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SYNOPSIS Forced vibration test result are presented for two large embankment dams. The results are compared with an approximately three dimensional finite element analysis. In addition a semi-empirical fundamental period expression is developed for use in the interpretation of the available natural period data. The period expression developed is based on the shear wedge solution for an inhomogeneous medium and takes the effect of the third dimension into account.

INTRODUCTION

A 62 m high earthfill and a 165 m high rockfill dam were tested by large vibration generators mounted on the crest. Natural frequencies and mode shapes for motion perpendicular to dam axis were observed. Shear strain amplitudes induced in the dam during testing were estimated to be in the order of 10^{-4} to 10^{-5} percent and thus the properties determined pertain to the small-strain (linear) range of the dynamic stress-strain relation of embankment materials. The results of these tests were than combined with the available data from similar studies by others with the following objectives :

1. To develop an empirical expression for the small-strain fundamental period of embankment dams for a quick and reliable estimate of the period in preliminary earthquake response analyses, and

2. To derive basic dynamic properties of embankment materials for use in selecting appropriate parameter values in more rigorous dynamic response analyses by shear wedge or finite element method.

TESTS ON TWO DAMS

The equipment used in the tests consists of a vibration generator system (Kinemetrics VG-1), accelerometers, seismometers and recorders. Each one of the two vibration generators used has a couple of rotating masses producing a horizontal sinusoidal force. The rotating mass units were installed at the middle part of the crest adjusted to exert the force perpendicular to the dam axis. Frequency of the dynamic force applied was varied between 0 and 10 cps with small increments. The amplitudes of the forced vibration recorded at various points along the crest and on the downstream face of the dam were plotted versus frequency to determine natural frequencies and mode shapes.

The first dam tested (Fig.1), Çubuk II, is a 62m high zoned earthfill dam with a wide, compacted clay core and sandy gravel (GW) shells of alluvial origin. The length of the crest is 230 m. The second dam tested, Keban, is a rockfill dam with compacted clay core and filter zones (Fig 2). The height of the embankment above its lowest point is about 200 m, but due to the peculiar geometry the effective height of the embankment was taken as 165 m.

The fundamental periods of the Çubuk II and Keban dams, measured by the forced vibration procedure described, are 0.44 and 0.61 sec, respectively. The damping ratios for the fundamental modes were estimated from the observed resonance curves to be about 3 to 4% for both dams. It may be noted again that these characteristics pertain to small-strain conditions. More complete information about the dams, test equipment, procedure, and test results may be found in Erdik et al. (1980).

Fig 3 shows the first three lateral (US/DS) mode shapes plotted on the plan of the embankment. The scale of displacement amplitudes is exaggerated as compared to the scale of the plan. In this figure dots represent the measurements and the continuous line shows the calculated mode shapes by an (approximately three dimensional) finite element method (Öner, 1980b). It is observed that the agreement is excellent in the fundamental mode, and poorer in the higher modes which is partly due to the high-frequency noise in the environment.

FUNDAMENTAL PERIOD

Purpose and Data Base

It is considered desirable to estimate the small-strain fundamental period of an embankment dam for use in preliminary and simplified dynamic analyses for earthquake resistant design. In such an analysis the effect of nonlinearity may be considered in a relatively simple manner (Makdisi and Seed, 1978). An empirical period expression that takes global characteristics such as the embankment material type and

