LIQUEFACTION EVALUATION OF EARTH DAMS – A NEW APPROACH

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SUMMARY

A new approach is presented for the seismic liquefaction evaluation of earth dams. The central concept is that: a) pore pressure buildup in the sand is controlled by the horizontal cyclic shear strain, $\gamma_C$; while b) flow failure is determined by the static shear stress, $\tau_s$, typically acting on a non-horizontal plane. A new type of laboratory test is described which simulates consecutive stages a) and b). A simplified procedure for determining $\gamma_C$ for a given design earthquake is presented, accounting for the nonlinearity of soil modulus and damping and for the stiffening effect of the rock abutments in narrow canyons.

INTRODUCTION

Pore pressure buildup in loose saturated sands and cohesionless silts within earth dams and embankments due to seismic shaking has repeatedly led to liquefaction flow failures or near failures (Refs. 1,2,3,4,5). Current techniques of liquefaction evaluation are based on the cyclic stress approach. The method usually combines the results of dynamic plane strain finite element (FE) analyses with stress-controlled, undrained cyclic triaxial tests on specimens anisotropically consolidated under stresses, $\sigma_{10}$ and $\sigma_{30}$, with $K_c = \sigma_{10}/\sigma_{30} > 1$ (Ref. 2). The cyclic strength of the sand is defined as the cyclic shear stress required to cause a specified accumulated axial strain in a given number of cycles. To evaluate the stability of the dam, this cyclic strength is compared with the cyclic shear stress, $\tau_C$, induced by the earthquake.

PROPOSED NEW APPROACH

The central concept of the new approach is that there are two different consecutive stages in the flow failure of an earth dam: a) a first stage of pore pressure buildup, mainly controlled by the level and duration of the seismic (cyclic) shear strains, $\gamma_C$, induced by the horizontal excitation at the base of the dam; and b) a second, flow failure stage driven by the static shear stresses, $\tau_s$, associated with the weight of the dam, which act on a soil softened by the earthquake-induced pore pressures. The existence of these two stages is illustrated by the fact that the slides in both the lower San Fernando Dam and the embankments in the Ojika, Japan earthquake occurred after
the main earthquake shaking had ended (Ref. 3). While typically the cyclic shear stress, \( \tau_c \), and strain acting on a soil element are horizontal, the static shear stress driving the slope failure, \( \tau_s \), is usually inclined and located along a potential sliding surface such as sketched in Fig. 1. This is in contradiction with the stress-controlled cyclic triaxial tests described above, where both \( \tau_s \) and \( \tau_c \) act on the same plane, which is inclined 45° with the horizontal.

![Diagram of stress conditions in an Earth Dam During an Earthquake](image)

**Fig. 1  Stress Conditions in an Earth Dam During an Earthquake**

\( (K_c = \sigma_1/\sigma_3 = \sigma_{10}/\sigma_{30} > 1) \)

Once the existence of the two stages is recognized, available experimental evidence can be used for a better understanding of each stage. The cyclic shear strain, \( \gamma_c \), is more fundamental than \( \gamma_t \) for the buildup of pore pressures (Refs. 6,7). Also, in both isotropically and anisotropically consolidated sand there is a threshold shear strain, \( \gamma_t \), such that if \( \gamma_c < \gamma_t \) there is no pore pressure buildup; for most clean quartz sands, \( \gamma_t = 10^{-2} \) (Refs. 6,8). This suggests using cyclic strain-controlled tests to generate pore pressures in the laboratory. Finally, it has been shown (Refs. 9,10,11) that flow failure can only occur in contractive sands.

The authors have developed a new type of axial/torsional triaxial test to simulate the two stages a) and b). In this strain-controlled CyT-CAU test (see Fig. 1), the static shear stress, \( \tau_s = q_s = 0.5 (\sigma_{10} - \sigma_{30}) \) is applied on a 45° plane by consolidating the specimen with \( K_c = \sigma_{10}/\sigma_{30} > 1 \). Then, a torsional horizontal shear strain, \( \gamma_c \), is applied in undrained condition (stage a). If the specimen is contractive, after a certain number of cycles, \( n \), flow failure is triggered, driven by \( \tau_s \) (stage b).

