CHARACTERISTIC PERIODS OF COHESIVE SOIL-FOUNDATION SYSTEMS

by

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ABSTRACT

Characteristic periods associated with both resonant and zero force level phenomena and important in the design of soil-structure systems under transient loadings are presented for prototype circular footings supported on a cohesive soil. Interrelated effects of footing diameters ranging from 5 ft-2 in to 10 ft-4 in, total weights from 6.41 tons to 25.64 tons, static pressure levels from 2.56 psi to 10.25 psi, and level of dynamic loading are considered. Periods vary as a power of the total weight and inversely as a power of the contact footing area. Dynamic force level effects indicate a nonlinear (underlinear) nature for the cohesive soil-foundation systems considered.

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INTRODUCTION

A primary factor in the design and analysis of soil-structure systems subjected to transient loadings, including those of a blast or earthquake nature, is the relative values of the characteristic time parameters of the disturbance or excitation function and the soil-structure system. For simplicity, consider a single degree of freedom situation with T the characteristic period of the soil-structure system and τ the characteristic period of the excitation. The ratio of the characteristic periods, τ/T, is a parameter of primary interest because it is a measure of the degree of "tuning" between the excitation and the soil-structure system. The response of the soil-structure system can be expressed in terms of a selected quantity in the response taken with reference to a quantity in the excitation, as a function of the ratio τ/T. Response-excitation quantity ratios are usually expressed as dimensionless ratios of characteristic displacements, velocities, or accelerations. Depending upon the excitation and system under consideration, certain values of the ratio τ/T may indicate excessive response characteristics leading to large settlements and structural damage to both the soil-structure system itself and adjacent structures. Since the characteristic time parameters, τ , of the excitation can be anticipated with reasonable accuracy, it is extremely important to be able to determine the characteristic periods of soil-structure systems in order that they may be designed in such a manner as to avoid undesirable response.

A particularly difficult aspect of the analysis of such systems is the determination of the characteristic period of the soil-foundation part of the system. The period of a soil-foundation system is a function of many factors, including size, mass, and geometry of the foundation, properties of the soil supporting the foundation, static loads being transmitted through the foundation, and nature as well as magnitude of the excitation. The interrelated effects of these factors on the characteristic period needs clarification. It is generally recognized that the response of a soil-foundation system is a nonlinear problem of a highly indeterminate nature. Highly simplified idealizations of such a system have led to useful solutions but they are limited with regard to the scope of their applicability to represent actual field conditions. In addition, most experimental studies of characteristic periods or resonant frequencies of soil-foundation systems have been on models or relatively small footings, with prototype investigations quite limited in scope.

The present paper deals with the effects of footing size, total weight, and level of loading on the vertical mode characteristic period of circular footings supported on the surface of a cohesive soil. The characteristic period, T, is expressed in terms of the resonant frequency, f, of the soil-foundation system and written as

$$T = \frac{1}{f} \qquad (1)$$

where f is expressed in cycles per second. The analysis is based on the results of a number of vertical mode tests of large scale reinforced concrete circular footings ranging from 5 ft-2 in to 10 ft-4 in in diameter.

EXPERIMENTAL PROGRAM

The author has been involved in the analysis of the results of a number of tests of reinforced concrete, circular, prototype footings supported on cohesive soil and subjected to vertical sinusoidal forces generated by a centrifugal force device due to a rotating eccentrically mounted mass. The footing diameters considered are 5 ft-2 in, 7 ft-4 in, 9 ft-2 in, and 10 ft-4 in with total weights ranging from 12,820 lbs to 51,280 lbs and applied force amplitudes between 525 lbs and 52,000 lbs. The footings were supported on the surface of a relatively uniform silty clay. Unfortunately, extensive soil test data were not available from the test area. Available information indicates the following typical soil characteristics; weight density of 120 lbs per cu ft, compression modulus varying with depth from approximately 10,500 psi near the surface to 22,000 psi at 29 ft, and a shear modulus ranging from approximately 4,000 psi to 8,500 psi over the same depth. Moduli were determined using seismic methods. Footing weight, weight of vibrator, and symmetrically secured ballast give static pressures ranging from 2.56 psi to 10.25 psi. For each footing, a particular eccentricity was selected for a constant magnitude of eccentric mass and steady state conditions were obtained for various values of frequency ranging from 6 cps to 30 cps, subject to the limitations of the vibrator. Four different eccentricity values were used for each footing. All footings were carefully instrumented with various configurations of transducers and pick ups for both test control and displacement measurement. Special instrumentation was used to measure the phase angle, δ , between the applied force and the footing displacement. Thus, for each frequency of oscillation, the force amplitude, vertical displacement amplitude, and phase angle between force and displacement were obtained.

