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EARTHQUAKE ENGINEERING RESEARCH CENTER

A SIMPLIFIED PROCEDURE FOR ESTIMATING EARTHQUAKE-INDUCED DEFORMATIONS IN DAMS AND EMBANKMENTS

by
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A report on research sponsored by
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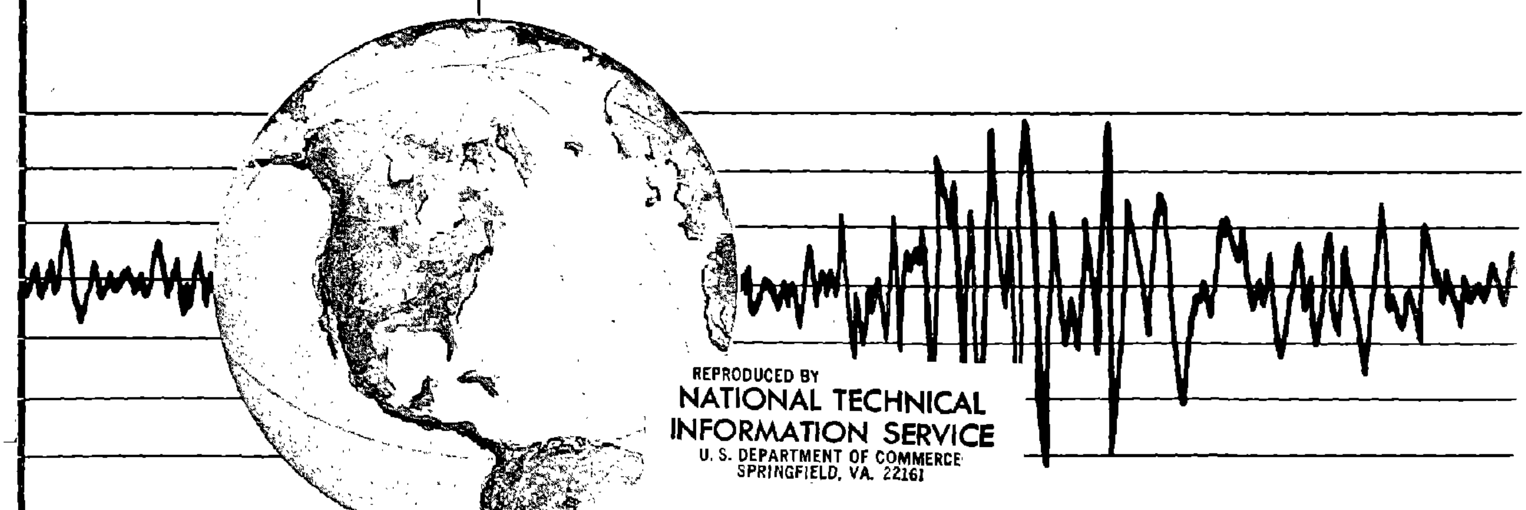

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The inadequacy of the pseudo-static approach recommended by the International Committee on Large Dams, to predict the behavior of low embankments during earthquakes has been clearly recognized and demonstrated. The committee pointed out the need for early revision of the method which "in several instances...predicts a safe condition for dams which are known to have had major slides."

This report presents a simplified procedure for the seismic design of small embankments. The method is an approximate one which uses the concept, originally proposed by Newmark, for calculating permanent deformations, but it is based on an evaluation of the dynamic response of the embankment rather than rigid body behavior.

The method is applied to dams with heights in the range of 100 to 200 ft and constructed of compacted cohesive soils or very dense cohesionless soils. Design curves on the basis of analyzed cases are presented with an example problem.

The method is an approximate one and involves simplifying assumptions. The design curves are averages based on a limited number of cases and should be updated as more data become available.

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F. I. Makdisi[1] and H. Bolton Seed[2]

INTRODUCTION

In the past decade major advances have been achieved in analyzing the stability of dams and embankments during earthquake loading. Newmark (1965) and Seed (1966) proposed methods of analysis for predicting the permanent displacements of dams subjected to earthquake shaking and suggested this as a criterion of performance as opposed to the concept of a factor of safety based on limit equilibrium principles. Seed and Martin (1966) used the shear beam analysis to study the dynamic response of embankments to seismic loads and presented a rational method for the calculation of dynamic seismic coefficients for earth dams. Ambraseys and Sarma (1967) adopted the same procedure to study the response of embankments to a variety of earthquake motions.

Later the finite element method was introduced to study the two-dimensional response of embankments (Clough and Chopra, 1966; Idriss and Seed, 1967) and the equivalent linear method (Seed and Idriss, 1969a) was used successfully to represent the strain dependent non-linear behavior of soils. In addition the nature of the behavior of soils during cyclic loading has been the subject of extensive research (Seed and Chan, 1966; Seed and Lee, 1966; Lee and Seed, 1967, Thiers and Seed, 1969, etc.). Both, the improvement in the analytical tools to study the response of embankments and the

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knowledge of material behavior during cyclic loading, led to the development of a more rational approach to the study of stability of embankments during seismic loading. Such an approach was used successfully to analyze the Sheffield Dam failure during the 1925 Santa Barbara earthquake (Seed, Lee and Idriss, 1969) and the behavior of the San Fernando Dams during the 1971 earthquake (Seed et al., 1973). This method has since been used extensively in the design and analysis of many large dams in the State of California and elsewhere.

From the study of the performance of embankments during strong earthquakes, two distinct types of behavior may be discerned:

(1) That associated with loose to medium dense sandy embankments; these are susceptible to rapid increases in pore pressure due to cyclic loading resulting in the development of pore pressure ratios of 100% in large portions of the embankment, associated reductions in shear strength, and potentially large movements leading to almost complete failure.

(2) The behavior associated with compacted cohesive clays, dry sands and some dense sands; here the potential for buildup of pore pressures is much less than that associated with loose to medium dense sands, the resulting cyclic strains are usually quite small and the material retains most of its static undrained shearing resistance so that the resulting post-earthquake behavior is a limited permanent deformation of the embankment.

The dynamic analysis procedure proposed by Seed and his co-workers has been used to predict adequately both types of embankment behavior using the "Strain Potential" concept (Seed et al., 1973). Procedures for integrating strain potentials to obtain the overall deformation of an embankment have been proposed by Seed et al. (1973), Lee (1975), and Serff et al. (1976).

The dynamic analysis approach has been recommended by the Committee on Earthquakes of the International Commission on Large Dams (ICOLD, 1975): "high embankment dams whose failure may cause loss-of-life or major damage should be designed by the conventional method at first, followed by a dynamic analysis in order to investigate any deficiencies which may exist in the pseudo-statical design of the dam". For low dams in remote areas the Committee recommended the use of conventional pseudo-static methods using a constant horizontal seismic coefficient selected on the basis of the seismicity of the area. However, the inadequacy of the pseudo-static approach to predict the behavior of embankments during earthquakes has been clearly recognized and demonstrated (Terzaghi, 1950; Seed and Martin, 1966; Seed, Lee and Idriss, 1969; Seed et al., 1973 and Seed, 1973). Furthermore in the same report mentioned above (ICOLD, 1975) the Commission refers to the conventional method as follows: "There is a need for early revision of the conventional method since the results of dynamic analyses, model tests and observations of existing dams show that the horizontal acceleration due to earthquake forces varies throughout the height of the dam....in several instances, this method predicts a safe condition for dams which are known to have had major slides."

It is this need for a simple yet rational approach to the seismic design of small embankments that prompted the development of the simplified procedure described in the following pages.

This approximate method uses the concept originally proposed by Newmark (1965) for calculating permanent deformations but it is based on an evaluation of the dynamic response of the embankment as proposed by Seed and Martin (1966) rather than rigid body behavior. It assumes that failure occurs on a well defined slip surface and that the material behaves elastically at stress levels below failure but develops a perfectly plastic behavior above yield. The method involves the following steps:

(1) A yield acceleration i.e. an acceleration at which a potential sliding surface would develop a factor of safety of unity is determined. Values of yield acceleration are a function of the embankment geometry, the undrained strength of the material (or the reduced strength due to shaking), and the location of the potential sliding mass.

(2) Earthquake induced accelerations in the embankment are determined using dynamic response analyses. Finite element procedures using strain-dependent soil properties can be used for calculating time-histories of acceleration, or simpler one-dimensional techniques might be used for the same purpose. From these analyses time-histories of average accelerations for various potential sliding masses can be determined.

(3) For a given potential sliding mass, when the induced acceleration exceeds the calculated yield acceleration, movements are assumed to occur along the direction of the failure plane and the magnitude of the displacement is evaluated by a simple double integration procedure.

The method has been applied to dams with heights in the range of 100 to 200 ft, and constructed of compacted cohesive soils or very dense cohesionless soils, but may be applicable to higher embankments. A similar approach has been proposed by Sarma (1975) using the assumption of a rigid block on an inclined plane rather than a deformable earth structure which responds with differential motions to the imposed base excitation.

In the following paragraphs the steps involved in the analyses will be described in detail and design curves prepared on the basis of analyzed cases will be presented, together with an example problem to illustrate the use of the method. It should be noted, however, that the method is an approximate one and involves simplifying assumptions. The design curves are averages based on a limited number of cases analyzed and should be updated as more data become available and more cases are studied.

DETERMINATION OF THE YIELD ACCELERATION

The yield acceleration, k_y, is defined as that average acceleration producing a horizontal inertia force on a potential sliding mass so as to produce a factor of safety of unity and thus to cause it to experience permanent displacements.

For soils which do not develop large cyclic strains or pore pressurs and maintain most of their original strength after earthquake shaking, the value of k_y can be calculated by stability analyses using limiting equilibrium methods. In conventional slope stability analyses the strength of the material is defined as either the maximum deviator stress in an undrained test, or that stress level which would cause a certain allowable axial strain, say 10%, in a test specimen. However, the behavior of the material under cyclic loading conditions is different from that under static conditions. Due to the transient nature of the earthquake loading an embankment may be subjected to a number of stress pulses at levels equal to or higher than its static failure stress which simply produce some permanent deformation rather than complete failure. Thus the yield strength is defined, for the purpose of this analysis, as that maximum stress level below which the material exhibits a near elastic behavior (when subjected to cyclic stresses of numbers and frequencies similar to those induced by earthquake shaking) and above which the material exhibits permanent plastic deformation of magnitudes dependent on the number and frequency of the pulses applied. Fig. 1 illustrates the concept of cyclic yield strength. The material in this case has a cyclic yield strength equal to 90% of its static undrained strength and as shown in Fig. 1(a) the application of 100 cycles of stress amounting to 80% of the undrained strength resulted in essentially an elastic behavior with very little permanent deformation. On the other hand the application of 10 cycles of stress level equal to 95% of the static undrained strength led to substantial permanent strain as shown in Fig. 1(b). On loading the material monotonically to failure after the series of cyclic stress applications, the material was found to retain the original undrained strength. This type of behavior is associated with various types of soils that exhibit small increases in pore pressure during cyclic loading. This would include clayey materials, dry or partially saturated cohesionless soils or very dense saturated cohesionless materials which will not undergo significant deformations, even under cyclic loading conditions, unless the undrained static strength of the soil is exceeded.

Seed and Chan (1966) conducted cyclic tests on samples of undisturbed and compacted silty clays and found that for conditions of no stress reversal and for different values of initial and cyclic stresses, the total stress required to produce large deformations in 10 and 100 cycles ranged between 90 and 110% of the undrained static strength.

Sangrey et al. (1969) investigated the effective stress response of clay under repeated loading. They tested undisturbed samples of highly plastic clay (LL = 28, PI = 10) and found that the cyclic yield strength of this material was of the order of 60% of its static undrained strength.

Rahman (1972) performed similar tests on remoulded samples of a brittle silty clay (LL = 91, PI = 49) and found that the cyclic yield strength was a

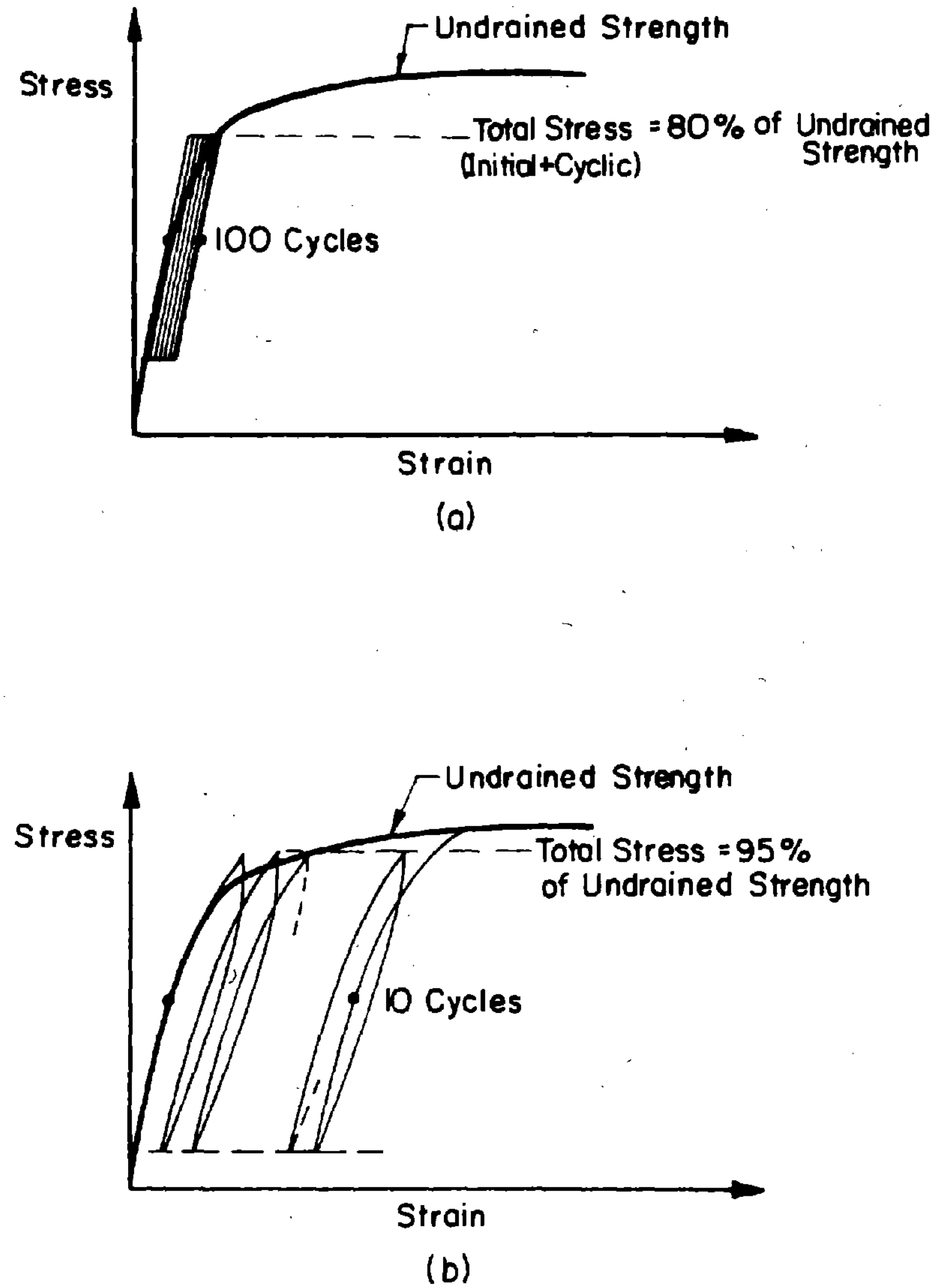


FIG. 1 DEFINITION OF DYNAMIC YIELD STRENGTH

function of the initial effective confining pressure. For practical ranges of effective confining pressures the cyclic yield strength for this material ranged between 80 and 95 percent of its static undrained strength. At cyclic stress levels below the yield strength, in all cases, the material reached equilibrium and assumed an elastic behavior at strain levels less than 2 percent irrespective of the number of stress cycles applied.

