EARTHQUAKE ENGINEERING RESEARCH CENTER

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H. Bolton Seed

Faiz I. Makdisi

Pedro De Alba

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NOTICE

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H. Bolton Seed[1], Faiz I. Makdisi[2], and Pedro De Alba[3]

Introduction

The study of the behavior of dams and embankments under conditions of earthquake loading has been the subject of much study during the past decade. As a result major advances have been achieved in understanding the nature of soils under cyclic loading and in the development of analytical tools for the study of the dynamic response of embankments to earthquake loading. This progress has led to the development of dynamic analysis procedures for evaluating the stability of embankments during earthquakes.

It was not, however, until the near catastrophic failure of the Lower San Fernando Dam during the 1971 earthquake that the attention of regulatory agencies and the design profession was seriously directed towards re-evaluating existing conventional design tools and attempting to adopt the more recently developed dynamic analysis procedures for evaluating seismic stability. Since then, many dams have been studied using dynamic analysis techniques.

It should be emphasized, however, that in any rational approach to the design of embankments against earthquakes, in addition to the proper use and understanding of the material properties and behavior during seismic loading, and in addition to the proper use of analytical procedures to

[1]Professor of Civil Engineering, University of California, Berkeley, Calif.
[2]Project Engineer, Woodward-Clyde Consultants, San Francisco, Calif.
[3]Assistant Professor of Civil Engineering, University of New Hampshire.

estimate the dynamic response, considerable insight and judgment are required; this can only be gained through an intimate knowledge of the strengths and limitations of the analysis and testing procedures themselves and by studying the behavior of similar embankments during actual earthquake loading conditions. Accordingly the purpose of the present paper is to summarize available information concerning the behavior of dams subjected to strong earthquake shaking. A similar survey was presented by Ambraseys (1960); the present paper is intended to complement this initial review by presenting further details concerning embankment construction materials and procedures and performance records in earthquakes during the past 17 years.

A review of information on embankment dam performance shows that most of the available data has resulted from studies of earth dam performance in 6 major earthquakes:

1. San Francisco earthquake of 1906.
2. The Ojika (Japan) earthquake of 1947.
3. The Fallon (Nevada) earthquake of 1954.
4. The Kern County (California) earthquake of 1952.
5. The Tokachi-Oki (Japan) earthquake of 1967.

and 6. The San Fernando (California) earthquake of 1971.

However, useful information on individual embankments has occasionally resulted from other earthquakes. It is hoped that the salient features of observed embankment performance during these various events can be summarized in the following pages and appropriate conclusions drawn to aid the design engineer in making more meaningful evaluations of the data provided by analytical design procedures.

Performance of Embankment Dams in the 1906 San Francisco (California) Earthquake

At the time of the 1906 San Francisco earthquake, which resulted from rupture along some 270 miles of the San Andreas fault, a significant number of earth dams had already been constructed in California and were subjected to strong shaking during the earthquake. Surveys show that 33 dams, ranging in height from 15 to 140 ft were located within 37 miles of the fault; the relative locations of these dams and the fault are shown in Fig. 1.

For each of these dams, data were collected on the following features:

(1) Date of construction.

(2) Distance from the fault and the probable maximum acceleration in rock at the site.

(3) Geometric configuration of the embankment, i.e., height, slopes and depth of foundation.

(4) Method of construction (where available).

(5) Types of soils comprising the embankment and foundation.

(6) Extent of damage due to shaking.

A concise summary of this information is presented in Table 1. Estimates of the maximum accelerations developed in rock at the various dam sites were estimated on the basis of acceleration attenuation curves developed for the Western United States (Schnabel and Seed, 1972).

The close proximity of many of these dams to the fault on which this major earthquake occurred is emphasized by the data summary in Table 2. Here the dams are divided into four groups and it is readily apparent that almost half (16) were located within 5 miles of the causative fault of what many people consider to be one of the strongest earthquakes in the known history of the United States.

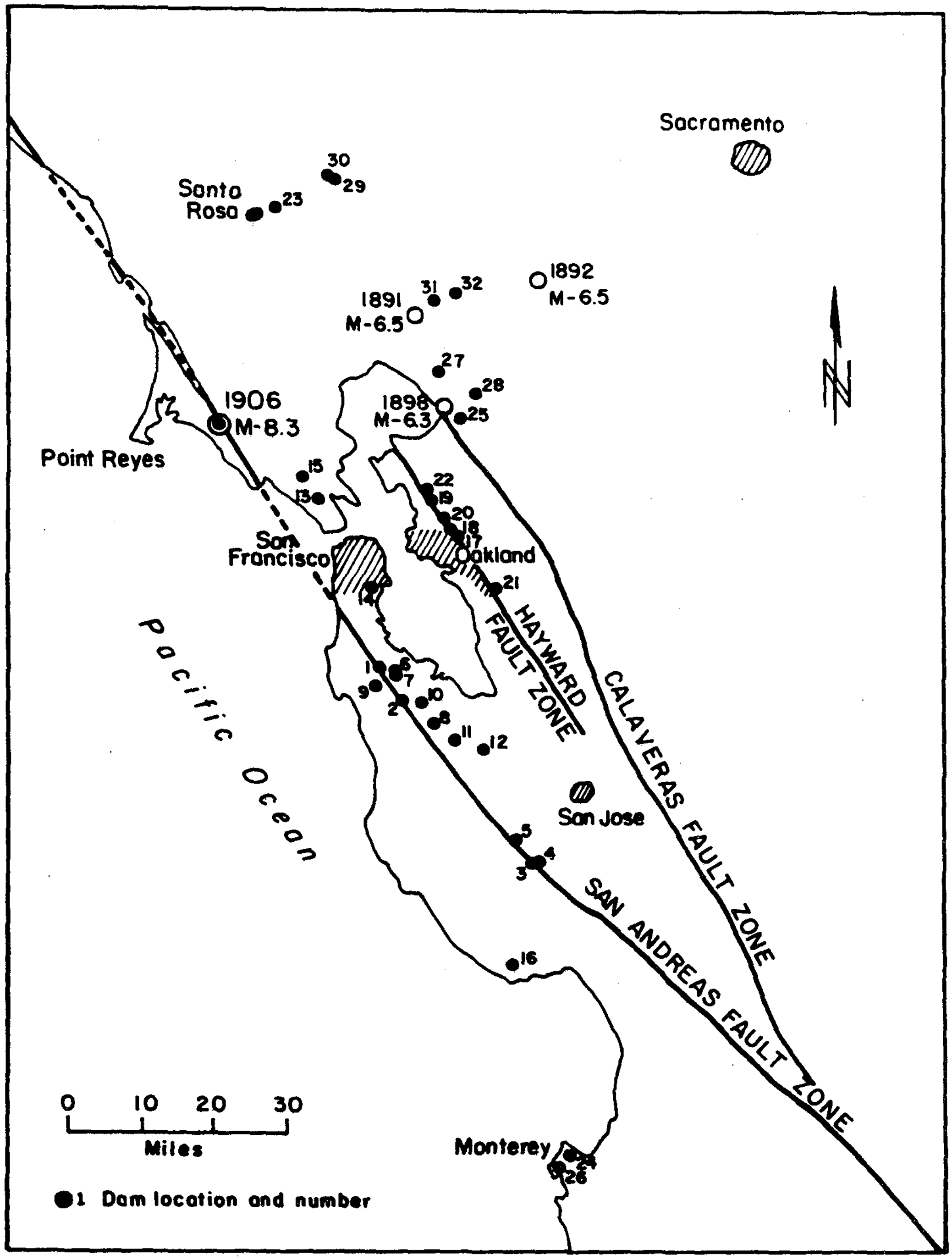

Fig. 1. LOCATIONS OF DAMS SHAKEN BY THE SAN FRANCISCO 1906 EARTHQUAKE.

Table 1: DAMS WITHIN 35 MILES OF SAN ANDREAS FAULT - 1906 EARTHQUAKE.