ANALYSIS OF EXPERIMENTAL RESULTS

The experimental results indicated changes in characteristic period or resonant frequency with total weight of the foundation system, cross sectional contact area of the foundation with the supporting soil, and eccentricity factor of the vibrator-foundation system. The eccentricity factor, ϵ , is defined as

$$\epsilon = \frac{M_o e}{M} \qquad (2)$$

where M_o is the eccentrically mounted mass, e the eccentricity, and M the total mass of the footing-vibrator system.

Figure 1 is a plot of the resonant frequency f in cps (1/T) versus the cross sectional contact area A in sq inches for various constant values of the eccentricity factor. The total weight, W, of the structural system was 30,970 lbs for all four footings and includes the weight of the footing, vibrator, and ballast. Footing diameters were 5 ft-2 in, 7 ft-4 in, 9ft-2 in, and 10 ft-4 in. As indicated in Fig. 1, resonant frequency increases with increased contact area but the rate of change with respect to area decreases with increasing area. In addition, resonant frequency decreases with an increase in the eccentricity factor. Thus, as indicated by Eq. (1), the resonant period decreases with an increase in contact area but increases as the eccentricity factor or excitation level increases. This is consistent with the known nonlinear restoration characteristics, underlinear or soft spring type relationship, of cohesive soils and qualitatively can be interpreted in analogy with a simple spring-mass system. Expressing the natural frequency, p, of a simple linear spring-mass system as

$$p = \sqrt{\frac{k}{M}} = \sqrt{\frac{kg}{W}} \qquad (3)$$

gives a natural period of

$$T_n = 2\pi\sqrt{\frac{M}{k}} = 2\pi\sqrt{\frac{W}{kg}} \tag{4}$$

For the constant static loading of footings on cohesive soil, the larger deflection occurs for the smaller diameter footing; hence, a larger secant modulus (point spring constant k) is obtained for the larger footing. In addition, the underlinear restoration characteristic gives a decrease in k for an increase in force level. Thus, an increase in diameter or contact area for a constant force level gives an increase in k and, hence, a decrease in period while the secant modulus k decreases with increasing excitation or forcing level, giving an increase in period.

Considering the resonant frequency-contact area relation to be parabolic, for simplicity, leads to the representation of the results of Fig. 1 as given in Fig. 2 which is a plot of log (1/T) versus log A. Use of straight line representations in Fig. 2 gives a relation of the form

$$T^{-1} = a\,A^{b} \tag{5}$$

where the intercept, a, is a function of the eccentricity factor and the weight W of the system while b equal 0.33 is the slope of the straight line approximations.

Assuming that the resonant frequency varies as $(A^{0.33})$, the influence of the weight of the system on the resonant period can be studied by dividing T^{-1} by $(A^{0.33})$ and considering the result as a function of W for a constant value of the eccentricity factor. Since the natural frequency of a simple linear spring-mass system varies inversely with the square root of the weight of the system as indicated in Eq. (3), it might be reasonable to represent the variation of f with W in the form of a simple power relationship. Figure 3 is a plot of log $(T^{-1}/A^{0.33})$ versus log W for a constant value of the eccentricity factor. Approximation of the response of Fig. 3 by the straight line given, leads to a relation of the form

$$\frac{T^{-1}}{A^{0.33}} = C\,W^{-h} \tag{6}$$

where C = 382 and h = 0.58.

Considering the influence of the weight of the system as $W^{-0.58}$, the effect of the eccentricity factor can be studied by plotting $(T^{-1}W^{0.58}/382\,A^{0.33})$ as a function of ϵ . Such a plot is presented in Fig. 4 in semi-log form. Approximation of the response by the straight line representation gives

$$\frac{T^{-1}\,W^{0.58}}{382\,A^{0.33}} = 1.25\,\exp\,(-19.1\,\epsilon) \tag{7}$$

where 1.25 and -19.1 are the intercept and slope, respectively. Rearrangement of Eq. (7) gives

$$T^{-1} = f = \left[\frac{478\ A^{0.33}}{W^{0.58}}\right] \exp\ (-19.1\epsilon) \qquad (8)$$

Eq. (8) represents the resonant frequency response as the product of three functions; area, weight and eccentricity factor functions. Because of the empirical nature of its development, Eq. (8) is not dimensionally homogeneous unless one considers the numerical coefficients to have dimensions. The units utilized in Eq. (8) are area in sq inches, weight in pounds, and eccentricity factor in inches. The degree of applicability of Eq. (8) to represent the results of the extensive prototype test program is indicated in Fig. 5. The measured values of the resonant frequency (T_m^{-1}), as determined from the peak points of the displacement-frequency response curves, are plotted versus the resonant frequency values (T_C^{-1}) calculated from Eq. (8). The straight line of Fig. 5 represents a perfect correlation factor of one and allows comparison of Eq. (8) and the measured response. Except for several cases, the response given by Eq. (8) is within 10 per cent of the measured response with no apparent phenomenological order due to area, weight, or eccentricity.