**DESCRIPTION OF METHOD**

The main steps of the proposed method are:

1) Determine the static principal effective stresses, \( \bar{\sigma}_1 \) and \( \bar{\sigma}_3 \) in the dam before the earthquake by using FE analyses, flow nets, etc.

2) Determine within the dam the zone(s) of contractive sand, where \( \sigma_3 > \bar{\sigma}_{3us} \). (The subscript 'us' denotes 'undrained steady state'.) The value of \( \bar{\sigma}_{3us} \) for a given sand (Refs. 10,11) depends only on the void ratio, \( e \), of the sand, and a \( \sigma_{3us} \) versus \( e \) steady state line (SSL) such as in Fig. 2(a), can be obtained from monotonic, load controlled CTB or CAU triaxial tests.

3) Determine the residual or steady state shear strength, \( S_{us} = 0.5 (\bar{\sigma}_{1us} - \bar{\sigma}_{3us}) \) of the sand, also a function of \( e \) only. The \( S_{us} \) SSL can be
obtained from the same triaxial tests as in 2) above. In general the $\bar{\sigma}_{\text{us}}$ SSL and the $S_{\text{us}}$ SSL are parallel (Ref. 12) and thus the residual or steady state strength envelope is defined by $\bar{\sigma}_{\text{us}}$ (Fig. 4), as follows:

$$\frac{q}{p_{\text{us}}} = \tan \bar{\sigma}_{\text{us}} = \sin \bar{\sigma}_{\text{us}} = \frac{(S_{\text{us}}/\bar{\sigma}_{\text{us}})/[1 + (S_{\text{us}}/\bar{\sigma}_{\text{us}})]}{1}$$ (1)

where $q = 0.5 (\sigma_1 - \sigma_3)$ and $p = 0.5 (\sigma_1 + \sigma_3)$. Values of $\bar{\sigma}_{\text{us}}$ are given in Table 1 for several sands (Ref. 12). Table 1 suggests that $\bar{\sigma}_{\text{us}}$ may increase with increasing angularity of the sand grains. It also indicates that a value of $\bar{\sigma}_{\text{us}} = 26.5^\circ$ ($S_{\text{us}} = 30^\circ$), corresponding to $S_{\text{us}} = \bar{\sigma}_{\text{us}}$, can be used for preliminary design. This is useful, as $S_{\text{us}}$ is relatively difficult to measure.

4) Perform a static, regular slope stability analysis of the dam, using $S_{\text{us}}$ as the shear strength of the contractive zone(s). This is conservative and assumes that for the whole contractive zone(s), pore pressures and strains have increased enough to drop the shear strength to the value $S_{\text{us}}$.

5) If the slope stability analysis in 4) indicates that the dam may be unsafe, the next step is to make a preliminary determination of the cyclic shear strains, $\gamma_c$, induced by the design earthquake. A simplified method for this determination is discussed in the next section. If most of the sand within the dam is subjected to $\gamma_c \leq \gamma_t$, the liquefaction evaluation ends here, and the dam can be considered safe.

6) If a significant portion of the cohesionless soil in the dam experiences $\gamma_c > \gamma_t$, a detailed investigation is necessary. This may include refining the calculation of $\gamma_c$ by dynamic FE analyses, and conducting a series of CyT-CAU
tests on soil specimens with void ratios and consolidation effective stresses covering the range within the dam. The results of a typical CyT-CAU test performed by the authors on a contractive specimen of Banding Sand is shown in Fig. 3. In the figure, during the undrained cyclic torsional straining, the pore pressure increased, while the static shear stress, $\tau_s$, remained constant. At point $T$, after $n = 109$ cycles, unidirectional flow deformation was triggered; this rapid flow failure was driven by the axial deviator load, not by the cyclic torsional load. As a consequence, the axial strain increased more than 15% in 0.32 sec.; points $S$ in the figure correspond to the steady state condition, with large deformations and flow of the sand under constant effective normal ($\sigma_{3us}$) and shear ($S_{us}$) stresses. In the tests performed by the authors, flow failure has invariably triggered when the effective stress path reached the $\sigma_{us}$ envelope, as shown by Fig. 4. The effective stress path for the same test shown in Fig. 3 is presented within the box in Fig. 4. Both points $T$ and $S$ lie on the same envelope. The values of $\sigma_{3us}$ and $S_{us}$ for points $S$ are the same it obtained from cyclic CyT-CAU tests or from monotonic CIU or CAU tests.