	Name of Dam	Date of Constr.	Distance from fault (miles)	Height (ft)	Embankment Material	Foundation Material	Performance During 1906 Earthquake	Estimated acceleration in Base Rock (g)
1	San Andreas	1870	0	97	sandy clays & clayey sands	clays mixed with sand & gravel	minor long & transverse cracks	0.8
2	Upper Crystal Springs	1877	0	75	probably similar to San Andreas	clays mixed with sand & gravel	minor long. cracks	0.8
	Old San Andreas	before 1875	0					0.8
3	Upper Howell	1878	0	38	probably same as lower Howell		cracks and settlement	0.8
4	Lower Howell	1877	0	38	sandy clays, clayey sand, weathered sandstone	crushed sandstone & shale	5' breach due to escaping water	0.8
5	Lake Ranch	1878	0	36	Gr. sandy clay, lean clay, silty sand	Weathered sandstone	No damage reported	0.8
6	Burlingame	1876	1	24	sandy clay, gravelly clay	sandy clay	No damage reported	0.8
7	Crocker	1890	1.3	45	silty sandy clay	sandy clay	No damage reported	0.75
8	Emerald Lake #1	1885	1.4	57	Mixed Clay and Adobe		No damage reported	0.75
9	Pilarcitos	1866	2	103	Silty Clay, some clayey sand	Clay mixed with gravel	No damage reported	0.75
10	Notre Dame		2	50	Silty Clay, sandy clay	Bedrock	No damage reported	0.75
11	Bear Gulch	1896	2	45	Silty Clay and sandy clay	Weathered shale & sandstone	No damage reported	0.75
12	Lagunita	1900	4	15	Sandy Clays	Sandy clays and silty sand	No damage reported	0.65
13	Belvedere	1905	5	48	Gravelly sandy clay		No damage reported	0.60
14	Univ. Mount N. Basin	1885	5	17	Silty Sand, clayey sand, Gr. sandy clay	Sandy Gr. Clay, silty sand	No damage reported	0.60
15	Lagunitas	1872	5	48	Sandy clay, gr. Sandy Clay	Sandy Gr. clay	No damage reported	0.60
16	Cowell	1890	12	50	Sandy Clay to clayey sand w/sand-stone fragments	Stiff sandy clay w/sand-stone fragments	No damage reported	0.45
17	Estates	1903	18	93	Sandy Clay, occasional sand and gravel	Sandy clay & Clayey sand (very stiff)	No damage reported	0.35
18	Piedmont	1905	18	52	Silty to Clayey fine sand w/ sandstone fragments	Sandy Clay & Clayey Sand (very stiff)	Minor cracks & settlement	0.35
19	Berryman	1905	18	40	Silty to sandy clays, occasional silty sands	Stiff silty clays w/gravel	No damage reported	0.35
20	Lake Temescal	1869	18	105	Sandy Clay, lean clay, shale and sandstone fragments	Bedrock	Minor cracks	0.35
21	Chabot	1892	19	135	Sandy Clays and Clayey sand w/gravel	Gr. Sandy clay & bedrock	No damage reported	0.35
22	Summit	1891	19	21	Sandy silty clay, stiff clay w/gravel	Silty Sandy clay (stiff)	No damage reported	0.35
23	Lake Ralphine	1882	22	35	Yellowish brown clay w/rock fragments		No damage reported	0.35
24	Pacific Grove	1882	26	20			No damage reported	0.3
25	Port Costa	1890	28	45	Clayey soil		No damage reported	0.25(0.5)*
26	Forest Lake	1892	28	60	Clayey soil		No damage reported	0.25
27	Lake Chabot	1870	29	50	Silty clay, lean clay w/traces of sand & gravel	Silty gravelly clay	No damage reported	0.25(0.4)*
28	Lake Herman	1905	32	50	Clayey soil	Clayey Allurium	No damage reported	0.25
29	Lower St. Helena	1878	32	50			No damage reported	0.25
30	Upper St. Helena	1900	32	50	Lean clay, gr. sandy clay, gr. cl. sand		No damage reported	0.25
31	Lake Camille	1800	33	30	Clayey soil		No damage reported	0.25(0.5)*
32	Lake Frey	1894	37	83	Lean clay	Bedrock	No damage reported	0.20

* Accelerations due to earthquake of 1891, 1892, 1898 of magnitude 6.5 in the vicinity (see Figure 1)

For an earthquake of magnitude about 8-1/4, it is estimated that these dams experienced base accelerations in rock of more than 0.6g at predominant periods of the order of 0.4 seconds, and durations of significant shaking of about 60 seconds. Considering the severity of such levels of shaking, and the fact that the natural periods of the embankments in question (median height 60') are not very different from the predominant period of the ground motion, one might expect substantial damage to have occurred. Yet the majority of the embankments survived the shaking without any significant damage at all. This behavior is in marked contrast to the behavior of the Upper and Lower San Fernando Dams in the earthquake of 1971, the behavior of a number of Japanese dams in the Tokachi-Oki earthquake of 1968 and the behavior of the Sheffield Dam in the Santa Barbara, California, earthquake of 1925. It is of interest, therefore, to explore the possible reasons for the excellent performance record in the San Francisco earthquake.

Three major factors are considered to contribute to the stability and performance of an embankment during an earthquake:

1) Section geometry (Upstream and downstream slopes),

2) Construction method and compaction procedure,

3) Type of embankment and foundation material.

Thus the data available on the 33 dams considered should be examined in the light of the above factors to determine if one or more of them was common to most embankments and might therefore serve to explain their excellent performance.

(1) Section Geometry:

Table 3 shows values of upstream and downstream slopes for 26 of the 33 embankments studied. Values of upstream slopes ranged between 1.3:1 and 3-1/2:1 and those for downstream slopes ranged between 1.3:1 and 3:1.

Table 2: LOCATION OF DAMS RELATIVE TO SAN ANDREAS FAULT (1906)

Distance from Fault - miles	No. of Dams	Estimated Max. Accn. in Rock -g
0 - 1	6	0.75 to 0.8
1 - 5	10	0.6 to 0.75
12 - 20	7	0.35 to 0.45
20 - 37	10	0.25 to 0.35

Table 3: EMBANKMENT SLOPES FOR DAMS AFFECTED BY 1906 EARTHQUAKE

Name of Dam	Upstream Slope	Downstream Slope
San Andreas	3-1/2:1	3:1
Upper Crystal Springs	2:1 & 3-1/4:1	2:1(75) 3-1/2:1
Old San Andreas		
Lake Ranch	2-3/4:1	2-1/2:1
Lower Howell	2:1	1-1/2:1
Upper Howell	2:1	2:1
Burlingame	2-1/2:1	2:1
Crocker	2-1/2:1	2:1
Emerald Lake #1	1.3:1	1.3:1
Pilarcitos	2-1/2:1	2:1
Notre Dame		
Bear Gulch	3:1	2.9:1
Lagunita	3:1	2:1
Belvedere	3:1	2:1
Univ. Mound N. Basin	3:1 & 1-1/2:1	2:1
Lagunitas	2-1/2:1	2:1
Cowell		
Estates	2:1	2:1
Piedmont	2:1	2:1
Berryman		3:1
Lake Temescal	1-1/2:1 & 3:1	2:1
Chabot	2:1	3:1 (with berm)
Summit	2:1	2:1
Lake Ralphine		
Pacific Grove	2-1/2:1	2:1
Port Costa		
Forest Lake	3:1	2:1
Lake Chabot	3:1	2:1
Lake Herman	3:1	2:1
Lower St. Helena	2:1	2:1 (berm) 2:1
Upper St. Helena	2:1	1-1/2:1
Lake Camille		
Lake Frey	2.7:1	2:1

Table 4: EXTENT OF DAMAGE, 1906 SAN FRANCISCO EARTHQUAKE

Dam	Height	Est. Max. Accn. in Rock	Damage
San Andreas	97	0.8g	Minor long and transverse cracks
Upper Crystal Springs	75	0.8g	Minor longitudinal cracks
Lower Howell	38	0.8g	5 ft breach due to ruptured outlet pipe
Upper Howell	38	0.8g	Some cracks and settlement
Piedmont	52	0.35g	Minor cracks and settlement

From an examination of Table 3 and from a comparison of the locations of the various embankments (the intensity of shaking they experienced and the resulting damage) and their corresponding slopes, there does not appear to be any correlation between the slopes of the embankments and their stability or performance during the earthquake. While the San Andreas Dam with slopes of 3 and 3-1/2:1, which was almost intercepted by the fault, suffered minor cracking, Pilarcitos Dam with similar height, construction procedure, foundation characteristics, and within 1.7 miles of the fault, but with slopes of 2-1/2 and 2:1 escaped without any damage.

Another example is Emerald Lake Dam with slopes of 1.3:1 and located within 1.5 miles of the fault, which escaped with no damage, in contrast to Piedmont Dam which is of similar height, at a distance of 18 miles from the fault, but with slopes of 2:1, and which suffered some cracking and crest settlement.

(2) Construction Procedure:

Unfortunately, not enough information is available on the construction procedures employed for the dams studied to permit a detailed comparative study of the effects of this factor; however, data are available on 12 dams and may be broken down as follows:

2 dams were compacted in 10" layers using 3-ton rollers

3 dams were compacted by teams and wagons

5 dams were compacted by moving livestock

2 dams were probably rolled in layers

From the information above and on the basis of the standard practice for that period, the general method of construction for these dams could be described as:

- placing the material in layers (various thicknesses have been used);
- using some means of compaction, either the hauling equipment, or moving live-stock, and at times light-weight rollers;
- water was used sometimes for wetting the soil, but moisture control methods did not exist at the time.

