Ground Motions and Liquefaction Potential

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Fifth International Conference on
Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
and Symposium in Honor of Professor I.M. Idriss
May 24-29, 2010 • San Diego, California

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ABSTRACT

Strong earthquakes have the potential to produce liquefaction of saturated and loose soils or to produce shear strength loss on sensitive clays, which may produce embankment failure with uncontrolled release of the reservoir. Earthfill Dam 101 has been classified as a high risk dam and this study attempts to determine the extent of the risk. It retains a 60,000 acre-feet reservoir with population downstream of the embankment.

The analysis of the earthquake through a bracketed accelerogram provides insight into how the embankment will respond to the seismic loading and to the soils properties. In the case study, the peak response accelerations, the frequency content and the soil properties were the dominant factors in predicting the embankment deformation, foundation liquefaction and the potential cracking. Several methods were used to investigate the site conditions and the results were compared indicating good correlations and helping to define property ranges.

The analysis of liquefaction and strength loss potential included two basic concepts that involve soil behavior: “sandy like” and “clay like”. The analysis of the SPT samples identified the proper soil behavior during the seismic loading. Determinations of residual strengths of layers of concern were performed in-situ with the vane borer and hollow-stem augers. Finally, conclusions determining the level of risk and recommendations for future activity are made based on the results.

INTRODUCTION

This paper presents considerations to perform the analysis of the response of the embankment built on a foundation with concerns of liquefaction and strength loss. The analysis includes: earthquake characteristic discussions, computations of the response peak accelerations of the embankment and foundation, subsurface explorations, methodology of the analysis, conclusions and recommendations. A Risk Analysis was performed and concluded that the dam is a high risk structure with potential to produce heavy losses of life and property damage. The results of the analysis were assessed carefully, which ultimately be used as the basis to perform embankment modifications to prevent catastrophic failure.

EARTHQUAKE CHARACTERISTICS

Earthquakes are ground motions produced by movements of geotectonic plates and/or fault displacements, and they are cataloged by the source and by the energy released during the ground motion. The characteristics of the earthquakes are shown on the recorded ground motion (accelerogram) which involves several parameters that are useful for prediction of earthquake effects.

The dam site has the potential to be subjected to seismic events from various sources: Subduction Zones (Plate collisions), Crustal Faults (Faults displacements), and Areal Seismicity (Unidentified geologic structures/historical events).

The earthquake magnitudes. In his book Kramer [1] presents four main methods to measure earthquake magnitudes. The earthquake magnitudes are based on the trace amplitude (M_L), surface wave magnitude (M_S), body wave magnitude (m_b), and the moment magnitude (M_W). The magnitude of the expected maximum earthquake to occur at the dam site is a subduction earthquake with a magnitude M_W 9 and a return period of 50,000 years. In this paper, a 500 years return period subduction earthquake (Earthquake No. 1) is used to evaluate the effects of smaller earthquakes on the embankment.

The subduction source earthquake is expected to be as large as M_W ~9 based on the analysis of core samples consisting of coseismic turbidite deposits off the continental margin; analysis has identified the dating of the seismic events to almost 10,000 B.C.

Shallow Crustal earthquake sources that might be closer than 20 to 30 km to the site are produced by surface deformations (faults) produced by clockwise rotation of large crustal blocks within the plate. The closest fault to the site extends about 21 km in a southeast direction, which can produce an earthquake of magnitude (M_W) that may vary between 6.7 and 7.1.

Areal seismicity earthquake sources are based on shallow crustal earthquakes of limited magnitude that occur in unidentified geologic structures, producing earthquakes known as random events, with magnitudes varying up to M_L 7.0.

Time acceleration histories or accelerograms**.** The ground motion propagates in all directions and is recorded by Strong Motion Accelerographs in a 3_D Cartesian coordinates: two horizontal components, and one vertical component. The accelerograms used in this study were determined by combining the characteristics of several accelerograms generated under similar sources. The ground motion characteristics are presented clearly by the accelerogram, which basically is a cyclic loading consisting of: acceleration amplitudes, frequency content, and duration. These parameters give clues on how to use efficiently the accelerogram minimizing the analysis costs as will be discussed later.

Ground motions produced by fault activity tend to have the horizontal accelerations H larger than the vertical accelerations V, while the ground motions produced by subduction sources have a tendency to have larger acceleration in the vertical direction. The duration of the subduction ground motions are significantly larger than other ground motions.

The acceleration amplitude is the representation of the movements in three orthogonal directions. The maximum amplitude is the most used parameter in seismic analysis and it is known as the peak acceleration. Since the earthquake is represented by three accelerograms (H1, H2, and V) and the performed analysis is 2-D, the horizontal acceleration used is the Peak Horizontal Acceleration (PHA), which is the vectorial sum of the two orthogonal horizontal accelerations H1 and H2. See Table 1. In this study the largest computed horizontal acceleration is used as the PHA.

The frequency content**,** as indicated by Kramer, describes how the amplitude of a ground motion is distributed among the different frequencies within the accelerogram. There are three parameters that predict the potential effects of the frequency content. According to the frequency content shown on Figure 1, the embankment will be shaken severely for about 30 sec which may cause significant number of transverse and longitudinal large cracks, especially below the crest to a probable depth of a crest width (30 ft). The effects of the frequency content can be analyzed by the Fourier analysis and the response spectra. The Fourier spectrum gives clues of how the ground motion may affect the structures and the Response spectrum describes the maximum responses of a structure that is represented by a single degree of freedom system to a particular accelerogram as a function of the natural period and damping ratio.

The Predominant period is the maximum amplitude that occurs in any one of the spectral plots. The earthquake No. 1 has a predominant period of 0.20 for the H and 0.13 for the V. The natural period of the embankment is 0.31, which when comparing with the predominant periods allow us to predict that the H acceleration will amplify more than the V accelerations. The H amplification was from 0.21g to 0.66g. Both periods are also shown on Figure 1.

The duration of the ground motion has a strong influence on the damages produced by the seismic event. The accelerogram measures the ground motion from beginning to end, which may include very small amplitudes at the beginning (10 seconds) and at the end (20 seconds) of the accelerogram. Seed [2,3] has defined the duration of the earthquake from a magnitude that may produce effects on the structure. Seed call to this condition bracketed duration which normally is indicated as 0.05g. Kramer [1] presents several methods to define the proper duration of the ground motion. The accelerogram had duration of 120 seconds, but after an assessment of the frequency content it was reduced to 90 seconds (Bracketed accelerogram) by removing very small amplitudes that will not affect the embankment. The duration of this particular earthquake can be reduced even more if Seed's bracketed criterion would be applied at the beginning and end of the accelerogram. Figure 1 shows the bracketed acceleration time history in a horizontal and vertical direction of a subduction zone earthquake that could shake the dam site.

