

A NEW DYNAMIC MODEL FOR EARTH DAMS EVALUATED THROUGH CASE HISTORIES

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ABSTRACT

Static finite element analyses and laboratory measurements suggest that the shear modulus of soil in earth dams and embankments increases approximately as the $2/3$ -power of the distance from the crest, a variation that is also confirmed from field measurements of shear wave velocities in actual earth and rockfill dams. A shear-beam model has been developed and is outlined in this paper that accounts for such an increase of modulus with depth. The main thrust of the paper is to evaluate this model by comparing its predictions with the recorded response of a number of dams and embankments subjected to earthquake or man-induced vibrations. It is demonstrated that the model successfully explains observed modal displacement shapes, peak accelerations and seismic coefficients experienced by several Japanese, one American and one European dam, as well as by a large-scale laboratory embankment model. Consequently, the model offers a significant improvement over the classical, homogeneous shear beam model that is presently used to evaluate the seismic safety of earth dams.

Key words: case history, dam, deformation, dynamic, earthfill, earthquake, heterogeneity, soil dynamics, vibration

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INTRODUCTION

Earth dams are three-dimensional structures of complicated geometry, made up of material whose behaviour during cyclic loading is very difficult to accurately predict and describe in mathematical terms. Moreover, earthquake ground motions are of a random nature, consisting of body and surface waves that strike at various directions in an unpredictable way. Therefore, precise detailed evaluation of the seismic response of earth dams is a formidable task and, in view of the tens-of-thousands of such dams that exist and are built in seismic regions throughout the world, engineers have developed a variety of simplified dynamic models that are used as a guide in the aseismic design process.

The viscoelastic 'shear-beam' model, employed in order to estimate the characteristics of lateral vibrations of earth dams built in relatively wide canyons, is the simplest and most widely used among a number of dynamic models (Mononobe et al., 1936; Hatanaka, 1955; Ambraseys, 1960; Seed et al., 1966). Based on the assumption of uniform horizontal lateral deformations and stresses, the model is very reliable in predicting natural fre-

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quencies, horizontal seismic accelerations and seismic coefficients, as has been demonstrated through comparative analytical studies (e.g., Chopra et al., 1969) and actual field measurements (e.g., Abdel-Ghaffar et al., 1978). In its present form the model makes another, unrealistic simplification by assuming the soil properties to be constant throughout the dam. It is well known, however, that soil modulus is a function of the effective mean stress, σ'_0 , which in an earth dam varies from point to point, being very small near the crest and very large near the center of the base of the dam.

This paper first shows that, with sufficient degree of accuracy, the average shear modulus, \bar{G} , over a horizontal plane increases in proportion to the $2/3$ -power of the distance, z , from the crest. An improved 'inhomogeneous' shear beam model is then outlined that can account for this variation in G while retaining the simplicity of the 'homogeneous' model. The model is evaluated through numerous case histories involving earth and rockfill dams. Close agreement exists between predicted and observed modal displacement shapes, peak accelerations and seismic coefficients. Phenomena such as the repeatedly observed sharp amplification of motions at the upper third of the dam can easily be explained with this model, whereas the classical 'homogeneous' shear beam theory fails to anticipate their occurrence or explain their causes. Therefore, it is believed that continuing use of the classical model in practice is unjustified. The proposed inhomogeneous model offers a simple rational alternative and its substantiation by a large body of field and experimental data guarantees its success in aseismic design of earth or rockfill dams and embankments.

EXPERIMENTAL-ANALYTICAL EVIDENCE ON THE VARIATION OF SHEAR MODULUS IN EARTH DAMS

Numerous experimental investigations (e.g., Hardin et al., 1972; Ohsaki et al., 1973) have confirmed that, in most soils, shear modulus depends primarily on the average effective confining pressure, σ'_0 , the void ratio, e , and the shearing strain amplitude, γ . The low-amplitude shear modulus, G , can be expressed mathematically as

$$G = F(e)(\sqrt{\sigma'_0}) \quad (1)$$

where $F(e)$ is a function of e that varies from soil to soil. In earth dams e is primarily a function of the placement procedure (e.g. hydraulic fill or rolled fill, etc.) and the compaction effort; thus, one can state that approximately e and $F(e)$ are constant throughout the dam. Eq. (1) then indicates that G is proportional to the square-root of the effective confining stress, σ'_0 . The spatial distribution of σ'_0 within a dam can be estimated by finite element or finite difference analyses, (e.g. Clough et al., 1967; Poulos et al., 1974).