It seems unlikely, in view of modern construction techniques, that such methods would consistently result in dense compact material; accordingly the construction procedure does not serve to explain the excellent performance of the dams under the severe loading conditions to which they were subjected.

(3) Embankment and Foundation Material:

Table 1 provides a description of embankment material for the 33 dams considered. Most of the embankments consisted of clays of low to medium plasticity, while a few contained in addition some clayey sands.

The embankments were divided into three groups according to the type of soil of which they were constructed. Table 5 shows the dams forming the three groups and a definition of the type of soil in each group.

It may be seen from Table 5 that 31 of the 33 embankments consist of predominantly clayey soils or of mixed sandy clays and clayey sands, with only 2 being comprised partly of sandy soils.

A similar grouping was made for the foundation materials of the dams as shown in Table 6. In this case it was found that all of the embankments were underlain by bedrock, clayey soils, or very dense sand foundations.

(4) Overall Assessment of Performance:

Out of the 33 dams shaken by the earthquake, only 5 of the embankments were reported to have experienced any form of damage. Table 4 summarizes

Table 5: CLASSIFICATION OF DAMS ACCORDING TO TYPE OF EMBANKMENT MATERIAL

Predominantly Clay[1]	Mixed Clays to Clayey Sands[2]	Predominantly Sands[3]
Lake Ranch	San Andreas	Univ. Mound N. Basin*
Burlingame	Upper Crystal Springs	Piedmont**
Crocker	Lower Howell	
Emerald Lake #1	Upper Howell	
Pilarcitos	Cowell	
Notre Dame	Chabot	
Bear Gulch	Pacific Grove	
Lagunita	Forest Lake	
Belvedere		
Lagunitas		
Estates		
Berryman		
Lake Temescal		
Old San Andreas		
Summit		
Lake Ralphine		
Port Costa		
Lake Chabot		
Lake Herman		
Upper St. Helena		
Lower St. Helena		
Lake Camile		
Lake Frey		

[1]"Predominantly Clay": greater portion of the material encountered consists of, for example, sandy clays, silty clays, lean clays.

[2]"Mixed Sands and Clays": material encountered consists of approximately equal portions of say, sandy clay and clayey sand.

[3]"Predominantly Sands": greater portion of the material encountered consists of, for example, clayey sand or silty sand.

*Reservoir totally lined concrete.

**Reservoir filled only few months before earthquake--fill probably not saturated.

Table 6: CLASSIFICATION OF DAMS ACCORDING TO TYPE OF FOUNDATION MATERIAL

Bedrock	Predom. Clay	Mixed Clays and Sands	Predominantly Sands
Lake Ranch	San Andreas	Lagunita	University Mound N. Basin
Lower Howell	Old San Andreas	Estates	
Upper Howell	Burlingame	Piedmont	
Notre Dame	Crocker	Pacific Grove	
Bear Gulch	Pilarcitos		
Lake Temescal	Lagunitas		
Upper St. Helena	Berryman		
Lower St. Helena	Chabot		
Lake Frey	Summit		
Lake Camille	Lake Chabot		
	Lake Herman		
	Cowell		
	Upper Crystal Springs		
	Emerald Lake #1		
	Belvedere		
	Lake Ralphine		
	Port Costa		

the extent of this damage, together with information on the heights of the embankments and the estimated maximum accelerations developed in rock. Except in the case of one dam, Lower Howell, where a breach formed due to water escaping from a ruptured outlet pipe, damage to the other four was restricted to minor cracks and settlement and all embankments remained operational without any need for repairs; this in spite of the fact that the fault passed through the Upper Crystal Springs Dam and within 100 ft of the abutments of the San Andreas, Upper Howell and Lower Howell Dams. The behavior of the Upper Crystal Springs Dam was not considered of particular significance, however, since it had water at equal heights on both sides and was used only as a causeway at the time of the earthquake.

Thus the remarkable fact emerges that 32 dams were shaken severely by an 8-1/4 magnitude earthquake without sustaining any significant damage. It has been determined that the inclination of the slopes did not offer any added feature for stability; nor did their method of construction provide any added advantage; in fact, construction procedures can be considered less than adequate by today's standards of compaction procedures.

The one feature found common to most of the dams is the clayey nature of the embankment material and foundation soils. Most of the embankments consisted of clayey soils of low to medium plasticity, and considering the wide variation in their slopes and the uncontrolled and probably substandard methods of compaction employed in constructing them, it seems reasonable to conclude that the behavior of the clayey soils was the major factor contributing to their stability.

Two embankments, however, (University Mound and Piedmont) were found to consist predominantly of sandy material. Located at distances of 5 and 18 miles from the fault, their adequate performance does not conform with the expected behavior of relatively loose saturated sands under conditions of strong shaking. Accordingly these two embankments were examined more closely to determine the reasons contributing to their stability. In the following paragraphs a description of the characteristics of the two embankments is presented along with the reasons for their adequate behavior.

University Mound North Basin:

Located about 5 miles from the fault, this dam has a height of 24 feet with upstream slopes of 3:1 and 1-1/2:1 and a downstream slope of 2:1. The embankment consists of predominantly silty sand (a product of the Colma formation) and some clayey gravels and clayey sands (a product of the Franciscan formation).

Its foundation is the Colma formation which consists of dense to very dense silty sand of the Pleistocene age.

The inside walls and the floor of the reservoir are lined with reinforced concrete to make it water tight. This condition would seem to eliminate the possibility of the embankment being saturated and would thus serve to explain its adequate behavior. The water table was encountered recently at a depth of 25 feet into the foundation (in the month of October); during the winter season the water table would still be within the foundation zone, which as mentioned earlier consists of very dense material and as such presents no problems of instability.

It thus appears that the dry condition of the embankment and the denseness of the foundation material were primarily responsible for the adequate behavior of the dam during the earthquake.

Piedmont Reservoir:

Located at approximately 18 miles from the zone of energy release, this dam has a height of 50 feet and upstream and downstream slopes of 2:1.

The embankment consists of predominantly medium dense (non-plastic) silty sand and clayey sands and some sandstone fragments. The material was spread in thin layers, watered, and compacted by rollers. The average dry density of the embankment material was of the order of 115 pcf with a water content of 17%, and a relative compaction of approximately 97% based on standard AASHO maximum densities. The foundation material consisted of a layer of residual soil consisting of medium dense clayey sand and some sandy clays, underlain by a layer of "weathered overburden" consisting of very stiff sandy clays and dense clayey sands with dry densities between 112 and 116 pcf and water content values of 17 to 19%.

One very important factor is the fact that the dam was completed in 1905 and the reservoir filled for the first time only a few months before the earthquake (Ambraseys, 1960). In addition, the upstream face was lined with 6" thick reinforced concrete slabs with the construction joints sealed with asphalt. Considering also the impervious nature of the dense silty and clayey sand embankment, it again seems unlikely that any sizeable portion of the embankment would be in a saturated condition during the earthquake. The possibility of the foundation being saturated does exist, however, especially since the floor of the reservoir is not lined. But considering the confining pressures due to the total weight of the embankment, and the denseness and the clayey content of the foundation soils, the foundation is not expected to cause any problems of instability.

Thus the behavior of the University Mound North Basin and Piedmont embankments seems quite reasonable due to the following two reasons:

1) The embankments were not saturated, and thus the potential for liquefaction and strength loss did not exist,

2) The foundation material seems to be strong enough to preclude any possibility of failure resulting in instability of the embankment.

In all other cases, it seems likely that the clayey nature of the construction materials, which generally exhibit little tendency for strength loss during shaking, was responsible for the excellent performance of the embankments.

Performance of Embankment Dams in the 1939 Ojika (Japan) Earthquake

The Ojika earthquake of 1939 was a magnitude 6.6 event which occurred in the northwestern portion of Honshu, Japan, causing estimated acceleration intensities in the zone of severe damage of the order of 0.3 to 0.4g.

A substantial number of low embankment dams forming reservoirs for irrigation purposes were damaged during the earthquake. As a result, a comprehensive survey (Akiba and Semba, 1941) was undertaken to investigate the damage to these embankments. A total of 74 embankments were severely damaged, of which 12 failed completely. Detailed surveys were made of 58 of the damaged embankments together with 12 which suffered no significant damage due to the shaking. The heights of the embankments ranged between 5 and 60 feet, but there was no apparent correlation between the height of an embankment and the extent of damage it experienced. Similarly although embankment construction records were found for 31 of the embankments investigated, no relationship between damage pattern and construction procedures could be determined.