Thiers and Seed (1969) performed tests on undisturbed and remoulded samples of different clayey materials to determine the reduction in static undrained strength due to cyclic loading. Their results are summarized in Fig. 2 which shows that the reduction in undrained strength after cyclic loading as a function of the ratio of the "maximum cyclic strain" to the "static failure strain". These results were obtained from strain controlled cyclic tests; after the application of 200 cycles of a certain strain amplitude, the sample was loaded to failure monotonically at a strain rate of 3 percent per minute. Thus from Fig. 2 it could be argued that if a clay is subjected to 200 cycles of strain with an amplitude less than half its static failure strain, the material may be expected to retain at least 90 percent of its original static undrained strength.

Andersen (1976), on the basis of cyclic simple shear tests on samples of Drammen clay, determined that the reduction in undrained shear strength was found to be less than 25% as long as the cyclic shear strain was less than ±3% even after 1000 cycles.

On the basis of the experimental data reported above and for values of cyclic shear strains calculated from earthquake response analyses, the value of cyclic yield strength for a clayey material can be estimated. In most cases this value would appear to be 80% or more of the static undrained strength. This value in turn may be used in an appropriate method of stability analysis to calculate the corresponding yield acceleration.

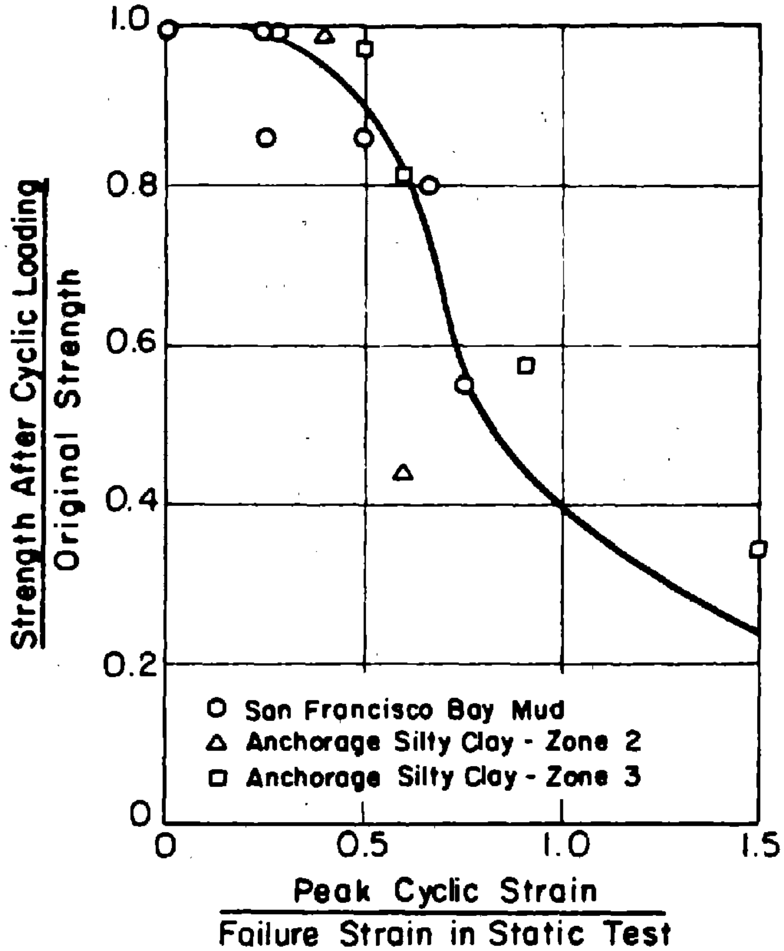
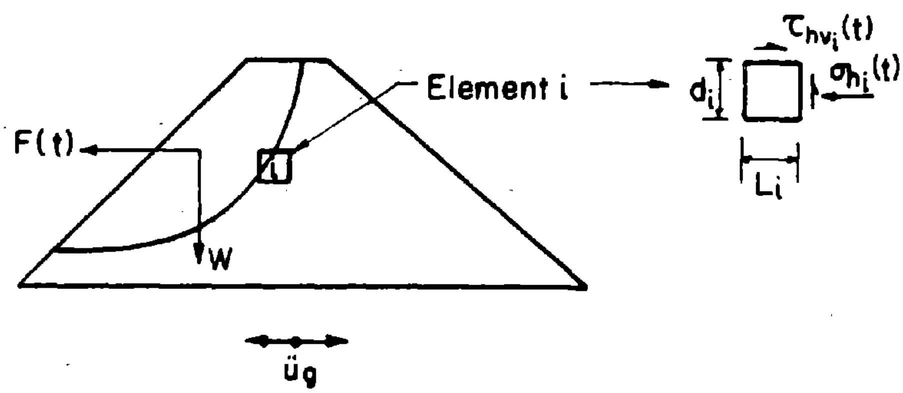


FIG. 2 REDUCTION IN STATIC UNDRAINED STRENGTH DUE TO CYCLIC LOADING (From Thiers & Seed, 1969)



$$F(t) = \sum_{i=1}^{n} \tau_{hv_i}(t)\, L_i + \sigma_{h_i}(t)\, d_i$$

n = number of elements along the sliding surface

$$k_{av}(t) = F(t)/W$$

FIG. 3 CALCULATION OF AVERAGE ACCELERATION FROM FINITE ELEMENT RESPONSE ANALYSIS

Finite element response analyses (as will be described later) have been carried out to calculate time histories of crest acceleration and average acceleration for various potential sliding masses. The method of analysis employs the equivalent linear technique with strain dependent modulus and damping. The ranges of calculated maximum shear strains, for different magnitude earthquakes and different embankment characteristics, are presented in Table 1. It can be seen from Table 1 that the maximum cyclic shear strain induced during the earthquakes ranged between 0.1% for a magnitude 6-1/2 earthquake with a base acceleration of 0.2g and 1% for a magnitude 8-1/4 earthquake with a base acceleration of 0.75g. For the compacted clayey material encountered in dam embankments "static failure strain" values usually range between 3% and 10%, depending on whether the material was compacted on the dry or wet side of the optimum moisture content. Thus in both instances the ratio of the "cyclic strain" to "static failure strain" is less than 0.5.

It seems reasonable therefore to assume that for these compacted cohesive soils,very little reduction in strength may be expected as a result of strong earthquake loading of the magnitude described above.

Once the cyclic yield strength is defined, the calculation of the yield acceleration can be achieved by using one of the available methods of stability analysis. In the present study the ordinary method of slices has been used to calculate the yield acceleration for circular slip surfaces. As an alternative Seed (1966) has suggested a method of combining both effective and total stress approaches, where the shear strength on the failure plane during the earthquake is considered to be a function of the initial effective normal stress on that same plane before the earthquake. This method is applicable to non-circular slip surfaces and the horizontal inertia force resulting in a factor of safety of unity can readily be calculated.

Table 1

Maximum Cyclic Shear Strains Calculated from Dynamic Finite Element Response Analyses

Magnitude	Embankment Height (ft.)	Slope	Max. Base Acceleration	Max. Shear Strain (%)
6-1/2 (Caltech record)	75	2:1	0.5	0.2 - 0.4
6-1/2 (Caltech record)	150	2:1	0.2	0.1 - 0.15
6-1/2 (Caltech record)	150	2:1	0.5	0.2 - 0.3
6-1/2 (Lake Hughes record)	150	2:1	0.2	0.1 - 0.15
6-1/2 (Caltech record)	150	2-1/2:1	0.5	0.2 - 0.3
7-1/2 (Taft record)	150	2:1	0.5	0.2 - 0.5
7-1/2 (Taft record)	150	2:1	0.2	0.1 - 0.2
8-1/4 (S-I record)	150	2:1	0.75	0.4 - 1.0
8-1/4 (S-I record)	135	-	0.4	0.2 - 0.5

Table 2

Embankment Characteristics for Magnitude 6-1/2 Earthquake

Case #	Embankment Description	Height (ft.)	Base Acceleration (g)	T_o (1)	k_{max} (2) (g)	Symbol
1	Example Case - slope = 2:1 - k_{2max} = 60	150	0.2 (Caltech record)	0.8	(a) 0.31 (b) 0.12	● ■
2	Example Case - slope = 2:1 - k_{2max} = 60	150	0.5 (Caltech record)	1.08	(a) 0.4 (b) 0.18	○ □
3	Example Case - slope = 2:1 - k_{2max} = 80	150	0.5 (Lake Hughes record)	0.84	(a) 0.33 (b) 0.16	⊙ △
4	Example Case - slope = 2-1/2:1 - k_{2max} = 80	150	0.5 (Caltech record)	0.95	(a) 0.49 (b) 0.22	◇ ▽
5	Example Case - slope = 2:1 - k_{2max} = 60	75	0.5 (Caltech record)	0.6	(a) 0.86 (b) 0.26	◐ ◩

(1) T_o = Calculated first natural period of the embankment.

(2) k_{max} = Maximum value of time history of:
(a) crest acceleration
(b) average acceleration for sliding mass extending through full height of embankment.

Having determined the yield acceleration for a certain location of the slip surface, the next step in the analysis is to determine the time history of earthquake-induced average accelerations for that particular sliding mass. This will be treated in the following section.

DETERMINATION OF EARTHQUAKE INDUCED ACCELERATION

In order for the permanent deformations to be calculated for a particular slip surface, the time-history of earthquake-induced average accelerations must first be determined.

Two-dimensional finite element procedures using equivalent linear strain-dependent properties are available (Idriss et al., 1973) and have been shown to provide response values in good agreement with measured values (Kovacs et al., 1971) and with closed form one-dimensional wave propagation solutions (Schnabel et al., 1972).

For most of the case studies of embankments used in the present analysis, the response calculation was performed using the finite element computer program QUAD-4 (Idriss et al., 1973) with strain dependent modulus and damping. The program uses the Rayleigh damping approach and allows for variable damping to be used in different elements.

To calculate the time-history of average acceleration for a specified sliding mass, the method described by Chopra (1966) was adopted in the present study. The finite element calculation provides time-histories of stresses for every element in the embankment. As illustrated in Fig. 3, at each time step the forces acting along the boundary of the sliding mass are calculated from the corresponding normal and shear stresses of the finite elements along that boundary. The resultant of these forces divided by the weight of the sliding mass would give the average acceleration, $k_{av}(t)$, acting on the sliding mass at that instant in time. The process is repeated for every time step to calculate the entire time-history of average acceleration.

For a 150-ft-high dam subjected to 30 seconds of the Taft earthquake record scaled to produce a maximum base acceleration of 0.2g, the variation of the time-history of k_{av} with the depth of the sliding mass within the embankment, together with the time-history of crest accelerations, is shown in Fig. 4.

Comparing the time-history of crest acceleration with that of the average acceleration for different depths of the potential sliding mass, the similarity in the frequency content is readily apparent (it generally reflects the first natural period of the embankment), while the amplitudes are shown to decrease as the depth of the sliding mass increases towards the base of the embankment. The maximum crest acceleration is designated by $\ddot{u}_{max}$, and k_{max} is the maximum average acceleration for a potential sliding mass extending to a specified depth, y.

It would be desirable to establish a relationship showing the variation of the maximum acceleration ratio ($k_{max}/\ddot{u}_{max}$) with depth for a range of embankments and earthquake loading conditions. It would then be sufficient, for design purposes, to estimate the maximum crest acceleration in a given embankment due to a specified earthquake and use the above relationship to determine the maximum average acceleration for any depth of the potential sliding mass. A simplified procedure to estimate the maximum crest acceleration and the natural period of an embankment subjected to a given base motion is described in Appendix A (Makdisi and Seed, 1977).