Fig. 1 displays the contours of ver-

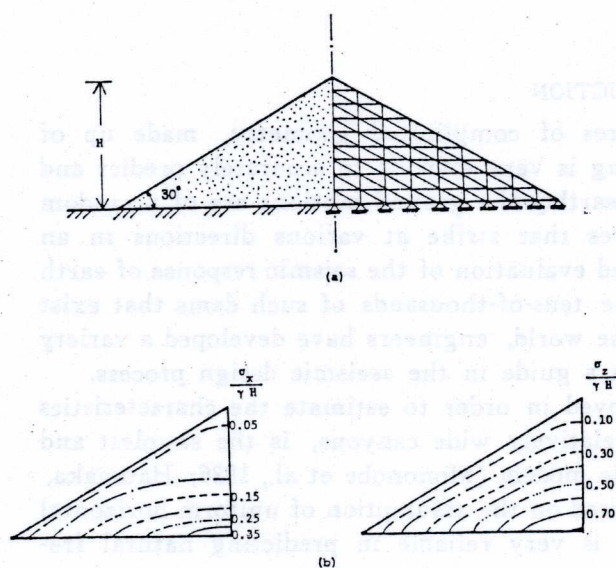


Fig. 1. (a) Finite-element discretization of a 30°-slope dam;
(b) Contours of horizontal (σ_x) and vertical (σ_z) normal stresses

tical and horizontal normal stresses in a 30°-slope earth dam made up of material with Poisson's ratio $\nu=0.30$. They were computed from an elastic finite-element analysis assuming "single-lift" construction (Poulos et al., 1974). Clough et al. (1967) have shown that the distribution of the above stresses is insensitive to realistic changes of Poisson's ratio, slope of the dam or consideration of a "mult-lift" construction. Therefore the stress distributions of Fig. 1 are typical of a variety of earth dams.

Reading σ_z and σ_x from the said figure and realizing that for plane-strain conditions

$$\sigma_y = \nu(\sigma_x + \sigma_z), \quad (2)$$

one can determine the mean stress, $\sigma_0 = 1/3(\sigma_x + \sigma_y + \sigma_z)$, and from Eq. (1) the shear modulus, G , in terms of $F(e)$. Fig. 2(a) portrays the distribution across horizontal planes of G , normalized with the maximum modulus, G_m . The latter obviously is found at the center of the base of the dam. One may observe that G is nearly uniform in the horizontal direction but increases appreciably with depth in the vertical direction. In fact, the average modulus over a horizontal plane, \bar{G} , which is used in the shear beam model, increases with distance from the crest as shown in Fig. 2(b). The agreement with the curve

$$\frac{\bar{G}}{G_m} = \left(\frac{z}{H}\right)^{2/3} \quad (3)$$

which is also plotted in Fig. 2(b), is excellent. Thus, Eq. (3) expresses very precisely the actual variation of modulus with depth and should be reflected in an improved dynamic shear beam model.

Since earth dams usually consist of more than one material (e. g. clayey core and sandy shell) and the pore-water pressures are not uniformly distributed but rather increase with depth below the seepage line, one might expect that Eq. (3) approximates the variation of G in actual earth or rockfill dams more accurately than in dams consisting of uniform material (as in Figs. 1, 2). Yet the results of some direct field measurements that are presented next clearly demonstrate that Eq. (3) is, in fact, adequate for many types of dams.

FIELD EVIDENCE ON THE VARIATION OF SHEAR MODULUS IN EARTH DAMS

Abdel-Ghaffar & Scott (1978) have reported a systematic investigation of the dynamic response of Santa Felicia Dam in Southern California. Santa Felicia (shown in Fig. 3(a)) is a modern rolled-fill embankment-dam with a central impervious clayey core and pervious shell upstream and downstream. It was subjected to strong shaking during the San Fernando earthquake of February 9, 1971 (maximum acceleration about 0.21 g) and to moderately strong shaking during Southern California earthquake of April 8, 1976

