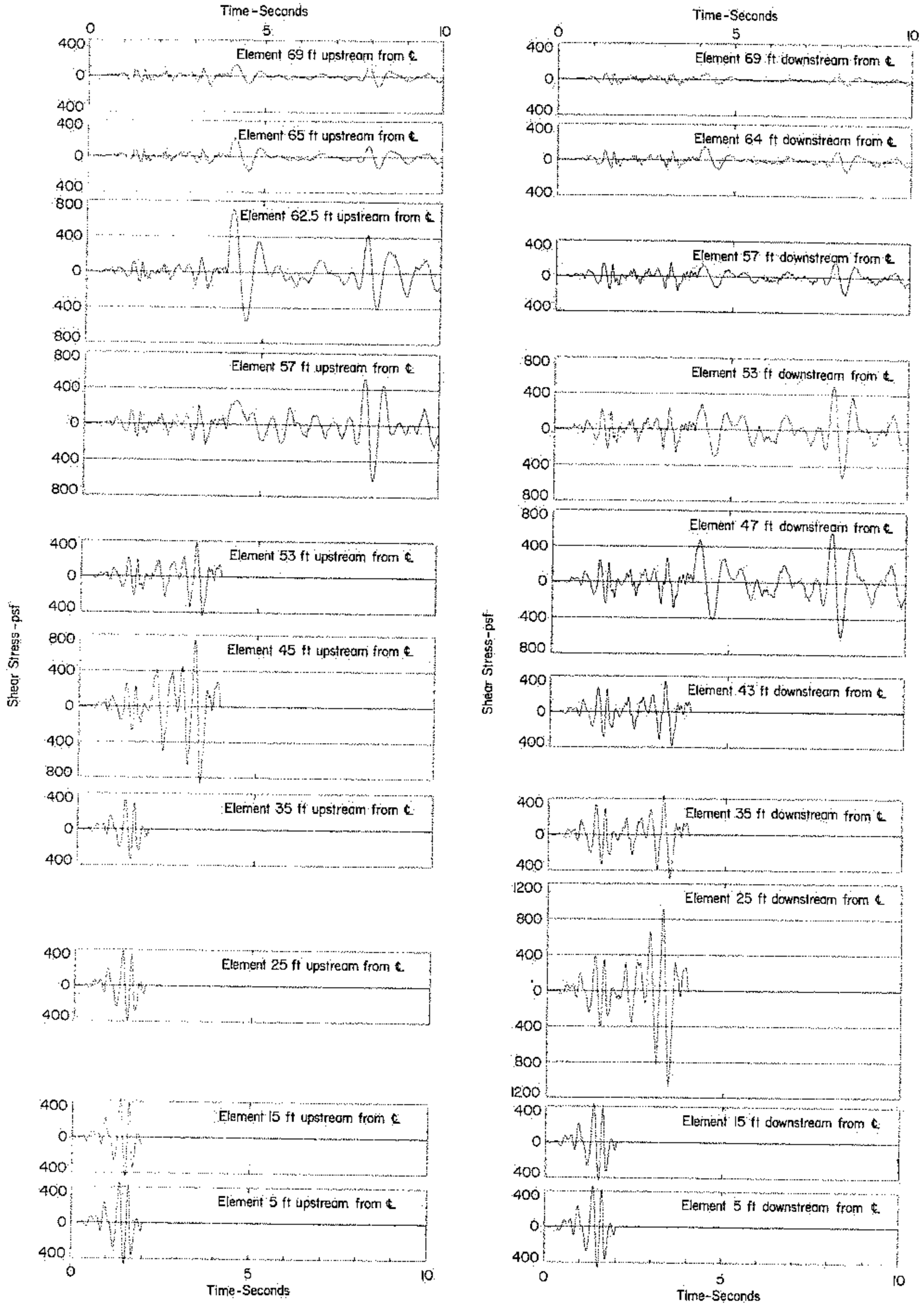



FIG. 41 PROGRESSIVE LIQUEFACTION ANALYSIS: TIME HISTORIES OF DYNAMIC SHEAR STRESSES DEVELOPED ALONG BASE OF DAM.

evaluation of the static stress condition along the base of the embankment after 4 seconds, and the additional liquefaction likely to occur in the ensuing 6 second period. The dynamic stresses induced in soil elements along the base of the embankment during the 4 to 10 second period of earthquake ground motions are shown in Fig. 41.