It must be emphasized that Eq. (8) represents the resonant frequencies or periods of the soil-foundation systems as a function of the particular type of sinusoidal loading used, and does not give the natural periods or frequencies of the systems. Because of the nonlinear nature of soil-foundation systems, such loading level effects are to be expected. However, letting the eccentricity factor, ϵ , take on the limiting value of zero in Eq. (8), gives

$$f = \frac{478\ A^{0.33}}{W^{0.58}} \qquad (9)$$

and use of Eq. (1) gives

$$T = (2.09 \times 10^{-3})\ \frac{W^{0.58}}{A^{0.33}} \qquad (10)$$

Thus, as a first order approximation, Eqs. (9) and (10) might be considered the natural frequency and natural period, respectively, of the cohesive soil-foundation systems under consideration.

The total weights considered above varied from 12,820 lbs to 51,280 lbs and the contact areas ranged from 20.97 sq ft to 83.89 sq ft. Although such variations may be characteristic of many practical situations, it is interesting to consider the characteristic frequency and period of a considerably larger foundation system supported on the same cohesive soil. Consider a 500 ton system supported on a 50 ft diameter circular footing. Eqs. (9) and (10) give a frequency of approximately 10 cps and a period of approximately 0.10 sec, respectively. A period of 0.1 sec is the order of magnitude expected for such a soil-structure system. Thus, Eq. (10) gives a range of values consistent with reported results obtained for various soil-footing configurations and may be useful in shedding insight into other studies of soil-foundation systems.

It must be emphasized that the present analysis only considers circular shapes, weights and areas over a limited range, a particular excitation method, and a single soil. It should be expected that soil type, shape of footing, and magnitude as well as type of excitation will have some effect on the characteristic periods of soil-foundation systems.

CONCLUSIONS

It has been shown that the characteristic periods of prototype circular footings supported on the surface of a cohesive soil are functions of the weight of the system, size of the footing, and level of loading. Footing diameters range from 5 ft-2 in to 10 ft-4 in, total weights from 6.41 tons to 25.64 tons, static pressure levels from 2.56 psi to 10.25 psi, and applied force levels from 525 lbs to 52,000 lbs. The characteristic time parameters used include resonant frequencies and periods as well as zero force level frequencies and periods. Characteristic periods vary as a power of the weight of the system and inversely as a power of the contact area between the footing and the supporting soil. Dynamic force level effects indicate a definite nonlinear (underlinear) nature for the cohesive soil-foundation systems analyzed.