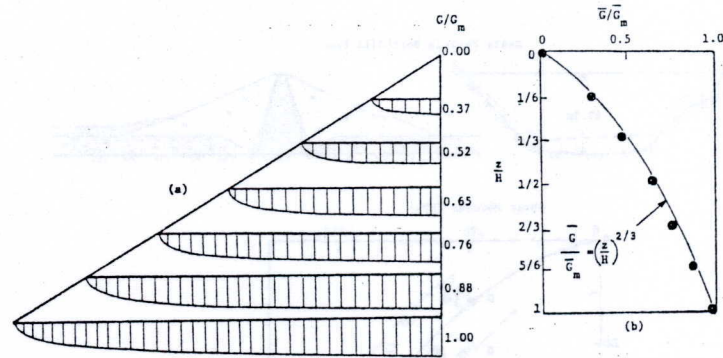


Fig. 2. (a) Distribution of shear modulus along six horizontal cross-sections; (b) Variation with depth from the crest of the average shear modulus over horizontal cross-sections

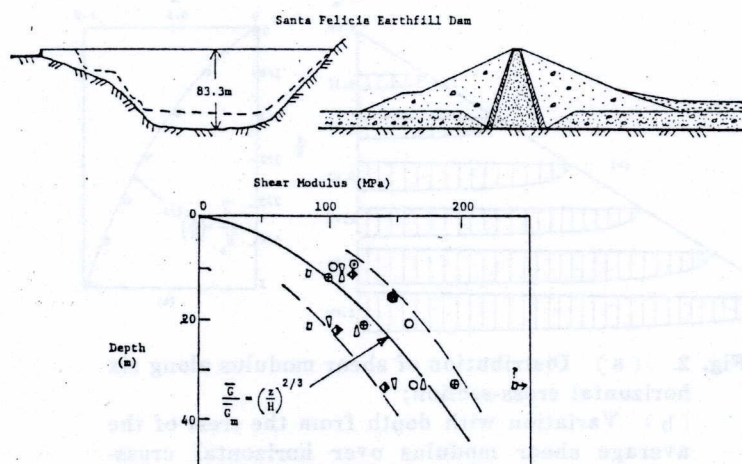


Fig. 3. Comparison of the variation of shear modulus in Santa Felicia Dam, obtained from field S-wave velocity measurements, with the theoretical-experimental distribution with depth (Eq. (3)).

upstream slope triggered P and S waves; the arrival time of these waves traveling along various paths inside the dam was accurately determined by means of a geophone and a signal enhancement seismograph. Thus S and P-wave velocities and then soil moduli were calculated.

Fig.3(b) portrays the variation with depth of the shear modulus determined from different tests; the area between the two dashed lines is suggested as the region where data points are clustered. Also shown for comparison is the curve corresponding to the 2/3-power variation law, i.e., Eq. (3), whose general applicability is herein under investigation. And indeed, the agreement is very good; the theoretical line passes through the center of the band defined by the two dashed lines and looks like an average line of all the data points.

Minami (1969) reported estimates of the shear wave velocities of two dams in Japan, Makio and Togo Dams. Makio is a rockfill dam with a clayey central core and a height of about 85 meters, while Togo is an earthfill dam with an inclined clayey core and a height of about 31 meters. A large number of seismometers were installed at various depths inside the dam and on the two slopes. By observing the time lags of first S-wave arrivals in the various seismometers when the dam was subjected to earthquake shaking, shear-wave velocities were estimated. Although, to the author's knowledge there is no documented evidence on the accuracy of this method, one can certainly accept the reported measurements as providing an indication of the variation of soil stiffness with depth. Thus, for Makio Dam, the S-wave velocity near the crest was estimated to be $C=350$ m/s while at the base $C_m=1000$ m/s. For Togo Dam the corresponding wave speeds were $C=125$ m/s and $C_m=250$ m/s. Table 1 compares the near-crest to base observed velocity ratio, C/C_m with the ratio computed from Eq. (3): $C/C_m = (z/H)^{1/3}$. H and z were estimated from the geometry of the dam and the location of the seismometers, as shown by Minami, 1969. The agreement between observed and computed values is again very good.

