EARTHQUAKE DEFORMATION IN EARTH AND ROCKFILL DAMS

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ABSTRACT

A finite element model capable of computing permanent deformations along pre-existing sliding surfaces in dams and embankments is utilized to evaluate the error introduced with the current state-of-practice two-step procedures which are based on the simplifying assumption that the computation of dynamic response and of the resulting sliding displacements can be decoupled. Both sinusoidal and real earthquake ground motions are employed as excitation and Coulomb's friction law governs sliding along any point of the tensionless interface. It is found that when the predominant frequencies of the input motion lie in the proximity of the fundamental frequency of the dam the permanent deformation based on the decoupling assumption may exceed the "exact" value significantly.

1 INTRODUCTION

Newmark[8] demonstrated that earth slopes do not necessarily fail just because the transient factor of safety against wedge sliding reaches (momentarily) a value below unity. Instead, permanent deformations of only a limited magnitude take place, as the direction of the "driving" acceleration quickly reverses and the sliding of the soil wedge stops.

Seed and Martin[10], Ambraseys and Sarma [1] and Makdisi and Seed[4], applying Newmark's sliding-block concept, have developed simplified procedures for predicting the permanent displacements of dams subjected to earthquake shaking. The magnitude of such displacements has since been introduced in safety performance criteria, in place of the "factor of safety" against limit-equilibrium failure[5]. In its complete form, this simplified procedure includes for any trial failure surface two steps (Fig.1): (1) perform a dynamic analysis of the dam

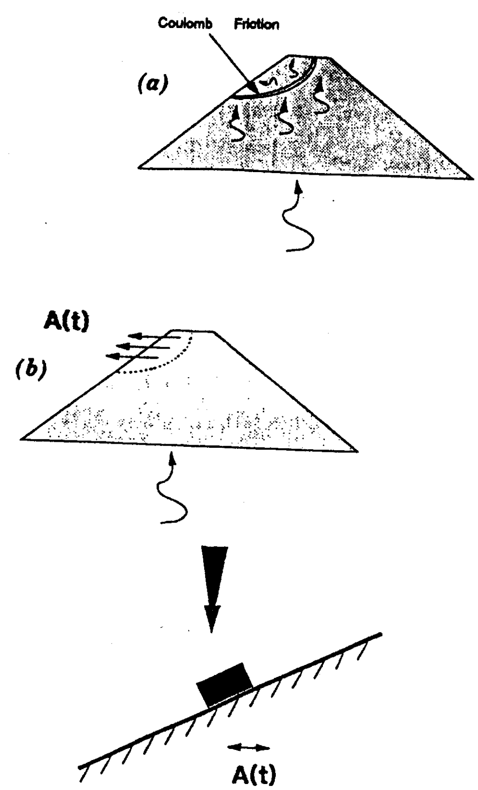


Fig. 1: (a) Single-Step Analysis (b) Two-Step (Decoupled Analysis)

assuming that no failure surfaces exist. Determine the time history of acceleration at a number of locations within a hypothetical potentially sliding wedge, and at each instant of time average the accelerations over the whole volume of this wedge. The result is the (spatially) average time history of acceleration for the soil above the hypothetical failure surface, and (2) Use this average time history of acceleration as the "driving" support motion in a sliding-block analysis, where the mass of the block equals the mass of the wedge, while the horizontal force required to initiate sliding is set equal to the horizontal yield acceleration (pseudo-statically determined) of the wedge. Compute the resulting permanent slip along the sliding interface. This simplified two-step procedure (which is hereafter also called "Decoupled" method) is based thus on three key assumptions: (i) computation of the *elastic* dynamic response and of the *inelastic* plastic slip can be decoupled and carried out separately, (ii) all permanent deformation occurs in concentrated fashion along the slip surface, and (iii) the sliding block behaves as a rigid body. Although intuitively logical, these simplifying assumptions have been tested only indirectly.

