Paper No. 205, Bull. ISET, Vol. 17, No. 2 June 1980 pp 11-24

A STUDY OF INDIAN STANDARD CODE PROVISIONS FOR EARTHQUAKE RESISTANT DESIGN OF GRAVITY DAMS

S. S. SAINI[1] AND V. V. PATRI[2]

Introduction

The Indian Standard Criteria for Earthquake Resistant Design of Structures, I.S : 1893 specifies the seismic forces to be adopted in the design of gravity dams. The code provisions have been revised from time to time to incorporate a more rational approach for design of dams. Several dams have been designed using these provisions as given in the various revisions of the code. It would be, therefore, useful to the designer to know the quantitative differerence in the various code provisions and their adequacy for dam design

This paper presents a comparative study of the seismic forces worked out as given in the various revisions of I.S. Code (1, 2, 3) and the dynamic analysis (4). A comparison of the dynamic moments and dynamic shears due to horizontal inertia forces has been made for dams of heights varying from 30 m to 200 m. The hydrodynamic forces as provided in the code are based on the assumption that water is incompressible and the dam is treated as rigid. The code also recommends approximate expressions to estimate the hydrodynamic moments and shears. In order to check the approximations involved in the use of these expressions, distribution of hydrodynamic moments and shears has been obtained based on the hydrodynamic pressure distribution given in the code and the percentage error is determined.

Simplified design procedures (5, 6) evolved on the basis of dynamic analysis of dams are also now available. These procedures account for the compressibility of water and dam-reservoir interaction. The feasibility of the use of simplified methods of design in actual practice using the presently accepted design criterion has been investigated. This has been done by applying the method for design of four dams with heights varying from 50 m to 150 m.

The design of a dam is considerably dependent on the choice of design seismic coefficient which, in turn, depends on the seismicity of the site and the properties of the structure. It is rather difficult to assess precisely the seismic activity at the site due to lack of seismic data and uncertainty involved in the phenomenon of earthquake occurrence. Hence, there is a tendency amongst field design engineers to estimate the design seismic forces on the higher side. This is generally done without giving any due consideration to the economic aspects. To make a judicious estimate of the design seismic coefficient, it would be useful, from economic considerations, to know the increase in the cost of the dam with an increase in the design seismic coefficient. The influence of the variation of design seismic coefficient on cost of gravity dams has, therefore, been investigated here by studying four dams of heights varying from 50 m to 150 m, using I.S. Code provisions (3, 7).

---

1 Professor of Civil Engineering, University of Roorkee, Roorkee.

2 Trainee Engineer, Water Resources Development Training Centre, University of Roorkee, Roorkee.

It is concluded that the latest code I.S : 1893-1975 specifying seismic forces for concrete and masonry gravity dams, is quite rational and compares well with dynamic analysis. The simplified design procedure which accounts for compressibility of water and dam-reservoir interaction provides a more rational approach but is uneconomical to use unless realistic and reasonable criteria for permissible tensile stressess is evolved. It is also noted that the cost of gravity dams increases rapidly with an increase in the design seismic coefficient. Thus, the design seismic forces should be estimated after detailed seismic investigations and using rational dynamic analysis procedures.

Study of Indian Standard Code Provisions

The Indian Standard Code I.S : 1893 (1, 2, 3) has been revised from time to time in an attempt to provide a more rational approach for design of dams. The essential differences in the seismic provisions for gravity dams in the three versions of the code are summarized below :

1. The seismic zones as demarcated in I.S. Code of 1966 (1) have been revised in subsequent revisions of the code (2, 3) based on additional seismic data and taking into consideration the tectogenesis and geological features.

2. There is a definite change in the provisions, specifying the distribution of horizontal seismic acceleration along the height of dam in three successive code revisions (1, 2, 3). The I.S. Code of 1966 (1) specifies a uniform variation of horizontal acceleration along the height of the dam as shown in Figure 1 (a). Based on the analysis of dynamic behaviour of several

