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Appendix II.—Notation

The following symbols are used in this paper:

$A_i \ldots R_i$ = arbitrary constants;
E = Young's modulus;
E_m = Young's modulus of medium;
G = shear modulus;
G_m = shear modulus of medium;
P = field stress in vertical direction;
Q = field stress in horizontal direction;
R_o = outer radius of liner;
t = thickness of liner;
u_r = displacement in radial direction;
u_θ = displacement in tangential direction;
μ = Poisson's ratio;
$\tau_{r\theta}$ = shearing stresses;
σ_r = radial stresses; and
σ_θ = tangential stresses.

Subscripts

1 and 2 = liner and to the medium, respectively.

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Seismic Stability and Deformation of Clay Slopes[a]

By Ignacio Arango[1] and H. Bolton Seed,[2] Members, ASCE

Introduction

It has long been recognized that failures of earth slopes constitute one of the major causes of damage during earthquakes. Major landslides due to this cause have been reported dating back to 383 BC, but the large majority of the slides have taken place in cohesionless soils due to liquefaction phenomena (9). Even in the few reported cases of slides occurring in clay soils (2,10,14), the cause of sliding has often been attributed to liquefaction of sand seams and lenses within the clay. The great scarcity of cases of slides in homogeneous clay deposits in spite of the large number of slides reported, gives cause for speculation whether cohesive soils are vulnerable to seismically-induced slope stability. However, the increasing number of slopes being constructed in clay soils makes it highly desirable to determine a satisfactory answer to this question.

Since the problem is unlikely to be clarified by field studies in the near future, it seemed desirable to undertake a special series of tests on small-scale embankments, which have proven useful for clarifying other aspects of soil behavior in previous studies, with the object of determining: (1) Whether or not sliding can be induced in homogeneous banks of clay subjected to reasonable levels of seismic excitation: (2) the mechanics of any slide movements which might occur: (3) quantitative data on conditions under which sliding may develop: and (4) the relationship between the results obtained from the small-scale embankments and those obtained by means of less sophisticated laboratory

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tests such as simple shear or cyclic triaxial, so that prediction of the behavior of prototype structures in the field could be advanced. It was for these purposes that the investigation described in the following pages was undertaken, and it is hoped that this presentation of results will encourage engineers who have studied or observed seismically-induced slides in clays in the past to record their experiences.

Small Embankment Tests

Equipment, Materials, and Testing Procedure.—The equipment used consisted of a horizontal shake table 7 ft wide by 10 ft long on which the clay banks

