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EARTHQUAKE RESISTANCE OF EARTH AND ROCK-FILL DAMS. REPORT I. DI--ETC(U)
MAY 71 R W CUNNY, J E AHLBERG

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MISCELLANEOUS PAPER S-71-17

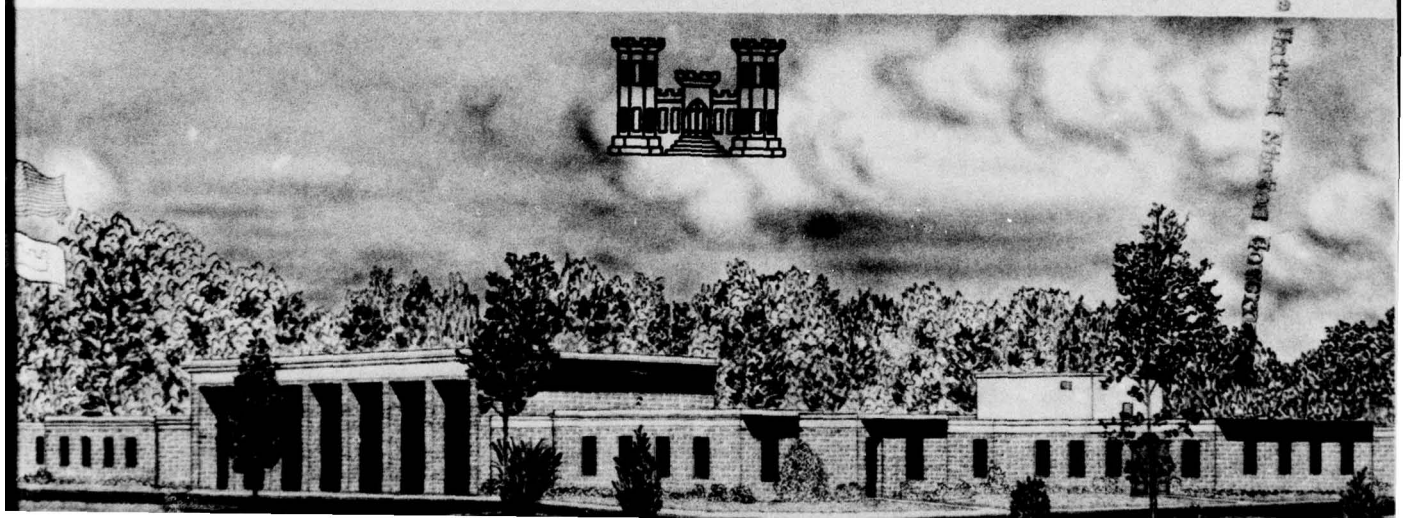
EARTHQUAKE RESISTANCE OF EARTH AND ROCK-FILL DAMS

Report 1

DISCUSSIONS BY PROFESSORS H. B. SEED AND R. V. WHITMAN

by

R. W. Cunny, J. E. Ahlberg



List of Associated Reports

Previous reports prepared under Engineering Study 540 are:

"A Comparative Summary of Current Earth Dam Analysis Methods for Earthquake Response," issued by Office, Chief of Engineers, as Inclosure 1 to Engineer Technical Letter No. 1110-2-77, 9 December 1969.

"Earthquake Studies for Earth and Rock-fill Dams," issued by Office, Chief of Engineers, as Engineer Technical Letter No. 1110-2-79, 12 January 1970.

"Motion of Rifle Gap Dam, Rifle, Colorado; Project Rulison Underground Nuclear Detonation," published by the Waterways Experiment Station as Miscellaneous Paper S-70-1, January 1970.

MISCELLANEOUS PAPER S-71-17

EARTHQUAKE RESISTANCE OF EARTH AND ROCK-FILL DAMS

Report 1

DISCUSSIONS BY PROFESSORS H. B. SEED AND R. V. WHITMAN

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R. W. Cunny, J. E. Ahlberg

May 1971

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted by U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

ARMY-MRC VICKSBURG, MISS.

FOREWORD

This report contains the records of the visits to the U. S. Army Engineer Waterways Experiment Station (WES) by Professors H. B. Seed, 15-16 September 1970, and R. V. Whitman, 26-27 October 1970, to discuss the earthquake resistance of earth and rock-fill dams. The Office, Chief of Engineers (OCE), authorized these visits under its Civil Works Program as a part of Engineering Study 540 entitled "Earthquake Resistance of Earth and Rockfill Dams."

Engineers of the Soils Division, WES, actively engaged in the discussions and report preparation were Messrs. S. J. Johnson, R. W. Cunny, L. W. Heller, LT J. E. Ahlberg, and SP5 W. C. Moss. The work was under the general supervision of Mr. James P. Sale, Chief, Soils Division. This report was prepared by Mr. Cunny and LT Ahlberg.

Director of WES during the visits and the preparation of this report was COL Ernest D. Peixotto, CE, and Technical Director was Mr. F. R. Brown.

CONTENTS

DEPARTMENT OF THE ARMY
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IN REPLY REFER TO: WESSD

11 February 1971

MEMORANDUM FOR RECORD

SUBJECT: Professor H. Bolton Seed Visit 15-16 September 1970, Earthquake Discussions

1. Professor H. B. Seed visited WES on 15-16 September to discuss the earthquake resistance of earth and rock-fill dams. A list of those who attended the discussions is given in Incl 1. Professor Seed was questioned regarding his views on earthquake design input, appropriate soil properties for earthquake analysis, and earthquake analysis procedures. The remarks that follow are the writers' interpretation of the comments made by Professor Seed.

Earthquake design input

2. Geologists are very important in selecting the magnitude of an earthquake for a particular site. They are very capable in locating faults and the development of fault history as well as presenting the regional geologic structure. Professor Clarence Allen of the California Institute of Technology and Dr. Lloyd Clough of Woodward-Clyde & Associates are particularly proficient in evaluating the potential effect of faults at a particular site. Geological records, as opposed to seismological records, have an advantage in determining the potential activity of an area because the geological records have a longer history than the latter.

3. For determining the appropriate rock motion at a site, Professor Seed recommends determining the maximum magnitude of an earthquake on a fault or faults likely to be critical for the site, a depth of focus, and a distance of the site from the fault. The details of this procedure are described in Professor Seed's paper, "Characteristics of Rock Motions During Earthquakes," ASCE Journal, Soil Mechanics and Foundations Division, September 1969, Professor Seed's paper, "The Response of Earth Dams During Earthquakes," included in the Proceedings of the Seismic Instrumentation Conference held in San Francisco in November 1969, and the University of California Earthquake Engineering Research Center Report on "Rock Motion Accelograms for High Magnitude Earthquakes," April 1969.

4. The rock motion at the bottom of the alluvial valley may be 5 to 15 percent less than the rock motion of the outcrops at the valley walls

and depends upon the amount of overburden in the valley. Professor Seed has used a value of 10 percent for certain analyses he has made.

5. Bedrock motion is easier to predict than ground motion at the surface of a soil or alluvial deposit. Ground motion can be very different for two locations that are near each other because of differences in soil properties. The maximum velocity of motion is limited by the shear strength of the material.

6. Ground motion amplification can be determined assuming either (a) deformable rock properties, or (b) rigid base rock. The method using the deformable rock properties was developed by Kanai, is one-dimensional, and permits energy to be radiated into the rock foundation. The rigid base rock method is a closed-energy system but can be used for either a one-dimensional or a two-dimensional analysis; the magnitude of the calculated ground motion is affected by the damping characteristics of the system.

7. A report on the effect of soil conditions on damage caused by the Caracas earthquake can be found in a University of California publication, Earthquake Engineering Research Center Report No. 69-2, entitled "Relations Between Soil Conditions and Building Damage in the Caracas Earthquake on July 29, 1967."

8. The following engineers have worked with strong motion earthquake records and are considered among the best qualified for developing design ground motion inputs for specific locations: Housner, Newmark, Blume, Ambraseys, Seed, Kanai, Rosenblueth, Esteva, and Whitman.

9. Three methods for obtaining an earthquake time history for analysis are: (a) use some previous record, (b) use some previous record which is modified by changing one or both of the intensity and time scales, depending upon the existing conditions at the site, and (c) generate an artificial earthquake using some technique such as filtered white noise. Professor Seed prefers using the method which best suits the particular site in question. If a previous record is available from that exact location, then that record may be adequate. If no previous record is available, then some type of record modified for site conditions might be used. Artificial earthquakes which have been described by Jennings, Housner, and Tsai in their Engineering Earthquake Laboratory Report, "Simulated Earthquake Motions," California Institute of Technology, April 1968, can be used as firm ground input but should not be interpreted as rock motion. Unless carefully constructed by appropriate filtering, an artificial earthquake motion is considered least desirable.