THE EMBANKMENT

Earthfill Dam No. 101 retains a 60,000 acre-feet reservoir with a significant population downstream of the dam. The freeboard of the embankment is about 10 ft during normal operations.

The embankment is about 150 feet high, zoned with a crest width of 30 feet, and is about 2000 ft long, with side slopes of 3:1 and 2:1 in the upstream and downstream slopes respectively. Figure 2 (Cross Section at Sta 7+00) shows the geometric configuration and zoning of the cross section, also it shows the soil properties, piezometric line, the critical slip surface and the minimum factor of safety (FS=1.49) under static loading. The residual shear strength of the Qal soil was determined by Vane Shear Tests (VST) as undrained strengths when dealing with seismic loading.

A post-earthquake analysis [3, 4] was performed by changing the strength of the Qal layer from peak values to residual values, which according to the Vane tests may vary from 2.5 psi to 7.5 psi (Table 2). Figure 3 (Post-Earthquake Stability Analysis) shows the model used in the post-earthquake analysis. The Qal layer below the crest has the strength of 7.0 psi and the Qal below the shell and the embankment toe has the strength of 2.5 psi. Under these conditions, the factor of safety is 0.83 indicating that the critical slip surface may lead to catastrophic embankment failure with large deformations.

SUBSURFACE EXPLORATION

Several field explorations were conducted at different stages of the analysis. The explorations consisted of Cone Penetration Tests (CPT), Standard Penetration Tests (SPT), Seismic Cross Hole tests, and Vane Shear Tests (VST) in the embankment and foundation. Also, the exploration program included sampling and laboratory testing. Figure 4 shows a plan of the embankment, spillway and the locations of the field testing. An assessment of the field exploration results has identified two horizons: alluvial soils and bedrock. The alluvial soils were subdivided in two sub-horizons: clayey soil (Qal), and basal gravely layer (Qalb). See Figures 2 and 3.

The CPT tests were performed along the downstream toe and below the downstream shell according to the ASTM Standard D-5778-95 procedure. The information provided by CPT-08-14 includes seismic velocity measurements, which are in close agreement with the results provided by the triplet Cross Hole Test. These tests are located at Sta. 7+25 near the downstream embankment toe. Figure 5 shows five parameters that are useful to the response analysis of the foundation. The seismic velocity or the shear wave velocity (Vs) is in close agreement with the Vs measurements by the Cross Hole Testing method. Other parameters of the CPT interpretation were used to confirm assumptions made on the analysis. Reference [5] presents the best uses of the CPT data.

The SPT tests were performed according to standard procedures for the determination of the liquefaction potential ASTM D-6066-96 [6]. SPT tests were performed below the crest, downstream shell, and the downstream toe [2]

Figure 6 shows the SPT data and the analysis of the foundation liquefaction on the right side of the embankment, which is the weakest zone of the dam site located on the downstream embankment toe, at about Sta. 7+00.

Some of the SPT results show potential for foundation liquefaction as shown in Figure 6 by comparing columns U and AC. However, since Qal has high content of fines as shown on column G of figure 6, it may not liquefy. The accelerations used on the liquefaction analysis were taken from the studies on the peak response accelerations discussed below. Thus, additional studies on the characteristics of these fine contents were required.

The shear wave velocities were measured along horizontal paths by the three cross hole method. Table 4 shows the Gmax values that were computed from the measurements of the Vs. The Vs measurements were made below the crest (V1 in figure 4), downstream slope (V2 in figure 4) and at the toe of the embankment (V3 in figure 4). Table 4 presents the average values, standard variation and the Gmax used in the analysis. During the analysis, Gmax values were varied within the ranges obtained from Table 4 until the results of the analysis were satisfactory. It was helpful to vary the Gmax Values until reasonable results were obtained during the intermediate steps of the analysis. The final results should be considered not as a number but as an indicator of how soon and how extensive the fixing will be. The Vs used on the determination of the natural period of the embankment was taken from Column AB and row 48 from Table 4.

The VST tests were performed to determine the peak undrained and the remolded residual stresses of the liquefiable soils with the Vane Borer M-1000. The test was performed in accordance with ASTM procedure D-2573.

The method consists of drilling with a double tube Auger to the depth where the tests would be performed. The shear vane is placed on the surface to be tested, pushed 1.5 ft and measure the required torque to define the peak undrained residual strength. The torque head located on top of the auger is connected to the shear vane by a rod, and the peak residual strength is measured by the stress applied through the torque. Several pushes and tests were made every 1.5 ft in a predetermined span, and then the shear vane is retracted. The auger is advanced to the next span for additional testing. The tested soil is over cored and the soil is retrieved for lab testing. The new span is tested until reaching its full depth.

The remolded tests are made immediately after the peak undrained strength is measured by relocating the torque dial to zero; the test restarts with a new applied stress after the shear vane is rotated 360 degrees. Several spans were tested in ten foot increments. Figure 7 (Recording Head with Casing Adaptor and Recorded Sample) shows the recording head and a sketch of the recorded sample test. A summary of the shear residual strengths are shown on Table

SOIL PROPERTIES

Index Properties, LL, PI, Soil Classifications, Fines Content and soil behavior studies were made intensely. Table 3 presents some of the soil properties of the alluvial foundation, which will allow us to predict its behavior respect to liquefaction. Most of the samples were taken from drillholes, and were tested in the laboratory. Because of the high fines content determined, the liquefaction potential predicted by the SPT method may not occur at the dam site. However, according to the studies performed by Seed [2] the clayey portions of the Qal may experience loss of strength during the earthquake event due to cyclic failure.

The soil strain dependent properties (Shear Modulus) are modified by the Shear Modulus and Damping Ratio Reduction Functions. Figure 9 and Figure 10 show the details of these functions.

THE ANALYSIS

The analysis of the embankment response to seismic loading consisted on determining (1) the response peak accelerations for the assessment of the liquefaction potential analysis, which will produce large deformations leading to failure. And (2) assuming that liquefaction and/or cyclic failure has not occurred, the potential deformation of the embankment will be determined.