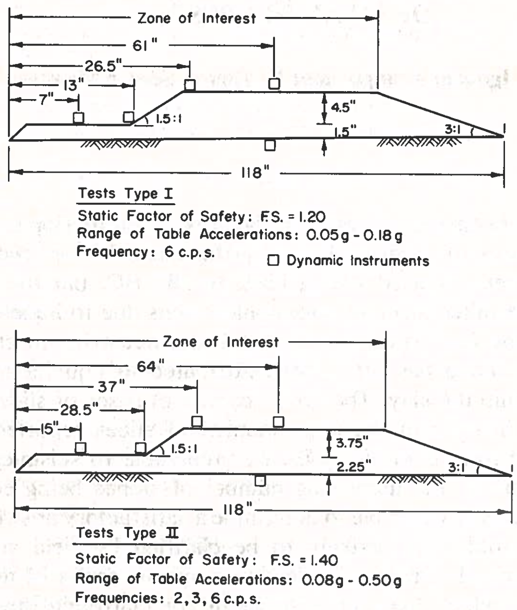

FIG. 1.—Test Bank Geometries and Dynamic Variables

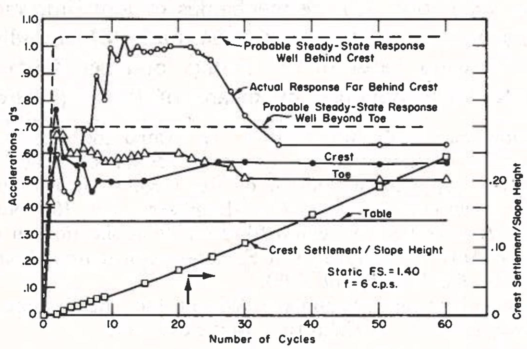

FIG. 2.—Acceleration and Deformation Data: Test No. 12

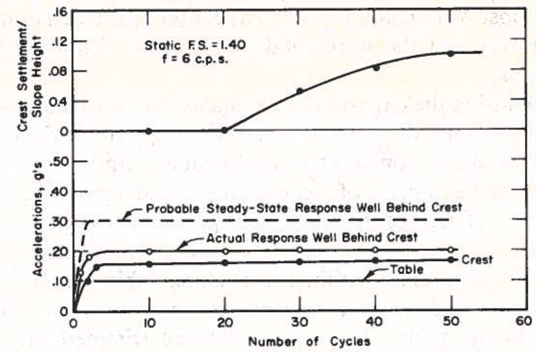

FIG. 3.—Acceleration and Deformation Data: Test No. 21

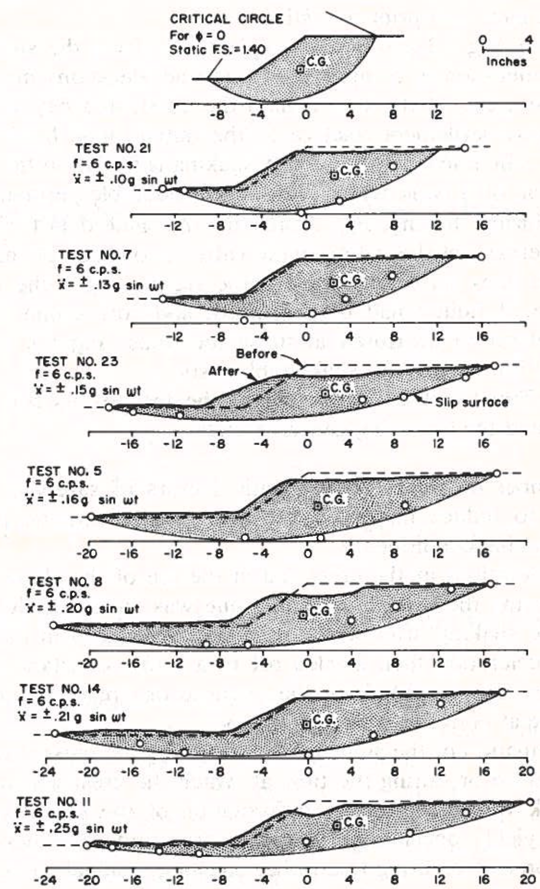

FIG. 4.—Bank Profiles After Testing

were constructed. An MTS actuator attached to the table produced the simulated earthquake motions. A six-channel Sanborn recorder provided a continuous record of the accelerations and the horizontal and vertical displacements at selected points on the banks.

The clay used during the experiments was a mixture of Kaolinite and Bentonite with the proper amount of water so that slopes with static factors of safety of 1.20 and 1.40 could be constructed on the table with the geometric properites shown in Fig. 1. The ranges of accelerations and operating test frequencies utilized with each of the geometrical configurations are also indicated in the same figure.

A 10-ft by 5-ft wide area within the shaking table was provided with two 6-in. high aluminum end forms to confine the clay laterally. The clay mixture was pumped into the table, the excess material trimmed off, and the whole setup protected with an impervious membrane which would prevent loss of moisture. The end slopes were trimmed about 24 hr after clay placement. Accelerometers, LVDT's and fixed reference markers were then placed on the surface of the bank just prior to testing.

Test Results.—Figs. 2 and 3 show typical data from the shaking table tests. The table acceleration is compared with the accelerations measured at points located on the crest, at the toe, behind the crest, and beyond the toe of the slope. The crest settlement relative to the initial slope height is also shown in the figures. The number of cycles of shaking is shown in the abscissa. While Fig. 2 is typical of a bank which showed considerable permanent deformation of the slope during shaking, Fig. 3 presents the same data for a bank showing only a slight permanent shear deformation after a considerable number of cycles. The two dash lines on each one of these figures show the probable steady state response, if failure had not occurred, and correspond to the maximum value of acceleration measured at those locations. Figures such as 2 and 3 were prepared for each of the shake table tests.

Analysis of Test Results.—The study of the test results for which Fig. 2 is typical, indicated the following general trends:

1. In a number of slopes having static factors of safety of 1.2 and 1.4, it was possible to induce major displacements representative of failure under earthquake-shaking conditions.
2. The accelerations at the crest and at the toe of the slope reached a peak value early during the test. This peak value was sustained during one or two cycles of table shaking after which a considerable drop in its amplitude took place. The acceleration then leveled off to a rather constant value during the rest of the test. The amplitude of this reduced or "residual" acceleration was about the same at both the crest and the toe.
3. No deformation of the slope (as measured by the crest vertical settlement) takes place before or during the time at which the crest and toe accelerations show the peak values. However, deformation of the slope starts as soon as the peak or "yield" acceleration starts to drop and continues to occur while the acceleration is decreasing to a rather constant residual value.
4. At a given frequency, the amplitude of the peak acceleration, at both the crest and the toe, increases as the table acceleration increases.
5. For a given amplitude of table acceleration and at least for the range of frequencies used in the tests during this investigation, the lower the frequency of the motion, the lower was the amplitude of the peak or "yield" acceleration observed.

Extent of Yielding Soil Mass; Average Shear Strain.—The final slope configurations after shaking as determined by the various measuring instruments and by profiles taken along the longitudinal axes of some of the test banks are shown in Fig. 4. It may be observed that for a given frequency, the larger the amplitude of the table acceleration, the larger the extent of the deformation of the clay bank. Since the external configuration of the yielding soil mass

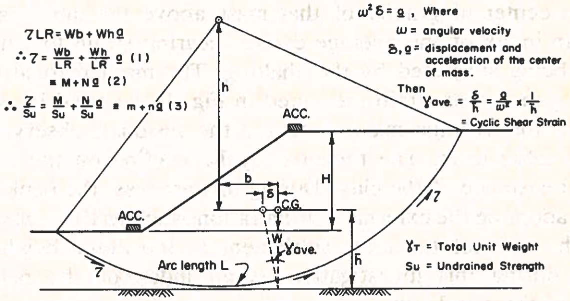

FIG. 5.—Dynamic Equilibrium Equations and Definition of Average Shear Strain (Cyclic)

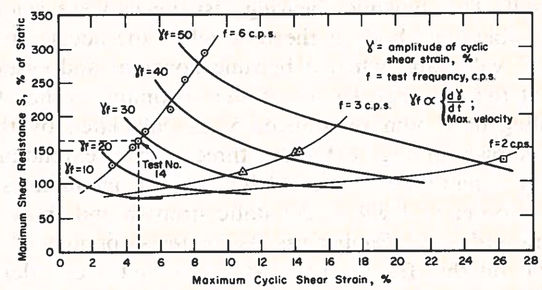

FIG. 6.—Maximum Shear Strain, Rate of Shear, Velocity, and Maximum Shear Resistance

was known, it was desirable to determine the depth of the slip surface and the extent of mass involved during the slope deformation. The approximate method reviewed in Ref. 6 was used to estimate the depth of the slip surfaces. The calculated depths are shown by open circles in Fig. 4. Arcs of circles best fitting the calculated depths were next drawn as shown in the same figure.