In conclusion, shear modulus in earth and rockfill dams varies in a manner which is best described with Eq. (3), that is, the average G over a horizontal plane increases as

(maximum acceleration about 0.05 g). Subsequently, full-scale dynamic tests were carried out, including forced and ambient vibration as well as popper tests. Important part of the whole investigation was the determination of soil stiffness through field wave-velocity measurements carried out on the dam. Thus representative values of the dam's material properties and their variation with depth were estimated. The wave-velocity measurements were performed using a sledge hammer whose impacts at a series of hammer stations located along the crest and the

the 2/3-power of the distance from the crest. Of course, additional systematic field measurements in existing or under construction dams could be quite helpful in establishing the reliability of Eq. (3) for a wider variety of embankment dams, than the

one considered above. Nevertheless, the development of a dynamic 'inhomogeneous' shear beam model that accounts for this increase of G with depth is at present greatly justified.

Table 1. Observed and predicted velocity ratios in two Japanese dams

Makio Dam		Togo Dam	
Observed C/C_m	Predicted C/C_m	Observed C/C_m	Predicted C/C_m
0.35	0.37	0.50	0.43

INHOMOGENEOUS SHEAR BEAM MODEL

The shear beam model is based on two fundamental assumptions: (a) only horizontal shear deformations take place; (b) the resulting shear stresses are uniformly distributed over any horizontal cross-section. Although of a simplifying nature, the two assumptions are reasonable approximations to a complicated pattern of stresses and deformations. Thus, although rocking and vertical deformations take place as a result of flexural-type deformations (Hatanaka, 1955) and of wave reflections on the faces of the dam (Ishizaki & Hatakeyama, 1962), horizontal deformations are, in fact, predominant in all but a few frequencies of vibration (e.g., Chopra et al., 1969). Also the distribution of shear stresses on horizontal planes may not be very different from a uniform one, as can be seen in Fig. 4 which displays the shear stresses that developed at a critical instant of time in an earth dam subjected to the El Centro 1940 NS accelerogram. The analysis was made with a plane-strain finite element formulation (Chopra et al., 1969) for a dam with relatively steep slopes (1.5 horizontal on 1 vertical). Dams with more realistic flatter slopes (e.g., 3 on 1) would lead to even more uniform shear-stress distribution.

With the above two assumptions, the following equation of motion governs the space and time variation of the horizontal displacement, u , in a dam modelled as an inhomogeneous truncated wedge (Fig. 5):

$$\rho \frac{\partial^2 u}{\partial t^2} = \frac{1}{z} \frac{\partial}{\partial z} \left[\bar{G}(z) \cdot z \cdot \frac{\partial u}{\partial z} \right] \quad (4)$$

The reader is referred to Ambraseys (1960) or Okamoto (1973) for a detailed derivation of Eq. (4) which, together with the boundary conditions of zero displacements at the dam-base interface and zero shear stresses at the crest, states an algebraic eigenvalue problem. If $G(z)$ is described by Eq. (3), an exact closed form solution to this problem has been obtained by the author (Gazetas, 1981). Only the results are summarized here.

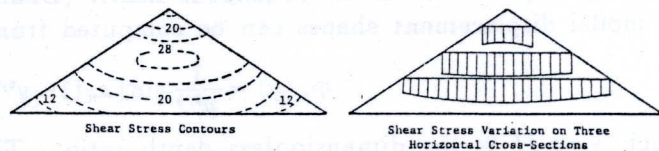


Fig. 4. The almost uniform distribution of shear stresses τ_{zx} that develop on horizontal planes in a dam subjected to earthquake excitation suggests that 'shear beam' is a realistic model of the dynamic response of earth dams

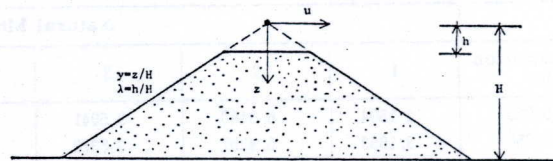


Fig. 5. Coordinate axes and notation

The natural periods of the dam are given by

$$T_n = \frac{3\pi}{a_n} \frac{H}{\bar{C}_m} \quad (n=1, 2, 3, \dots) \quad (5)$$

in which: $\bar{C}_m = (\bar{G}_m/\rho)^{1/2}$ is the base shear wave velocity and a_n is a function of the truncation ratio (See Fig. 5) for each n ; it has been numerically calculated and is offered in Table 2 for the first six modes of vibration ($n=1-6$) and the complete spectrum of truncation ratios ($\lambda=0-1$). In the interesting case of very small λ (tall dams), Eq. (5) yields for the fundamental period:

$$T_1 \approx 2.57 \frac{H}{C_s} \quad (6)$$

where $\bar{C}_a = (6/7)\bar{C}_m$ is the average velocity of the dam. The value of T_1 from the above equation is only 2% smaller than the one deduced from the classical homogeneous shear beam theory (Okamoto, 1973). Notice also that for very small λ Eq. (5) yields

$$T_n = T_1/n \quad (7)$$

which implies that the higher natural periods are closer to each other than the classical theory predicts. For example, $T_2 = T_1/2$ and $T_3 = T_1/3$ from Eq. (7) as opposed to $T_2 \simeq T_1/2.34$ and $T_3 \simeq T_1/3.78$ of the homogeneous model (Okamoto, 1973).

