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Ground Motion Amplification from Vertical Propagation of Earthquake Waves

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GROUND MOTION AMPLIFICATION FROM VERTICAL PROPAGATION OF EARTHQUAKE WAVES

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ABSTRACT

The paper is directed to follow a procedure where seismic responses are identified and recommendations to proceed with their applications are made quickly and economically, before engaging in a complex analysis. It is not intended for selecting design soil parameters or earthquake parameters for the final design of any specific dam. A selected earthquake accelerogram will produce response accelerations and strains that will cover a significant variety of accelerograms in terms of severity of earthquake damage to an earth dam. Scaling-up and scaling-down the selected outcrop motion broadens the spectrum of seismic response analyses using simple and effective SHAKE program for making decision on the need for further more sophisticated analyses, particularly when the 1-D response analysis shows potential problems. It is inappropriate to base the judgment of final seismic response of dam on 1-D analysis,. The traditional dynamic modulus reduction curves have been shown to be appropriate up to one percent strain. The analysis performed in this paper shows the maximum strain to be smaller than 0.5%. The Mexico City (MC) clay is used to demonstrate different soil types do exist in nature and, in some situation, can show quite different responses in different intensity seismic events. First, hypothetical soil deposits are used in evaluating factors affecting PHA variation in a soil deposit, and then, the findings are referenced in the seismic response of an embankment dam in the west coast of the United States. The shear wave velocities of this embankment dam were carefully selected, using educated judgment, to represent a compacted embankment.

INTRODUCTION

During an earthquake, seismic waves propagate from its hypocenter to a given site through rocks and then vertically through soil deposits. The local site effects on the ground motion alteration can be assessed using one-dimensional (1-D), two-dimensional (2-D) or three-dimensional (3-D) analysis depending on the complexity of the site. Depending on soil properties, shape and thickness of soil deposits and ground motion characteristics, the seismic wave can either attenuate or amplify. The two most distinctive cases of motion amplification were observed in the 1985 Michoacán earthquake at Mexico City (Chiang and Chang, 1994; Chang and Chiang, 1994) and the 1989 Loma Prieta earthquake Emeryville record at Oakland, California. After a long distance travel from the rupture sites, the ground motions were significantly weakened by the time they reached both cities.
Then, these severely weakened motions propagate through the very sensitive Mexico City clay and the less sensitive Oakland Bay mud, the motions were significantly amplified (as measured in seismographs) when reached the ground surface, strong motion durations lengthened, predominant frequencies lowered and high frequency motion filtered.

The amplification from the bedrock to ground surface at Mexico City was also affected by the distance from the edge of the ancient lake basin (Chang and Chiang, 1994). The above studies indicated that both 1-D analysis for slight bedrock inclination and 2-D analyses for steeper bedrock inclination can reasonably estimate the surficial ground motion intensity. For horizontal soil deposits, Aubeny (1984) used Fourier Transformation to evaluate the surficial motion amplification characteristics and found that, for a 3-layer system with shear-wave velocities of 600, 1000 and 1500 fps, respectively, the property and the location of the softest layer significantly affects the surficial motion characteristics. When a soft layer overlies a stiff layer, ground motion typically amplifies and the natural frequency of the layered system is much higher than that of a homogeneous deposit of soft soil. When the soft soil underlies the stiff soil, the motion attenuates and the natural frequency is of the same order of magnitude as the homogeneous soft soil deposit.

This paper illustrates the ground-motion alteration when the shear wave propagates upwards through a hypothetical soil deposit with nearly horizontal layers having three different shear-wave velocities of 500, 800 and 1100 fps representing soft, medium and hard soils, respectively. The SHAKE 2000, 1-D shear-wave propagation program was used. Typical shear-modulus reduction and damping factor curves (Seed and Idriss, 1970) were used to represent the nonlinear relationship between the dynamic properties and shear strains.

The analysis includes hypothetical layered deposits of different thicknesses for differing layers of sand and clay. An embankment dam located on the west coast of the United States was also analyzed in this case study. Dynamic properties of the embankment materials were calculated using selected shear-wave velocities and typical nonlinear relationships between shear modulus and shear strain. Finally, the results obtained from the one-dimensional wave analysis can be used as a tool for assessing the performance of earth dams under seismic shaking or to determine if further more sophisticated study is needed.

PROPERTIES OF SOILS

The dynamic soil properties including six shear modulus reduction curves and six damping ratio curves are shown in Figs. 1 and 2. The shear modulus curves for sand also reflected the effect of confining pressures, and were used accordingly. The clayey soils use: the “Clay PI=10” for Modulus reduction and “Soil PI=15” for damping. The references of the dynamic properties of the MC Clay are also included in Fig. 1 and Fig. 2.

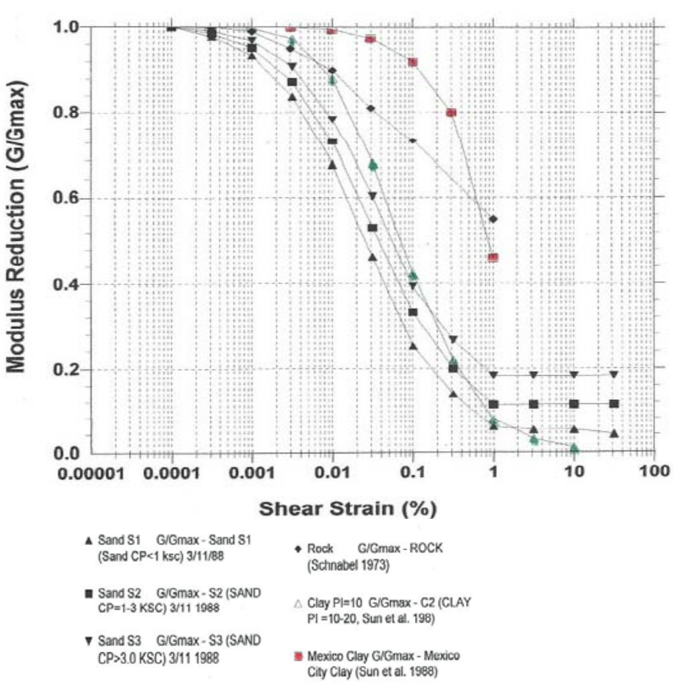

Fig. 1. Shear modulus reduction Curves (SHAKE2000)

