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EARTHQUAKE INDUCED EXCESS PORE WATER PRESSURES IN THE UPPER SAN FERNANDO DAM DURING THE 1971 SAN FERNANDO EARTHQUAKE

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ABSTRACT

The excess pore water pressure developed in the Upper San Fernando Dam during the 1971 San Fernando Earthquake has been evaluated in several studies. Almost all of these studies indicate large excess pore pressure ratios developed only in the upstream and downstream shells which are not consistent with the limited deformation of the dam and the piezometer responses during the earthquake. In this paper, the construction and field observations of the behavior of the Upper San Fernando Dam are reviewed and a simple approach involving Newmark's (1965) and Makdisi-Seed's (1978) permanent deformation and limit equilibrium slope stability analyses are used to estimate the excess pore water pressures developed in the core and downstream shell areas during the earthquake for comparison with field measurements. The major differences of this analysis with previous studies lies in the assumptions regarding the selection of the failure plane, liquefiable zones, and mobilized shear strengths. The results explain the field piezometric observations and the limited displacement of the dam.

INTRODUCTION

Earthquakes have caused significant damage to dams and in some cases loss of the reservoir or impoundment (Casagrande 1965; Seed et al. 1969; Seed et al. 1975; Marcuson et al. 1979; Seed 1987). Given the importance of maintaining a reservoir, evaluating the stability and permanent deformation of dam slopes during earthquakes is vitally important in geotechnical engineering. Several methods (Kramer 1996) have been widely used to calculate the amount of seismically-induced deformations which a slope may undergo during an earthquake. The limited deformation experienced by the Upper San Fernando Dam (USFD) during the 1971 San Fernando Earthquake makes it an ideal case for the validation of these methods by many researchers (e.g. Seed et al. 1973; Wolfgang et al. 1993; Moriwaki et al. 1998; Beaty 2001; Wu 2001). However, the amount and pattern of displacements as well as the earthquake-induced pore water pressures estimated with these methods have not been in complete agreement with the observed behavior of USFD. For example, Moriwaki et al. (1998) modeled USFD using the Fast Lagrangian Analysis of Continua (FLAC) software package assuming a non-liquefiable core and predicted complete liquefaction of the upstream and downstream shells of the embankment, which led to unrealistic deformation patterns of the embankment. Wu (2001) uses the finite element method (FEM), again assuming a non-liquefiable core and found better agreement between the calculated and measured deformations of USFD. However, Wu predicted large pore water pressure ratios in the upstream and downstream shells. Beaty (2001) also used FLAC with a non-liquefiable core and underestimated the deformations of USFD as well as predicting complete liquefaction in the shells. The discrepancies among the results of these methods and the actual response of USFD is possibly due to the assumptions regarding the failure plane, strengths, and liquefaction potential of the materials in USFD. In this study assumptions are made about the seismic stability of USFD which better match the actual behavior and specifications of USFD and the measured displacements are used in conjunction with Newmark (1965) and Makdisi and Seed (1978) permanent deformation methods and limit equilibrium slope stability analyses to estimate the excess pore water pressures developed during the earthquake in USFD.