10. The three most significant characteristics of earthquake motion are: (a) maximum acceleration, (b) the predominant period or periods, and (c) duration. Duration is especially important in problems involving soil stability; each cycle of stress can cause increases in pore pressure which may cause large permanent deformations.

11. For major earthquakes, one can move the earthquake along the fault for its full length of break (for magnitude 8 earthquakes, the fault break would be about 200 miles). This movement can produce very different time histories depending upon where the fault break begins with reference to the site.

12. Items which Professor Seed believes need more study in the area of earthquake design input are: (a) effect of motions with different predominant periods on the seismic response of a structure, (b) the effect of different time histories of motion having the same general characteristics, and (c) the effect of the two horizontal acceleration time histories which are similar but have very different response spectra.

Soil properties for earthquake analysis

13. The two most important properties necessary for finite element analysis are damping and dynamic shear modulus. Although Poisson's ratio is also required, accuracy of this property is not critical. Both damping and shear modulus vary as a function of shear strain and should be selected on the basis of estimated shear strain for analysis purposes.

14. Dynamic shear modulus can best be estimated from field in situ tests. For some soils such as saturated natural clays, disturbance can have a great effect on modulus and if this property is measured in the laboratory, it should be adjusted to more accurately represent the in situ conditions. A field test used by Weston Geophysical for determining shear modulus involves propagation of a shear wave from one borehole to another. Shear wave velocities can also be determined by surface vibrators and second arrivals from refraction seismic tests.

15. Damping cannot be accurately evaluated from field tests, so lab tests must be run to determine this property. Different tests are needed to determine the relationship of damping over a wide range of shear strain.

16. Cyclic load triaxial tests are used to evaluate the response of the soil to the repeated earthquake loading. For sands, either undisturbed samples or samples remolded to appropriate densities can be used. The confining pressure and initial axial stress applied to the specimens should cover a range of stress conditions dictated by the

calculated stress conditions in the embankment. The cyclic stress should be equivalent to that produced by the earthquake loading.

17. Susceptibility to liquefaction for cohesionless material can be determined with the cyclic triaxial test by determining the natural density of the deposit and running a cyclic test on a specimen prepared at that density. A small change in density near maximum makes a large difference in the number of cycles to failure during the cyclic load test. Relative density is rather difficult to determine because different procedures are used in different laboratories and this results in different maximum and minimum densities. A one lb/cu ft change in dry density may cause a 5 to 10 percent difference in relative density for some soils.

18. One difficulty with cyclic triaxial tests is the 90 degree rotation of the principal stresses during the application of the deviator axial stress when initial stress ratio is either one or near one. When the confining pressure exceeds the axial stress, extension and necking develop in the specimens and nonrepresentative conditions are produced during the test. This situation becomes aggravated as pore pressures are developed and the effective confining pressure exceeds the effective axial stress by increased amounts and increasing strains in the specimen develop rapidly.

19. Dr. Casagrande has pointed out that pore pressure concentrations build up in the specimen around the cap in the cyclic load triaxial test; this results in a conservative value of strength (failure or large deformations at fewer cycles of load). Professor Seed also points out, however, that using an incorrect value for confining pressure ($\sigma_1 = \sigma_3$) at the beginning of the test leads to an unconservative value of strength (an increase in number of cycles for failure). Taking both of these considerations into account, the cyclic shear stress which will cause failure on a given number of cycles for the cyclic triaxial specimen is greater than that for actual field conditions; for this reason Professor Seed reduces the cyclic shear stress causing failure by a factor of 0.6 when the cyclic tests are run with $\sigma_1 = \sigma_3$. The basis for this correction is described in a report by Seed and Idriss entitled "Applicability of Laboratory Test Procedures for Measuring Soil Liquefaction Characteristics Under Cyclic Loading."

20. The cyclic loading simple shear test appears to provide a better means for determining liquefaction characteristics. This apparatus more closely simulates field conditions in that it utilizes the correct initial stress conditions and the rotation of the principal plane is more like that in the field; however, there are difficulties with

assembling the apparatus and running the tests, and great care must be taken in working with the apparatus.

Earthquake analysis procedures

21. There are various methods for earthquake analysis available to determine the seismic response of earth dams. Professor Seed prefers to use the approach of (a) determining the stresses in the embankment due to an earthquake loading, (b) subjecting a laboratory specimen to an equivalent number of appropriate cyclic stresses, (c) evaluating the strain observed from the laboratory specimens, and relating this to the response of the embankment. One example of such an analysis is given in Incl 2.

22. Another method of analysis is to construct a circular arc through the dam and compute seismic coefficients at various times during the excitation and from this compute a minimum factor of safety. See Incl 3. Professor Seed has sometimes used this method to make simple analyses of small structures.

23. Guidelines or arbitrary criteria that can be used for the dynamic analysis are: (a) if the shear wave velocity is less than 3000 ft/sec, the material may be considered soil; if it is more than 3000 ft/sec, it may be considered rock; (b) a magnitude 7 earthquake has approximately ten equivalent cycles of loading, (c) a magnitude 8 earthquake has approximately 30 equivalent cycles of loading; (d) soil in a dam subjected to 5 percent strain could be considered stable; (e) soil in a dam subjected to a 20 percent strain is no doubt unsatisfactory; (f) soil in a dam subjected to 7 to 10 percent strain is probably okay, but this would depend upon the particular circumstances.

24. Professor Seed believes that Professor Newmark's method for determining the deformation of slopes from seismic excitation can be used with confidence for cohesionless materials where pore pressures do not develop.

25. Professor Seed reviewed the proposed earthquake analysis for Warm Springs Dam. An acceptable analysis is shown in Incl 4.

Miscellaneous comments

26. The reservoir should have little effect on the seismic response of an earth dam because of the relatively flat embankment slopes; however, the reservoir loading is important in determining initial effective stress conditions.

27. Judgment must be used in evaluating the drainage conditions in a rock-filldam. In the case of Oroville Dam, part of the embankment was considered drained whereas, further in, the material was considered undrained. Sometimes it might be desirable to make calculations with and without drainage in areas where pore pressures could develop to determine the significance of drainage before drawing a conclusion as to its significance.

28. For earthquake resistant design, a freeboard of 15 ft has been used; this should not, however, be used in lieu of making a good dynamic analysis. The freeboard height is dependent upon potential reservoir landslides, height of dam, effect of overtopping on stability of embankment, and consequences of overtopping on facilities downstream.

29. Other provisions that should be made for earthquake resistance are: filter zones should be as thick as possible, core should be as thick as possible, riprap should be used to protect against erosion, and embankment material should be compacted to a high density.

30. Mr. Tom Leps is doing studies on the effect of overtopping on the flow of water through rock fill. Bob Weigel at the University of California is doing work on model tests of sliding reservoir slopes and the resulting wave action.

31. Professor Seed agreed to furnish the following which have since been obtained by mail:

a. A paper by Clarence Allen given at an international AEC conference in Tokyo. This paper discusses the importance of using geologic data in determining the seismicity of an area.

b. The report of Professor Seed's findings from the Caracas earthquake study; also, the Weston Geophysical Report containing the shear wave velocity profile of that area.

c. A copy of a paper given by Housner at a Geological Conference on Reservoir Induced Earthquakes held at Berkeley in May 1969.

d. A card deck of the computer program used to calculate free-field motion from bedrock motion input by a lumped mass analysis.

e. A card deck of the digitized record of a magnitude eight earthquake moving along a fault and any report available in which this approach is discussed.

WESSD 11 February 1971

SUBJECT: Professor H. Bolton Seed Visit 15-16 September 1970, Earthquake Discussions

Professor Seed also agreed to furnish a working card deck for his equivalent linear finite element computer program which was obtained during LT Ahlberg's visit to Berkeley during the first week of December.

