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Model Parametric Studies of the Earthquake Response of the Embankment Dam

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ABSTRACT: Several centrifuge model dams were tested with the conditions similar to those of O'Neill Forebay Dam, California to examine the structural behavior under seismic excitation. A few experiments were carried out to study the performance of the dam, whose geometry was still the same as O'Neill Dam, but the soil properties were altered by adding some gravel or increasing the compaction density. The results of either test clearly demonstrated a stiffer response to seismic excitation than the model with the original prototype material properties. Characteristics of O'Neill Forebay Dam with different structural modifications were examined. These included construction of a berm at the downstream side of the model dam, which has been proposed and designed recently for the prototype dam by the Bureau of Reclamation personnel as a part of dam rehabilitation program to strengthen the structure, as well as addition of a rip-rap layer on downstream. The earthquake simulation tests conducted on these model dams with the various structural modifications suggested that the performance of the dam with any of above-mentioned reinforcement was some what stiffer than that of the current O'Neill Dam configuration. The response of the structure under various excitation intensities was also examined by increasing the magnitude of the input excitation but keeping the same frequency contents. Some models were tested with the embankment overlaying on a rigid base, while the others included an alluvium foundation underneath the dam.

INTRODUCTION

The seismic stability of an earth dam greatly depends on the mechanical properties of materials, geometry of structure, type of underlying foundation, and magnitude of an earthquake. The centrifuge modeling is an excellent tool to understand the effect of each parameter. This paper presents the outcomes of a series of centrifuge tests on model embankment dams whose conditions were close to those of O'Neill Forebay Dam, California to examine the behavior under earthquake loading. O'Neill Forebay Dam is located on San Luis Creek approximately 40 miles southeast of San Jose and 12 miles west of Los Banos, California. During the Loma Prieta earthquake which occurred on October 17,1989 with an epicenter within a 50 miles radius of the O'Neill Forebay structure, acceleration data were recorded in of the dam. The motion similar to the recorded prototype base acceleration was used to excite the centrifuge models. Since dynamic magnification of base motion has been amain focus of the research, only accelerations were measured at various locations in the model.

Centrifuge testing has been widely applied to examine embankment problems under earthquake loading; Kutter and James (1989) tested on clay embankments to investigate dynamic magnification, the existence of a yield acceleration, and a delayed failure; Arulanandan et. al. (1988) examined the mechanism causing flow failure of an embankment dam with a less permeable layer resting on a more permeable layer; Lee and Schofield (1988) studied pore pressure generation and subsequent events of homogeneous sand embankment; Ketcham (1989) conducted on an embankment comprising loose, water-saturated sand by using ambient vibration and base excitation. His objectives of the experiments were to identify the fundamental shear mode of the structure and to study the contractive behavior. Astaneh (1994) examined the behavior of both homogeneous and zoned embankments with replacement fluid. He demonstrated the importance of replacement fluid for centrifuge experiment in order to achieve correct modeling. Because full scale measurements in the field are extremely difficult to perform, it is generally recognized that the centrifuge modeling technique provides convenient access to examine many geotechnical problems especially in the geotechnical earthquake engineering area. The centrifuge models can be constructed and instrumented with a minimum effort, and the testing event can be repeated fairly easily. The typical scaling relations for centrifuge modeling are summarized in Table 1.

Table 1. Scaling relations for centrifuge modeling. N is the scale factor.

Quantity	Model	Prototype
Gravity	N	1
Length	1/N	1
Strain	1	1
Stress	1	1
Force	$1/N^2$	1
Density	1	1
Acceleration	N	1
Time (dynamic)	1/N	1
Time (diffusion)	$1/N^2$	1

TESTING PROGRAM

A total of 23 tests was conducted at the 125th scale. They are labeled as Tests A through W, as shown in Table 2. These model tests were carried out in the 400 g-ton centrifuge at the University of Colorado, using the electro-hydraulic shaker. The base motions of the models were similar to the field acceleration recorded at the toe of O'Neill Dam.