UPPER SAN FERNANDO DAM AND PERFORMANCE DURING 1971 EARTHQUAKE

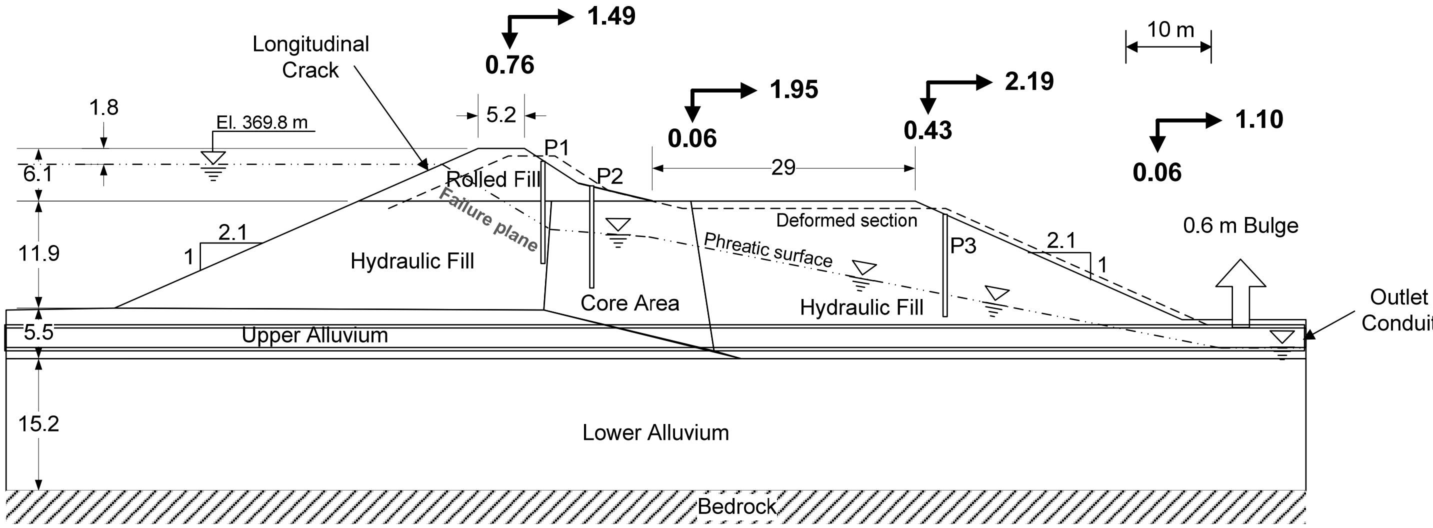


Fig. 1. Cross section of the USFD. The bold arrows indicate the displacements after the 1971 San Fernando Earthquake (Serff et al. 1976; Wu 2001) – all dimensions are in meters

USFD, built on San Fernando Creek northwest of Los Angeles, was completed in 1922 and was 25 m high and 530 m long (Seed et al. 1973). Although it was not constructed to its full intended height, a 5.5 m high rolled fill section was placed on the downstream portion of the hydraulic fill, leaving a 29 meter-long bench on the downstream slope. This gave the dam a wide profile for its height which was founded on 15 m of alluvial deposits overlying bedrock (Huynh et al. 2006). A typical cross section of the dam is shown in Fig. 1. Due to the lack of abundant water during construction, the semi-hydraulic fill method was used for USFD construction instead of the hydraulic fill method. In this method, dikes are constructed at the outer limits of the embankment to provide containment of the sedimentation pool. Borrow material is loaded by scrapers or excavators and transported to the site. The borrow material is then dumped on the inner slopes of the containment dikes. Afterwards, the borrow material is spread by sluicing it with a water cannon using water from a barge floating on the sedimentation pool. Similar to the hydraulic fill method, the finer material is transported down into the pool forming the core, and the coarser material is deposited on the outer slopes forming the shells. A central core of highly stratified sand, silt and clay layers was produced in USFD by the semi-hydraulic fill construction method (Harder et al. 1989).

On February 9, 1971 a 6.6 Richter Magnitude earthquake with a peak acceleration of 0.55g to 0.60g, and an epicenter at 13.5 km northeast of USFD hit the region (Scott 1973). This earthquake caused the USFD to move a maximum of 2.2 m (see Fig. 1) with the crest moving about 1.5 m downstream and formed several longitudinal cracks running the full length of the upstream face of the dam near the reservoir level (Serff et al. 1976). The reservoir level at the time of the earthquake was 1.8 m below the crest of the dam at an elevation of 369.8 m (Wu 2001). Although the reservoir was not lowered for some time after the earthquake, primary descriptions of the dam response do not mention any significant displacements of the upstream slope (Seed et al. 1973; Serf et al. 1976) which implies that liquefaction did not occur in the upstream slope. Shearing mostly occurred through the looser fine grained core and hydraulic fill material and there was little or no evidence of slide movement in the foundation alluvium because the relative density and cyclic strength of the alluvium was significantly greater than that of the hydraulic fill of the embankment under the same confining pressures (Seed et al. 1975).

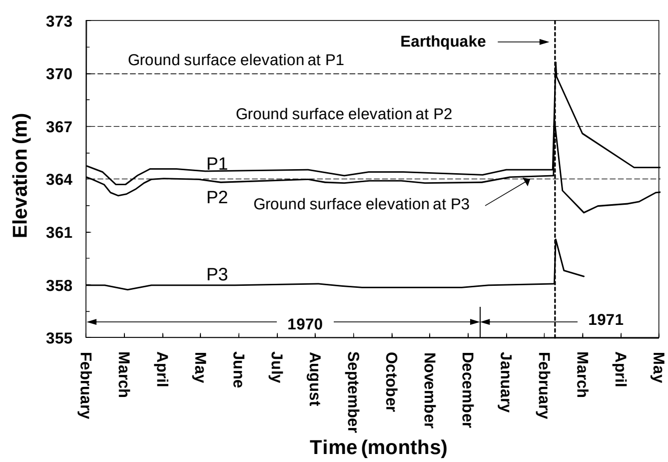


Fig. 2. Change in water levels in piezometers during and following the 1971 earthquake (after Serff et al. 1976)

Instrumentation of USFD consisted of three piezometers (observation wells) to locate the phreatic surface (see Fig. 1) and survey monuments embedded in the embankment to measure deformations prior to the 1971 earthquake (Serff et al. 1976). Figure 2 shows the sudden variation of the water levels in the piezometers before, during, and after the earthquake. The water level in piezometers P1 and P2 (indicating the pore water pressure in the central core area of the embankment) increased during the earthquake such that water exceeded the top of the piezometers. According to Fig. 2 the excess pore water pressure ratios at the time of the earthquake were at least 30.7%, 18.9%, and 12.0% in piezometers P1, P2, and P3, respectively.