One main objective of this paper is to investigate the validity of the Two-Step procedures, by assessing the effect of the presence of a sliding interface on the acceleration response of the dam. Moreover, in many cases, zones or surfaces of weakness were found to have formed in dams after earthquake shocks or after other causes of distress; the response of the earth structures containing such a discontinuity is of interest. Another example of pre-existing potentially sliding surfaces is earth structures containing inclined geomembranes and geotextiles for a variety of purposes, where the geomembrane may act as a weak surface during seismic shaking. Hence, concern about the dynamic response of a geotechnical facility, such as a landfill containing geosynthetics, should primarily be associated with the permanent relative displacement that may accumulate along the interface between the two geosynthetics.

Attempting to provide an improved understanding of the mechanism of the sliding deformations in earth dams, this paper presents results with a more general "*Single-Step*" procedure to estimate the earthquake induced permanent displacements, and examines the likely error introduced by the two-step procedures. A finite-element formulation, which can implement Coulomb's friction law along a sliding tensionless interface has been employed to this end. Note that a similar type of analysis was advocated by Nadim and Whitman[7] for gravity retaining walls, which determined the permanent seismic displacements after assuming the presence of a pre-defined Coulomb-type sliding surface in the backfill.

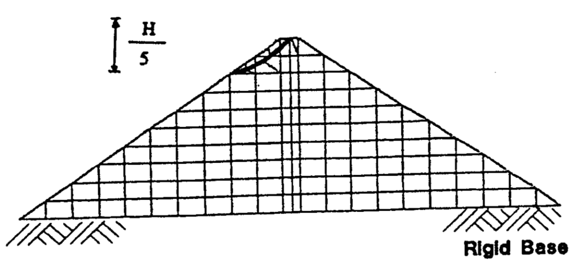
2 GEOMETRY AND MATERIALS OF THE STUDIED DAM

A typical 100 m tall rockfill dam, is studied herein to compare the response computed using the decoupling assumption with that calculated by a "single-step" procedure. The dam is symmetric with 1.5H:1V slopes and a 10 m wide crest. The dynamic behavior of rockfill is described through: (i) the small-strain shear modulus, G_{max}; (ii) the decrease of secant modulus G with increasing strain γ; (iii) the hysteretic damping ratio, β, which is an increasing function of the amplitude of shear strain γ; and (iv) the Poisson's ratio v. Published experimental results from the literature were utilized to assign realistic values to the material parameters[11]. Thus, the spatial distribution of G_{max} was estimated as a function of the effective confining pressure, $\sigma_m = (\sigma_1+\sigma_2+\sigma_3)/3$, of each rockfill element in which K_{2max} has units of square-root of stress and its value for compacted gravels and rockfill is in the range of 150-250 if stresses are in lb/ft^2 (or about 40-70 if stresses are in kPa). Back analysis by Mejia[6] of the response of the Orovile Dam (California) to a weak seismic shaking gave K_{2max} = 170. In our study K_{2max} was varied parametrically between 150 and 250, if only to confirm that the conclusions regarding the performance of the dam are not sensitive to the exact value of this parameter. Static analyses were first carried out using the FE code ADINA[2], to obtain the effective mean principal stress σ_m in all elements. Poisson's ratio v for dry or nearly dry rockfill is taken equal to 0.25. Note that while v has only a marginal influence on lateral oscillations, it may play an appreciable role in rocking-type, as well as in longitudinal and especially vertical oscillations. Some limited data *is* available for estimating the decline of secant shear modulus, G, with increasing γ, in rockfill; they reveal a slightly faster rate of decline than for clayey soils. On the other hand, the data for the dependence of β on γ show a slightly faster rate of increase than for sands and clays[11]. Having modeled the plane geometry and dynamic properties of the dam, iterative 2D "equivalent linear" visco-elastic analyses were performed in which the shear modulus G and damping ratio β were obtained from the "effective" shear strain level of the previous iteration. The "effective" shear strain was obtained as the 2/3 fraction of the peak value. All analyses were performed with ADINA[2], which uses Newmark's time integration algorithm for a direct step-by-step solution.

