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SYNOPSIS: As a result of the shaking from the Loma Prieta Earthquake of 1989, the concrete lining of an earth embankment suffered cracking at numerous locations on its upstream side and developed in its bed lining, crater-like pot-holes ranging in size up to 10 ft. in diameter. Field investigation and preliminary studies indicated the differential dynamic vibrations of the earth embankment and the concrete lining to be the cause of the cracking, and the liquefaction of the underlying stratum below the bottom of the reservoir to be the cause of the pot-holes formed as a result of the pore pressure bursting through cracks in the lining. This paper presents a summary of the investigation and the related studies.

INTRODUCTION

Case histories of the experience of seismic shaking of earth dam embankments is of major interest to dam designers. Presented herein is such an experience of the FMC Test Pond embankment subjected to the 20 seconds long shaking due to October 17, 1989 Loma Prieta Earthquake. This earthquake affected the California's San Francisco Bay Area including San Jose where the embankment is located approximately 20 miles from the epicenter.

The dam embankment was constructed in 1959 by a cut and fill procedure of the native soil which consists of sandy silty clay with some gravels. A test pond reservoir was created by excavating the natural ground and using the resulting material to construct the surrounding embankment. A typical section of the embankment is shown in Figure 1. Each of the upstream and downstream slope is 2 horizontal to 1 vertical; and the crest width is 10 feet. The maximum height of the dam embankment is 21 feet. The upstream face is lined with 5 inch thick concrete which extends to the pond bed horizontally in the form of an apron. Details of the construction procedure are not available, but FMC officials stated that the embankment was compacted in six inches lifts to a 95 percent compaction of the optimum density based on Proctor test. The dam section is essentially homogeneous. Typical depth of water in the pond is 17.5 ft; and has a free board of 3.5 ft. The pond-reservoir is used for performing swim tests on U.S. Army Bradely tanks.

FMC Test Pond is located within a mile of the San Jose Municipal Airport and the Guadalupe River. At these locations, minor settlement of a tower foundation suggestive of liquefaction-related ground softening occurred, but this could not be confirmed. At another nearby site, where an evidence of probable liquefaction was observed (EERC, 1990), was on the east bank of the Guadalupe River. Minor lateral spreading and settlement caused minor cracking in the pavement of the airport frontage road at this location.

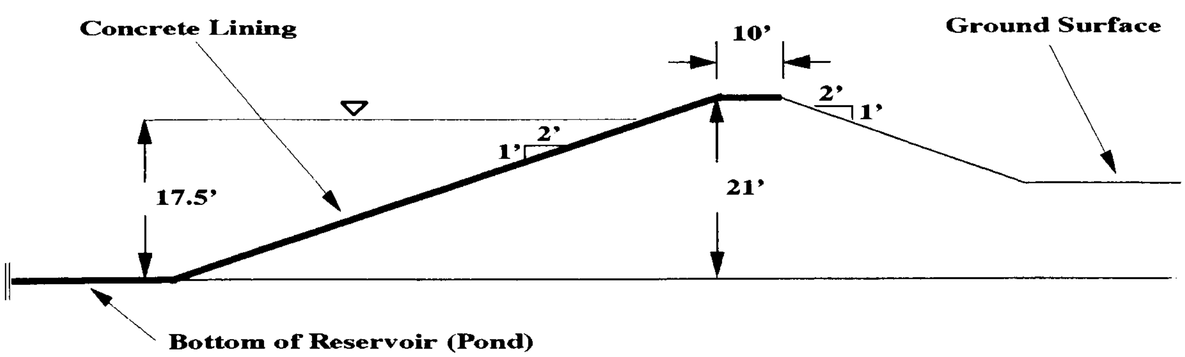

Figure 1: Cross Section of the Dam

On the other hand, the damage to the Test Pond was considered highly significant because of the seepage loss which caused an average draw down of 1 inch per day and was far in excess of the loss of water due to evaporation. Also observed were longitudinal cracks in the concrete lining of the upstream face of the embankment above the water level. Two drainage pipes located near the downstream toe of the embankment were observed to have tilted significantly.

This paper presents the results of investigations and studies made to evaluate the causes of excessive leakage and cracking of the lining.

FIELD INVESTIGATIONS

A field survey was performed to establish cross sections at pertinent locations where cracking or distress to dam was noticed. When the field measured cross section was superimposed on the original cross section of the dam, a bulging of the dam at the base was noticed (Fig 2).

earthquake shaking. Apparently, the cause of leakage did not appear to be related to the instability of the embankment. This necessitated the emptying of the pond reservoir completely for inspection of any cracking below the water table. The depth of water in the pond was about 17.5 ft.