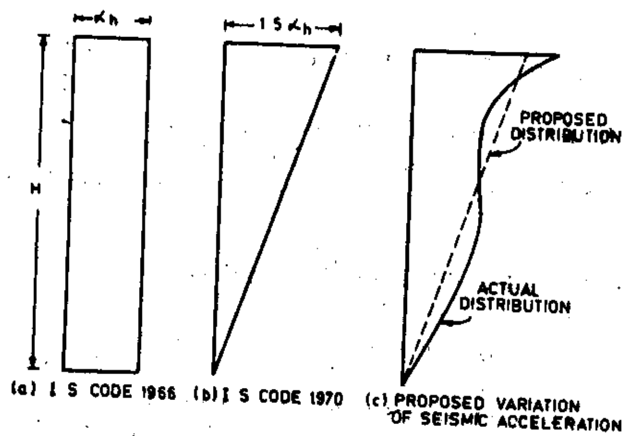


Fig. 1 Variation of horizontal seismic acceleration along the height of dam

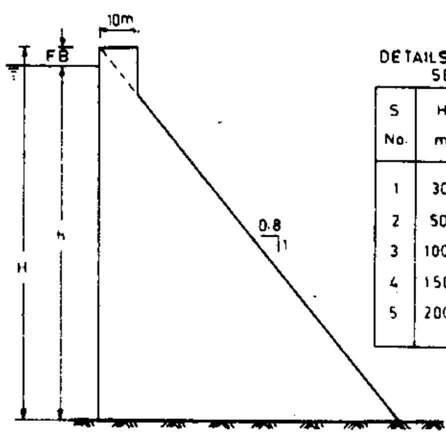
dams (8), it is observed that the distribution of horizontal seismic acceleration along the height of a dam is not uniform but has a general shape as shown in Fig. 1 c) with zero value at base and maximum at the top. Hence, the uniform distribution of seismic acceleration as suggested in the I.S. Code of 1966 (1) has been subsequently modified in the second revision of I S. Code (2). For specifying the seismic acceleration along the height of the dam, the curve has been approximated as varying linearly, from zero at base to maximum at top, the top value being 1.5 times the design horizontal seismic coefficient as shown in Figure 1 (b).

In I.S. Code revision of 1975 (3) which is the latest revision, two methods have been suggested for determination of earthquake forces, namely the Response spectrum method and the seismic coefficient method. In Response spectrum method of analysis, average design curves giving distribution of dynamic moments and dynamic shears along the height of dams have been specified which are based on the dynamic analysis of several dams (4, 11). It is, however, noted that for low dams, the details of the dam near the top contribute greatly to the overall response (4, 10) and hence the average design curves can not be specified in this case. Accordingly, the average design curves have been specified for dams of heights more than 100 m. The idea of choosing a value of 100 m is also that most of the dams upto this height will have periods shorter than 0.30 seconds and the spectral acceleration may not be influenced by change of natural period in this range. In seismic coefficient method, which is to be applied for dams upto 100 m in height, the distribution of horizontal earthquake acceleration is the same as specified in I.S. Code of 1970 (2).

3. The vertical seismic coefficient has been specified as half of the horizontal seismic coefficient. This is reasonable in accordance with the observations that peak ground acceleration in vertical component caused by an earthquake is about one half that of peak horizontal component. The distribution of vertical seismic acceleration along the height of the dam is similar to the horizontal seismic acceleration distribution. It is uniform in the I.S. Code of 1966 (1) and it is specified to have a shape of an inverted triangle with value at top of dam being equal to half the horizontal acceleration in I.S. Code of 1970 (2) and 1975 (3).

Assessment of Horizontal Inertia Forces

The effect of an earthquake on a dam is to induce inertia and hydrodynamic forces. It is of interest to know the effect on distribution and magnitude of inertia forces due to modifications in the nature of distribution of horizontal seismic acceleration along the height of dam. This will give an assessment about the underestimation or overestimation of seismic forces in the design of dams based on provisions in the various code revisions. A comparison of these code provisions (1, 2, 3) with dynamic analysis (4) would be of interest to the designer to assess the adequacy of the latest code provisions.



DETAILS OF CROSS-SECTIONS

S No.	H m	FB m	h m
1	30	2	28
2	50	2	48
3	100	4	96
4	150	5	145
5	200	5	195

Fig. 2 Dam cross-sections selected for comparison of seismic provisions

For the sake of comparison of distribution of dynamic moments and shears along the height of the dam, five non-overflow dam cross-sections have been selected as shown in Figure 2. The cross sections have a vertical upstream face, downstream slope of 0.8 : 1, top width of 10 m and dam heights of 30 m, 50 m, 100 m, 150 m, and 200 m. The dynamic moments and dynamic shears at various elevations along the height of dams have been worked out using the I.S. Code provisions (1, 2, 3) and dynamic analysis (4) and these have been expressed as follows :

$$M = C_m W H \alpha_h \tag{1}$$

$$V = C_v W \alpha_h \tag{2}$$

Where M is the dynamic moment, V the dynamic shear, C_m the dynamic moment coefficient, C_v the dynamic shear coefficient, W the total weight of the dam, H the total height of the dam and α_h the design horizontal seismic coefficient.

The variation of dynamic moment and shear coefficients, C_m and C_v as worked out for the selected dam sections has been shown in Figures 3 to 7. Considering the forces obtained by dynamic analysis as more accurate, it is noted from the figures that the variation of dynamic moment and shear coefficients as per I.S. Code of 1966 (1) is markedly different from those obtained by using later code revisions (2, 3) and dynamic analysis (4). The dynamic moments and shears are overestimated at base using I.S. Code provisions as compared to dynamic analysis whereas these are underestimated in the top portion of the dam. It is noted that I.S. Code provision of 1966 (1) overestimates the dynamic moments at base for all the dam sections under consideration by about 20 to 35 percent, where as dynamic shears are overestimated to the extent of 70 to 90 percent at base, in comparison to those obtained using dynamic analysis. The underestimation of dynamic moments in the upper portion of the dams varies from 30 to 80 percent and for dynamic shear, from 30 to 60