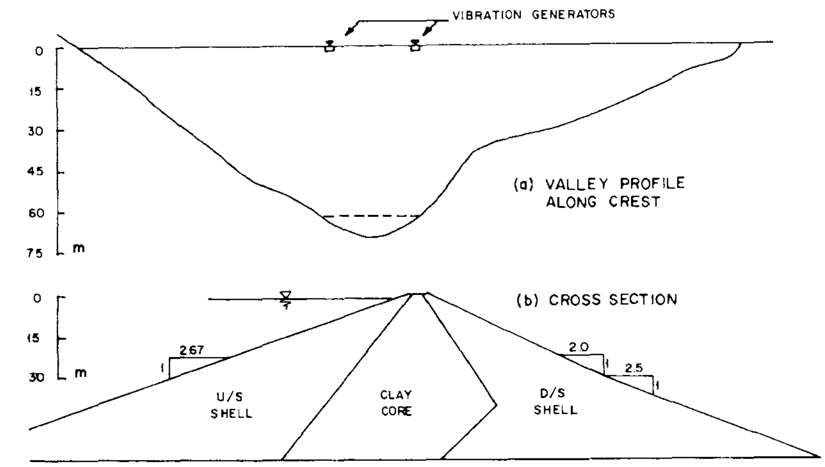

Fig 1. Valley profile and cross section of Çubuk II Dam

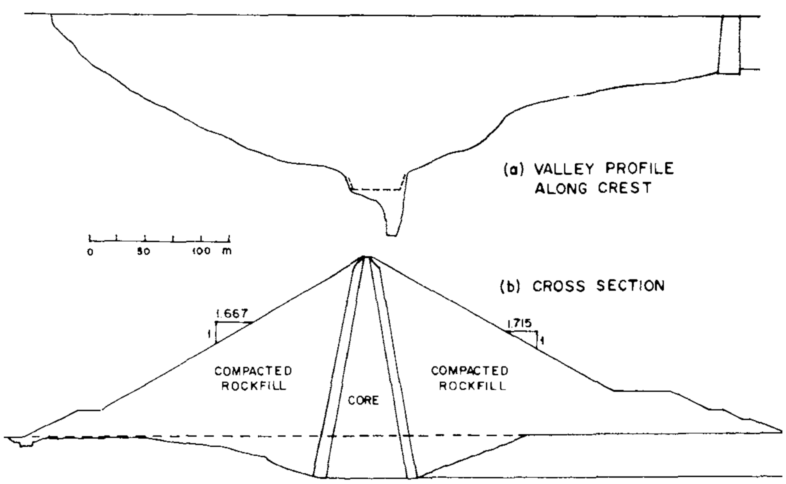

Fig 2. Valley profile and cross section of Keban Dam

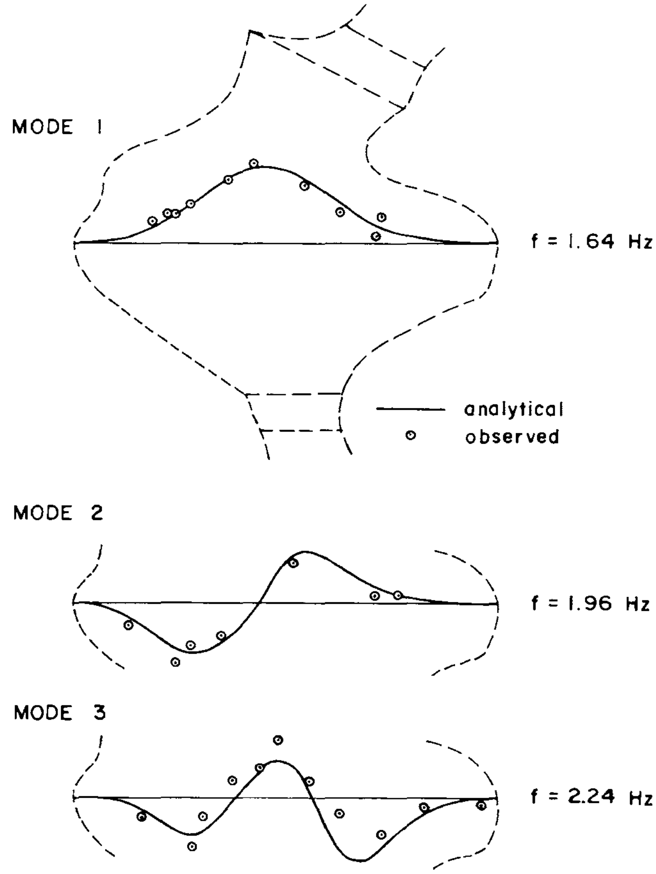

Fig 3. Comparison of predicted and measured mode shapes

TABLE 1. Characteristics of the Embankment Dams Considered

Dam Name	Type (1)	Height (m)	Crest Length (m)	Average Slope (2)	L/H (4)	Fundamental Period, T(sec)	Reference
Bouquet Canyon	E	63	363	3.00	5.74	0.45	Keightley (1964)
Brea	E	26.5	538	3.58	20.3	0.37	A.-Gaffar and Scott (1980)
Carbon Canyon	EH	30(31) (3)	587	3.25	13.0	0.64	A.-Gaffar and Scott (1980)
Çubuk II	E	62	230	3.00	3.20	0.44	This Study
Dry Canyon	EH	18(23)	148	2.53	8.09	0.52	Keightley (1964)
Keban	R	165	610	1.69	3.22	0.61	This Study
Kisenyama	R	95	225	2.48	2.68	0.37	Okamoto (1973)
Kuzuryu	R	128	355	2.25	2.77	0.37	Okamoto (1973)
Miboro	R	131	405	2.13	3.09	0.42	Okamoto (1973)
Rema	R	90	230	1.35	2.00	0.23	Paskalov et al. (1980)
Rifle Gap	E	37(31)	457	-	12.5	0.32	Heller and Ahlberg (1973)
Sannokai	E	37	145	2.75	3.92	0.35	Okamoto (1973)
Santa Felicia	E	61(23)	389	2.81	5.88	0.61	A.-Gaffar and Scott (1979)
Tajik	EH	30	129	4.00	4.30	0.50	Atrakova et al. (1973)
Tarumizu	R	39	137	3.05	2.00	0.26	Yanagisawa et al. (1980)
Yanase	R	115	202	2.30	1.76	0.23	Okamoto (1973)
Yugo.	R	100	191	2.00	1.91	0.39	IZIIS (1978)

(1) E: Zoned earthfill, EH: Homogeneous earthfill, R: Rockfill.
(2) Average of the upstream and downstream slopes.
(3) Alluvium thickness, if not excavated.
(4) See text for "effective" L/H ratio.
(5) Average of the two values reported.

geometry as governing factors would be appropriate for this purpose.

Table 1 shows the characteristics of 17 embankment dams, including the two presented herein, for which natural period data are available. One may find data about a number of other dams in literature than those given in Table 1. However period data obtained purely from analyses were disregarded, besides those lacking completeness of data either on embankment characterisistics or test conditions or both. Also period values from strongly non-linear response during a strong earthquake were excluded.