To determine the variation of maximum acceleration ratio with depth, use was made of published results of response computations using the one-dimensional shear slice method with visco-elastic material properties (Seed and Martin, 1966; Ambraseys and Sarma, 1967). Martin (1965) calculated the response of embankments ranging in height between 100 and 600 ft and with shear wave velocities

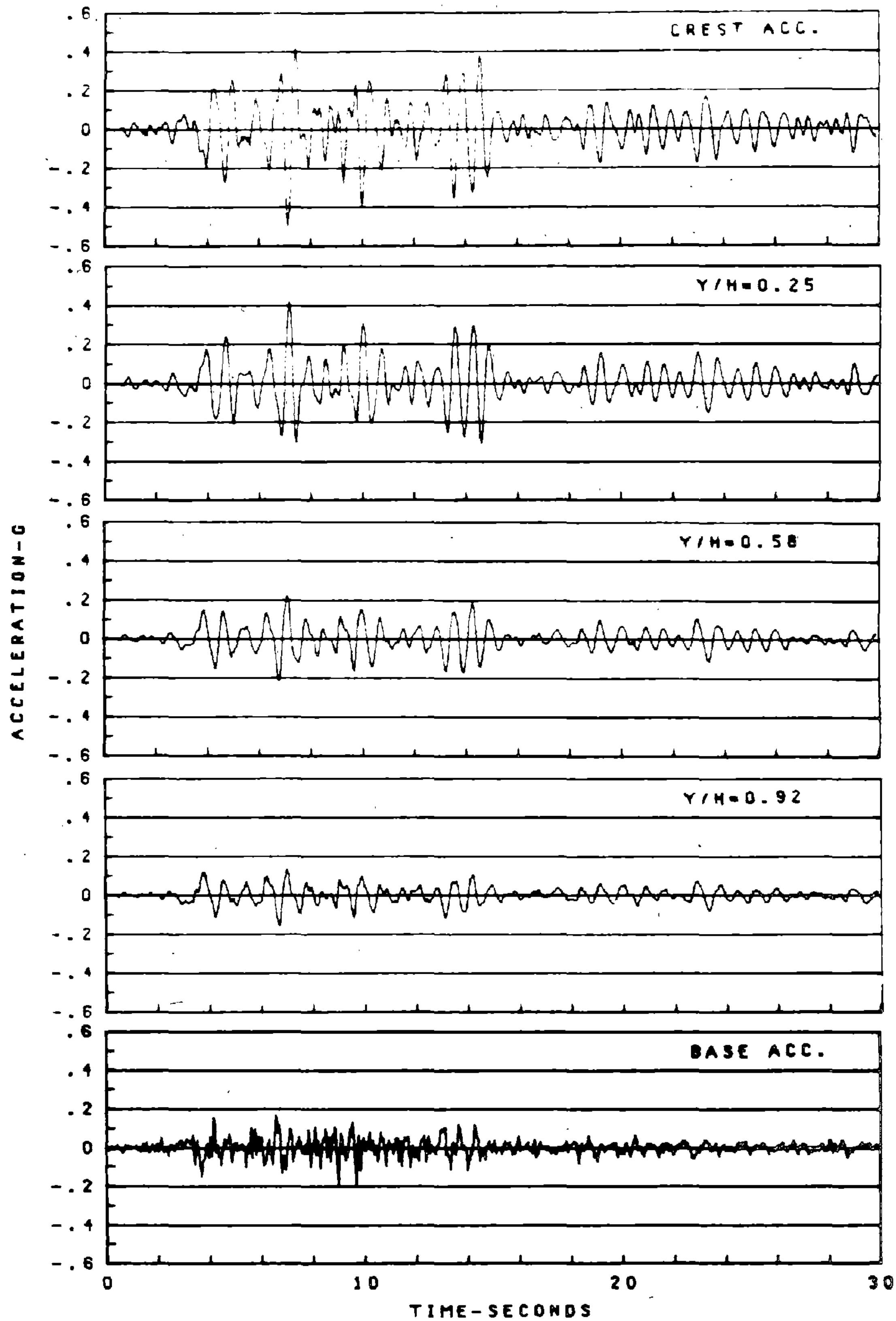


FIG. 4 TIME-HISTORIES OF AVERAGE ACCELERATION FOR VARIOUS DEPTHS OF POTENTIAL SLIDING MASS.

between 300 and 1000 fps. Using a constant shear modulus and a damping factor of 0.2, the average acceleration histories for various levels were computed for embankments subjected to ground accelerations recorded in the El Centro earthquake of 1940. The variation of the maximum average acceleration, k_{max}, with depth for these embankments with natural periods ranging between 0.26 and 5.22 sec. is presented in Fig. 5. The maximum average acceleration in Fig. 5 is normalized with respect to the maximum crest acceleration and the ratio $(k_{max}/\ddot{u}_{max})$ plotted as a function of the depth of the sliding mass is presented in Fig. 6.

Ambraseys and Sarma (1967) used essentially the same method reported by Seed and Martin (1966) and calculated the response of embankments with natural periods ranging between 0.25 and 3.0 seconds. They presented their results in terms of average response for 8 strong motion records. The variation of maximum average acceleration with depth based on the results reported by Ambraseys and Sarma (1967) is shown in Fig. 7 and that for the maximum acceleration ratio $(k_{max}/\ddot{u}_{max})$ is shown in Fig. 8. A summary of the results obtained from the different shear slice response calculations mentioned above is presented in Fig. 9 together with results obtained from finite element calculations made in the present study. As can be seen from Fig. 9 the shape of the curves obtained using the shear slice method and the finite element method are very similar. The dashed curve in Fig. 9 is an average relationship of all data considered. The maximum difference between the envelope of all data and the average relationship ranges from ±10 to ±20% for the upper portion of the embankment and from ±20 to ±30% for the lower portion of the embankment.

Considering the approximate nature of the proposed method of analysis, the use of the average relationship shown in Fig. 9 for determining the maximum average acceleration for a potential sliding mass based on the maximum

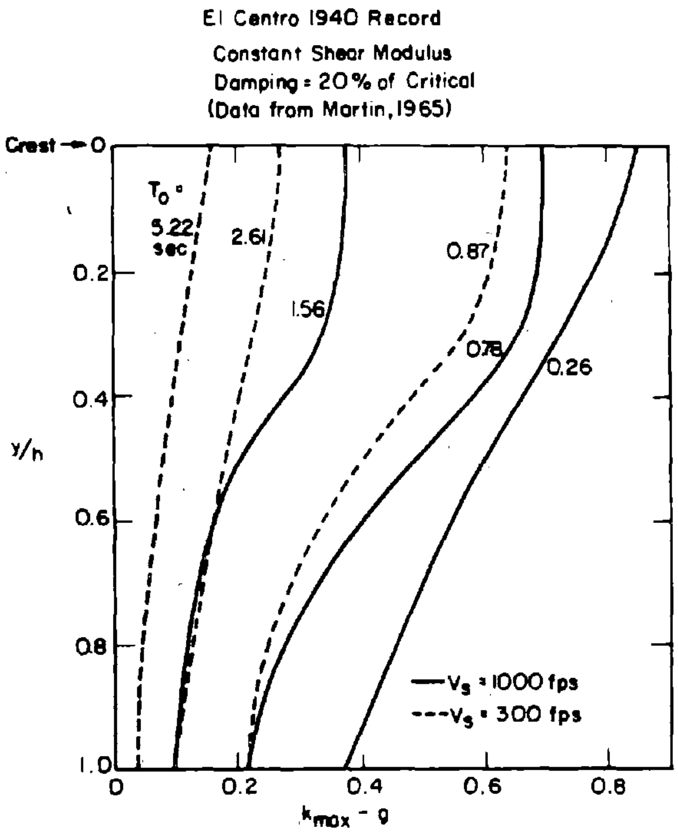


FIG. 5 VARIATION OF MAXIMUM AVERAGE ACCELERATION WITH DEPTH OF SLIDING MASS - EL CENTRO RECORD

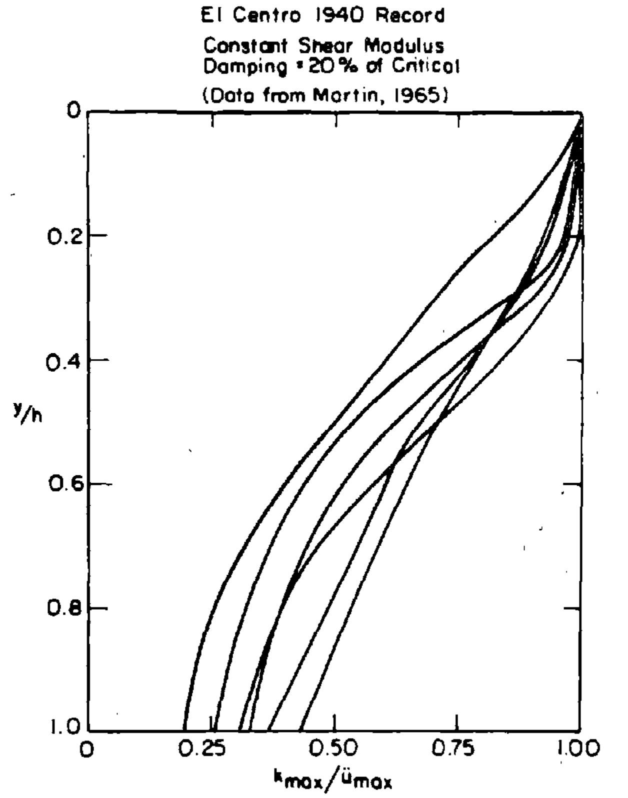


FIG. 6 VARIATION OF RATIO OF "MAX. AVERAGE ACCELERATION TO MAX. CREST ACCELERATION" WITH DEPTH OF SLIDING SURFACE - EL CENTRO EARTHQUAKE

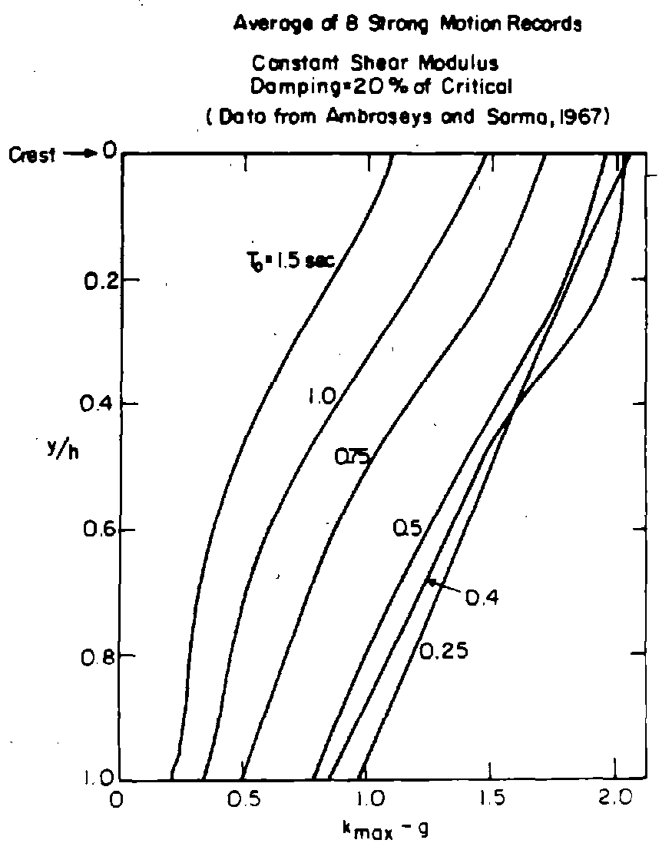


FIG. 7 VARIATION OF MAXIMUM AVERAGE ACCELERATION WITH DEPTH OF SLIDING MASS - AVERAGE OF 8 STRONG MOTION RECORDS

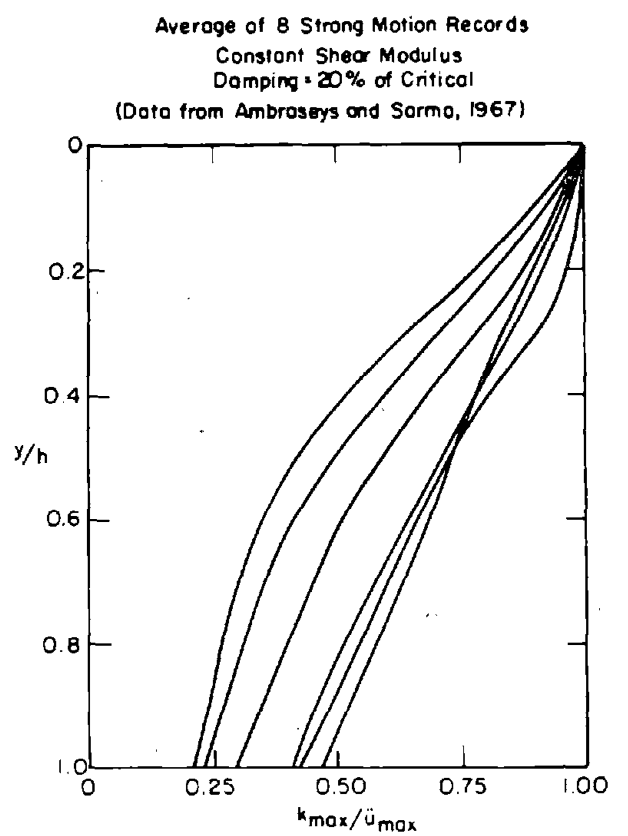


FIG. 8 VARIATION OF RATIO OF "MAX. AVERAGE ACCELERATION TO MAX. CREST ACCELERATION" WITH DEPTH OF SLIDING SURFACE - AVERAGE OF 8 STRONG MOTION RECORDS