Fig. 3 Typical Stress-Strain-Pore Pressure Response During a CyT-CAU Test on Banding Sand ($D_r = 23\%$, $\sigma_{30} = 5.5$ kg/cm$^2$, $K_c = 2$)

Fig. 4 Effective Stresses at Triggering of Flow and During Steady State Flow From CyT-CAU Tests on Banding Sand, $D_r = 12\%$ to $25\%$
The excess pore pressure ratio, $u^* = \Delta u/\sigma_{yy}$, induced by $\gamma_c$ is a function of $K_c$, $\gamma_c$ and $n$. When $u^* = u^*_T$, the stress path reaches the failure envelope, and flow failure is triggered if the soil is contractive, with $u^*_T$ given by:

$$u^*_T = \frac{1}{2} \left[ K_c + 1 - (K_c - 1)/\tan \bar{\alpha}_{us} \right]$$

(2)

Therefore, for a soil element in a contractive zone within the dam, the value of $u^*_T(\gamma_c, n, K_c)$ at successive times during the earthquake must be compared with $u^*_T$ for that element. When $u^* = u^*_T$, the soil element is assigned a shear strength equal to $\sigma_{us}$. A static slope stability analysis is conducted to evaluate the danger of flow failure at the end of the shaking, and can also be performed at different times during the earthquake. A factor of safety of 1 or less predicts flow failure of the dam.

**DETERMINATION OF SEISMIC SHEAR STRAINS**

The shear strains induced within an earth dam subjected to horizontal earthquake excitation depend on a number of factors, including soil properties, dam-canyon geometry and ground motion characteristics. The soil properties include the shear modulus, $G$, and the damping of the soils within the dam; both properties are strain dependent.

A simplified analysis of a typical dam subjected to horizontal excitation (Ref. 13) was performed by the authors using the shear beam model. The dam was idealized as an infinitely long, viscoelastic, truncated, almost triangular wedge deforming only in shear and thus experiencing only horizontal seismic shear stresses and strains. The model assumes a uniform distribution of horizontal lateral displacements and of shear stresses and strains across the width of the dam. To account for the soil shear modulus dependence on confining pressure, the shear modulus is taken as:

$$G(x) = G_b \left( \frac{x}{H} \right)^\mu = G_b y^\mu$$

(3)

where $G_b$ is the shear modulus at the base of the dam, $H$ = height of the dam, $x$ = depth from the crest and $\mu$ is a constant coefficient to express inhomogeneity.

Considerable field and analytical evidence indicates that $\mu = 2/3$ in actual dams (Fig. 5; Refs. 14, 15). For $\mu = 2/3$, the expression for the shear strain $\gamma_j$ at the normalized depth $y = x/H$ corresponding to the $j$th mode is:

$$\gamma_j(y) = -\frac{2}{3y} \left[ y^{-2/3} \sin[a_j(1 - y^{2/3})] + a_j\cos[a_j(1 - y^{2/3})] \right]$$

(4)

where $j = 1, 2, 3, \ldots$, and $a_j$ is the eigenvalue of the $j$th mode (the values of $a_j$ for the first six modes are tabulated in Ref. 16). The data points in Fig. 6 show the distribution with depth of the maximum seismic shear strain, $\gamma_{max}$, normalized with respect to the strain at the base, $\gamma_b$, and corresponding to four historic input earthquake records. These strains in Fig. 6 were calculated using time history earthquake analyses. Although Fig. 6 has some scatter, it is quite consistent and an average representative curve could be defined. The
maximum shear strain at the base, \( \gamma_b \), can be approximated by:

\[
\gamma_b = \left[ \gamma_{b1} + \gamma_{b2} \right]^{1/2} ; \quad \gamma_{b1} = \frac{3pgH}{G_b}, \quad \gamma_{b2} = \frac{A_i}{a_j} \cdot \left( \frac{S_a}{A'} \right) T_j
\]