Liquefaction potential was analyzed by the Shear-Wave Based method as presented by Andrus [8]. The liquefaction resistance and the Vs are based on: confining pressures, plasticity, void ratio, moisture content, and the degree of cyclic loading, and so the method is appropriate. However, on this case the results obtained by this method were not satisfactory. The Vs method consists in comparing the cyclic stress ratio (CSR) and the cyclic resistance ratio (CRR). The CSR was computed by the classical formula CSR= $0.65 \times (a_{max}/g) \times (\sigma_v/\sigma'_v) \times r_d$. The author believes that the r_d factor or the stress reduction factor is not yet well defined to be used in all cases. The CRR formulae presented on reference [8] is quite sound.

The model that will analyze the embankment response includes assessments of the accelerograms, shear strengths, shear modulus, adjusted piezometric line during. The stratigraphy of the foundation is defined by layers that have clay like properties and sand like properties. The materials have selected ranges of shear strengths determined by Vane tests and CPT tests. This model was used for the static slope stability analysis, and slightly modified for the dynamic response analysis with small variations of the Qal zones, to portray the dynamic soil properties measured on the site, as shown on Figure 11.

Peak response accelerations under the Earthquake No. 1 loading were determined using QUAKE/W. The dynamic shear modulus reduction and damping functions shown on Figures 9 and 10 were selected from typical values based on their gradation and Atterberg limits. The functions were extrapolated to cover a required 10% strain. Figure 12 and Figure 13 show details of the peak response accelerations used in the determination of the liquefaction potential by the SPT method. Table 5 shows a summary of the response peak accelerations at the center of gravity of the critical slip surface, the average peak response acceleration on the upper quarter of the embankment, and below the downstream toe area.

Samples taken from Qal were investigated for strength loss potential. According to the data presented in Table 3, 40% of the samples fall in the Zone A and Zone B indicating that the soils will experience loss of strength due to cyclic failure. See Figure 8 for location and details of these Zones A and B.

The deformations of the embankment during the Earthquake No. 1 loading were computed by QUAKE/W using the Newmark Method option. The critical slip surface was used for the computation of the deformation by the Finite Element Newmark method option of QUAKE/W and it was found that the maximum deformation corresponding to the minimum factor of safety of 1.49 is 1.3 feet, which occurs 60 seconds after the earthquake started. Figure 14 shows the history of the deformations during the earthquake; the time corresponds to the bracketed accelerogram. The deformation is large for an embankment with FS=1.49 under an earthquake with peak acceleration of 0.21g H. However, when the frequency content parameters are introduced to this assessment, the deformation appears to be reasonable.

CONCLUSIONS AND RECOMMENDATIONS

The dam site has the potential to be shaken by earthquakes produced by different sources; a subduction earthquake within 500 years reoccurrence was selected as the most appropriate earthquake to perform the analysis, which features are: M_W=9.0, α_H = 0.286g, α_V=0.214g and duration of 120 sec.

The acceleration histories were analyzed and indicate: (1) The response peak accelerations in the embankment and foundation shows amplification from 0.21g to 0.66g and from 0.286g to 0.55g, H and V respectively; a comparison of the natural period of the embankment/foundation and the predominant period of the earthquake also indicates strong amplification and (2) The reduction of the duration of the accelerogram is reasonable and beneficial because the computation time was reduced and did not change the embankment response characteristics.

Subsurface explorations were performed according to the progress of the analysis, and the tests were selected to complement each other. The soil properties measured in-situ are well measured and reliable.

The analysis scope of the embankment response under seismic loading includes: (1) Determination of the response peak accelerations for the liquefaction potential assessment has indicated that some Qal sandy soils will liquefy, (2) according to the plasticity chart other Qal clayey soils will have shear strength reduction due to cyclic failure, (3) The embankment deformations considers non-liquefiable soils and indicates a displacement of 1.3 ft.

The subduction Earthquake No. 1 may produce a catastrophic release of the reservoir.

It is recommended that the analysis should be extended for larger return periods to assess the urgency and extent of the embankment modifications to prevent human losses and property damages.

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TABLES

Table 1. Summary of the Peak Accelerations of the Accelerograms

Magnitude (M_W)	H1	H2	V	
8	0.285	0.286	0.214	
	*			Used as PHA

Table 2. Shear Residual Strength by VST

Shear Strength	Min. (psi)	Max (psi)
Peak strength	15	20
360 degree remolded	2.5	7

Table 3 Soil Properties

Depth (Ft)	%<.005	% Fines	LL	PI	Moisture %	Classification	Source
4 to 36	31	54	44	21		CL	DH-2
5 to 30	19	51	37	12		ML-CL	DH-3
0 to 42	18	56	34	10		CL-ML	DH-4
0 to 10	12	34	28	6		CL-ML	DH-5
9.6 to 11.5	42	56	42	16	37.6	CL-ML	DH-8
22.4 to 24.5	66	30	83	51	44.0	CH	DH-8
8.2 to 10.9	34	65	42	11	37.9	CL-ML	DH-11
15.4 to 17.7	33	67	42	13	38.5	CL-ML	DH-11
24.2 to 26.9	60	40	66	38	47.0	CH	DH-11
3.5 to 5.6	65	34	63	20	56.4	MH	DH12
7.9 to 10.3	54	45	59	23	52.6	MH	DH-12
12.8 to 15.2	50	48	53	19	57.0	MH	DH-12
24.6 to 26.9	39	49	46	17	45.1	ML	DH-12
29.1 to 31.6	29	49	38	12	46.9	ML	DH-12
31.6 to 34.0	25	47	38	8	45.9	ML	DH-12
6.2 to 8.6	35	65	41	13	39.4	CL-ML	DH-17
22.4 to 24.8	51	29	52	29	36.6	CH	DH-17
27.2 to 29.6	40	35	46	19	48.8	CL-ML	DH-17
45 to 7			59	28	42.8	MH-CH	DH-73
50 to 52			55	25	45.1	MH	DH-73
55 to 57			54	24	44.7	MH	DH-73
62 to 65			38	12	39.2	ML	DH-73
41				NP	25.2	ML	DH-74
46			60	27	44.5	MH	DH-74
51	42	54	52	22	45.4	MH-CH	DH-74
56			48	20	45.9	ML-CL	DH-74
61			42	16	150	ML-CL	DH-74
128 to 130			47	16	37.3	CL_S	AP-2-92
124			48	19	38.2	CL	AP-2-92
133			60	28	33.4	CH	AP-2-92
139			38	13	33.5	$_S$(CL)	AP-2-92
144			41	11	26.4	$_S$(CL)	AP-2-92