MATERIAL: The soil was a natural material obtained from the construction site in the field and was the same material as O'Neill Forebay Dam. Its gradation is shown in Figure 1, however, only the portion passing No. 4 sieve was used to construct the model. It is classified as clayey sand and as SC group. Atterbergs Limits are 22 (liquid limit) and 8 (plasticity limit). The friction angle and cohesion are 30.5 and 8.8 psi, respectively.

CENTRIFUGE: All tests were conducted in the 400 g-ton centrifuge customs built for the University of Colorado. The maximum payload and g level are 2 ton and 200 g, respectively. During the full-speed flight, the top surface of the swing platform is extended at a radius of 18 ft. The machine is equipped with a 64-channel data acquisition system with a 5 millie-volt resolution. The maximum sampling rate is 10 Khz. However, in this testing program, the sampling rate of 2 Khz was used.

SHAKE TABLE: The shake table is an electro-hydraulic system with a 2-stage servo mechanism which is supplied with 3000 psi hydraulic oil. Motions of the table can be controlled fairly accurately up to 300 Hz by using the correction algorithm (Ketcham, 1989). Its force capacity is approximately 9000 lb.

CONTAINER: A container having inside dimensions of 48 in. long, 12 in. wide, and 9 in. high was constructed to fit the model dam. It is entirely made up of 6063 grade aluminum. Appropriate reinforcements were included to minimize the deflections of the container. All connection joints were sealed with a silicon sealant to achieve a watertight container.

SAMPLE PREPARATION: Besides the container, a mold that consisted of several wooden blocks was prepared to cast the model dam. By stacking the wooden blocks, the mold would form the outer shape of the model dam. The weight of soil in each layer of mold was precalculated to produce a uniform desired density. All models except models A and B were prepared by mixing the soil with 15.2 % water content and compacting in layers in the mold to produce 134 pcf unit weight. These placement conditions were targeted in order to produce O'Neill Dam's properties. The mold was removed before filling water in the reservoir. Figure 2 shows the pictures of a model embankment in the container, in-flight shake table and a part of the centrifuge.

RESULTS

Test A was conducted with the O'Neill Forebay dam's original geometry but was scaled 125 times, as shown in Figure 3. However, the soil was compacted with a slightly higher unit weight than the prototype in order to study the effect of the compaction effort during construction. The unit weight was 145 pcf. The model dam that was instrumented mainly with accelerometers was shaken with an earthquake similar to the 1989 Loma Prieta Earthquake. Figure 3 shows the motions recorded at various locations in the dam. Accelerometers acc21 and acc22 were mounted at the container's base, and were oriented parallel and perpendicular to the shaking direction, respectively. They are, therefore, considered as horizontal and vertical base accelerations to the model. Test B was also another test to examine the effect of material properties. In this test model, 10 percent gravel, ranging from 0.25 to 0.5 inch diameter, by weight was added to the soil passing No. 4 sieve and the mixture was compacted to yield 145 pcf. in the model preparation. The geometry was the same as the prototype.

Tests C, D, E, F, and G were conducted on the models constructed with the prototype's geometry and placement density (135 pcf). They were excited with the earthquakes having similar frequency contents but different magnitudes. This was to study the structure's response due to shaking intensity. Among them, the three samples (C, D, and E) were newly constructed models, but the others had been excited in the previous test. From the physical evidence and measurements from shaking experiments, it appeared that the model structures were not degraded as a result of previous shaking, and hence their properties were assumed to be similar to those of newly constructed samples. In Tests H, I, and J, the dam was reinforced with a berm at the downstream side. The berm was

