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SYNOPSIS The paper describes the investigation of the seismic safety of a 131 m high rockfill dam presently under construction across the Rio Chixoy in Guatemala. The dam is located in a region of high seismicity, the recently active Motagua fault being only 40 km distant. Due to the possibility of high peak ground accelerations it was considered necessary to conduct a full dynamic analysis, which was carried out in stages following the Seed-Lee-Idriss method, the latter being modified where it was deemed necessary.

INTRODUCTION

The investigation of the seismic stability of a dam involves three phases: the assessment of the seismicity of a site; the choice of a mathematical model for the dynamic analysis, which requires both accelerograms and material properties as input data; and the evaluation of the effects of the loading on the stability of the dam according to the criteria of limiting deformation or slides.

REGIONAL SEISMICITY FOR THE SITE

The seismicity of the given region is influenced by the strong but very deep earthquakes originating from the subduction zone running parallel to the Pacific Ocean, by volcanic activity in the nearby mountain range and by the active fault systems (Cuilco-Chixoy-Polochic (CCP) and Motagua) located about 6 km and 40 km respectively from the site. The disastrous Guatamala earthquake of 1976 was due to activity on the Motagua fault, which marks the boundary between the North American and the Caribbean Plates. For this event the magnitude on the Richter scale was $M = 7.5$. To obtain a design earthquake either a deterministic or a probabilistic approach could be taken, but the former was considered to be decisive here on account of the potential danger to human life of a dam failure. For the Motagua fault the maximum credible event is regarded as having an upper limit of magnitude $M = 8.5$, whereas for the CCP fault it is thought that due to its great length $M = 7.7$ to 8.0. Each fault is assigned an estimated peak acceleration on the basis of empirical relations. Since there are no acceleration records for an event $M = 8.0$ at the close proximity (6 km) to the dam site of the CCP fault, and generally there is very little data available even at epicentral distances of a few tens of km for near field $M > 8$ events (Idriss, 1979) it was necessary to extrapolate existing data cautiously. On this basis the peak acceleration was assessed to be 0.65 g. The artificial 60 sec duration seismogram developed by Seed and Idriss was selected which exhibits high amplification over a large range of periods (Bossoney and Dungar, 1980).

STATIC ANALYSIS

Although the design of an earth dam is based mainly on limit equilibrium principles the evaluation of its seismic safety necessitates a good knowledge of the pre-earthquake state of stress in the dam.

In the first analysis the shell was assigned relatively stiff properties, while the value of Poisson's ratio ν for the core was fairly low. This led to excessive stress concentrations together with a volume reduction of the core. The stiffness of the shell was then reduced based on the consideration that the larger blocks of shell material would tend to crush and ν for the core was increased, which resulted in a more reasonable stress plot for the end of construction stage. However, when applying the water load to the core and the accompanying uplift forces to the upstream shell for the full reservoir condition, several filter and shell elements showed anomalous stress values. The results were improved by using buoyant unit weights for appropriate zones in the first stage and seepage forces corresponding to the steady state condition in the impounding stage with effective stress parameters throughout. The final calculated pre-earthquake stress state is shown in Fig. 1. It may be generally remarked:

(i) a significant stress transfer from the core to the shell is observed, especially at the end of construction stage, the effect being enhanced by the wide trench excavated in the alluvium.

(ii) in some areas, e.g. near the surfaces where the overburden stresses are low, the calculated stresses are close to failure. Theoretically, this would have consequences for earthquake loading as this would lead to seismic settlements or slips.

(iii) the analysis is sensitive to variations in the soil parameters, required in the hyperbolic stress-strain law.

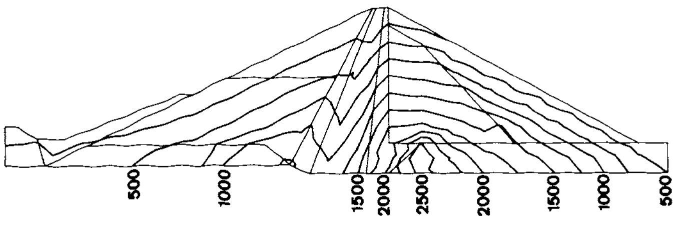

Fig. 1 Major principal stresses (kN/m^2) for full reservoir condition

DYNAMIC ANALYSIS

The dynamic analysis was conducted using the equivalent linear procedure (standard programs QUAD-4, FLUSH). Laboratory testing provided information about these parameters in the range of γ > ca. $5 \cdot 10^{-3}$ %. The initial values G_{max} were estimated using empirical equations. Alternatively, a hyperbolic law could be used to extrapolate the triaxial data to values in the range $\gamma < 10^{-4}$ %. For the rockfill a value of $K_2 = 200$ was used in the relation