3 RESULTS USING SINUSOIDAL GROUND MOTION

Fig. 2 sketches the studied rockfill dam. Two different models have been used, corresponding roughly to the cases of shallow and deep failure wedges in the dam (Interface 1 and Interface 2 respectively), as indicated by the FE meshes in the figure. Using the properties discussed in previous sections, the response of the dam is first analyzed in this section using as input a simple sinusoidal ground motion with five pulses of acceleration. (This was done in order to study some details of response that may be obscured by the erratic nature of an actual ground motion.)

(a)



(b)

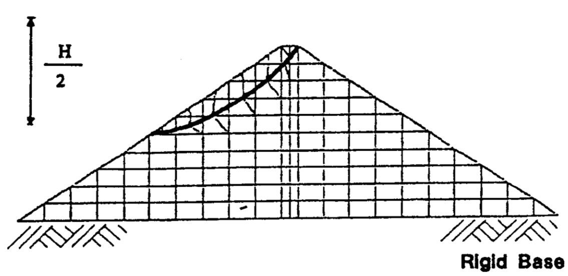


Fig. 2: Mesh with PEPSI: (a) A_y=0.3g (Interface 1), and (b) A_y = 0.5g (Interface 2)

The "critical" accelerations A_c of the sliding interfaces ("PEPSI") 1 and 2 were determined with slope stability calculations to equal about 0.30g and 0.50g, respectively. Ground excitation amplitudes of A=0.27g and 0.40g along with several different values of the frequency ratios f/f_1, where f is the excitation frequency and f_1 is the fundamental frequency of the dam, were applied. 20 interface elements model the sliding interface. Non-linearities may thus occur in only 20 elements of the finite element assemblage. Fig. 3 shows some details of typical results using the "Single-Step" method, at a frequency ratio f/f_1 = 0.75 and ground acceleration A = 0.40 g. Fig. 3a portrays the acceleration history of a point inside the sliding block, while Fig. 3b portrays the history of a point just outside the sliding block for the dam with "PEPSI" No. 1. First we note that the time history of Fig. 3b is rather similar to that derived for the intact dam, without a sliding interface (Fig. 3c). However, some high-frequency spikes are generated during the half-cycles where the wedge tends to move inward relative to the supporting dam ("sticking" phase). Similar spikes have been observed in other studies of sliding systems.

On the other hand, the acceleration histories of the points inside the PEPSI (Fig. 3a) consistently exhibit an asymmetric pattern, which is comparable to the acceleration response of a block sliding on an inclined plane. During the "sticking" phase, the absolute acceleration of the sliding mass is the same as that of the dam. During slippage, however, the absolute acceleration of the sliding mass deviates from that of the dam. A peak acceleration of roughly constant amplitude is maintained during the slip phase. Apparently, this asymmetry between negative and positive peak values of acceleration is due to the "truncation" of the positive peaks because of the slippage. (The maximum inertia force that can be transmitted to the sliding mass is limited by the "critical" acceleration of the wedge). The acceleration history of Fig. 3a, however, exhibits some sudden acceleration spikes, which are not present at the responses of Fig. 3b, for the points just outside the sliding block. It appears that these sharp spikes (observed at the end of each slipping phases), which will be explained later in this section, are due to additional dynamic excitation of the sliding mass triggered by the re-attachment of the sliding mass with the underlying body of the dam.