The equilibrium of the soil mass bounded by the slope surface and the circular failure can readily be written as shown in Fig. 5.

Evidence presented earlier indicates that the accelerations at the crest and toe of the slope tend to equalize once a portion of the clay bank starts to yield, suggesting that the sliding soil mass behaves as an essentially rigid body and moves at a relatively uniform acceleration. Based on this, the concept of "average acceleration" was developed. The "average acceleration" is that acceleration at which the whole sliding soil moves at a particular time. This average acceleration has a peak value or "maximum" at the onset of the deformation process and a "residual" value which develops while the deformation takes place.

Assuming that the sliding mass of soil behaves as a rigid body subjected to cyclic accelerations and consequently, displacements, then the relative cyclic deformation of that rigid body with respect to the base might be used as an indication of the cyclic shear strain within the soil. The ratio of the average cyclic displacement, whether maximum or residual, of the sliding mass to the height of the center of gravity of that mass above the table base might then be used as an index of the average cyclic shearing strain to which the sliding soil mass is being subjected by the shaking. The mathematical expression for the average cyclic shear strain presented in Fig. 5 was used in calculating both cyclic strains, the average maximum, and the residual, observed during each of the shake table tests. The repeated cyclic loading on the slope decreases the shearing resistance of the clay. During this process, the bank slope deforms permanently adopting the external configurations such as those previously shown in Fig. 4. The ratio of the crest settlement to the initial height of the slope was chosen during this investigation as an index of the permanent shear deformation of the clay bank.

Soil Strength During Dynamic Testing.—Eq. 1 in Fig. 5 shows that the dynamic shearing resistance acting along any given failure surface is a function of the average acceleration acting on the sliding mass and the geometrical characteristics of the mass itself. The dynamic shearing resistances were calculated for each of the shaking table tests both at the time when the acceleration reached the peak or "yield" value and when it became constant and essentially equal at the crest and at the toe. The values of the maximum cyclic shear strain and the corresponding maximum resistance, S, as calculated by the Eq. 1 given in Fig. 5, are shown in Fig. 6 by the three curves extending upwards and to the right. For example, Test No. 14 indicated a maximum shear resistance developed of the order of 178% of the static strength and an average maximum cyclic shear strain of 4.6%. Similar results for tests conducted at 6 cps defined the upper curve on this figure. Tests at 2 cps and 3 cps defined the lower curves. A similar plot relating the residual shearing resistance and cyclic strains was also prepared from the test data.

Fig. 6 shows that the larger the cyclic shear strain amplitude, the larger the shearing resistance mobilized by the soil. This trend, however, involves the combined effect of the amplitude of the shear strain and of the rate of shear. During harmonic motion, the frequency, f, and the period of vibration, T, are interrelated by the expression $f = 1/T$. The rate of shear is, therefore, proportional to the expression $\gamma/T = \gamma f$, which can also be shown to be a measure of the maximum velocity of the ground motion (1). Contours of equal values of the product, γf, were superimposed on the curves presented in Fig. 6. It is apparent from this figure that: (1) The shearing resistance mobilized increases as the rate of strain increases; and (2) at a given rate of shear strain, the larger the amplitude of the shear strain, the lower is the resistance mobilized by the material.

Crest Deformations.—The information presented so far indicates that the permanent deformation of the slope during an earthquake begins as soon as the acceleration within the bank of soil reaches a certain critical value which has been called the yield acceleration. An expression has been presented in Fig. 5, which relates the value of this yield acceleration and both the maximum dynamic shearing resistance, S, and the actual shearing resistance developed, τ, by the clay. The amount of permanent deformation which develops during

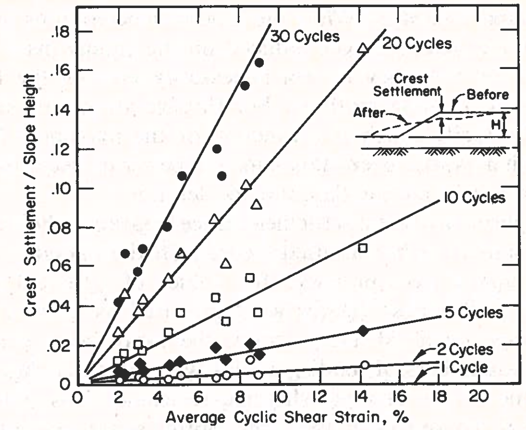

FIG. 7.—Average Shear Strain Versus Permanent Deformation—Shake Table Tests

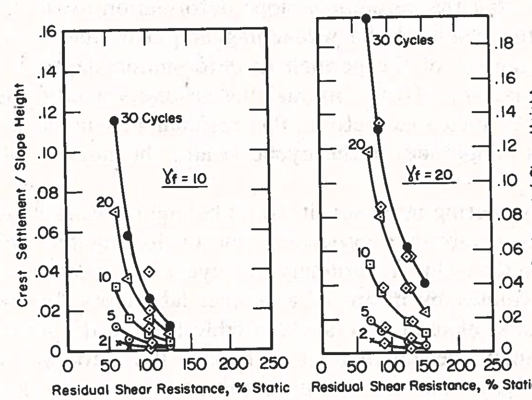

FIG. 8.—Residual Resistance Versus Permanent Deformation—Shake Table Tests

the earthquake depends on whether the resistance developed within the slope reaches the maximum dynamic soil resistance or not.