$$G_{max} = 221\ K_2\ (\sigma'_m)^{1/2} \tag{1}$$

where σ'_m is the mean effective stress in units of kN/m^2. In later computer runs K_2 was reduced to 150 to allow for some degradation of soil modulus due to a seismically-induced build-up of pore pressures in the upstream shell.

A typical stress response for a shell element is shown in Fig. 2.

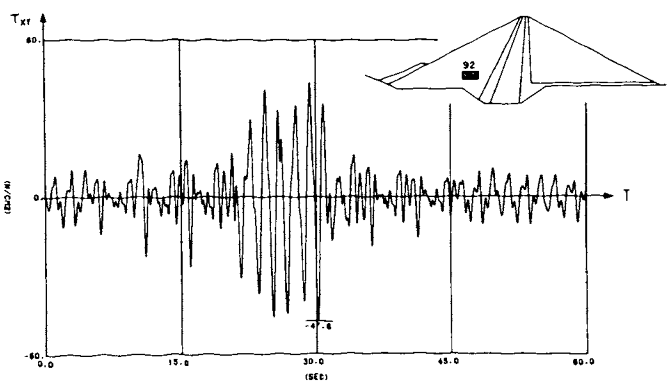

Fig. 2 Time history of horizontal shear stress

For the given input motion greater shell stiffness gives rise to higher dynamic shear stresses. However, to increase strength and reduce settlements it is necessary to have well-compacted rockfill material. A consequence is that in the dynamic analysis there is a strong tendency for stress transfer between the core and the rockfill leading to high stress concentrations in the lower filter zone.

As a comment to the equivalent linear procedure it may be mentioned that there is some arbitrariness in fixing the characteristics strain. More important, however, are the consequences of this assumption for the vibrational behaviour. The shear moduli are too soft for the lower levels of shaking in the earthquake record and vice-versa. There is a tendency to overamplify the motion at the fundamental frequency and filter out higher frequency motions. This effect could possibly be significant for dams located in the near field of causative faults.

EVALUATION OF PERMANENT DEFORMATION

The investigation of the permanent deformation involves:
(i) the deformation of the saturated zones.
(ii) possible deformation along the dry downstream slope.
(iii) checking the overall stability of the saturated zones for the excess pore pressures generated by the earthquake.
In this way the loss of freeboard for this case of extreme earthquake loading may be evaluated.

Having obtained the stress time histories for various elements within the dam the final step in the Seed-Lee-Idriss analysis was to evaluate the dynamic effects on the dam with the aid of triaxial cyclic laboratory tests, usually, for practical reasons, employing uniform sinusoidal loading. In case (i) the cyclic strains developed at different stress levels for the period of shaking are obtained. Then by means of a "strain harmonizing" technique the strains within the dam are estimated. With regard to the accuracy of this part of the analysis the development of non-homogeneous strain fields in the soil element (triaxial sample) should not be overlooked (Casagrande, 1976). This problem will be more acute for dense, overconsolidated samples than for loose samples.

For case (ii) the ultimate rather than the peak value of drained strength ϕ' would be adopted, as only deformations exceeding a few tens of cm would be of interest in a maximum credible event. A sliding block type of analysis was adopted for this part.

In the following attention is focused on case (iii). As far as core material is concerned it appears that pore pressures are induced only in the case of contractive clays, and that well-compacted clays develop pore suctions under cyclic loading (Sangrey et al, 1979). This would confirm the observed behaviour of clay fill dams in practice. For instance, they stood up extremely well to very intense shaking in the 1904 San Francisco Earthquake (Seed, 1979). However, even very compact granular materials can develop pore pressures under cyclic loading, a phenomenon that is caused by grain crushing effects and a slight rearrangement of particles. Rockfill as a class of granular material is considered to have excellent earthquake-resistant strength properties due to its rapid draining capabilities. However, if the rockfill contains some finer material, which is unavoidable with quarry-blasted rockfill unless expensive screening of the material is specified, the permeability k is reduced. In this study tests indicated that the value of k is about 10^{-3} to 10^{-2} cm/s, and since the rockfill comprises the major portion of the dam a laboratory investigation of the cyclic loading properties of the saturated material was undertaken. The tests were conducted under undrained conditions, which is a conservative assumption. However, simultaneous generation and dissipation of pore pressure can be accounted for computationally.

