A CASE STUDY OF SHEFFIELD DAM FAILURE DURING AN EARTHQUAKE

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SUMMARY

Learning from past failures is a strong tool for the designer, but not much information on the damages caused to earth dams during the earthquake is available. Sheffield earth dam failed in Santa Barbara Earthquake of 1925. A case study was made by Seed et al. (1969). This study was based on the laboratory test results obtained from triaxial and simple shear tests under cyclic loading conditions and the main cause of failure was assessed to be liquefaction. Since small sample studies have attracted many doubts, the case study of sheffield dam failure is discussed using a new approach of liquefaction analysis which uses the test data obtained from a large sample study on a horizontal vibration table.

INTRODUCTION

The Sheffield dam failed during the Santa Barbara, California earthquake of June 29, 1925. This is one of the very few recorded cases in which a dam failure has been attributed to an earthquake. A case study of the failure was carried out by Seed, Lee and Idriss (1969) to assess the factors leading to the failure. The information on the earthquake parameters and the soil conditions was provided by the records of the failure made soon after the earthquake by engineers and seismologists and by a study conducted by the U.S. Army Corps of Engineers in 1949. In this case study small sample laboratory test data was used to study the behaviour of dam during the earthquake. The main cause of failure has been assessed as liquefaction of sand in earth dam and in foundation.

Laboratory test data used was obtained from small size samples subjected to triaxial and cyclic loading tests. However, various investigators have raised doubts about the laboratory test results on small size samples in cyclic loading triaxial or simple shear apparatus. There has been no uniform agreement in both qualitative as well as quantitative results obtained by different investigators on small samples. In the test carried out by Seed and his coworkers, any sand could be liquefied under defined test conditions while in tests carried out by Castro the medium dense to dense sand samples could not be liquefied and they showed a dilative response. From the tests conducted on small size samples under cyclic loading conditions, Castro and Poulos (1977) believe that observed behaviour of cyclic deformations in dilative clean sands are due to a test error-redistribution of void ratio, which is not representative of field behaviour. They have also stated that the use of cyclic load test results should be avoided for quantitative evaluation of insitu deformations.

The progressive development of liquefaction and spreading of the liquefaction zone is one of the most important aspect of liquefaction during the earthquake. The liquefaction of soil layer in first few cycles of ground motion would reduce the overburden pressure on lower layers. This may cause

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favourable conditions for liquefaction to develop in the underlying layer in the next few cycles, thereby further reducing the overburden pressure on deeper layers. In this way the liquefaction may travel to sufficiently deep layers during the earthquake. This aspect of the problem was also not considered in the above case study.

Now a method has also been made available to study the liquefaction problem using vibration table test data on large size samples which takes into account the progressive nature of liquefaction (Gupta 1977). In this paper the liquefaction of Sheffield dam has been predicted using this new approach.

DESCRIPTION OF THE SHEFFIELD DAM

Detailed information regarding the dam embankment and foundation soil properties have been reported by Seed et al. (1969) and the same are used in the present analysis. The Sheffiold dam was constructed in 1917 in a ravine north of the city of Santa Barbara. A section of the dam at its maximum height is shown in Figure 1. The dam 220 m (720 feet) long with a maximum height of 7.6 m (25 feet) was constructed of soil from the reservoir excavation and compacted by routing the construction equipment over the fill. The body of the dam was composed of silty sand and sandy silt containing some cobles and boulders. The upstream slope was faced with a 1.2 m (4 feet) thick clay blanket which was extended upto 3 m (10 feet) into the foundation to serve as a cut-off wall. The clay blanket was overlain with a 5 inch concrete facing. Relative density of the embankment material is reported to be 35% to 40% only. Since the dam was constructed nearly sixty years back and advanced compaction techniques were not available, this value of relative density is quite possible. At the time of earthquake, the depth of water in the reservoir was about 5 m. It is also reported that seepage had been noted near the toe of the downstream and the water level in the dam was expected to be as shown in Figure 1.

The foundation soil about 2 m (6 feet) thick below the dam consisted mainly of silty sand and sandy silt containing some cobbles varying from 75 mm to 150 mm in diameter overlying the firm base. Grain size analysis of the silty sand and sandy silt samples showed percent coarser than 0.02 mm varying from about 40% to 60%. The upper about 0.5 m of this soil was much looser than the deeper material. The relative density for the top 0.5 m soil are reported to be about 35% to 40% like the dam material.

