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SIMULATION OF PORE PRESSURES IN EARTH STRUCTURES DURING EARTHQUAKES

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ABSTRACT

Assessment of stability of an earth structure can be carried out reliably if pore pressures developed during earthquake loading can be simulated with confidence. The pore pressure coefficients, which are a function of the number of cycles, are used to demonstrate the increase of pore pressures in an earth dam through an earthquake time history. The usefulness and reliability of a pore pressure equation utilising the pore pressure coefficients is demonstrated. The simulation procedure for pore pressure developed in an earth structure during earthquake is explained and attention is drawn to the way in which failures can be explained on the basis of excess pore pressures. The use of dynamic pore pressure coefficients along with limit equilibrium and sliding block approaches has a vast scope in the practice of geotechnical earthquake engineering.

KEY WORDS

Earth dams, Slopes, Pore pressure, Pore pressure parameter A_n, Dynamic response, Deformation

INTRODUCTION

An important aspect of geotechnical earthquake engineering concerns the dynamic response of earth structures such as embankments, slopes and earth dams to earthquakes. In order to quantify overall stability and to estimate stresses, strains and permanent deformations, it is essential to determine good quality soil properties as well as analytical and numerical methods need to be developed. Evaluation of soil properties for seismic stability analyses of slopes has recently been reviewed by Martin (1992). Reviews concerning the assessment of seismic stability and permanent deformation have also been carried out by Finn (1990) and Marcuson *et al* (1992). The latter publication has focused on the merits and limitations of both simple and sophisticated methods of analysis (two-dimensional and three-dimensional). Simplified method of determining the permanent deformation is generally based on the sliding block technique, Newmark (1965), Sarma (1975,1981), Hynes-Griffin and Franklin (1984), Tika *et al* (1993), Chowdhury and Xu (1993, 1994) and Chowdhury (1995), in which a critical (or yield) acceleration is determined for a most likely failure surface and it is assumed that deformations occur along the failure surface when the earthquake accelerations exceed this critical acceleration. In determining the critical acceleration, soil properties, particularly the excess pore water pressure generated within the soil mass during the earthquake plays a very important part. Seed *et al* (1973), Seed(1979) developed the liquefaction analysis technique, in which the dynamic stresses are first determined in the body of the structure and its foundations and subsequently, the possible rise in pore water pressure in the soil mass (leading to liquefaction) due to the cyclic loading is determined in the laboratory. If liquefaction is indicated, then the structure is assumed to have failed. This approach facilitated an understanding of the failure of the Lower San Fernando Dam in the earthquake of 7 February, 1971. Further developments in understanding soil mass behaviour under earthquake conditions have included reassessments of this failure (Seed *et al* 1988, Marcuson et al 1990, Stark and Mesri 1992). An alternative method of assessing the excess pore pressure during cyclic loading was proposed by Sarma (1979), Sarma and Jennings(1980) and later modified by Tsatsanifos and Sarma(1982), in which a cyclic pore pressure parameter A_n (for n cycles) was defined and this was related to A_1 and the number of cycles through a second parameter β. For triaxial cyclic loading conditions, these were defined as:

$$\frac{\Delta u}{\sigma'_{3c}} = A_n \frac{\Delta \sigma_1}{\sigma'_{3c}} \qquad (1)$$

$$\sqrt{A_n} = \sqrt{A_1} + \beta \log n \qquad (2)$$

An expression similar to 1 was also developed for simple shear loading conditions. However, it was found that the parameters are dependent on the loading conditions. Whereas Sarma and Jennings (1980) assumed that the parameters were independent of stress level below the failure stress level, Tsatsanifos and Sarma showed that these are dependent on stress level. For particular time histories, the pore pressures estimated by this approach were found to be comparable to those given by Martin *et al*(1975).