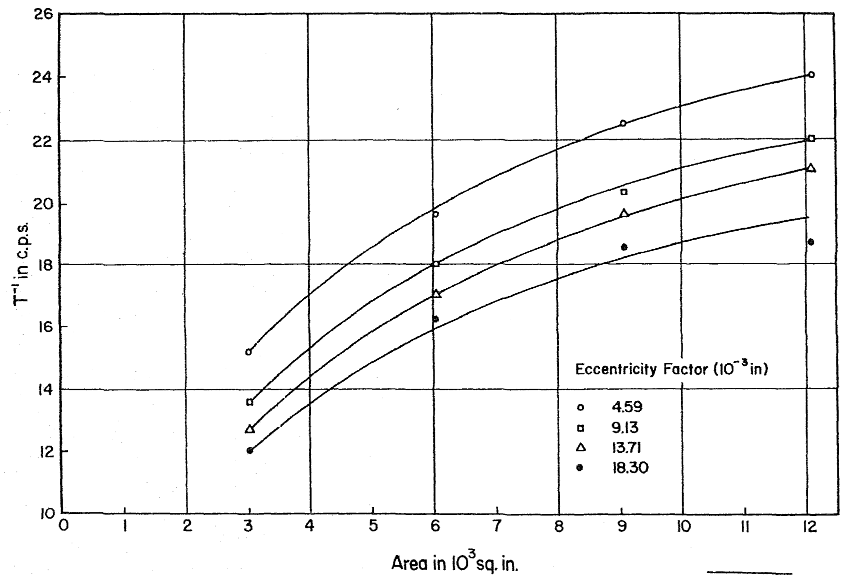

Figure 1. Resonant Period versus Footing Area

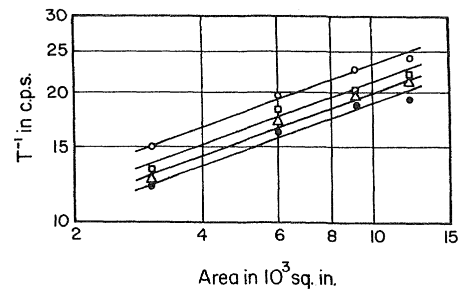

Figure 2. Resonant Period versus Footing Area

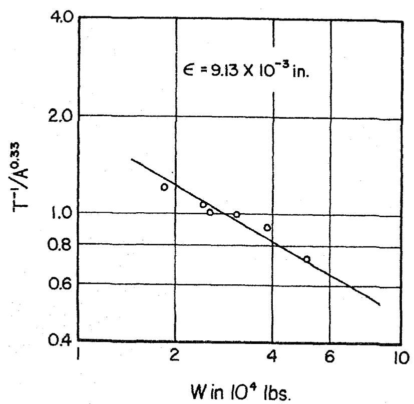

Figure 3. Period-Area Parameter versus Weight

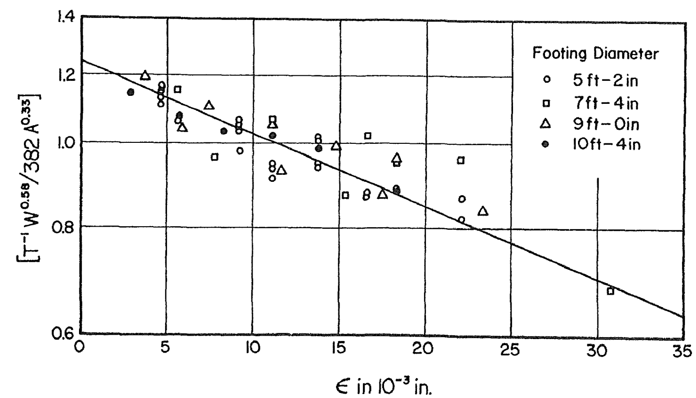

Figure 4. Period Parameter versus Excitation Level

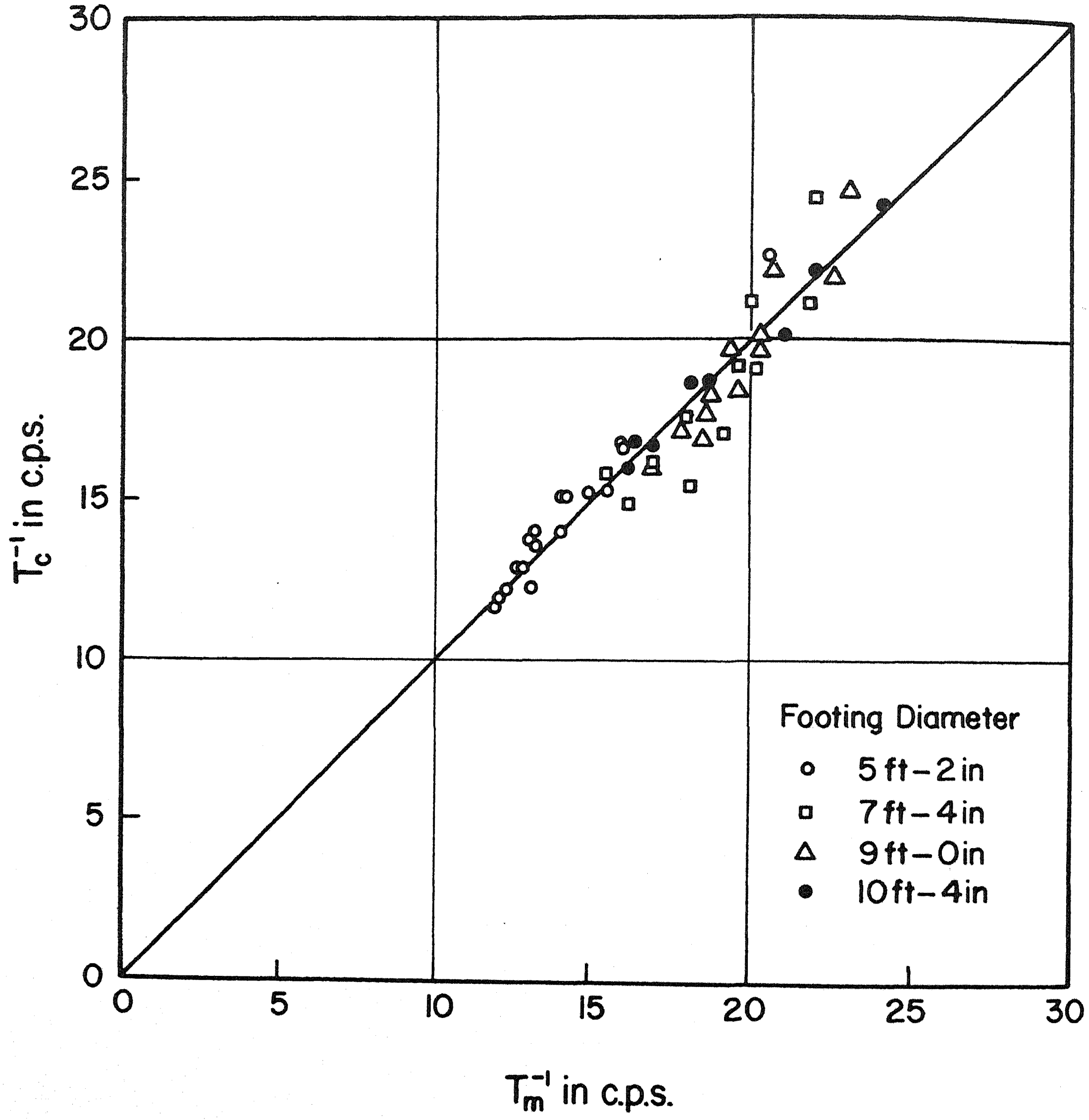

Figure 5. Correlation of Calculated and Measured Periods

EARTH DAMS SUBJECTED TO EARTHQUAKES

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SYNOPSIS

This paper presents, factors affecting the stability of an earth dam under earthquake forces with particular reference to Ramganga dam, 122 m high. Field blasting tests to determine the dominant period of ground and other necessary data alongwith the results of model tests on a large shock vibration table have been described. The model tests offer a good scope for the qualitative study of the problem of stability under earthquake loads. It has been shown that the variation of acceleration with the height of the dam is similar to that predicted theoretically.

INTRODUCTION

The importance of earthquake loads on earth dams has enhanced recently when a number of high earth dams are being planned in seismically active zones in different parts of the World. In India, Ramganga dam about 122 m high, and Beas dam about 100 m high, belong to this category. All of them are either close to active faults or so situated that they are likely to experience a shock of moderate or severe intensity in their life time. The problems connected with behaviour and design of earth dams against earthquake forces are many and no clear cut and quantitative answer to almost any of them is available. Efforts have been directed towards theoretical analysis of idealized cross sections (1, 2) based on theory of elasticity, and model analysis and model testing (3, 4). Because of the very nature of the earthquakes, no field study is