SANTA BARBARA EARTHQUAKE

Santa Barbara earthquake of June 29, 1925 occurred at about 6 hours 42 minutes (Nunn, 1925). The macroseismic data obtained in and around the area by Willis (1925) and Byerly (1925) formed the basis of estimating the violence of the earthquake at different places. These isoseismals supplement the far field data in the absence of any strong motion instrument in the area, in assigning the release of energy at the source and its probable location. The earthquake was assigned a magnitude of 6.3 on open ended Richter scale and location being around 11 km from the dam site (Seed et al., 1969).

Even though there are numerous lineaments in the vicinity of Santa Barbara and early reports attribute the earthquake to movement along one of these, yet there is no evidence of any horizontal or vertical movement of the ground surface during the earthquake (Willis, 1925). The more recent studies also do not associate the triggering of this earthquake because of movement alone of the lineaments close to Sheffield reservoir, which is across Santa Ynez Lineament (Seed et al., 1969).

As per isoseismals, the intensity at dam site can be interpolated between VIII and IX on Rossi-Forell scale (Byerly, 1925). Willis (1925) had earlier estimated the duration of the main earthquake as 15 sec. This estimate appears to be reasonable and is supported by the data published in Bolt (1975). From these accounts available in the past reports, the maximum ground acceleration in the vicinity of the Sheffield dam is estimated as 0.15 g. from Gutenberg and Richter (1942) formula and is confirmed by the recent works cited in Bolt (1973). Comparison with earthquakes of similar magnitude occurring at 11 km epicentral distance, the maximum ground acceleration at similar geological setting indicates reasonable accord with the previous estimate and also gives the value of 3 Hz as predominant frequency in the motions at ground surface. The time history of such a ground motion, obtained by Seed et al. (1969) from the appropriate scaling of 1940 El Centro accelerogram is given in Figure 2, which has been used in the present analysis.

METHOD OF LIQUEFACTION ANALYSIS FROM VIBRATION TABLE TESTS

The following physical phenomenon constitutes the basis of liquefaction analysis.

a) An increase in pore water pressure during vibrations at a depth in a saturated soil mass causes an equal increase in pore water pressure through out the deposit below this depth. Thus the pore pressures developed in first few cycles of ground motion at a depth are transmitted equally in all the deeper layers and the effective overburden pressures in these deeper layers are reduced. In the next few cycles and under reduced effective overburden pressures further pore water pressure may develop in deeper layer resulting in still further reduction in effective overburden pressure on still deeper layers. In this process, if the pore water pressure becomes equal to the initial effective overburden pressure, progressive liquefaction may occur starting at some depth and moving to deeper layers.

b) The pore water pressure developed during the earthquake dissipate only by upward movement of water thus setting up a water flow in the deposit. During this process a hydraulic gradient will be set up in the deposit and may develop quick conditions in the upper layers which may not have liquefied earlier. Thus partial liquefaction in lower layers may cause complete liquefaction in upper layers.

Based on these physical concept a method of analysis is developed. This method offers a way to estimate the loss of effective overburden pressure on account of increase in pore water pressure during the earthquake. The following information is required for carrying out the analysis.

a) Accelerogram of the anticipated earthquake: Approximate accelerogram for the site in question is first estimated.

b) Laboratory data: The laboratory tests were performed on a horizontal vibration table on the sand samples. The saturated sand sample is prepared in the tank (size 105 cm x 60 cm x 40 cm high) mounted on the table. The frequency and amplitude of the table can be independently controlled. 26 cm was the height of the sample. The tests are required to be performed at predominent frequency of the accelerogram and at different accelerations for corresponding field relative densities and under different dead wiehgt surcharge conditions on sand obtained from site. Plots of excess pore water pressure versus effective overburden pressure for corresponding relative densities at different accelerations serves as laboratory data for the analysis.

The liquefaction analysis is now carried out in the following steps graphically.

1. Mark the thickness of different strata of different relative densities.

2. Plot the effective overburden pressures versus depth.

3. Consider the accelerogram to consist of suitable different number of cycles of different accelerations keeping the sequence of vibrations the same as in accelerogram.