USE OF DYNAMIC PORE PRESSURE PARAMETER A_n IN LIMIT EQUILIBRIUM ANALYSIS

A two-stage approach under earthquake conditions was proposed by Sarma (1979,1988) as a significant modification of an earlier limit equilibrium solution with effective stresses, Sarma (1973),. The first stage consists of determining the static stresses along the potential slip surface and the second stage includes the application of earthquake load with pore-pressure parameters to dynamic loading conditions. A value of the factor of safety F and the corresponding value of critical seismic coefficient k_c corresponds to any given value of the dynamic pore pressure parameter A_n. This method was applied to the Lower San Fernando Dam Failure of February 9, 1971 which has been the subject of many previous studies such as Seed et al (1973). The dynamic pore pressure parameters $A_1 = 0.67$ and $A_5 = 1.35$ were evaluated from the results reported by Seed *et al* (1973). From Sarma's analysis a value of $k_c = 0.21$ was evaluated for the case with no dynamic pore pressure. However, corresponding to a value of $A_n = 1.8$ the value of k_c was evaluated as 0.11. Such a value is still significantly higher than zero and, therefore, does not represent overall failure. Nevertheless, it is consistent with a significantly decreased stability. This approach was proposed mainly for the purpose of illustrating how the value of the dynamic pore pressure parameter can sensibly be used in an analysis. It was not considered to be a complete solution of a real problem involving a specific earthquake ground motion with a unique acceleration - time history. Even so, if liquefied zones were assumed as in the studies of Seed *et al* (1973), a logically consistent interpretation of the failure could be provided on a qualitative basis.

INCREMENTAL TIME-STEP APPROACH

A new approach for the estimation of excess pore water pressures in an earth structure has been proposed by Chowdhury and Xu (1993). This approach recognises that critical acceleration is, in general, a function of time during the earthquake. The critical acceleration may change because of change of shear strength associated with mechanisms such as pore pressure generation and strain-softening or even strain hardening. The proposed new approach requires that analyses be performed in small time steps. For a given slip surface, which may be the critical slip surface under static conditions or any other reasonable surface, the initial value of the critical seismic coefficient k_c is calculated. This critical acceleration must necessarily be positive; otherwise, a static failure is indicated. The pore pressure parameters A_l and β are determined from appropriate laboratory tests. Then, for any time step, the increase in pore pressure and the corresponding critical acceleration is computed for the slip surface. If the applied load is bigger than the critical acceleration, then possible deformation in the earth structure are computed using the sliding block technique before proceeding to the next time step. The magnitudes of principal stresses which are needed to compute the excess pore water pressure can be estimated from the mobilised normal and shear stresses on the failure plane.

The pore water pressures computed using the above technique represent a highly variable time distribution due to the variable acceleration as represented by the acceleration-time history of any real earthquake. Moreover, the determination of the equivalent number of cycles prior to the current time step, so that the appropriate A_n value can be used, makes the application difficult. Therefore, a simplified approach is used, in which the total equivalent number of cycles correspondiong to the end of the earthquake is prejudged. At any stage during the earthquake, the equivalent number of cycles is considered to be proportional to the elapsed time from the start of the earthquake. The dynamic pore pressure parameter A_n is assumed to vary according to equation 2.

APPLICATION OF INCREMENTAL APPROACH

Implementing this approach required a comprehensive extension of the Newmark sliding block method of analysis. A suitable computer program has been developed to handle the computations involving iterative limit equilibrium analyses and dynamic analyses for many small time steps. The computer program can handle slip surfaces of both circular and arbitrary shape within an earth structure which may have several different soil zones.

The comprehensive time-step approach was used for the analysis of the Lower San Fernando Dam. A cross-section of the dam is shown in Fig. 1 and it is the one which has been used in previous analyses by other researchers. An acceleration-time history, closely resembling the one used in previous studies by other researchers, was generated using a simulation approach developed at the University of Wollongong. This acceleration time-history is shown in Fig. 2 and the soil properties for the different zones shown in Fig. 1 are presented in Table 1. These are the properties which have been presented in previous publications concerning the analysis of the 1971 failure of this dam.

The potential failure mass above the assumed slip surface shown in Fig. 1 was considered to be divided into 16 vertical slices and it is of interest to consider the excess pore water pressure generated at the base of each slice. The slices were numbered from 1 to 16 consecutively starting from the toe of the upstream face of the dam. The prediction of pore water pressure generated with time for two slices, slice 5 and slice 10 is shown in Fig. 3 considering the dynamic pore pressure parameters $A_l = 0.67$ and $A_n = 2.4$. The pore water pressure generated with time at the base of slice 1 for different values of A_n is shown in Fig. 4 assuming $A_l = 0.67$. A similar plot for pore pressure generated at the base of slice 5 is shown in Fig. 5.

