EARTHQUAKE-INDUCED CRACKING OF DRY CANYON DAM

by

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Introduction

The Dry Canyon Dam was constructed about 60 years ago primarily of fine to coarse silty sand using hydraulic filling techniques. In 1952 it was subjected to a magnitude 7.7 earthquake at an epicentral distance of 46 miles. Severe longitudinal cracking developed, indicative of a potential failure of the upstream slope. Subsequent drilling and sampling provided information on the soil conditions within the dam. Using the available field data, cyclic loading laboratory tests and an analysis to determine the seismic coefficient, a slope stability analysis was made using the method recently proposed by Seed (12).

The object of this method is to evaluate the deformations of an embankment resulting from any given earthquake, and for design purposes to limit the deformations to tolerable values. For this purpose the "strength" of the soil is defined as the stress required to cause a given amount of strain in cyclic load tests on representative samples. It is necessary to select the limiting strain in the laboratory tests on the basis of the corresponding deformations it will cause in the embankment.

Very few case history examples are available to provide a basis for selecting this limiting strain. From a study of the behavior of the Otterbrook Dam which deformed noticeably during construction, Seed suggested that "for design purposes the maximum working stresses should be kept below values producing approximately 13% strain in triaxial test specimens" (12).

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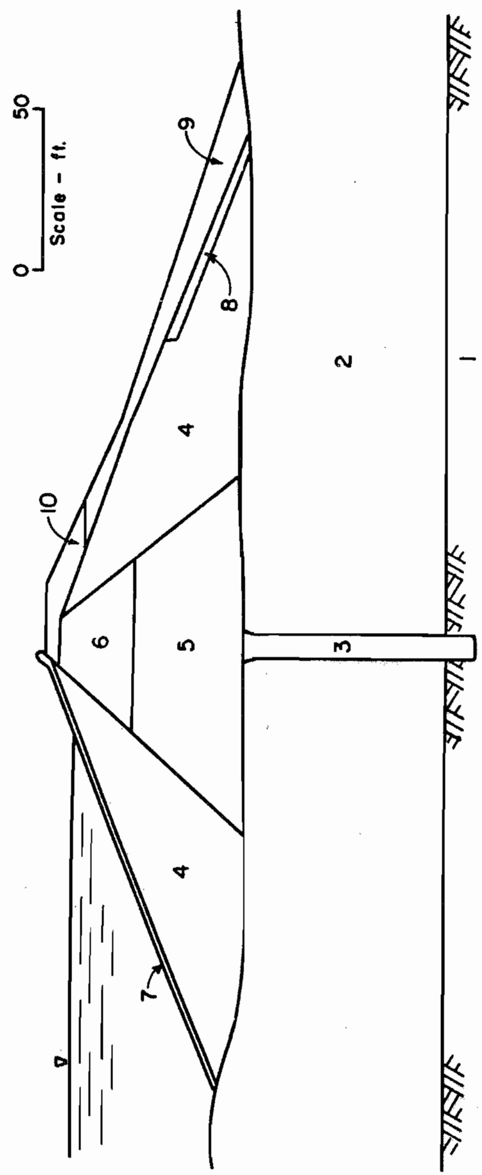
Ellis and Hartman used a limiting value of 5% strain in cyclic loading triaxial tests as a failure criterion in the seismic design of banks for the San Luis Canal where only limited slope movement could be tolerated without danger of damaging the concrete lining (3). The following study of the Dry Canyon Dam was conducted to evaluate the validity of the proposed method for determining the behavior of an earth dam during an earthquake and to provide additional information on the desirable strain criteria to be used in interpreting the results of laboratory test data for design purposes.

The results of the analysis indicated that the computed factor of safety increased substantially with increasing values of the assumed failure strain criterion which was used in interpreting the cyclic triaxial test data.

A criteria of $\epsilon_1 = 5\%$ gave cyclic loading strength data which when used in the seismic stability analysis led to a computed factor of safety of 1.0. Thus this study suggests that 5 percent axial strain is an appropriate lower value to define failure in cyclic loading triaxial tests for the purpose of seismic slope stability analysis.