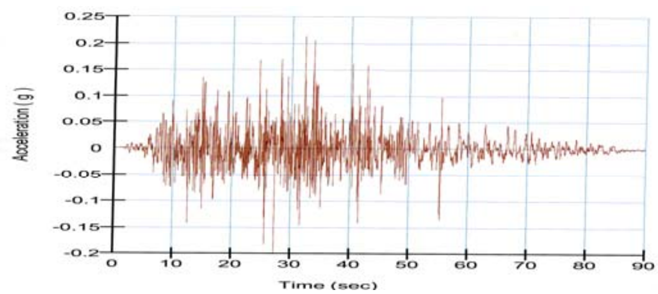
Table 4. Gmax

	X	AB	AC	AD	AH	AI	AJ	AN	AO
1	DH-08-1 TO -3			DH-08-06 X DH-08-04			DH-08-11 TO DH-08-12		
2	Scoggins Dam Crest			Mid slope			Right D/S toe		
3	Layer No.	Gmax		Layer No.	Gmax		Layer No.	Gmax	
4	1	3.78E+06		1	4.60E+06		1	7.56E+05	
5	2	5.34E+06		2	5.17E+06		2	8.77E+05	
6	3	5.37E+06		3	4.79E+06		3	8.88E+05	
7	4	5.05E+06		4	4.98E+06		4	6.34E+05	
8	5	4.87E+06		5	4.68E+06		5	7.79E+05	
9	6	5.29E+06		6	5.28E+06		6	6.56E+05	
10	7	5.44E+06		7	5.42E+06	**Zone 3**	7	5.77E+05	**Qal**
11	8	5.05E+06		8	5.33E+06		8	8.77E+05	
12	9	5.31E+06		9	5.12E+06		9	9.81E+05	
13	10	5.37E+06		10	5.73E+06		10	1.15E+06	
14	11	5.60E+06		**Average**	**5.11E+06**		**Average**	**8.18E+05**	
15	12	5.25E+06		**STDEV**	3.53E+05		**STDEV**	1.74E+05	
16	13	5.28E+06		Av+/-STD	5.47E+06	4.76E+06	Av+/-STD	9.92E+05	6.43E+05
17	14	5.06E+06		**Used**	**5.11E+06**		**Used**	**8.18E+05**	
18	15	4.48E+06	**Zone 1**	11	5.83E+06		11	2.30E+06	
19	16	4.21E+06		12	6.31E+06	Zone2 U	12	2.18E+06	**Qalb**
20	17	4.57E+06		13	5.18E+06		13	2.51E+06	
21	18	5.05E+06		14	5.57E+06		**Average**	**2.33E+06**	
22	19	5.57E+06		15	5.63E+06		**STDEV**	1.68E+05	
23	20	5.79E+06		16	5.19E+06		Av+/-STD	2.50E+06	2.16E+06
24	21	5.53E+06		17	5.26E+06		**Used**	**2.33E+06**	
25	22	5.21E+06		18	6.38E+06				
26	23	4.76E+06		19	7.05E+06	**Zone 2**			
27	24	4.96E+06		20	7.17E+06	Satyrated			
28	25	5.10E+06		21	6.05E+06				
29	26	5.37E+06		22	5.68E+06				
30	27	5.09E+06		**Average**	**5.94E+06**				
31	28	4.80E+06		**STDEV**	6.76E+05				
32	29	4.07E+06		Av+/-STD	6.62E+06	5.27E+06			
33	30	2.99E+06		**Used**	**5.74E+06**				
34	**Average**	**4.99E+06**		23	4.99E+06				
35	**STDEV**	5.96E+05		24	4.05E+06				
36	Av+/-STD	5.58E+06	4.39E+06	25	2.44E+06				
37	**Used**	**5.10E+06**		26	2.67E+06	**Qal**			
38	31	2.97E+06		27	3.06E+06				
39	32	2.96E+06		28	3.16E+06				
40	33	2.89E+06		**Average**	**3.39E+06**				
41	34	2.96E+06	**Qal**	**STDEV**	9.55E+05				
42	35	3.19E+06		Av+/-STD	4.35E+06	2.44E+06			
43	36	3.23E+06		**Used**	**2.83E+06**				
44	37	3.21E+06		29	6.05E+06	**Qalb**			
45	**Average**	3.06E+06		**Average**	**6.05E+06**				
46	**STDEV**	1.44E+05		**STDEV**	0				
47	Av+/-STD	3.20E+06	2.91E+06	Av+/-STD	6.05E+06	6.05E+06			
48	**Used**	**3.05E+06**		**Used**	**6.00E+06**				
49	38	5.73E+06	**Qalb**	30	1.05E+07	rock			
50	39	9.79E+06		31	1.44E+07				
51	**Average**	7.76E+06		32	1.84E+07				
52	**STDEV**	2.87E+06		33	2.18E+07				
53	Av+/-STD	1.06E+07	4.89E+06						
54	**Used**	**6.00E+06**							
55	40	1.59E+07	Rock						
56	41	2.12E+07							

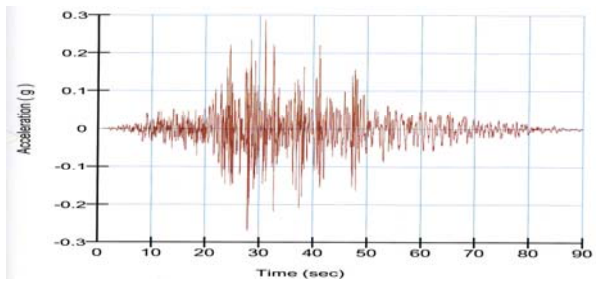
Table 5. Summary of Peak Response Accelerations

Location	X component in g's	Y component in g's
Center of Gravity	0.42	0.34
Upper Quarter of Embankment	0.5 to 0.6	0.28 to 0,34
Downstream Embankment toe		
El. 210	0.50	0.35
El. 191	0.5	0.30
El. 184	0.45	0.30
El. 178	0.40	0.25
El. 172	0.35	0.25

FIGURES



X Component



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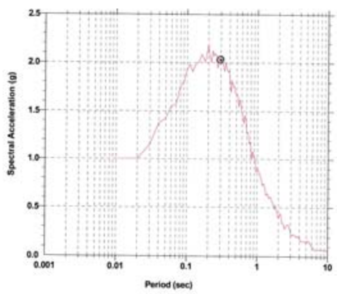