DISCUSSION AND CONCLUSIONS

The concept and relevance of dynamic pore water pressure parameters in the stability and deformation analysis of earth structures has been discussed. It has been argued that estimates of residual undrained shear strength for cohesionless soils are only necessary if a total stress approach is used under undrained conditions. On the other hand, if an effective stress approach is used, reliable estimates of excess pore water pressure generated during earthquakes will be sufficient for analysis and, therefore, estimates of residual undrained shear strength are not required. In total stress analyses reported in the literature, liquefied soil zones are identified and residual undrained shear strength values assigned to such zones. If an effective stress approach is adopted for analysis, it is not necessary to identify zones of liquefaction separately from the main analysis.

In order to estimate pore water pressures as a function of time during the earthquake, an incremental time-step approach has been developed. Brief details of this approach, based on a comprehensive extension of the Newmark sliding block model, are given in the paper. For the Lower San Fernando Dam the estimated excess pore water pressure results are presented along an assumed slip surface.

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TABLE 1

Soil properties used in the analysis

Zone	γ (kN/m^3)	c' (kN/m^2)	ϕ' (Degrees)	c_u (kN/m^2)
1	17.3	0	38	-
2	20.5	0	37	-
3	19	-	-	81.4
4	20	0	33	-
5	19	-	-	81.4

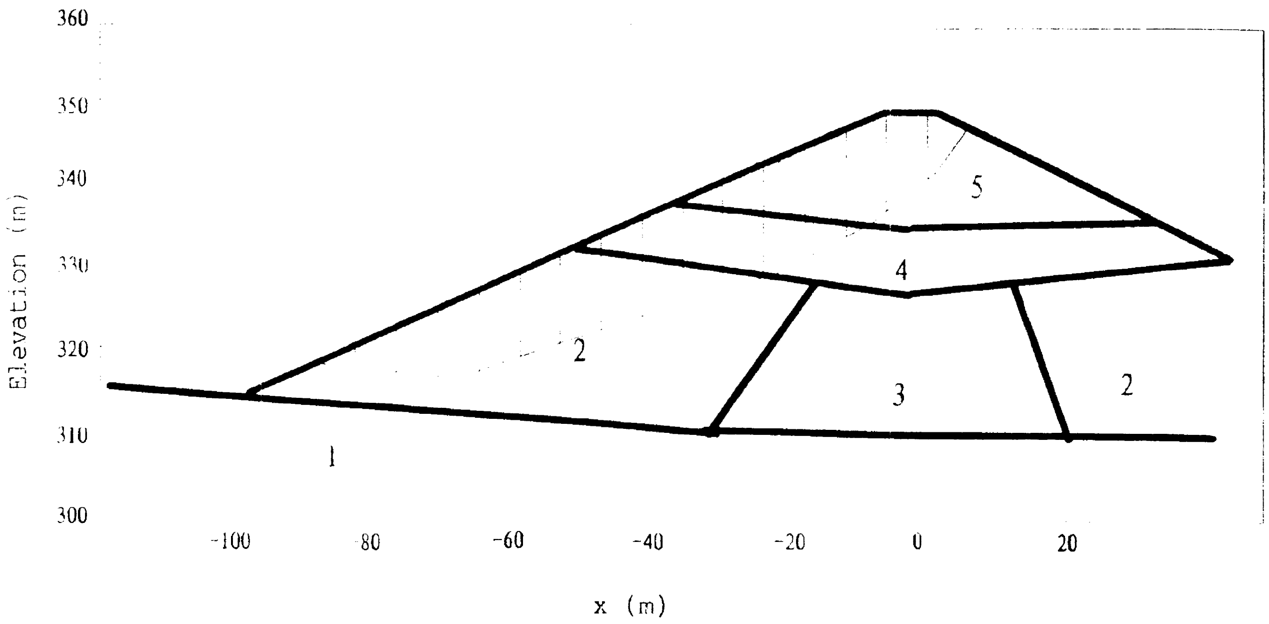

Fig 1 : Cross section of Lower San Fernando dam used for analysis; the particular slip surface is also shown