apparently possible under actual loading conditions. Further, because of huge mass of earth dam involved, no field testing is economically feasible and even if vibrations are caused by blasting in the vicinity or by placing an eccentric vibrator on the top of dam, it will be difficult to determine the portion of the dam actually participating in the vibrations that we record on the instruments. Thus theoretical analysis, which must be based on many assumptions, difficult to realise in practice, lack the backing of confirmation by field data. Although model studies suffer from certain disadvantages, models offer a useful tool in studying the effect of certain variables.

In this paper, factors affecting the stability of an earth dam under earthquakes have been discussed with particular reference to Ramganga dam. Also results of field observations and model study of certain typical sections on a large shock Vibration Table 5.2 m x 2.8 m have been presented.

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FACTORS AFFECTING STABILITY

An earth dam may fail in any one of the following ways when subjected to an earthquake shock (2) :-

i) Compaction of dam and/or foundations and consequently slumping and possible over-topping by waves.

ii) Sliding of the dam on its base when the overall earthquake acceleration exceeds the coefficient of friction between the dam and the foundations.

iii) Separation of the dam from its abutments due to unequal rigidity of the dam and the abutments and consequent out of phase vibrations (Similar would be the situation when a concrete or masonry spillway section joins the earth structure) or shearing of the dam across its section if a fault crossed it (which should be rare if proper geological investigations have been carried out before selection of a site).

iv) Slipping of the slopes resulting in longitudinal cracks near the top due to the horizontal and vertical inertia forces.

v) Liquefaction and failure of foundations.