The modal displacement shapes can be computed from

$$\phi_n(y) = \frac{1}{y^{2/3}} \sin[a_n(1-y^{2/3})] \quad (8)$$

in which $y=z/H$ is the dimensionless depth ratio. The reader is referred to Gazetas 1981, for a detailed derivation of Eq.(8). However, direct substitution of (8) in (4) is sufficient to show its validity. From Φ_n one can easily derive expressions for the modal shapes of shear stress, τ_n , shear strain, γ_n , and seismic coefficient on a potential sliding mass, k_n . The response of the dam to an earthquake can then be evaluated by using the

Table 2. Roots (a_n) of the period relation[illegible]

principle of modal superposition. The participation factor of each mode in the total response is

$$\Gamma_n = \frac{\int_h^H \phi_n z dz}{\int_h^H \phi_n^2 z dz} = \frac{2}{a_n} \frac{1}{1 - \lambda^{2/3} - \sin[2a_n(1 - \lambda^{2/3})]/2a_n} \quad (9)$$

For very small truncation ratios λ , Eq. (9) simplifies to $\Gamma_n = 2/n\pi$.

The ability of the inhomogeneous model to predict the dynamic behaviour of earth or rockfill dams is now evaluated through comparisons with the observed response of actual or large-scale dams to earthquake or man-induced vibrations.

CASE HISTORIES

Actual Dams During Earthquakes

Mori and Kawakami (1975) and Okamoto (1973) have reported studies of the behaviour of two earth and two rockfill dams during a number of earthquakes recorded on seismometers that had been installed at the crest, the downstream slope, the abutments, as well as at various locations inside each dam. The results were presented in the form of acceleration time-histories, Fourier amplitude spectra and amplification spectra of the dam, whereby natural frequencies, modal displacement and average shear wave velocities were estimated.

A comparison between the observed fundamental modal shapes and those predicted by the inhomogeneous shear beam theory is first presented for three of the dams, Ainono, Ushino and Sannokai. Since no field wave-velocity measurements were carried out, the mentioned authors backfigured average shear wave velocities from the observed fundamental frequencies of the dams by using the classical homogeneous shear beam theory. As shown previously, inhomogeneous and homogeneous theory yield almost identical fundamental frequencies. Thus no comparison can be made herein regarding fundamental frequencies and the discussion is restricted to mode shapes.

Lateral cross-sections of the three dams are shown in Fig. 6. Ainono Dam (completed in 1961) is an earth-fill dam with maximum height of 40.8 m and crest length of 132.9 m. It is composed of 28% sand, 40% silt and 32% clay and has an inserted drainage filter.

The foundation consists of hard shale. Ushino Dam (completed in 1968) is a rock-

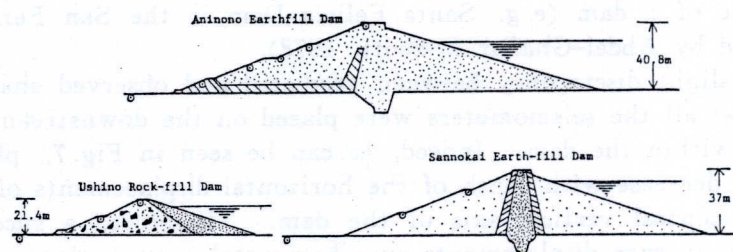


Fig. 6. Cross-sections of three Japanese dams (Refs. 12 and 15)

fill dam with an inclined impervious clayey core forming the upstream slope. Its maximum height is 21.4 m, its crest length 160 m and it is founded on tuff. (More details about these two dams can be found in Mori & Kawakami, 1975). Sannokai Dam is an earthfill dam with a central impervious clay core, maximum height of 37 m and crest length of 145 m. It is also founded on green tuff. (See Okamoto, 1973, for additional information.)

