ON THE FORCED VIBRATION TESTING OF DAMS

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SUMMARY

This paper presents a summary of the results of twenty three forced vibration tests on dams. The vibration testing procedure is described and a mode classification scheme is introduced. The importance of comparing theory and experiment is discussed and the conclusions which have been drawn from such comparisons are presented.

INTRODUCTION

When a dam is subjected to forced vibration tests the following modal characteristics will be obtained for several of the lowest frequency modes of vibration.

1. Resonance (or natural) frequency
2. Damping
3. Mode shape
4. Modal stiffness

This information can be used in two ways

1. To check the mathematical model of the structure
2. As basic input data to predict how the structure will behave in an earthquake

Furthermore, if the results of tests on a large number of dams are available, together with any comparisons with mathematical models, it is possible to

1. Assess how good the mathematical models are at modelling the actual structural behaviour
2. Provide a database of test results for creating empirical models when more complex theory is inadequate (eg damping).

The purpose of this paper is to draw together the results from a number of actual tests and thereby provide some information on the accuracy of modelling dams. Within this paper the various types of dam will be placed into four categories. These catagories which are not totally unrelated are:

1. Arch dams
2. Gravity dams
3. Buttress dams
4. Embankment dams

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FORCED VIBRATION TESTING

There are several ways of testing dams, but they fall into two principal categories:

1. forced vibration tests
2. ambient vibration tests

The ambient vibration tests rely on natural excitation (wind or seismic) to excite the dam and the dam behaviour is monitored. Because the analysis will be performed on short records of unstationary data, large errors are possible and it will be difficult to obtain accurate values of damping and mode shapes. On the other hand, forced vibration tests use vibration generators to provide a forcing function which can be controlled by the engineer and thus provide the opportunity to make accurate measurements of the structural characteristics. Indeed, forced vibration studies have provided the major source for detailed measurements of dam behaviour. Forced vibration studies do have one drawback with regards to earthquake studies, and that is because they are concerned with vibrations of much smaller amplitude than those normally encountered in earthquakes. Consequently, the forced vibration studies must be used to confirm the basic mathematical modelling of a dam, which can then be used in conjunction with non-linear extrapolations to predict how the dam would behave under earthquake loading.

In order to perform forced vibration tests, the dam must be excited using some type of vibration generator and the resultant vibrations must be monitored. The equipment which the Building Research Establishment (BRE) uses to perform the forced vibration tests has been described in detail elsewhere (ref 1). Basically it consists of four individual vibration generators which are controlled from one master control unit and these provide the excitation force which causes the dam to vibrate. The frequency of the whole system can be varied from 0 - 20 Hz in steps as small as 0.001 Hz and the four generators can be phase locked to within 0.01 radians. The vibrations are monitored using servo-accelerometers together with appropriate signal conditioning (amplifiers and filters). Servo-accelerometers are used because they can be calibrated on site, the calibration of any monitoring equipment being of the utmost importance. BRE has performed forced vibration tests on many different types of structure over the past eight years, and although the type of structure and local considerations do influence the testing, the general testing procedure is the same. Usually the objective of the tests is to obtain detailed measurements of the lowest frequency modes of vibration of the structure and this is accomplished in the following manner.

1 The vibration generators are located at positions and in directions where they are likely to produce the largest response (usually at the top of the structure).
2 Two accelerometers are positioned at different places to monitor the structural response.
3 The vibration generators are set to produce a known force, the frequency is incremented and the response is monitored. The plot of normalised displacement against frequency is termed the *spectrum of response* and can be used to determine the resonance frequencies, associated damping values and modal stiffnesses.

4 When a mode has been located the generators are set to the resonance frequency and the mode shape is obtained by moving one accelerometer to various positions on the structure. The other accelerometer is left in one position to provide a reference signal. The mode shape can indicate if there are any modes to be expected at lower frequencies.
5 The vibration generators are moved in either position or direction to attempt to locate further modes and the procedure is repeated.