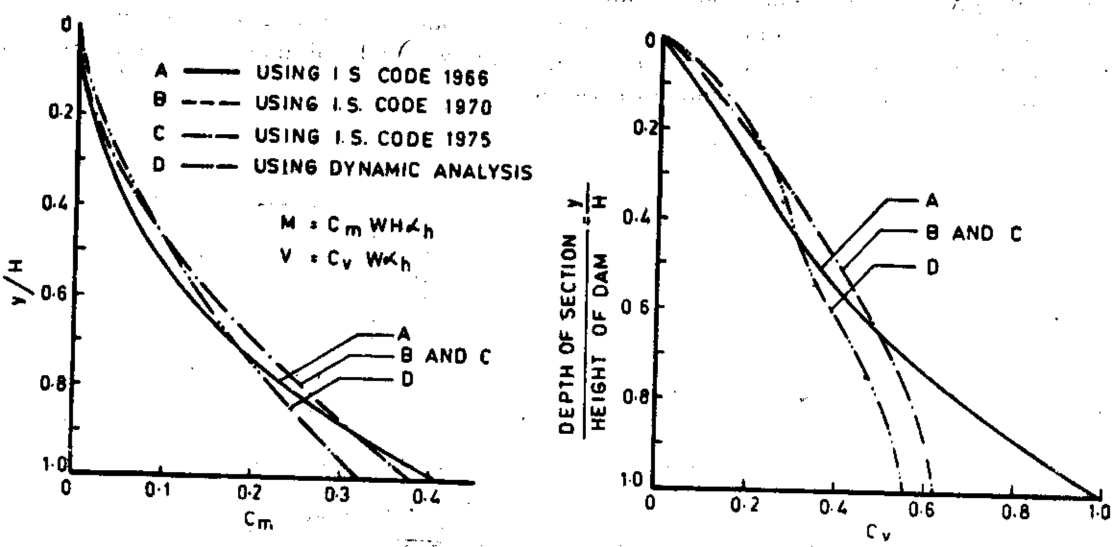


Fig. 3 Comparison of dynamic moments and shears due to horizontal inertia forces for 30 m high dam

percent. Thus, the I.S. Code of 1966 is inadequate to give realistic pattern of distribution of dynamic moments and shears.

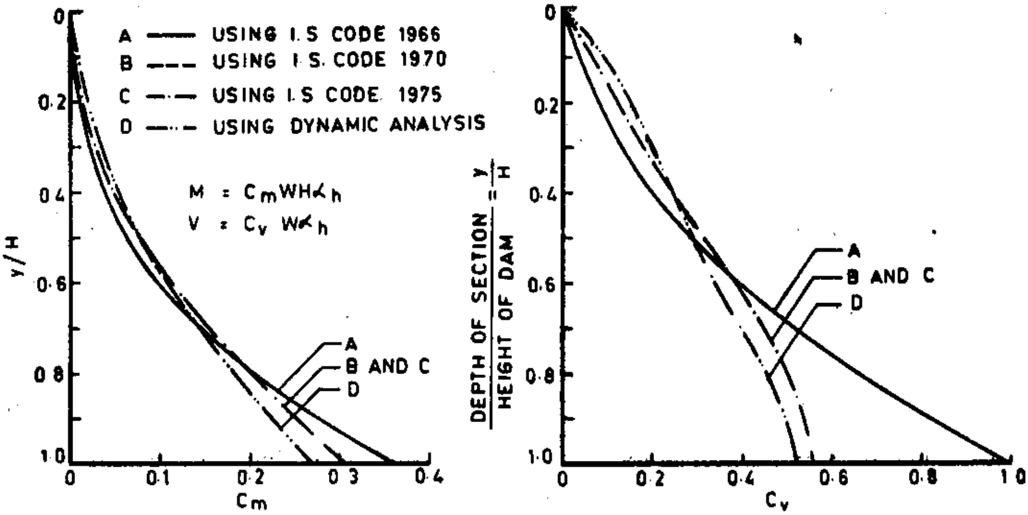


Fig. 4 Comparison of dynamic moments and shears due to horizontal inertia forces for 50 m high dam