EVALUATION OF DAMAGE

After the reservoir was emptied, a detailed inspection was performed to identify the extent and severity of the damage to the lining and to look for other potential sources of leakage. It was clear that the lining had developed numerous cracks; most conspicuous of which was a continuous crack at the heel almost along the entire length of the embankment. Yet the most striking observation of all was a number of crater like pot holes in the bed lining at the bottom of the reservoir. The maximum size of the crater like pot holes was measured to be approximately 10 feet in diameter (Fig. 3). It appeared that these craters might have been caused by the liquefaction and the high pore water pressure in the foundation soils resulting in a

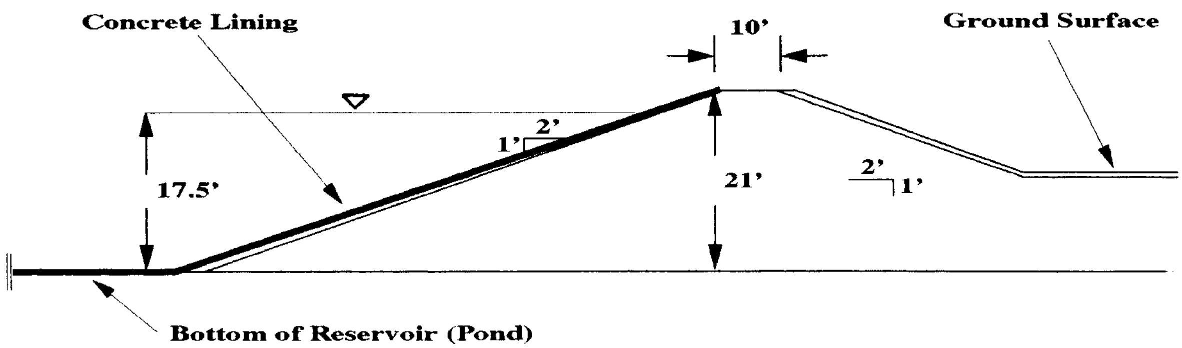

Figure 2: Superposition of As Built Cross Section over the Current Cross Section

A geotechnical field investigation by drilling bore holes and collecting undisturbed samples was carried out. Three bore holes were drilled to a maximum depth of 30 ft. Figure 3 shows the location of the bore holes. Two of these holes were drilled through the crest of the dam. California drive sampler was used for obtaining undisturbed samples. One of the bore holes (bore hole #3), drilled through the crest, was located where tilting of the drainage pipes was observed. The geotechnical investigation indicated silty clay to be the material the entire embankment is made of. Furthermore, laboratory testing indicated this silty clay to be highly plastic (PI = 41).

PRELIMINARY STABILITY ANALYSES

A psuedostatic stability analysis indicated a relatively high factor of safety of 2.14 for a seismic coefficient of 0.2g and for a strength value as low as 400 psf. Accordingly it was concluded that because of the absence of ground water table within the embankment of silty clay having an average shear strength of 3,000 psf, the dam remained generally stable during the

punching shear failure mode of the existing lining. Field measurements indicated that the existing concrete lining was 5" to 6" deep. Studies were then directed to analyze the cracking in the lining and the development of pot holes.

ANALYSIS OF CRACKING IN THE LINING

The rupture in the concrete lining in the longitudinal directions near the base and in the transverse direction near the southern bend of the test pond was believed to be related to differential seismic responses between the embankment and the concrete lining. Accordingly in an effort to have a preliminary understanding of the maximum seismic response of concrete lining, a highly simplified finite element model was developed. This model excludes any consideration of hydrodynamic interaction between the embankment and the test pond. Furthermore, the soil structure dynamic interaction effects are not fully recognized. The material properties were obtained or estimated on the basis of experimental results.

The base of the model was assumed as fixed. For the preliminary analysis the seismic loading specified was based on the UBC response spectrum with Zero Period Acceleration (ZPA) equal to 0.2 g horizontally. Effects of vertical component of the earthquake were disregarded.

The simplified finite element model was analyzed using PCI-SAP computer program (Pal Consultants, 1991) and the maximum displacement response indicated was on the order of 7 inches horizontal and 3 inches vertical: It was apparent from the order of displacement responses that concrete lining, owing