The data obtained indicate that when the accelerations developed within the slope have a value below the maximum acceleration obtainable from the clay slope under the given geometry and eqrthquake motion, the slope deformations are small and develop after a considerable number of stress cycles. The data also show that when the actual shearing resistance developed within the clay mass equals or exceeds the dynamic resistance, S, of the material: (1) At a given frequency, the larger the table acceleration, the larger the permanent crest deformation; (2) the larger the number of cycles of shaking, the larger the permanent slope deformation; and (3) at a given table acceleration and for a given number of cycles, the lower the frequency, the larger the permanent crest deformation. The relationship between the crest deformation, number of cycles, and corresponding values of average cyclic shear strain are shown in Fig. 7.

Influence of Ground Velocity.—While the ground accelerations provide a good index of the maximum lateral forces induced on the sliding mass of the slope, it has been shown (9) that they do not necessarily provide the best index of the damaging effects of these motions. For flexible structures such as a clay bank, the *ground velocity* which is a function of the maximum force and the time during which it acts, has been shown to be a better measure of the potential damaging effect of a base motion than the acceleration.

Test data on the permanent crest settlement of the slopes and their corresponding cyclic residual resistance can be classified according to the velocity proportionality factor, γf. Two typical plots prepared for values of γf equal to 10 and 20 are presented in Fig. 8. These figures were prepared as follows. For a given value of the average cyclic shearing strain, the ratio crest settlement/slope height after various numbers of loading cycles were obtained from Fig. 7. The corresponding value of the residual shearing resistance was obtained from a plot similar to Fig. 6. These values were then plotted separately for equal values of the parameter, γf, as shown in Fig. 8. Well defined curve trends are readily seen in these graphs.

Thus, it appears that the permanent slope deformation of the test banks can be explained as the result of the weakening action of the cyclic loading on the clay. The magnitude of the permanent deformation depends on the value of the residual resistance of the clay and the ground velocity imposed by the seismic loading. As indicated before, the residual resistance of the clay is determined by the amplitude of the cyclic strain, the number of cycles, and the rate of shear.

For practical engineering purposes, it would be highly desirable to use simpler laboratory tests to establish a procedure for evaluating the deformation of prototype slopes in the field. Accordingly, the cyclic stress strain characteristics of the clay were studied by means of a simpler laboratory test (simpler shear tests) and the results related to the shaking table tests in the hope of obtaining a practical and simple design method to bypass the elaborate shaking table tests. This test program and the results obtained from it are examined in the following section.

Dynamic Simple Shear Tests

Cyclic simple shear tests have been shown to provide a close simulation of the stress condition that a soil undergoes during seismic loading (5). Fig. 9 shows the cyclic motions induced in a soil element when shear waves are transmitted upward through the soil by an earthquake. The simple shear test is intended to reporduce the same type of deformations in soil samples.