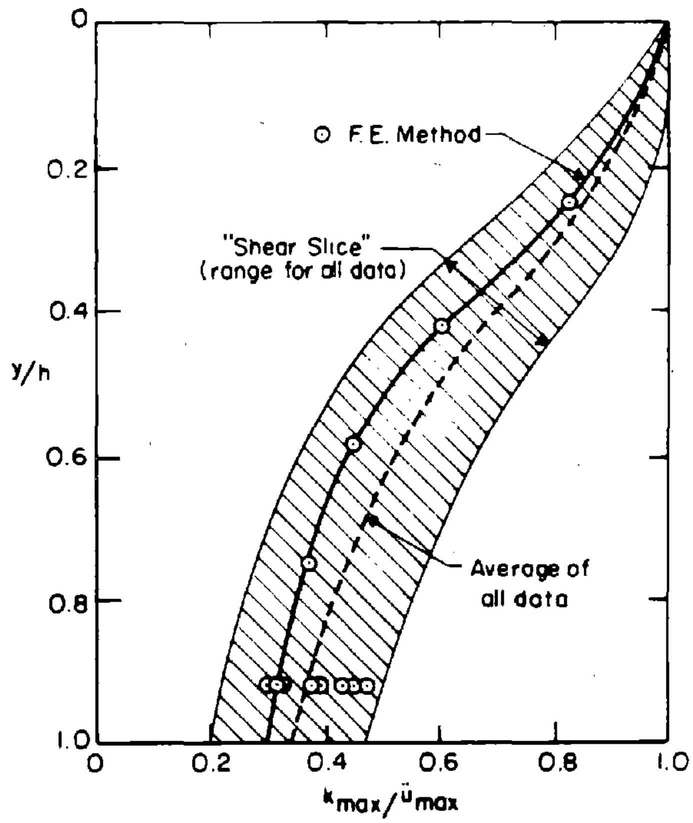


FIG. 9 VARIATION OF "MAXIMUM ACCELERATION RATIO" WITH DEPTH OF SLIDING MASS

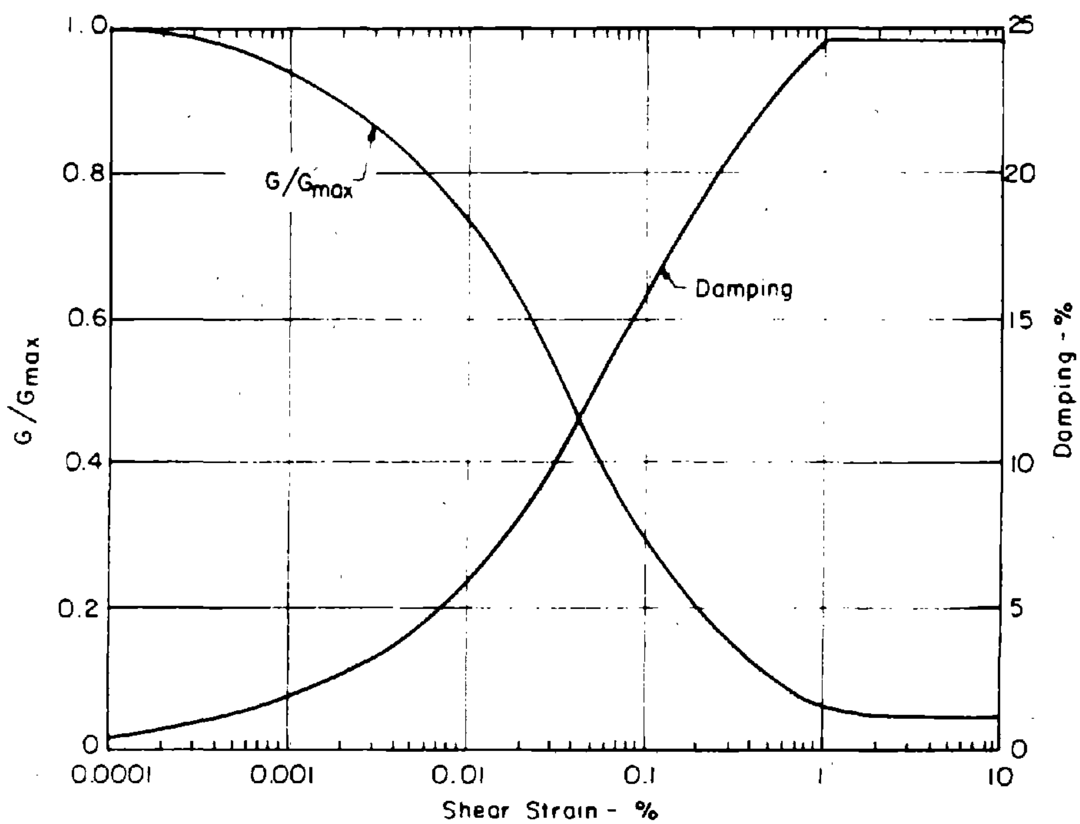


FIG. 10 SHEAR MODULUS AND DAMPING CHARACTERISTICS USED IN RESPONSE COMPUTATIONS

crest acceleration is considered accurate enough for practical purposes. For design computations where a conservative estimate of the accelerations is desired the upper bound curve shown in Fig. 9 may be used leading to values which are 10 to 30% higher than those estimated using the average relationship.

CALCULATION OF PERMANENT DEFORMATIONS

Once the yield acceleration and the time-history of average induced acceleration for a potential sliding mass have been determined, the permanent displacements can readily be calculated.

By assuming a direction of the sliding plane and writing the equation of motion for the sliding mass along such a plane, the displacements which would occur any time the induced acceleration exceeds the yield acceleration may be evaluated by simple numerical integration. For the purposes of the soil types considered in this study, the yield acceleration was assumed to be constant throughout the earthquake.

The direction of motion for a potential sliding mass once yielding occurs was assumed to be along a horizontal plane. This mode of deformation is not uncommon for embankments subjected to strong earthquake shaking, and is manifested in many cases in the field by the development of longitudinal cracks along the crest of the embankment. However studies made for other directions of the sliding surface showed that this factor had little effect on the computed displacements.

To calculate an order of magnitude of the deformations induced in embankments due to strong shaking a number of cases have been analyzed during the course of this study. The height of embankments considered ranged between 75 and 150 ft with varying slopes and material properties. The embankments were subjected to ground accelerations representing three different earthquake magnitudes: 6-1/2, 7-1/2 and 8-1/4.

The method used for calculating the response, as mentioned earlier, is a time-step finite element analysis using the equivalent linear method. The strain dependent modulus and damping relations for the soils used in this study are presented in Fig. 10. The response computation for each base motion was repeated for a number of iterations (mostly 3 to 4) until strain compatible material properties were obtained. In each case both time histories of crest acceleration and the average acceleration for a potential sliding mass extending through almost the full height of the embankment were calculated, together with the first natural period of the embankment. In one case however, time histories of average acceleration for sliding surfaces at 5 different levels in the embankment were obtained (see Fig. 4), and the corresponding permanent deformations for each time-history were calculated for different values of yield acceleration. It was found that for the same ratio of yield acceleration to maximum average acceleration at each level, the computed deformations varied uniformly between a maximum value obtained using the crest acceleration time-history to a minimum value obtained using the time-history of average acceleration for a sliding mass extending through the full height of the embankment. Thus it was considered sufficient for the remaining cases to compute the deformations only for these two levels.

Table 2 shows details of the embankments analyzed using ground motions representative of a magnitude 6-1/2 earthquake. The two rock motions used were those recorded at the Cal Tech Seismographic Laboratory (S90W Component) and at Lake Hughes Station #12 (N21E) during the 1971 San Fernando earthquake, with maximum accelerations scaled to 0.2 and 0.5g. The computed natural periods and maximum values of the acceleration time-histories are also presented in Table 2. The computed natural periods ranged between a value of 0.6 seconds for the 75-ft-high embankment to a value of 1.08 seconds for the 150-ft-high embankment. Because of the non-linear strain-dependent behavior of the material, the response of the embankment is highly dependent on the amplitude of the base motion. This is clearly demonstrated in the first two cases in Table 2, where the same embankment was subjected to the same ground acceleration history but with different maximum accelerations for each case. In one instance, for a base acceleration of 0.2g the calculated maximum crest accelerations was 0.3g with a magnification factor of 1.5 and a computed natural period of the order of 0.8 seconds. In the second case, for a base acceleration of 0.5g the computed maximum crest acceleration was 0.4g with an attenuation factor of 0.8 and a computed natural period of 1.1 seconds.

From the time-histories of induced acceleration calculated for all the cases described in Table 2 and for various ratios of yield acceleration to maximum average acceleration, k_y/k_{max}, the permanent deformations were calculated by numerical double integration. The results are presented in Fig. 11 which shows that for relatively low values of yield acceleration, k_y/k_{max} of 0.2 for example, the range of computed permanent displacements was of the order of 10 to 70 cm (0.3 - 2.5 ft). However, for larger values of k_y/k_{max}, say 0.5 or more, the calculated displacements were less than 12 cm (0.5 ft). It should be emphasized that for very low values of yield accelerations (in this case $k_y/k_{max} \leq 0.1$) the basic assumptions used in calculating the response by the finite element method, namely the equivalent linear behavior and the small strain theory, become invalid. Consequently the acceleration time-histories calculated for such a case do not represent the real field behavior and the calculated displacements based on these time-histories may not be realistic.

The procedure described above was repeated for the case of a magnitude 7-1/2 earthquake. The base acceleration time-history used for this analysis was that recorded at Taft during the 1952 Kern County earthquake and scaled to

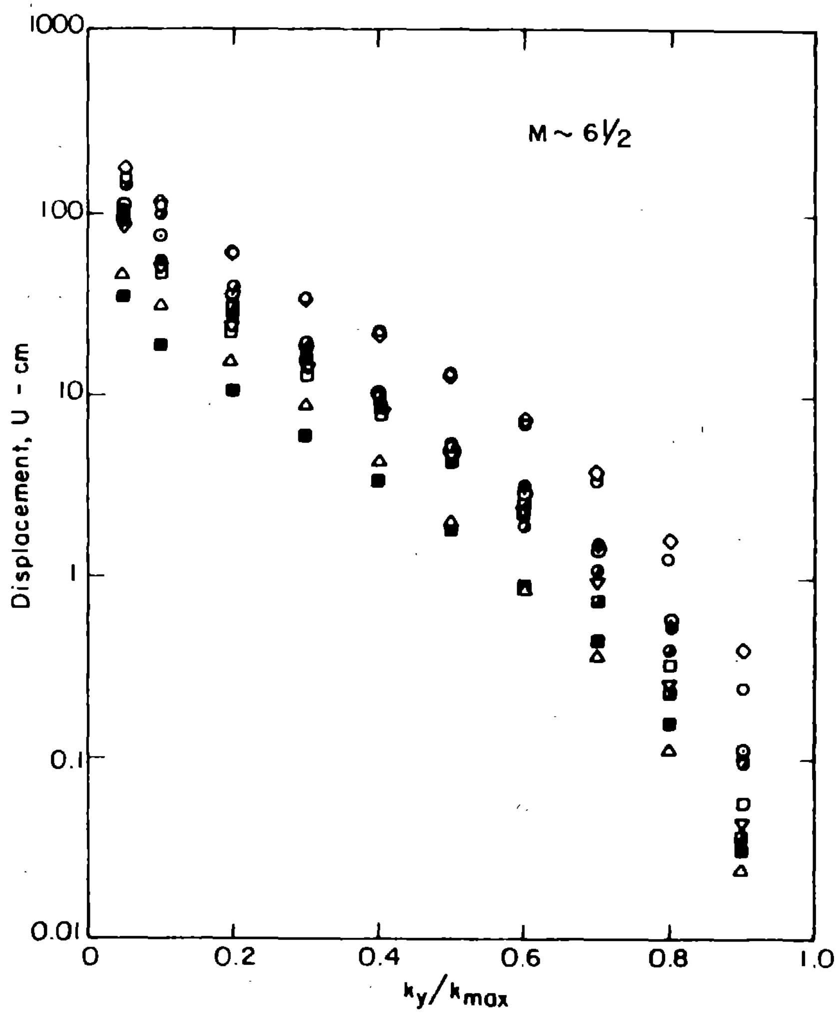


FIG. 11 VARIATION OF PERMANENT DISPLACEMENT WITH YIELD ACCELERATION - MAGNITUDE 6-1/2 EARTHQUAKE

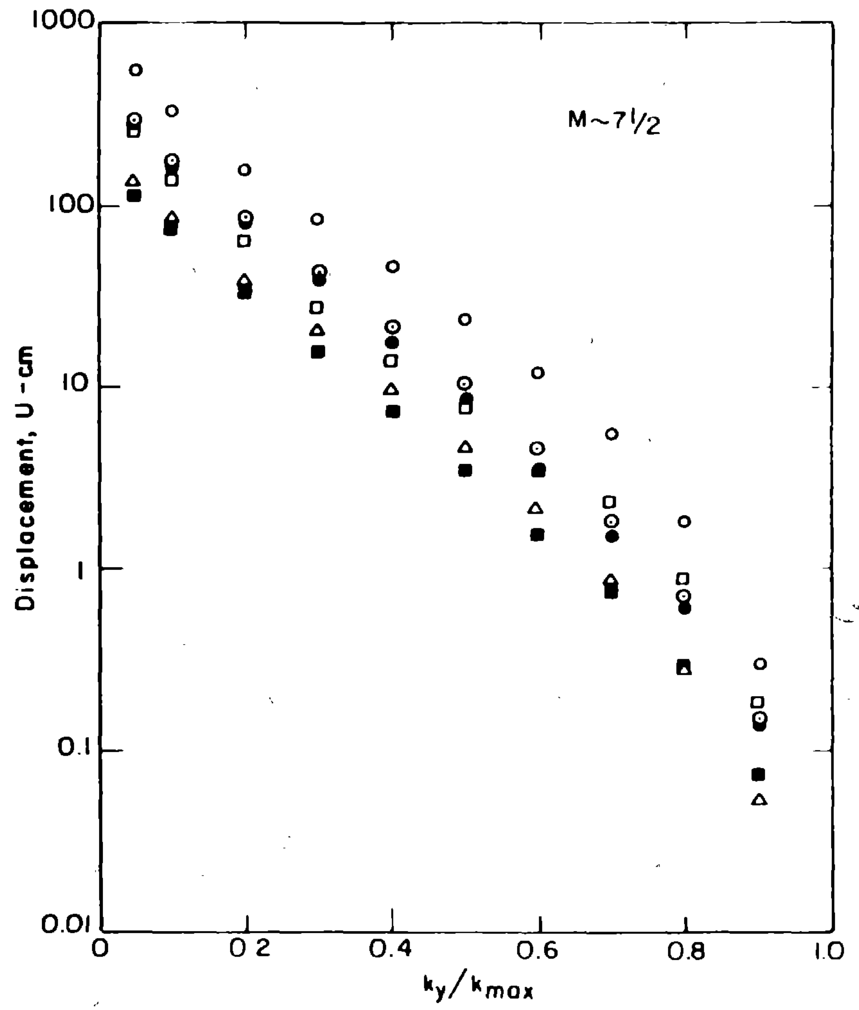


FIG. 12 VARIATION OF PERMANENT DISPLACEMENT WITH YIELD ACCELERATION - MAGNITUDE 7-1/2 EARTHQUAKE

maximum accelerations of 0.2 and 0.5g. The details of the 3 cases analyzed are presented in Table 3 and the results of the computations of the permanent displacements are shown in Fig. 12. For a ratio of k_y/k_{max} of 0.2 the calculated displacements in this case ranged between 30 and 200 cm (1 and 6 ft), and for ratios greater than 0.5 the displacements were less than 25 cm (0.8 ft).