4. For the first set of number of cycles of known acceleration, plot the pore pressure with depth with the help of laboratory data for corresponding relative density and surcharge till maximum pore pressure at a particular depth is obtained. The pore pressure below this depth equals the maximum value and remains constant.

5. Repeat step 4 for second set of cycles of known intensity and obtain further loss in effective overburden pressure.

6. Repeat the process step by step for the entire accelerogram.

7. Extend this final line to meet the line of initial effective overburden pressure. The depth to the point of intersection is considered to have liquefied.

FAILURE ANALYSIS OF THE DAM

The laboratory tests on vibration table were performed on a silty sand sample obtained from a dam site in India. The grain size characteristics of this sand are very close to the reported characteristics of silty sand of Sheffield dam. Therefore the laboratory data obtained on this soil was used for the analysis of sheffield dam. In laboratory tests the soil sample at relative density of about 35 to 40% liquefied completely at accelerations of more than 10% g (g acceleration due to gravity). Therefore the accelerogram expected at site (Figure 2) can be considered to consist of the following combinations of acceleration intensity and number of cycles without much error.

	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Acceleration % g	10	10	5	5	5	5	5
Number of cycles	10	10	10	10	10	10	10

For a typical section AA (Figure 1) the analysis is shown in Figure 3, and Table 1 illustrates it. At water table the initial overburden pressure is 944 g/cm^2. After first 10 cycles of 10% g a maximum loss of 60 g/cm^2 occurs and residual effective overburden pressure remains 884 g/cm^2. Thus proceeding step by step for the entire earthquake it is indicated that soil layer between 5.5 m to 6.5 m will completely liquefy.

Similarly the liquefaction analysis for the dam was carried out at different sections. Zone of liquefaction in the dam section indicated during the earthquake according to this method is shown in Figure 4. It will be expected under these circumstances that the dam may float on its base and may move during the earthquake as has been observed during the 1925 earthquake.

CONCLUSION

The dynamic analysis procedure incorporating test data from vibration table tests on large size samples predicts the failure of dam by sliding along its base as a result of soil liquefaction. This type of analysis takes into account the progressive spreading of liquefaction during the earthquake.

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TABLE - 1

LIQUEFACTION ANALYSIS OF DAM AT SECTION AA

Sl. No.	Acceleration % g	No. of Cycles	Depth m	Effective Overburden Pressure g/cm^2	Excess pore pressure g/cm^2	Residual Overburden g/cm^2
(1)	10	10	5.5	944	60	884
			6.5	1036	60	976
			7.5	1128	60	1068
(2)	10	10	5.5	884	80	804
			6.5	976	80	896
			7.5	1068	80	988
(3)	5	10	5.5	804	90	714
			6.5	896	90	806
			7.5	988	90	898
(4)	5	10	5.5	714	120	594
			6.5	806	120	686
			7.5	898	120	778
(5)	5	10	5.5	594	170	424
			6.5	686	170	516
			7.5	778	170	608
(6)	5	10	5.5	424	240	184
			6.5	516	240	276
			7.5	608	240	368
(7)	5	10	5.5	184	184	0
			6.5	276	276	0
			7.5	368	280	88