Materials, Equipment, and Testing Procedure.—The soil used in this study

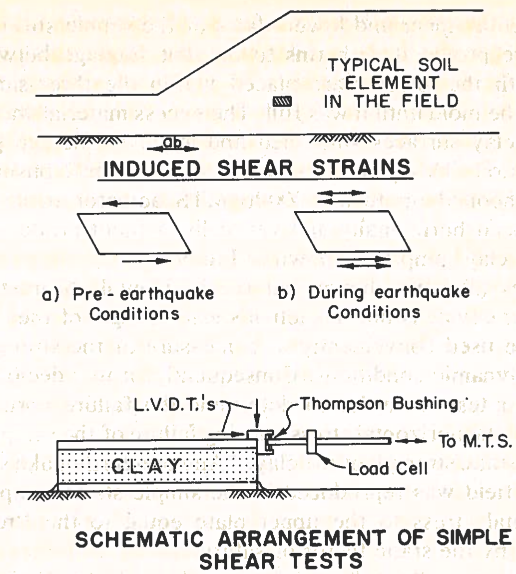

FIG. 9.—Field and Laboratory Cyclic Loading of Soil Element

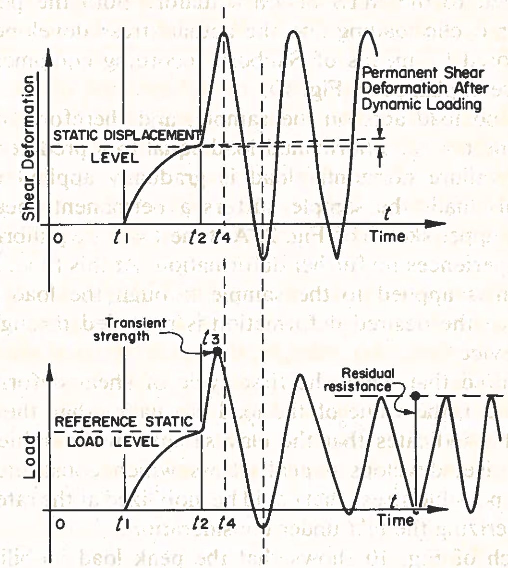

FIG. 10.—Dynamic Clay Strength; Strain Controlled Tests

was a mixture of 75% Kaolinite to 25% Bentonite at a water content of about 100%. Typical data on this clay mixture have been presented elsewhere (1,5). Samples were 1 ft square in plan dimensions and 2.4 in. high. It was decided to use a three-sided mold provided with removable bottom and top plates which constituted the actual upper and lower plates of the simple shear sample setup. These plates were provided with fins to prevent slippage between the plates and the soil. With the lower plate placed within the three-sided mold, clay was deposited in the mold until it was full. The excess material was then screeded off and the two clay surfaces smoothed and leveled. The top plate was then pressed into position. This plate was provided with a ball bushing attachment which could be hooked up to a 2,000-lb MTS actuator in such a way that it could be displaced horizontally and vertically without producing any torsion or bending of the clay sample as shown in Fig. 9.

Previous studies (3) have shown that the horizontal shear stress measured in the simple shear device is not the actual shear strength of the clay. However, this stress may be used conveniently as a measure of the strength of the clay under static and dynamic conditions. Consequently, it was decided to run some static simple shear tests in order to determine the failure horizontal load for the material used. The horizontal stress causing failure of the sample was adopted as the "reference static strength of the clay." Any pre-earthquake stress condition of a slope in the field was reproduced in the simple shear sample by applying an initial horizontal stress to the upper plate equal to the "reference static strength" divided by the static factor of safety.

The test sample was allowed to deform and reach equilibrium (no further deformation) under the applied static load. When equilibrium had been reached, a constant amplitude cyclic shear strain was applied to the upper plate through a load cell attached to the MTS piston actuator. Both the permanent shear deformation under cyclic loading and the actual stress developed during each cycle were monitored by means of Sanborn recording equipment. The results of a typical test are explained in Fig. 10.

At time $t = 0$, no load acts on the sample, and therefore, no deformation takes place. At time $t = t_1$, a horizontal load equal to a predetermined fraction of the calculated failure horizontal load is gradually applied to the sample. As a result of this load, the sample suffers a permanent shear deformation as indicated in the upper sketch of Fig. 2. At time $t = t_1$, equilibrium is reached; i.e., the sample experiences no further deformation. At this time, a precalculated cyclic shear strain is applied to the sample through the load cell. The load required to produce the desired deformation is recorded through the load cell on the Sanborn device.

It may be observed that when the first cycle of shear deformation is at its highest point ($t = t_4$), the value of the load is smaller than the value attained at time $t = t_3$. This indicates that the clay sample, when subjected to a high rise time strain pulse, develops a peak stress which constitutes its transient strength or highest possible stress that could be mobilized at the rate and amplitude of loading characterizing the test under consideration.

The lower sketch of Fig. 10 shows that the peak load mobilized during the second strain cycle has lower amplitude than that during the first cycle. The same holds true for all subsequent cycles. The load decrease is especially noticeable during the first three cycles, after which a fairly constant "residual load" is mobilized. The stresses measured between the fourth and 12th cycles were taken as representative values of the residual clay resistance under simple shear test conditions and have been used in analyzing the clay sample deformations.

Simple Shear Test Results.—The maximum shearing resistances obtained by the shaking table tests are compared in Fig. 11 with the values of the transient strengths obtained by the simple shear tests. For reference purposes, the static strength of the clay is also shown. Stresses are shown for different values of the factor, γf. The results presented in this figure point out that the shearing resistances developed by a clay during the cyclic loading imposed by an earthquake-like motion are always lower than the transient strength of the clay under similar conditions of shear amplitude and rate of loading. The maximum

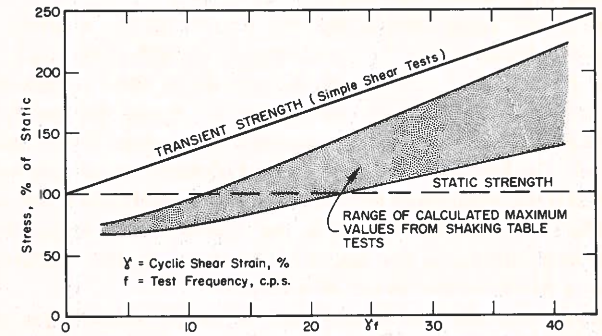

FIG. 11.—Clay Strength

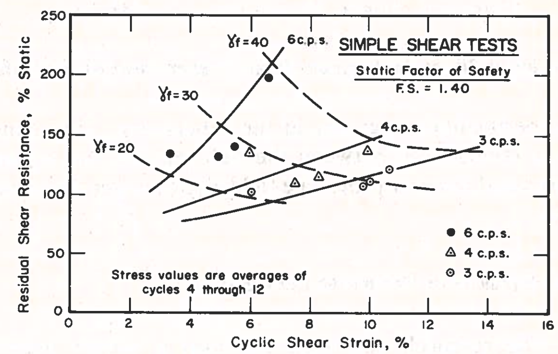

FIG. 12.—Cyclic Shear Strain Versus Residual Shear Resistance

shearing resistance, S, developed by the clay during the cyclic loading may be either lower or higher than the undrained strength of the clay depending on the value of the velocity parameter, γf.

The relationship between the cyclic shear strain and the residual shearing resistance as developed in the simple shear tests is shown in Fig. 12 where lines of equal frequency and equal γf have been superimposed. The data from the same tests were also used in preparing Fig. 13 which shows, after six and 12 cycles of loading, typical variation trends of the permanent shear deformations and the residual shearing resistance for two values of γf.