Table 7 shows a breakdown of the number of embankments associated with different types of damage. It shows that slope failures and cracks (bulging being considered as a mild slope failure) were causes of damage in at least 80% of the embankments, with the type of damage being undetermined for most of the others due to the fact that they suffered complete failure. The grain size distributions for the embankment materials comprising the 12

Table 7: OJIKA EARTHQUAKE - RESERVOIR DAMAGE*

Statistics on slope failures and crack development

Upstream slope failure	17
Downstream slope failure	6
Both upstream and downstream slope failure	8
Undetermined due to complete failure	9
Undetermined (others)	2
Cracking and bulding	6
Bulging only	1
Cracking only (primarily vertical)	9
	58

*After Akiba and Semba (1941)

Table 8: DAMAGE TO DAMS IN OJIKA EARTHQUAKE 1939*

Grain Size Distribution for Dams Suffering Complete Failure

Dam No.	>0.1 mm	0.01 to 0.1 mm	<0.01 mm	Notes
1	85	15	0	
2	84	15	1	
3	82	15	3	
4	66	18	16	
5	72	21	9	
6	79	12	9	
7	77	13	10	
8	92	5	3	
9	81	6	13	
10	34	38	28	Failure in poorly compacted material over outlet conduit
11	14	59	27	
12	36	31	33	Failure of conduit leading to piping

*After Akiba and Semba (1941)

embankments which failed completely is shown in Table 8. As may be seen from the data in this table, 9 of these 12 embankments were constructed primarily of sands. For the three failures occurring in embankments constructed of clayey sands, two of the failures were found to be due to a washout of poorly compacted soil over recently repaired outlet conduits while the third was due to rupture of an outlet conduit which led to piping.

Of particular interest are the main conclusions reported by the authors of this study (Akiba and Semba, 1941) as follows:

1. There were very few cases of dam failures during the earthquake shaking. Most of the failures occurred either a few hours or up to 24 hours after the earthquake.
2. The majority of the damaged and failed embankments consisted of sandy soils; no complete failure occurred in embankments constructed of clay soils.
3. Even at short distances from the epicenter, there were no complete failures among the embankments built of clayey soils; however, at greater epicentral distances, there was a heavy concentration of completely failed embankments composed of sandy soils.

This experience provides further evidence of the vastly superior stability of embankments constructed of clay soils under strong seismic loading conditions. It also suggests that the critical period for an embankment dam subjected to embankment shaking is not only the period of shaking itself but also a period of hours following an earthquake, possibly because piping may occur through cracks induced by the earthquake motions or slope failures may result from pore pressure re-distribution.

Performance of Embankment Dams in the 1952 Kern County (California) Earthquake

The embankments shaken by this earthquake provide a comparison between the behavior of rolled fill embankments of clayey material and the comparatively poorer performance of hydraulic fill dams of sandy material. The earthquake registered 7.6 on the Richter magnitude scale and shook the better part of Kern County and the northern part of Los Angeles County. Seven embankments, ranging in height between 20 and 190 feet were located within 50 miles of the epicenter where the shaking intensity ranged from very strong to moderate; some damage occurred however at one dam (South Haiwee) at a distance of 95 miles from the zone of energy release. Table 9 shows the heights of the affected embankments, their distance from the fault, the estimated base accelerations, the type of embankment material and construction procedure, and a summary of the reported damage.

As can be seen from Table 9, of the three hydraulic fill sandy embankments, two suffered longitudinal cracking with one (Dry Canyon) developing cracks indicative of a potentially incipient sliding condition. The estimated shaking intensities at these two embankments were approximately 0.12g and 0.04g. In contrast, three rolled fill embankments of compacted sandy clays subjected to estimated accelerations ranging between 0.06g and 0.12g, suffered no significant damage.

Fairmont Dan, however, which is also a hydraulic sand fill embankment located about 36 miles from the fault, was shaken by estimated accelerations of the order of 0.18g and suffered no significant damage.

The settlements reported in the Buena Vista embankment were due to cavities caused by leaching of gypsiferous beds in the foundation (Esmiol, 1965), and thus were not considered representative of the behavior of the embankment.

Again in this case there was no correlation between the heights and slopes of the embankments and their performance during the earthquake. However, the consistent pattern of behavior observed in the previous cases was also noticed in this earthquake; namely, that embankments of compacted clayey material at relatively short distances from the epicenter faired much better than embankments of sandy material at larger distances from the fault and at much lower shaking intensities.

Performance of Embankment Dams in the 1954 Fallon (Nevada) Earthquake

Two earthquakes with magnitudes about 6.7 occurred near Fallon, Nevada on July 6 and August 23, 1954, producing local ground motions of Modified Mercalli Intensity VIII to IX, corresponding to peak ground accelerations of the order of 0.25 to 0.4g in the near-epicentral region (Trifunac and Brady, 1976).

Damage to water supply facilities and embankment dams in the area is reported by Ambraseys (1960): "Extensive damage was caused to canal and drainage facilities of the Newlands Project. Culverts collapsed and longitudinal cracking and sloughing occurred in many places along both drainage channels and irrigation canal embankments. Canal levees settled from 1 to 3 ft and the bottoms of canals were raised by 1 to 2 ft. In one case the bottom of a drainage ditch was raised by 6 ft.....At the Wildlife Area, the East Canal was severely damaged for about 2 miles and 200 yds were completely destroyed. West of Fallon a 40 ft portion of embankment, about 20 ft high failed completely..."

Damage to embankment dams was reported by Ambraseys (1960) as follows:

Lahoutan Dam

"The foundation material (for this dam) consists of seamy broken sandstone and shale of varying degrees of hardness. The overburden, composed

Table 9: 1952 KERN COUNTY EARTHQUAKE

Dam	Date of Const.	Height -ft	Dist. from fault -miles	Estimated Max. Accn.	Type of Fill	Effect of Earthquake
Tejon Storage	1946	32	4	≃0.6g	Rolled fill (sandy clay)	No significant damage
Buena Vista	1890	20	19	≃0.3g	Unknown	1 to 2 ft settlement over cavities in gypsum beds
Fairmont	1912	121	36	≃0.18g	Hydraulic fill	No reported damage
Drinkwater	1923	105	42	≃0.13g	Rolled fill	No damage
Dry Canyon	1912	66	45	≃0.12g	Hydraulic fill (sand, silty sand)	Longitudinal cracks along crest
Bouquet Canyon	1934	190	46	≃0.12g	Rolled fill (sandy clay)	No damage
South Haiwee	1913	81	95	≃0.04g	Hydraulic fill (sand, clayey silt)	Slight cracking

Table 10: EARTH DAMS DAMAGED BY THE TOKACHI-OKI EARTHQUAKE*

	Under 10 meters		Over 10 meters	
	Number	%	Number	%
Sliding causing failure	9	10.5	1	12.5
Sliding of upstream slope	21	24.7	4	50.0
Sliding of downstream slope	10	11.7	0	0
Sliding of both slopes	4	4.7	0	12.5
Cracking in dam body	24	28.2	1	12.5
Settlement of dam	7	8.2	1	12.5
Damage of appurtenant structures	22	25.8	2	25.0

Note: Number of dams investigated - 93 earth dams
Dams under 10 meters high - 85
Dams over 10 meters high - 8
When two or more types of damage were found in the dam, they were counted separately.

*After Moriya (1974)

of sandy clay and gravel, was removed and the dam was built on mudstone and stiff clay." The dam was located about 30 miles from the epicenter of the earthquakes, where peak ground accelerations may be expected to be about 0.15g, and apparently sustained no damage from either shock.

Coleman Dam

"This is a composite concrete-earth diversion structure and it is located one mile north-west of Fallon. Reports indicate that the structure was destroyed during the earthquake. This structure failed because of displacement and cracking of the earth-fill abutments, which in turn permitted water to erode around the concrete portion, causing it to partially overturn, crack and settle. The earth abutments were completely washed out."

Rogers Dam

"This dam was reported to have failed during the August 23rd earthquake in Fallon. No details were found but it appears that the structure, like Coleman Dam, was composed of a central concrete spillway with two earth-fill embankments. It was situated 3 miles north-east of Lovelock in Nevada. The southeast part of the dam composed of earth-fill gave way and a portion of the concrete structure was broken off and turned around into the spilling basis."