In the cases analyzed for the 8-1/4 magnitude earthquake, an artificial accelerogram proposed by Seed and Idriss (1969) was used with maximum base accelerations of 0.4 and 0.75g. Two embankments were analysed in this case and their calculated natural periods ranged between 0.8 and 1.5 seconds. Table 4 shows the details of the calculations and in Fig. 13 the results of the permanent displacement computations are presented. As can be seen from Fig. 13 the permanent displacements computed for a ratio of k_y/k_{max} of 0.2 ranged between 200 and 700 cm (6 and 23 ft), and for ratios higher than 0.5 the values were less than 100 cm (3 ft). It should be noted in this case that values of deformations calculated for a yield ratio less than 0.2 may not be realistic.

An envelope of the results obtained for each of the three earthquake loading conditions is presented in Fig. 14 and reveals a large scatter in the computed results reaching, in the case of the magnitude 6-1/2 earthquake, about one order of magnitude.

It can reasonably be expected that for a potential sliding mass with a specified yield acceleration, the magnitude of the permanent deformation induced by a certain earthquake loading is controlled by the following factors:

a) the amplitude of induced average accelerations, which is a function of the base motion, the amplifying characteristics of the embankment, and the location of the sliding mass within the embankment;

b) the frequency content of the average acceleration time-history, which is governed by the embankment height and stiffness characteristics,

Table 3

Embankment Characteristics for Magnitude 7-1/2 Earthquake

Case #	Embankment Description	Height (ft.)	Base Acceleration (g)	T_o (1)	k_{max} (2) (g)	Symbol
1	Example Case - slope = 2:1 - k_{2max} = 60	150	0.2 (Taft record)	0.86	(a) 0.41 (b) 0.13	● ■
2	Example Case - slope = 2:1 - k_{2max} = 60	150	0.2 (Taft record)	1.18	(a) 0.54 (b) 0.21	○ □
3	Example Case - slope = 2-1/2:1 - k_{2max} = 80	150	0.2 (Taft record)	0.76	(a) 0.46 (b) 0.15	⊙ △

(1) T_o = Calculated first natural period of the embankment.

(2) k_{max} = Maximum value of time history of:
(a) crest acceleration
(b) average acceleration for sliding mass extending through full height of embankment.

Table 4

Embankment Characteristics of Magnitude 8-1/4 Earthquake

Case #	Embankment Description	Height (ft.)	Base Acceleration (g)	T_o (1)	k_{max} (2) (g)	Symbol
1	Chabot Dam (average properties)	135	0.4 (S-I Synth. record)	0.99	(a) 0.57	○
	Chabot Dam (Lower bound)	135	0.4 (S-I Synth. record)	1.07	(a) 0.53	△
	Chabot Dam (Upper bound)	135	0.4	0.83	(a) 0.68	□
2	Example Case - slope = 2:1 - k_{2max} = 60	150	0.75	1.49	(a) 0.74 (b) 0.34	● ■

(1) T_o = Calculated first natural period of the embankment.

(2) k_{max} = Maximum value of time history of:
(a) crest acceleration
(b) average acceleration for sliding mass extending through full height of embankment.

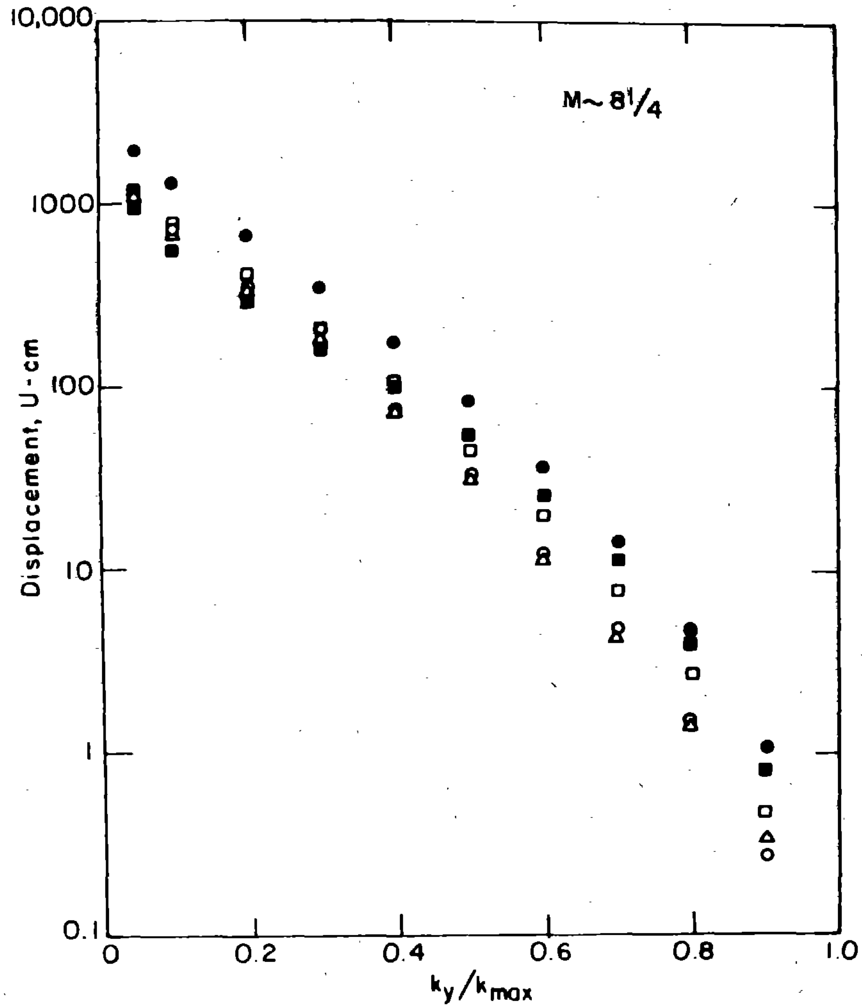


FIG. 13 VARIATION OF PERMANENT DISPLACEMENT WITH YIELD ACCELERATION - MAGNITUDE 8-1/4 EARTHQUAKE

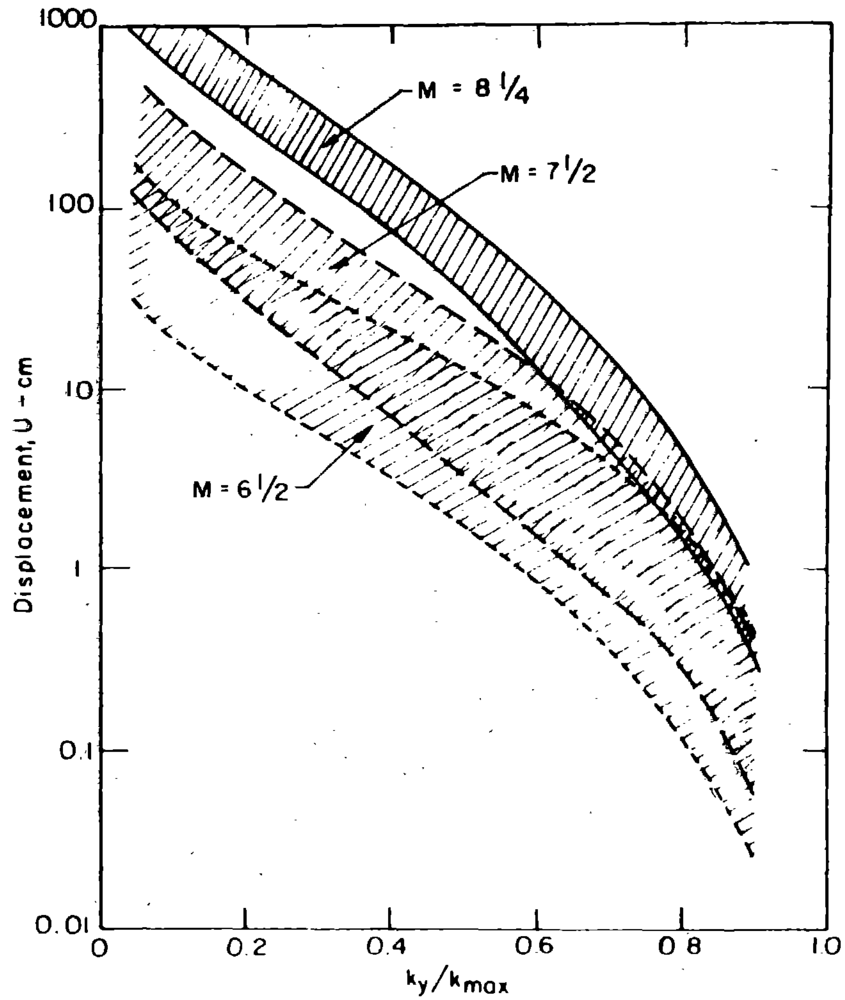


FIG. 14 VARIATION OF PERMANENT DISPLACEMENT WITH YIELD ACCELERATION - SUMMARY OF ALL DATA

and is usually dominated by the first natural frequency of the embankment;

c) the duration of significant shaking, which is a function of the magnitude of the specified earthquake.

Thus to reduce the large scatter exhibited in the data in Fig. 14, the permanent displacements for each embankment were normalized with respect to its calculated first natural period, T_o, and with respect to the maximum value, k_{max}, of the average acceleration time-history used in the computation. The resulting normalized permanent displacements for the three different earthquakes are presented in Fig. 15. It may be seen that a substantial reduction in the scatter of the data is achieved by this normalization procedure as evidenced by comparing the results in Figs. 14 and 15. This shows that for the ranges of embankment heights considered in this study (75 to 150 ft or 50 to 65 meters) the first natural period of the embankment and the maximum value of acceleration time-history may be considered as two of the parameters having a major influence on the calculated permanent displacements. Average curves for the normalized permanent displacements based on the results in Fig. 15 are presented in Fig. 16. Although some scatter still exists in the results as shown in Fig. 15, the average curves presented in Fig. 16 are considered adequate to provide an order of magnitude of the induced permanent displacements for different magnitude earthquakes. At yield acceleration ratios less than 0.2 the average curves are shown as dashed lines since, as discussed earlier, the calculated displacements at these low ratios may be unrealistic.

Thus to calculate the permanent deformation in an embankment constructed of a soil which does not change in strength significantly during an earthquake, it is sufficient to determine its maximum crest acceleration, $\ddot{u}_{max}$, and first natural period, T_o, due to a specified earthquake. Then by the use of the relationship presented in Fig. 9, the maximum value of average acceleration