MODE CLASSIFICATION

In order to classify the modes of vibration of dams and hence simplify interpretation, it is useful to relate each mode to the principal direction in which modal deformations occur. For dams it is convenient to define motion using three orthogonal directions, namely upstream-downstream (U.D.), across valley (A.V.) and vertical (V.). For each direction there is a whole family of modes and it is useful to use a convention for describing modes. Normally a series of specific types of deformation will be encountered along the crest of the dam in the mode direction and these will be mirrored on parallel horizontal sections throughout the dam. If the crest is represented by a straight line and the crest displacement is represented by a displacement of this line (for any one mode), then the shape will be somewhat similar to the modes of vibration of a string which is held at both ends, where the lowest frequency mode will have no nodes, the next lowest frequency mode will have one node etc. For the purposes of definition let the mode with no nodes be termed 1, and the mode with one node be termed 2 etc. Also there will be a series of specific types of deformation encountered throughout the height of the dam. If for example a vertical section through the dam is considered, the series of deflected shapes will be somewhat similar to that of a vertical cantilever, with the lowest frequency mode having no nodes, the next lowest frequency mode having one node etc. Again let these modes be termed 1, 2,... If a mode of vibration is to be defined completely it is necessary to define

1. Direction
2. Crest deflected shape
3. Vertical deflected shape

So for an upstream-downstream mode with 2 nodes along the crest and no nodes down the dam let the terminology be U.D. 3-1 etc. This mode classification is used to define the modes which are included in the table at the end of the report and is useful when a large number of modes are investigated on any one dam.

MATHEMATICAL MODELS

In order to design a dam or predict its response to an earthquake, it is necessary for the engineer to use some form of mathematical model of the structure, and the accuracy of any calculations will depend on the accuracy of this model. Therefore it is desireable to have some means of checking the accuracy of the mathematical models and the theories upon which they are based. The provision of results from full scale tests is therefore an important step towards justifying mathematical models and confirming the various assumptions which have been made. A perfect correlation between theory and practice should not be expected because there are many factors, such as material properties, which cannot be specified exactly. If the mathematical model can predict similar behaviour to the measured behaviour then a real understanding of the dam's behaviour can be achieved. If this is consistently repeated for many dams, confidence in the theoretical methods will accrue. However, if theory and practice do not correspond, discrepancies in present design analyses will be revealed and with them the impetus to discover why the mathematical model (and/or the physical testing) is in error. Although technology has provided the means there has, as yet, been little systematic comparison between design models and actual behaviour, so this potential gain for the engineering profession remains untapped.

In this short paper it is not possible to reproduce all of the results of forced vibration tests on dams, but table 1 provides a summary of the results of tests on 23 dams. In table 1 only the two lowest frequency modes of vibration in any one direction are presented. If the mode shapes are not available then the mode classification is assumed but indicated by a question mark following the mode descriptor. Also one column in the table indicates whether a theoretical comparison is presented in the relevant paper. General conclusions from this work are discussed in the next four sections.

ARCH DAMS

In the design of arch dams it is important to have a good mathematical model of the dam so that the stresses in the dam can be calculated for various load combinations. Consequently arch dams have attracted more recent attention with regards to dynamic testing and modelling than any other type of dam. Fortunately, despite their somewhat complex shape, arch dams are perhaps the easiest type of dam to model accurately. They are usually built on solid foundations, so that soil-structure interaction is negligible, they are built of mass concrete whose material properties can be estimated as well as those of any other type of material used for dam construction, and their shape is well defined providing an ideal case for solution using the finite element method (F.E.M). The comparisons which have been performed show a good correlation between experimental measurements of frequencies and mode shapes and theory (both F.E.M and moment distribution predictions). The prediction of damping values is more difficult, but if the values noted in table 1 are examined, it can be seen that there is some correlation between damping and frequency (for small amplitudes of vibration). Considering the frequency and damping for the fundamental mode, it appears that the frequency in Hertz is approximately equal to the damping expressed as a percentage of critical. Furthermore, if both of these values are plotted against dam height, then similar curves can be fitted to both graphs, thus providing an empirical method of predicting damping and frequency. These are obviously crude approximations but in the absence of further data they may provide useful guides. The damping of higher frequency modes can be considered to be of similar order to the first mode damping, although again considerable scatter should be expected.