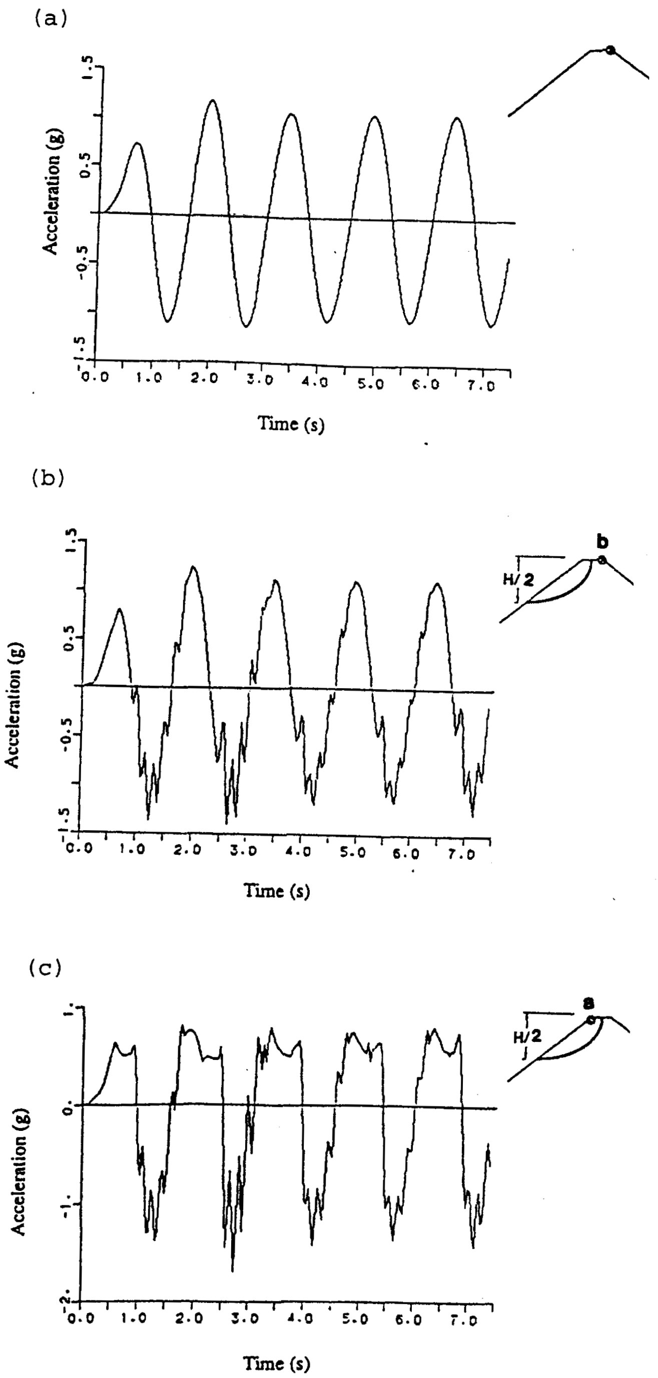


Fig. 3: Acceleration History Results: Single-Step Analysis for (a) a point inside the sliding mass and (b) a point outside the sliding mass, and (c) Two-Step Analysis for a point at the intact dam.

Fig. 4 shows the effect of the presence of "PEPSI" on the permanent displacement of the sliding mass at different excitation frequencies, for peak ground acceleration A = 0.27g and A = 0.40g. This effect is represented as the ratio **R** of the permanent