Description of Dry Canyon Dam

The Dry Canyon Dam is located about 8 miles north of the city limits of Los Angeles, California, and is one of several such dams constructed early this century as part of the Los Angeles aqueduct system bringing water to the city from the Owens River some 250 miles to the north (2). The reservoir capacity is 750 acre feet, and the embankment is a maximum of about 63 ft high and 530 ft long at the crest. A sketch of the cross section of the dam is shown on Fig. 1. It is founded on about 60 ft of recent alluvium consisting mainly of silty sand and gravel, overlying a



Boundaries between zones are very approximate

1 Bedrock (Shale and sandstone)
2 Recent Alluvium (silty sand and gravel)
3 Puddled Clay cut-off wall
4 Shell (silty sand) (1911-1912)
5 Hydraulic Fill (silt-sand) (1911-1912)
6 Wagon Rolled Core (1912)
7 6" Thick Concrete Face (1912 and 1933)
8 Sand Filter (1933)
9 Uncompacted SS and Shale (1933)
10 Compacted SS and Shale (1933)

Fig. 1 MAXIMUM CROSS SECTION OF DRY CANYON DAM. 1952 thinly bedded sandstone and shale bedrock. It was constructed during the years 1911 and 1912 using standard methods and techniques of that era. Because of the permeable nature of the alluvial foundation, a 6 ft wide puddled clay cut-off wall was sunk to bedrock and extended longitudinally some 500 ft from rock abutment to rock abutment.

The upstream and downstream shells shown as Zone 4 on Fig. 1 were constructed of silty sand derived from soft shale excavated by steam shovels and hauled to the site by teams and wagons. Compaction was accomplished by the hauling operations.

The lower part of the core, shown as Zone 5 on Fig. 1, was constructed by hydraulic sluicing material from a bank 500 ft away within the reservoir area. When the core had reached 35 ft high a shortage of water developed and the remainder of the core, Zone 6, was constructed of selected material in the dry and compacted by wagon rolling as was done for the shell. Both upstream and downstream faces slope at 2.5 to 1. The dam was completed by placing a 6 inch thick concrete slab on the upstream face.

From the beginning, the dam was troubled by leakage problems, mostly through the abutments. In 1932-33 repairs were made which included raising the height by 3 ft and placing Zones 8, 9 and 10 as shown on Fig. 1. Between 1933 and 1952 the crest settled about 0.75 ft. As shown on Fig. 1 the water level in the reservoir was about 52 ft above the base and 11 ft below the crest at the time of the earthquake.

Subsequent to the earthquake other repairs and modifications have been made. In 1966 the reservoir was drained and an extensive drilling and sampling program carried out to study the feasibility of partial or complete reconstruction. The data and soil samples obtained from this program formed much of the basis for this study.

One of the findings from the 1966 field study was that the boundaries

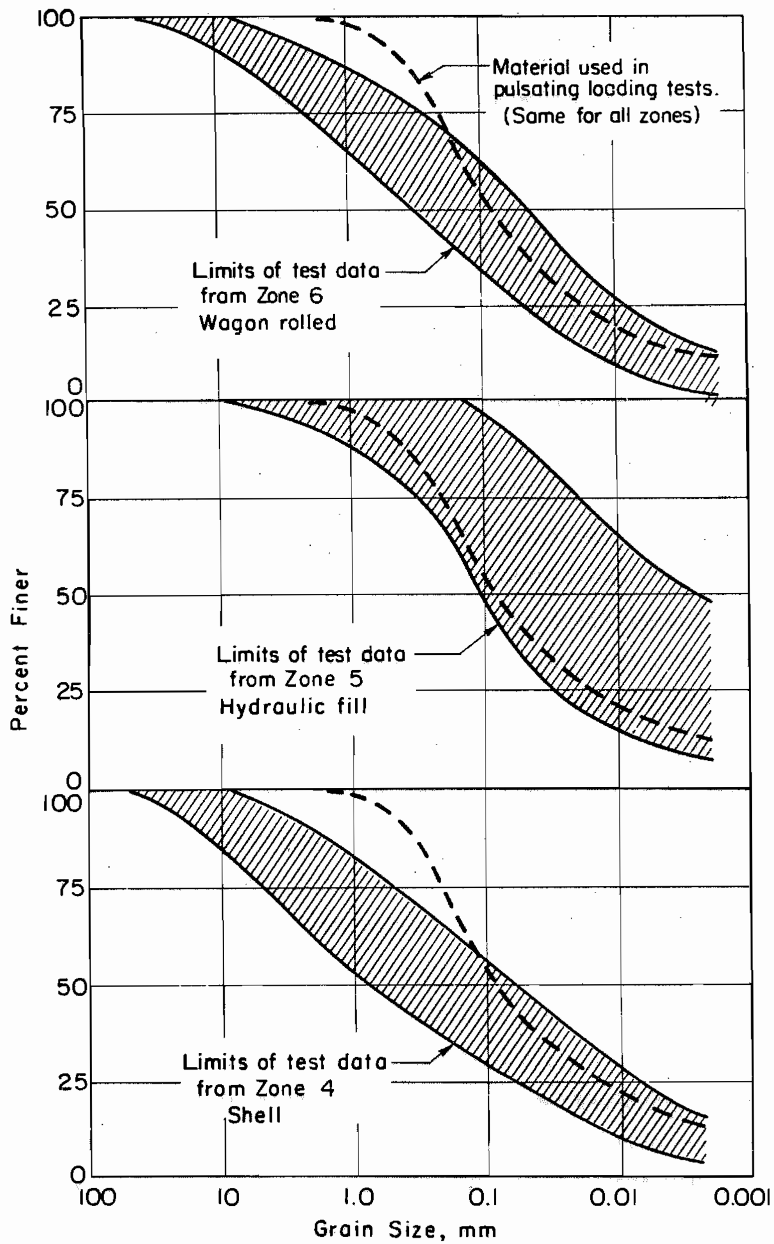
between the various zones were not nearly so well defined as indicated by the construction drawing reproduced on Fig. 1. In each of the zones the soil was very heterogeneous and stratified in thin layers of silt and sand ranging from a fraction of an inch to 2 or 3 inches thick. Because of this closely spaced variability it was difficult to detect well defined differences between samples from different zones.