NEWMARK'S SLIDING BLOCK ANALYSIS

One of the first methods to calculate the seismically induced permanent displacement of slopes was proposed by Newmark (1965). In this procedure it is assumed that slope movement is initiated if the acceleration applied to a potential slide mass is large enough to overcome the yield acceleration or shear strength of the involved slope materials. By computing the acceleration at which the inertia forces cause yielding and integrating the effective acceleration on the sliding mass in excess of this yield acceleration as a function of time, the velocities and permanent displacements of the slope can be calculated (Goodman and Seed 1966; Seed 1979).

Newmark's sliding block analysis has several limitations and shortcomings when applied to the seismic displacement of embankments including: not considering upslope movements (Ambraseys and Menu 1988), assuming rigid-perfectly plastic soil, ignoring the effects of deformability of the failure mass (Newmark 1965; Chang et al. 1984; Bray 2007), and neglecting the effects of rate- and displacement- dependent strength (Kramer 1996). Because of its simplicity and ease of use, it is used herein as an approximation and comparison with the Makdisi and Seed (1978) analysis. Because there were no accelerometers to record the acceleration of USFD during the earthquake, the motion recorded at the abutment of Pacoima Dam has been modified (Seed et al. 1973) and commonly used in the analyses of the San Fernando Dams (Scott, 1973; Seed et al. 1973; Seed and Harder 1990; Inel et al. 1993; Moriwaki et al. 1998; Wu 2001) and it is also used herein (see Fig. 3). Using this record and an average downstream sliding of about 1.6 m (see Fig. 1), a yield acceleration (k_y) of 0.014g is back-calculated using the computer code developed by Jibson and Jibson (2003) for the Newmark (1965) sliding block analysis.

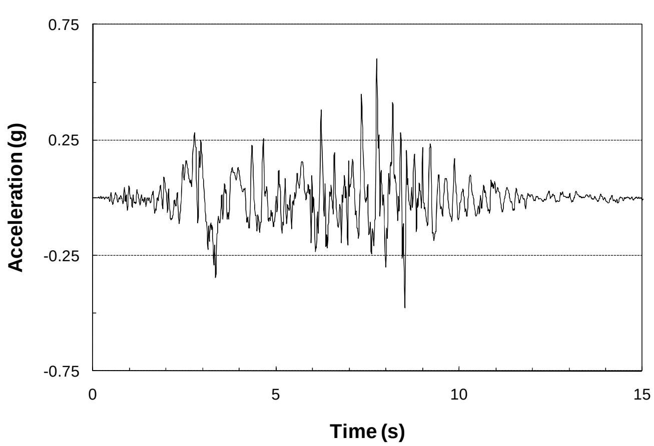


Fig. 3. Modified acceleration time history at the abutment of Pacoima Dam (Seed et al. 1973)

MAKDISI-SEED'S PERMANENT DEFORMATION ANALYSIS

Makdisi and Seed (1978) found that the peak average acceleration of a potential sliding mass decreases with increasing depth of the slip surface within the embankment and proposed a method for calculating the earthquake induced permanent slope deformation of earth dams based on the Newmark (1965) sliding block method. As opposed to the original Newmark (1965) rigid sliding block model which ignores the dynamic response of a deformable sliding mass, Makdisi and Seed (1978) introduced the concept of an equivalent acceleration to represent the seismic loading of a potential sliding mass based on the work of Seed and Martin (1966). In this method, the average peak acceleration coefficient ($k_{max\text{-}average}$) at the center of gravity of the sliding mass is estimated using the peak acceleration at the crest of the dam ($a_{max\text{-}crest}$), and the depth of the sliding mass (y) from Fig. 4. Then the permanent deformation of the embankment is estimated from Fig. 5 using k_y, $k_{max\text{-}average}$, and the earthquake magnitude (M). The direction of movement for a potential sliding mass once yielding happens is assumed to be in the downslope direction.

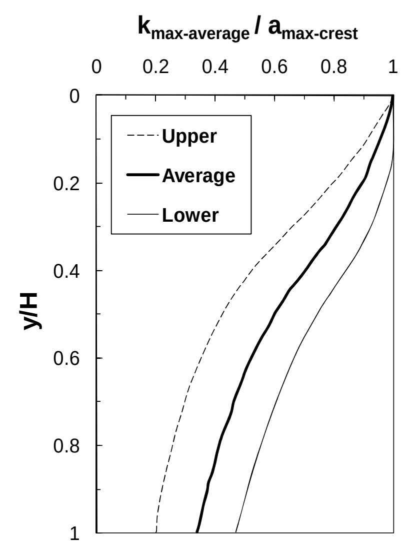


Fig. 4. Variation of the average peak acceleration coefficient ($k_{max\text{-}average}$) with depth of the potential sliding mass (y) (after Makdisi and Seed 1978).

