SEISMIC CONSIDERATIONS IN THE DESIGN OF CONCRETE GRAVITY DAMS

A DISSERTATION
submitted in partial fulfilment
of the requirements for the award of the Degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

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CERTIFICATE

Certified that the dissertation entitled "SEISMIC CONSIDERATIONS IN THE DESIGN OF CONCRETE GRAVITY DAMS" which is being submitted by Er. M.V. Deshpande in partial fulfilment of the requirements for the award of the Degree of Master of Engineering in Water Resources Development of ~~the~~ University of Roorkee is a record of candidate's own work carried out by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other Degree or Diploma.

This is to further certify that he has worked for a period of more than nine months for preparing this dissertation.

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A C K N O W L E D G E M E N T

The author deems it a proud privilage to work under Dr. S.S.Saini, Reader in Civil Engineering, Water Resources Development Training Centre, University of Roorkee, Roorkee and expresses his deep sense of gratitude for his valuable guidence, encouragement and critical review at each stage in preparation of this dissertation.

The author is thankful to his wife Sau Alka Deshpande who has helped him a lot in furtherance of his studies.

A number of articles on the subject as mentioned in the list of references have been consulted and material from them freely used in preparation of the dissertation for which the author expresses his thanks to the authors of these articles.

M.V.DESHPANDE

SYNOPSIS

Earthquake is an important consideration in the design of gravity dams which are located in seismically active areas. Therefore these dams must be properly designed after taking into account the probable seismic forces along with other loads.

Past as well as current procedures which are based on assuming a design seismic coefficient in the aseismic design of dams are entirely inadequate as they do not take into account the properties of the structure its damping characteristics and ground motion.

Theoretical techniques such as beam analysis and the detailed analysis by finite element method can predict the behaviour of concrete gravity dams subjec-ted to earth-quake accurately. The experimental technique of testing the models of gravity dams can be used to verify the beha-viour calculated theoretically.

The IS code for earthquake resistant design of structures are under constant revision from time to time (1966, 1970 and 1974). The dynamic moments and shear distribution as given by these codes when compared with the one obtained by dynamic analysis (proposed after carrying out dynamic analysis of a number of practi-cal dam profiles), show that the latest regision (IS code

1974) is most rational as the moment and shear distribution tallys very well with dynamic analysis. Therefore the Is code of 1974 can be used with more confidence in preliminary design of major dams. For final design however detailed dynamic analysis should be used.

The hydrodynamic shears and moments as given by the approximate formulae in IS codes (1966, 1970 and 1974) over-estimate the values over the actual. The percentage of overestimation is 54 per cent at 10 per cent depth to 2.2 per cent at bottom in case of hydrodynamic moment and the same is 15 per cent and zero per cent in case of hydrodynamic shear. Correct Distribution of hydrodynamic moment and shear is proposed.

Light weight structural system should be provided at the top of the dam to reduce tensile stresses at the top. The upstream face of the dam should be kept sloping instead of vertical. To reduce tensile stresses at top. Abrupt changes in the slopes of the dam should be avoided by providing proper transitions or fillets to reduce concentration of tensile stresses. The theoretical techniques of dynamic analysis of dam have been found to be adequate in predicting the behaviour of dam, subjected to earthquake. The experimental techniques of testing of models of dam on shake table are used to verify the theoretical analysis.

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CHAPTER I

INTRODUCTION

1.1 INTRODUCTION

Concrete gravity dams usually form an important element of multipurpose projects e.g. hydropower, irrigation and flood control. In India a number of river valley projects are being taken up. The gravity dams need a huge amount of investment. Therefore the dam sec tions would have to be designed properly after taking into account the forces that would be acting on it. Amongst the other forces ac ting on the dam, earthquake force is an important one governing its stability. This is more true if the dam is located in a seismically active area. Hence the dams located in seismically active areas would be subjected to dynamic forces caused by earthquakes. It is therefore essential that these dams should be designed taking into account the anticipated earthquake forces so that they can safely withstand future shocks without serious damage Since the failure of a dam is much more disastrous to a community than that of any other structure. The observed behaviour of Koyna dam during the earthquake of December 11, 1967 , indicates that gravity dams would be expected to suffer comparable damage during similar earthquake. Therefore utmost care is necessary to account for the seismic forces in the design of gravity dams.

Past as well as current practices for the design of dams from a seismic point of view are based on assumption of a design seismic coefficient. Thus the seismic forces are accounted for as equivalent static forces. The Indian standard codes of earthquake resistant design of dams is revised from time to time (1966, 1970 , and 1974). The provisions of 1966 IS code are entirely inadequate in estimating dynamic moments and shears. The IS code of 1970 is a bit improved over the 1966 code in that it is first time recognised that the accelerations at the top of the dam are more than these at bottom and thus the 1970 code specifies a triangular variation of design seismic coefficient from top to bottom. The IS code of 1974 differentiates for the first time between high dams (more than 100 m height) and low dams (less than 100 m height) and specifies two approaches for aseismic design viz, the Response spec-trum Approach in case of high dams and the seismic coefficient Method in case of low dams. The provisions of different IS codes as regards the dynamic moment and shear distribution are compared with those given by dynamic analysis for four cases of dam heights of 50 m, 100m , 150 m and 200 m. It is found that the IS Code of 1974 gives a realistic distribution of dynamic moments and shears which is very close to the one given by dynamic analysis.