Table 2. Test Program

Test	Embankment's Material Conditions	Model Configuration	Peak base acceleration	RMS base acceleration
A	-4 soil, 145 pcf high compaction	Embankment only	6.34	1.41
B	-4 soil, 145 pcf 10 % gravel	Embankment only	7.04	1.90
C	-4 soil, 134 pcf	Embankment only	7.51	2.19
D	-4 soil, 134 pcf	Embankment only	8.16	2.39
E	-4 soil, 134 pcf	Embankment only	9.75	2.72
F	-4 soil, 134 pcf	Embankment only	10.74	3.02
G	-4 soil, 134 pcf	Embankment only	12.60	2.96
H	-4 soil, 134 pcf	Embankment 1/2 berm	9.76	2.29
I	-4 soil, 134 pcf	Embankment 1/2 berm	9.76	2.81
J	-4 soil, 134 pcf	Embankment 1/2 berm	10.74	2.76
K	-4 soil, 134 pcf	Embankment full berm	6.44	1.11
L	-4 soil, 134 pcf	Embankment full berm	9.18	2.36
M	-4 soil, 134 pcf	Embankment full berm	9.48	2.29
N	-4 soil, 134 pcf	Embankment full berm	11.14	3.01
O	-4 soil, 134 pcf	Embankment on foundation	27.2	2.78
P	-4 soil, 134 pcf	Embankment on foundation	10.94	2.12
Q	-4 soil, 134 pcf	Embankment on foundation	25.78	5.49
R	-4 soil, 134 pcf	Embankment /w berm on foundation	9.88	2.02
S	-4 soil, 134 pcf	Embankment /w rip rap on foundation	19.44	3.80
T	-4 soil, 134 pcf	Embankment /w rip rap on foundation	24.60	6.51
U	-4 soil, 134 pcf	Embankment /w rip rap on foundation	13.86	2.88
V	-4 soil, 134 pcf	Embankment /w rip rap on foundation	18.36	4.34
W	-4 soil, 134 pcf	Embankment /w rip rap on foundation	25.0	6.61

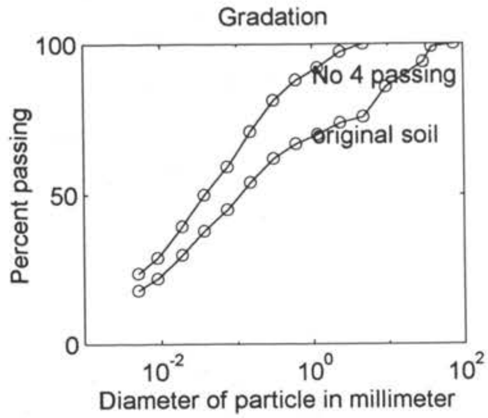

Figure 1. Gradation of soil

5.5 inch wide, as illustrated in Table 2, and was only one half the proposed size by the Bureau of Reclamation in terms of the 125th model scale. The material and placement density of the berm were the same as the main embankment. Differences among these tests were the levels of shaking intensities. In Tests K, L, M, and N, the model embankments were reinforced with an 11-inch wide berm at the down stream side, which was the full width of the proposed berm (See Table 2). Both the embankment and berm were compacted to achieve 134 pcf wet density, and were excited with different intensities.

Tests O, P, and Q were conducted with the model embankment overlaying a layer of foundation. The foundation was 2.75 in. deep and was prepared with the same soil as the embankment Again, the motions obtained from shaking with different intensities were examined. Test R model consisted of the embankment, downstream berm, and foundation. The berm was the full width of the proposed structure (11 in. wide), and the foundation was 2.75 in. deep. The model illustration can be seen in Table 2. All components of the model were compacted with the same soil to yield the 134 pcf unit weight. In this category, only one experiment was performed. In Tests S and T, the model embankment was constructed on the foundation, and the dam was covered with a rip rap blanket at the down stream side. The rip rap material consisted of gravel ranging from 0.25 to 0.5 inch diameter. It extended from the dam toe to the crest, and was parallel to the down stream slope forming 3 inch thick blanket, as shown in Table 2. The shaking intensity of Test T was higher than that of Test S. In Tests U, V and W, the rip rap face formed an angle much gentler than the dam's down steam slope making more reinforcement at the toe than the crest. But it also extended from the toe to the crest, as shown in Table 2. The shaking intensities were successively higher in those tests also.