4 Incl
as

R. W. CUNNY
Engineer
Chief, Soil Dynamics Branch

CF w/incl:
Mr. S. J. Johnson
Mr. J. R. Compton
Dr. C. R. Kolb
Mr. W. C. Sherman

J. E. AHLBERG, 1LT
Engineer
Analytical Section

LIST OF ATTENDEES

H. B. SEED DISCUSSIONS

15-16 September 1970

University of California

Professor H. B. Seed

WES

Mr. S. J. Johnson

Mr. R. G. Ahlvin*

Mr. R. W. Cunny*

Mr. J. R. Compton*

Mr. W. C. Sherman*

Dr. C. R. Kolb*

Mr. L. W. Heller

Mr. W. E. Strohm*

Dr. E. L. Krinitzsky

Mr. F. K. Chang

Mr. G. R. Skoglund

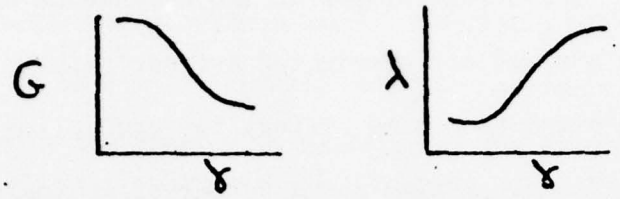
LT J. E. Ahlberg

PFC W. C. Moss

*Part-time attendance

SEISMIC ANALYSIS I

1. Estimate the strain in the dam.
2. Determine shear modulus and damping from the relationships:



3. Determine the response of the embankment using a computer program such as the equivalent linear finite element method, and compare the computed strains with those assumed in step 1.
4. Using a new estimate for strain, determine new values of shear modulus and damping and repeat calculations until the strains computed are near the estimated strains.
5. Test laboratory samples under the stress conditions encountered in the embankment.
6. Evaluate response of embankment from deformation of laboratory specimens subjected to the simulated seismic loadings.

SEISMIC ANALYSIS II

1. The embankment response analysis is about the same as that given in Seismic Analysis I. The accelerations in the embankment are determined.
2. A failure circle is assumed and a weighted average seismic coefficient k is computed and plotted as a time history for each slice or appropriate group of slices. Estimate the number of equivalent cycles of a weighted average k that is compatible with the computed k-time history.
3. Find the stresses on the base of the slices of the failure circle and subject the laboratory specimens to these same stresses at the same number of equivalent cycles determined in paragraph 2. Obtain strain of specimens.
4. The strains at various points along the embankment are assumed equal to specimen strains. If a maximum failure strain criterion is assumed, then a factor of safety of the slope can be computed as a ratio of failure criterion strain divided by average computed strain.

Professor Seed's Comments on
Dynamic Analysis for Warm Springs Dam

If a dynamic earthquake analysis is to be made, it should include a complete treatment as follows:

1. Determine design earthquakes by consulting with Clarence Allen (Cal Tech) or some other equally competent engineering seismologist.
2. Determine the shear wave velocity of the foundation and embankment for dynamic analysis. Conduct seismic field tests on the dam foundation materials and on the two test embankments.
3. Determine static and dynamic stresses induced in the foundation and the embankment. Obtain equivalent linear program from University of California (finite element method). Construct mesh, assign modulus and damping values, apply design earthquakes and compute stresses and strains; repeat analysis using improved modulus and damping values. Full reservoir condition.
4. Perform cyclic loading triaxial tests on embankment materials. Sample density same as fill. Use consolidation ratios, $\frac{\sigma_1}{\sigma_3}$, of 1.0 and 2.0, three different confining pressures and three different deviator stresses, the largest being adequate to cause at least 15 percent strain during the number of significant cycles of the largest deviator stress induced by the earthquakes. These tests would require 18 valid sample tests, which might require about twice this number of individual tests.
5. Interpret the effect of the computed stress history on the embankment materials. Assume that the strain induced at various points in the dam by the design earthquake is the same as the strain on a cyclic loaded sample subjected to similar stresses. Assess dam safety in terms of strain and prepare report of findings.

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2 February 1971

MEMORANDUM FOR RECORD

SUBJECT: Discussions with Professor R. V. Whitman 26-27 October 1970

1. Professor Whitman presented two lectures on 26 October 1970. The notes of the first lecture, "Amplification," are shown in Incl 1. The "Choosing of a Design Earthquake" was the second lecture and the notes are shown in Incl 2. During the subsequent discussions, Professor Whitman answered a prepared set of questions. These questions along with briefs of his answers are given in Incl 3. Additional remarks that Professor Whitman made regarding earthquake studies are recorded below.

Soil properties for
earthquake analysis

2. The three methods used by Whitman to determine the shear modulus of soil in decreasing desirability are:

a. In situ, crosshole techniques or surface vibratory methods.

b. Laboratory.

c. Hardin's empirical formulas.

These methods are for modulus values at low strains. An initial value to be used in a dynamic analysis at an assumed strain (1×10^{-4} to 5×10^{-4}) is determined from the relationship

3. Equivalent linear theory does not rigorously account for material properties. Ramberg and Osgood* have developed equations that simulate soil behavior quite well. Professor Whitman has used the following equation to evaluate a damping value which also accounts for the radiated waves.

$$D_n = D_i + \frac{(\gamma c)_s}{(\gamma c)_r} \frac{2}{\pi} \frac{1}{2n - 1}$$

n = mode number

D_n = total damping for mode n

D_i = internal soil damping

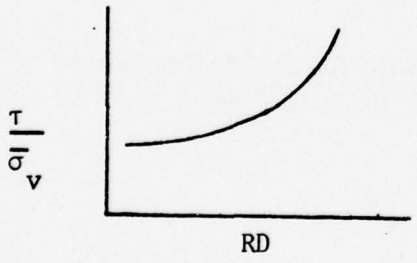
γ = density of the material

c = shear wave velocity of the material

s = represents overlying layer

r = represents underlying layer

4. Whitman has recognized that a susceptible material becomes liquefied when the properties of that material plot above the line on the following type of graph:



Assumption: Horizontal ground surface

τ = dynamic shear stress on a horizontal plane

$\bar{\sigma}_v$ = vertical effective stress

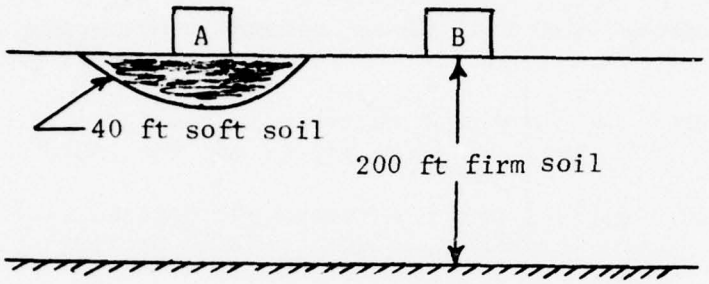
RD = relative density of the material

* "A Description of Stress Strain Curves by Three Parameters," National Advisory Committee for Aeronautics, Technical Note 902, Washington, D. C., July 1943.

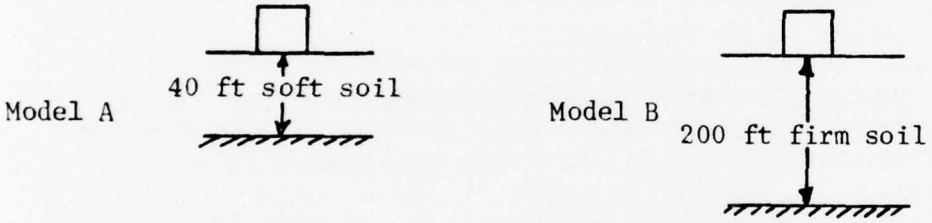
WESSD 2 February 1971
SUBJECT: Discussions with Professor R. V. Whitman 26-27 October 1970

Earthquake analysis procedures

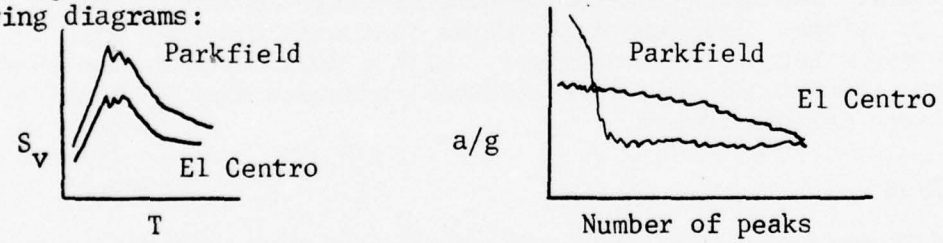
5. Amplification theory is very sensitive to surface conditions and may not be accurate for shallow surface deposits. This is because the shallow layers have a period far from the fundamental period of the entire deposit. As an example:



The best mathematical model for structures A and B would be:



6. Any relationships between magnitude or intensity versus acceleration are averages and do not take into account the effect of duration. The following diagrams:



show that the Parkfield earthquake had a larger maximum acceleration and a higher maximum response spectrum than the El Centro earthquake, but the El Centro was more damaging due to its large duration of substantial peaks. Whitman suggests that duration included with maximum acceleration and response spectrum would be a more true indicator of earthquake motion.