(5)

where \( T_j = \frac{3\pi H}{a_j} (\rho/G_b)^{1/2} \)

(6)

and \( j = 1, 2 \); \( (S_a/g)T_j \) is the acceleration spectral value corresponding to the modal period \( T_j \) for the selected damping ratio; and \( \rho \) is the mass density of the soil. \( A_i \) is a dam geometry coefficient; for usual dams, \( A_i = 1 \). A comparison of \( \gamma_b \) obtained from the same four time history analyses of Fig. 6 with those estimated using Eq. 5 showed that Eq. 5 is accurate within 15%. Once \( \gamma_b \) is found, the distribution of \( \gamma_{\text{max}} \) with \( x \) can be estimated with the average curve of Fig. 6.

The use of Eq. (5) and Fig. 6 to estimate \( \gamma_b \) and \( \gamma_{\text{max}} \) versus \( x \), will in general require consideration of the nonlinearity of the soil modulus \( G_b \) in Eqs. (5) and (6) as well as of the damping ratio. Fortunately, both soil parameters change very slowly with shear strain and, thus, the variation of \( \gamma_{\text{max}} \) with depth in Fig. 6 can be neglected and an average maximum shear strain, \( \gamma_{\text{ave}} = 1.75 \gamma_b = 1.75 (\gamma_{b1}^2 + \gamma_{b2}^2)^{1/2} \), can be used for the whole dam. The use of this constant \( \gamma_{\text{ave}} \) preserves the validity of \( g = G_b(x/H)^{2/3} \) and, thus, allows use of the same analytical solutions, but with \( G_b \) adjusted to be consistent with \( \gamma_{\text{ave}} \). Because \( \gamma_{\text{ave}} \) is a peak seismic strain value, it is convenient to select a fraction \( \delta \gamma_{\text{ave}} = 1.75 \delta \gamma_b \), where \( \delta < 1 \), to enter the curves of \( G_b \) and of damping ratio versus constant cyclic shear strain. Iterations are needed to find \( \gamma_b \), \( G_b \) and damping ratio in this simplified procedure, as is usually done for soils in equivalent linear response analyses.

Once convergence occurs, the desired value of \( \gamma_b \) is obtained, and \( \gamma_{\text{max}} \) for different elevations is determined with the curve of Fig. 6.
For dams located in narrow canyons the assumption of plane strain (Ref. 20) used in the shear beam model may not be valid. The rigid rock boundaries stiffen the whole dam and alter its dynamic response. Idealized canyon shapes are illustrated in Fig. 7, which shows the effect of the L/H ratio on the fundamental period of a homogeneous (μ = 1) dam. In Fig. 7, Tps corresponds to the shear beam, plane strain case, L/H = ∞. Figure 7 can be used as a guide to modify T1 (and T2) for the more realistic inhomogeneous model of Eq. (6), so as to include the stiffening effect of the narrow canyon on Ymax.

Once Ymax has been obtained, a representative cyclic shear strain, Yc = δ Ymax, is calculated for each depth x of the dam. These values of Yc, in conjunction with the number of equivalent strain cycles, n, and of the corresponding Cyt-Cau test results, can then be used to predict the pore pressure ratios, u′, at the end of the earthquake, as discussed in steps 5) and 6) of the previous section.

![Figure 7: Effect of Canyon Shape on the Fundamental Period of the Dam, T1 (μ = 1)](image)

CONCLUSIONS

A new approach has been presented for seismic liquefaction evaluation of earth dams. Field evidence, new laboratory results and simplified analyses strongly suggest that the approach is more realistic than the present state-of-the-art. Especially important is the explicit recognition that the pore pressure buildup prior to failure is controlled by the seismic shear strains, while the flow slide itself is driven by the static shear stresses.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance of Messrs. Gregory E. Thomas and Steven D. Thorne in conducting the laboratory tests presented in Figs. 2, 3 and 4. This work was performed under U.S. National Science Foundation Grant No. CEE-8205345.

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