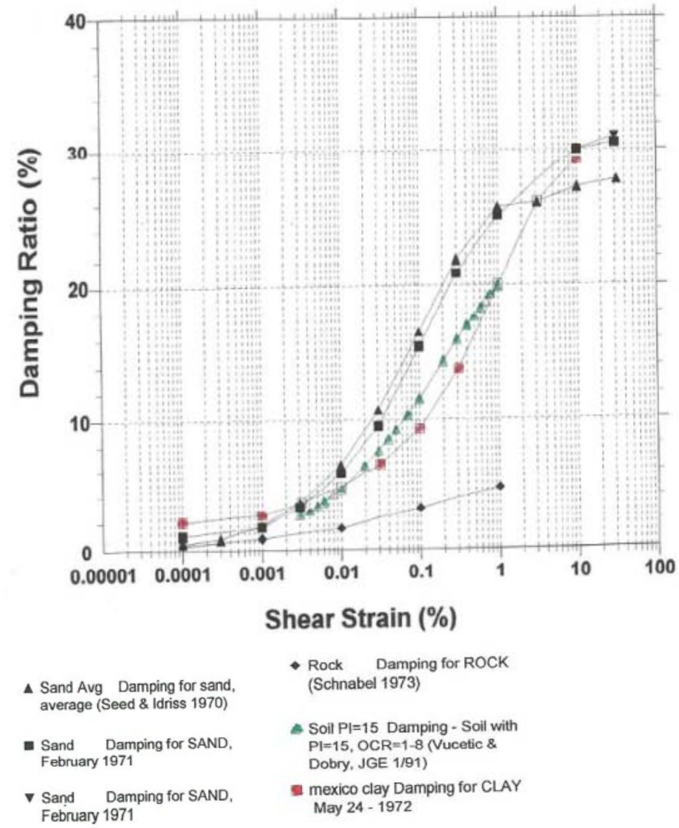

Fig. 2 Damping Ratio curves (SHAKE2000)

Analyses were first performed on hypothetical soil deposits with layers of soft, medium and hard soils with the shear-wave velocities selected as 500, 800 and 1100 fps, respectively, and were followed by the analysis of an embankment from the west coast of the United States. The unit weight of the bedrock is assumed 150 pcf, and the soil 120 pcf for the hypothetical deposits.

1 – D SHEAR-WAVE PROPAGATION THROUGH HYPOTHETICAL STRATIFIED SANDY SOIL DEPOSITS

The 1989 Loma Prieta earthquake time history (Idriss, 1991), shown in Fig. 3, was used in all SHAKE analyses, unless otherwise specified. It has a peak horizontal acceleration (PHA) of 0.24g.

An extensive analysis program is carried out on the hypothetical soil deposits to study the effect of depositional sequences and thicknesses of soft, medium hard and hard soils on the ground motion amplification, expressed as the ratio between the calculated peak horizontal acceleration (PHA) anywhere in a soil deposit and the PHA of the outcrop earthquake ground motion.

The Peak Horizontal Acceleration Ratio is referred to as PHAR, the ground motion is amplified when PHAR is greater than 1.0 and is attenuated when PHAR is less than 1.0.

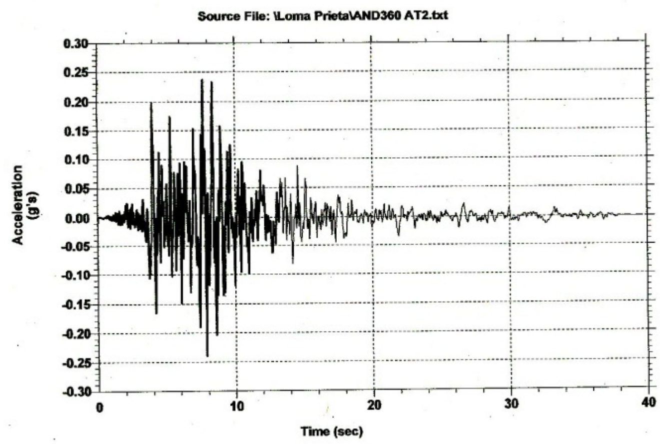

Fig. 3. Accelerogram from the 1989 Loma Prieta Earthquake

The first group of analyses covered a hypothetical soil deposit with ten 10-foot horizontal layers of sandy soils with shear wave velocities, Vs, of 500, 800 and 1100 fps (feet per second) for loose, medium dense and dense sands, respectively, and three rock Vs velocities: 2500, 5000 and 10000 fps. Table 1 summarizes the results of 18 analyses devoted to studying the effects of differing rock and sand velocities on ground motion amplification. Results showed that, within the range of rock velocities used in the analyses, the bedrock velocity exerted significant effects on surficial ground motion amplification for 90-ft of homogeneous sand deposits with greater differences occurring at higher Vs like 1100 psf. No difference for Vs soil with 500 psf and small difference for Vs soil with 800 psf, as shown in Figure 4. Also shown in Table 1, the surficial PHA of dense and medium sands increases and the motion amplifies, with PHAR ranging from 1.16 to 1.79 and the PHA of loose sand decreases and the ground motion attenuates with PHAR of around 0.67. At the same rock Vs, the sand deposit with linearly increasing shear wave velocities from 500 fps to 1100 fps with depth experiences amplification with PHAR of 2.08, with decreasing velocities 1100 fps to 500 fps attenuation with PHAR of 0.67, with increasing velocities from 500 fps to 1100 fps but with insertion of two loose sand layers directly above the bedrock attenuation with PHAR of 0.67.