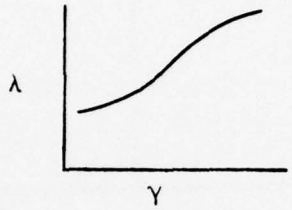
7. Whitman's procedure for design of a building is to assume a maximum acceleration and velocity and construct a maximum velocity response spectrum for this motion. Artificial earthquakes would be generated to correspond with this spectrum. (Note that an adjustment of amplitude and not frequency is all that is necessary to change the spectrum.) This

is an appropriate method for buildings because its modes have very different frequencies.

8. Whitman's additional steps for earth dam design are:

a. Look at the input motion to estimate the shear strain in the dam.

b. Find a damping value from the relationship of damping versus shear strain.



This is internal damping from laboratory values and one has to estimate radiation damping.

c. Estimate modulus as in paragraph 2.

d. Using an appropriate time history for input, find response of structure and iterate damping and modulus as a function of computed shear strains. An average strain value of two-thirds peak strain is a good value for iteration.

9. The accuracy of amplification theory decreases as the soil deposit becomes deeper. The first reason for this is that the input becomes more difficult to define. The second reason is that with a deeper soil deposit the higher modes become more important. With a deep deposit, one should use a finite element analysis that includes radiation damping, such as that developed by John Lysmer.

Miscellaneous

10. The following items are those that Professor Whitman believes the Corps of Engineers should review more closely:

a. Shaking table tests done at the University of Mexico by George Prince on rock-fill dams and the investigation of the breaking of particles under loading.

b. Experiences in other countries (i.e., Japan, Portugal, Chile, Mexico).

WESSD 2 February 1971
SUBJECT: Discussions with Professor R. V. Whitman 26-27 October 1970

11. The following items were furnished by Professor Whitman to the WES:

a. "An Investigation into the Nature of Microtremors Through Experimental Studies of Seismic Waves," by Ahmed Allam.

b. A computer program "Dynamic Fourier Analysis of Layered Systems" which uses a one-dimensional Fourier transform analysis to compute the response of linear, visco-elastic, non-uniform soil deposits, subjected to a base excitation.

c. MIT Civil Engineering Research Report R70-14, "Damping in Soils: Its Hysteretic Nature and the Linear Approximation" by R. Doby.

d. MIT Civil Engineering Research Report R69-15, "Theoretical Background for Amplification Studies," by J. M. Roesset and R. V. Whitman.

e. MIT Civil Engineering Research Report R70-37, "Fundamental Period and Amplification of Peak Acceleration in Layered Systems," by G. A. Madera.

f. MIT Civil Engineering Research Report R68-17, "Earthquake Simulation Models and Their Application," by S. Hou.

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Dr. C. R. Kolb
Mr. W. C. Sherman

Lecture on Amplification
by
R. V. Whitman

26 October 1970

1. The amplification of earthquake motions from bedrock to the soil surface has been noted. Figure 1 shows the acceleration response spectra for two sites relatively close together with respect to their distance from the epicenter. One site was underlain by only stiff soil whereas the second site was underlain by a layer of soft soil. The peak accelerations were quite different as well as the shapes of their respective response spectra.

2. A reliable theory is needed to explain and predict amplification. However, not much data is now available for validation of amplification theory. Complicated building codes are being introduced and theory is needed for their substantiation. Localized damage in cities (e.g., Caracas) where it was not expected has promoted a closer look into amplification theory.

3. A comparison of two amplification theories, wave propagation solution (Kanai) and lumped shear beam (Seed), is given in fig. 2. These are one-dimensional analyses that consider linear viscoelastic material which, although it does not depict actual soil behavior, simulates the behavior satisfactorily. A necessary assumption for both theories is that horizontal wave fronts propagate vertically to the free surface.

4. The lumped shear beam analysis, with the aid of a high speed digital computer, can be carried out by using mode superposition techniques or the more time consuming step-by-step procedures. The soil properties of shear modulus and damping, as a function of shear strain, are necessary for the computations. A rigid base is assumed at the rock surface which does not account for radiation damping. The mode superposition technique uses one value of damping for the system. This means that an average value must be determined from the various soil layers and each mode. Most of the computer time is used to compute the eigenvalues, and time histories at any level can be produced with little additional effort.

5. The wave propagation theory allows energy to be radiated from the soil profile through the bedrock. The effect of using radiation is shown in fig. 3. The dashed curve represents the velocity spectrum obtained from a lumped shear beam analysis. The solid line represents the values obtained from the wave propagation analysis. The difference has been interpreted as due to energy being trapped in the system. The soil in this example was 100 ft thick overlying bedrock and both methods used the same damping value. In the wave propagation analysis, a Fourier spectrum has to be generated for each soil layer and this method is not suited to represent a large number of soil layers.

6. The present shortcomings of the wave propagation amplification theory are shown in fig. 4. Of special note is the lack of confidence in selecting the input earthquake to be used with the theory.

7. Presently, work is being done at MIT with the wave propagation theory. An amplification spectrum can be produced from the Fourier spectra produced from the rock and soil acceleration time histories (fig. 5). This amplification response spectrum, fig. 6, shows the natural frequency of the deposit and the amount of damping in the soil layer. This damping can be computed using three techniques:

a. Amplitude of peaks.

b. Band width of spikes.

c. Q-theory (area under amplification curves).

Figure 7 shows a comparison of damping computed from data in Mexico City. Note that damping calculated from the Q-theory seems to agree best with the laboratory value.