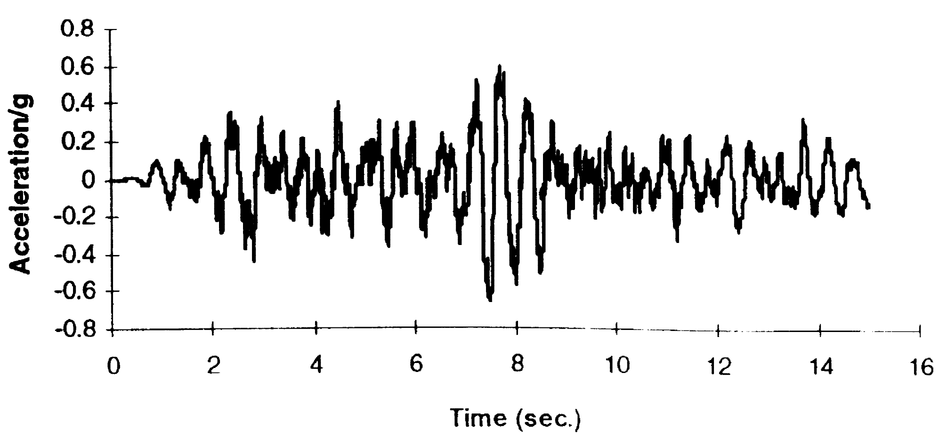

Fig 2: A generated earthquake for 15 seconds

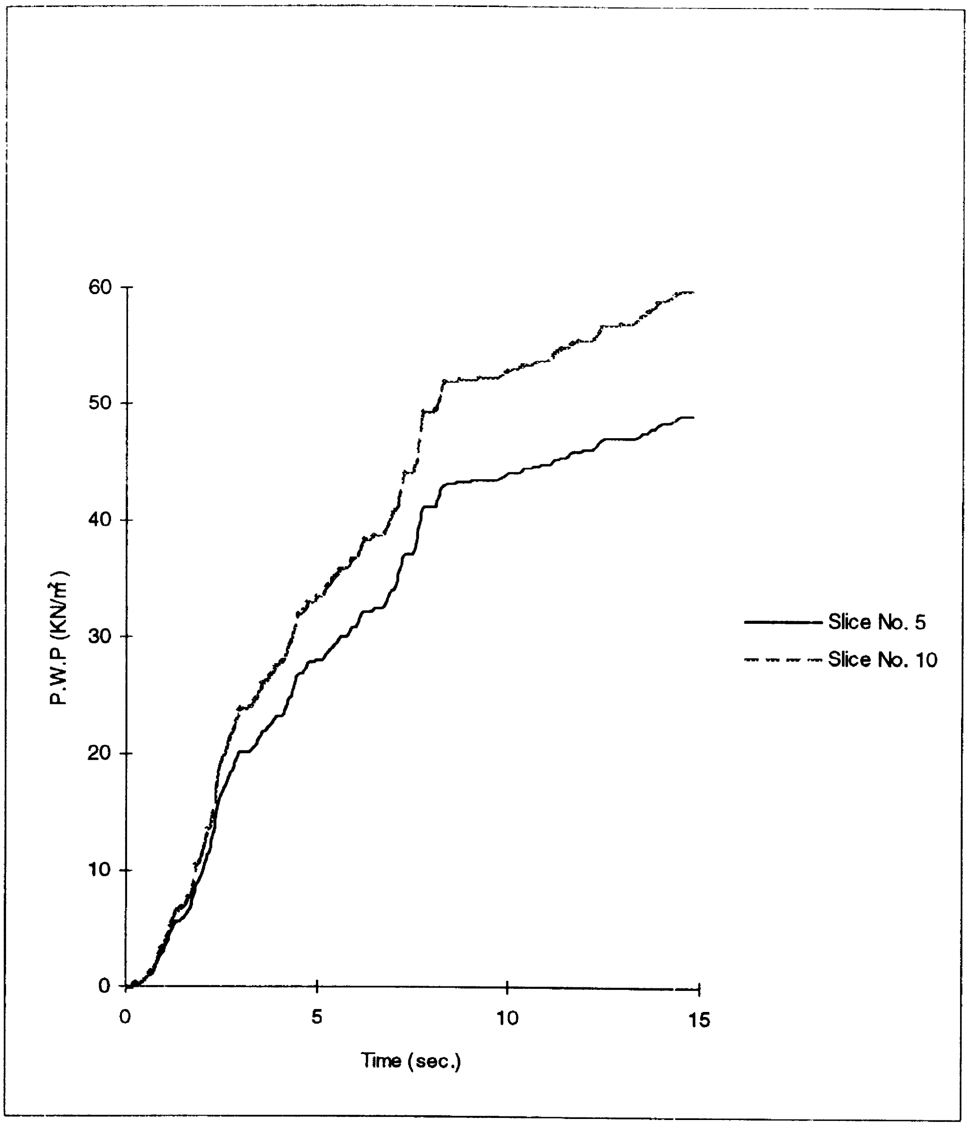

Fig. 3 Pore water pressure development during earthquake in slices 5 and 10 considering A_1 =0.67 and A_n=2.4