The Fig. 4 and Fig. 5 show the relationships between PHAR and Vs. Figure 4 shows the increase of PHAR as the Vs of the bedrock increases. Fig. 5 shows that, at the rock Vs of 10,000 fps, PHAR increases with the increase in Vs for all three types of soils as Vs varies from 500 to 800 fps in an ascending from sand, lean clay to the Mexico City clay. The Mexico City clay shows the decrease in PHAR when its Vs exceeds 800 fps.

Table 1. Analysis of 100 ft sand deposit in ten 10-ft layers

Vs (fps) Bedrock			Vs (fps) Soil						Surface PHA (g)	PHAR
10000	5000	2500	1100	800	500	500 to 1300	1300 to 500	500 to 1100		
X			X						0.43	1.79
X				X					0.31	1.29
X					X				0.16	0.67
	X		**X**						**0.40**	**1.67**
	X			**X**					**0.30**	**1.25**
	X				**X**				**0.16**	**0.67**
		X	X						0.35	1.46
		X		X					0.28	1.16
		X			X				0.16	0.66
X						**X**			**0.50**	**2.08**
X							**X**		**0.16**	**0.67**
X								**X**	**0.16**	**0.67**
	X					X			0.47	1.94
	X						X		0.16	0.66
	X							X	0.16	0.67
		X				**X**			**0.41**	**1.69**
		X					**X**		**0.16**	**0.65**
		X						**X**	**0.16**	**0.65**

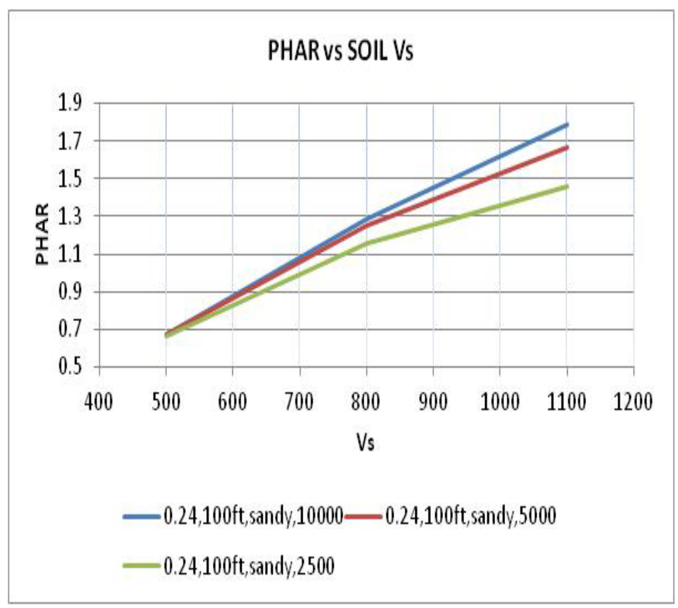

Fig. 4. Includes PHA=0.24g, 90 ft thick model, type of soil and Vs bedrock. Data taken from Table 1

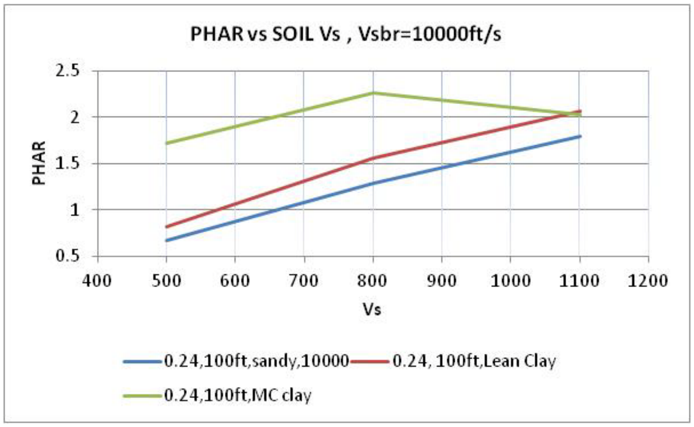

Fig. 5 Effects of Vs on the PHAR

The second group of analyses was performed on three 30-ft thick sand layers of different stacking orders: (1) loose sand [S] with Vs of 500 fps, (2) medium dense sand [M]) with Vs of 800 fps, (3) dense sand [H] with Vs of 1100 fps and the bedrock with Vs of 10000 fps, as shown in Table 2. Table 2 shows only attenuation with PHAR ranging from 0.40 with loose sand right above the bedrock to 1.00 with the loose sand on top.