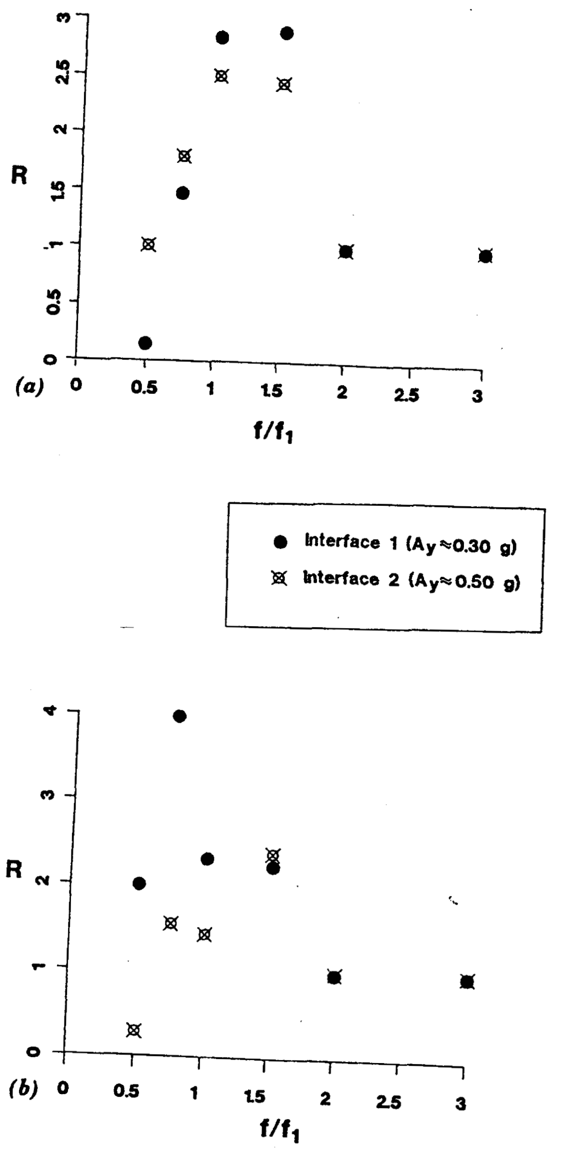


Fig. 4: Ratio R of Residual Displacements Computed with Two-Step (decoupled) and with Single-Step (PEPSI) Procedure

displacement from the "Two-Step" method to the permanent displacement from the "Single-Step" method, at the end of five cycles of motion. The following trends are worthy of note in Fig. 4:

1. The most severe overestimation of permanent displacement by "Two-Step" method occurs when the dam is excited close to its resonant frequency. This may be explained by the fact that in the "Single-Step" method, build-up of the response is drastically limited by the shearing strength of the interface - - - a constraint that is particularly effective at resonance. By contrast, the "Two-Step" method allows the dam response (and hence the driving

acceleration) to grow unrestrictedly, thereby leading to unrealistically high deformations.

2. for a low frequency ratios, e.g. $f/f_1 < 0.5$, the "Two-Step" method may underestimate the displacements whereas for $f/f_1 >> 1.0$, both "Single-Step" and "Decoupled" methods give similar results.

The above observations can be better appreciated by drawing the analogy with the study of Westermo and Udwadia[13], who considered the steady-state response of a simple oscillator supported on a frictional interface (Fig. 5a). The base excitation is harmonic with amplitude A and frequency ω while friction is of the Coulomb type with coefficient μ.

For every value of the frequency ratio ω/ω_n where $\omega_n = \sqrt{(k/m_1)}$ = the natural frequency of the fixed-base oscillator, slippage will occur whenever

$F = \mu g/A \le F_{critical}$, where $F_{critical} = (\beta^2 + 2\beta\cos\theta + 1)^{1/2}$, $\beta = \gamma\,(\omega/\omega_n)^2\,[(1-(\omega/\omega_n)^2)^2 + (2\zeta\omega/\omega_n)^2]^{1/2}$, θ is the phase angle and γ is the mass ratio $m_1/(m_1+m_2)$.