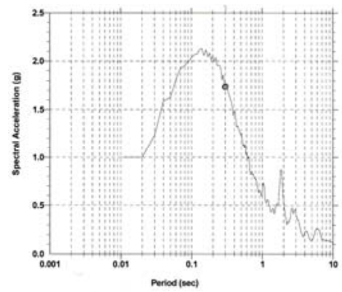

Figure 1 *Time Acceleration History for the Horizontal and Vertical Components*

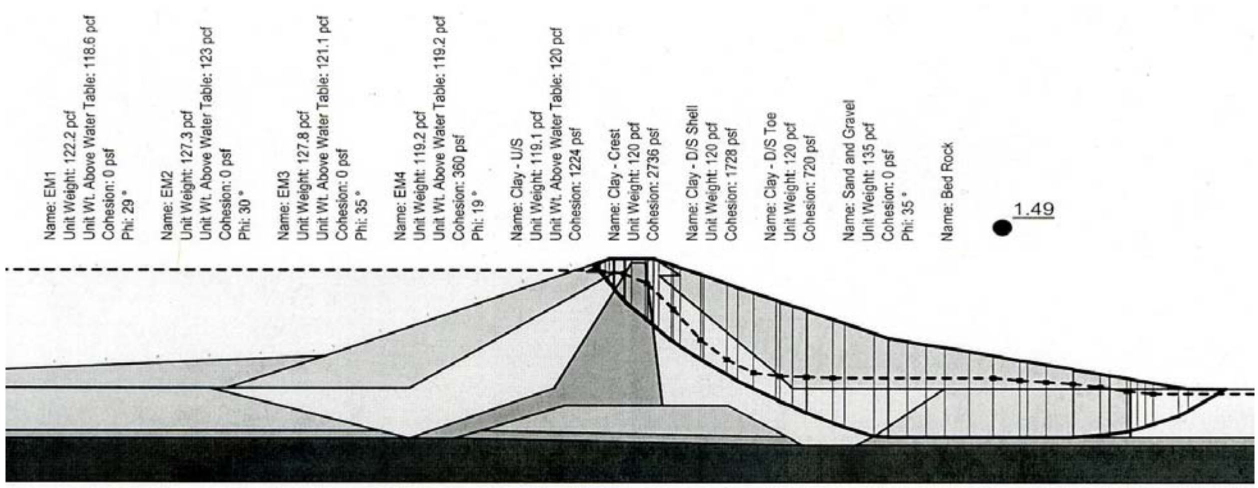

Figure 2. Factor of Safety During Normal Conditions

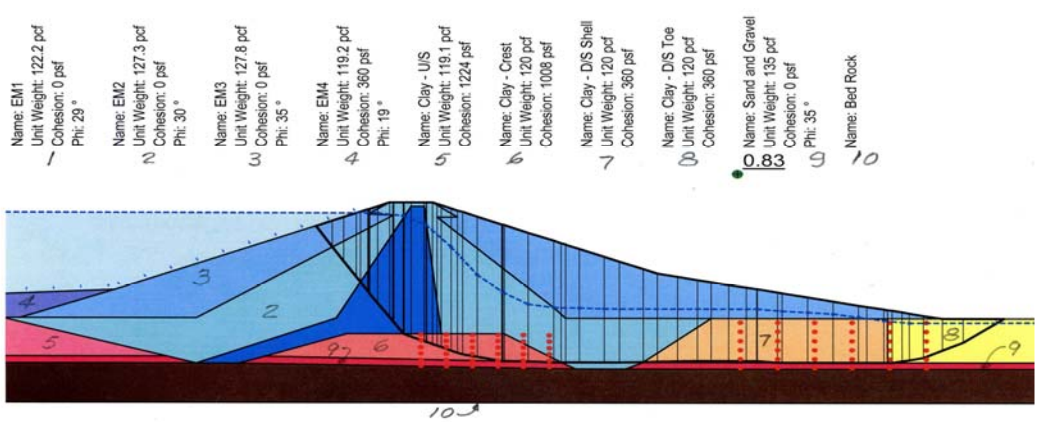

Figure 3. Post-Earthquake Stability Analysis

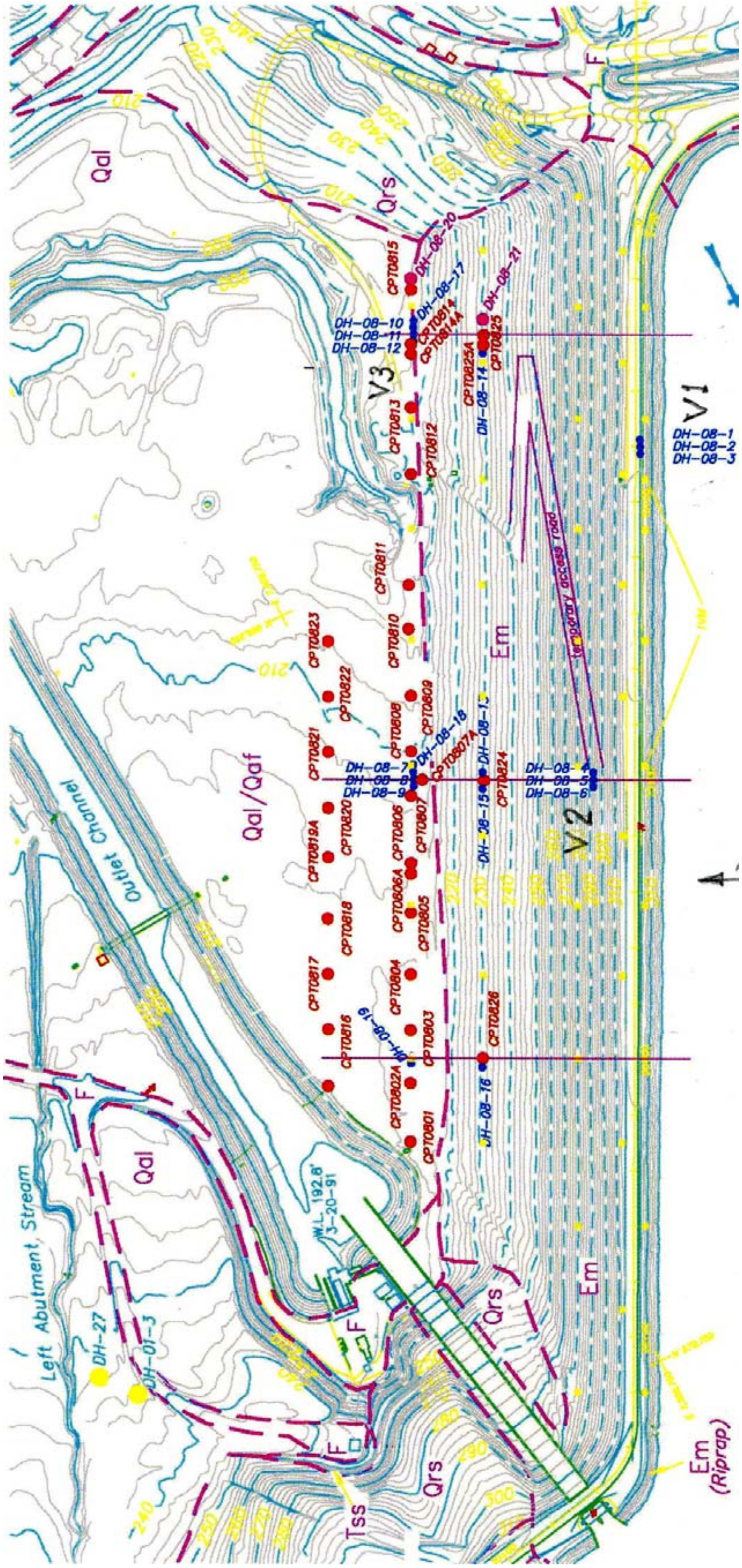

Figure 4. Subsurface Exploration Program

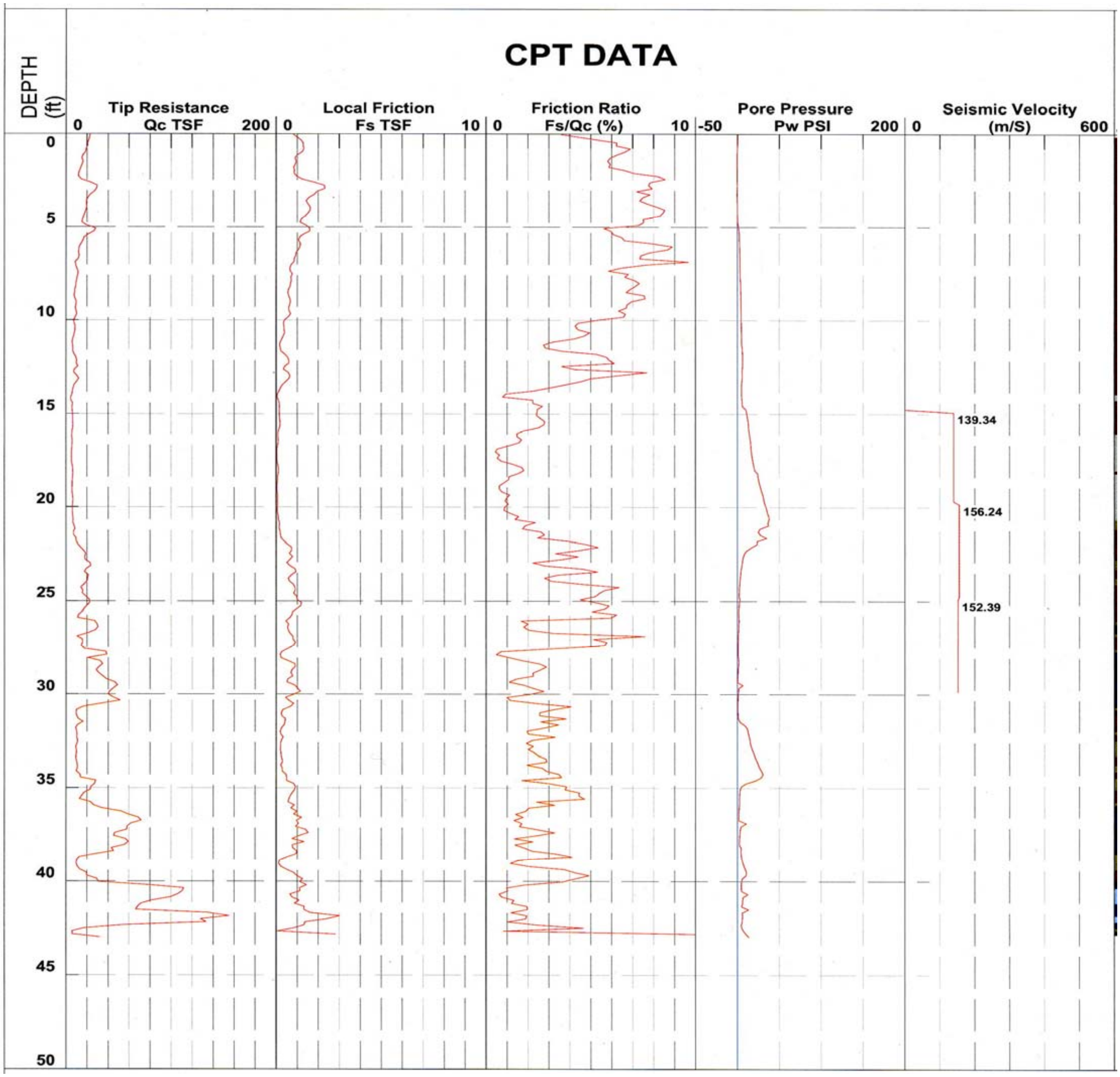

Figure 5. CPT Data

Hole Location DH08-10 Left downstream embankment toe

Ground Surface	207.5	feet		
Water Surface	204.5	feet		
Atmospheric Pressure	2.09	ksf	100	kPa
Drill Hole Diameter	100	mm		
Liner?	yes			
Energy Correction	50%	0.83		
Earthquake Magnitude	6.80			

500 yrs r.period
Peak Acc.: 0.35 g

A	B	C	D	E	F	G	I	J	K	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC