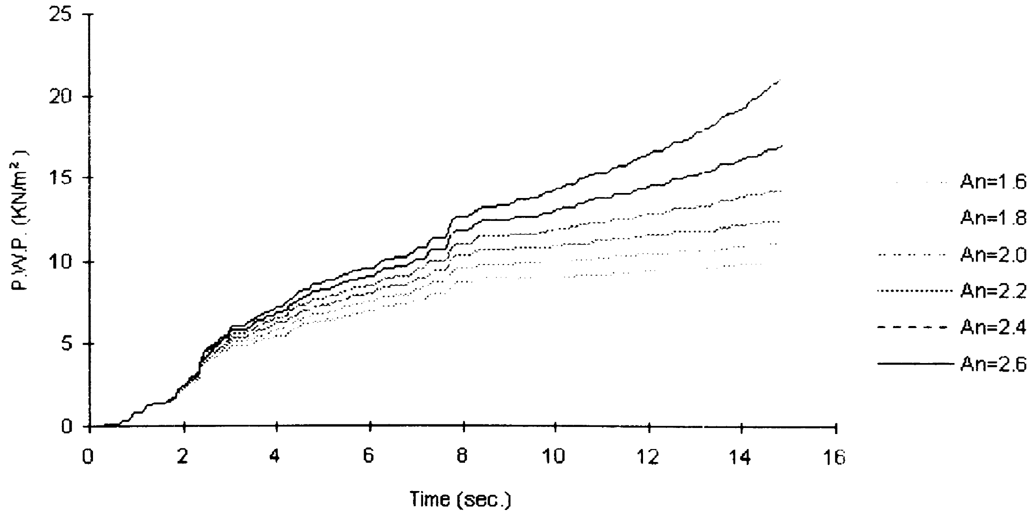

Fig. 4 Pore water pressure development during earthquake in slice 1 showing the influence of the value of A_n assuming A_1=0.67

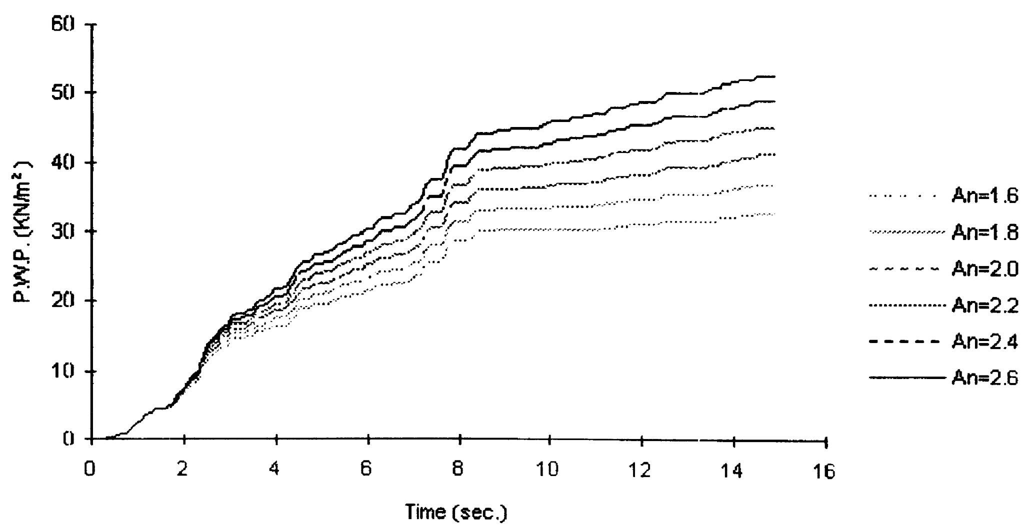

Fig. 5 Pore water pressure development during earthquake in slice 5 showing the influence of the value of A_n assuming A_1=0.67