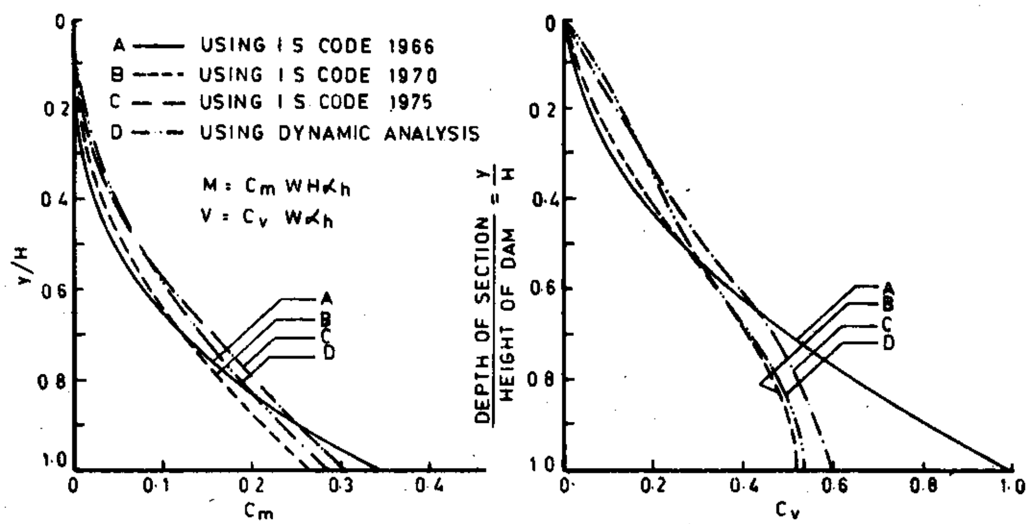


Fig. 5 Comparison of dynamic moments and shears due to horizontal inertia forces for 100 m high dam

The I.S. Code provision of 1970 (2) shows definitely an improvement over the earlier code provision of 1966 (1) but still the distribution of dynamic moments and shears does not follow the variation as obtained by using the dynamic analysis. It is noted that the dynamic moments and shears are overestimated by marginal amounts at the base but the

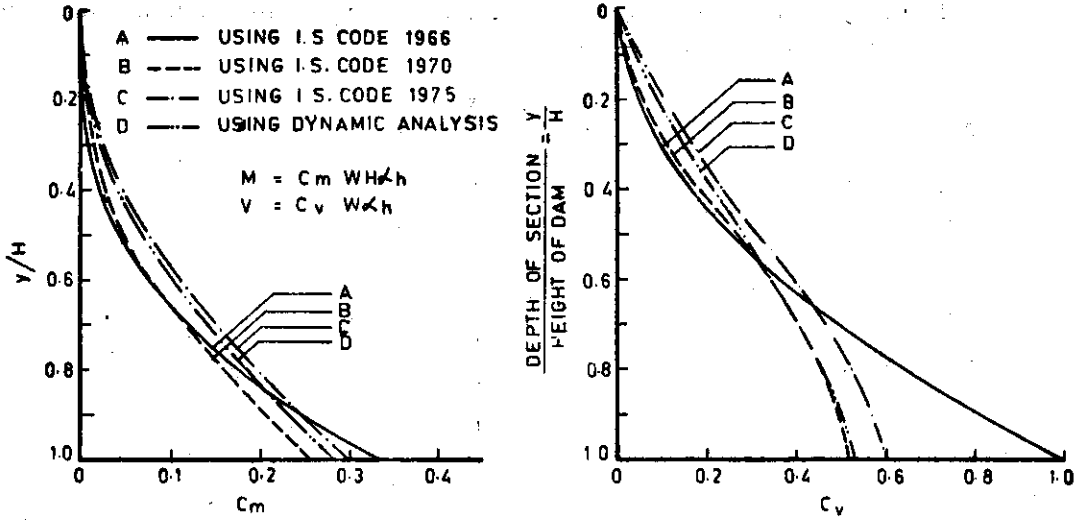


Fig. 6 Comparison of dynamic moments and shears due to horizontal inertia forces for 150 m high dam

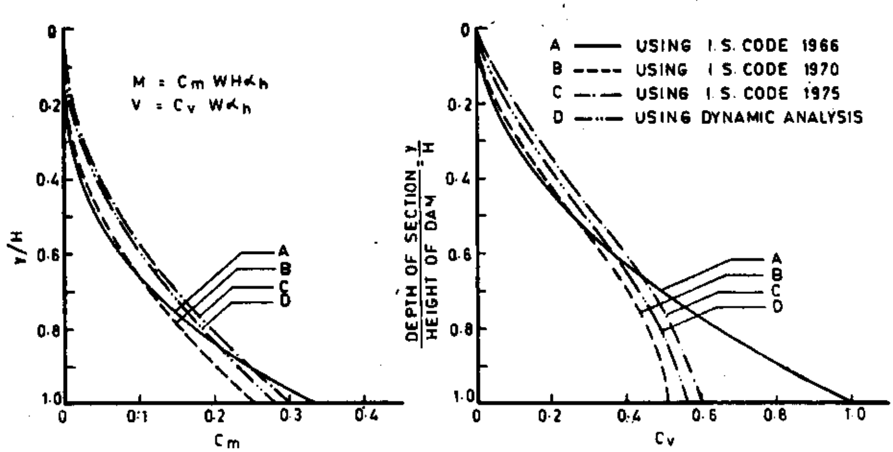


Fig. 7 Comparison of dynamic moments and shears due to horizontal inertia forces for 200 m high dam

code provisions indicate large error in evaluating the dynamic moments and shears in the top portion of the dams. For a 30 m high dam, the dynamic moments are overestimated by 14 percent at the base but these are underestimated by about 32 percent in the top portion of the dam. Similarly, the dynamic shears are overestimated by 12 percent at the base and these are underestimated by about 25 percent in the top portion of the dam. The underestimation of dynamic moments and shears in the top portion of the dams is more for higher dams. This can be observed from Figs. 6 and 7 which give the distribution of dynamic moments and shears for dams of 150 m and 200 m in height.