The laboratory data from some 150 samples were grouped and averaged to indicate the general nature of the soils in the different zones. The ranges of grain size distribution curves for soil samples from Zones 4, 5 and 6 are shown on Fig. 2. The average moist and dry field densities of the undisturbed samples for the various zones are shown on Table 1 along with relative density values to be discussed later.

These data indicate that in spite of the wide variation from sample to sample, and from thin layer to thin layer within the embankment, there is nevertheless definable differences between the soils in the different zones. The low dry density and finer nature of the hydraulic fill as compared to the wagon rolled shell and core is particularly evident. Although unimportant to this study because of the small zone involved, it is nevertheless of interest to note the benefit obtained by the 1933 compaction operation as compared to the 1912 wagon rolling method. It is recalled that at this period of time (1930's) the theories of compaction were just being developed by R. R. Proctor (6) working for the Los Angeles Department of Water and Power, owners of this dam.

The Earthquake

Beginning in mid-1952 this area of Southern California was subject to a large earthquake and many strong aftershocks. The seismic, geological and engineering aspects of these earthquakes were thoroughly investigated at the time and numerous well documented accounts of various important technical



aspects are available (cf Ref. 5, 21). The main and largest shock occurred at 4:52 p.m. local time July 21, 1952, and was the prime cause of damage to the Dry Canyon Dam. It had a Richter Magnitude of 7.7 and the depth of focus was about 10 miles. The earthquake occurred on the NE-SW trending White Wolf fault and produced surface rupture for a total distance of about 33 miles with a maximum offset of about 2.5 ft horizontally and 4 ft vertically (1). The maximum assigned Modified Mercalli intensities in the epicenter region ranged between VIII and XI.

Two strong motion records were obtained of this shock. The strongest was recorded at Taft, some 25 miles NW of the epicenter at a station located on about 25 ft of alluvium overlying sedimentary rock (4). The other was recorded at the seismological station in Pasadena some 77 miles SE of the epicenter and located on granitic rock. The peak recorded accelerations at these two sites were 0.18 g and 0.055 g, respectively. The duration of the strongest shaking was about 30 seconds. The Dry Canyon Dam is located some 46 miles SE of the epicenter, and the average Modified Mercalli intensities in this region were about VII.

Seed et al (18, 19) have recently suggested that for engineering purposes the shortest distance to the causative fault may be a more significant parameter than the epicenter distance. For this case it so happens that a straight line drawn between Taft and Pasadena passes very close to the damsite, is normal to the White Wolf fault and intersects it almost at the epicenter. Thus, the epicenter distances are also the shortest distances to the causative fault.

On the basis of the above information, and allowing for some magnification at the damsite due to the extra thickness of foundation soil as compared to the recording sites, it was estimated that the maximum peak ground surface acceleration at the damsite was probably about 0.15 g.