The similarities between the test result trends obtained by the shake table tests, Fig. 8, and the simple shear tests, Fig. 13, are readily apparent. In both of these tests, the clay behavior is essentially the same in nature, and the factors influencing the deformation and stress changes are the same. This conclusion is of practical importance since it indicates that the behavior of a bank of saturated clay exposed to seismic shaking can be predicted by results

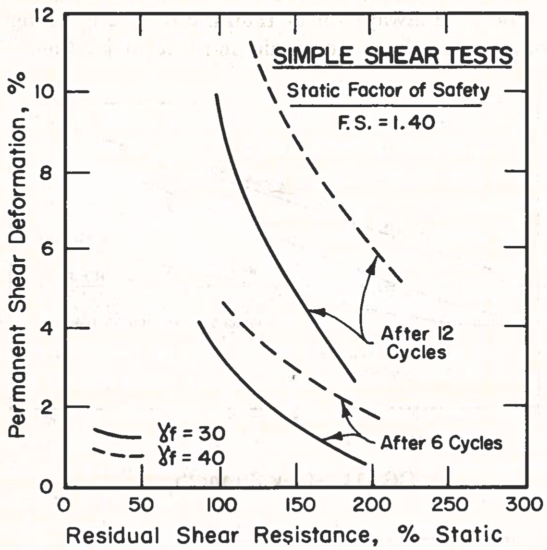

FIG. 13.—Residual Shear Resistance Versus Permanent Shear Deformation

obtained from experimental programs in the laboratory. This concept implies that there exists a relationship between the deformation and stresses obtained in simple shear tests and those of prototype field slopes under the same earthquake loading conditions.

Analysis of Seismic Stability of Saturated Clay Slopes

Introduction.—The research described provides a deeper understanding of the stability of slopes during seismic loading and of the factors influencing their deformation. The data obtained enables the formulation of a method for the evaluation of the behavior of saturated clay slopes under seismic loading. The method is essentially a modified form of that proposed by Seed (8) in that it utilizes response analysis to determine the stresses and strains induced in the earth bank and cyclic loading tests to evaluate the deformation resulting from these stresses. However, it utilizes strain-controlled rather than stress-controlled, cyclic loading tests, and it also incorporates the concepts of a rigid sliding mass and a yield acceleration proposed by Newmark (7). The method embraces the following steps: (1) Definition of the problem—geometry and soil properties; (2) selection of the appropriate design earthquake; (3) dynamic response analysis of the earth bank to the given earthquake—determination of accelerations and shear strains developed within the bank; (4) laboratory

soil testing to determine dynamic cyclic soil properties; and (5) maximum acceleration and crest deformation assessment. These steps are analyzed in the following.

Definition of Problem.—The geometric characteristics of a saturated clay slope of height H, foundation thickness D and angle β are shown in Fig. 14. The clay has an undrained strength, S_u, corresponding to a defined static factor

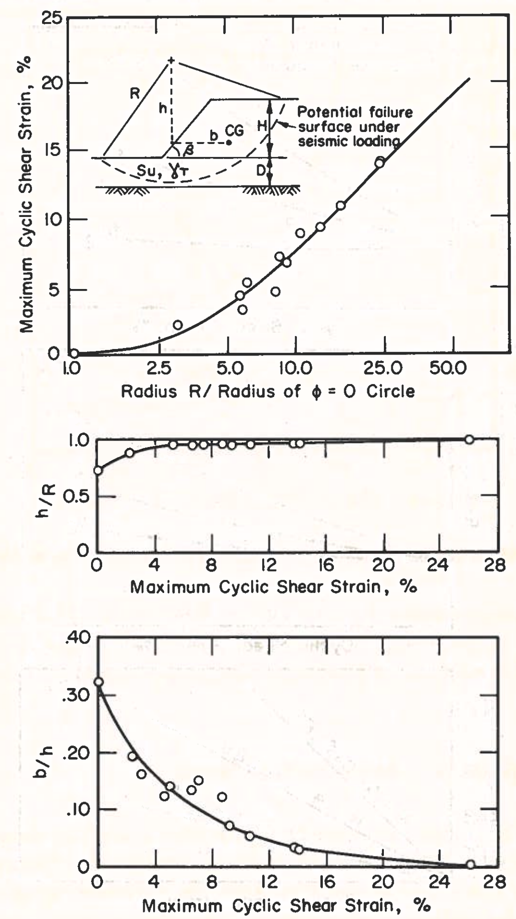

FIG. 14.—Relationships Between Maximum Shear Strain and Geometrical Parameters of Slip Surfaces

of safety. The base of the slope is subjected to a design earthquake. It is now desired to determine whether the slope will withstand the seismic loading without appreciable permanent deformation or whether it is necessary to find the extent of any deformation that might take place.

Design Earthquake; Dynamic Response Analysis.—For analysis purposes, it is necessary to determine the time history of motion in the rock-like base material. Although earthquake motions are random in nature, they do possess certain basic characteristics including: maximum amplitude of accelerations, predominant frequency of motions, and duration. Methods to develop reasonable estimates of the time history of the rock motion at a given site have been examined elsewhere (11).

The successful application of analytical finite element methods to the evaluation of the response of earthbanks under seismic loading has also been considered by several authors within the past recent years (4).