Framework of critical state soil mechanics

It is pertinent at this stage to review briefly our knowledge of the behaviour of saturated, non-cohesive materials under monotonic and cyclic loading and conditions of no drainage.

In the case of loose materials pore pressures are induced under both monotonic and cyclic loading. Liquefaction is a special type of behaviour associated with an unstable grain structure, which may be regarded as strain softening at constant volume.

Dense non-cohesive materials develop pore suctions under monotonic loading but pore pressures under cyclic loading.

This behaviour in undrained shearing is illustrated in Fig. 3. Samples normally consolidated (C) and slightly overconsolidated (D) develop pore pressures, whereas a heavily overconsolidated or compacted material (E) develops at failure pore suction under monotonic loading. If the compacted sample (E) is subjected to cyclic loading pore pressures are induced, i.e. the effective stress at the point of stress reversal moves to B. However, if it is now loaded statically to failure it exhibits more or less the same undrained strength (A) as before. The build-up of pore pressure under cyclic loading appears at first sight to contradict the critical state concept. However, the latter is essentially a model for states near failure. Nevertheless it still provides a useful framework for understanding soil strength behaviour for samples subjected to pre-cycling at lower stress levels. An actual test result for a sample of gravel material from the rockfill shell is shown in Fig. 4. After 12 cycles a state of almost 100 % pore pressure increase is obtained. The effective stress path is in the position DE. The sample strained progressively reaching a permanent extension ε_a = 1.2 % at 22 cycles. Subsequent static compression results in the stress path DF, the ultimate strength being reached at ε_a = 14 %.

If an element of soil is brought to a condition of 100 % pore pressure increase and static forces induce plastic flow it is evident that movement would be very quickly retarded by the increase in strength due to the developed pore suctions. In situ, however, one is not dealing with a homogeneous stress condition, and, further, the drainage restriction does not apply. It is necessary, therefore, to examine alternative mechanics of failure. One possible mechanism is strain softening due to dilatency. This is illustrated in Fig. 5 by means of a hypothetical test. The initial stress condition is represented by point C, which is on the "dry" side of the

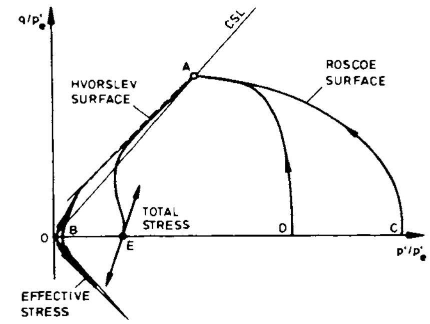

Fig. 3 Behaviour of normally consolidated and overconsolidated samples in normalized p' (mean effective stress): q (deviator stress) diagram. p_e': normal consolidation pressure at given volume. Terminology after Atkinson and Bransby (1978).

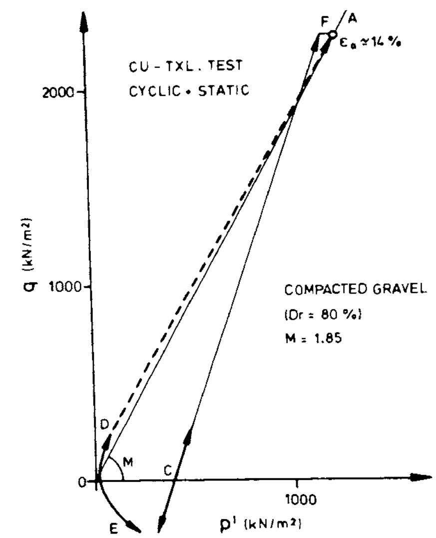

Fig. 4 Strength of cyclically pre-loaded rockfill material.

critical state line, CSL. Cyclic loading shifts the effective stress to the left (P). Subsequent static loading at constant volume tends to dilate the grain structure inducing pore suctions. At failure the critical state is reached (Q). Now drainage is permitted and water is sucked in. To remain on the CSL-line the total deviator stress must be reduced. Point R corresponds to the ultimate drained strength of the material in triaxial compression, which for a dense, compact material is lower than the undrained strength. In practice it would be unwise to rely on the pore suctions to maintain stability.