Top DEPTH (FT)	Bottom DEPTH (FT)	SPT DEPTH (FT)	SPT ELEV. (FT)	Soil Density (lb/ft^3)	Soil Type	Fines Content	N (BPF)	Effective Stress (ksf)	Total Stress (ksf)	$(N_1)_{60}$ Corrected (BPF)	Fines Correction α	β	(BPF)	$(N_1)_{60\text{-}CS}$ (BPF)	r_d	A_{max}/g	CSR	K_m	K_α	K_σ	$CSR_{7.5}$	$(N_1)_{60}$ Required
33.0	34.0	33.5	174.0	110	CL-SC	45.0	2	1.78	3.69	2	5.00	1.20	5	**7**	0.90	0.35	0.424	1.28	1.0	1.00	0.332	**27**
35.5	36.5	36.0	171.5	110	(ML)s	65.0	3	1.90	3.96	3	5.00	1.20	6	**8**	0.88	0.35	0.418	1.28	1.0	1.00	0.326	**27**
38.0	39.0	38.5	169.0	110	(SM)g	15.0	3	2.02	4.24	3	2.50	1.05	3	**5**	0.86	0.35	0.411	1.28	1.0	1.00	0.321	**27**
40.5	41.5	41.0	166.5	110	(CL)s	60.0	29	2.14	4.51	24	5.00	1.20	10	**34**	0.84	0.35	0.403	1.28	1.0	0.99	0.318	**27**
44.0	45.0	44.5	163.0	110	GC-GM	30.0	100	2.31	4.90	79	4.71	1.15	17	**96**	0.81	0.35	0.392	1.28	1.0	0.98	0.313	**27**