Despite the many limitations of the Makdisi-Seed's method (Makdisi and Seed 1978; Chang et al. 1984; Ambraseys and Menu 1988; Bray 2007) it has provided reasonable estimates of seismic displacement for many cases (e.g. Lin and Whitman 1983; Rathje and Bray 2000) and is used herein to estimate k_y of USFD. The observed failure surface (Fig. 1) which extends through the entire height of the embankment (y/h = 1) is used in Fig. 4 and the upper bound, average, and lower bound values of $k_{max\text{-}average}/a_{max\text{-}crest}$ are found to be 0.470, 0.340, and 0.200, respectively. Then using $a_{max\text{-}crest}$ of 0.6g for USFD [from dynamic finite element analysis of Seed et al. (1973)], $k_{max\text{-}average}$ values of 0.120g, 0.204g, and 0.282g are calculated. Similar to Newmark's analysis, an average movement of 1.6 m is assumed for the downstream sliding mass according to Fig. 1 and upper bound, average, and lower bound values of k_y = 0.114g, 0.083g, and 0.049g, respectively are obtained for a 6.6 magnitude earthquake from Fig. 5.

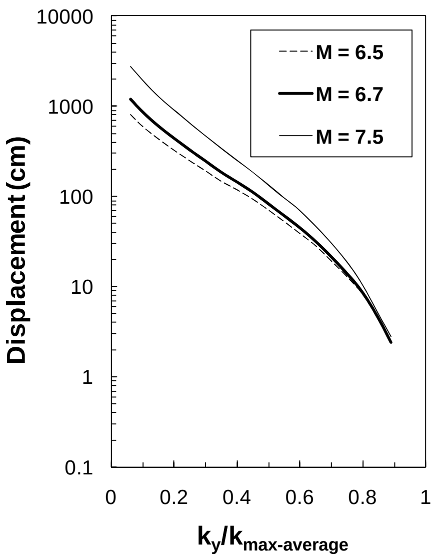


Fig. 5. Average values of the permanent displacement of an embankment caused by different levels of earthquake shaking (after Makdisi and Seed 1978).

EARTHQUAKE INDUCED EXCESS PORE WATER PRESSURE PREDICTIONS

The k_y defined from Newmark's and Makdisi-Seed's methods are used to find the excess pore water pressures which triggered failure and produced a factor of safety against slope stability (FS_S) of unity in USFD during the earthquake. Pseudo-static limit equilibrium slope stability analyses were performed by applying the k_y and using Spencer's (1967) stability method as coded in SLOPE/W (Geo-Slope International 2007).

Seed et al. (1973) and others who studied the USFD (Wolfgang et al. 1993; Moriwaki et al. 1998; Beaty 2001; Wu 2001) assumed an entirely clay core area which did not liquefy under cyclic loading (Seed and Chan 1966). Although their finite element analysis indicates a tendency to develop large strains in the core area (Seed et al. 1973) they did not consider any excess pore water pressures in the stability analysis of this area. As discussed before, the semi-hydraulic fill placement adopted in USFD produced a core area which was stratified with layers of loose sand, silt, and thin clay (Harder et al. 1989). The fine silt and clay layers would impede drainage of the excess pore water pressure developed in the loose sand layers and form water films (Kokusho 2003) at the base of the silt and clay layers significantly reducing the strength of the core area (Byrne et al. 2006). The considerable rise of pore water pressure in piezometers P1 and P2 (see Fig. 2) could be an indication of a similar phenomenon. The sand, silt, and clay layers would also mix during the failure process and this would further reduce the strength of the soil (Baziar and Dobry 1995). Therefore, in the slope stability analyses both the hydraulic fill and core area are treated as liquefiable materials.

Swaisgood (2003) examined observed crest settlements (as a parameter to represent earthquake damage) during past earthquakes and found that seismically induced crest settlements of earth and rockfill dams are largely dependent on the peak ground acceleration (PGA) at the base of the dam and earthquake magnitude (M). Other factors such as composition of the dam, whether earth or rock, had only minor effects on the observed vertical settlements. Figure 6 summarizes the observations reported by Swaisgood (2003). In this figure the observed settlements are expressed as a percentage of the embankment and foundation height and are related to the peak ground acceleration at the base of the dam. The triangular data points in Fig. 6 correspond to embankment dams constructed by hydraulic fill techniques. For USFD the peak ground acceleration is 0.60g and the observed crest settlement is 0.76 meters (about 2% of the total dam and foundation height) which is somewhat above the upper range of the observed settlements from other embankment dams shown in Fig. 6. This is probably caused by USFD being a semi-hydraulic fill dam in which the core was susceptible for pore water pressure generation and strength loss and therefore it is not surprising that the observed settlements plot close to those of hydraulic fill dams in Fig. 6.