In many cases this effect of dilatancy softening would not be enough to explain failure occurrence. In fact, a good design would be based on ultimate rather than peak drained strength, and if this were the only mechanism operating the static design would take care of the contingency of catastrophic collapse. Thus other factors involved have to be examined.

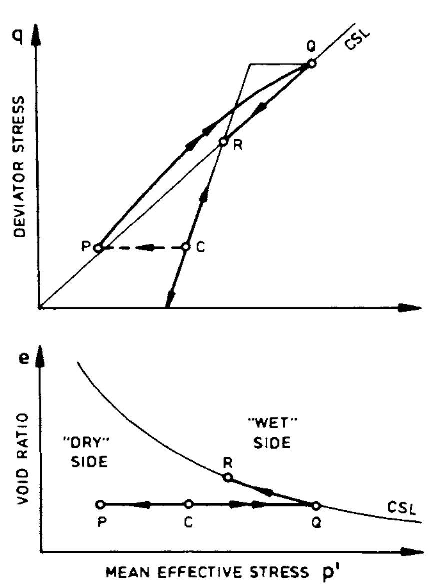

Fig. 5

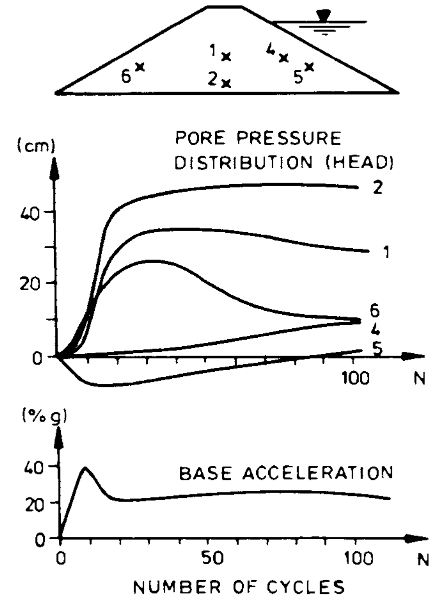

Fig. 6

Effect of transient seepage forces

The development of pore pressures due to shaking gives rise to transient seepage forces, or viewed in terms of Terzaghi's effective stress law, the strength is reduced. It appears that this subject has not been discussed as explicitly as it ought to have been (Prater, 1981), and thus opinions have differed widely with regard to the importance of cyclic pore pressure effects. It was shown above that the strength of an element of soil is governed by the static strength, irrespective of cyclic pore pressure build-up. If, however, we allow the material in the slip zone to reach the point R (Fig. 5) and there is still a reservoir of excess pore pressure in zones adjoining the slip zone, then water will flow across the slip zone as dissipation and drainage take place and the pore pressures will rise again due to the hydraulic effects.

It follows that there is an interplay of three pore pressure phenomena: generation of pore pressure due to shaking; dissipation of pore pressure in connection with transient flow (a problem of consolidation); reduction of pore pressure due to dilatancy effects in induced slip zones. The last of these effects is the least amenable to analysis. If the slip zone is very thin it might be possible to neglect strain-induced pore suctions. However, if the plastic zones are more widespread pore suctions will play an important rôle. An illustration of this is given in the results of shaking table tests on 0.9 m high homogeneous sand dams under seepage conditions (Richter et al, 1979), see Fig. 6. In the lower central portions of a high dam the material would tend to be normally consolidated, with reduced pore suction effects.

The increase in total head will be equal to the excess pore pressure and since the latter is maximum near the base of the dam the flow will tend to be upward. However, soils compacted in layers will have anisotropic permeability conditions and flow is greater in the horizontal direction. The net result is an unfavourable flow direction from the point of view of stability unless interceptor drains are installed.

PORE PRESSURE GENERATION

The generation of pore pressures under cyclic loading is a complex process and usually empirical means are used to describe it. There is, however, the further problem of applying the results of laboratory tests, in which restrictions are placed upon the state of stress, to general states of stress in the field, i.e. there is the question of the influence of the intermediate principal stress, σ_2. Here we investigate the limiting condition for the residual pore pressure at the end of shaking. If the CSL-line (Fig. 3) determines the maximum value, then

$$(\Delta u_r)_{max} = p_o' - q_o' / M \qquad (2)$$

where p_o' , q_o : anisotropic consolidation stress state.