The Approach

Combining the available analytical solutions with empirical data is believed to be a rational approach for developing an expression for the small-strain fundamental period of embankment dams. In this semi-empirical approach one may make use of the traditional shear wedge solutions (e.g., see Ambraseys, 1967). The most elementary one of these solutions which considers a homogeneous triangular elastic wedge under plane strain conditions, gives the fundamental period as

$$T = 2.61\ H/V_s \tag{1}$$

where V_s is the shear wave velocity of the embankment material. Fundamental period (and mode shape) by this solution agrees fairly well with plane strain finite element solutions (Chopra 1967) implying that the pure shear assumption involved in the shear wedge methods yields acceptable results at least for the fundamental mode. The other two basic assumptions, however, do not seem to be quite realistic:

(1) Plane strain condition may not be found the great majority of real dams with a V or U shaped valley profiles.

(2) Dynamic material properties of real soils are more complex than one which is represented by a single constant V_s, even for small-strain conditions.

It is necessery to modify the basic shear wedge solution for the effect of the third dimension and soil property heterogeneity. An approximate procedure is described below.

Effect of the Third Dimension

It may be shown that the fundamental period of a three dimensional embankment may be expressed by multiplying the one dimensional shear wedge solution (Eq.1) by a function of L/H ratio, f(L/H), where L is the crest length and H is the height.

The most common valley shape is assumed to be close to a parabola which is the basis for defining the L/H ratio herein. For other valley shapes an equivalent L/H value based on an equal-areas criterion, may be used.