GROUND AMPLIFICATION

The performance of the model embankments under different configurations, materials and loading conditions is examined by using the measurements of the accelerometers oriented in horizontal direction. Since those accelerations are irregular time histories having frequencies up to 500 Hz, as shown in the earlier plots, it is very difficult to compare one trace to others. Thus, RMS acceleration is used to define a single acceleration history. The RMS acceleration which is an average value is defined as:

$$RMS = \left[\frac{1}{T}\int a^2 dt\right]^{1/2}$$

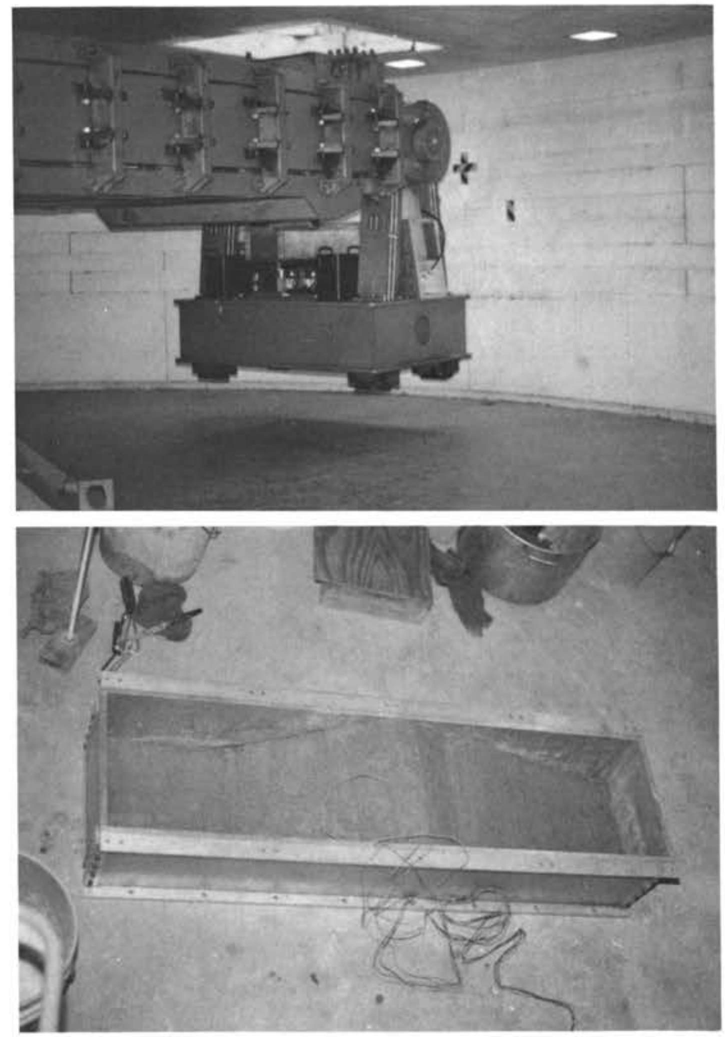

Figure 2. Pictures of the model embankment in the container, in-flight shake table, and a part of the centrifuge

where,
RMS=root mean square acceleration
T = duration of the earthquake
a = acceleration

Values of RMS accelerations were evaluated for the same duration of records (0.47 sec.) and were plotted against the heights of measurement points to provide acceleration profile. The profile describes how the motions are transmitted through the soil and is a good indication of amplification or attenuation. Figure 4-a shows the comparison of amplification profiles between Tests A and C. The model in Test A was compacted with slightly higher than Test C; the unit weights of Test A and C were 145 and 134 pcf, respectively. For this comparison, the results of Test C were selected among the other tests having same test conditions (i.e., Tests C, D, E, F, and G) because its input level of excitation was close to Test A. The figure suggests that the model with high compaction effort (Test A) exhibits less amplification, and hence stiffer response than the other. Figure 4-b is the plot comparing Tests B and C. Test B contained 10 percent of gravel in the soil while Test C did not. As in the previous comparison, Test C was chosen among the others (Tests C, D, E, F, and G) for having similar input shaking intensity to Test B. It can be seen that the model with gravel mixtures shows stiffer response (less amplification) than that with pure No 4 sieve soil.