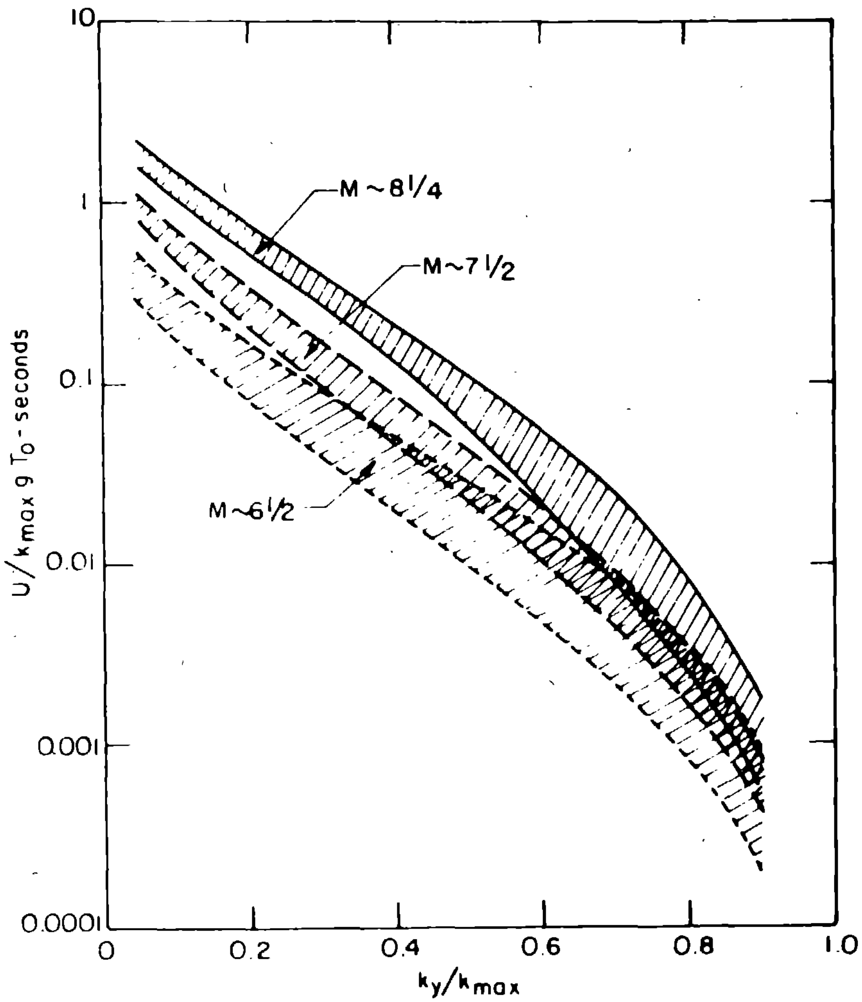


FIG. 15 VARIATION OF NORMALIZED PERMANENT DISPLACEMENT WITH YIELD ACCELERATION - SUMMARY OF ALL DATA

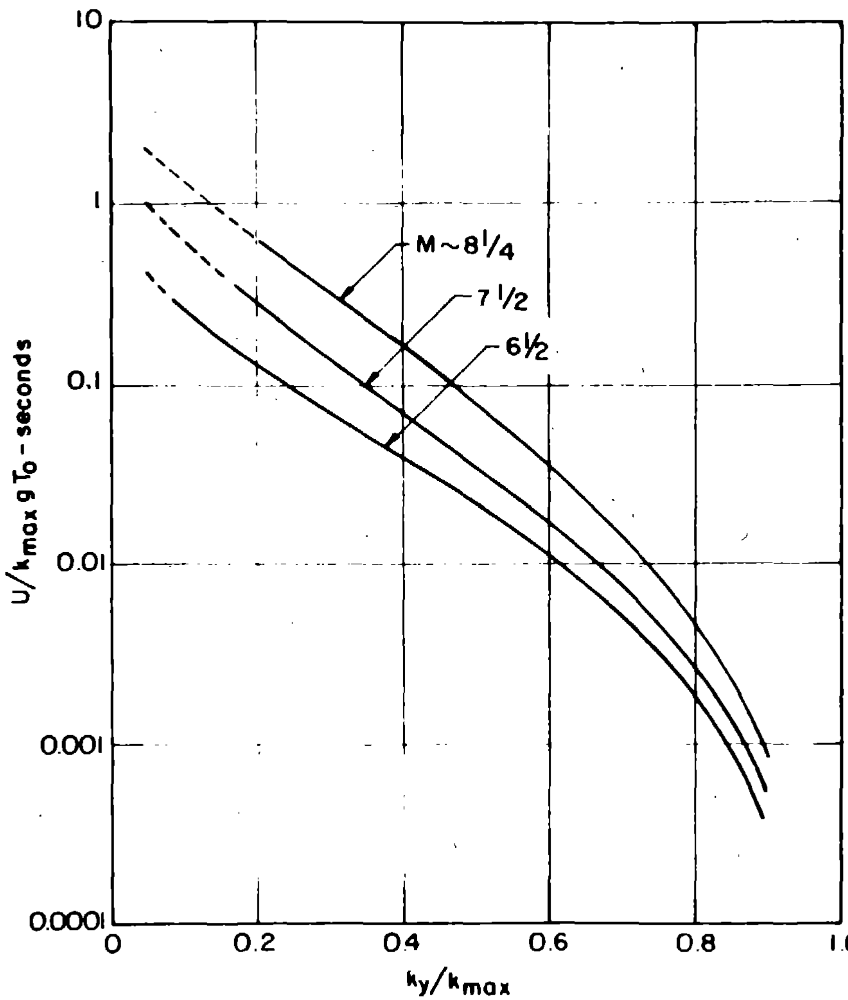


FIG. 16 VARIATION OF AVERAGE NORMALIZED DISPLACEMENT WITH YIELD ACCELERATION

history, k_{max}, for any level of the specified sliding mass may be determined. Entering the curves in Fig. 16 with the appropriate values of k_{max} and T_o, the permanent displacements can be determined for any value of yield acceleration associated with that particular sliding surface.

It has been assumed earlier in this paper that in the majority of embankments permanent deformations usually occur due to slip of a sliding mass on a horizontal failure plane. For those few instances where sliding might occur on an inclined failure plane it is of interest to determine the difference between the actual deformations and those calculated with the assumption of a horizontal failure plane having the same yield acceleration. A simple computation was made to investigate this condition using the analogy of a block on an inclined plane for a purely frictional material. It was found that for inclined failure planes with slope angles of 15° to the horizontal, the computed displacements were 10 to 18% higher than those based on a horizontal plane assumption.

APPLICATION OF METHOD TO AN EMBANKMENT SUBJECTED TO AN 8-1/4 MAGNITUDE EARTHQUAKE

To illustrate the use of the simplified procedure for evaluating earthquake-induced deformations, computations are presented below for the 135 ft high Chabot Dam, constructed of sandy clay and having the section shown in Fig. 17.

The shear wave velocity of the embankment was determined from a field investigation and the strain-dependent modulus and damping were determined from laboratory tests on undisturbed samples. The dam, located about 20 miles from the San Andreas fault, was shaken in 1906 by the magnitude 8-1/4 San Francisco earthquake with no significant deformations being noted; peak accelerations in the rock underlying the dam in this event are estimated to have been about 0.4g. Accordingly the response of the embankment to ground

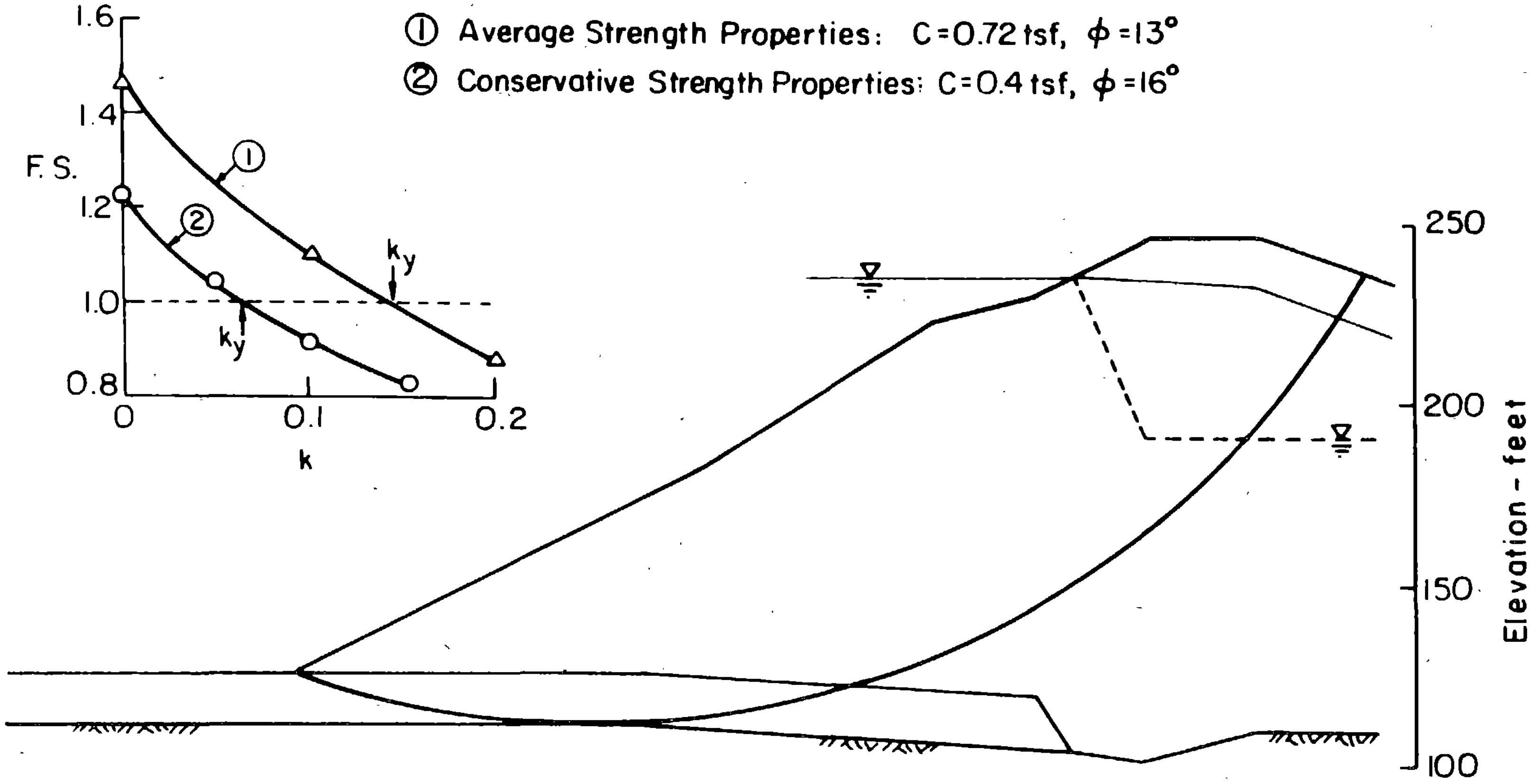


FIG. 17 YIELD ACCELERATION VALUES FOR SLIDE MASS EXTENDING THROUGH FULL HEIGHT OF EMBANKMENT.

accelerations representative of a magnitude 8-1/4 earthquake and having a maximum acceleration of 0.4g was calculated by a finite element analysis. The maximum crest acceleration of the embankment, $\ddot{u}_{max}$, was calculated to be 0.57g and the first natural period, T_o = 0.99 seconds. The maximum values of the calculated shear strain were less than 0.5%. On the basis of static undrained tests on the embankment material, the static failure strains ranged between 3% and 8%, so that for the purposes of this analysis the cyclic yield strength of this material can be considered equal to its static undrained strength. From consolidated undrained tests on representative samples of the embankment material two interpretations were made for the strength of the material: one, based on an average of all the samples tested resulting in a cohesion value, c, of 0.72 tons/sq. ft (or 0.72 kg/cm^2) and a friction angle, ϕ, of 13°; the other, a conservative interpretation, based on the minimum strength values with a cohesion of 0.4 tons/sq. ft (or 0.4 kg/cm^2) and a friction angle of 16°. Using these strength estimates, values of yield accelerations were calculated for a sliding mass extending through the full height of the embankment as shown in Fig. 17.

Considering the average relationship of $k_{max}/\ddot{u}_{max}$ with depth shown in Fig. 9, the ratio for a sliding mass extending through the full height of the embankment is 0.35, resulting in a maximum average acceleration, k_{max}, of $0.35 \times 0.57g = 0.2g$. From Fig. 17 the yield acceleration calculated for the average strength values is 0.14g. Thus the parameters to be used in Fig. 16 to calculate the displacements for this particular sliding surface are as follows:

$$\text{Magnitude} \simeq 8\text{-}1/4$$

$$T_o = 0.99 \text{ sec.}$$

$$k_{max} = 0.2$$

$$k_y/k_{max} = \frac{0.14}{0.20} = 0.7$$

From Fig. 16:

$$U/k_{max}g\ T_o = 0.013 \text{ seconds}$$

$\therefore$ The displacement $U = 0.013 \times 0.2 \times 32.2 \times 0.99 = 0.08$ ft. (or 2.4 cm).

Using the most conservative value of $k_{max}/\ddot{u}_{max}$ shown in Fig. 9 of 0.47, the computed displacement would have been 0.58 ft (17.5 cm). Similarly using the conservative strength parameters for the soil (giving $k_y = 0.07$) and the average curve for $k_{max}/\ddot{u}_{max}$ shown in Fig. 9, the computed displacement would have been 1.5 ft (45 cms). All of these values are in reasonable accord with the observed performance of the dam during the 1906 earthquake.

The calculation was repeated for a sliding mass extending through half the depth of the embankment. The computed permanent displacements ranged between 0.02 and 1.08 ft (0.6 to 32 cms) indicating that the critical potential sliding mass in this case was that extending through the full height of the embankment.

CONCLUSION

A simple yet rational approach to the design of small embankments under earthquake loading has been described herein. The method is based on the concept of permanent deformations as proposed by Newmark (1965) but modified to allow for the dynamic response of the embankment as proposed by Seed and Martin (1966) and restricted in application to compacted clayey embankments and dry or dense cohesionless soils which experience very little reduction in strength due to cyclic loading. The method is an approximate one and involves a number of simplifying assumptions which may lead to somewhat conservative results.

On the basis of response computations for embankments subjected to different ground motion records, a relationship for the variation of induced average acceleration with embankment depth has been established. Design curves to estimate the permanent deformations for embankments, in the height range of 100 to 200 ft, have been established based on equivalent-linear finite element dynamic analyses for different magnitude earthquakes. The use of these curves requires a knowledge of the maximum crest acceleration and the natural period of an embankment due to a specified ground motion.

It should be noted that the design curves presented are based on averages of a range of results which exhibit some degree of scatter, and are derived from a limited number of cases. These curves should be updated and refined as analytical results for more embankments are obtained.

Finally, the method has been applied to an actual embankment which was subjected to a magnitude 8-1/4 earthquake at an epicentral distance of some 20 miles. Depending on the degree of conservatism in estimating the undrained strength of the material and in estimating the maximum accelerations in the embankment, the calculated deformations for this 135 ft clayey embankment ranged between 0.1 ft and 1.5 ft. These approximate displacement values are in good accord with the actual performance of the embankment during the earthquake.

Whereas the method described above provides a rational approach to the design of embankments and offers a significant improvement over the conventional pseuso-static approach, the nature of the approximations involved requires that it be used with caution and good judgment especially in determining the soil characteristics of the embankment to which it may be applied.

For large embankments, for embankments where failure might result in a loss of life or major damage and property loss, or where soil conditions cannot be determined with a significant degree of accuracy to warrant the use of the method, the more rigorous dynamic method of analysis described earlier might well provide a more satisfactory alternative for design purposes.

Acknowledgment

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Appendix A

A SIMPLIFIED PROCEDURE FOR COMPUTING MAXIMUM CREST ACCELERATION AND NATURAL PERIOD FOR EMBANKMENTS

A SIMPLIFIED PROCEDURE FOR COMPUTING MAXIMUM CREST ACCELERATION AND NATURAL PERIOD FOR EMBANKMENTS

by

F. I. Makdisi[1] and H. Bolton Seed[2]

Introduction

For many types of embankments, constructed of dry or very dense sands or clayey soils, an estimate of the magnitude of the deformations which might be induced by earthquake shaking can be made from a knowledge of the yield acceleration for a potential sliding mass, the maximum crest acceleration induced at the crest of the dam by the earthquake and the natural period of vibration of the dam (Makdisi and Seed, 1977). The yield acceleration, that is, the average acceleration at which a condition of incipient failure is induced in the potential sliding mass is determined by the strength parameters of the soil and an appropriate method of stability analysis.