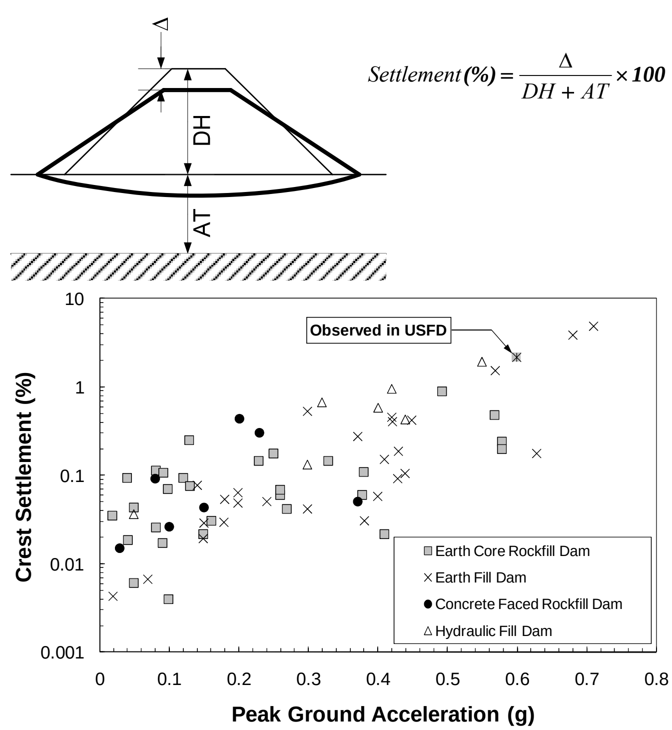


Fig. 6. Peak ground acceleration versus crest settlement (after Swaisgood 2003)

The overall displacement of USFD shows that the slide moved as a block (Huynh et al. 2006) and the longitudinal cracks running the entire upstream face of the dam (Serff et al. 1976) and the 0.6 m tall bulge observed at the downstream toe (Serff et al. 1976) indicate the extent of the sliding block. Also, axial compression (at the downstream) and extension (at the upstream) of the outlet conduit indicates that only minor shearing occurred at this depth and therefore could be the lower portion of the shear plane. Therefore, the potential failure surface is selected according to this deformation pattern and is shown in Fig. 1. Failure surfaces that have not complied with these observations have failed to capture the field deformation pattern (Moriwaki et al. 1998).

One of the most important parameters in predicting the displacements is the selection of appropriate strengths. While a high strength may provide too much resistance to deformation, a strength that is too low can reduce the ability of an element to transfer dynamic shear stresses to higher elements, or may affect the triggering calculations by reducing the peak stress that can occur in an element. Without any information about the pore water pressures, undrained slope stability analysis (USSA) can be used because the conditions at the triggering of the earthquake are sought before any large shear displacements occur. Thus, pre-liquefaction undrained yield strengths, s_u(yield), are required in these analyses. There has been a considerable interest in correlating s_u(yield) with in-situ test results such as SPT and CPT. This is because undisturbed sampling of liquefiable sands is difficult, if not impossible, and information on consolidation and aging history of cohesionless soil deposits is not readily available for properly reconstituting laboratory specimens. The s_u(yield) which is overcome to trigger liquefaction, are back-calculated by stability analyses of field liquefaction failures. Furthermore, the most useful information on undrained shear strength of contractive soils is expressed in terms of a ratio of undrained shear strength to consolidation pressure (Stark and Mesri 1992; Olson and Stark 2002), and the most appropriate consolidation pressure for normalizing s_u(yield) is the pre-consolidation pressure σ'_p, which is a point on the yield surface (Terzaghi et al. 1996). However, because information on σ'_p of cohesionless soil deposits is not readily available, s_u(yield) has been normalized by the pre-earthquake in-situ effective overburden pressure, σ'_{v0} (Olson and Stark 2003; Mesri 2007).

The first comprehensive set of data on $s_u(\text{yield})/\sigma'_{v0}$ was published by Seed et al. (1984) for liquefaction of level ground subjected to seismic shaking. However, if the pre-earthquake shear stress (τ_{c0}) applied during the consolidation stage increases (sloping ground) then the yield strength would increase but the seismic shear stress required to trigger liquefaction would decrease. Based on laboratory cyclic test data on liquefiable sands (Rollins and Seed 1990; Seed and Harder 1990), Terzaghi et al. (1996) suggest the following equation to obtain the mobilized yield strength ratio of a liquefiable sloping ground:

$$\frac{S_u(yield)}{\sigma'_{v0}} = \frac{\tau_{c0}}{\sigma'_{v0}} + 0.011\left(1 - 2\frac{\tau_{c0}}{\sigma'_{v0}}\right)(N_1)_{60} \quad (1)$$

in which $(N_1)_{60}$ is the dynamic standard penetration test blow count corresponding to a combined efficiency of 60%, normalised to an effective overburden pressure of 100 kPa (Skempton 1986; Terzaghi et al. 1996) . The effect of τ_{c0} on $(N_1)_{60}$ is indirectly included in Equation (1) based on back-analyses of observed field behavior.