Table 2. PHA values for three stratified 45 ft layers

Case on Fig 6	Layers			PHA (g)	PHAR
	Top	Medium	Bottom		
1	H	S	M	0.11	0.45
2	H	M	S	0.10	0.40
3	M	H	S	0.10	0.40
4	M	S	H	0.13	0.53
5	S	M	H	0.24	0.99
6	S	H	M	0.24	1.00

Figure 6 shows the maximum strains in the deposits of different stacking sequences: (1) Case 2 and Case 3 (having the S layer immediately above the BR) show the largest strains above the BR in the deposit; (2) Case 5 and Case 6 with S Layers on top show the largest strains of all cases analyzed, and (3) Case 1 and Case 4 with S layer between H and M layers show strains between those explained in (1) and (2).

In the third group, analyses were performed on 90-ft and 135-ft thick deposits of uniform sand and clay, respectively with Vs of 500 fps (S), 800 fps (M) and 1100 fps (H), respectively.

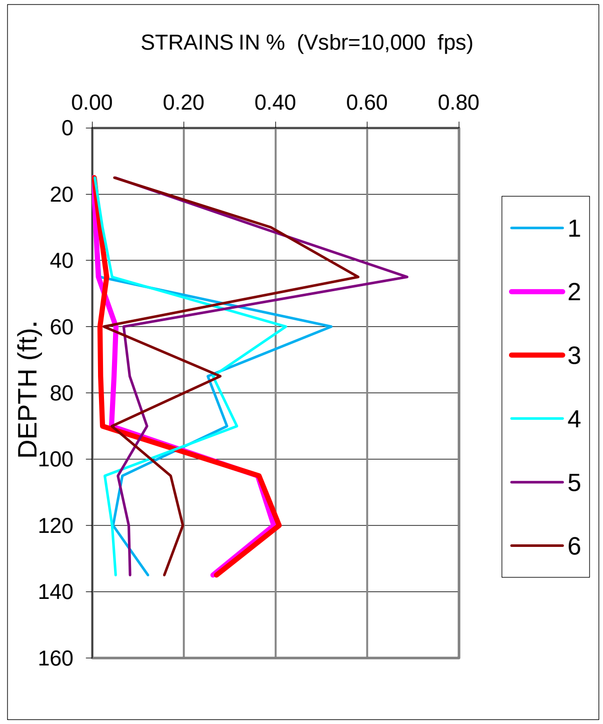

Fig. 6 Strains vs Depths for Cases 1 to 6 shown on Table 2

Results are shown in Table 3. It was found that, in general, the 90-ft deposit yielded higher PHA than 135-ft deposit, whether it is clay/sand and the clay deposit yielded higher PHA than sand. The results, summarized in Fig. 7, show that, in a soft soil deposit, the ground motion tends to attenuate and, in a hard soil deposit, it amplifies, and the clay deposit allows the motion to amplify at a smaller Vs than the sand deposit.

Table 3. PHA and PHAR for 90-ft and 135-ft homogeneous sand deposits

Case	Vs	90-ft deposit		Case	135-ft deposit	
		PHA	PHAR		PHA	PHAR
7	H	0.43	2.05	10	0.31	1.28
8	M	0.31	1.29	11	0.21	0.88
9	S	0.16	0.67	12	0.14	0.60

Figure 8 shows the strains on the 90-ft model with homogeneous deposits where: (1) The general trend of the strains are in agreement with the trend in Vs values, which implies that the strains are properly computed. (2) The maximum strain for each case is located on the layer immediately above the BR, and (3) The strains at the top layer are very small compared to the strains in the lower layers.

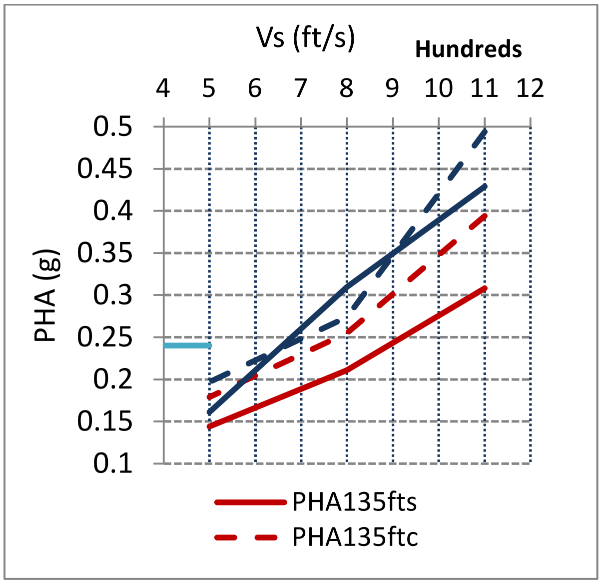

Fig. 7 PHA vs Vs for 90-ft and 135-ft models sandy (s), and clayey (c)

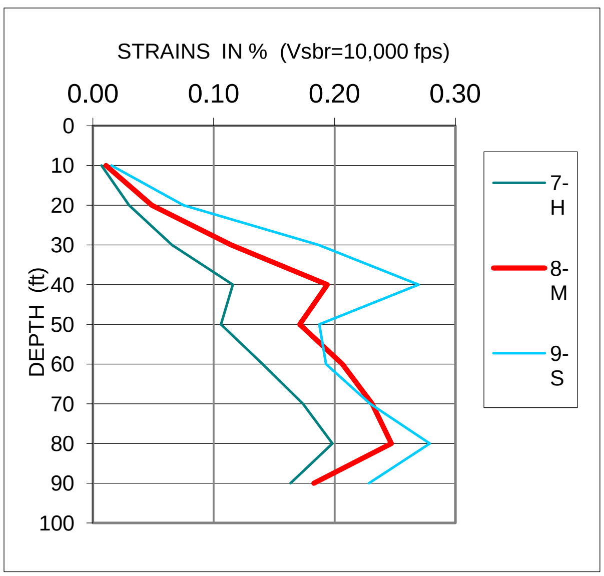

Fig. 8 Plot of Table 4 for the 90 ft high model

Fig. 9 shows the strains on the 135 ft deposit where: The strains have similar pattern as that for the 90-ft thick deposit with minor differences.