8. Whitman's concluding remarks are given in fig. 8.

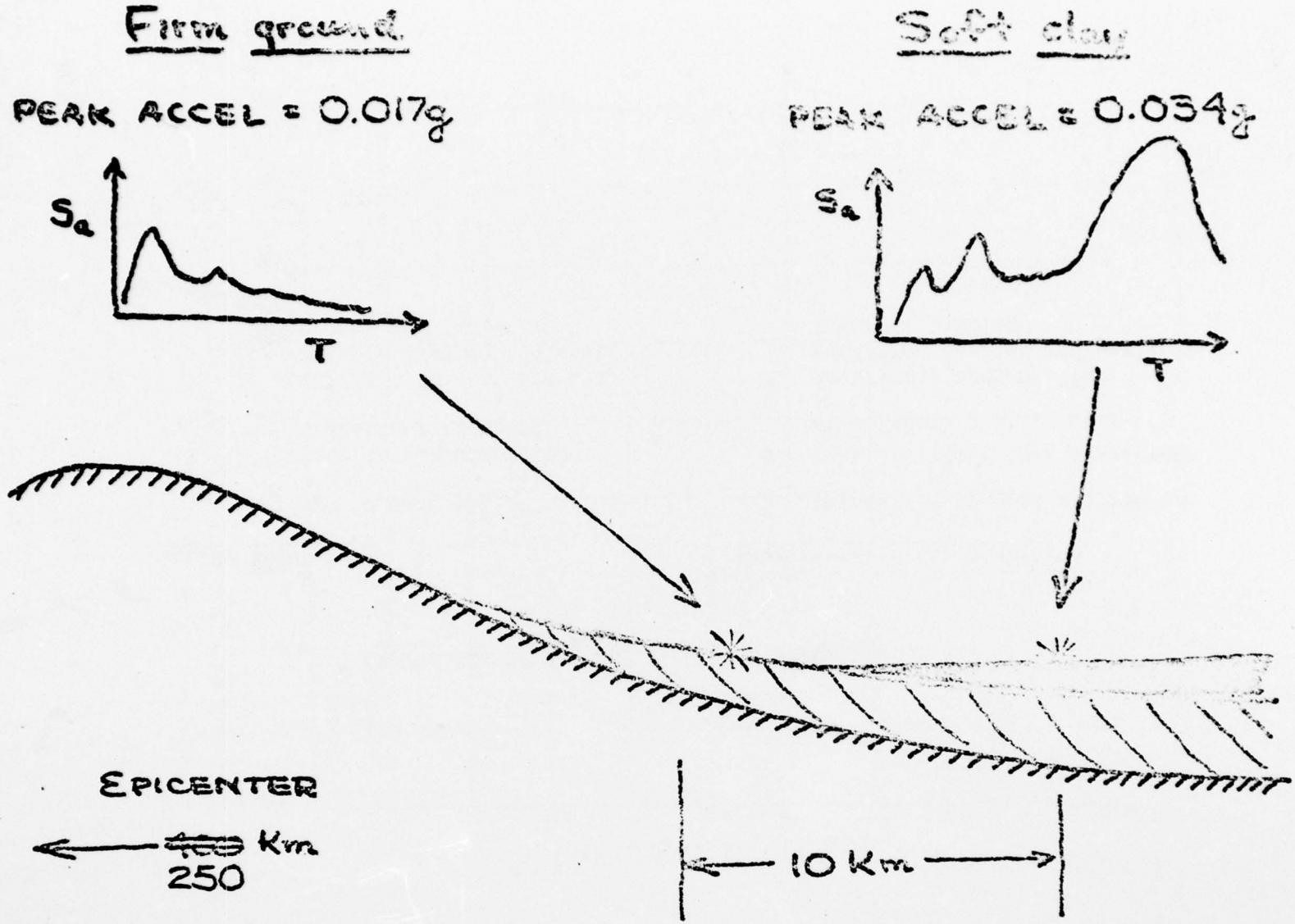


Fig. 1

Incl 1
Sheet 3

COMPARISON OF 1-D AMPLIFICATION THEORIES

Lumped shear beam

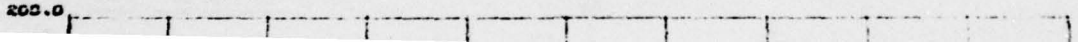
1. Familiar to many engineers
2. Efficient in computer time when iterations required
3. In usual form, cannot account for variable damping and does not account for radiation damping

Wave propagation solution

1. Unfamiliar to most engineers
2. Less efficient when iteration required
3. Accurately accounts for radiation damping and variable internal damping

Fig. 2

EARTHQUAKE AT FREE SURFACE 1



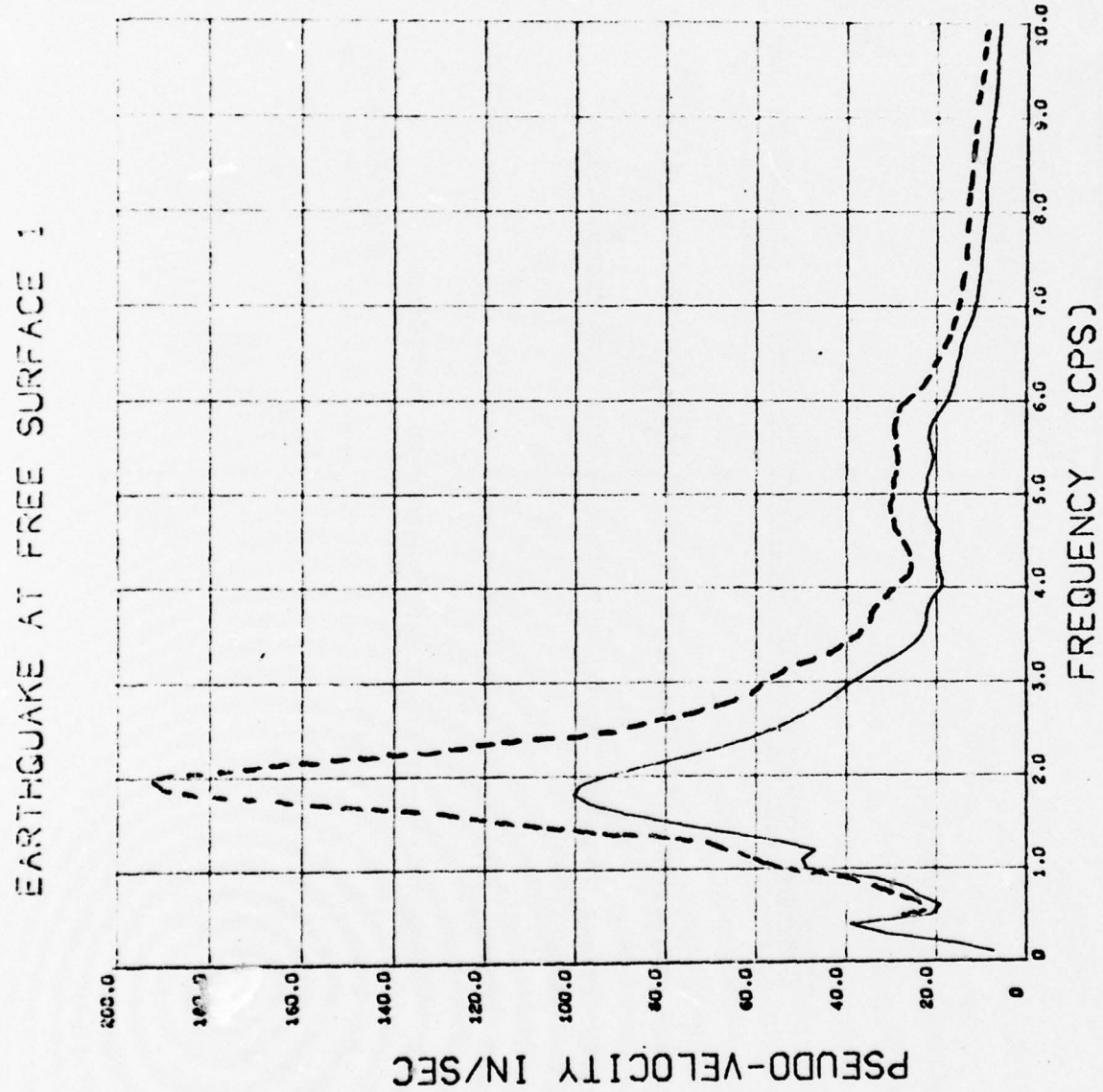


Fig. 3

Incl 1
Sheet 5

PRESENT SHORTCOMINGS OF AMPLIFICATION THEORY

1. Limitations of 1-D assumption still poorly understood

2. Effect of using "equivalent linear" solution incompletely understood

3. Cannot estimate damping accurately

4. Unable to select input earthquake with confidence

Fig. 4

TOP OF SOIL

Fig. 5

Incl 1
Sheet 7

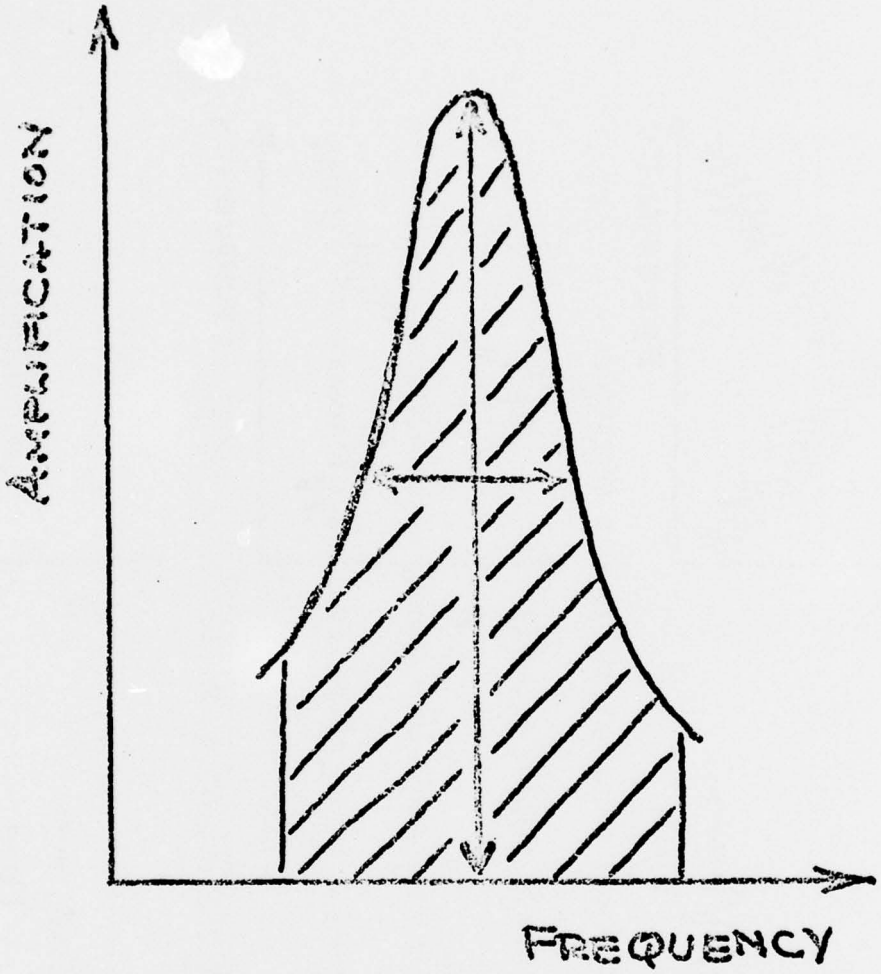


Fig. 6

Incl 1
Sheet 8

DAMPING DEDUCED FROM OBSERVED AMPLIFICATION SPECTRA - MEXICO CITY

Cycles of smoothing	Damping ratio - %		
	Amplitude	Bandwidth	Q-theory
0	2.2 to 4.1	3.5 to 9.4	4.8 to 8.8
200	3.1 to 5.7	9.1 to 16.9	7.4 to 8.0