Here, the pre-earthquake shear stress ratios (τ_{c0}/σ'_{v0}) on the failure plane are found by a static slope stability analysis. Figure 7 shows these values for each slice along the failure plane. The large value of $\tau_{c0}/\sigma'_{v0} = 0.379$ in the first slice (in the rolled fill) is caused by its steep base which increased τ_{c0}, and its intersection with the upstream slope and the resulting smaller triangular area which reduces σ'_{v0} on the failure plane. An average $\tau_{c0}/\ \sigma'_{v0} = 0.188$ is selected from Fig. 7 for the portion of the failure surface in the downstream hydraulic fill to be used in Equation (1) and in the slope stability analyses. This average value is in the range of those (0.1 to 0.3) from the thirty liquefaction flow failures of sloping ground studied by Olson (2001) and Olson and Stark (2003) and is close to the most typical value of 0.2 (Mesri 2007). Using an average $(N_1)_{60} = 17$ (Seed et al. 1973), $s_u(\text{yield})/\sigma'_{v0}$ is estimated to be 0.305 for the downstream hydraulic fill. The other input parameters for a USSA are provided in Table 1. $FS_S = 1$ is obtained using these parameters, with a horizontal seismic acceleration of 0.080g which is close to the average yield acceleration of 0.083g from the Makdisi-Seed’s method and indicates that the same amount of average displacement (1.6 m) would be produced with Makdisi-Seed’s approach. This further confirms the input parameters for USSA in Table 1. $s_u(\text{yield})/\sigma'_{v0}$ and effective friction angle (φ') of the core area have been obtained from small in-situ torvane tests and two consolidated undrained triaxial compression tests on samples taken from the dam (Seed et al. 1973). Without considering the excess pore pressures developed in the sand interlayers and perhaps water film formation (as done by Seed et al. 1973), using an average effective stress of 150 kPa on the part of the failure plane passing through the core area, substantially different shear strengths would result from these two parameters. This further indicates the importance of considering a liquefiable core area and including excess pore water pressures in its mobilized strength.

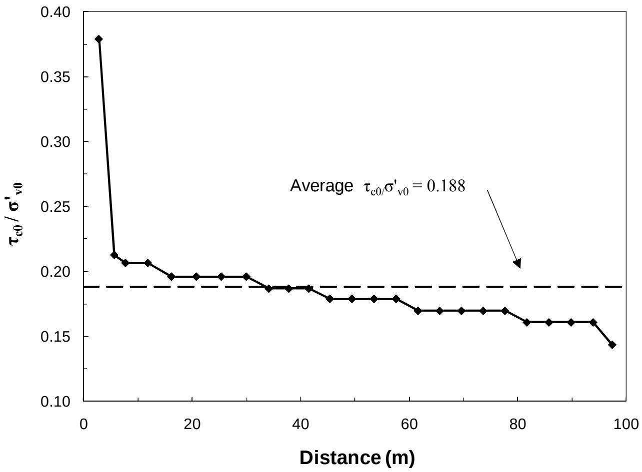


Fig. 7. τ_{c0}/σ'_{v0} versus distance on the failure plane from the upstream shell.

The main scope of this study is to estimate the excess pore water pressures developed during the earthquake using an effective stress stability analysis (ESSA). Because the average amount of excess pore water pressures developed in the core area and the downstream hydraulic fill could be considerably different and the average pore water pressure in only one of these zones can be estimated from each series of stability analyses, two series of analyses are performed. In the first series, USSA is performed for the core area, and ESSA is used in the downstream hydraulic fill to find the range of the excess pore water pressure ratios (r_u) in the hydraulic fill which produce $FS_S = 1$. In the second series, USSA and ESSA are used for the downstream hydraulic fill and core area, respectively and the range of the triggering r_u is estimated for the core area.

TABLE 1. Input parameters used in the slope stability analysis of the USFD (from Seed et al. 1973)

Soil	$\gamma_{saturated}$ (kN/m^3)	USSA			ESSA
		τ_{c0}/σ'_{v0}	$(N_1)_{60}$	s_u/σ'_{v0}	φ' (°)
Rolled fill	22.0	0.379[1]	25	0.444[2]	-
Hydraulic fill	19.2	0.188[1]	17	0.305[2]	37
Core area	19.2	-	-	0.240	37
Alluvium	20.3	0	55	0.600[2]	-

[1] From Fig. 7.
[2] Calculated from Equation (1).