The above expression is plotted in Fig. 5(b),

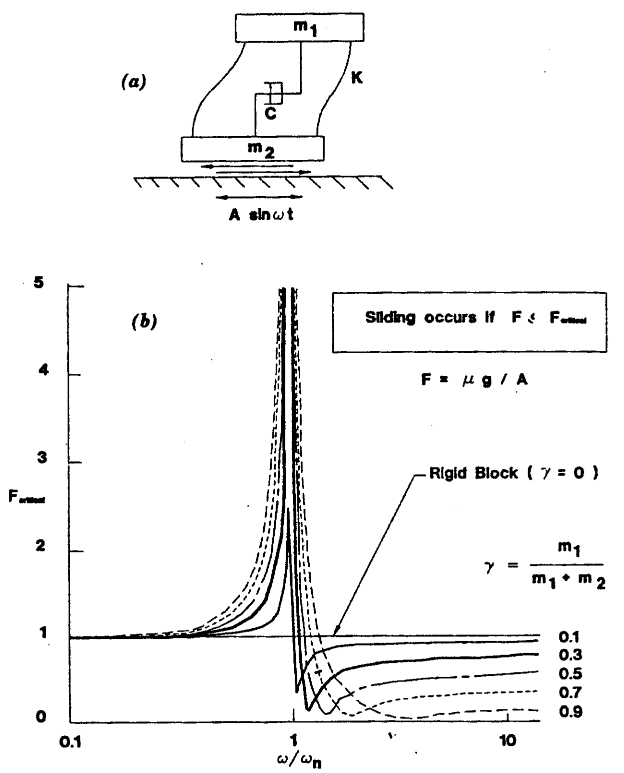


Fig. 5: Effect of Frequency Ratio Required to Initiate Oscillator Sliding

from which the following trends are noticed:

(i) Slippage of a rigid block ($\omega_n \to \infty$) initiates whenever $F < 1$ or $A > \mu g$, as of course expected.

(ii) Slippage of a flexible oscillator, on the other hand, initiates at higher values of F (and therefore lower values of A) if $\omega/\omega_n \approx 1$, and at lower values of F (higher value of A) if $\omega/\omega_n \gg 1$. This is because at $\omega/\omega_n \approx 1$ the shear force exerted on mass m_2 from the oscillating mass m_1 is much larger than the inertia force $m_2\ddot{u}$ that m_2 would have experienced if no sliding took place. Hence, sliding is sustained primarily by the resonant oscillation of mass m_1. Basically, then, during the transient part of the motion energy builds up in the oscillator, and at the steady state, the input energy balances the energy dissipated in frictional slippage and by viscous structural damping.

When $\omega/\omega_n > 1$, m_1 moves 180°-out-of-phase with the excitation and with m_2 (at least when there is no sliding). Therefore, the oscillation of m_1 acts in a "stabilizing" way and reduces the extent of sliding for a given acceleration level. Hence, the acceleration A needed to initiate sliding is larger than for a rigid block of mass m_1+m_2.

As we observed earlier that the acceleration-time history of Fig. 3a exhibit some sudden acceleration spikes which are not present at the response of Fig. 3b, which are just outside of the sliding block. To explain it we consider a finite element mesh with a horizontal sliding interface at the height of H/5. Yield acceleration of this sliding mass is artificially reduced to 0.30 g to pick up the details of the response, which would otherwise be impossible. The symmetry in the peak positive-negative value is due to the truncation of the peaks as a consequence of yield or slip in both directions. These peak responses represent a value of average 0.30g which is equal to the critical acceleration of the interface. Fig. 6 shows the crest acceleration response and its Fourier spectrum. Fig. 7 shows the same but artificially truncating the high acceleration spike (>0.30g) and its Fourier spectrum. By comparing these two Fourier spectra we can conclude that these sudden spikes are not the actual response of the dam but its source is somewhere else. It can be explained by the fact that the friction force is always opposed to the motion. The second derivative of the motion is the acceleration, which is proportional to the sum of all the forces acting on the mass. Since the frictional