Excessive settlement of the dam and foundations and over-topping by waves may be prevented by increasing the density of section and foundation materials and providing additional free board, while shear keys at the base will check sliding on the base. Further, a dam would not normally be constructed right across a fault. The joint of the dam with abutments needs be strong, and it would perhaps be advantageous if the dam section is altered near the junction in order to adjust its stiffness and thereby reduce the out-of-phase shears. Slipping of the slopes may result from various reasons discussed below. Liquefaction and failure of foundations are beyond the scope of this paper.

Slope failure : In order to ascertain if slope failure is likely to take place in an earth dam, the size of shock expected in the region is assumed and the dominant period of vibration determined.

For analysis of the stability of the sections, the following factors need be considered :-

i) Strength characteristics of soils under dynamic loads.

ii) Pore pressures set up during dynamic loading.

iii) Acceleration pattern within the dam.

The strength of soils usually increases with increase in rate of loading. Quantitative values have also been assigned to the increase in strength under purely dynamic loading, but in an actual dam, the load is static as well as dynamic, the latter being a fraction of the former. Therefore the actual increase in strength under the combined loads remains undetermined. The information simulating actual loading conditions is scanty. Arbitrary increases of strength have been adopted in practical design work.

Pore pressures present one of the most intricate problems in stability analysis under dynamic loads. The major difficulties are the ascertaining of its magnitude and its rate of dissipation. Some information is available on magnitudes of pore pressures developed in non-cohesive dense sands under fast rates of loading (5, 6). However, the behaviour of earth dam models of dense sand with respect to their settlement do not conform to that predicted on the basis of laboratory tests (4). Patel (7) has suggested a procedure to compute pore pressures in earthen embankments due to hydrodynamic pressures based upon his tests in an electrical analogy tank. This gives instantaneous values of pore pressures on the upstream slope of the dam for the full reservoir condition only. The strength of the section will thus be affected by the nature of pore pressures induced due to shock loading. This leaves many other problems unsolved for example earthquake forces will affect the pore pressures inside the dam irrespective of the effect of hydrodynamic pressures. Further, the pore pressures get dissipated with time either during the period of shock itself or after it. If the dissipation of pore pressure leads to greater stability its effect could be neglected, but if it leads to instability as it would be in dilatant soils, its effect has to be kept in view. However, if the pore pressure dissipation occurs after the shock, its effect may be important in determining the strength in a subsequent shock if it occurs before the conditions are normalised.

Most of the codes of practice provide for a uniform horizontal force in the design of earth dams against earthquake forces. It has been shown (2) that for the same total force on the dam height, the intensity of force allowed in the top quarter can be 62.5 percent higher than the corresponding uniform force.

In addition to these factors, damping of the soil and position of the core also affect dynamic stability of an earth dam. If the soil offers large damping, the amplitude of deformation will not build up to a high value. Although no account is taken of deformations in a stability analysis, it is well known that failure will occur only if a certain amount of deformation has taken place.

Failure of a section may take place as in Figure 1 a and b. This represents failure of the down stream slope. In figure 1 a, a portion of the downstream slope has moved towards the toe. Small movements of this nature are not harmful. The resulting section shown dotted is stronger than the initial section since the mass of soil has been transferred to a more stable position, and the section can be repaired.

In figure 1 b, the failure of slope is of catastrophic nature especially if the water waves are high enough to overtop the dam.

During draw-down, the loading conditions are severe for the upstream slope the failure of which is more likely if an earthquake occures in such condition. But the failure will be inconsequential except for expenditure involved in repairs as long as water in the reservoir does not over top the dam. It may probably be worthwhile spending the money in repairs to a section later rather than invest it initially in making the section strong. Thus excepting for the damage shown in Figure 1 b, the other two types could be considered as permissible damage. It is easy to show that such permissible deformations would absorb tremendous amount of energy and no great harm to such structures may occur. The only case is which a high factor of safety in design is called for is illustrated by Figure 1 b. In this case higher freeboard than is normally recommended would be essential.

A section with sloping core is more stable than the one with central core. In the latter, the upstream shell, the core and the downstream shell behave as three units of different rigidity and have different periods of vibration. Hence these have a tendency to vibrate out of phase and separate out. A section with the sloping core has the core resting on the downstream shell and the upstream shell in turn resting on the core. This leads to a superior bond of the three units and the section behaves as one mass, and leads to greater stability.