The progressive development of the extent of the liquefied zone along the base of the embankment, corresponding to the analyses shown in Figs. 39, 40 and 42, is summarized in Fig. 43. It may be noted that by taking into account the progressive nature of this development, the extent of this zone after 10 seconds of the earthquake is substantially larger than that shown by the analysis in Fig. 38, where progressive action was not considered.

In fact, the inclusion of the progressive nature of liquefaction in the analysis, leads to a potentially improved understanding of the observed performance of the Sheffield Dam. If the extent of foundation liquefaction after 10 seconds of earthquake ground motions were similar to that shown in Figs. 42 and 43, the embankment would be expected to slide downstream for the following reasons:

(1) Lateral movements along the base of the embankment would tend to reduce the internal horizontal force to a value approaching the active pressure value; this can be estimated to be less than about 5000 lbs per ft length of embankment.

(2) The horizontal water pressure due to 15 to 18 ft of water in the reservoir would be between 6200 and 9400 lbs per ft length of embankment. Since these pressures would act on the impervious upstream face of the embankment, they would be more than sufficient to prevent sliding in the upstream direction.

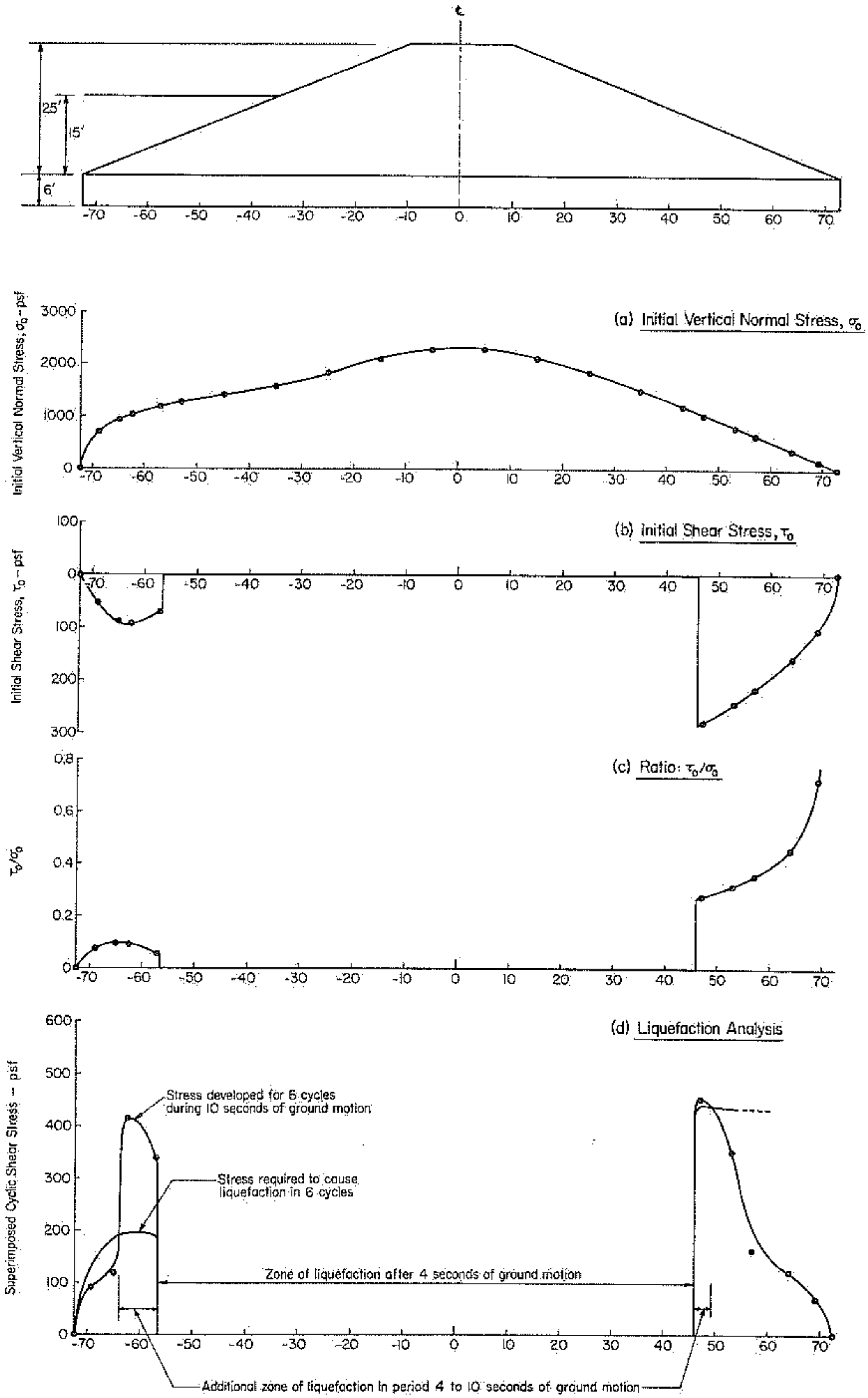



FIG. 42 PROGRESSIVE LIQUEFACTION ANALYSIS: DETERMINATION OF ZONE OF LIQUEFACTION AFTER 10 SECONDS OF GROUND MOTION.