Tests C, D, E, F, and G were the experiments with the same geometry and soil conditions, but difference shaking intensity. The shaking level of each test is indicated by the RMS value at the 0

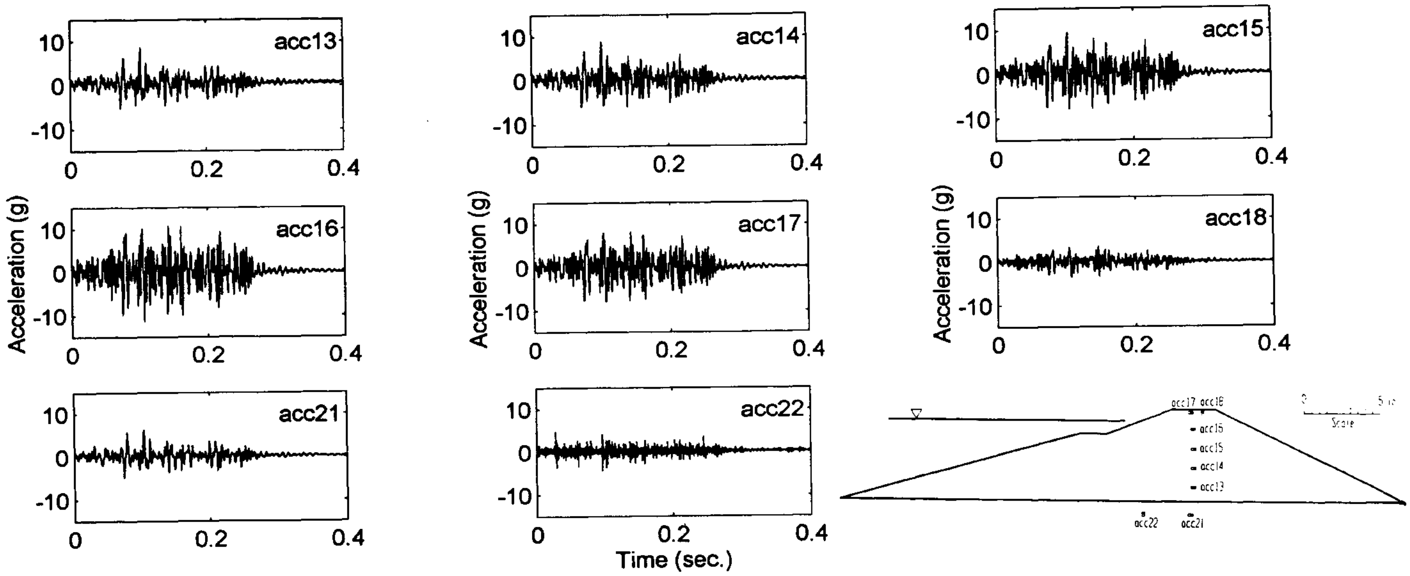

Figure 3. Transducer locations and measurements of Test A

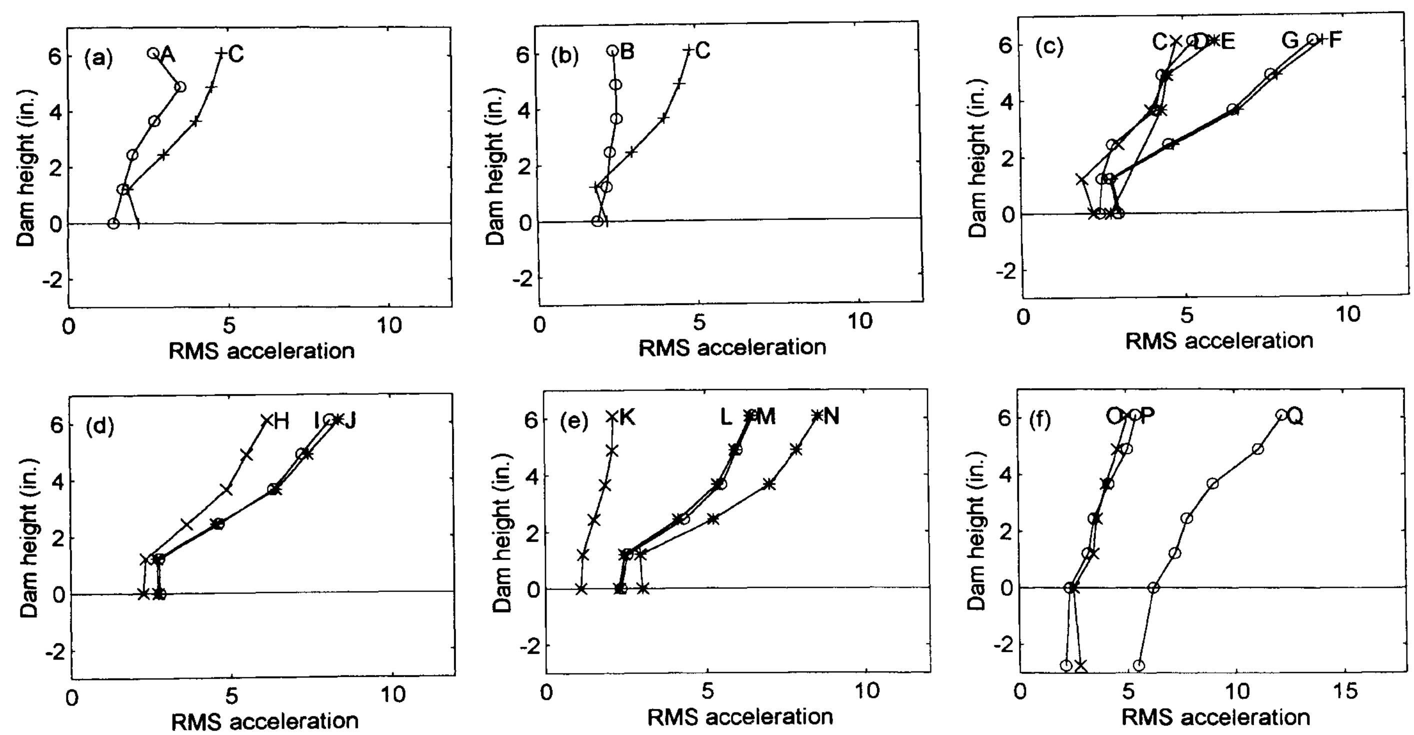

Figure 4. Amplification profiles. Continue to next page

depth (at the toe) in Figure 4-c. Although the range of variations in shaking intensities within this group of tests is small, the general response of the embankment shows a tendency of large amplification for the intense shaking. The comparison of Tests H, I, and J is presented in Figure 4-d illustrating the effects of earthquake magnitude on the embankment that was reinforced with a 5.5-inch wide berm in the downstream. The excitation levels of Tests I and J are similar, and the responses are almost identical illustrating the repeatability and credibility of the experiments. Comparing Test H to either Test I or Test J, it is noted that the motions transmitted from the intense shaking are amplified significantly. Figure 4-e compares Tests K, L, M and N where the embankment was reinforced with a 10-inch wide berm in the downstream. The plot apparently shows the effects of earthquake intensity very clearly. When the intensity is small, the acceleration profile is almost a straight line; as the intensity increases, the profile begins to show a curvature indicating much higher amplification near the crest. Figures 4-f, 4-g, and 4-h also show the effect of earthquake intensities for different model configurations. Generally the large magnitude earthquake yields much higher amplification than the small earthquake.