Performance of Embankment Dams in the 1968 Tokachi-Oki (Japan) Earthquake

This earthquake, with magnitude 7.8, caused extensive damage to embankments in the Aomori Prefecture in the northern tip of the main island of Honshu, Japan. The shaking intensity in the damaged area had peak accelerations of the order of 0.15 to 0.2g. Most of the embankments were small earth structures used for irrigation purposes with heights ranging between 15 and 60 feet. The embankment material consisted mostly of loose volcanic sand characterized by low values of standard penetration resistance constructed on foundation materials of a similar nature. A study of the damage to these embankments was reported by Moriya (1974). Table 10 shows a statistical summary of the type of damage observed in the 93 embankments investigated. It shows that sliding in the upstream and downstream slopes (or both) was the reported cause of damage in at least 50% of the embankments under 30 feet in height, and in 75% of the embankments over 30 feet in height. In 12% of the cases the sliding resulted in complete failure of the embankment. The slopes, in embankments under 30 feet ranged between 1:1.5 for the downstream face and 1:2.5 for the upstream face, while those for embankments over 30 feet ranged between 1:2.5 and 1:3.5 for the downstream and upstream sides respectively. There was no observed correlation between the slopes of the embankments and their performance during the earthquake. Most of the embankments were of homogeneous section; however, some were constructed with an impervious clay central core and it was noted that "sliding of the downstream slope did not occur in center-core dams with good water-tightness."

It seems likely from the above summary that slope failures in these embankments, as with those of the Ojika 1939 earthquake, can be attributed primarily to the phenomenon of liquefaction and/or strength loss associated with cyclic loading effects in loose saturated sands, leading to sliding and slumping mostly in the upstream slope and in some cases to complete failure of the embankment.

Performance of Embankment Dams in the 1971 San Fernando (California) Earthquake

A large number of earth dams in the County of Los Angeles were shaken by the 1971 San Fernando earthquake which registered a magnitude of 6.6 on the Richter scale. Within a 25-mile radius around the epicentral region (see Figure 2) some 44 dams, ranging in height between 30 and 190 feet, were subjected to peak accelerations of the order of 0.2 to 0.7g.

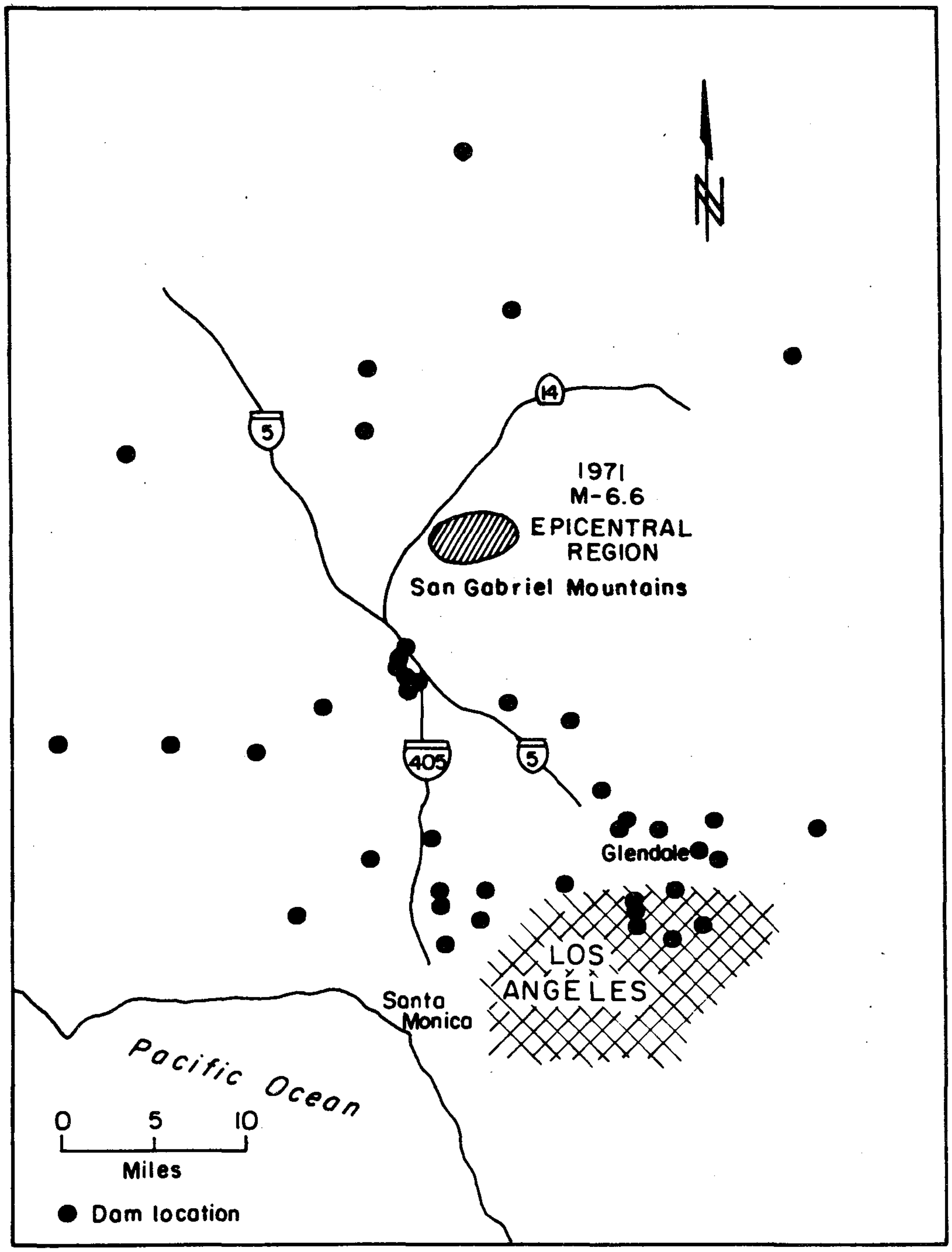

Fig. 2. DAMS LOCATED WITHIN 25 MILES OF THE EPICENTRAL REGION - SAN FERNANDO 1971 EARTHQUAKE.

The available information on these embankments in terms of their height, type, distance from fault, estimated base acceleration, and their performance during the earthquake is shown in Table 11. Of the 44 embankments listed in Table 11, 13 were non-operational (empty reservoir) during the earthquake and one was not saturated, so that only 30 of the embankments may be considered representative of fully operational dams. Of these operational embankments, 25 consisted of rolled fill dams of compacted clayey materials, and 5 were hydraulic sand fill dams. Whereas there was no damage observed in any of the 25 rolled fill embankments, two of the 5 hydraulic fill dams (Upper and Lower San Fernando dams) suffered substantial damage and major slides (Seed et al, 1973). The intensity of shaking at these two embankments was of the order of 0.5 to 0.6g. The shaking was of sufficient intensity and duration to induce large pore pressures in the saturated sandy embankments resulting in liquefaction and loss of strength. In the case of the Lower Dan the extent of the liquefied zone resulted in a major slide on the upstream side. In the Upper Dam, however, the result was a slide movement of 5 feet in the downstream direction and a crest settlement of 3 feet. The behavior of the Lower San Fernando Dam was typical of failures observed in the sandy embankments of the Ojika and Tokachi-Oki earthquakes in Japan.

However, the remaining 3 hydraulic fill embankments (Fairmont, Lower Franklin, and Silver Lake), all subjected to estimated maximum acceleration levels of the order of 0.2g performed adequately and suffered no significant damage. A summary of the performance of hydraulic fill dams with reference to the accelerations developed is presented in Table 12.

The contrasting behavior between rolled fill embankments and hydraulic fill embankments during this earthquake is essentially the same as that observed during the Kern County earthquake.

Table 11: EARTH DAMS WITHIN 25 MILES OF EPICENTRAL REGION - SAN FERNANDO EARTHQUAKE.