The important underlying assumption here is that the initial static state of stress is not changed by the earthquake. If it is assumed that ϕ' is the basic strength parameter

$$M = 6 \sin\phi' / (3 \mp \sin\phi') \qquad (3)$$

for compression and extension failure conditions respectively. For compact materials equ. (2) may slightly underestimate the amount of residual pore pressure, as the stress path reaches the state boundary (Hvorslev) surface, see Fig. 3. This appears to correspond to the cyclic limit state postulated by Sangrey et al (1979).

SAFETY EVALUATION OF UPSTREAM SHELL

Preliminary calculations showed that due to the given material permeability values the dissipation times would be relatively slow (Prater and Studer, 1979). For the rockfill material it was also found that the rise in pore pressure, i.e. in the relationship r_u against N/N_p, N_p being the number of cycles to 100 % pore pressure rise was very rapid, especially for high K_c values ($K_c = \sigma'_{1c}/\sigma'_{3c}$), compared to previously published data. Thus for the stability analysis, as a first approximation, the conservative assumption of neglecting the dissipation during shaking was adopted. The next step was to calculate the equivalent number of cycles for each element stress history, whence the residual pore pressure. The limiting value, given by equ.(2), was reached in some elements (Fig. 7). The assumption of 100 % pore pressure ratio (Idriss et al, 1979; Sadigh et al, 1979; Seed, 1979) was not adopted as it violates the strength law for $K_c > 1$, unless the static stresses are substantially altered.

Slope stability analyses were carried out using Bishop's method, in which the values of Δu were inserted at the base of each slice to account approximately for seepage forces. The analysis was carried out for the reservoir full condition. For this case the safety factor F fell from 1.80 to 1.17. The critical slip surface (1) together with two others which have greater safety (F = 1.19, 1.30 for (2),(3) respectively) is also shown in Fig. 7.

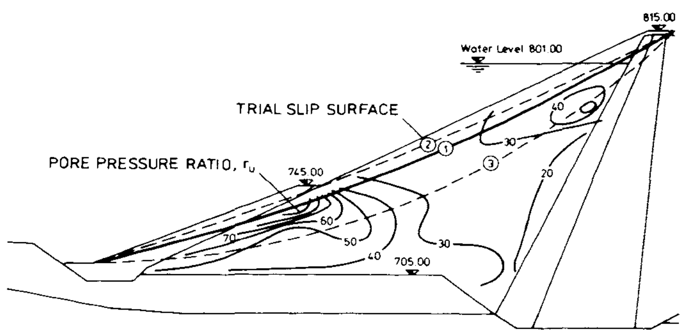

Fig. 7 Cyclically-induced pore pressure in shell as a function of overburden stress, $r_u = \Delta u/\gamma' h$.

The presence of the cofferdam influences the drainage and pore pressure generation/dissipation, since its upstream slope is impermeable. Thus it was decided, as a preventive measure, to remove the impermeable zone and construct a chimney drain along the downstream slope of the coffer dam. In this case the build-up of pore pressure is reduced and the direction of the seepage forces is improved from the point of view of stability. The safety factor by Bishop's method is now increased to at least 1.32.

FINAL COMMENTS

The behaviour of high rockfill dams under very strong ground shaking is largely an unknown quantity. For the saturated upstream shell pore pressure generation, dilatancy effects and the transient flow of water is a complex process. The development of pore pressures in compact materials is mainly related to crushing at grain contacts, and is a problem affecting non-cohesive soils. Thus the threat to stability does not come from collapse of the grain matrix (classical liquefaction) but rather from the adverse effects of transient seepage forces, i.e. the seismically induced pore pressures create a potential field within the saturated shell. For sliding to be triggered and prolonged the slip zone must be able to suck in water without diminishing greatly the reservoir of excess pore pressures in adjacent zones. To prevent hydraulic gradients acting in an unfavourable direction attention must be given to internal drainage. Also, from the practical viewpoint, the more compact the material, the less likely it is that high pore pressures will be induced. Thus not only does the seismic investigation and design require a careful analytical procedure, but also a good measure of engineering judgement.

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