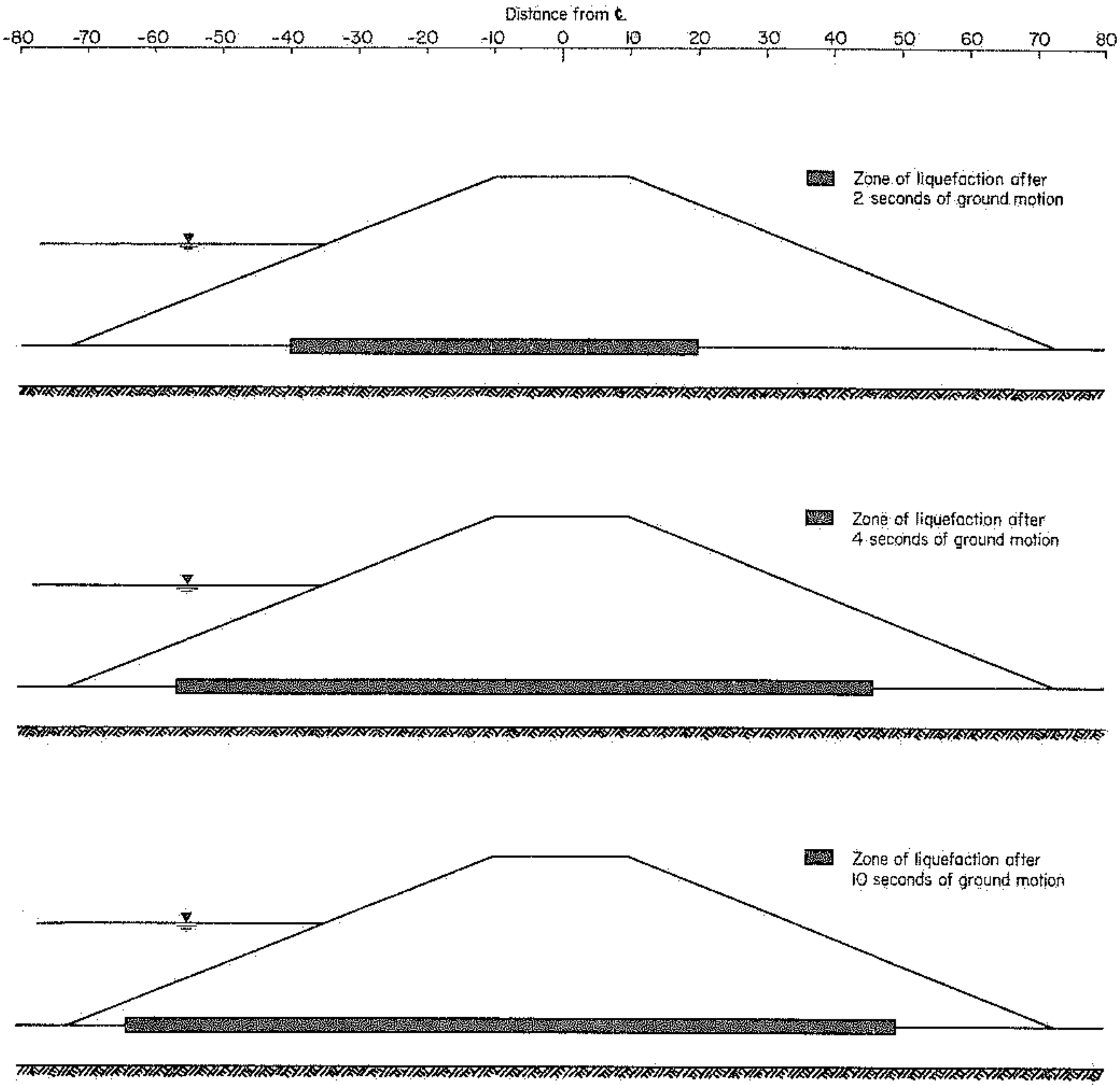



FIG. 43 PROGRESSIVE LIQUEFACTION ALONG BASE OF EMBANKMENT.

(3) In fact, since the horizontal water pressure exceeds the internal horizontal soil pressure and liquefaction extends virtually all the way to the upstream toe of the embankment, the small section of non-liquefied soil near the downstream toe would be called upon to resist the horizontal water pressure. Since the undrained shearing resistance of the soil in the nonliquefied section is only about 7000 lbs per foot length of embankment, a water depth of 16 ft in the reservoir would produce sufficient pressure to cause a slide movement of the entire embankment along its base; this depth of water is within the range believed to exist at the time of the earthquake. Sliding would also be facilitated by the continuing earthquake ground motions, which would help to maintain the soil in a liquefied condition and increase pore water pressures in non-liquefied sections of the base.

Thus a slide involving the downstream movement of the entire embankment might be expected to develop after about 10 seconds of ground shaking. As the slide occurred and water was released from the reservoir, the water pressures on the upstream face would progressively decrease until they could no longer hold the embankment together, at which stage the embankment would collapse by spreading laterally, leaving the overall appearance shown in Fig. 3.