The approximate formulae given by the various IS codes (1966, 1970 and 1974) for calculating hydrodynamic shear and moment were verified and it is observed that the assumption, that the hydrodynamic shear and moment coefficients, are constant over the depth of the reservoir is not correct. In case of hydrodynamic moments the percentage of error varies from as high as +54 per cent at a depth of 10 per cent from the reservoir water surface to as low as +2.2 per cent at the bottom of the reservoir. In case of hydrodynamic shear the values obtained from the formula given in IS Code are on higher side. The percentage error at bottom is nil and it is about +15 per cent at a point which is at a depth 10 percent of depth below the reservoir water surface. Thus the hydrodynamic moments and shears as given by the IS Code formulae are on a higher side than those given by theoretical analysis. Correct distribution of hydrodynamic moment and shear is proposed.

1.2 OUTLINE OF DISSERTATION

Chapter II deals with a critical review of current design procedure adopted in the design of dam from seismic point of view. The current procedure to calculate the dynamic moments and shears and the hydrodynamic pressure is presented.

Chapter III deals with the analytical and experimental techniques for estimating the dynamic behaviour of gravity dam. The two theoretical techniques viz the beam method and the finite element analysis have been reviewed. The method of dynamic testing of gravity dams on shake table to verify its behaviour as predicted by the theoretical techniques is briefly discussed.

Chapter IV presents a comparison of dynamic moment and shear distribution in case of four selected dam profiles with 50 m, 100 m, 150 m and 200 m height and downstream slope 0.8 : 1 and vertical upstream face as calculated by the provisions of the different Indian Standard Codes of earthquake resistant design of structures (IS: 1893 of 1966, 1970 and 1974). These moment and shear distributions are compared with the dynamic analysis. The formulae for determining hydrodynamic shear and moment as given in various IS codes are verified. The hydrodynamic moments as given by the formula in IS codes are higher than actual by about 54 per cent at 10 per cent depth and by about 2.2 per cent at bottom of reservoir. The corresponding figures in case of hydrodynamic shear are 15 per cent and zero. A correc-t distribution of hydrodynamic moment and shear distribution is proposed.

Chapter V deals with the significant results obtained by various investigators after studying the dynamic behaviour of a number of gravity dam sections theoretically and experimentally . The effect of reservoir water and vertical ground motion on the dynamic response of the dam is discussed. The importance of using light weight system at top of dam is to reduce the tensile stresses at the top of the dam is brought out. The effect of abrupt changes in downstream face slopes of the dam is discussed. The effect of sloping upstream face of the dam is discussed. The adequacy of theoretical techniques of dynamic analysis is discussed.

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CHAPTER - II

PRESENT DESIGN PROCEDURE

2.1 GENERAL

The forces which are considered in the design of gravity dams are well known. These forces are broadly divided into two main categories for the purpose of design. They are as follows :

(a) Normal Loads -

These are the loads which are always acting on the dam. They are, the weight of dam and its superstructure, the water pressure corresponding to the reservoir level and uplift. The weight of the dam and superstructure can be determined to a fairly reasonable amount of accuracy. Similarly the water pressure can be determined fairly accurately. The uplift pressures are however not precisely predictable. The dam shall have adequate stability under normal loads. The fac-tor of safety and the permissible stress in the dam in case of normal loads shall not be exceeded.

(b) Abnormal Loads

These loads are those which are occasional in nature and which are not always acting on the dam. They are, the wave pressure, silt load, ice thrust and earthquake force. The earthquake force is the most important of them all. But, unfortunately the earthquake forces are not precisely predictable. The effect of the earthquake on the reservoir-dam system is two fold. First, as the ground vibrates the dam also vibrates giving rise to the inertial forces. Secondly, the reservoir also vibrates, giving rise to the hydrodynamic pressures. The two forces i.e. the inertial force and the hydrodynamic pressure are discussed below:

2.2 INERTIA FORCE

The Indian Standard criteria[1,2] for Earthquake resistant Design of Structures (IS: 1893 , of 1966 and 1970) specify a basic seismic coefficient (α_o) for each of the seismic zones in which the country is divided.