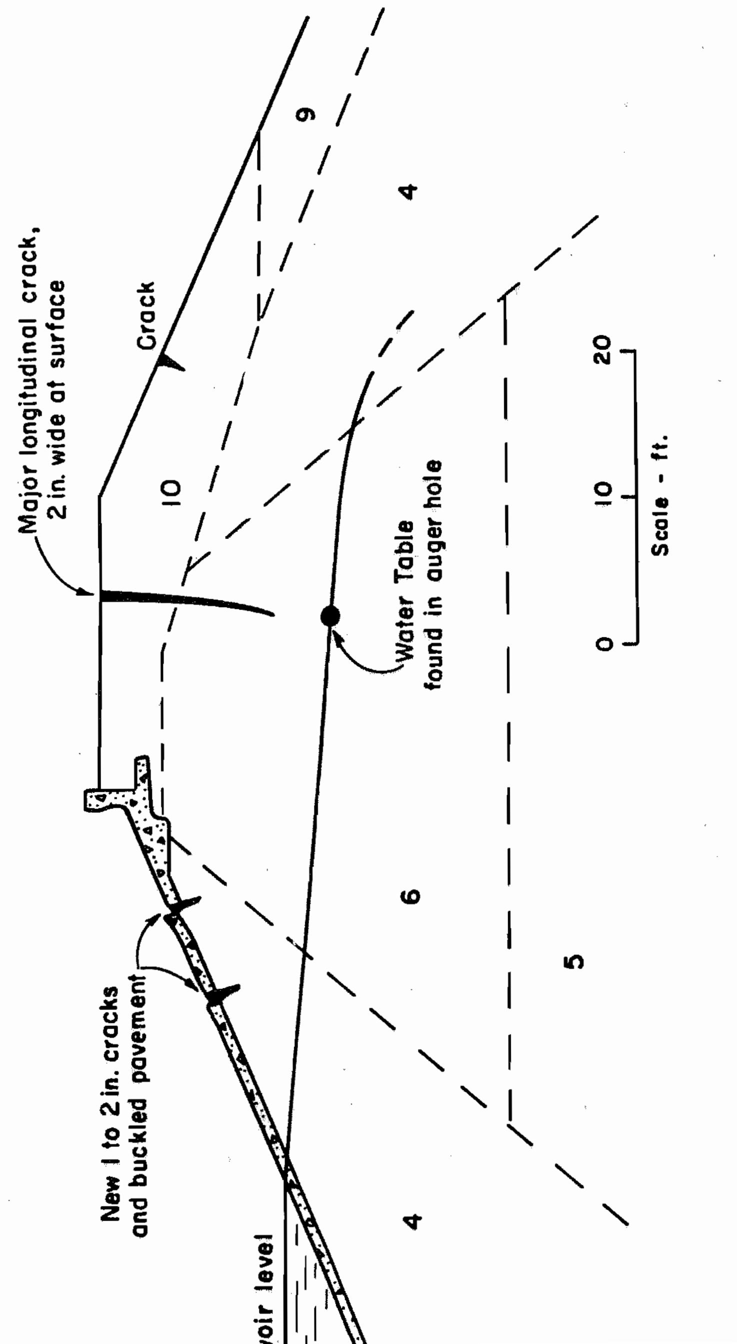
Earthquake Damage

Immediately following the earthquake the dam was inspected for possible damage. Quoting from one of the inspector's reports in the owner's files, "Horizontal Cracking had occured for almost the entire length of the dam, the main crack being about 18 ft downstream from the parapet wall. Small cracks appeared on the downstream face paralleling the major crack, about 10 ft down from the crest....old cracks in the upstream concrete facing became larger, and new ones appeared; also some buckling occurred." The main crack was about 2 inches wide at the surface, and the crest on the upstream side of the crack had settled over 1/2 inch with respect to the downstream side.

A 4x5x14 ft deep test pit was dug from the crest to determine the nature and extent of this crack. The crack width narrowed with depth, and could not be identified deeper than 13 ft below the surface. It sloped slightly upstream and at the bottom was offset 3 ft towards the reservoir. A hand auger hole was drilled at the bottom of the test pit and encountered the water table about 3 ft beyond the last trace of the crack. A sketch of the upper part of the dam is shown on Fig. 3 indicating the nature and location of the features just described. The position of the phreatic surface within the dam was estimated from the known reservoir elevation and the water table found in the auger hole.

It is of interest to point out in passing that another old hydraulic fill structure, the Haiwee Dam of similar vintage and construction, 90 ft high on a 120 ft deep alluvium foundation was located some 100 miles from the epicenter of the same earthquake and was subjected to an estimated maximum peak ground acceleration of about 0.05 g. Some longitudinal cracking also developed, but much less extensive than at the Dry Canyon Dam.

The nature of the observed cracking of Dry Canyon Dam suggested that the earthquake had induced an incipient deep arc failure of the upstream



ig. 3 UPPER ZONES OF DRY CANYON DAM SHOWING CRACKS INDUCED BY 1952 EARTHQUAKE.

slope, and although no large permanent movements did develop, it appeared that the maximum available strength of the dam material must have been almost completely mobilized during the earthquake. An analytical method has recently been proposed by Seed (12) to analyze the stability of earth dams during an earthquake. This case therefore appeared to offer a unique and valuable opportunity of checking these proposed procedures with the observed performance of this dam.