One point which has generated much interest is the effect of water-structure interaction. This effect was demonstrated by tests on Kolbrein dam (ref 2) when the dam was tested with the reservoir full and empty. At Kolbrein dam the frequency of the fundamental mode reduced by 15.7% and the damping increased by 14.6% when the reservoir was filled.

GRAVITY DAMS

Gravity dams are usually constructed as a series of monoliths, with contraction joints between adjacent monoliths. These contraction joints are then either grouted or left with a water seal to prevent leakage. This procedure can lead to two types of characteristic features in dams.

1. If the grouting is adequate, then the individual monoliths will be effectively combined, so that the dam will behave essentially as one complete structure.
2. If there is no grouting each monolith will have its own characteristics and there will be varying amounts of interaction between monoliths. This type of behaviour has been noted in at least three dams.

The mathematical modelling of the dams will therefore be determined by the flexibility across the contraction joints. The most common type of behaviour appears to be when the contraction joints are effectively rigid, but soil-structure interaction is common, producing high damping values. The measured values range from 3-12.5% of critical but no obvious empirical relationships emerge for estimating damping values. With regards to modelling gravity dams it has been suggested (ref 10) that a 2-D analysis of planar vibrations of a typical section of the dam is reasonable in situations where the length of the dam is many times the cross sectional dimensions. However, this can only estimate the frequency of the fundamental upsteam-downstream mode. Work on Alpe Gera dam provides the one study where predictions made using a 3-D model have been compared with experiment. After preliminary models it was found that the best correlation between mode shapes was obtained when a very stiff foundation was selected. This produced frequencies which were, on average, 15% higher than the measured frequencies.

When there is no grouting between contraction joints, and each monolith acts quasi-independently, the situation is far more complicated. In fact the dam will behave in a similar fashion to a buttress dam and this will be discussed in the next section.

BUTTRESS DAMS

Buttress dams are perhaps the most difficult type of dam to model accurately. Although the buttresses are designed to act independently, there is a considerable amount of interaction between adjacent buttresses and between buttresses and the ground, and consequently high damping factors are encountered. This interaction is extremely difficult to model and recourse to modelling individual buttresses is probably necessary. Because each buttress has its own characteristics only typical values for one buttress from Wimbleball (buttress 4, March 80) and one monolith from Upper Glendevon (monolith 16, April 82) are presented in the table. At both Wimbleball and Upper Glendevon (a gravity dam with no grouting) measurements were made at various water levels and the water height is found to effect all of the modal characteristics. Theoretical calculations for one buttress at Wimbleball are presented in ref. 1. The calculations were made using a finite element model and assumed no interaction with the soil or adjacent buttresses. A satisfactory agreement was obtained between the experimental and theoretical frequencies, although because of the restrictions on the mathematical model the mode shapes could not be reproduced.

EMBANKMENT DAMS

Embankment dams are by far the most common type of dam throughout the world, yet very few dynamic tests have been reported. One reason for this is that it is not easy to fix any vibration generator to the crest of the dam. Consequently for BRE investigations 2m concrete cubes are cast into the crest of the dam to provide a base for fixing the generators. There are several published papers concerned with the mathematical modelling of embankment dams but the present state-of-the-art is illustrated by the following (ref 11):