The I.S. Code provision of 1975 (3) is observed to be reasonably realistic and gives results close to those obtained by dynamic analysis. For a 150 m high dam, the dynamic moments are overestimated by about 11 percent at base and about 17 percent in the top portion of the dam. For other dam heights also, it can be seen that the distribution of dynamic moments and shears as obtained by using the I S. Code provisions of 1975 (3) closely follows the variation as obtained from dynamic analysis (4). The overestimation by small amounts as obtained in the distribution of dynamic moments and shears using response spectrum analysis can be accepted because of the many uncertain factors on which estimation of seismic forces is based. Thus, the code provisions in the third revision (3) are quite rational and can be used with more confidence in the preliminary design of dams.

Assessment of Hydrodynamic Forces

The I.S. Code provisions (1, 2, 3) specify Zanger's expressions (9) for estimating hydrodynamic pressures on dams which are based on the assumption of treating the dam as rigid and neglecting compressibility of water. The I.S. Code also gives approximate expressions for obtaining hydrodynamic moments and shears due to hydrodynamic effect as follows :

$$V_h = 0.726\, p_e\, y \tag{3}$$

$$M_h = 0.299\, p_e\, y^2 \tag{4}$$

Where V_h is the hydrodynamic shear, M_h the hydrodynamic moment, p_e the intensity of hydrodynamic pressure and y the depth of the section below the reservoir water surface.

Since, the hydrodynamic pressure has parabolic distribution, the hydrodynamic shear coefficient of 0.726 and hydrodynamic moment coefficient of 0.299 as given in the approximate expressions are not expected to be constant along the depth of reservoir. It is, therefore, essential to assess the percentage error due to approximation made in the simplification. This will help to check the adequacy of the approximate expressions for evaluating hydrodynamic shear and moment.

Knowing the intensity of hydrodynamic pressure along the depth of the reservoir, the hydrodynamic shear and moment can be obtained as :

$$V_h = \int_0^y p_e\, dz \tag{5}$$

$$M_h = \int_0^y p_e\, (y-z)\, dz \tag{6}$$

Where p_e is the intensity of hydrodynamic pressure, y the depth of section below the free surface of the reservoir, z the depth at which element strip of the pressure diagram is considered and dz the depth of elemental strip.

The hydrodynamic shears and hydrodynamic moments are expressed as :

$$V_h = C_{vh}\, p_e\, y \tag{7}$$

$$M_h = C_{mh}\, p_e\, y^2 \tag{8}$$

where C_{vh} is the hydrodynamic shear coefficient and C_{mh} the hydrodynamic moment coefficient.

The variation of hydrodynamic shear coefficient C_{vh} and hydrodynamic moment coefficient C_{mh} has been plotted along the depth of the reservoir and is shown in figure 8.

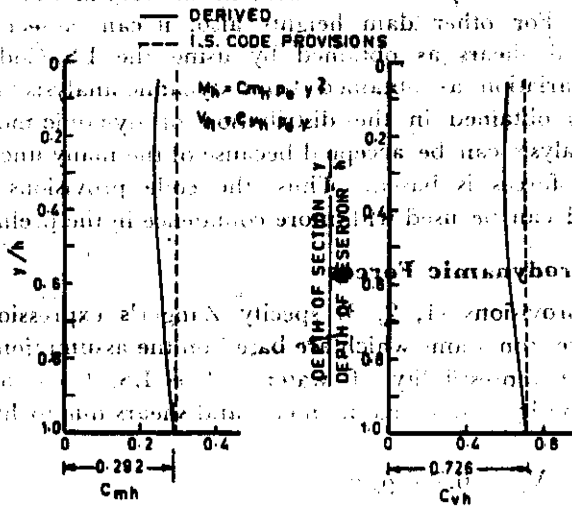


Fig. 8 Distribution of hydrodynamic moment and shear coefficients

It can be seen from the figure that the value of hydrodynamic shear coefficient of 0.726 at base compares exactly with the value as given in approximate expressions given in the code (1, 2, 3). The hydrodynamic shears are, however, overestimated using code provisions at higher elevations. The overestimation varies upto 15 percent along the depth of reservoir, the maximum overestimation being at a section which is at 0.3 h below the free reservoir surface, h being the depth of the reservoir. It is also observed from the figure that the derived value of hydrodynamic moment coefficient of 0.292 at base closely compares with the value of 0.299 given in the approximate expression given in code (1, 2, 3). The hydrodynamic moments at higher elevations are overestimated as per provisions in code (1, 2, 3). The overestimation varies upto about 20 percent along the depth of reservior, the maximum overestimation being at a section which is at 0.3 h below the reservoir surface. It is, thus, concluded that hydrodynamic shears and moments are marginally overestimated using the approximate expressions given in I.S. Code (1, 2, 3). These errors are, however, considered small and can be accepted for the preliminary design of dams.