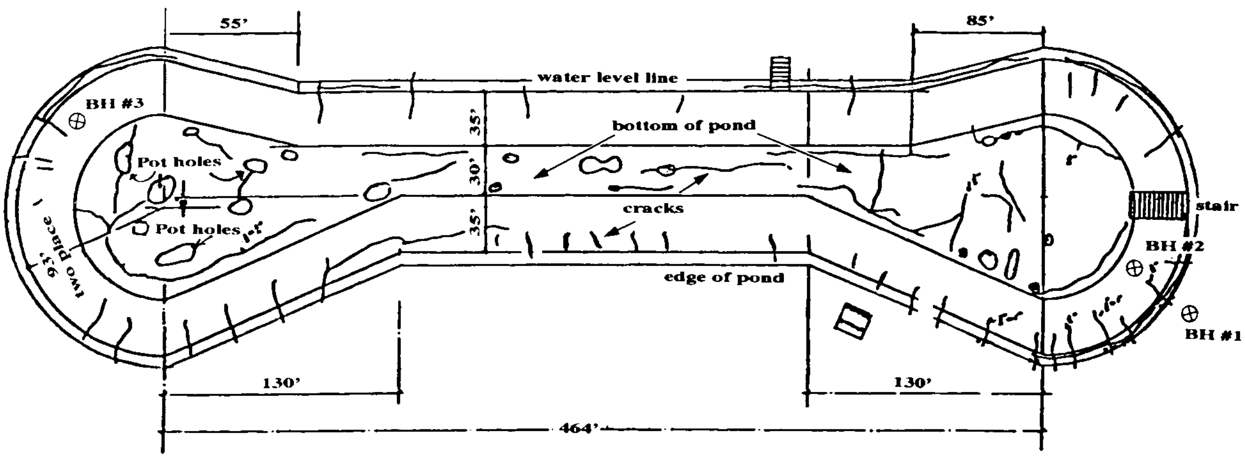

Figure 3: Mapping of Cracks & Craters (Pot-Holes) at FMC Test Pond

to its brittleness, undergoes certain limited cracking. The maximum stress responses are higher than the assumed dynamic tensile strength (=400 psi) near the base of the lining, indicating, thereby, the potential for cracking there. It may be pointed out that due to the limited modeling of the problem and the simplifying assumptions used in the preliminary analysis, the results are merely indicative of the order of displacement and stress responses. In view of the foregoing preliminary analysis, it follows that cracking in concrete lining of an embankment slope are significantly impacted by the differential dynamic vibrations of the earth embankment and the concrete lining. A more detailed modelling utilizing extensive soil testing and involving soil-structure and hydrodynamic seismic interaction effects is not completed in time for this writing.

ANALYSIS OF LIQUEFACTION

The occurrence of liquefaction due to the October 17, 1989 Loma Prieta Earthquake was quite wide spread in the San Francisco Bay Area. At many locations such as the San Francisco Marina, Oakland Airport and Bay Farm Islands Alameda, liquefaction phenomenon manifested itself in the form of sand boils on the ground surface. These sand boils are the result of the jetting out of soil-water slurry from underneath the ground where liquefaction occurs. The dynamic loading due to the earthquake shaking generates excess pore water pressures in the soil strat. Whereas a sloping ground or an embankment resting on liquefied strater can undergo undesirable movements until the excess pore water pressure drops sufficiently low and strength regain occurs to arrest the sliding; on a relatively level ground the excess pore pressure often bursts out in the form of a spring carrying with it the liquefied soil and depositing it on the ground surface in the form of a sand boil. Fine sands, silty sands & sandy silts are the types of soils which are most susceptible to liquefaction. As the reservoir at the FMC pond was emptied, three distinct crater-like pot holes with an approximate diameter ranging from 3 ft. to 10 ft. were observed. A careful examination of the sloughed out material in these pot holes was made. Several grab samples were obtained & washed to examine the type of gravels in the pot holes and the gravels which formed as a constituent of the concrete pad. Three borings were drilled through the pot holes and provided significant information on the nature and strengths of the subsoil. On the basis of all of the foregoing information, it appeared that the clayey fine sand and silty fine sandy clay to clayey silt underneath the concrete pad, liquefied or suffered a loss of strength during the earthquake shaking. Excessive pore pressures generated in these soils created uplift pressures underneath the lining, causing it to fail at the pot hole locations. The reason that such a failure mechanism was not observed at other locations is that at the pot hole locations a water leakage over the years through a possible crack in the concrete pad created an excessively wet condition in the soil immediately underneath the cracks. The seismic shaking generated high pore pressures such that the excessive force immediately underneath the concrete lining burst through it causing it to fail in punching shear. It may be noted that there are inconsistencies in the blow counts and the dry densities data of the liquefiable soils underneath the reservoir, for example boring #4 at ten ft. depth has a relatively high blow count of 14 but the dry density is quite low. Also, this soil has a low PI value of 10. Similarly in the case of boring #5, where as the blow count data of 5 and 7 around 2 ft. depth clearly indicates liquefaction; however, the soil has a relatively high dry density at these locations. We believe that this is due to the nature of the subsoil conditions which contain high percentages of silts with fine sands which have been shown to liquefy (Puri, 1984, Seed et al; 1984, Ishihara and Koseki, 1989 and Prakash and Sandoval, 1991).

CONCLUSIONS

On the basis of the preliminary studies made to evaluate the causes of excessive leakage and the damage to the lining of the concrete faced embankment of the FMC test pond, the following conclusions may be drawn:

1. The cracks in the upstream face of the lining are believed to have been caused by the differential dynamic vibrations of the earth embankment and the concrete lining. A detailed dynamic analysis is warranted to more clearly define the cause.

2. Possible liquefaction of the underlying soils appears to have caused the bed lining to fail in punching shear due to the uplift pressure.

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