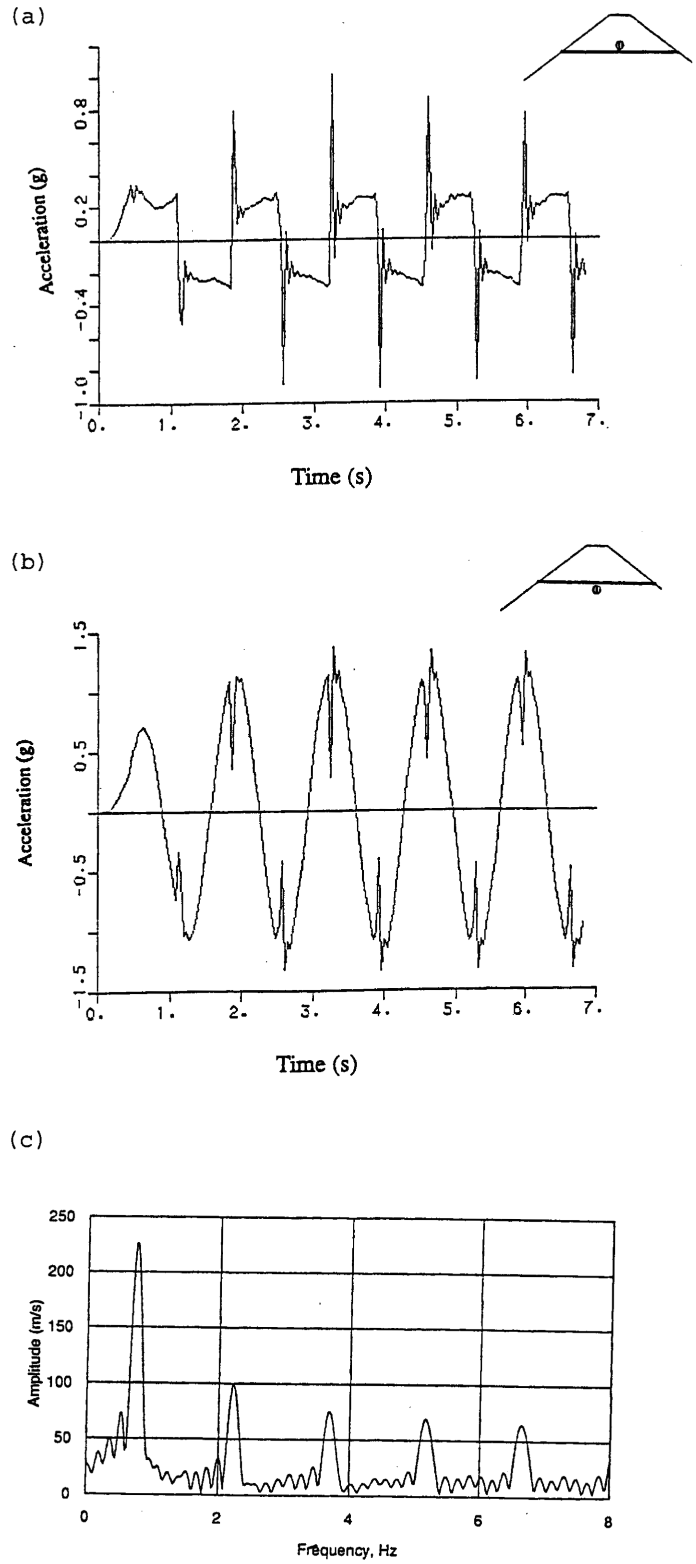


Fig. 6: Response of Dam with Horizontal Interface: Acceleration Histories (a) within the sliding mass, (b) Outside the sliding mass, and (c) response spectrum for the acceleration within sliding mass