The maximum crest acceleration induced in the embankment and the natural period of the embankment can readily be determined either by a finite element analysis (Clough and Chopra, 1966; Idriss and Seed, 1967) of the embankment section or by a shear slice analysis (Ambraseys, 1960; Seed and Martin, 1966). For many purposes, however, a simplified procedure may provide evaluations of these embankment characteristics with sufficient accuracy for many practical purposes. Such a procedure, which enables the determination, by hand calculation, of the maximum crest acceleration and the natural period of an embankment due to a specified earthquake loading is described in the following pages. The method also allows, through

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iteration, the use of strain dependent material properties. The steps involved in this procedure are described in the following paragraphs, and an example problem is solved to compare the results from the approximate method to those obtained from a finite element solution.

1. Evaluation of initial properties

Consider the dam section shown in Fig. 1(a) with height H, shear wave velocity v_{max}, and a mass density ρ. The section is assumed to be homogeneous and of infinite length. The maximum shear modulus, G_{max}, is related to the shear wave velocity, v_{max}, by the relation:

$$G_{max} = \rho \, v^2_{max} \tag{1}$$

For the first iteration of computations, assume any initial value of shear modulus G, and determine the ratio G/G_{max}. From Fig. 1(b), for the calculated value of G/G_{max} the corresponding values of shear strain, γ_{ave}, and damping, λ, could then be determined.

2. Calculation of maximum acceleration and natural period

In the derivation of the shear slice theory for a dam section with the properties described above, the expression for the acceleration at any level, y, as a function of time is given by:

$$\ddot{u}(y,t) = \sum_{n=1}^{\infty} \frac{2J_o\ (\beta_n y/h)}{\beta_n\ J_1\ \beta_n}\ \omega_n\ V_n(t) \tag{2}$$

where: J_o, J_1 = Bessel functions of first kind of order zero and one

β_n = the zero value of the frequency equation $J_o(\omega h\sqrt{\rho/G}) = 0$

ω_n = $\beta_n\ v_s/h$, where $v_s = \sqrt{G/\rho}$

= natural frequency of the n^{th} mode.

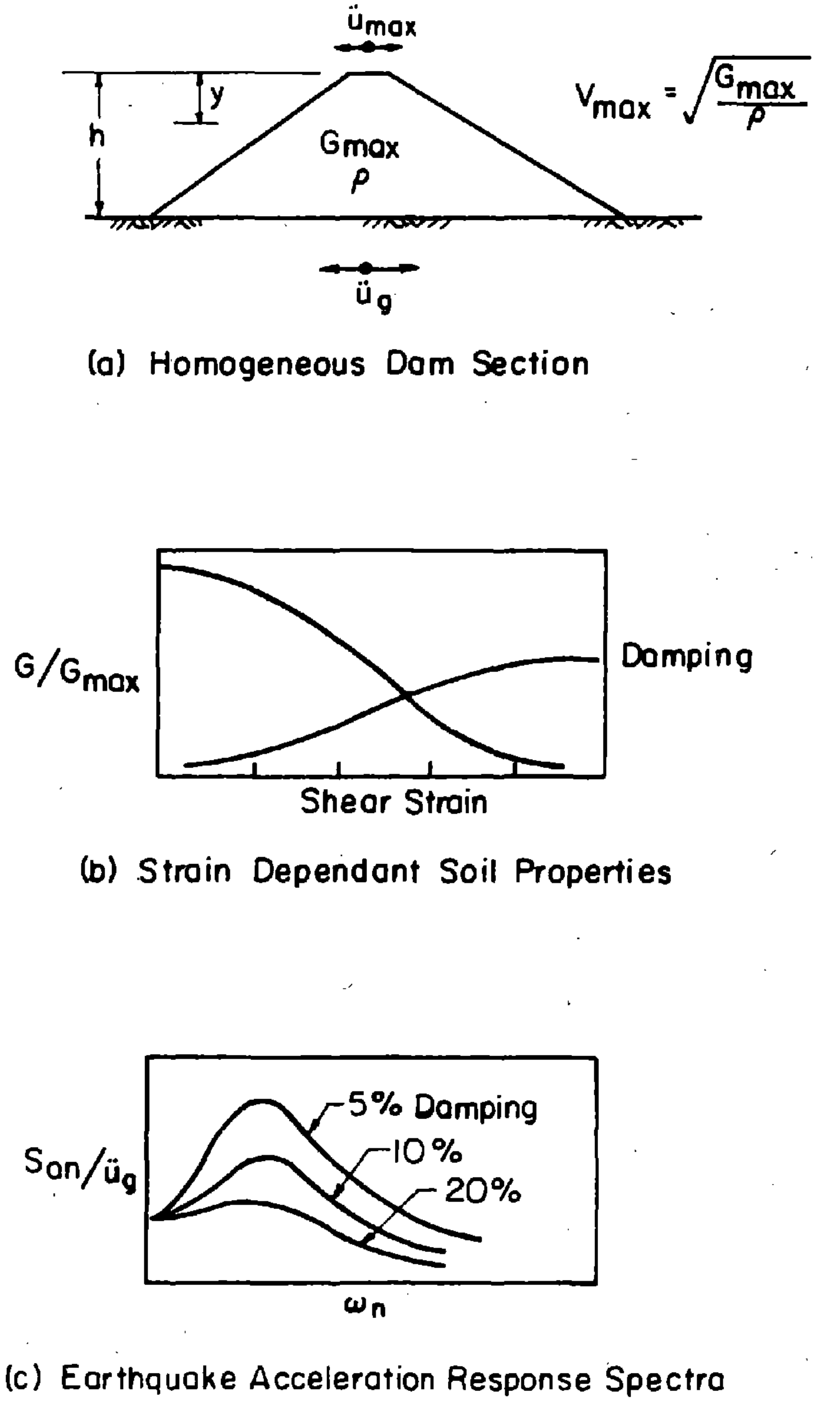

FIG. 1 CALCULATION OF MAXIMUM CREST ACCELERATION AND NATURAL PERIOD - (APPROXIMATE PROCEDURE).

$V_n(t)$ known as Duhammel's integral is given by the expression:

$$V_n(t) = \int_o^t \ddot{u}_g \, e^{-\lambda_n \omega_n (t-\tau)} \sin[\omega_{dn}(t-\tau)]\, d\tau \tag{3}$$

where: $\omega_{dn} = \omega_n \sqrt{1-\lambda_n^{\ 2}} \approx \omega_n$ for small values of λ_n

λ_n = fraction of critical damping

Thus,
$$\ddot{u}(y,t) = \sum_{n=1}^{\infty} \phi_n(y)\ \omega_n V_n(t) \tag{4}$$

where: $\phi_n(y) = \dfrac{2J_o(\beta_n y/h)}{\beta_n J_1(\beta_n)}$

= mode participation factor.

Considering the first three modes of vibration, the corresponding values of β_n are always: $\beta_1 = 2.4$, $\beta_2 = 5.52$, $\beta_3 = 8.65$, and the corresponding values of the first natural frequencies are:

$$\begin{aligned} \omega_1 &= 2.4\ v_s/h \\ \omega_2 &= 5.52\ v_s/h \\ \omega_3 &= 8.65\ v_s/h \end{aligned} \tag{5}$$

At the crest of the dam, y = 0, and the corresponding values of the mode participation factors $\phi_n(o)$ for the first three modes are given by:

$$\begin{aligned} \phi_1(o) &= 1.6 \\ \phi_2(o) &= 1.06 \\ \phi_3(o) &= 0.86 \end{aligned} \tag{6}$$

∴ The value of acceleration at the crest in each mode is given by the expression:

$$\ddot{u}_n(o,t) = \phi_n(o)\ \omega_n\ V_n(t) \tag{7}$$

and the maximum value of the crest acceleration in each mode is given by:

$$\ddot{u}_{n_{max}} = \phi_n(o)\ \omega_n S_{vn} \tag{8}$$

where S_{vn}, known as the spectral velocity, is the maximum value of $V_n(t)$, and is a function of ω_n, λ_n and the characteristics of the ground motion $\ddot{u}_g(t)$. For small values of λ_n the spectral acceleration S_{an}, is approximately equal to $\omega_n S_{vn}$ and thus the expression for the maximum crest acceleration for each mode could be written as:

$$\ddot{u}_{n_{max}} = \phi_n(o)\ S_{an} \tag{9}$$

The value of S_{an} as a function of ω_n and λ_n is readily available for most earthquake ground motion records and average values have been published by various authors (Housner, 1959; Newmark and Hall, 1969; Newmark, Blume and Kapur, 1973; Seed, Ugas and Lysmer, 1976).

The maximum crest acceleration for the first three modes is thus given by

$$\begin{aligned} \ddot{u}_{1_{max}} &= \phi_1(o)\ S_{a1} = 1.6\ \ S_{a1} \\ \ddot{u}_{2_{max}} &= \phi_3(o)\ S_{a2} = 1.06\ S_{a2} \\ \ddot{u}_{3_{max}} &= \phi_3(o)\ S_{a3} = 0.86\ S_{a3} \end{aligned} \tag{10}$$

As the maximum values in each mode occur at different times, the maximum values of the crest acceleration is approximated by taking the square root of the sum of squares of the maximum acceleration of the first three modes, hence:

$$\ddot{u}_{max} = \left[\sum_{n=1}^{3} (\ddot{u}_{n_{max}})^2 \right]^{1/2} \tag{11}$$

Therefore, having determined the value of v_s and λ in step (1), Eq. 5 is then used to determine the corresponding values of the first three natural frequencies. These in turn are used in Fig. 1(c) to determine the corresponding values of spectral acceleration and with the aid of Eqs. 10 and 11 the value of the maximum crest acceleration is readily determined.

3. Determination of average shear strain

To estimate the strain compatible material properties, an expression for the average shear strain over the entire section should be determined. From the shear slice theory, the expression for shear strain at any level in the embankment as a function of time is given by:

$$\gamma(y,t) = \sum_{n=1}^{\infty} \frac{2J_1\ (\beta_n y/h)}{h\ \omega_n\ J_1\ (\beta_n)}\ V_n(t) \tag{12}$$

$$= \frac{h}{v_s^2} \sum_{n=1}^{\infty} \frac{2J_1(\beta_n y/h)}{\beta_n^2\ J_1\ (\beta_n)}\ \omega_n V_n(t)$$

Thus

$$\gamma(y,t) = \frac{h}{v_s^2} \sum_{n=1}^{\infty} \phi_n'(y)\ \omega_n V_n(t) \tag{13}$$

where:

$$\phi_n'(y) = \frac{2J_1\ (\beta_n y/h)}{\beta_n^2\ J_1\ (\beta_n)} \tag{14}$$

= shear strain mode participation factor.

The variation of ϕ_n' with depth for the first three modes (after Martin, 1965) is shown in Fig. 2. Considering the small contributions of the higher modes compared to the first mode over the entire depth, it is sufficient for all practical purposes to consider the contributions of the first mode only in calculating the average shear strain. Thus from Eq. 13 the expression for the maximum shear strain at any level, y, may be written as:

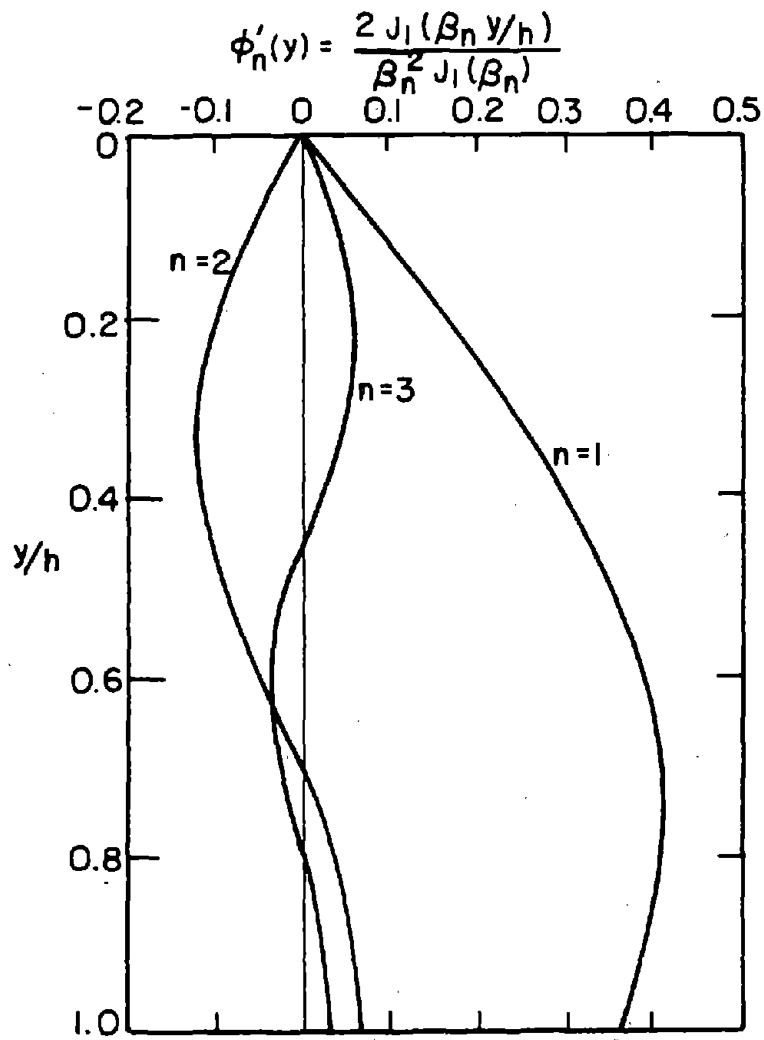

FIG. 2 VARIATION OF SHEAR STRAIN MODE PARTICIPATION FACTORS WITH DEPTH - SHEAR SLICE THEORY. (After Martin, 1965).

$$\gamma_{max}(y) = \frac{h}{v_s^2} \phi_1'(y) \; S_{a1} \tag{15}$$

where ϕ_1' is the first mode participation factor as shown in Fig. 2, and S_{a1} is the spectral acceleration corresponding to the first natural frequency ω_1.