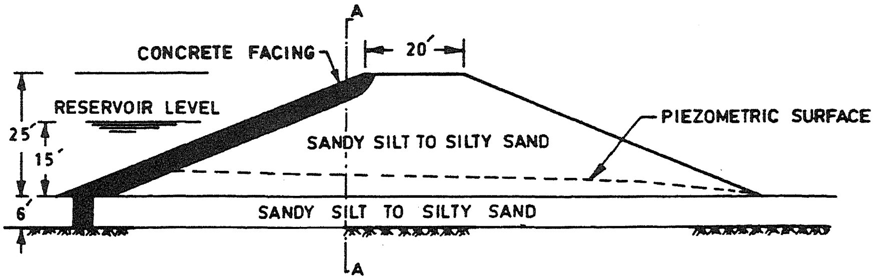

FIG. 1 _ CROSS SECTION THROUGH EMBANKMENT

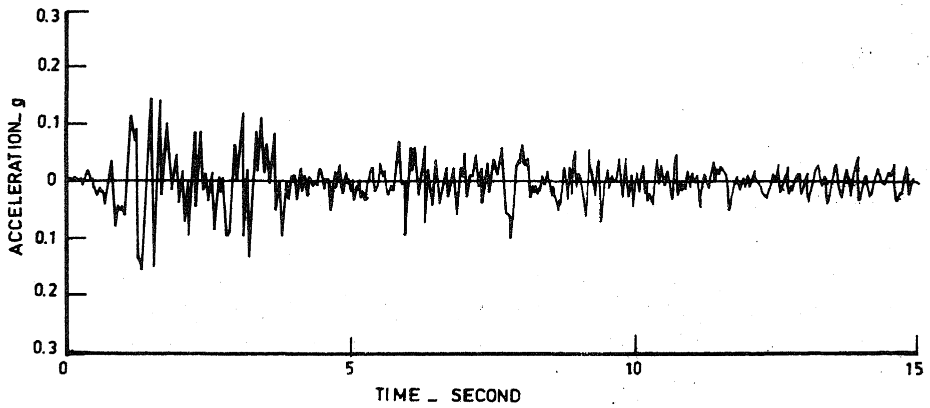

FIG. 2 _ ACCELEROGRAM

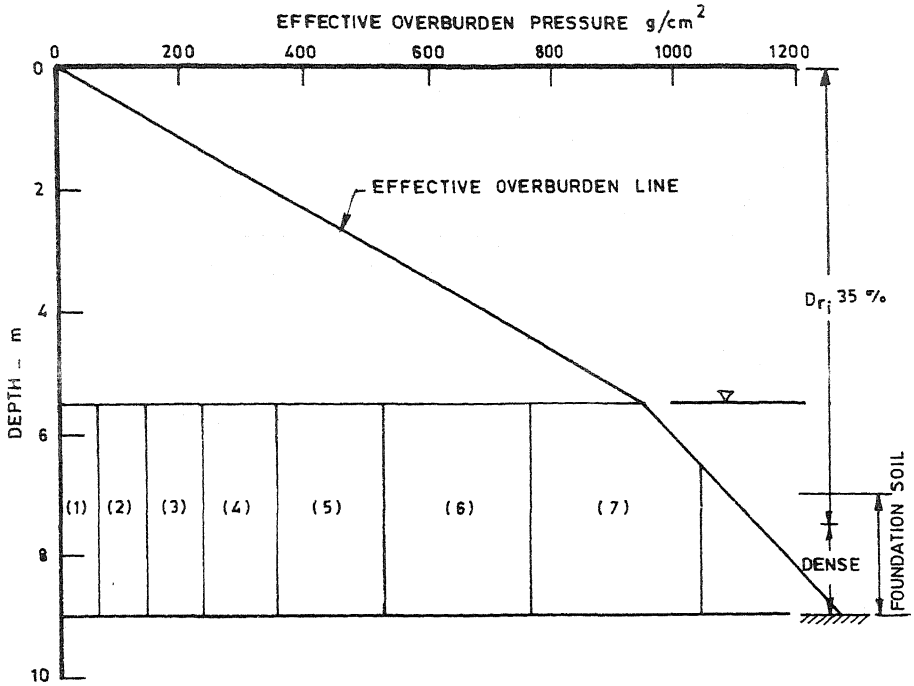

FIG. 3 _ LIQUEFACTION ANALYSIS AT SECTION A A

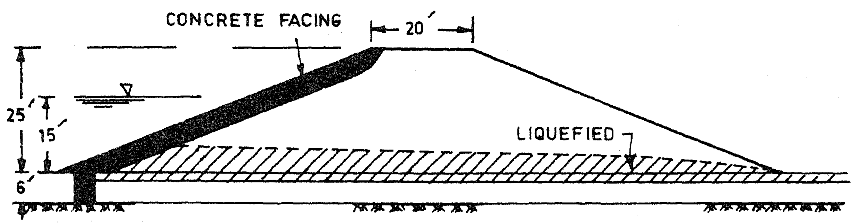

FIG. 4 _ ZONE OF LIQUEFACTION IN DAM DURING THE EARTHQUAKE