In addition to throwing light on the manner in which failure might well have developed, the foregoing analysis also indicates that minor changes in conditions might have led to a totally different pattern of behavior. For example, if the depth of water in the reservoir had been only 5 or 10 ft rather than 15 to 18 ft, the horizontal water pressure might well have been insufficient to prevent sliding in the upstream direction while the non-liquefied zone near the downstream toe might have resisted downstream movement. In this case the direction of sliding would have been completely the reverse of that which actually occurred.

The reasonable agreement between the performance predicted by this analytical procedure and the observed behavior of the Sheffield Dam would seem to indicate its potential value for analyzing problems of this type. At the same time the complexity of the failure mechanism, as indicated by the progressive changes in stress patterns once failure is initiated, indicates the potential dangers of placing undue reliance in over-simplified procedures for analysis purposes.

## Summary and Conclusions

In the preceding pages an attempt has been made to reconstruct the circumstances leading to the failure of the Sheffield Dam during an earthquake near Santa Barbara in 1925. The dam, with a maximum height of 25 ft and a length of 720 ft was constructed of silty sand from the reservoir excavation on a foundation of similar material about 6 ft thick. The upper 1 to 1.5 ft of the foundation soil was substantially looser than the deeper material, having a dry density of about 90 lb per cu ft corresponding to a degree of compaction, based on the Standard AASHO compaction test of about 75 percent or a relative density of about 35 percent. The embankment material, which was compacted only by the construction equipment, probably had similar properties.