DAMPING RATIO MEASURED IN LABORATORY = 5.5%

Fig. 7
Incl 1
Sheet 9

CONCLUDING REMARKS

1. There is considerable advantage, as well as some disadvantage, in using the "continuous" amplification theory.

2. Theory has been validated by a number of case studies.

3. There are still shortcomings to use of theory, especially inability to select input earthquake.

4. Work at MIT currently is to develop method for evaluation of field parameters.

Fig. 8

Lecture on the
Choosing of a Design Earthquake
by
R. V. Whitman

26 October 1970

1. The choosing of a design earthquake is a complex problem and it should be done in a combined effort by a panel. This panel should be comprised of members from the following disciplines:

a. Seismology.

b. Geology.

c. Structural Engineering.

d. Soil Mechanics.

2. One needs to use a rational approach in choosing the design earthquake. Throughout the world there are differences in seismicity and a single design earthquake cannot be used. The overdesign of nuclear power plants leads to substantial monetary penalties. The increased construction cost for an earthquake increased from 0.1 to 0.2 g acceleration is one-half to one and one-half million dollars. The engineering design costs alone are one-quarter million dollars for an earthquake analysis.

3. An example of the fast rate that thinking has changed concerning maximum design earthquakes is shown in fig. 1. The Parkfield earthquake was larger than the maximum probable estimated only two years earlier. Since then even larger earthquakes have occurred.

4. Figure 2 shows three questions which arise while choosing a design earthquake. Two levels of risk, the operational basis earthquake (OBE) and the design basis earthquake (DBE), are presently being used and are explained in fig. 3. Some designers require a time history while others need a response spectrum (fig. 4). Difficulties arise in using either input. The use of the response spectra restricts the analysis to mode superposition techniques. A time history input may not include adequate representation of frequencies most critical for structural response. Some firms use more than one time history which when combined gives a smoother response spectrum. This eliminates the peaks and valleys of the response spectra curves. The advantage of using artificial time histories versus an actual time history is that they have a smoother response spectrum. The third question which arises is where should the earthquake be placed with regard to the profile. In the profile of fig. 5, three possible locations exist for input. Location 1, in the bedrock, would give the best simulation of soil behavior as well as soil-structure interaction.

5. Magnitude is a measure of the size of an earthquake. The more common Richter magnitude is determined by estimation of the motion of a standard seismometer, 100 kilometers from the epicenter. During a large earthquake, energy is released along the fault break. Because of this the distance from the site to the fault is more important than the distance from the site to the epicenter (fig. 6). Intensity is the qualitative measurement of an earthquake at a particular location. The modified Mercalli intensity was originated before strong motion instruments were developed and is based on people's reactions and damage caused by the seismic disturbance. Figure 7 shows a relationship between intensity and peak accelerations. The short dashed line was proposed by Guttenberg and Richter and the long dashed line is the more recent prediction of Hershberger. The solid vertical lines represent data of some 30 earthquakes for which measurements of both intensity and peak accelerations were known. This shows that there is no good relationship between the quantities presented and care should be taken when trying to predict quantitative maximum accelerations from qualitative intensities. A more useful intensity description would include three additional criteria:

a. Maximum acceleration.

b. Duration.

c. Nature of building damaged.

6. The AEC presently follows a general procedure to produce a design earthquake:

a. Find intensity from historical records.

b. Relate maximum acceleration to intensity.

c. Find time history or response spectrum.

In the areas where active faults are present (California), faults near the site are located and a maximum historical earthquake is moved along the fault to the point nearest the site. Empirical charts are then used to determine the decrease of intensity with distance to the site. In less active areas (Eastern United States), a seismo-tectonic approach is used. The maximum intensity is determined for the region of like geology or tectonics. This earthquake intensity is then considered to occur under the site and is additionally increased one unit to take into account the possibility of an even larger earthquake occurring at the site. From this a maximum acceleration is determined. A response spectrum, such as Newmark's, is then developed showing the relation of maximum acceleration, velocity, and displacement to frequency. It should be noted that, when using Newmark's chart, high frequencies (2-5 cps) correspond to maximum acceleration, middle frequencies (1/4 - 2 cps) correspond to maximum velocity, and low frequencies (less than 1/4 cps) correspond to maximum displacement.

7. A new approach suggested by Whitman is based upon the occurrence of earthquakes in areas for which a long period of record is available and a plot of intensity versus return period is found to have a constant slope. By using an equation as in fig. 8 and assuming a probability of failure and the probability that the earthquake will cause failure of parts and machines one can calculate a return period which would then give a design intensity.

8. A design example given by Professor Whitman involved a nuclear power plant to be built in a valley of deep sediments 4 to 5 kilometers thick (fig. 9). The known faults are shown in fig. 10 and can be grouped into three systems as in fig. 11. The problem was to find the design earthquake and three independent techniques were used. Figure 12 gives the values obtained from using the seismo-tectonic approach for the earthquakes in the deep valley sediments. The maximum historical intensity was increased by one magnitude and this maximum credible intensity gave a corresponding 0.2 g maximum acceleration. A second approach was to consider the effect of nearby earthquakes and distant earthquakes (fig. 13). In the first case, it was assumed that earthquakes beneath the deep valley sediments which have occurred near the site could occur at the site. The effects of nearby earthquakes is shown in fig. 14 and based upon these data and judgment it was concluded that a magnitude 6 earthquake at the site would produce a maximum acceleration 0.22 g and a maximum velocity 7 in./sec. These are less than the Parkfield values but no rupture has occurred at the site in question and no intensity that large is expected. The effect of the magnitude 7 earthquake located 40 km away was then considered which resulted in a maximum acceleration of 0.10 g and a maximum velocity of 11.0 in./sec as in fig. 15. Esteva's equations were used to predict the effect of the earthquake at 40 km distance and these were modified for local site conditions. Return periods were used as the third approach; 2000 years of historical records were available. Data were plotted for the last 100 years of data and also for 2000 years of record (fig. 16). The thought was that the record for the last 100 years was the most accurate and an intensity of 7 or 8 (10,000 year return period) was chosen for design. The three approaches are summarized in fig. 17 and give consistent results. Figure 18 shows the recommended envelope for response spectrum obtained from the combination of the nearby and distant fault system. The maximum acceleration of .2 g and maximum velocity of 20 in./sec were used as design values.

9. The concluding remarks are shown in fig. 19. In this country, due to our abundant resources, we find ourselves overdesigning. Other countries, with limited resources, are designing on a more rational basis and perhaps this line of thought should be taken up in the United States.

	Maximum acceleration g	Maximum velocity in/sec	Maximum displacement in
Max. recorded in California (as of 1965)	0.32	14	12
Max. probable in California (estimate in 1965)	0.50	24 to 30	24
Parkfield earthquake (1967)	0.51	30	15

Fig. 1

QUESTIONS RE CHOICE OF DESIGN EARTHQUAKE

1. Level of risk which can be accepted

2. Time history vs. response spectra

3. Elevation within profile at which spectra is to be applied

Fig. 2

Incl 2
Sheet 5

LEVEL OF RISK	Ordinary Buildings	Nuclear Plants
Maximum probable earthquake; operational basis earthquake	Should not experience damage requiring expensive repairs	Should not be thrown out of operation
Maximum credible earthquake; design basis earthquake	Must not collapse	Must not cause accident; must shut down safely