The stability analysis procedure for seismic conditions proposed by Seed differs from that used in static analyses in two major ways: 1) the driving forces include seismic forces which are computed from an analysis of the dynamic response of the dam to the input base acceleration-time history; and 2) the soil strengths are determined by cyclic, pulsating loading tests. Since the seismic analysis procedure has been presented in detail elsewhere (12) it is not appropriate to repeat the description here.

Soil Samples for Laboratory Tests

Undisturbed Shelby tube samples of the soil were obtained. The soil was thinly layered and very heterogeneous, and it was not possible to obtain a sufficient number of uniform undisturbed samples of any one soil type on which to perform the tests required to obtain the properties. Accordingly, a number of soil samples were mixed together and the tests performed on reconstituted specimens from this mixture. The grain size distribution curve of this reconstituted batch of test soil is shown on Fig. 2 where it can readily be compared with the range of sizes pertaining to the different important zones in the dam.

Several investigators have presented collections of laboratory test data which demonstrate indirectly that possible errors associated with the

use of reconstituted soil rather than undisturbed samples are likely to be negligible (9, 10, 14). Although the strength is somewhat dependent on grain size, between the range D_{50} = 0.01 to 0.5 mm the strength variation is less than 10 percent. Most of the soil in this dam, as well as the test specimens, had mean grain sizes within this range. Samples formed by sedimentation which results in considerable particle segregation have been found to yield approximately the same strength as samples of the same soil carefully prepared to avoid segregation. Accordingly, it was believed that the test data were reasonably representative of the properties of the soil in the embankment.

Density Determinations

The maximum density of this soil was assumed to be equivalent to the maximum density obtained from a Modified Proctor Compaction test. The minimum density was obtained by carefully sedimenting small batches of soil in a 1000 ml glass graduate filled with water. A small batch of soil sufficient to form a 3/4-in. layer was added once a day for about 10 days. The soil was allowed to consolidate under its own weight with careful precautions to avoid any vibration or disturbance. The field densities were obtained from measurements on the Shelby tube samples. The relative density of the various soil strata computed from these data are summarized in Table I. The relative density of the hydraulic fill ranged from about 36 to 60 percent with an average of 47 percent which is typical of values obtained for many other hydraulic fills of silty sands (22).

To simplify the testing, all strength tests were performed on samples prepared to an initial relative density of 50 percent. As shown by Lee and Seed (7, 8), the static and cyclic loading strength of sand increases in direct proportion with the relative density. On this basis the strengths

Table 1

Densities of Dry Canyon Dam Soils

Material	Dry Density - lb/ft^3		e (G_s = 2.75 est)	Dr %	Strength Multiplying Factor
	Range	Ave			
Zone 1 Bedrock		114			
Zone 2 Alluvium (Natural)		108	0.59	77	
Zone 4 Shell (1912)	88 - 107	98	0.75	62	1.24
Zone 5 Hydraulic Fill (1912)	83 - 96	89	0.93	47	0.94
Zone 6 Wagon Rolled (1912)	90 - 110	102	0.68	68	1.36
Zone 10 Compacted Fill (1933)	107 - 114	111	0.55	80	1.60
Minimum Density		69	1.49	0	
Maximum Density		131	0.31	100	
Laboratory Tests		90	0.90	50	1.00

measured at 50 percent relative density were later modified to obtain strengths at the other relative densities corresponding to the various zones of the dam. This was done by multiplying the measured strengths by ratio of the relative density in the field to the relative density of the laboratory tests. These multiplying factors for each zone are shown in Table I.

Static and Pulsating Loading Triaxial Tests

Triaxial tests were performed on 1.4-in. diameter saturated samples which were prepared by pouring and compacting the dry soil into a metal-forming mold lined with a membrane. After application of an initial confining pressure they were saturated by a combination of flushing and back pressuring. Conventional static loading, isotropically consolidated-undrained tests with pore pressure measurements gave the following strength parameters, corresponding to consolidation pressures within the range of 10 to 30 psi:

$$\text{Total stress basis: } C = 3 \text{ psi}, \ \phi = 13^\circ$$

$$\text{Effective stress basis: } C' = 0, \ \phi' = 33^\circ$$

Although the soil in the dam was consolidated under anisotropic stress conditions, static isotropic consolidated laboratory tests appear to give the same effective stress parameters as obtained from anisotropically consolidated samples (7). Therefore, these effective stress parameters were used in the static effective stress analysis corresponding to conditions prior to the earthquake. The static analysis was required to obtain the effective stress ratios $K_c = \sigma'_{1c}/\sigma'_{3c}$ to be used in the pulsating loading tests (12).