RAMGANGA DAM

This dam is planned in a region affected seismically due to the faulting and folding of the Himalayan and other subsidiary mountain ranges. There are several faults near the site and some others at some distance away (8). In the last 130 years or so, no earthquake of substantial size has originated from the faults in the immediate neighbourhood of the proposed site, but the faults lying further away have been active. Recent geological investigations and occurrence of earthquakes seem to indicate that there are perhaps some faults deep down the alluvium. The Earthquake Research School, Roorkee, placed a Sprengnether seismograph near the site of the dam on firm rock for some time. It did not record any tremors originating from any of the closeby faults, but it did record some micro tremors originating from a fault at a distance of 50 miles and another from a fault about 120 miles away. Geological evidence and the site observations indicate that the nearby faults are dormant and the nearest active fault is about 50 miles from the site which may cause future shocks. A designer then needs to estimate the size of the earthquake which may be expected from these faults. Expressing in terms of Richter's magnitude, the biggest earthquakes that have occurred, are 6.7 and 7.5 with epicenters 90 miles and 130 miles away, respectively. The past records indicate that the maximum intensity felt near the site is on the lower side of MM VIII. Such an earthquake does not cause much harm to an earth dam which is a very stiff structure. Further due to plasticity of the material, it absorbs energy without appreciable damage.

Based on these assumptions and using magnitude-distance-acceleration curves (9) and the Standard Spectrum Curves the design coefficient works out to 15 % g. This could be applied to the mass in the top region and reduced towards the base according to the indications of the model studies.

EXPERIMENTAL WORK

Field Study : Dominant period of ground is determined to ascertain if the natural period of the dam is not close to it. Also, this information is required for model tests in which the periods have to be reduced in a certain ratio to simulate field loading (3). To determine the dominant period of ground, blasting tests were carried out with special gelatin 80 percent. The amount of charge used was 10, 20, 50 and 100 lbs.

In all twelve explosions were fired, the details of which are as follows;-

TABLE 1

Distance from observation site ft.	No. of blasts	Quantity of Explosive lbs.
500	2	10
500	1	20
500	1	50
1050	2	20
1050	2	50
3000	2	100
3000	2	200

For determining the accelerations, particle velocity and displacements caused by the waves, acceleration, velocity and displacement were recorded, along the axis of the dam and perpendicular to it. Velocity and displacement pick-ups were arranged to measure the velocity and displacement of the rock particles in a direction perpendicular to the dam axis.

Typical records of acceleration, velocity and displacement gave values of P and S wave velocities, dominant period of vibration and the elastic constants.

Model Tests : Model studies of Ramganga dam and a sand section were carried out on a shock vibration table having a free movement in one horizontal direction. The following observations were made :

i) Section of Ramganga dam. A model of Ramganga dam was constructed from the materials obtained from the site. The linear scale ratio was 1/200. The blows given had the intensity from 0.2 g to 0.5 g. The cracks developed on both the surfaces of the slopes, the depth of which, was 1.5 in. to 3 in. parallel to the axis of the dam. This pattern was similar to the cracks reported about ONO dam in Japan (10) during 1923 earthquake. For the ultimate load situation, the behaviour of this model represented that of an actual dam qualitatively.

Another observation was that the core and shells on its two sides did not vibrate as a homogeneous mass. By visual observation these could be seen to be vibrating out of phase quite often resulting in a crack along the joint. The depth of this crack was one fourth to one-third the height of the model. This confirmed the observations made earlier about the desirability of the sloping core.

ii) A Section of sand with steep slopes as shown in Figure 2 was next tested. The slope corresponds to the angle of repose. The first shock with 0.2 g caused only a few cracks on both the slopes, where as a second shock of similar intensity caused slumping of slopes as shown dotted in the figure.

iii) Another section of sand with 30° slopes was tested with some acceleration pick-ups embedded close to the top in one test and close to the bottom in another, Figure 3,4. From the records, the acceleration pattern with height of the dam was plotted as in Figure 5. Theoretical values calculated in (2) have also been plotted. Also values reported by Seed and Clough (4) have been included in this plot. The experimental values are very close to the theoretical ones.

Further work on the stability of embankments and strength of soils is currently in hand at the School.

CONCLUSIONS

One of the significant conclusion which can be drawn from these preliminary tests is that the technique of model testing offers a good scope for the qualitative study of stability problem under earthquake loads. The pattern of acceleration with the height of dam in these tests is very similar to the theoretically predicted one.