Fig. 3

Incl 2
Sheet 6

Fig. 4

Incl 2
Sheet 7

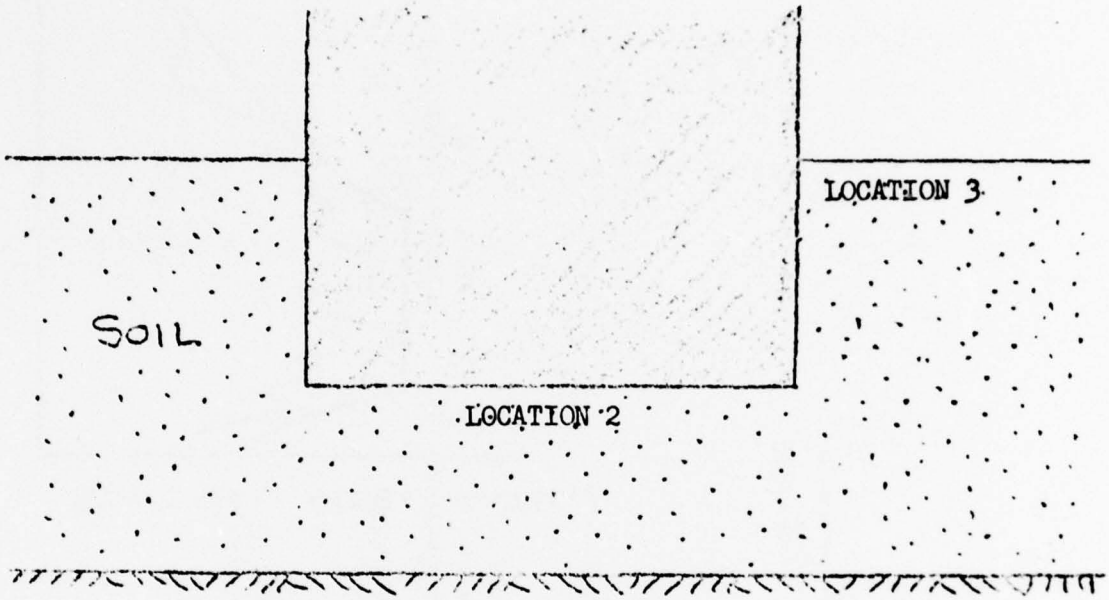


Fig. 5

Incl 2
Sheet 8

Fig. 6

Incl 2
Sheet 9

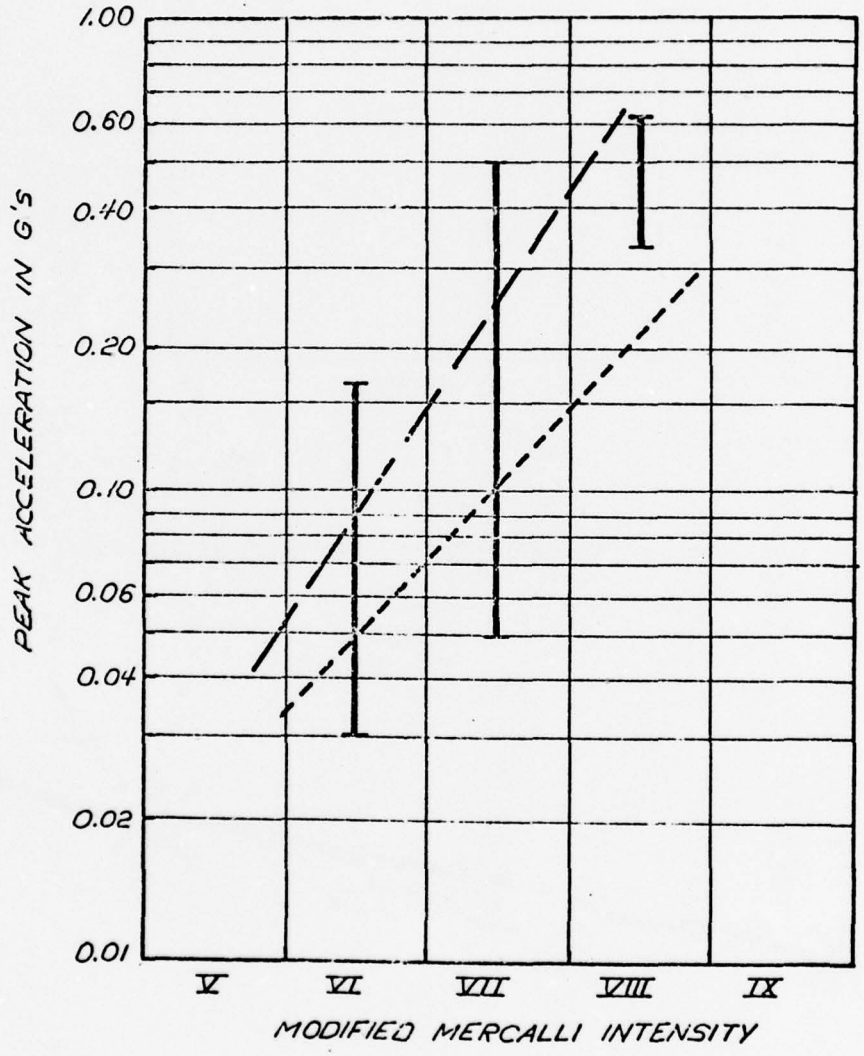


Fig. 7

Incl 2
Sheet 10

Probability of failure caused by earthquake	=	Probability that earthquake will occur	X	Probability that earthquake will cause failure

Fig. 8

Incl 2
Sheet 11

Incl 2
Sheet 12

Fig. 9

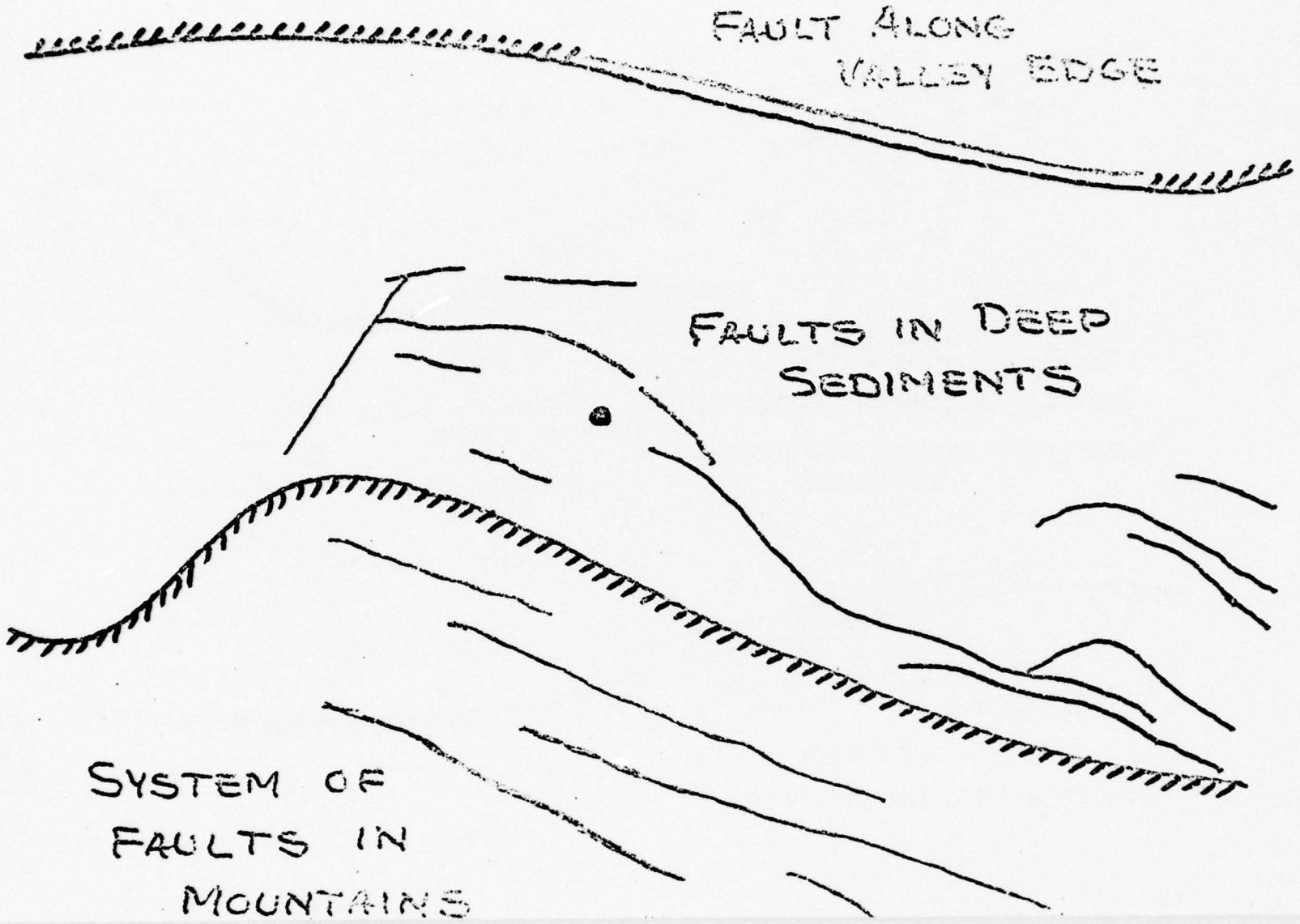


Fig. 10
Incl 2
Sheet 13

Incl 2
Sheet 14

Fig. 11

SEISMO-TECTONIC APPROACH

Maximum historical intensity at site	VI to VII
Maximum credible intensity	VII to VIII
Corresponding acceleration	0.2 g