Figure 8 shows the factors of safety from the first series of the slope stability analyses. According to this figure the range of triggering r_u from ESSA with k_y corresponding to the Makdisi-Seed's method for the hydraulic fill is 47% to 33% which is larger than r_u (= 12.0%) observed in piezometer P3. It could be that since piezometer P3 was not extended far enough into the hydraulic fill and the failure plane, the pore water pressure which it was measuring was likely less than that required to trigger failure on the failure plane.

Figure 9 shows the factors of safety and the range of r_u for the core area from the second series of analyses. The range of r_u = 31% - 12% corresponding to the k_y values from Makdisi-Seed's approach capture the range observed in piezometers P1 and P2 (30.7% - 18.9%). These piezometers were deep enough to observe the pore water pressure in the failure plane and although water overflew from both of them but the agreement here indicates that the overflow was not significant.

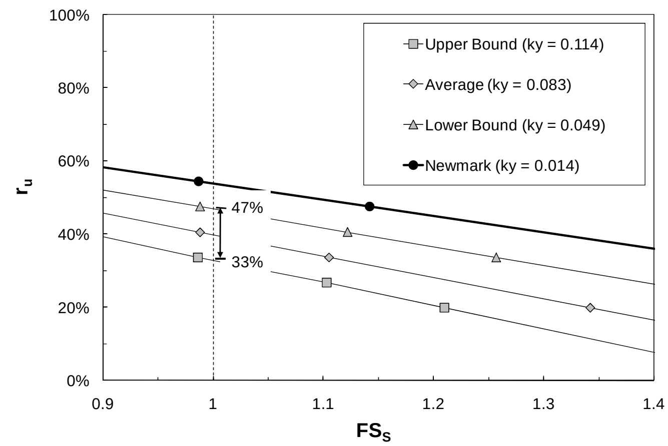


Fig. 8. FS_S for different r_u values in the downstream hydraulic fill corresponding to upper bound, average, and lower bound values of k_y from Makdisi-Seed's method and k_y from Newmark's analysis.

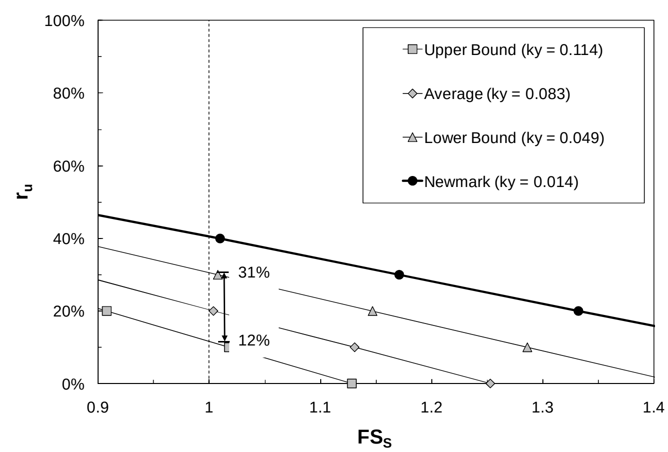


Fig. 9. FS_S for different r_u values in the core area corresponding to upper bound, average, and lower bound values of k_y from Makdisi-Seed's method and k_y from Newmark's analysis.

Without any excess pore water pressure generation (r_u = 0), the FS_S according to Figs. 8 and 9 are all above unity and would not trigger failure. This indicates that it was not only the inertial forces which were the fundamental cause of the deformations, but also the excess pore pressures and strength loss were the root causes of the deformations. Moreover, the unrealistic assumption of not considering the dynamic response and deformability of the deep sliding mass in USFD by the rigid sliding block method of Newmark necessitated a much lower yield strength (k_y = 0.014) in order to produce the average displacement of 1.6 m and this lead to larger r_u in the slope stability analyses corresponding to FS_S = 1 in Figs. 8 and 9.

Figure 10 shows ranges of r_u for factors of safety against liquefaction (FS_{Liq}) from laboratory experiments on sands (Tokimatsu and Yoshimi 1983). According to this figure, FS_{Liq} corresponding to the r_u developed in the core and hydraulic fill areas are all above one (1.03 – 2.1) and although it is possible that complete liquefaction may have had happened in limited areas of the hydraulic fill where FS_{Liq} = 1.03 but the overall range of r_u developed in USFD was not likely sufficient to cause complete liquefaction and failure of the dam and explains the limited movement of USFD. These low r_u values and the limited movements can be attributed to the massive section of the USFD, the semi-hydraulic filling method used to construct the dam which produced a denser [relative density = 45% - 70% according to Seed et al. (1973)] deposit in comparison to the hydraulic filling method used for Lower San Fernando Dam, and the dilative trend of the coarser outer shells (as indicated in the triaxial tests on this material in Fig. 11) which reduced the r_u values and the material became stronger as movement continued through the shell. Subsequently, there could have been a tendency for redistribution of the pore water pressures as the water moved towards the potential shear zone and areas of lower water pressures from the areas of higher water pressures. If water had been released from the reservoir of the USFD after the earthquake, it could have caused overtopping of the remainder of the Lower San Fernando Dam and considerable damage and loss of life could have resulted.