1 – D SHEAR WAVE PROPAGATION THROUGH SENSITIVE MEXICO-CITY TYPE CLAY DEPOSITS

To examine the effect of extremely sensitive clay on the ground motion amplification, the seismic wave was allowed to propagate through a Mexico City type clay deposit with the

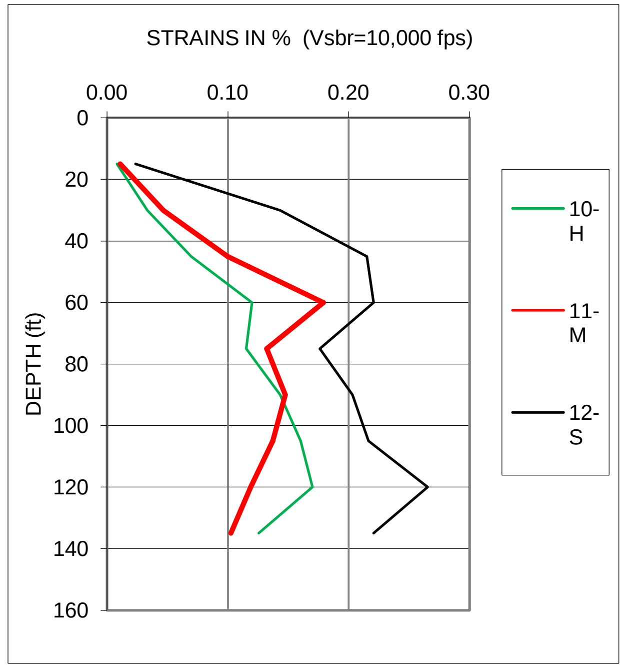

Fig. 9. Plot of Table 3 for the 135 ft high model

featured characteristics of not having significant modulus degradation until the shear strain reaches 0.1 percent and a high sensitivity as shown in Fig. 1. The Loma Prieta earthquake motion accelerogram was scaled to 0.07 and 0,40 and used in the analysis to examine the effect of ground motion intensity on the surficial PHA of different thicknesses.

Table 4 shows the surficial PHA increases from 0.32g (PHAR of 1.34) for the 15-ft thick deposit to 0.551g (PHAR 2.30) for a 150-ft deposit and then decreases to 0.271g (PHAR 1.13) for the 600-ft deposit. So PHA increases as the thickness increases, but after reaching a peak at 150 feet, it decreases and no definite trend is observed. The PHAR is sensitive to the input ground motion intensity.

Table 4. 0.24g PHA and PHAR - Deposits of MC clay

Soil above bedrock (Ft)	PHA (g)	PHAR
15	0.32	1.34
30	0.50	2.06
45	0.49	2.03
60	0.46	1.89
75	0.45	1.89
135	0.51	2.12
150	0.55	2.30
300	0.28	1.16
600	0.27	1.13

Table 5 shows the analysis result for weak intensity wave with PHA of 0.07g, the scaled down accelerogram of the Loma Prieta record. The resulting surficial motion peaked at a PHA of 0.18g or PHAR of 2.61. Table 6 presents the result using PHA of 0.40g, the scaled up accelerogram of the Loma Prieta earthquake. The clay is assumed to have V_s of 1100 fps with MC clay dynamic property curves.

Table 5. Surficial PHA and PHAR under 0.07g outcrop PHA

Layer No. and thickness (ft)	Vs (fps)	PHA (g)	PHAR
1- 15.0	800	0.18	2.61
2 - 15.0	800	0.17	2.48
3 - 15.0	800	0.16	2.25
4 - 15.0	800	0.17	2.38
5 - 15.0	800	0.16	2.21
6 - 15.0	800	0.13	1.88
7- 15.0	800	0.12	1.75
8 -15.0	800	0.10	1.43
9 - 15.0	800	0.06	0.09
Outcrop	10000	0.06	0.09

Table 6. Surficial PHA and PHAR under 0.40g outcrop PHA

Layer No. and thickness (ft)	Vs (fps)	PHA (g)	PHAR
1-15.0	800	0.76	1.90
2 - 15.0	800	0.72	1.82
3 - 15.0	800	0.67	1.68
4 - 15.0	800	0.64	1.61
5 - 15.0	800	0.59	1.48
6 - 15.0	800	0.56	1.41
7- 15.0	800	0.51	1.26
8 -15.0	800	0.41	1.02
9 - 15.0	800	0.29	0.75
Outcrop	10000	0.38	0.94

Table 6 shows the surficial PHA of 0.76g with PHAR of 1.90 under the motion with 0.40g input PHA. It is, thus, observed that a weak outcrop motion yielded a higher amplification than a strong outcrop motion, if the deposit is MC clay type.

1 – D SHEAR-WAVE PROPAGATION THROUGH HYPOTHETICAL DEPOSITS

This section investigated amplification effect when the Loma Prieta motion propagated through the uniform sand or clay deposits of two different thicknesses of 185 feet and 135 feet. The surficial PHA in Tables 7 through 10 showed: 1) the ground motion attenuated in both 185-ft deposits, sand or clay; 2) The motion attenuated in the 135-ft sand deposit, while amplified in the 135-ft clay deposit. Thus, the ground motion amplification depends on: deposit thickness, dynamic properties, and soil types and 3) the clay deposit allowed higher amplification.