The upstream slope of the dam was faced with an impervious blanket which was extended about 10 ft into the foundation material to serve as a cut-off wall. However both the foundation material and the lower portion of the embankment were saturated by seepage around and underneath the cut-off. The water in the reservoir was about 15 to 18 ft deep at the time of the earthquake.

The earthquake which caused the failure was a shock of magnitude about 6.3 with an epicenter believed to be located about seven miles from the dam-site. The resulting ground accelerations at the site probably had a maximum amplitude of about 0.15g, a predominant frequency of about 3 cycles per second and a duration of significant shaking of 15 to 18 seconds.

The dam failed by a central section, about 300 ft in length, sliding downstream about 100 ft along a slip surface near the base of the embankment.

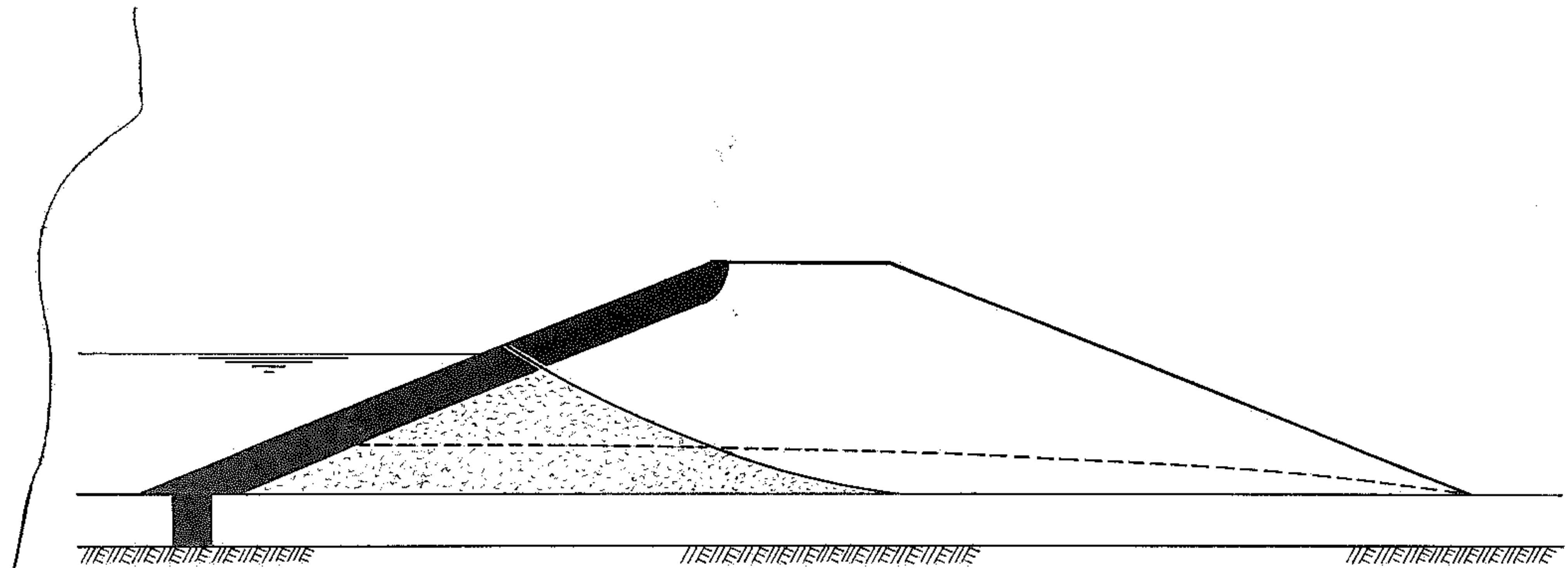
To study the failure, a comprehensive series of strength tests were performed on samples of silty sand obtained from the foundation deposits at the dam-site and recompacted to approximately the same density as the foundation and embankment material. These tests included conventional consolidated-undrained static loading tests, consolidated-undrained triaxial compression tests under cyclic loading conditions and consolidated-undrained simple shear tests under cyclic loading conditions. These data, together with the information concerning conditions at the time failure occurred, were used as a basis for investigating the applicability of various procedures for analyzing the seismic stability of the embankment.