A large number of distant earthquakes triggered the seismometers installed in the three

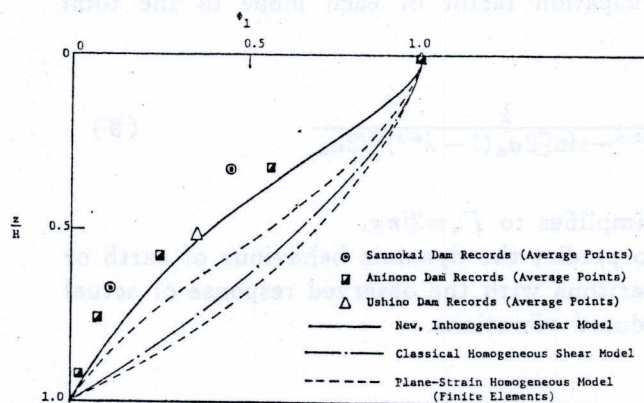


Fig. 7. The proposed, inhomogeneous shear beam model explains the average recorded fundamental mode shapes of the three dams with sufficiently greater accuracy than the currently available models (1 or 2-Dimensional) that assume constant soil modulus with depth

genous (in the vertical direction) dam. Very little similarity appears to exist between observed and predicted mode shapes. This, however, is no longer true when the new inhomogeneous theory is used. Indeed, for the very small truncation ratios of the three dams, $a_n \approx n\pi$ from Table 2; Eq. (8) then yields for the fundamental shape:

$$\phi_1 = y^{-2/3} \sin[\pi(1 - y^{2/3})] \quad (10)$$

Eq. (10), normalized to a unit crest displacement, is also displayed in Fig. 7. The agreement with field data points is very good. This fact is an additional evidence of the need to consider the variation of modulus with depth in dynamic analysis of earth/rockfill dams. Furthermore, it provides confidence on the ability of the presented inhomogeneous shear beam theory to satisfactorily predict fundamental mode shapes. Note that very often vibrations in the fundamental mode dominate the earthquake response of a dam (e.g. Santa Felicia Dam in the San Fernando 1971 earthquake, as reported by Abdel-Ghaffar & Scott, 1978).

The slight discrepancy between predicted and observed shapes is probably due to the fact that all the seismometers were placed on the downstream slope and not at 'average' points within the dam. Indeed, as can be seen in Fig. 7, plane-strain analysis predicts sharper decrease with depth of the horizontal displacements of the two slopes than those of the central vertical axis of the dam. Therefore, a successful shear beam model, yielding average displacements over horizontal cross-sections of the dam, must predict a slightly milder deamplification with depth. In other words, the agreement of Eq. (10) with the actual, average fundamental mode response of the three dams is probably even closer than what Fig. 7 indicates.

Another case study is now presented to further demonstrate the potential of the advocated new dynamic model. A moderately strong earthquake was recorded on September 9, 1969, with a number of seismometers inside the Kiseniyama Dam in Japan, as reported by Okamoto (1973). Kiseniyama Dam, shown in cross-section in Fig. 8(a), is a

* To account for the canyon geometry, 'effective' heights were considered for each dam smaller than their maximum height. Thus, for the Sannokai Dam, H was taken equal to 32 m; for the Ainono Dam, $H=35$ m; and for the Ushino Dam, $H=19$ m.

dams. By analysing the Fourier amplitude spectra and the amplification spectra of the recorded motions the above-mentioned authors determined the natural frequencies and the corresponding normalized deflection curves (modal shapes) of each dam. Fig. 7 portrays the average normalized deflection from 5 records of the Sannokai Dam, 6 records of the Ainono Dam and 6 records of the Ushino Dam.* Notice the relatively sharp attenuation of motions with depth from the crest. Also shown in this Figure for comparison are the fundamental mode shapes predicted by the classical shear beam theory and by plane-strain finite-element analysis of a homo-

tall rockfill dam completed in 1969, shortly before the earthquake. It is 95 m high and 267 m long at the crest; it has a central clayey core and is founded on rock.

During the earthquake the dam experienced very high acceleration levels only at and near the crest while below the upper third the motion was much smaller and essentially equal to the ground motion. Fig. 8 (b) depicts 6 accelerograms recorded at corresponding points in the dam and on the ground. Fig. 8 (c) displays the distribution of the peaks of these accelerograms with depth from crest.