Table 7. Surficial PHA and PHAR for 185-ft Clay deposit

Layer No. and thickness (ft)	Vs (fps)	PHA	PHAR
1-23.3	800	0.17	0.72
2 - 23.3	800	0.15	0.63
3 - 23.3	800	0.14	0.57
4 - 23.3	800	0.12	0.51
5 - 23.3	800	0.13	0.53
6 - 23.3	800	0.15	0.61
7- 15.0	800	0.19	0.80
8 -15.0	800	0.23	0.94
9 - 15.0	800	0.22	0.94
Outcrop	10000	0.24	1.000

Table 8. PHA and PHAR for 185-ft Sand Deposit

Layer No. and thickness (ft)	Vs (fps)	PHA (g)	PHAR
1-23.3	800	0.16	0.65
2 - 23.3	800	0.14	0.57
3 - 23.3	800	0.13	0.53
4 - 23.3	800	0.11	0.44
5 - 23.3	800	0.11	0.46
6 - 23.3	800	0.14	0.58
7- 15.0	800	0.19	0.78
8 -15.0	800	0.20	0.83
9 - 15.0	800	0.22	0.90
Outcrop	10000	0.24	1.00

Table 9. PHA and PHAR for 135 ft Clay Deposit

Layer No. and thickness (ft)	Vs (fps)	PHA	PHAR
1-15.0	800	0.26	1.07
2 - 15.0	800	0.24	0.99
3 - 15.0	800	0.20	0.87
4 - 15.0	800	0.201	0.84
5 - 15.0	800	0.16	0.65
6 - 15.0	800	0.17	0.72
7- 15.0	800	0.21	0.88
8 -15.0	800	0.24	0.98
9 - 15.0	800	0.24	1.00
Outcrop	10000	0. 24	1.00

Table 10. PHA and PHAR for 135-ft Sand Deposit

Layer No. and thickness (ft)	Vs (fps)	PHA (g)	PHAR
1-15.0	800	0.21	0.88
2 - 15.0	800	0.19	0.80
3 - 15.0	800	0.16	0.67
4 - 15.0	800	0.16	0.67
5 - 15.0	800	0.13	0.54
6 - 15.0	800	0.13	0.55
7- 15.0	800	0.17	0.71
8 -15.0	800	0.19	0.79
9 - 15.0	800	0.24	0.99
Outcrop	10000	0. 24	1.00

CASE STUDY OF AN EMBANKMENT DAM

General

Embankment dams are critical civil structures for which failure can have disastrous consequences. The characteristics of an incident ground motion and the associated responses are of paramount importance to the seismic safety and risk assessment of an embankment dam. A west coast dam is adopted in the paper as a case study for demonstrating the judgment needed when SHAKE analyses are used in its preliminary seismic safety assessment. A comprehensive subsurface exploration program was performed during its seismic safety assessment. Ground water was detected at 10 feet below the original ground surface.

Typically an old zoned embankment dam is composed of a central impervious core, protected by the upstream and downstream shells, including upstream and downstream transition filters and outer shells with increasing permeability. The filter zones are in direct contact with the core followed by shells. The core zone is usually composed of a mixture of clay, silt, sand and gravel compacted by tamping rollers in 6-inch lifts; shells are the mixture of silt, sand, cobble and boulder compacted by pneumatic tire rollers in 12-inch lifts; outer shell zones contained miscellaneous materials and compacted in 12-inch lifts travelling construction equipment.

The maximum structural height of the cross section is assumed to be 130 ft. The dam has a cut-off trench underneath the core and the filters and shells were placed directly over the soft alluvial soils. The maximum shear moduli of soils are shown in Table 11. Shear moduli are related to shear strains following typical modulus reduction curves. The shear-wave velocities of the dam selected in this study show that the embankment materials are nearly homogeneous and the alluvial deposits underlying the dam is soft. The density of the embankment materials is assumed 120 pcf. The soil shear modulus and damping ratio curves are selected from the SHAKE data bank, where the dynamic properties of sand also reflect the effect of effective overburden pressure. The dam was modeled in nine layers with thicknesses shown in Table 13, first eight layers for embankment and one layer for alluvial deposit.

Field Shear Wave Velocities

Table 11 shows the shear wave velcities measured using geopyysical seimic testing and higher Vs for the embankment materials than for the alluvial deposit even after decades of consolidation under the embankment weight. The natural alluvial deposit near and beyond the toe has even lower Vs values. This indicates potential strain and liquefaction problems under strong seismic shaking.

Table 11. Vs measurements below crest

Depth (ft)	Average Vs (fps)
0 to 70	1200
70 to 90	1100
90 to 120	1200
120 to 140	900
Bedrock	2500

Earthquake Ground Motion and SHAKE Analysis Results

Through a complex seismotectonic study during the seismic safety assessment, a magnitude 6.5 earthquake was recommended for the investigation. Fig. 3 shows the accelerogram with the peak horizontal acceleration of 0.24g from the 1989 Loma Prieta earthquake (10/18/89, 360 USGS Station 1652) selected for the SHAKE analysis. Fig. 10 shows the response acceleration-time history on the top layer. Table 12 the surface peak horizontal acceleration of 0.29g and 0.31g, when the embankment material is considered sandy or clayey, respectively and the variation of average response PHAs.