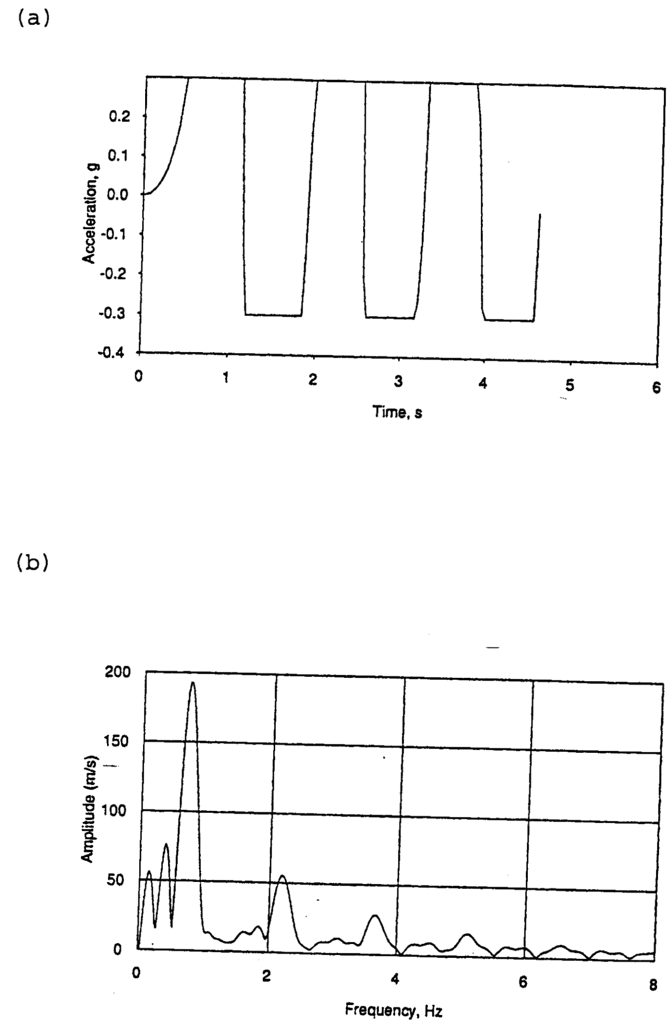


Fig. 7: Acceleration Response of Fig. 6(a) but with Truncated Spikes (>0.3g) and (b) its Response Spectrum

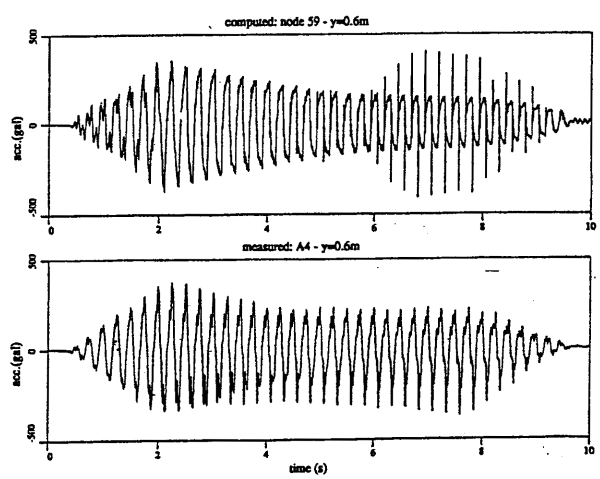


Fig. 8: Measured Horizontal acceleration Record at Central Section (Prevost, J.H., 1992, Personal Communication)

force suddenly reverses at the peaks, the curvature there must show a discontinuous change. Hence motion continues with a smooth transition in displacement and velocity but with a jump in acceleration i.e., slip occurs.

It appears therefore that sharp spikes observed at the end of each sliding phase are due to additional dynamic excitation of the sliding mass triggered by the reunion of the sliding mass with the dam. The abrupt change in inertial force associated with this reunion may also cause a recording strong motion instrument to overshoot the response observed by J. H. Prevost and R. Popescu as shown in Fig. 8.

4 SUMMARY AND CONCLUSION

Estimation of the permanent deformations of embankment dams in practice is based on the simplifying assumption that dynamic acceleration response and wedge sliding are two separate processes (decoupled "elastic" and "rigid-slip" features of dynamic response). An alternative hypothesis has been utilized in this paper, namely that these two processes occur simultaneously. To this end, the dam was assumed to contain an *a priori* assigned potentially sliding interface, and the dynamic response was computed in a *Single Step*. The results for both harmonic and real-earthquake type excitations have provided a considerable insight to the problem. It has been found that the Two-Step approximation leads to usually (but not always) conservative estimates of sliding deformations, compared with the Single-Step approach. The largest overestimation occurs when the predominant frequency of excitation is in the broad neighborhood of the fundamental frequency of the dam. The charts in Figs. 6 and 7 can be used to estimate this overestimation for a harmonic excitation.

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