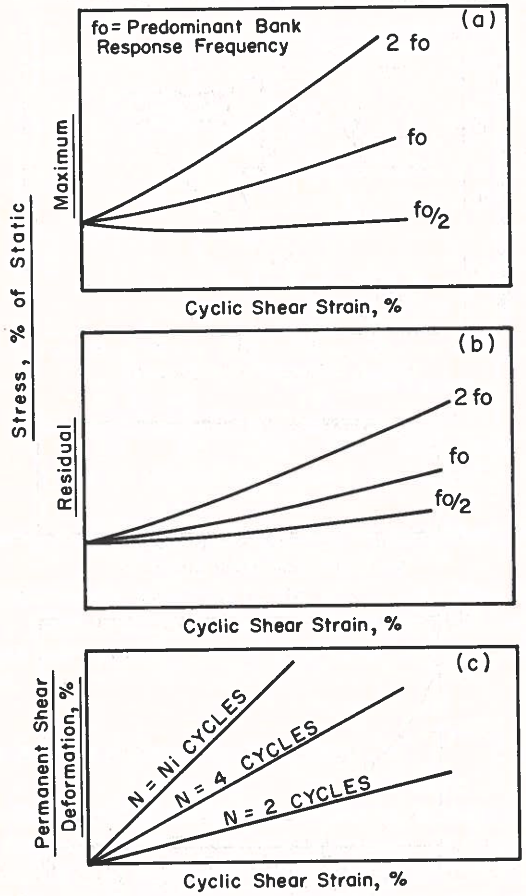

FIG. 15.—Summary of Laboratory Test Results (Simple Shear or Triaxial)

It was shown in previous paragraphs that the actual extent of the yielding zone is a function of the maximum cyclic shear strain developed within the bank. Only an estimate of the maximum cyclic shear strains that the slope will develop during the earthquake is at first needed. This estimate may be obtained by plotting the strains at various heights and distances from the face of the slope of the bank as a function of time and observing their relative magnitude. The maximum average shear strain amplitude may then be chosen by inspection.

With this estimated value of the shear strain amplitude, Fig. 14 can be used to define the extent of the mass of soil which would be involved in any potential slope movement. The various geometrical relationships presented in Fig. 14 were obtained by analyzing the slip circles of the deformed slopes as those shown in Fig. 4. The centers of gravity of the sliding masses were obtained by graphical means. The parameters *h*, *b*, and *R*, defined in the upper portions of Fig. 14 were scaled off and plotted versus the maximum cyclic shear strain corresponding to each individual test.

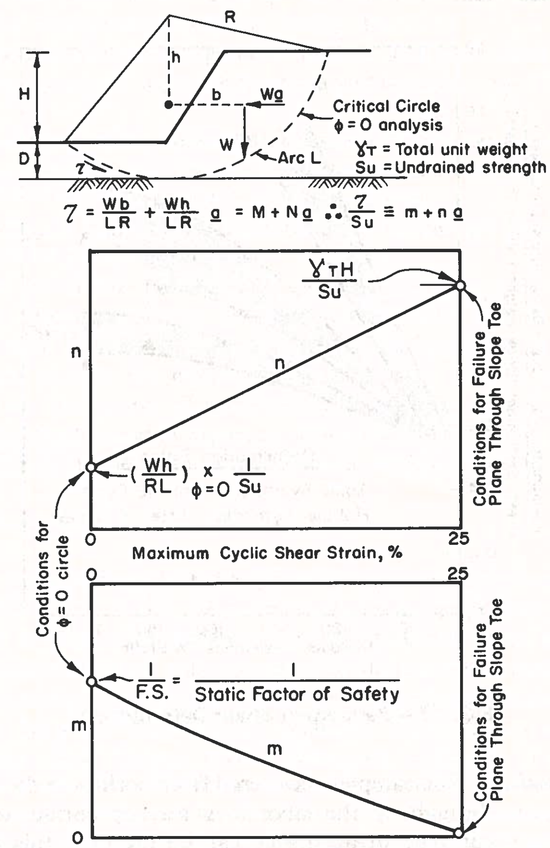

FIG. 16.—Parameters *m* and *n*

The information contained in this figure provides the basic data to define the slip circle to be analyzed in subsequent stages. A better estimate of the maximum accelerations and shear strain within the yeilding mass of soil can now be undertaken. Owing to the irregular nature of the acceleration distribution, it is not accurate to average the accelerations themselves. A method similar to that used in Ref. 12 in computing the average seismic coefficient can be utilized in this case. The maximum acceleration and cyclic shear strain can be taken as the average of the values during the first few seconds of shaking.

An assessment of the significant number of cycles and their corresponding amplitudes and predominant frequencies can also be made. Methods and several examples of these assessments are given in Ref. 8.

The analysis so far has provided the following basic information: (1) Maximum accelerations and cyclic strains within the potential yielding zone; and (2) number of significant stress cycles of equal amplitude to which the slopes will be subjected.

With these basic data, the slope behavior evaluation can proceed to the last two steps of the analysis; i.e. the laboratory testing and finally, the permanent deformation assessment.