Figs. 11 to 12 show that the ground motion attenuates in the weak layer, and amplifies thereafter and Figs. 13 and 14 show higher PHA response at higher Vs. The amplified motion can lead to cracking in dam due to differential displacement and, for the dam selected in this case history study, the high motion amplification can cause the alluvial deposit to liquefy. Thus, it might be necessary to enhance the liquefaction resistance of the alluvial foundation soils. Figure 15 shows that the alluvial layer immediately above the bedrock experienced the largest strains and the strains decreased towards the top of the dam in similar fashion as shown in Tables 13 and 14.

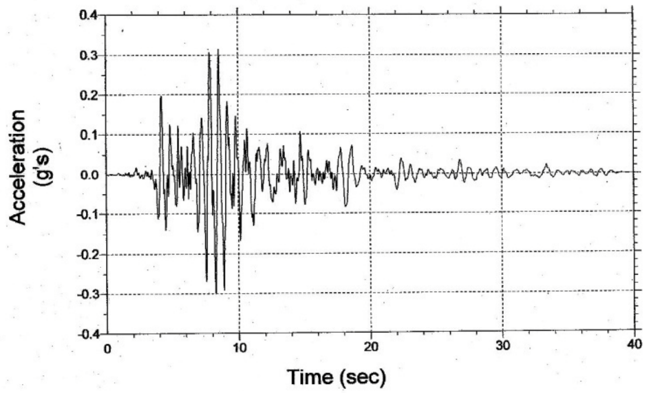

Fig. 10- Motion on the top layer of Clay model

Table 12. PHA and PHAR for 120-ft Embankment and 10-ft Clay/Sand deposits

Layer & thickness		Vs (ft/s)	PHA Sand	PHA Clay	PHAR	
					Sand	Clay
1	15	1200	0.29	0.31	1.21	1.28
2	15	1200	0.29	0.30	1.18	1.25
3	15	1200	0.26	0.28	1.08	1.18
4	15	1200	0.26	0.29	0.95	1.21
5	10	1200	0.22	0.28	0.84	1.17
6	20	1100	0.22	0.26	0.88	1.06
7	15	1100	0.21	0.23	0.78	0.94
8	15	1200	0.17	0.23	0.63	0.97
9	10	900	0.16	0.21	0.97	0.87
Outcrop		10000	0.24	0.24	1.01	1.01