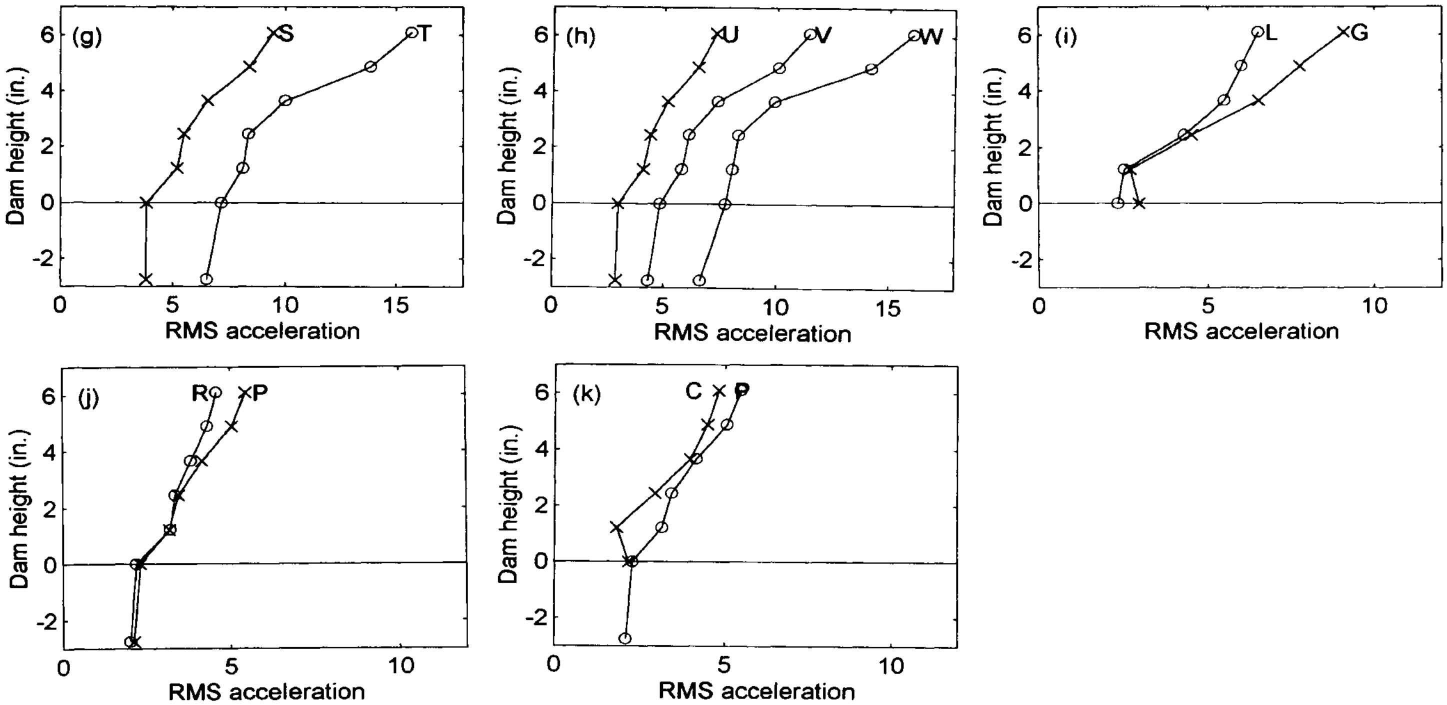

Figure 4 (continued). Amplification profiles

Tests G and L are compared in Figure 4-i to show the effects of the presence of the berm. These two models were the embankments reinforced with no berm and a 10-inch wide berm, respectively; they were excited with very similar earthquakes. Test L (with the berm) shows somewhat stiffer response than Test G (without berm). Figure 4-j also compares the effect of reinforcing berm with an underlying foundation layer. Like in the previous comparison, the model with a berm shows somewhat stronger behavior than that without berm.

Test C (embankment without a foundation) and Test P (embankment with a foundation) are compared in Figure 4-k. The existence of a foundation underlying the embankment clearly makes the difference between the two due to the interaction between embankment and foundation. Intuitively, the model with a foundation would represent a better simulation of the field problem due to the completeness of all the structure's components in the model.

CONCLUSIONS

A total of 23 tests was conducted in this parametric study. It was found that the embankment constructed with the unit weight of 145 pcf showed stronger response than that with 135 pcf due to an increase in shear stiffness of the structure. Adding some gravel in the soil during the embankment construction also improved the dynamic behavior. Generally, the embankments with a reinforcing berm were somewhat stronger than those without berm. From the responses of the structures excited with different earthquake intensities, amplification of motions was generally observed in all tests and very large amplification occurred in the intense shaking test. The effects of this earthquake intensity were very prominent near the crest.

ACKNOWLEDGEMENT

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