Hole Location DH08-14 Mid slope embankment, Left

Ground Surface	230.2	feet		
Water Surface	226.2	feet		
Atmospheric Pressure	2.09	ksf	100	kPa
Drill Hole Diameter	100	mm		
Liner?	yes			
Energy Correction	50%	0.83		
Earthquake Magnitude	8.00			

500yr r.period
Peak Acc.: 0.35 g

A	B	C	D	E	F	G	I	J	K	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC
Top DEPTH (FT)	Bottom DEPTH (FT)	SPT DEPTH (FT)	SPT ELEV. (FT)	Soil Density (lb/ft^3)	Soil Type	Fines Content	N (BPF)	Effective Stress (ksf)	Total Stress (ksf)	$(N_1)_{60}$ Corrected (BPF)	Fines Correction α	β	(BPF)	$(N_1)_{60\text{-}CS}$ (BPF)	r_d	A_{max}/g	CSR	K_m	K_α	K_σ	$CSR_{7.5}$	$(N_1)_{60}$ Required
55.0	56.0	55.5	174.7	110	SC	29.0	15	2.89	6.11	11	4.64	1.15	6	**17**	0.72	0.35	0.347	0.84	1.0	0.94	0.437	**30**
56.9	57.9	57.4	172.8	110	SP	45.0	9	2.98	6.31	6	5.00	1.20	6	**13**	0.71	0.35	0.340	0.84	1.0	0.94	0.431	**30**
59.5	60.5	60.0	170.2	110	(SM)g	20.0	17	3.11	6.60	12	3.61	1.08	5	**16**	0.69	0.35	0.331	0.84	1.0	0.93	0.423	**30**
61.7	62.7	62.2	168.0	110	(SM)g	15.0	34	3.21	6.84	23	2.50	1.05	4	**26**	0.67	0.35	0.324	0.84	1.0	0.92	0.416	**30**
64.5	65.5	65.0	165.2	110	(ML)g	65.0	50	3.34	7.15	33	5.00	1.20	12	**45**	0.64	0.35	0.314	0.84	1.0	0.91	0.407	**29**
67.1	68.1	67.6	162.6	110	(GP)s	30.0	20	3.47	7.44	13	4.71	1.15	7	**20**	0.62	0.35	0.304	0.84	1.0	0.91	0.398	**29**