'A tridimensional finite element analysis of the stresses in an embankment dam is today theoretically possible, but remains extremely costly for the analysis of dynamic stresses. For this reason and according to current practice, one performs only two-dimensional (2D) plane strain analysis of a typical cross-section of each dam'. It is therefore worthwhile examining what a 2D model could represent. Because there is no scope for across valley motion, no across valley modes can be predicted. Also because different deformations along the crest cannot be represented, only the fundamental modes in the upstream-downstream and vertical directions can be modelled. It should also be noted that a 2D model (usually a slice through the dam at the tallest section) does not represent the true mass, stiffness or correct foundation conditions of the dam, but nevertheless a good correlation between upstream-downstream modes has been noted in several investigations. Whether this is purely fortuitous remains to be seen. Shear wedge models which assume only shearing deformation in the upstream-downstream direction have been used to provide accurate predictions of upstream-downstream modes, but a full 3D model (ref 1) predicts modes in the across valley and vertical directions and these have been noted in both ambient and forced vibrations (ref 7). The 3D model underestimated the frequencies by an average of 20% but provided a good estimate of the mode shapes. Again damping cannot be predicted theoretically and a range of 2-4.8% has been encountered with one exception. This was the fundamental upstream-downstream mode at Lower Glendevon where a damping of 16.3% was encountered. This result was obtained in two seperate tests, with the excitation and monitoring positions being changed between tests, thus confirming the accuracy of the measurement.

FUTURE WORK

With important structures like dams, it is important to be confident that they will behave in a similar manner to that assumed in their design. The comparison of theoretical and experimental studies provides one method of justifying design assumptions, yet only a small amount of this type of work has been completed. The majority of the work has been carried out on arch and gravity dams and this has been conducted principally by an Italian group (ref 4). However, little work on buttress and embankment dams has been published, so the conclusions remain tentative. In order to obtain the most from future work, it is important that the results of investigations are available in as much detail as possible, and that comparisons consider mode shapes as well as frequencies. Also to prove that mathematical models are adequate it is best to perform the calculations before the experimentation so that their mutual independence is assured.

REFERENCES

1. SEVERN R T, JEARY A P and ELLIS B R. "Forced vibration tests and theoretical studies on dams." Proc. Instn. Civ. Engrs., 69, Sept 1980. & Discussion Proc. Instn. Civ. Engrs., 71, June 1981.
2. FLESCH R and EISELMAYER M. "Dynamic in-situ tests on Kolbrein arch dam." Seventh World Conf. on Earthq. Eng. Vol 8, Istanbul, 1980.
3. DEINUM P J et al. "Vibration tests on Emosson arch dam, Switzerland." Earthq. Eng. and Struct. Dynamics, Vol 10, 1982.
4. CALCIATI F et al. "Experience gained during in-situ artificial and dynamic excitation of large concrete dams in Italy." Trans. 13 Int. Congr. Large Dams, Q51, R32, New Delhi, 1979.
5. PASKALOV T A et al. "Full scale vibration studies and mathematical model formulation of arch concrete dams." Proc. Conf on Dams and Earthquakes, Instn. Civ. Engrs., London, Oct 1980.
6. ROUSE G C and BOUWKAMP J. "Vibration studies of Monticello dam." Water Resources Technical Publication No. 9, US Dept. of Interior, 1967.
7. ELLIS B R et al. "Vibration studies of the Glendevon dams." to be published.
8. PASKOLOV T et al. "Comparitive analysis of a rockfill dam." Seventh World Conf. on Earthq. Eng. Vol 8, Istanbul, 1980.
9. KEIGHTLEY W O. "Vibrational characteristics of an earth dam." Bull. Seism. Soc. Amer. Vol 56, No 6, Dec. 1966.
10. CHAKRABATI P and CHOPRA A K. "Earthquake analysis of gravity dams including hydrodynamic interaction." Earthq. Eng. and Struct. Dyn. Vol 2, No. 2, 1973.
11. BALLISATT M and BOSSONEY C. "Dynamic behaviour of three rockfill dams." Trans. 13 Int. Congr. Large Dams, Q51, R18, New Delhi, 1979.