The function f(L/H) developed by Öner (1980a)

$$f(L/H) = \left[1 + 4/(L/H)^2\right]^{-1/2} \tag{2}$$

was compared with the results of the studies recently reported by Martinez and Bielak (1980) and by Gazetas (1980). It was found that Eq.(2) is sufficiently accurate for most practical purposes.

Heterogeneity due to Stress Dependence

To improve the basic shear wedge solution further, more realistic models for the dynamic stress-strain relationship of soils (embankment materials) must be incorporated in the analysis. One may employ Hardin-Drnevich (1972) expression for this purpose. With slight modifications to be used with all unit systems this expression for small-strain shear modulus, G, reads,

$$G = AF\sqrt{\sigma_o'\ p_a} \tag{3}$$

where p_a is atmospheric pressure in the same stress unit as G or σ_o, the mean effective stress,

$$\sigma_o' = \frac{1 + 2K}{3}\ \sigma_v' \tag{4}$$

in which K is the coefficient of lateral pressure, and σ_v' is the vertical effective stress. The coefficient AF in the modulus expression is dimensionless, and varies mainly with density, void ratio, and type of soil.

It may be shown that a triangular, plane strain shear wedge with a modulus distribution defined by Eq.(3) has a fundamental natural period given by Öner, 1980b),

$$T = 4.708\ H^{0.75} \Big/ \sqrt{\frac{AF}{\rho}\ (1+2K)\,\gamma' p_a} \tag{5}$$

where γ' denotes the average effective unit weight, and ρ is the mass density of the embankment.

Finally, Equations (2) and (5) may be combined to yield the desired period expression for a three dimensional wedge :

$$T = 4.708\ H^{0.75} \Big/ \sqrt{\frac{AF}{\rho}\left[1+\frac{4}{(L/H)^2}\right]\ (1+2K)\gamma' p_a} \tag{6}$$

DEDUCED DYNAMIC PROPERTIES

By using the semi-empirical period expression developed the modulus coefficient, AF may be backfigured from the data available. When suitable material data are not available it was assumed that $K = 0.4$, $\gamma_{av} = 13\ kN/m^3$, and $\rho = 1.9\ t/m^3$. Table 2 gives the values of AF for each dam deduced in this manner. Corresponding K_{2max} values (as used by Seed and Idriss, 1970) are also listed in the same table for comparison with the available data on varuous soils.

It is observed that the AF parameter is about 500 for homogeneous earth dams, between 1000 and 2000 for zoned earth dams, and 3000 to 6000 for rockfill dams.

TABLE 2. Deduced Dynamic Parameters

Dam name	Type	AF(dim.less)	$K_{2max}(psf^{1/2})$
Buquet Canyon	E	1903	87
Brea	E	808	37
Carbon Canyon	EH	584	27
Çubuk II	E	1557	71
Dry Canyon	EH	234	11
Keban	R	4334	197
Kisenyama	R	3718	169
Kuzuryu	R	5960	271
Miboro	R	5151	234
Rema	R	808	37
Rifle Gap	E	1809	82
Sannokai	E	1250	57
Santa Felicia	E	1278	58
Tajik	EH	467	21
Tarumizu	R	1537	70
Yanase	R	4490	396
Yugo.	R	2688	122

CONCLUSIONS

Forced vibration test data for two large embankment dams have been presented. The results have been evaluated combining them with similar data presented by others in order to develop a semi-empirical fundamental period expression. This expression takes into account the three dimensional effect (valley shape) and the heterogeneous distribution of soil modulus within the embankment.

Dimensionless modulus coefficient, AF (Eq.3), values have been calculated for the dams for which sufficient data are available, using the period formulation developed.

Typical AF values for homogeneous alluvial fill dams were found to be about 500 while the values for modern zoned earthfill dams are in 1000 to 2000 range. Corresponding results for rockfill dams are about 3000 to 6000. The representation of the stiffness characteristics of embankment dams by the dimensionless AF factor is believed to be more useful in practice as AF parameter is independent of the embankment geometry.

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