Dam	Type	Approx Distance From Epicentral Region (Miles)	Estimated max. Rock Acceleration (g)	Height	Reservoir Condition	Damage
Wilson Debris	Rolled fill	3½	0.70		Empty	Minor cracking of embankment
Lopez	Rolled fill	5	0.6	166	Filled with debris	No significant damage
Yarnell Debris Basin	Rolled fill	5½	.68	49	Empty	No significant damage
Upper San Fernando	Hydraulic fill	6	.55	82	Operational	Slide movement 6ft. downstream
Dry Canyon	Hydraulic fill	6½	.55	66	Empty	No significant damage
San Fernando Dike B	Rolled fill	6½	.55	36	Empty	Some slumping of embankment
Channel Diversion Dike	Rolled fill	6½	.55	43	Empty	Some cracking in embankment
Van Norman Bypass	Rolled fill	7	.50		Operational	No significant damage
Lower San Fernando	Hydraulic fill	7	.50	142	Operational	Major slide upstream
Hansen	Rolled fill	9	.43	97	Empty	No significant damage
Green Verdugo	Rolled fill	9	.43	117	Operational	No significant damage
Drinkwater	Rolled fill	9	.43	105	Operational	No significant damage
Porter Estate	Rolled fill	10	.40	41	Empty	No significant damage
Bouquet Canyon	Rolled fill	11	.36	190	Operational	No significant damage
Reservoir No. 1	Rolled fill	14	.29	35	Operational	No significant damage
Chatsworth	Hydraulic fill	14½	.28	44	Empty	No significant damage
Brand Park	Rolled fill	15	.28	96	Operational	No significant damage
10th and Western	Rolled fill	15½	.27	28	Operational	No significant damage
Sepulveda	Rolled fill	15½	.27	57	Empty	No significant damage
Harold	Rolled fill	16½	.25	30	Operational	No significant damage
Diederich	Rolled fill	17	.24	60	Operational	No significant damage
Encino	Rolled fill	17	.24	168	Operational	No significant damage
Santa Felicia	Rolled fill	17	.24	200	Operational	No significant damage
Chevy Chase	Rolled fill	18	.23	35	Operational	No significant damage
Runkle	Rolled fill	18	.23	41	Empty	No significant damage
Upper Hollywood	Rolled fill	18	.23	87	Operational	No significant damage
Upper Franklin	Rolled fill	18	.23	55	Operational	No significant damage
Upper Stone Canyon	Rolled fill	18	.23	110	Operational	No significant damage
Glenoaks	Rolled fill	19	.22	62	Operational	No significant damage
Stone Canyon	Rolled fill	19	.22	185	Operational	No significant damage
Fairmont	Hydraulic fill	20	.20	121	Operational	No significant damage
Lower Franklin	Hydraulic fill	20	.20	103	Operational	No significant damage
Eagle Rock	Rolled fill	20	.20	113	Operational	No significant damage
Rowena	Rolled fill	20	.20	29	Operational	No significant damage
Rubio Div. Debris	Rolled fill	20	.20	55	Empty	No significant damage
Highland	Rolled fill	20	.20	12	Operational	No significant damage
Ivanhoe	Rolled fill	20	.20	20	Operational	No significant damage
Sawtell	Rolled fill	21	.19	34	Operational	No significant damage
J.W. Wisda	Rolled fill	21	.19	50	Empty	No significant damage
Silver Lake	Hydraulic fill	21	.19	53	Operational	No significant damage
Eaton Wash	Rolled fill	21	.19	63	Operational	No significant damage
Ascot	Rolled fill	22	.18	73	Operational	No significant damage
Elysian	Rolled fill	22	.18	72	Operational	No significant damage
Wood Ranch	Rolled fill	23	.17	146	Operational	No significant damage

* Embankment not saturated--not representative. ** Under re-construction

Table 12: DAMAGE TO HYDRAULIC FILL DAMS WITHIN 25 MILES OF EPICENTRAL REGION - SAN FERNANDO EARTHQUAKE, 1971

Dam	Approx. distance from epicentral region - miles	Approx. max. rock accelerations	Performance
Upper San Fernando	6	≃0.55g	Crest slid downstream 5 ft
Dry Canyon	6-1/2	≃0.55g	Not in use - reservoir empty - no damage
Lower San Fernando	7	≃0.50g	Major slide upstream
Chatsworth	14-1/2	--	Under reconstruction - reservoir empty
Fairmont	20	≃0.20g	No significant damage
Lower Franklin	20	≃0.20g	No significant damage
Silver Lake	21	≃0.19g	No significant damage

Table 13: PERFORMANCE DURING EARTHQUAKES OF FIVE DAMS WITH SANDY SHELLS AND CENTRAL CLAY CORES

(see material characteristics in Fig. 3)

Dam	$(K_2)^*_{max}$	Field Performance
Chabot	65	Survived magnitude 8-1/4 earthquake at distance of 20 miles with no apparent damage; would apparently survive magnitude 7 earthquake at distance of 2 miles.
Lower Franklin	50	Survived magnitude 6-1/2 earthquake at distance of about 20 miles ($a_{max} \approx 0.2g$) with no apparent damage.
Fairmont	50	Survived magnitude 6-1/2 earthquake at distance of about 20 miles ($a_{max} \approx 0.2g$) with no apparent damage.
Lower San Fernando	45	Major upstream slide including upper 30 ft of dam due to magnitude 6-1/2 earthquake at distance of about 5 miles ($a_{max} \approx 0.55g$); performance and analysis indicate dam would have survived same earthquake at distance of 20 miles without significant damage but would have failed catastrophically for magnitude 8-1/4 earthquake at distance of 20 miles.
Upper San Fernando	30	Downstream slide movement of crest of dam due to magnitude 6-1/2 earthquake at distance of about 5 miles ($a_{max} \approx 0.55g$); this embankment had an extremely wide downstream berm which together with its lower value of $(K_2)_{max}$ no doubt contributed to its better performance than the Lower San Fernando dam.

*Shear modulus $G = 1000\ K_2 (\sigma'_m)^{1/2}$

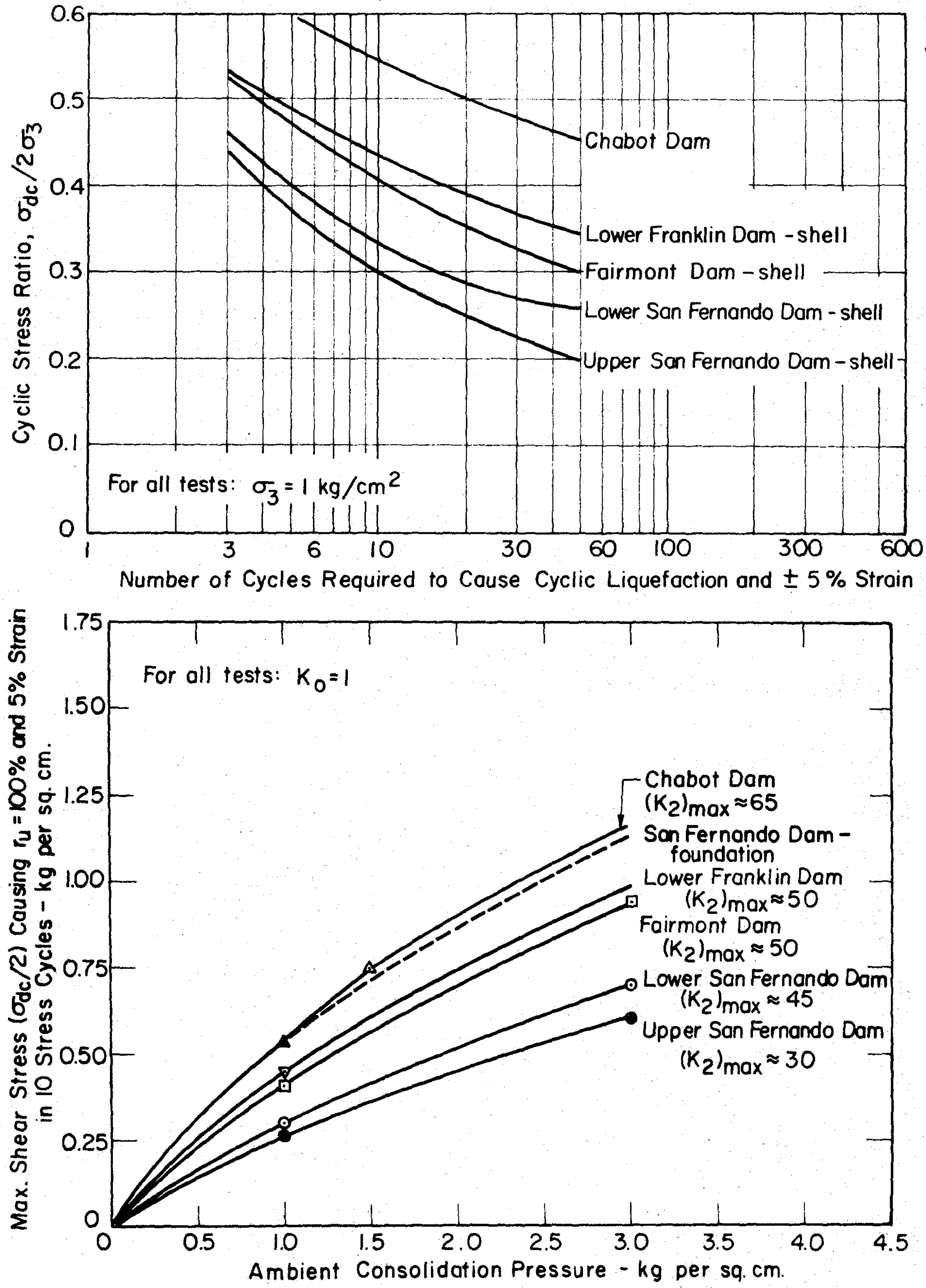

Fig. 3. CYCLIC LOADING CHARACTERISTICS OF CONSTRUCTION MATERIALS FOR FIVE DAMS WITH KNOWN PERFORMANCE DURING STRONG EARTHQUAKE SHAKING (See Table 13).