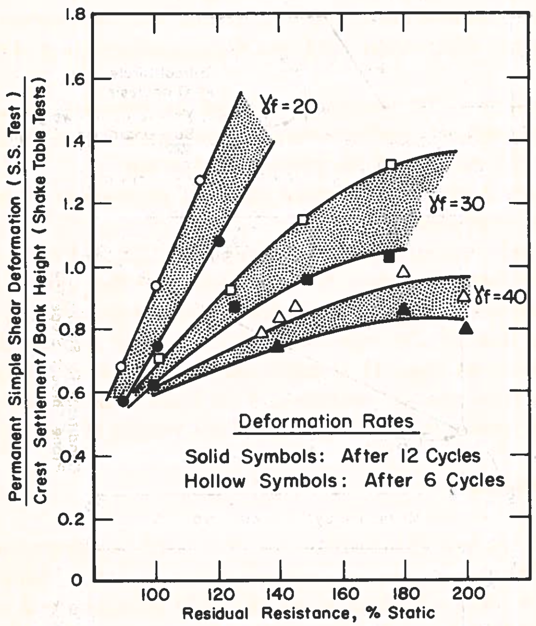

FIG. 17.—Permanent Shear Deformations

Laboratory Testing.—Soil samples recovered from borings in the field adjacent to the slope can be brought to the laboratory for appropriate testing. Either triaxial or simple shear type of apparatus can be used for this purpose. The two methods of testing and the procedures to interpret the test results are considered elsewhere (13,15).

The results of the laboratory tests, whether simple shear or triaxial, may be presented as shown in Fig. 15. The upper sketch of this figure shows the maximum cyclic shearing resistance of the clay under cyclic loading at a given shear strain amplitude and frequency. The middle sketch shows the residual resistance of the clay and the lower one depicts the permanent shear deformation of the test specimen after the application of a number of strain cycles.

Maximum Acceleration.—It has been shown that the maximum accelerations actually observed in the shaking table tests were a function of the geometry of the yielding slope and of the dynamic resistance of the clay. This information

is presented in Fig. 16, where the dimensionless parameters, *m* and *n*, are plotted versus the maximum cyclic shearing strains. The lower and upper boundaries for the two parameters are defined in the sketches. The almost linear variation between the limiting conditions has been supported by the test data obtained during this investigation (1).

Since the value of the maximum average cyclic shear strain has been calculated through the finite element analysis, it is now possible to extract the corresponding values of the parameters *m* and *n* (from Fig. 15) and of the maximum dynamic stress, *S*, from the laboratory data, Fig. 16. The maximum acceleration $\underline{a}$ can be computed by the equation presented in Fig. 16.

The maximum acceleration computed in this way represents the threshold acceleration beyond which appreciable permanent deformation of the slope would take place.

Slope Deformation.—The maximum average acceleration, *a*, as obtained by the finite element analysis must now be compared with the maximum acceleration as computed by the method of the preceeding section.

If $a < \underline{a}$, then it must be inferred that the permanent slope deformation will be small for all practical purposes.

If $a \geq \underline{a}$, the deformation of the slope must be expected to be of engineering concern. Such deformation may be estimated with the aid of Fig. 17 which represents the ratio between the permanent shear deformation of the laboratory test sample and that of the equivalent shaking table test as defined by the settlement of the crest. Fig. 17 requires the knowledge of cyclic shear strain amplitude, of the predominant frequency of the bank response and of the residual stress of the clay available from Fig. 15 and the results of the FE Analyses.

Summary and Conclusions

Studies have been made of the behavior of saturated clay slopes under simulated earthquake conditions. It may be concluded that in large shaking table tests, there is a peak or yield acceleration which might be developed within the clay banks. The development of this acceleration determines whether or not the slope will be permanently deformed under cyclic loading conditions.

The value of the yield acceleration is related to the geometry of the potential yielding soil mass and to the available maximum resistance of the clay when subjected to a given seismic loading. The maximum resistance shown by clay banks under seismic loading was lower than the transient strength of the material. The actual value of the resistance may be either higher or lower than the static undrained strength of the clay depending on factors such as the amplitude of ground motion and the rate of strain.

The permanent deformation of the slope is a function of the residual stress of the clay and the duration of the seismic motion. The velocity of ground motion was found to be as important in determining the extent of the permanent slope deformation as the number of significant stress cycles and their amplitudes.

The research indicates a close interrelationship between the results obtained in a shaking table and in a simple shear device.

Finally, based on the findings of this investigation, a method for predicting field slope behavior under seismic conditions is advanced. Although the data presented were obtained for slopes of a relatively insensitive saturated clay with only two different static factors of safety, it is believed that the method is general and may be extended to slopes of different materials and geometries.

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Performance Instrumentation Installed in Oroville Dam

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Introduction

The California Department of Water Resources' Oroville Dam with a 770-ft (235-m)-high embankment included a comprehensive performance-instrumentation program. (Construction began in 1963 and was concluded in 1967.) Many of the standard earth-dam instrument systems were employed, as well as several new systems designed especially for Oroville Dam. The unprecedented height of this embankment dam posed a unique challenge to the effectiveness and durability of the various performance-instrumentation systems. This paper describes the instrumentation equipment, installation methods, and operational response throughout the construction period, as well as the current operational status (1973).

Oroville Dam is a zoned, earth-fill dam of approx 80,000,000 cu yd (61,000,000 m^3) of material. The zones included a clayey, sandy gravel, impervious core, and shell zones of mostly rounded gravel, dredger tailings resulting from early days hydraulic gold-mining operations along the Feather River. Fig. 1 shows a general cross section.

Coreblock Instrumentation—General

The initial instrumentation installed was in the 290,000-cu yd (222,000-m^3) concrete coreblock beneath the impervious core. The coreblock structure was primarily intended to fill the steep-walled river channel beneath the impervious

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