Comparison of I.S. Code Provisions with Simplified Design Procedures

The I.S. Code provisions do not take into account the dam-reservoir interaction and neglect the compressibility of reservoir water. Simplified design procedures (5, 6) are now available which account for these effects. The I.S. Code provisions have been compared here with the simplified design procedure for a 100 m high dam. To evaluate the lateral seismic forces, computational steps as given in the simplified method of design (5, 6) have been followed. The dynamic shears and dynamic moments have been expressed as :

$$V_1 = C_{v1} W \alpha_h \qquad (9)$$

$$M_t = C_{mt} W H \alpha_h \qquad (10)$$

in which, C_{vt} and C_{mt} are dynamic shear and moment coefficients considering the effect of inertia forces and hydrodynamic forces, W the total weight of dam, H the total height of dam and α_h the design horizontal seismic coefficient.

The provisions in I.S. Code (3 treat the dam-reservoir as an uncoupled system. Hence the inertia and hydrodynamic forces due to earthquake have been worked out separately as per provisions in the I.S. Code (3) and the two uncopled solutions are then combined to obtain the total dynamic shears and moments. The variation of dynamic shear and moment coefficients, C_{vt} and C_{mt} has been plotted in figure 9.

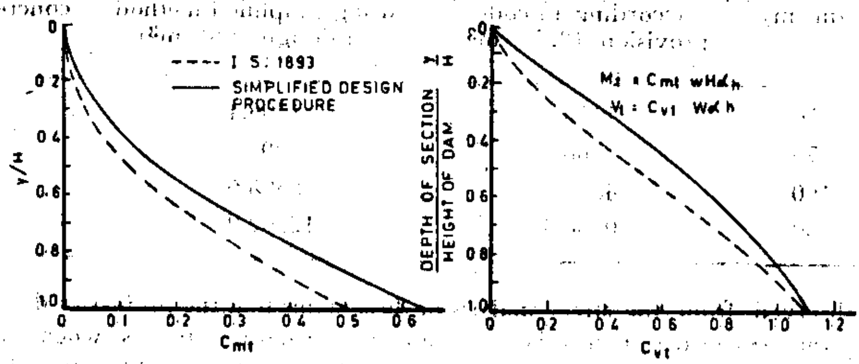


Fig. 9 Distribution of total dynamic moments and shears for 100 m high dam

It is observed from figure 9 that the dynamic moments and shears as obtained using simplified design procedure (5, 6) are substantially higher than those obtained using I.S. Code provisions (3). The simplified design procedure (5, 6) indicates that the dynamic moments are 24 percent higher at base than those obtained by using code provisions (3). These are higher to the extent of 34 percent at depth 0.5 H from top of dam and more than 80 percent at depth 0.2 H and 0.3 H from top of dam. The dynamic shears as obtained by simplified method of design (5, 6) are marginally higher at base but at higher elevations, the dynamic shears are considerably higher to the extent of 34 percent at depth 0.5 H and 68 percent at depth 0.1 H below the top of the dam, as compared to corresponding values obtained using code provisions (3).

It is, thus, seen that the effect of compressibility of water and dam-reservoir interaction is considerable and need to be included for a rational design of dams. It is, however, to be noted that higher tensile stresses will be indicated in the dam section and hence concrete volume required for the dam would be considerably increased to satisfy the present criteria of stability (7) which does not permit the occurrence of tensile stresses even under critical loading condition. Hence, it is essential to check the suitability of the simplified design procedure (5, 6) for use in design before adopting.

Suitability of the simplified design procedure

To study the suitability of the simplified procedure (5, 6) in design practice with the presently accepted design criteria (7), its effect on the design of gravity dams is analysed. To investigate this aspect, gravity dams of heights 50 m, 75 m, 100 m, and 150 m have been designed using the simplified design procedure (5, 6) and the I.S. Code provisions (3, 7). Table 1 gives the concrete volume and percentage increase of concrete volume in dams obtained by designing according to simplified method of design over and above the conventionally designed dams using I.S. Code provisions (3, 7).

