Pseudo-static analysis of clay slopes subjected to earthquakes

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The seismic stability of slopes is a subject about which much uncertainty still exists. In dam engineering until the late 1960s the simple pseudo-static approach (the Author's equivalent static force (ESF) method) was universally applied. However, with the notable failure of the lower San Fernando hydraulic fill dam attention was given to material technological aspects, i.e. to seismically induced liquefaction. To meet the needs of analysis and design the now widely used stress-path-type method of Seed, Lee & Idriss (1969) was introduced. Today computer programs in which the soil is represented as a two-phase material with an appropriate non-linear description of the grain skeleton behaviour are being developed. Experience has shown, however, that most well-engineered dams, especially good draining, well-compacted rockfill dams and clay dams, are unlikely to fail under seismic loading (Seed, 1979). Thus complicated analysis techniques are not warranted in their case. Nevertheless, as a compromise a Newmark-type analysis (Newmark, 1965) is often carried out.

Based on the concept of a sliding mass of soil, the Newmark approach is really an extension of the traditional static stability analysis of slopes. If pore pressures are induced by the shaking action the method is difficult to apply because the magnitude of these pore pressures is generally not known. Further, Kutter (1984) centrifuge models have shown that the internal deformation patterns obtained are more complex than the sliding block concept assumes and that distinct slip surfaces may not develop under dynamic excitation. Nevertheless, as a design tool the Newmark method is more convenient than the simple pseudo-static method of analysis. Leaving aside the difficult problem of selecting a representative seismic input motion, the reason for this is discussed here.

More recent seismological observations have shown that actual peak accelerations, especially in the near field, are much higher than was supposed when pseudo-static methods were introduced. However, design becomes prohibitively uneconomic for horizontal seismic coefficients exceeding $k_h \approx 0.25$. Furthermore,

vertical acceleration components, traditionally neglected, can also be substantial, e.g. $k_v \approx 2k_h/3$. A seismic coefficient equal to some fraction, say one-third, of the peak acceleration may more or less take into account the effects of transient inertia loading. However, a rough estimate of the plastic irrecoverable deformation as given by Newmark's method provides a greater measure of rationality for the aseismic design of slopes. The Author did not distinguish clearly enough between these two methods (ESF and Newmark). In many countries dams and slopes are not specifically considered in codes of practice, so that appropriate seismic coefficients are not available for design.

Various theoretical investigations have shown that the predicted rigid block displacements in Newmark's analysis are very sensitive to the value of the computed yield acceleration. The yield acceleration can be determined by any of the established methods of slope stability analysis. For general situations, e.g. including inhomogeneity, seepage, action of external forces etc., one of the various methods of slices, e.g. Bishop's or Janbu's methods, would normally be employed. The Author provides a solution for a purely cohesive soil with a linear variation in cohesion with depth. In soil mechanics this corresponds to a $\phi = 0$ condition corresponding to no drainage of porewater and its application in earthquake loading situations is justified.

The Author states that the Writer's analysis (Prater, 1979) does not allow for deep-seated slip surfaces other than those passing through the toe of the slope. However, it is necessary to point out that the question of deep-seated failure was considered, since for the static analysis of clay slopes ($\phi = 0$) of angle $i < 53^\circ$ deep-seated failure is more critical. It is an easy matter to extend the Rendulic (1935)–Taylor (1937) logarithmic spiral analysis to include this case (Prater, 1985). This simply requires, using triangular elements, area and centre of gravity determinations analogous to those made by the Author for the slip circle analysis. However, on the basis of considerations of wave amplification effects through a soil layer, especially for a dam profile, only toe failure is deemed to be relevant. If cohesion increases with depth this considera-

Fig. 1. Comparison of horizontal yield acceleration (factor of safety $F=1$) for planar and logarithmic spiral slip surfaces: slope angle, 30° ; varying strength parameters ϕ and $N_s = c/\gamma H$

tion would be even more pertinent. However, possible sliding surfaces above the toe of the dam may be more critical. These can also be treated using the equations for toe failure. Ambraseys & Sarma (1967) calculated various acceleration distributions and concluded that a deep slide will be subject to smaller overall accelerations, since it is more likely that in large dams shallow slides near the crest will occur. Similar conclusions will apply to natural slopes.

Often very simple planar sliding models are used to calculate block displacements, even for dam safety calculations. In a later paper the Writer (Prater, 1981) has compared the yield acceleration in a homogeneous slope for planar and logarithmic spiral sliding surfaces. For steep

slopes the differences are not very great, but for a slope angle of less than 45° they can be substantial. Fig. 1 shows a plot of horizontal yield acceleration against friction angle for a 30° slope with a family of curves for stability number N_s . Planar sliding, it can be seen, can overestimate yield acceleration very considerably, with a consequent influence on the calculated displacement.

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