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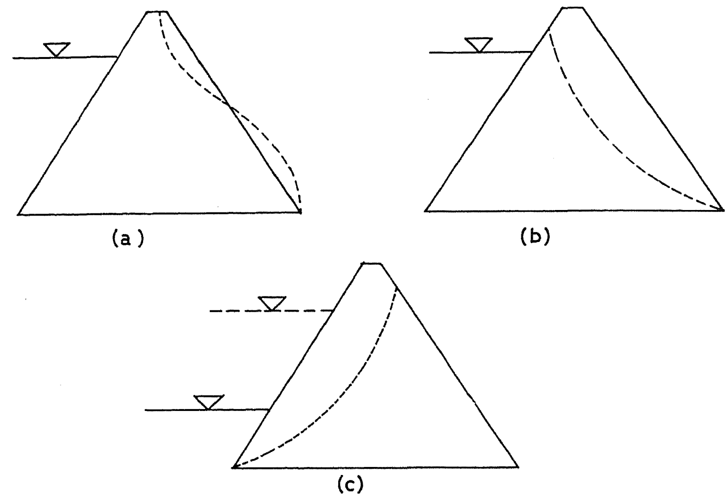

FIG.1 FAILURE OF SECTION

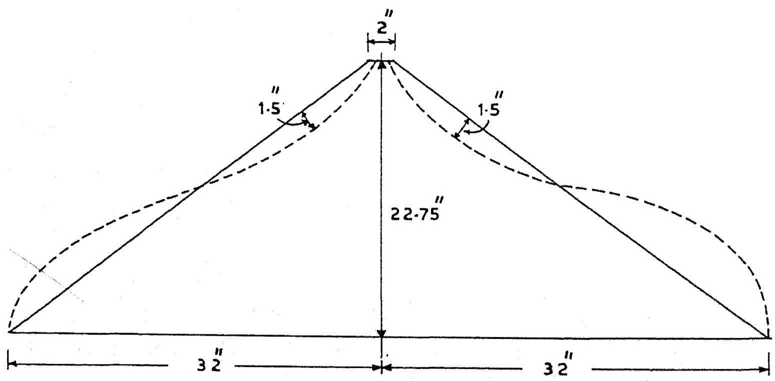

FIG. 2 TYPICAL SECTION (III)

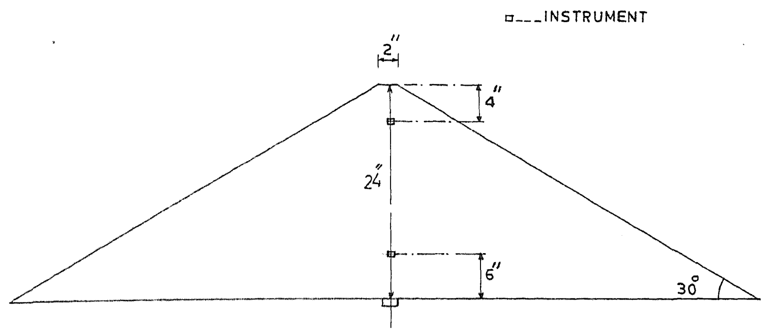

FIG. 3 TYPICAL SECTION (II)

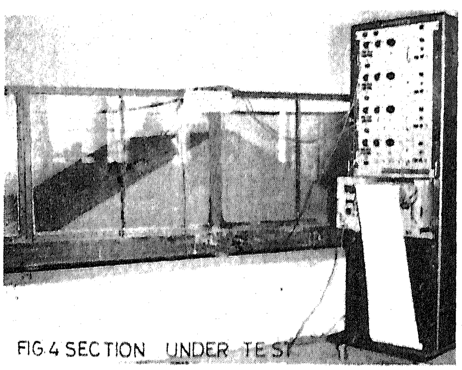

FIG.4 SECTION UNDER TEST

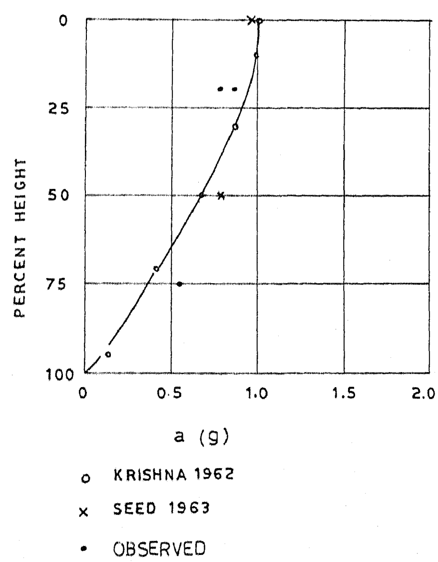

FIG. 5 ACCELERATION PATTERN