A total of 21 consolidated-undrained pulsating loading, triaxial tests were also performed and the data interpreted using procedures described elsewhere (8, 9, 10, 13, 14, 15). The pulsating loading tests were performed on samples consolidated to three anisotropic stress conditions corresponding to (K_c = 1.0, 1.5 and 2.0) and at two values of σ_{3c} (19 and 25 psi) to bracket the range of consolidation stresses in the dam.

The results of a typical test performed on an anisotropically consolidated (ACU) sample are shown on Fig. 4. Typical of the data from ACU tests, the sample fails by progressive deformation which accumulates with each cycle. There is no sudden collapse or liquefaction, as the pore pressure never reaches the liquefying state that would reduce the effective stress to zero (8, 15). This is in contrast to the behavior of isotropically consolidated samples of saturated sand which do fail by a complete collapse due to the high pore pressure which produces liquefaction (10, 14).

Using the method of data reduction described elsewhere (15), curves were prepared showing total accumulative axial strains vs number of cycles, and by a series of cross plots the data were reduced to a final form suitable for use in the analysis (12). In making this data reduction it was necessary to establish a criterion for failure. This was done in terms of an accumulative axial strain. The first analyses were made using strengths corresponding to a failure criterion of 20 percent axial strain. Subsequently, other analyses were made using lower strains as a criterion for failure.

In addition to the strength tests, some 16 samples were subjected to special cyclic loading tests for the purpose of determining the secant modulus and hysteretic damping corresponding to various axial strains. The data were interpreted according to the procedures described by Seed and Idriss (17), and the results were found to be very similar to that compiled by them for a wide variety of soils of this general type (17, 20). As the Seed-Idriss data covered a wider range of stresses and strain conditions

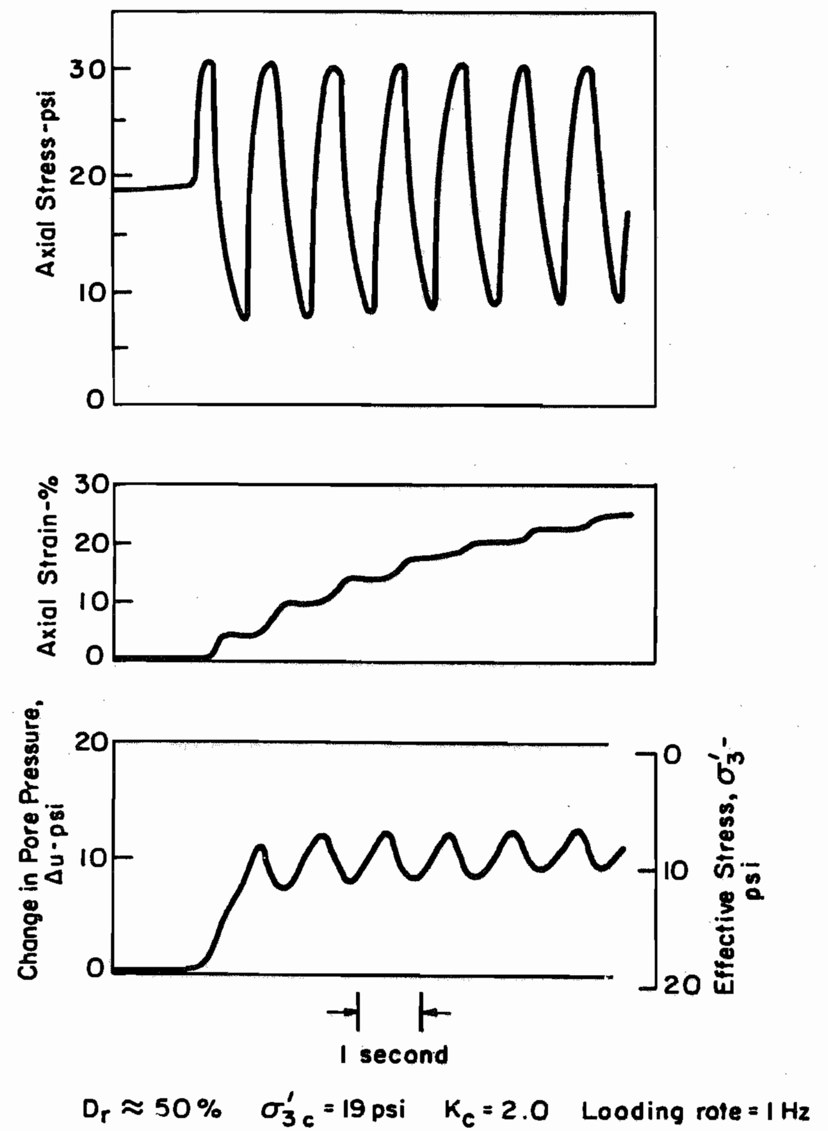


FIG. 4 TYPICAL PULSATING LOAD TESTING RECORD

than the tested performed for this study, and as the data coincided within the range tested, the Seed-Idriss modulus and damping relations were used throughout in the seismic response analysis.