TABLE 1

Dam name Location	Type	Ht. (m)	Crest Lg(m)	Mode	Freq Hz	Damp %	Mode Shape	Theor. Comp.	Ref
Contra	Arch	220	380	UD 1-1	1.94	1.72	Y	Y	-
Switzerland				UD 2-1	1.74	1.76	Y		
Kolnbrein	Arch	200	626	UD 1-1	1.43	1.5	Y	Y	2
Austria				UD 2-1	1.21	1.3	Y		
Emosson	Arch	180	424	UD 2-1	1.85	2.10	Y	Y	3
Switzerland				UD 3-1	2.96	3.51	Y		
Place Moulin	Arch/	155	678	UD 1-1?	2.033	1.15	N	N	4
Italy	Grav.			UD 2-1?	2.032	1.18	N		
Lumiei	Arch	136	210	UD 1-1?	5.65	1.00	N	Y	4
Italy				UD 2-1?	3.99	1.17	N		
-	Arch	123	400	UD 1-1	2.71	1.20	Y	N	5
Yugoslavia				UD 2-1	2.51	1.07	Y		
Monticello	Arch	92	321	UD 1-1	3.13	2.7	Y	N	6
U.S.A.				UD 2-1	3.55	2.2	Y		

Fiastra Italy	Arch/ Grav.	87	254	UD 1-1? UD 2-1?	4.72 5.97	3.27 2.40	N N	N	4
Talvacchia Italy	Arch	78	218	UD 1-1? UD 2-1?	3.80 3.68	3 - 5 3 - 5	N N	N	4
Ambiesta Italy	Arch	59	145	UD 1-1? UD 2-1?	4.70 4.11	1.90 6.00	N N	Y	4
- Yugoslavia	Arch	53	194	UD 1-1 UD 2-1	3.61 3.29	1.90 2.70	Y Y	Y	5
Barcis Italy	Arch	50	71	UD 1-1? UD 2-1?	10.1 7.6	4.0 7.0	N N	Y	4
Alpe Gera Italy	Grav.	174	528	UD 1-1 UD 2-1	3.47 4.72	4.41 4.54	Y Y	Y	4
Suviana Italy	Grav.	97	225	UD 1-1 UD 2-1	6.00 8.66	8.65 7.27	N N	N	4
Campo Moro Italy	Grav.	97	198	UD 1-1? UD 2-1?	6.24 8.73	4.03 3.00	N N	N	4
Morasco/Italy	Grav.	59	564	UD 1-1?	6.49	8.40	N	N	4
Baitings England	Grav.	54	472	UD 1-1 UD 2-1	5.0 6.1	6.5 12.9	Y Y	Y	1
U Glendevon Scotland	Grav.	45	390	Mono UD Mono UD	6.37 8.31	17.7 10.7	Y N	N	7
Wimbleball England	Butt.	63	300	Butt UD Butt UD Butt AV Butt AV	7.87 9.05 6.07 7.65	9.37 3.72 12.25 7.24	Y Y Y Y	Y	1
Rama Yugoslavia	Embank Rock	96	230	UD 1-1 UD 2-1	4.3 6.3	2.55R 2.0R	N N	Y	8
LLyn Brianne Wales	Embank Rock	90	280	UD 1-1 UD 2-1	2.81 3.44	4.82 2.75	Y Y	Y	1
Bouquet Can. U.S.A.	Embank Earth	61	363	UD 1-1 UD 2-1	2.23 2.68	4.1 3.5	Y Y	Y	9
L Glendevon Scotland	Embank Earth	35	240	UD 1-1 UD 2-1 AV 1-1 AV 2-1 V 1-1 V 3-1	3.18 3.51 3.47 4.13 4.19 5.48	16.3 2.8 4.5 4.1 4.0 3.0	Y Y Y Y Y Y	N	7