Figure 6. Liquefaction Potential Assessment by SPT Method

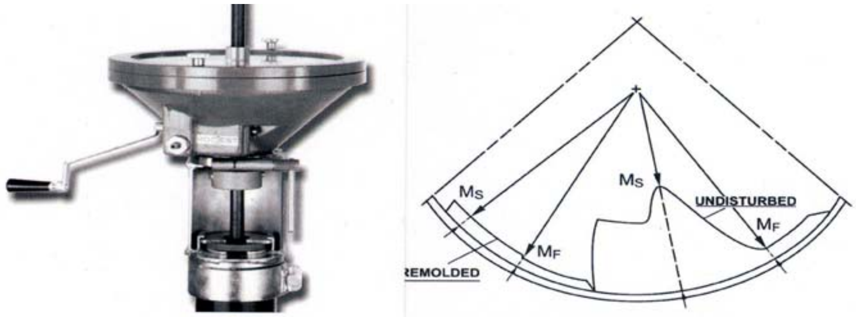

Figure 7 Torque Head and Test Recorded Sample

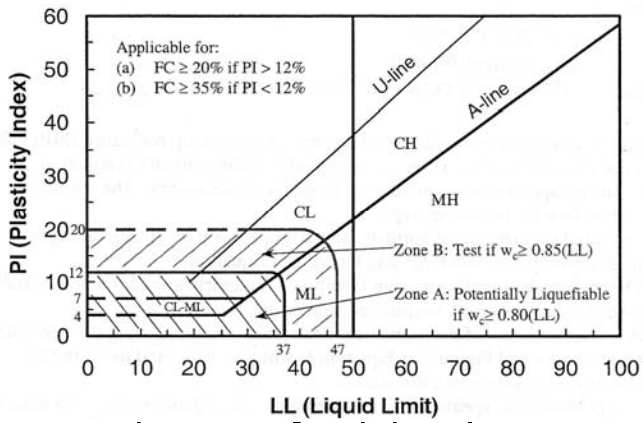

Figure 8 Plasticity Chart

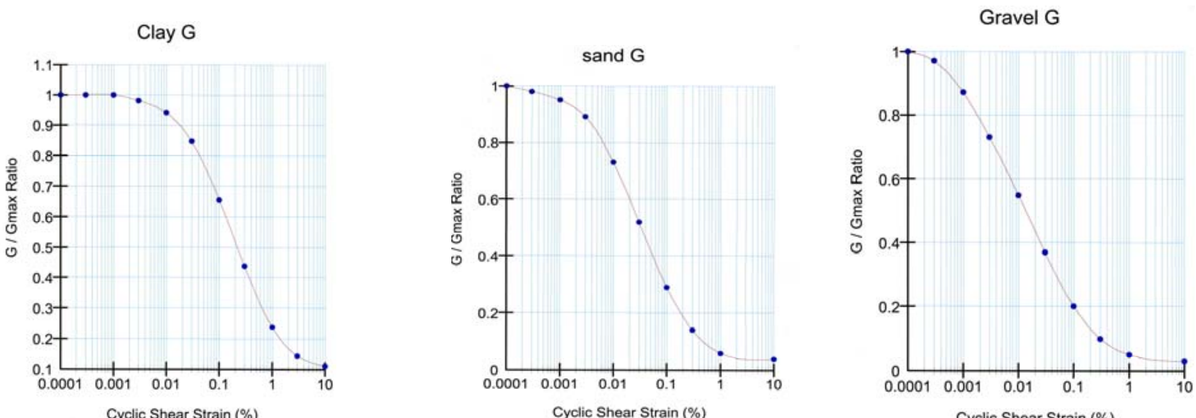

Figure 9. Shear Modulus Reduction,

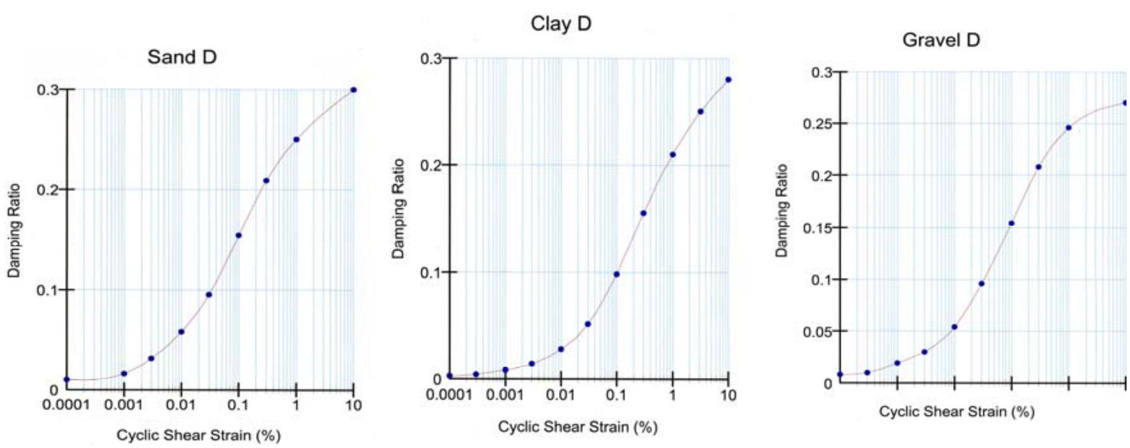

Figure 10. Damping Ratio Reduction

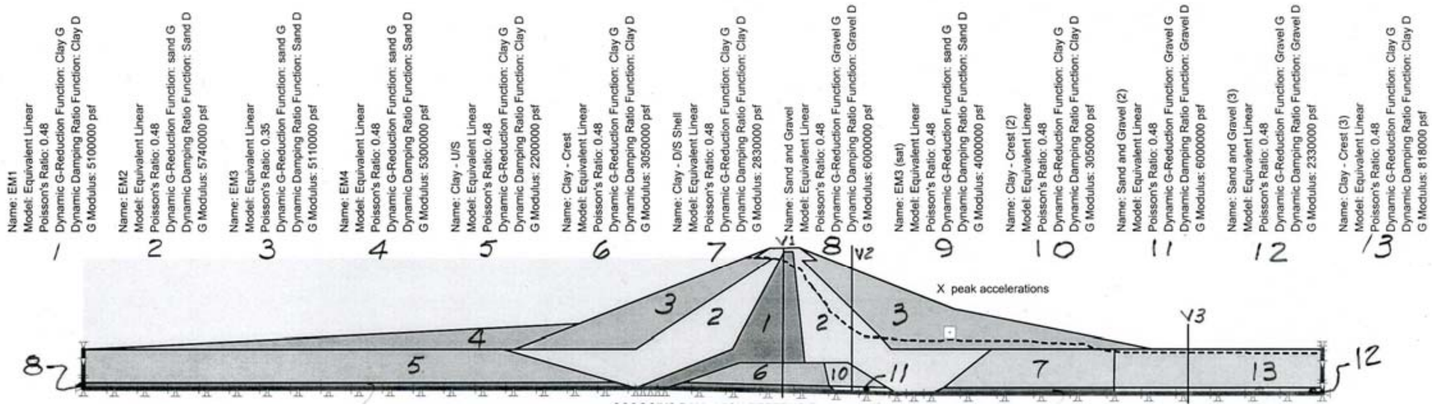

Figure 11 Model for the Dynamic Response of the Embankment

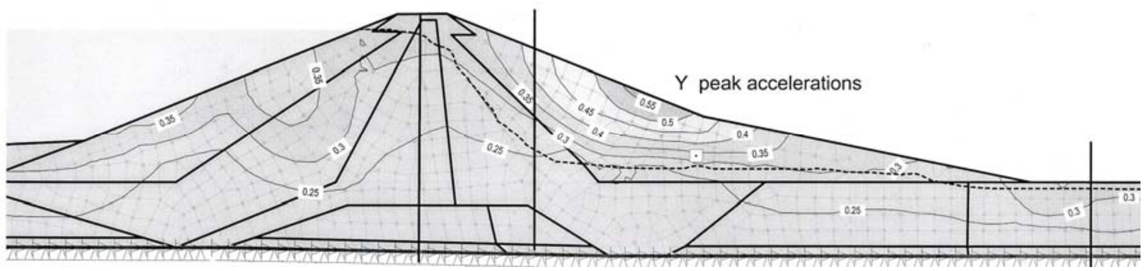

Figure 12. Vertical Peak Accelerations

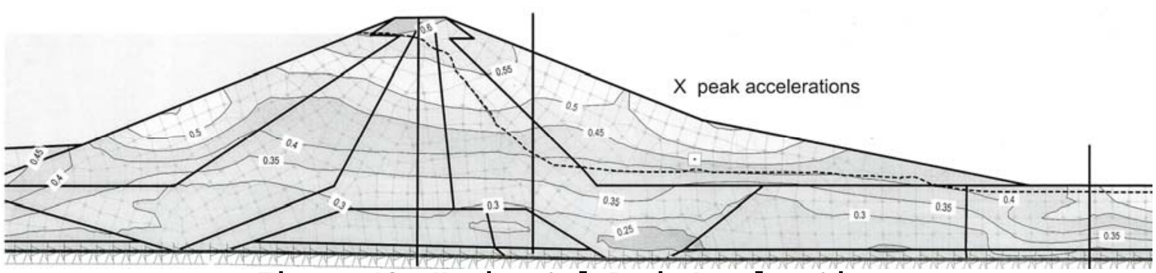

Figure 13. Horizontal Peak Accelerations

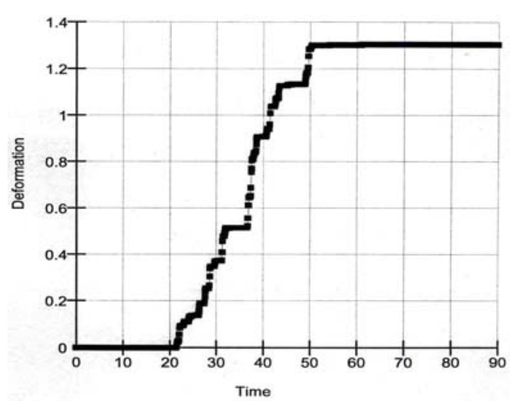

Figure 14 Deformation – Time History