The very sharp amplification in the upper third of the dam is apparent. This author attributes the phenomenon to the important contribution of the higher modes which have a 'whip-lash' effect on such a tall (therefore flexible) structure. Notice, nevertheless, the satisfactory performance of the inhomogeneous shear beam model. Using the ground record as input and assuming 5% damping ratio in the fundamental mode and 8% in the higher modes, the curve shown in Fig. 8(c) is computed.* It clearly exhibits the large amplification of motion at the upper third and only slightly underpredicts the peak crest acceleration. On the contrary the classical homogeneous theory fails to even roughly anticipate the actual behaviour.

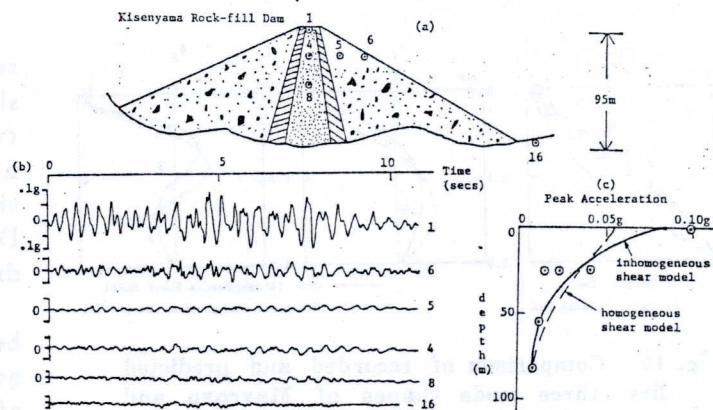


Fig. 8. (a) Cross-section of the Kiseniyama rock-fill dam in Japan;

(b) Acceleration time-histories recorded at various points in the dam during a moderately strong earthquake;

(c) The proposed inhomogeneous model predicts with good accuracy the recorded peak acceleration distribution with depth

Full-Scale Dynamic Tests

Petrovski et al. (1972) and Abdel-Ghaffar & Scott (1979) have reported the results of full-scale forced vibration tests involving the Mavrovo Dam in Yugoslavia and the Santa Felicia Dam in California, USA, respectively. Both groups excited the dam into resonance in various modes (in the upstream downstream as well as in the longitudinal direction) by a coupled pair of mechanical shakers capable of producing a total force up to 45,000 kN. The motion was recorded on several seismometers located at selected measurement stations covering the entire crest and downstream slope of the dam.

Santa Felicia Dam has already been described (Fig. 3(b)). Mavrovo is also an earthfill dam with central clayey core and an upstream rock-fill shell (Fig. 9). Its maximum height is 56 m but an effective height of 40m was assumed in our analysis, to account for the effect of the V-shaped canyon**. The length of the crest is 215 m and the foundation consists of weathered diabase.

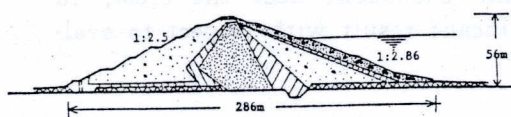


Fig. 9. The Mavrovo earth-fill dam in Yugoslavia

* only five modes were considered in the analysis

** In fact, the records showed that below this depth (approximately) the motions essentially always vanished.

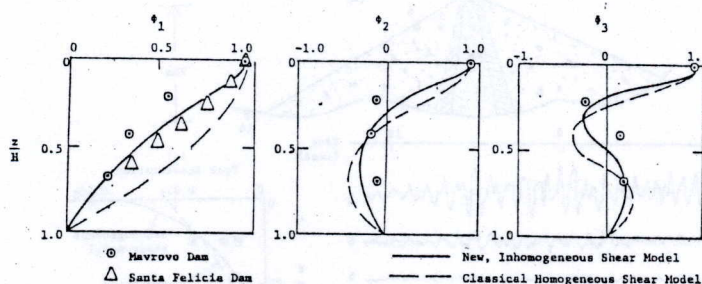


Fig. 10. Comparison of recorded and predicted first three mode shapes of Mavrovo and Santa Felicia dams