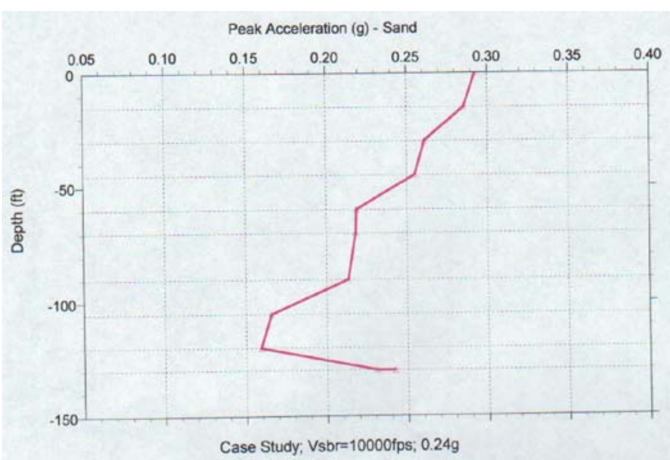

Fig. 11 Case Study – Sandy soils – PHA on layers

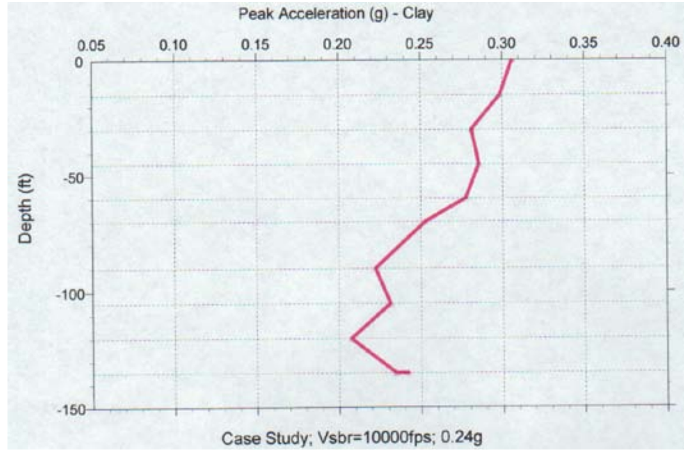

Fig. 12 Case Study – Clayey soils – PHA on layers

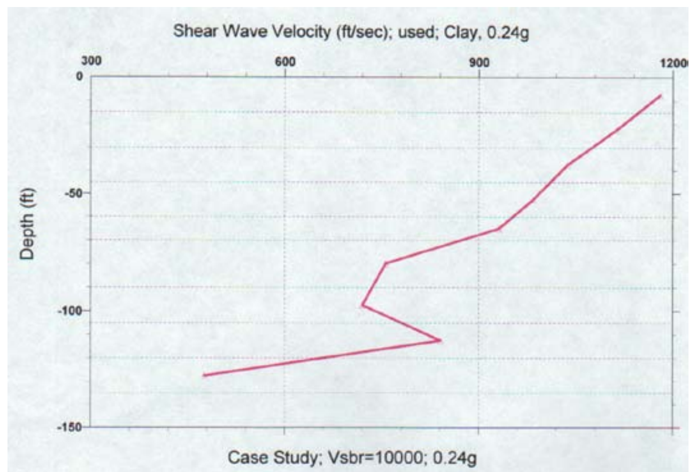

Fig 13 – Case study – Sandy Soils – Vs

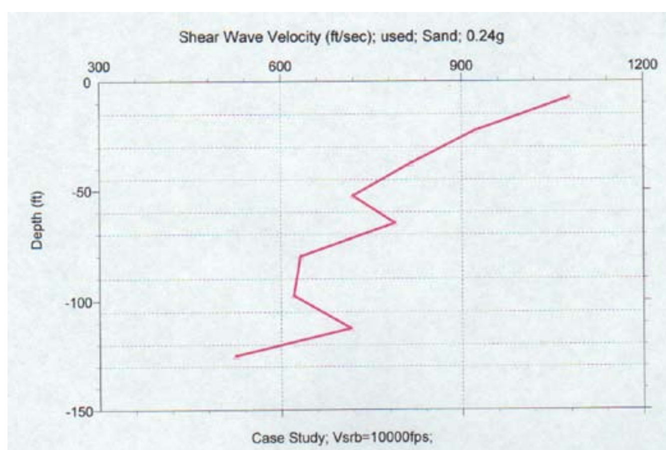

Fig. 14 Case study – Clayey soils – Vs

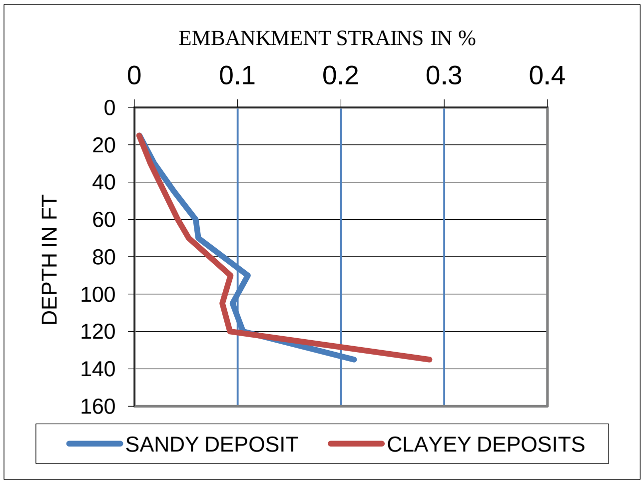

Fig. 15 Strains in the 135 ft embankment with a Vs for bedrock of 2500 fps

Table 13 PHA and PHAR for sandy embankment

DEPTH	STRAIN.	PHA	PHAR
15	0.00	0.25	1.05
30	0.02	0.24	1.00
45	0.04	0.22	0.92
60	0.06	0.21	0.88
70	0.06	0.19	0.80
90	0.11	0.17	0.71
105	0.10	0.16	0.67
120	0.11	0.12	0.50
135	0.21	0.19	0.80

Table 14. PHA and PHAR for clayey embankment

DEPTH	STRAIN.	PHA	PHAR
15	0.00	0.28	1.17
30	0.02	0.27	1.13
45	0.03	0.25	1.05
60	0.04	0.24	1.00
70	0.05	0.23	0.96
90	0.09	0.22	0.92
105	0.09	0.20	0.84
120	0.09	0.21	0.88
135	0.29	0.18	0.75

Questions pertaining to the accuracy of SHAKE analysis for seismic response of an embankment dam are frequently raised. The colleagues from the Itasca Consulting (2011) performed seismic response analyses using both SHAKE and FLAC for an embankment dam with nonlinear dynamic properties and found that SHAKE analysis yielded results that were in close agreement with the results from FLAC analyses, as shown in Figure 16.

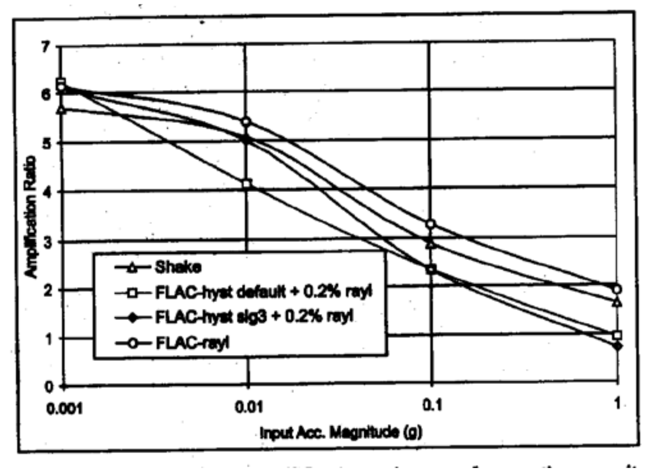

Fig. 16 PHAR vs Input acceleration magnitude in g (Itasca Consulting, 2011)

CONCLUSIONS

This article addresses the factors affecting the ground motion amplification during its upward propagation in hypothetical deposits and the selected case embankment dam. The findings are briefed as follows:

Motion through Hypothetical Deposits

The findings are: 1) The motion amplifies as it propagates through a deposit with increasing soil stiffness with depth and attenuates through a deposit with decreasing soil stiffness with depth; 2) As a motion propagates through a soft soil deposit its high frequency portion is filtered out, and its strong motion duration lengthened; 3) In thin alluvial deposits with low shear wave velocities, the ground motion is significantly amplified with the implication of potential for soil liquefaction and embankment cracking.

Case Study of Selected Earth Dam

In this selected zoned earth dam, the findings include: 1) The motion is amplified with peak horizontal acceleration amplification factor of 1.21 when embankment is composed of sandy soils and 1.28 when clayey soils and the actual amplification factor lies in between the two values; 2) The amplification in the thin alluvial foundation soils is much higher than the embankment materials with implication of potential foundation liquefaction or embankment cracking; 3) Finding from SHAKE analyses should be used only as a guide for the decision pertaining to the necessity of more rigorous two or three dimensional analyses.

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