Fig. 12

Incl 2
Sheet 16

Fig. 13

EFFECT OF SMALL NEARBY EARTHQUAKE

Earthquake	Dist. from fault Km	Maximum acceleration g	Maximum velocity in/sec
Parkfield (M = 5.5)	5	0.41	11
	6	0.47	8
	9	0.28	5
El Centro (adjusted to M = 6)	6	0.19	8
Assumed		0.22	7

Fig. 14

Incl 2
Sheet 17

EFFECT OF MODERATE EARTHQUAKE AT DISTANCE OF 40 KM

	Maximum acceleration g	Maximum velocity in/sec
From Esteva equations	0.08	5.5
Modified for local conditions	0.10	11.0

Fig. 15

Fig. 16

Incl 2
Sheet 19

SUMMARY OF PREDICTIONS

Seismo-tectonic province	VII to VIII + 0.2g
Magnitude and distance	0.22g
Return period	VII to VIII + 0.2g

Fig. 17

Incl 2
Sheet 20

Fig. 18

Incl 2
Sheet 21

CONCLUDING REMARKS

1. It is essential that "design earthquake" be chosen realistically.

2. Intuitive procedures for making this choice are widely used by AEC.

3. Rational analytical tools are available.

4. The main difficulty lies in being realistic about the acceptable risk.

Fig. 19

Questions and Answers
From
R. V. Whitman Discussions
26-27 October 1970

Design Earthquake

1. For analysis, is an artificial or an actual earthquake the best?

   One artificial record can simulate many actual records. Allen Cornell, MIT, is capable of easily producing the artificial records.

2. How many earthquake inputs should be used?

   Three or four, as a minimum.

3. If more than one input earthquake is to be used, should these be varied according to duration and/or amplitude?

   Amplitude, if one is trying to produce a smooth response spectrum.

4. How does one judge when enough earthquakes have been used to analyze a structure?

   When one has a smooth response spectrum.

5. What is the validity of scaling the El Centro earthquake for designs in the Midwest? Isn't this a common practice?

   This is common practice although it has no validity.

6. How does one develop an adequate time history from a response spectrum?

   Allen Cornell, MIT, is very proficient at this.

7. With respect to the many artificial earthquakes proposed, has anyone made an engineering analysis of the input evidence (data and assumptions) to evaluate the validity of various artificial earthquakes?

   Not to Whitman's knowledge.

8. What is your opinion of using 1D lumped-mass analysis for obtaining soil layer response?

   This is a good tool for iterative purposes but does not take into account radiation damping.

9. Should earthquake input be placed in the bedrock or at the base of a structure?

   If the depth of the soil in the foundation is more than twice the width of the structure, then the bedrock input should be used.

Design Analysis

10. What is the validity of using Ambraseys' $\bar{k}$ values from the envelope of numerous earthquakes?

    This approach does not take into account particular site conditions and foundation effects. A shear wedge analysis may be sufficient when using this approach in lieu of the finite element method.

11. What is the validity of using k factors in a dynamic analysis?

    This is not a desirable approach.

12. How does Newmark's method for determining deformations compare with methods proposed by other people?

    Whitman has used this method but has made no particular comparisons.

13. What are the "keys" to the most critical times during an earthquake?

    a. % plastic?

    b. g max?

    c. v max?

    d. d max?

    e. stress amplitude?

    f. strain amplitude?

    No comment.

14. What determines a reasonable "cutoff" time for a dynamic analysis?

    When no further change occurs in the response spectrum.

15. What amount of permanent deformation or strain is excessive for a plane strain finite element program?

    Not known.

16. For dams, what is the most important condition for analysis (e.g., after construction, steady seepage, or rapid drawdown)? Can one tell before running the analysis?

    This depends upon the particular investigation.

17. What is the effect of the reservoir on the stability of the dam? Do you know of any work being done in this area?

    Not known.

    No.

18. What interpretational procedures can be used to relate seismic response observed from small earthquakes to anticipate response under larger earthquakes?

25.

No comment.

19. What are some recommendations regarding defensive design?

a. Freeboard requirement?

26.

b. Thickness of filter zones?

c. Thickness of core?

d. Special provisions for spillway and outlet works?

27.

e. Riprap?

The usual earthquake design provisions should be made and also non-erodible materials in zones where cracking is anticipated and on the downstream face should be used.

28.

20. What are your comments on evaluation of landslide stability and feasible methods for estimating effects of potential wave action?

No comment.

29.

21. What, do you feel, are the most critical structures for a dam?

No comment.

22. How much design effort is reasonable for the earthquake problem as compared to a static analysis?

30.

The first few times a dynamic analysis is made it may be very costly, but with experience, this cost should decrease.

23. What information do you have on reservoir induced earthquakes?

31.

None.

24. Would you be concerned if a major portion of a dam went plastic under earthquake loading?

No, but one guideline for failure is if the vertical deformation exceeds 0.1 freeboard.

32.

Liquefaction

25. What is the minimum earthquake acceleration that can cause liquefaction of sands?

    Shaking table tests have shown that loose natural sands can liquefy with .07 g acceleration.

26. What relative density is required to withstand liquefaction?

    Somewhere around 70 percent relative density, the material should be stable.

27. How would you assess relative density of natural sand deposits?

    Penetration resistance values are the best available at the present time. However, there is a need for a better technique to assess relative density.

28. Would sands under a slope (subject to high shear stresses) be less likely to liquefy than sands under level ground surface?

    This answer may be yes, but more work needs to be done for substantiation.

29. Is there a decrease in susceptibility of liquefaction with depth as lateral pressures increase?

    Yes, because the ratio of shear stress to effective overburden pressure decreases with depth.

30. What types of laboratory tests are best suited to evaluate liquefaction susceptibility of sands?

    Shaking table tests with large specimens (2 to 3 ft) that are instrumented for pore pressure measurements.

31. What influence does permeability have on progress of liquefaction?

    This has a large effect; with fine sand as compared with gravel one gets higher pore pressures during cycling and, therefore, higher susceptibility to liquefaction.

32. What types of field tests are best suited to evaluate liquefaction susceptibility of sands?

    No comment.

33. How adequate is the Corps method for determining a soil deposit's liquefaction susceptibility?

No comment.

Laboratory and Field Testing

34. How does one interpret cyclic load tests and apply them in design?

No comment.

35. How does one extract modulus and damping from cyclic tests (if possible)?

No comment.

36. Are full-scale field tests desirable and necessary for determining dynamic material properties or are laboratory tests enough?

These are desirable but damping cannot be measured in the field.

37. For field testing, how deep should one test in a homogeneous dam? (Or can one use a portable vibrator and save on the shipping costs of a large vibrator?)

Perhaps one could use a small vibrator but the effect of depth would have to be taken into account.

38. Is the value of damping for a soil layer different than that for a soil structure?

Yes, from the viewpoint of radiation damping.

39. What is the current practice and application for laboratory tests with respect to earthquake analysis?

Repeated load tests are very useful.

40. What are the criteria for estimating pore pressures in pervious shells during seismic excitation?

No comment.

Warm Springs Dam

41. What earthquake input should WES use for Warm Springs Dam?

    An historical record, if possible, moved along the appropriate fault to the point nearest the site.

42. What are your comments to the proposed analysis method(s) for Warm Springs Dam?

    a. Determine susceptibility of foundation materials to pore pressure buildup.

    b. Estimate amplification in structures and determine the acceleration levels that would be present.

    c. Use some technique to determine the factor of safety along a failure plane. If the FS < 1, use Newmark's equations to estimate deformation.

    d. If a nonlinear analysis is used, Ambraseys' or Newmark's methods are not needed.

    e. Use a linear method to compare with nonlinear methods.

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13. ABSTRACT

Professor H. B. Seed of the University of California and Professor R. V. Whitman of the Massachusetts Institute of Technology visited the U. S. Army Engineer Waterways Experiment Station on 15-16 September 1970 and 26-27 October 1970, respectively, to discuss the earthquake resistance of earth and rock-fill dams. This report contains the records of these visits.

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