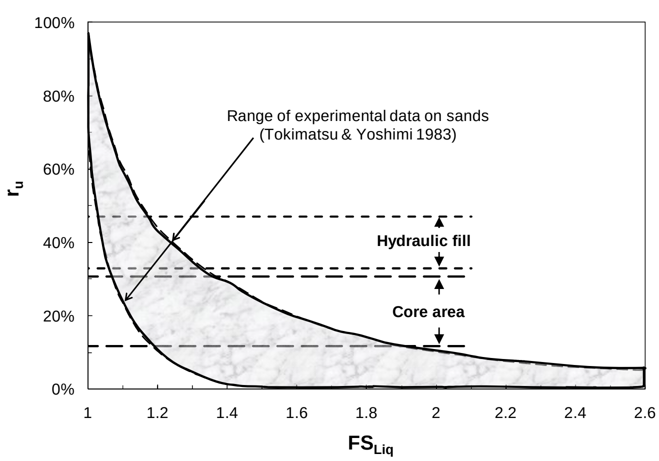
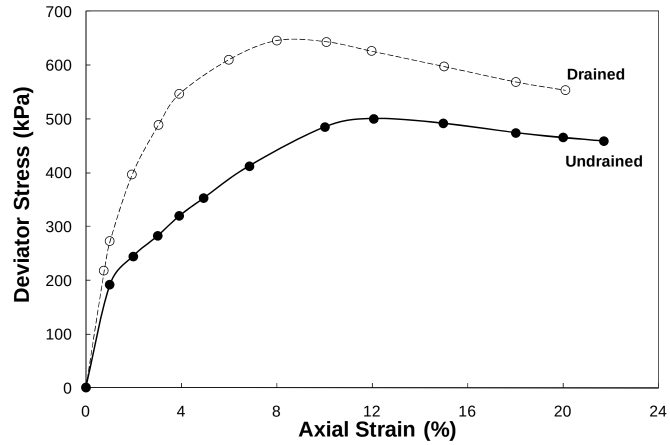
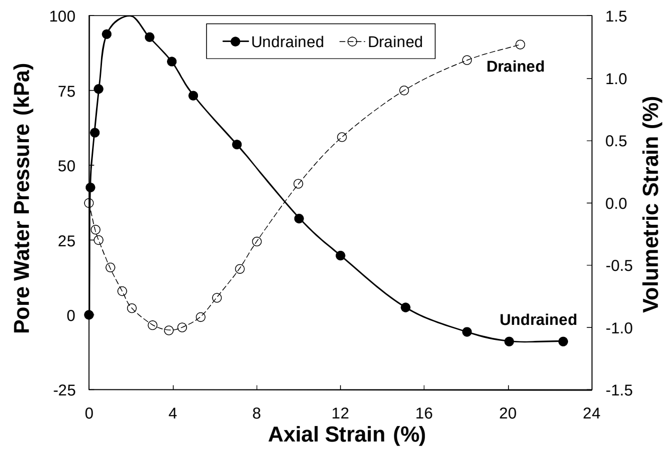


Fig. 10. FS_{Liq} for different values of r_u.



(a) Stress-strain behavior



(b) Volumetric-strain response

Fig. 11. Undrained and drained triaxial compression behavior of the hydraulic fill silty sand from USFD consolidated to a stress of 196 kPa (after Seed et al. 1973)

CONCLUSIONS

One of the best ways to corroborate soil properties is to compare predicted performance with field observations. This is particularly important when there is limited laboratory testing and field measurements. In this study, the excess pore water pressures developed in the Upper San Fernando Dam during the 1971 San Fernando Earthquake are estimated using the actual displacement of the dam with Nemarks's and Makdisi-Seed's permanent deformation methods combined with pseudo-static limit equilibrium analyses. Because of the interbedded sand layers within the fine silt and clay it was assumed that the central core area was liquefiable, and a failure plane which corresponded to the deformation pattern of USFD was selected. The predicted range of the excess pore water pressure ratios (r_u) in the core area from the Makdisi-Seed's method (31% - 12%) agree well with those indicated by the piezometers installed in the dam. However, the range of r_u predicted in the hydraulic fill (47% - 33%) is larger than the observed value (12%) which could be due to the fact that the piezometer was not deep enough to measure the pore water pressure developed in the failure plane. None of these excess pore water pressure ranges would produce complete liquefaction of USFD.

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