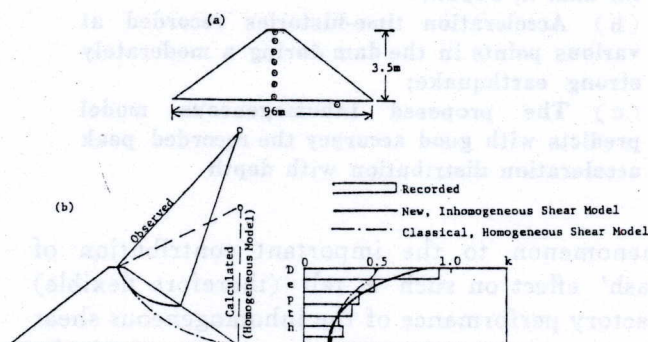


Fig. 11. (a) Scale model of a dam tested on a shaking Table (Ref. 13);
(b) the recorded high values of the seismic coefficient in the upper third of the dam cause shallow sliding failures which only the proposed theory can anticipate

(Fig. 11(a)) was subjected to sinusoidal excitations through a large shaking table consisting of a 14.5 m long by 6 m wide and by 3 m high reinforced concrete box mounted on eight steel pipe piles (Noda et al., 1973). Detailed results of the experimental investigation can be found in the original Reference. Here, in Fig. 11(b), the observed distribution of seismic coefficient, k , with depth of the corresponding sliding mass are depicted for a frequency $f=6.67$ cps. As a result of the much higher values of k near the crest, the embankment experienced a shallow sliding failure that was restricted in the upper half of the dam, as shown in Fig. 11(b). The classical shear beam theory again fails to explain the sharp amplification of the seismic coefficient near the crest, in contrast with the presented theory. This is a significant result with respect to evaluating the safety of dams during earthquakes.

SUMMARY AND CONCLUDING REMARKS

Because of the dependence of soil properties on the effective normal octahedral stress, shear modulus in earth or rockfill dams is not constant but increases with the $2/3$ -power of depth from the crest. Analytical, laboratory and field evidence is presented in the paper that verifies that such a variation is representative of actual earth dams and a new 'inhomogeneous' shear-beam model is outlined to account for the phenomenon. To demonstrate

Fig. 10 compares the three first recorded modal displacement shapes with the predictions of the two shear beam theories. (Only first mode data are available and shown for the Santa Felicia Dam.) Two conclusions can be drawn from this comparison:

(1) The inhomogeneous shear beam model (Eq. (8)) can adequately foresee the distribution of displacements with depth in all three modes; this is hardly true for the homogeneous model.

(2) The differences between the new theory and reality are largest in the two higher modes, a result also confirmed from the recorded second mode shape of the Sannokai Dam (not shown here). It is believed that rocking vibrations, which take part in the motion but can not be accommodated with any shear beam theory, are primarily responsible for the observed discrepancies.

Shaking-Table Tests of Large-Scale Embankments

A 3.5 m high embankment

the reliability of the model, case histories are then presented, involving a large number of actual earth and rockfill dams. The model very satisfactorily predicts the observed response of these dams to several earthquakes and to man-induced or ambient vibrations, and thus it constitutes a useful engineering tool in the aseismic analysis and design of embankment dams.

In closing this paper, the Author wishes to emphasize once again that the 'shear-beam' assumption, which is the basis of the proposed model, may in some cases only crudely approximate reality, and certain phenomena that are ignored may have an effect on the dynamic performance of earth dams. For instance, in dams built on narrow valleys, three-dimensional effects can be of importance and their consequence should, therefore, properly be assessed. Three-dimensional dynamic finite-element analyses are certainly a possibility and a technically sound choice for such a task. However, computer storage and budget limitations make such analyses a less attractive alternative, especially during the design process when the engineer has not yet decided on the geometry and exact location of the dam and has, at best, only a crude knowledge of the geotechnical properties of its materials. Thus, in practice, finite-element analyses are not undertaken except, perhaps, at the final stage of very important projects in order to verify conclusions drawn by simpler design-oriented methods, such as the one proposed in the paper.

NOTATION

- a_n =eigenvalue parameter that is function of n and the truncation ratio; it is given in Table 2
 C =shear-wave velocity
 \bar{C} =average C over a horizontal plane section
 e =void ratio
 G =shear modulus of soil
 \bar{G} =average G over a horizontal plane section
 H =height of dam (measured from the origin of coordinate axes)
 T_n = n th natural period of vibration (Eq. (5))
 Γ_n =participation factor of n th mode (Eq. (5))
 Φ_n =displacement shape of n th mode (Eq. (8))

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