Table 1

Percentage increase of concrete volume in dams

Sl. No.	Height of dam (m)	Concrete volume of dam per unit length designed according to code provisions (3, 7) (m^3)	Concrete volume of dam per unit length designed by using simplified method of design (5,6) (m^3)	%age increase in concrete volume
1.	50	1179.1	1296.4	10.0
2.	75	2668.3	2893.1	8.5
3.	100	4865.0	5395.0	11.0
4.	150	10688.4	12202.9	14.0

It is observed from the stress analysis of these dams that the profiles of dams designed according to the present design criteria (3, 7) show higher tensile stresses when analysed by using simplified method of design (5, 6). It can be seen from Table 1 that the currently accepted design criteria for tensile stresses in dams designed using simplified method of design can be achieved only at the cost of substantial increase of 10 to 14 percent in concrete volume over and above the concrete volume required for conventionally designed dams. It is to be noted that simplified method of design (5, 6) though provides a more rational approach to estimate seismic forces acting on dams but its use in design practice is some what limited, at present, until the improved design criteria for permissible tensile stresses in dams is evolved based on laboratory and field tests of material strength under dynamic conditions.

Effect of seismic provisions on cost of gravity dams

The seismic zones of India have been revised and upgraded to some extent in the various revisions of I.S. Code (2, 3). The dam sites which were located in the seismic zones with lower seismicity as per provisions in the seismic zoning map (1) are now coming under seismic zones with higher seismicity in the revised zoning map (2, 3) of the country. In view of this, the safety and economy of such dams should be re-evaluated. The dam sites located on the boundaries of seismic zones are always subject to discussion as to the value of seismic coefficient to be adopted in the design. It would, therefore, be useful to study the effect of variation in design coefficient on cost of gravity dams. This will help to arrive at the judicious decision of design seismic coefficient. Since the cost of dam involves many other factors, percentage increase in concrete volume is taken as indicative of increase in the cost of the dam.

To study the effect of variation of design seismic coefficient on the cost of gravity dams, dams of heights 50 m, 75 m, 100 m and 150 m have been selected as these are in the common range of heights in case of concrete and masonry storage dams in India. The dimensions of the profile of dams, like top width, free board etc. have been based on the data collected from actual dams (4). These parameters of dam profiles as assumed in the study are given in Table 2.

Table 2

Parameters of dam sections

Sl. No.	Ht. of dam (m)	Top width (m)	Free board (m)	Fetch (km)
1.	50	6.5	2.5	5.0
2.	75	7.5	3.0	7.0
3.	100	9.0	4.0	9.0
4.	150	10.0	5.0	11.0

The pertinent data which is assumed in the design is as follows :

Density of dam material	$= 2.4$ t/m^3
Modulus of elasticity of dam material	$= 2.2 \times 10^6$ t/m^2
Damping	$=$ 5 percent of critical damping
Shear strength of foundation rock	$= 500$ t/m^2
Coefficient of friction between concrete and rock	$= 0.75$

The variation in design seismic coefficient is assumed to be on zonal basis i.e. each dam section is designed for seismic coefficient pertaining to seismic zone I, II, III, IV and V as given in code (3). In order to find out the total percentage increase in concrete volume of dam due to seismic forces, concrete volume of dam designed with design seismic coefficient equal to zero has been taken as the basis. Dam sections have been designed using gravity method of analysis for the loading conditions as stipulated in I.S. 6512 : 1972 (7) and the earthquake loading considered as specified in I.S. Code 1893 : 1975 (3). A computer program which takes into account these criteria has been developed and extensively used for the stress and stability analysis of dams. The design has been carried out by ensuring safety against sliding as well as ensuring that the stresses are within permissible limits. The maximum permissible compressive stress does not generally form a restricting design criteria in the design of dams as the compressive strength of concrete mix. specified will be adequate to take up the compressive stresses. As far as the permissible tensile stresses are concerned, the I.S. Code (7) does not permit the tensile stresses to occur at the upstream face of the dam. It is, however, a common practice to allow nominal tensile stresses of 1 to 2 kg/cm^2 under critical loading condition. Accordingly, nominal tensile stresses of 2 kg/cm^2 have been permitted in the design of dam sections.

The influence of variation in design value of horizontal seismic coefficient on concrete volume of gravity dams is shown in figure 10 for various heights of the dams considered. It is noted from the figure that the relationship between variation in design seismic coefficient and the percentage increase in volume of concrete in dams is not linear. The cost of the dam

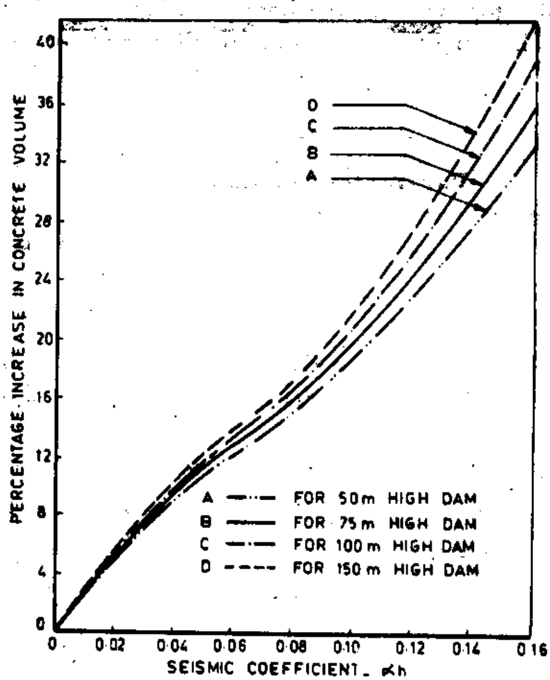


Fig. 10 Effect of design seismic coefficient on increase in concrete volume of dams