It was noted that possible variations in choices of computational details make it possible to obtain a wide range of computed results using a conventional pseudo-static analytical approach. However using the most common choices of analytical details leading to essentially the lowest values for the computed factor of safety, it was found that the incorporation of a static seismic force represented by a seismic coefficient of 0.1, led to a computed factor of safety in the range 1.0 to 1.2. However sliding along the critical failure surface indicated by this analysis would not have led to any loss of freeboard or to a catastrophic failure of the entire embankment section. The seismic coefficient required to indicate a condition of marginal stability was found to be in the range 0.1 to 0.17. However if seismic coefficients of this order of magnitude are appropriate for the type of earthquake experienced by the Sheffield Dam, it would appear to be necessary to use much higher values, of the order of 0.3 or more, to analyze similar types of failure for the largest earthquakes which might be expected to occur in California. Finally, for the conditions believed to exist at the time failure occurred, it was found that a seismic coefficient of about 0.15 to 0.21 would have been required in a pseudo-static analysis to predict failure along any sliding surface approximating the actual failure surface.

Dynamic response analyses indicate that during the earthquake the body of the dam was probably subjected to a series of cycles of earthquake-induced inertia forces which might be represented by about 10 uniform force cycles with a maximum amplitude corresponding to a seismic coefficient of 0.15. A dynamic stability analysis procedure incorporating this result together with rational choices of computational details and strength values determined by cyclic loading triaxial compression tests indicated a factor of safety of 1.15 against catastrophic failure due to soil liquefaction and a predicted failure surface in close proximity to that on which the sliding developed during the earthquake. When the results of cyclic loading simple shear tests were used in the same type of analysis, the computed factor of safety against a liquefaction failure was 0.75, and the predicted failure surface was in excellent agreement with that deduced by engineers who inspected the dam after the failure occurred.

Finally, a finite element analysis of the anticipated behavior of the embankment was conducted and used in conjunction with the cyclic loading simple shear test data to investigate the stability of the embankment. This analysis showed that liquefaction of soil near the base of the embankment would be likely to develop progressively, starting near the center line and extending towards the toes, progressing somewhat faster upstream than downstream. However the water pressure on the upstream impervious face of the dam would be sufficient to prevent sliding in this direction and would ultimately have to be resisted by the non-liquefied zone of soil near the downstream toe. This zone would progressively decrease in length until after about 10 seconds, its resistance would be insufficient to withstand the pressure due to about 16 ft of water in the reservoir and the entire embankment would slide downstream. As it did so the depth of water pressing on the upstream face would decrease and ultimately the water pressure would be insufficient to hold the embankment together leading to a collapse of the embankment due to lateral spreading. This mode of failure is also in excellent accord with the apparent behavior of the dam during the earthquake.

Probably the most convenient method of comparing the results of the various stability analysis procedures is to compare the values of the seismic coefficients which would have to be incorporated in them to predict failure along slip surfaces approximating that on which failure actually occurred. On the basis of the opinions of post-failure observers and the consequences of failure, this surface must have been near the base of the dam, intersecting the upstream face of the dam below the reservoir level. Thus for all practical purposes it can be considered to be located within the zone shown in Fig. 44. For surfaces in this zone,



| Method of Analysis | Seismic Coefficient Required to Indicate Failure for Critical Slip Surface in Probable Failure Zone (shaded) |
|---|---|
| Pseudo-static analyses using strength parameters from static loading consolidated-undrained tests (total stresses) | 0.15 to 0.21 |
| Analyses using test data from cyclic loading anisotropically consolidated-undrained triaxial compression tests (10 cycles) | 0.17 to 0.19 |
| Analyses using test data from cyclic loading simple shear tests (10 cycles) | 0.11 |

Earthquake Characteristics: Maximum ground acceleration about 0.15g; duration of strong shaking about 15 seconds

Response Analysis: Indicates about 10 cycles with seismic cofficient equal to 0.15 for slides extending to base of embankment

Fig. 44 - SUMMARY OF ANALYTICAL RESULTS.

and for the conditions believed to exist in the embankment at the time of failure, values of the seismic coefficient, $k_f$, leading to a computed factor of safety of unity using the various analytical procedures are shown in Fig. 44. Using a pseudo-static analysis it was found that the value of $k_f$ would have to be in the range of 0.15 to 0.21; using a dynamic analysis with cyclic loading triaxial compression test data the value of $k_f$ would be about 0.17 to 0.19; and using a dynamic analysis with cyclic loading simple shear test data the value of $k_f$ would be about 0.11.