On the basis of the behavior of these embankments, it seems reasonable to conclude that hydraulic fill embankments generally are more susceptible to damage and failure than rolled fill embankments when subjected to earthquake shaking. This in no way indicates, however, that all hydraulic fill dams are inherently unstable; on the contrary, the performance record of the Fairmont, Lower Franklin and Silver Lake Dams shows that dams of this type can withstand shaking intensities of the order of 0.2g with no significant damage. At the same time the performance of a wide variety of rolled earth dams in this earthquake clearly shows that reasonably well-built structures of this type can withstand the shaking of a magnitude 6.5 earthquake, even in regions where the peak acceleration ranges from 0.2 to 0.5g, without significant damage.

The majority of the embankments consisted of clayey soils of medium to low plasticity. Thus their performance can be considered as confirmation of the excellent performance of embankments constructed of clayey soils in the 1906 and other earthquakes.

Quantitative data relating the material characteristics and the earthquake performance of hydraulic fill and wagon rolled earth fill dams in the San Francisco (1906) and San Fernando (1971) earthquakes is presented in Fig. 3 and Table 13. In general the three basic embankments parameters likely to have a major influence on the performance of a dam with reasonably full reservoir during an earthquake are:

1. The geometry of the dam; except in special circumstances, this is usually quite similar for most older dams.

2. The stiffness of the soils comprising the dam which influences the stresses induced by the earthquake. In the case of dams constructed of cohesionless soils, this may be approximately designated by the soil parameter $(K_2)_{max}$ in the equation for the shear modulus is

$$G = 1000\ K_2\ (\sigma_m')^{1/2}\ \text{psf}$$

where G is the shear modulus of the soil and σ_m' is the mean effective principal stress. In a general way, the stiffer the material comprising the dam, the higher will be the stresses induced in it by any given earthquake motion.

3. The resistance of the soils comprising the shell of the dam to pore pressure build-up and strain development under cyclic loading conditions. This may be represented by data of the type shown in Fig. 3.

The results of cyclic load rests at a standard confining pressure of 1 kg per sq cm, and the corresponding values of $(K_2)_{max}$ for 5 dams with sandy shell materials whose performance is known (see Table 13) during specific earthquakes is shown in Fig. 3. The possible performance of other dams of the same general type (sand shells with central clay core) can often be evaluated from this data by direct comparison of material characteristics and prior prototype performance.

Performance of Embankment Dams in Other Earthquakes

While the performance of embankment dams in the six earthquakes described previously each provide data on the behavior of multiple structures in the same event, there are a number of other cases of known dam performance which merit the attention of the design engineer:

1. Mexico Earthquake, 1915

Failure of the Volcano Lake Dike is reported to have occurred during an earthquake in Mexico in 1915, probably as a result of liquefaction of loose foundation soils.

2. Kanto Earthquake (Japan), 1923

During the Kanto earthquake of 1923 (with a magnitude of 8.2) damage developed in three earth dams located at some distance from the epicenter where the maximum ground accelerations were estimated to be of the order of 0.3g. Reported damage is as follows (Moriya, 1974):

Ono Dam: This dam was constructed in 1912; many fissures 2" to 10" wide developed along the crest of the dam ranging from 100 to 180 ft in length and extending to a depth of about 35 ft. One fissure was 70 ft deep. In addition, two local slides occurred on the downstream side of the dam.

Lower Murayama Dam: This was a rolled fill dam with a puddle clay core; longitudinal cracks developed along the crest accompanied by outwards movement of the downstream slope. The downstream berm settled about 4 ft and moved downstream about 6 ft with a heaving of 2 ft near the downstream toe.

Upper Murayama Dam: This dam was under construction at the time of the earthquake but a deep longitudinal crack about 1" wide formed in the completed portion of the embankment.

3. Santa Barbara (California) Earthquake, 1925

This earthquake reportedly had its epicenter about 7 miles northwest of the Sheffield Dam site and has been assigned a magnitude of 6.3. Reliable observers report that the shaking in the area of the dam had an intensity of about IX on the Rossi-Forrel scale which would correspond roughly to a peak acceleration of the order of 0.05 to 0.10g. However, an earthquake of magnitude 6.3 occurring at a distance of 7 miles is likely to produce ground accelerations substantially higher than these values.

The shaking produced a complete failure of the dam, reports indicating that sliding occurred near the base of the embankment causing a section of the dam about 300 ft in length to move bodily downstream as much as 100 ft, breaking up as it did so.

The embankment, about 720 ft long and having a maximum height of 25 ft was composed of silty sand and sandy silt but the upstream slope was faced with a 4-ft thick clay blanket which was extended as much at 10 ft down into the foundation to serve as a cut-off. The foundation soil consisted mainly of silty sand and sandy silt containing some cobbles. The upper 2 ft was somewhat looser than the underlying soil, having a relative density of the order of about 40 percent and it was apparently in this layer that the sliding occurred, since there was no formal stripping of this upper layer prior to construction. Examination after the failure indicated that there was no leakage of water through the upstream core but that seepage around and underneath the cut-off had saturated the foundation soils and the lower part of the main structure. At the time of the failure, the depth of water in the reservoir was about 15 to 18 ft (Seed et al, 1967).

4. Chile Earthquake, 1943

A strong earthquake (Magnitude 8.3) shook a number of dams in Chile in the earthquake of April 6, 1943. One of these, the Cogoti dam was subjected to shaking of intensity IX on the Rossi-Forrel scale corresponding to a peak acceleration of the order of 0.1g.

The dam was constructed of dumped rockfill with a laminated concrete facing on the upstream slope which served as the sole impervious element of the embankment. As a result of the earthquake, the crest of the embankment settled 15 inches and these were minor rock slides on the downstream slope. Overall damage to the structure, however, was insignificant.

5. El Centro Earthquake, 1940

Failure is known to have occurred in several dykes and canal banks during this earthquake (Ross, 1968), which had a magnitude of about 6.6.

Ground accelerations in the area of damage are estimated to be of the order of 0.2 to 0.5g and failure was apparently due to liquefaction of foundation soils.

6. Russian Earthquakes

Several hydraulic fill dams in Russia have been subjected to earthquake motions ranging from about 0.10g to 0.17g with little or no damage (Ambraseys, 1960). Data on the particular dams involved and their performance are summarized in Table 14.

7. Nankai Earthquake (Japan), 1946

During this earthquake, (Magnitude $\simeq$ 8.1), approximately 50 dams constructed between 760 and 1944 with heights ranging from 50 to 130 ft were subjected to ground motions with peak accelerations estimated to range from 0.08g to 0.25g with no reports of damage.

8. Fukui Earthquake (Japan), 1948

Failure occurred in the Hosorogi embankment located in an area where the maximum ground acceleration was estimated to be about 0.45g. The embankment was constructed of silty clay on a foundation of soft organic silt. Failure was apparently due to lateral spreading and settlement of the embankment into the foundation soil (Ambraseys, 1960).

9. Hebgen Lake Earthquake, 1959

The Hebgen Lake earthquake (Magnitude $\simeq$ 7.6) of August 17, 1959, occurred on a fault located only a few hundred feet from the Hebgen Dam, a compacted earth fill structure constructed on foundation soil with depths ranging from 10 to 90 ft and provided with a central concrete core wall extending through the dam and foundation soils to the underlying rock. The fault ran parallel to the reservoir and fault movements were essentially vertical, resulting in a lowering of the dam and reservoir of about 10 ft due to tectonic movements and a corresponding rise in ground level on the upthrown side of the fault.

Ground shaking in the vicinity of the dam was undoubtedly very strong and it produced the following effects on the dam: (1) settlement of the crest of the dam relative to the underlying rock; the maximum settlement was about 4 ft on the downstream side and 6 ft on the upstream side of the crest; (2) several cracks, the largest being about 3 inches wide in the concrete core wall; (3) severe longitudinal and some transverse cracking, with crack widths varying from 3 inches to 12 inches on the upstream and downstream sides of the crest; (4) some leakage near the abutment contact in the general vicinity of the core-wall cracks; this leakage stopped when the badly cracked spillway were sealed; (5) a number of waves in the reservoir which caused the crest to be overtopped at least four times for periods of about 10 minutes; the depth of water flowing over the crest was estimated at about 3 ft during the second of these overflows; (6) a number of landslides around the edges of the reservoir and (7) a massive rock-slide, estimated at about 50 million tons of rock which dammed up the river about 7 miles downstream and created a lake which backed up to the downstream toe of the dam.