Seismic Response Analysis

A seismic response analysis to determine an appropriate value of the dynamic seismic coefficient was performed for the writers by Dr. I. M. Idriss using the shear slice method described by Seed and Martin (16).

The input motion at the base of the dam was the strong motion accelerogram recorded at Taft with all of the accelerations multiplied by a constant factor such that the peak base acceleration was 0.15 g. The equivalent average seismic coefficients and number of significant cycles corresponding to an assumed failure wedge over different portions of the dam section are shown in Table II.

Seismic Slope Stability Analysis

The essential differences between the seismic stability analysis used (12) and pseudo static methods of analysis are that both the seismic coefficient and the soil strength are determined by dynamic methods. As shown in Table II, the dynamic response analysis indicated the equivalent number of significant cycles to be 10. Therefore, the strengths corresponding to 10 cycles were interpolated from cyclic loading data using the procedures described elsewhere (12, 15).

Using the seismic coefficients in Table II and the strengths from the pulsating loading tests, modified for the appropriate relative density condition, a number of trial failure circles were analyzed.

The first series of trials was made using circular arcs tangent to the relatively dense alluvial foundation. The most critical of these

TABLE II

Results of Seismic Response Analysis - Dry Canyon Dam

Assumed Sliding Wedge	Equivalent Average Seismic Coefficient	Number of Significant Cycles
Top 1/4	0.23	10
Top 1/2	0.18	10
Top 3/4	0.14	10
Full Height	0.10	10

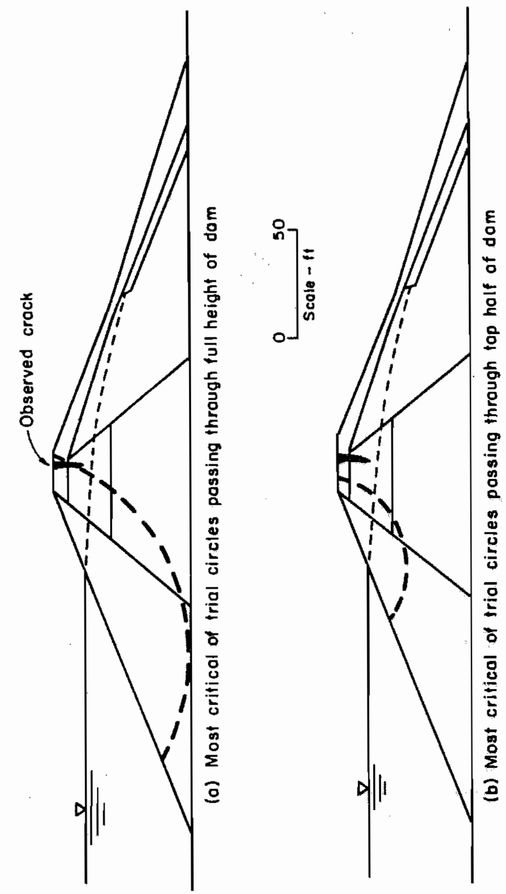
Input Base Acceleration: Taft Record Modified from a_{max} = 0.18 g to a_{max} = 0.15 g.

circles is shown on Fig. 5. It passes close to the observed crack and has a theoretical factor of safety of 1.35.

The next series of trials were made using circular arcs which extended only through the top half of the embankment. The most critical of these trials is also shown in Fig. 5. The critical circle is displaced by a small amount from the crack, and has a theoretical factor of safety of 1.27. Additional trials were made for more shallow circular arcs but it was found that much higher theoretical factors of safety were computed for any circle that did not penetrate part way into the weak hydraulic fill.

To put these results in perspective it must be emphasized that the strengths used for these analyses correspond to a failure criterion of 20 percent axial strain in a laboratory test. It seemed reasonable that the observed permanent field deformations of about 1/2 inch vertical and 2 inches horizontal across a crack would not be compatible with as much as 20 percent axial strain in a laboratory test. Therefore, using only the upper most critical circle, stability calculations were repeated using strength data corresponding to 10 and to 5 percent axial strain, respectively. The resulting minimum factors of safety for each of these conditions is shown in Fig. 6. Extrapolating from these data, a theoretical factor of safety F = 1.0 corresponds to a failure criterion of about 4-1/2 percent axial strain in a laboratory pulsating loading triaxial test.