The significance of these values must be judged in the light of the fact that the earthquake inducing failure was of strong but not major intensity, probably inducing a maximum ground acceleration of about 0.15g at the dam-site, a duration of shaking of about 15 seconds and effects comparable to a series of about 10 cycles of inertia forces represented by a seismic coefficient of about 0.15. With this framework it seems appropriate to draw the following conclusions:

(1) Conventional pseudo-static analyses do not appear to provide a satisfactory basis for predicting the type of slide which led to catastrophic failure of the Sheffield Dam during the Santa Barbara earthquake of 1925. Depending on the strength values and water level elevations considered in the computations, this method of analysis either indicates an adequately stable section or it predicts failure on an incorrect sliding surface if the seismic coefficient is equal to about 0.66 to 1 times the maximum ground acceleration likely to have developed during the earthquake and conservative choices of analytical details are used in the analyses; to predict failure on a surface approximating that on which sliding occurred it would be necessary to use a seismic

coefficient equal to about 1 to 1.4 times the maximum ground acceleration likely to have developed during the earthquake.

(2) The dynamic analysis procedure incorporating test data from cyclic loading triaxial compression tests predicts catastrophic sliding along a surface approximating the failure surface if the ground motions had been 10 to 20 percent higher than those which probably developed.

(3) The dynamic analysis procedure incorporating test data from cyclic loading simple shear tests correctly predicts a catastrophic failure by sliding along the base of the embankment as a result of soil liquefaction but without the factor of safety dropping to unreasonably low values. Thus this approach appears to provide a satisfactory basis for analyzing the Sheffield Dam failure.

(4) The complexity of the probable mechanics of failure and the most satisfactory analysis of the actual behavior of the Sheffield Dam is provided by a dynamic analysis using finite element techniques in conjunction with cyclic loading simple shear test data. However the computational techniques are considerably more involved than the methods discussed above.

It should be noted that failure by sliding along the base of the embankment might have resulted from the fact that the embankment soil was somewhat stronger than the surface layer of foundation material. However this condition would not significantly affect the quantitative results presented above for sliding along the observed failure surface.

The authors also recognize that somewhat more elaborate analyses of the progressive development of liquefaction along the base of the dam could have been made with the aid of recent knowledge concerning relationships between soil damping factors and strains and techniques for applying programmed cyclic stress applications to soil samples. However it was considered that the dynamic analysis procedure adopted was adequate for the purposes of the present study.

In the light of the fact that both field observations and dynamic analyses indicate that the sliding of the Sheffield Dam was due to a liquefaction failure of the loose saturated silty sand near the base of the embankment, it is of interest to conjecture how the dam would have behaved if the earthquake ground motions had been smaller in intensity or the soil near the base of the embankment had been somewhat denser. For this purpose it would seem reasonable to assume that the same type of analysis which predicts the observed performance would be adequate to evaluate the effects of minor changes in conditions.

On this basis, the dynamic analysis incorporating cyclic loading simple shear test data would indicate that the earthquake would have had no significant effects on the stability of the embankment if

(1) the earthquake had been smaller in magnitude or had occurred at a greater distance from the dam-site so that the maximum ground acceleration would have been only about 0.1g rather than the probable value of 0.15g; or

(2) the relative density of the foundation and embankment soils had been about 55 percent rather than the actual value of about 35 to 40 percent; this estimation is based on the experimental observation[15,16] that for samples of saturated sand initially consolidated under $K_o$ conditions and with relative densities less than

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[16] Lee, Kenneth L. and Seed, H. B., "Cyclic Stress Conditions Causing Liquefaction of Sand," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 93, No. SM1, January, 1967.