increases rapidly with an increase in the design horizontal seismic coefficient. For a variation of 0.02 only in design horizontal seismic coefficient from 0.10 to 0.12, the net percentage increase in concrete volume of dams varying in height from 50 to 150 m is about 4 to 5 percent. Thus, for the projects located on the boundaries of seismic zones, the value of design seismic coefficient will have to be determined taking into account its effect on the cost of the dam also.

It is also observed from the figure that for a design value of horizontal seismic coefficient equal to 0.05, which is usually considered as not very large, the percentage increase in concrete volume of dams varying in height from 50 m to 150 m, on account of seismic forces is about 10 to 12 percent. Thus the extra investments called for on account of earthquake provisions are of a major order and the value of design seismic coefficient should be properly selected giving due consideration to the economic aspects also. Fig. 10 is expected to serve as a good guide to estimate the increase in the cost of the dam due to an increase in the design seismic coefficient.

Conclusions

Based on this investigation, the following conclusions can be drawn :

1. The distribution of inertia forces due to horizontal earthquake acceleration as specified in I.S : 1893 - 1975 (3), is reasonably realistic and compares well with the dynamic analysis.

2. The approximate expressions given in the I.S. Code (1, 2, 3) to estimate the hydrodynamic forces due to earthquake overestimate the hydrodynamic moments to the extent of 20 percent and hydrodynamic shears to the extent of 15 percent. The use of these expressions is, however, considered to be reasonable in the preliminary design of dams.

3. The simplified design procedure (5, 6) which accounts for compressibility of water and dam-reservoir interaction provides a more rational approach for realistic assessment of seismic forces as compared to I.S. Code provisions (3) which do not account for these effects. It is, however, noted that it is uneconomical to use the method in design practice with presently accepted design criteria and an improved design criteria for permissible tensile stresses should be evolved. It is observed that for the dam sections designed by using simplified method of design (5, 6) and present design criteria (7), a substantial increase of 10 to 14 percent in concrete volume of dam is required, over and above that required for conventionally designed dam as per I.S Code provisions (3, 7).

4. The cost of gravity dams is influenced to a great extent even by a small variation in the design value of the seismic coefficient. It is observed that for commonly adopted design seismic coefficient equal to 0.05, the total percentage increase in concrete volume due to seismic forces is about 10 to 12 percent. The extra investments called for on account of earthquake provisions are of a major order and hence judicious decision is essential to determine the design seismic coefficient.

Acknowledgements

Thanks are due to Professor and Head, Water Resources Development Training Centre, University of Roorkee, Roorkee for permission to use the computer facilities at I.P.E., Dehradun.

References

Indian Standard Institution, I.S. : 1893-1966, "Indian Standard Criteria for Earthquake Resistant Design of Structures" New Delhi.

Indian Standard Institution, I.S. : 1893-1970, "Indian Standard Criteria for Earthquake Resistant Design of Structures", New Delhi.

Indian Standard Institution, I.S. : 1893-1975 "Indian Standard Criteria for Earthquake Resistant Design of Structures", New Delhi.

Saini, S. S. and V. H. Kulkarni, "Behaviour of Concrete Gravity Dams subjected to Earthquakes", Proc. Fifth Symposium on Earthquake Engineering, Roorkee, November 1974.

Chopra, A. K., "Earthquake Resistant Design of Concrete Gravity Dams", Journal of Structural Division, Proc. ASCE, Vol. 104, No. ST 6, June 1978.

Chopra, A.K. and C.F. Corns, "Dynamic Method for Earthquake Resistant Design and Safety Evaluation of Concrete Gravity Dams", Trans. ICOLD, New Delhi, Oct. 1979.

Indian Standard Institution, I.S. : 6512-1972, Indian Standard Criteria for Design of Solid Gravity Dams", New Delhi.

Saini, S.S., "Vibration Analysis of Dams" Ph.D. Thesis, University of Roorkee, Roorkee, Sept. 1969.

Zanger, C.N., "Hydrodynamic Pressures on Dams Due to Horizontal Earthquakes", Proc. of the Society for Experimental Stress Analysis, Vol. 10, No. 2, 1953.

Arya, A.S., S.K. Thakkar and G.I. Prajapati, "Earthquake Stresses in Gravity Dams with Light Structural System at Top", Journal of the Central Board of Irrigation and Power, Vol. 30, No. 4, October 1973.

Arya, A. S. and R. K. Agarwal, "Optimum Design of Concrete Gravity Dam Sections Considering Earthquake Forces", M.E. Dissertation, University of Roorkee, Roorkee, 1970.