Concerning this behavior, J. L. Sherard (1966) who visited the site immediately after the earthquake, and later made a detailed appraisal of the effects, reports as follows: "One does not have to reflect very long on the events which occurred before concluding that an extremely fortunate array of conditions prevented a complete failure--and with such a large reservoir, the failure would have been a disaster even in that sparsely populated area."

Fortunately the dam was constructed of a slightly cohesive well-graded sand and gravel which resisted erosion and the development of high pore pressures during the earthquake shaking. Never-the-less, slumping on the upstream side amounted to about 7 percent of the height of the dam at some sections (Seed, 1968).

10. Kita-Muto Earthquake (Japan), 1961

This earthquake with magnitude 7 occurred with its epicenter about 20 kms from the Miboro Dam, a large rockfill structure 131 m high with a sloping upstream core, an upstream slope of 1 on 2.5 and downstream slope of 1 on 1.75. Ground motions in the rocky terrain in the area of the epicenter are said to have been of short period and violent; the shaking produced several large landslides and a great amount of falling rock (Okamoto, 1973).

Accelerations in the vicinity of the Miboro Dam are estimated to be within the range 0.1g to 0.25g but the only effects on the dam were a settlement of 3 cm and a horizontal displacement of 5 cm at the crest.

11. Alaska Earthquake, 1964

Failure occurred in the embankment of a private dam near Anchorage in the Alaska earthquake of 1964. The dam had a homogeneous section, about 20 ft high but no construction records are available. Ground accelerations in the area are estimated to have been about 0.15g (Seed, 1968).

Overall Summary

It is apparent from the reports presented in the previous pages that earth dams have fared both well and poorly when subjected to earthquake ground motions. An attempt to summarize all of the available data is presented in Tables 15, 16, and 17. Table 15 lists known failures and damages to earth dams due to earthquakes in the United States; Table 16 lists failures and damages to earth dams due to earthquakes in Japan; and

Table 14: PERFORMANCE OF HYDRAULIC FILL DAMS IN RUSSIA

1944	Boz'suiskaya Dam	Fill dumped through water	$a_{max} \simeq 0.17g$	About 1 ft settlement
1957	Kairakkumskaya Dam	Hydraulic fill	$a_{max} \simeq 0.12g$	No damage
1957	Minguechaurskaya Dam	Hydraulic fill	$a_{max} \simeq 0.10g$	No damage

Table 15: DAMS KNOWN TO HAVE FAILED BY EARTHQUAKE ACTION IN NORTH AMERICA

Volcano Lake Dyke	Mexico	1915	Loose foundation soils probably liquefied
Sheffield Dam	California	1925	Liquefaction of loose silty sand near base ($a_{max} \simeq 0.2g$)
Private Dam	California	1952	
Coleman Dam	Nevada	1954	Near epicentral region of Mag. 6.6 Eq.
Rogers Dam	Nevada	1964	
Private Dam	Anchorage	1964	

<u>Dams known to have major damage due to earthquake action in North America</u>

Upper San Fernando	California	1971	5' downstream slide in hydraulic sand fill; $a_{max} \simeq 0.55g$
Lower San Fernando	California	1971	Major upstream slope slide in hydraulic sand fill due to liquefaction; $a_{max} \simeq 0.55g$
Hebgen Dam	Nevada	1959	Slumping and cracking of embankment; $a_{max} \simeq 0.7g$

Table 16: DAMS KNOWN TO HAVE SUFFERED SLIDE DAMAGE DURING EARTHQUAKES IN JAPAN

Earthquake	Maximum Acceleration	No. of dam failures	No. of dams with embankment slides	Height of embankments	Nature of embankment or foundation soils
Ojika (1939)	0.3 to 0.4g	12	40	10 to 50 ft approx.	Sandy soils
Tokachi-Oki (1968)	0.15 to 0.20g	10	49	10 to 40 ft approx.	Sandy volcanic soil

Other Dams Known to have Been Damaged in Japan

Kanto (1923)	
Ono Dam	$a_{max} \simeq 0.3g$. Built 1912. Many fissures 100 to 180 ft long and 2 to 10" wide extended to depth of 35 ft. One fissure was 70 ft deep. Two local slides occurred on downstream side.
Lower Murayama Dam	$a_{max} \simeq 0.25g$. Rolled fill with puddle clay core. Longitudinal cracks at crest. Downstream slope moved outwards. Downstream berm settled 4 ft and moved downstream about 6 with heaving of 2 ft near toe.
Upper Murayama Dam	Under construction. $a_{max} \simeq 0.25g$. Deep crack 1" wide formed in embankment.
Fukui (1948)	
Hosogori Embankment	$a_{max} \simeq 0.45$. Silty clay embankment on soft organic silts. Failure due to base spreading and settlement of embankment into soil - nearby houses also settled about 3 ft into foundation.

Table 17: DAMS KNOWN TO HAVE ADEQUATE PERFORMANCE DURING EARTHQUAKES

Rockfill	- Miboro Dam, Japan (420' high) - no damage from M = 7 @ 10 km producing $a_{max} \simeq 0.6$. Cogoti Dam - concrete faced rockfill, 275' high; $a_{max} \simeq 0.25$ to 0.5g; 15" crest settlement; some sliding on downstream face @ 1 on 1.8.
Clay Dams	- 31 earth dams of clay, sandy clay or clayey sand had negligible damage at shaking levels of 0.25 to 0.8g in San Francisco earthquake (1906). At least 12 irrigation dams constructed of clay soils were undamaged at shaking levels of 0.4 to 0.5g in Ojika earthquake (1948); sand dams had failures at lower shaking levels.
Rolled earth dams	- 25 rolled earth fill dams with shaking levels ranging from 0.2 to 0.4g had no damage in San Fernando earthquake (1971).
Hydraulic fill dams	- 3 hydraulic fill dams had no damage at shaking levels of about 0.2g in San Fernando earthquake (1971). 3 hydraulic fill dams in Russia had no failures at shaking levels of 0.10 to 0.17g approx. 2 hydraulic fill dams had minor damage at shaking levels of 0.12 to 0.18g in Kern County earthquake (1952).
Miscellaneous dams	- 50 dams built between 760 and 1944 with heights ranging from 50 to 130 ft had no damage at shaking levels of 0.08 to 0.25g in Nankai earthquake in Japan (1944).

Table 17 lists the many cases where no failures or damage have occurred during major earthquakes in the United States, Japan, South America, and Russia.

A careful review of the experience presented in these tables would seem to lead to the following conclusions:

1. Hydraulic fill dams have been found to be vulnerable to failures under unfavorable conditions and one of the particular unfavorable conditions is the shaking produced by strong earthquakes.
2. Many hydraulic fill dams, however, have performed well for many years and when they are built with reasonable slopes on good foundations they can survive moderately strong shaking--say up to about 0.2g from Magnitude 6-1/2 to 7 earthquakes with no harmfull effects.
3. Virtually any well-built dam can withstand moderate earthquake shaking, say with peak accelerations of about 0.2g and more, with no detrimental effects.
4. Dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking ranging from 0.35 to 0.8g from a magnitude 8-1/4 earthquake with no apparent damage.
5. Two rockfill dams have withstood strong shaking with no significant damage and if the rockfill is kept dry by means of a concrete facing they should be able to withstand extremely strong shaking with only small deformations.
6. Since there is ample field evidence that well-built dams can withstand moderate shaking with peak accelerations up to at least 0.2g with no harmful effects, we should not waste our time and money analyzing this type of problem--rather we should concentrate our efforts on those dams likely to present problems either because of strong shaking or because they incorporate large bodies of cohesionless materials (usually sands)

which, if saturated, may lose most of their strength during earthquake shaking and thereby lead to undesirable movements.

8. For dams constructed of saturated cohesionless soils and subjected to strong shaking, a primary cause of damage or failure is the build-up of pore water pressures in the embankment and the possible loss of strength which may accrue as a result of these pore pressures. Great caution is required in attempting to predict this type of failure by psuedo-static analyses, and dynamic analysis techniques seem to provide a more reliable basis for evaluating field performance.

A knowledge of field performance data of this type can provide a valuable supplement to analytical studies in the final assessment of the seismic stability of an earth dam and in some cases can eliminate entirely the need for analytical studies. In fact such knowledge of past performance, combined with guidance provided by dynamic analyses when appropriate, and the application of good judgment are the tools required to reach final decisions on the seismic stability of dams at the present time; with the aid of such information, it should indeed be possible to provide a higher degree of safety of dams against the damaging effects of earthquakes than ever before possible.

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