The most critical theoretical circle missed the observed failure crack at the crest by about 7 or 8 ft. The stability analysis method is not limited to circular surfaces, and it may be that a critical non-circular surface could have been drawn which came closer to the crack. However, considering the approximations that were involved in selecting the input



Case	Radius	K_c	Seis. Coef.	Factor of Safety Before E.Q.	Factor of Safety During E.Q.
(a) Full ht	100 ft	2.00	0.10g	2.0	1.35
(b) Top half	40 ft	2.12	0.18g	2.5	1.27

Failure criterion for cyclic loading triaxial tests: $\epsilon = 20\%$ in 10 cycles in pulsating loading triaxial tests

Fig. 5 SUMMARY OF STABILITY ANALYSES.

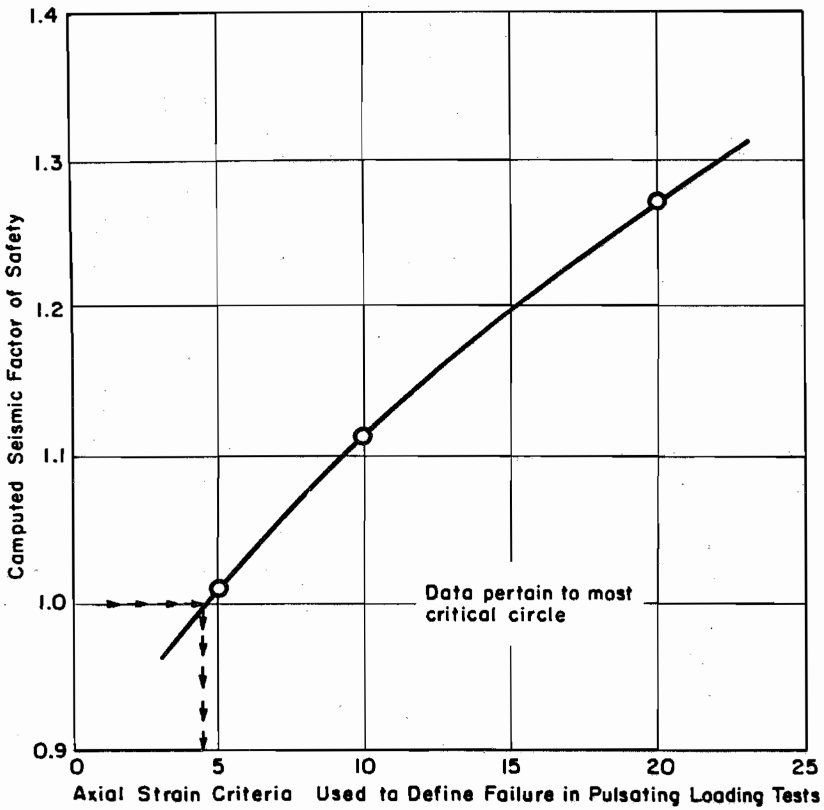


Fig. 6 SUMMARY OF SEISMIC FACTOR OF SAFETY CALCULATIONS.

data, a more refined analysis using non-circular surfaces did not seem to be justified. Considering the extreme variability and heterogeneity of the soil the imprecise boundaries between zones, and that the position of the critical circle is dependent on the relative strengths of the various soils through which it passes, no special significance can be attached to a disagreement of 7 or 8 ft between the theoretical and observed position of the failure surface.

Summary and Conclusions

The seismic stability analysis of the upstream slope of the Dry Canyon Dam when subjected to a magnitude 7.7 earthquake at 46 miles epicenter distance was performed to check on the ability of recently proposed methods and techniques to predict actual field behavior. These techniques include seismic response analyses to determine appropriate seismic coefficients, and soil strength determinations by pulsating loading tests. The method for obtaining soil strength requires a definition of failure in terms of accumulative axial strain. Lacking a rigorous correlation between laboratory strain and field deformations, soil engineers have developed empirical criteria to guide their designs based on experience and engineering judgement. Considering the nature of the observed field deformations compared to other cases for which correlations between laboratory and field deformations have been proposed, the computed limiting laboratory strain of 4 to 5 percent seems reasonable and the method of seismic stability analysis seems to be valid.

ACKNOWLEDGEMENTS

The City of Los Angeles, Department of Water and Power are owners of the dam. The firm Converse Davis and Associates of Pasadena were consultants to the owners in recent studies to repair and modify the dam. Each were very helpful in providing data and soil samples. The California Department of Water Resources also provided much useful information from their files.

The response analysis to obtain the seismic coefficients was performed by Dr. I. M. Idriss, Research Associate, University of California, Berkeley.

The study was supported in part by a grant from the National Science Foundation, No. GK20,578.

Grateful appreciation is expressed to each of these organizations and individuals for their support, without which this study would not have been possible.

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