The average maximum shear strain for the entire section may be determined by calculating an average value $\left(\phi_1'\right)_{ave}$ of the first mode participation factor in Fig. 2:

$$\therefore \left(\phi_1'\right)_{ave} \approx \frac{1}{5} (0.38 + 0.41 + 0.35 + 0.24 + 0.1) \tag{16}$$

$$\approx 0.3$$

and

$$(\gamma_{ave})_{max} = \frac{h}{v_s^2}\left(\phi_1'\right)_{ave} S_{a1} \tag{17}$$

Assuming the equivalent cyclic shear strain is approximately 65% of the maximum average shear strain, $(\gamma_{ave})_{max}$, then

$$(\gamma_{ave})_{eq} = 0.65 \times 0.3 \times \frac{h}{v_s^2} \; S_{a1} \tag{18}$$

Having obtained a new value for the average shear strain from Eq. 18 a new set of modulus and damping values can be determined from Fig. 1(b). If these values are different from those assumed in step (1), a new iteration must be performed starting from step (2) and the process repeated until strain compatible properties are obtained. The process usually converges in 3 iterations.

Example Problem

To evaluate the accuracy of the approximate method proposed above, the response of a 150-ft-high embankment, for which a solution has been obtained by the finite element method, will be calculated herein.

In the finite element solution the maximum shear modulus was calculated as a function of the square root of the effective confining pressure; therefore a weighted average based on values from all the elements was calculated and that value was used for the homogeneous section analyzed by the approximate method. The properties of the embankment are as follows:

Height, h	= 150 ft
Unit weight, γ	= 130 pcf
Mass density, ρ	= 4.04 slugs/ft^3
Maximum shear modulus, G_{max}	= 3650 ksf (average of all elements)
Maximum shear wave velocity, v_{max}	= 950 fps

The finite element response was calculated for ground accelerations obtained from the N-S component of the Taft record of the Kern County (1952) earthquake, adjusted to have a maximum acceleration of 0.2g. A plot of the normalized response spectra for this record is shown in Fig. 3. The strain dependent modulus and damping properties used in the computations are presented in Fig. 4.

Iteration #1

Assume $v_s = 600$ fps.

$\therefore\ v_s/v_{max} = 600/950 = 0.63$

and $G/G_{max} = (v_s/v_{max})^2 = 0.4$

From Fig. 4 : for $G/G_{max} = 0.4$

shear strain = 0.06%

and λ = 13%

From Eq. 5:

$\omega_1 = 2.4 \times 600/150 = 9.6$ rad/sec, $T_1 = 0.65$ sec.

$\omega_2 = 5.52 \times 600/150 = 22.1$ rad/sec, $T_2 = 0.284$ sec.

$\omega_3 = 8.65 \times 600/150 = 34.6$ rad/sec, $T_3 = 0.182$ sec.

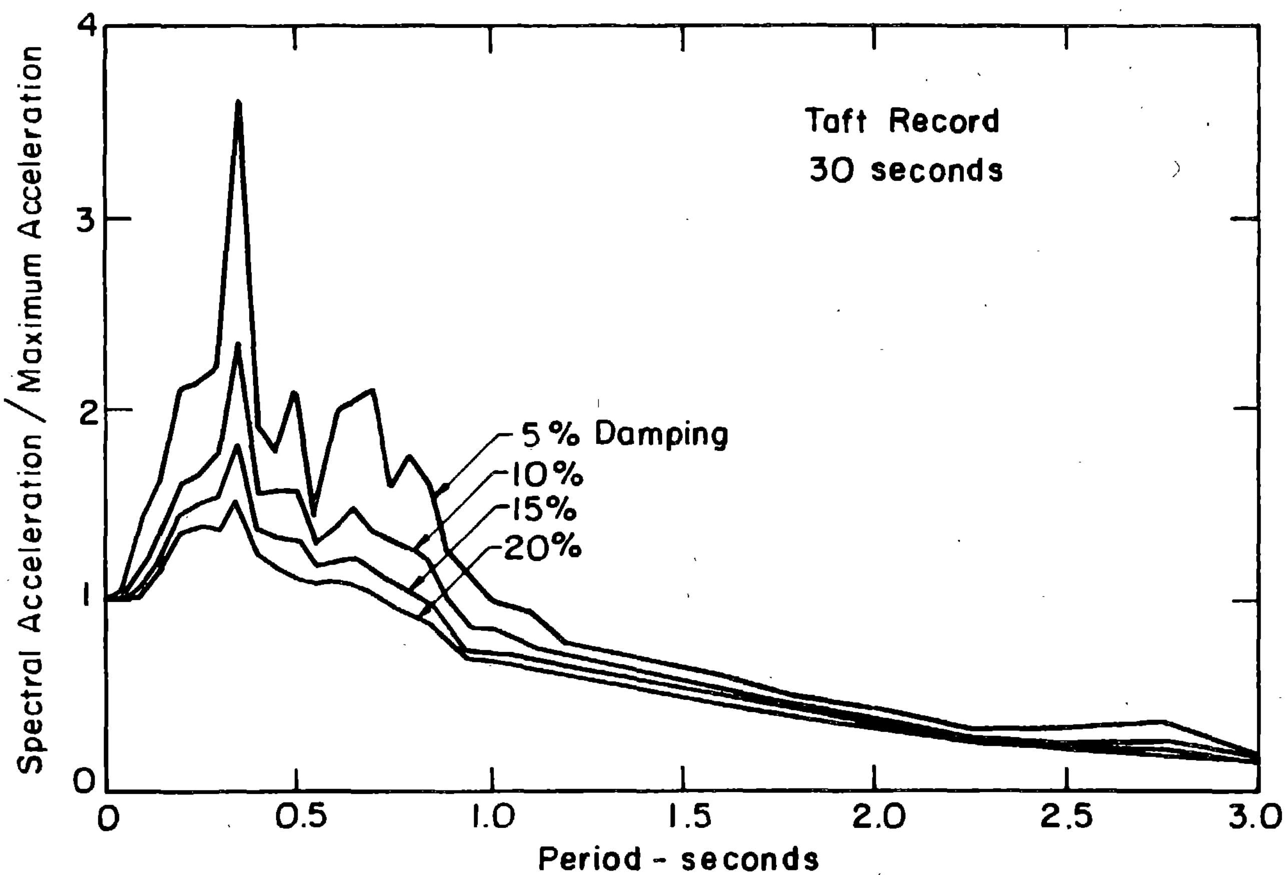

FIG. 3 NORMALIZED ACCELERATION RESPONSE SPECTRA - TAFT RECORD (N-S COMPONENT)

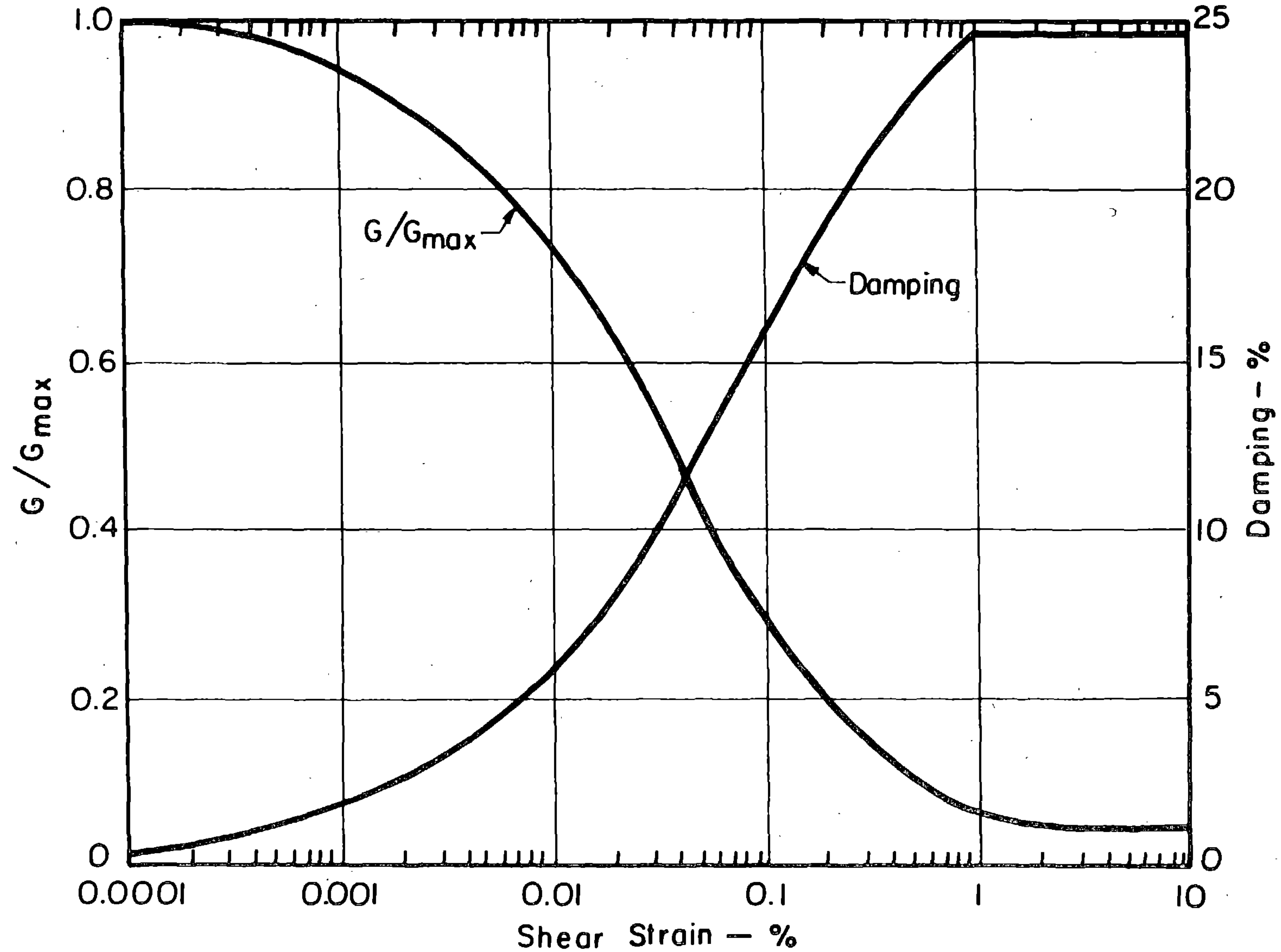

FIG. 4 SHEAR MODULUS AND DAMPING CHARACTERISTICS USED IN RESPONSE COMPUTATIONS

The values of spectral acceleration, for, ω_1, ω_2, ω_3 are obtained from Fig. 3 for λ = 13%, and from Eq. 10 we get:

$$\ddot{u}_{1max} = 1.6 \times 0.26 = 0.416g$$

$$\ddot{u}_{2max} = 1.06 \times 0.316 = 0.335g$$

$$\ddot{u}_{3max} = 0.86 \times 0.29 = 0.249g$$

from Eq. 11 the maximum crest acceleration

$$\ddot{u}_{max} = 0.59g$$

the average equivalent shear strain from Eq. 18 is then:

$$(\gamma_{ave})_{eq} = 0.65 \times 0.3 \times \frac{150}{(600)^2} \times 0.26 \times 32.2$$

$$= 0.068\%$$

Iteration #2

From Fig. 4: for shear strain = 0.068%

$$G/G_{max} = 0.36$$

$$\lambda = 13.7\%$$

and $v_s/v_{max} = 0.6$ $\quad\therefore v_s = 570$ fps

$$\therefore \omega_1 = 2.4 \times 570/150 = 9.12 \text{ rad/sec}, \qquad T_1 = 0.69 \text{ sec.}$$

$$\omega_2 = 5.52 \times 570/150 = 20.97 \text{ rad/sec}, \qquad T_2 = 0.3 \text{ sec.}$$

$$\omega_3 = 8.65 \times 570/150 = 32.87 \text{ rad/sec}, \qquad T_3 = 0.19 \text{ sec.}$$

and

$$\ddot{u}_{1max} = 1.6 \times 0.244 = 0.39g$$

$$\ddot{u}_{2max} = 1.06 \times 0.32 = 0.339g$$

$$\ddot{u}_{3max} = 0.86 \times 0.294 = 0.253g$$

$$\therefore \ddot{u}_{max} = 0.575g$$

$$(\gamma_{ave})_{eq} = 0.65 \times 0.3 \times \frac{150}{(570)^2} \times 0.244 \times 32.2$$

$$= 0.071\%$$

Repeating the same calculations for Iteration #3 we get the following results:

$$\left.\begin{array}{ll} \ddot{u}_{max} & = 0.57g \\ T_o & = 0.7 \text{ sec.} \\ \gamma_{ave} & = 0.07\% \\ G & = 1270 \text{ ksf} \\ \lambda & = 14\% \end{array}\right\} \text{Approximate Procedure}$$

The finite element solution gave the following results:

$$\left.\begin{array}{ll} \ddot{u}_{max} & = 0.51g \\ T_o & = 0.75g \\ \gamma_{ave} & = 0.065\% \\ (G)_{ave} & = 1410 \text{ ksf} \\ (\lambda)_{ave} & = 11\% \end{array}\right\} \text{Finite Element Solution}$$

As can be seen from the comparison above there is a fairly good agreement between the results obtained by the approximate procedure and those from the finite element calculation. Thus it would appear that for practical purposes the above procedure can be used for estimating the maximum crest acceleration and natural period of an embankment subjected to a given base motion. These values may in turn be used to estimate the permanent displacements induced by earthquake shaking as described elsewhere (Makdisi and Seed, 1977).

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