The horizontal seismic coefficient α_h as specified by the IS:1893 of 1966 and the IS: 1893 of 1970 is as given in Fig. 2.1 below

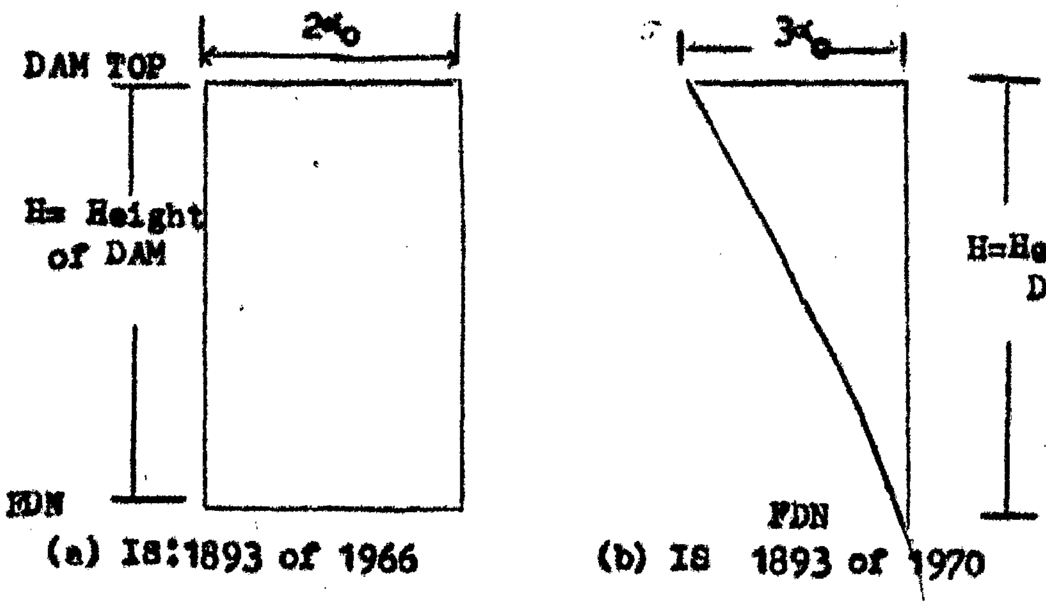

Fig.2.1 Distribution of Seismic Coefficients along height of dam.

The effect of vertical earthquake acceleration is to change the unit weight of water and concrete. An acceleration upwards increases the weight and an acceleration downwards decreases the weight. The vertical and horizontal forces and moments computed for the analysis without earthquake effect are multiplied by the factor $(1 \pm \alpha_v)$ to include effect of vertical acceleration. The vertical seismic coefficient α_v shall be taken as half of the horizontal seismic coefficient at any point according to both the IS codes[1,2].

The uplift is assumed to be unaffected by the earthquake, according to both the IS Codes[1,2]. The duration of earthquake is too short to permit building up of pore pressure in the concrete and foundation[1,2].

Experimental and analytical methods indicate that an earthquake acceleration is one half as effective in silt or soil masses as it is in water[1,2]. This is due to the internal shear resistance of the silt. Since the unit weight of water is also approximately one-half that of silt, it is sufficient to determine the increase in silt pressure as if the water is extended to the base of the dam. This increase is then added to the static silt pressures[1,2].

The U.S.B.R.[10,11] recommends that the earthquake loadings should be selected after consideration of the horizontal and vertical accelerations which may be reasonably expected at a particular site. These are determined from the geology of the site, proximity to major faults, the previous earthquake history of the region, and such seismic records as are available[10]. It envisages use of uniform horizontal and vertical seismic coefficient along the height of the dam[10,11]. The joint ASCE-USCOLD Committee[4] states that there are different prac-tices in the United States as regards the earthquake forces. Seismic forces are generally considered in areas where there has been a history of seismic activity[4]. Seismic effects are applied as loads on the dam and are computed from the concrete inertia effects due to acceleration. Horizontal accelerations ranging upto 0.1g are used. Hinds, Creaguer and Justin[3] also recommend a uniform seismic acceleration along the height of the dam. The inertia force is assumed to act through the centre of gravity of the section being considered[3].

Thus in all the previous publications[2,3,10,11] it was a common practice to use uniform seismic coefficient all alongthe height of the dam. The IS code of 1970 for the first time specifies a triangular variation of seismic coefficient along the height of the dam. For calculation of inertia force, the dam is divided into a number of horizontally divided sections and the weight of each section is found out. The weight of the section multiplied by the seismic coefficient at the centre of gravity of the section gives the inertia force acting on the section. The cumulative total of these inertial forces from top to bottom of the section gives the dynamic shear and sum of the moment of these inertial forces about the centre of gravity of the section under consideration gives the dynamic moment.

2.3 HYDRODYNAMIC PRESSURE

There are different approaches for the determination of the hydrodynamic pressure. The nature of water pressure caused by earthquake may be represented by a curve the true equation of which is complex. Davis(5) assumed this curve to approximate on ellipse. Westergaard(6) determined that the hydrodynamic pressure curve approached a parabola. But ,the formulae given by him are applicable to dams with vertical upstream face only. It has been shown by Kotsubo(8) that Westergaard's solution is valid only when the period of harmonic excitation is greater than the fundamental natural period of the reservoir.

Braktz and Heilburn(7) demonstrated that if the reservoir is of finite length in the upstream direc-tion , the pressure increase is not more than 0.5 % for $L/H > 2$. Where, L is the length of the reservoir and H the depth of the reservoir. If the upstream end of the reservoir is assumed to vibrate with the ground, the effect of length is negligible for $L/H > 3$. It has also been shown[9] that the pressure response is not sensitive to reservoir length. It may be said in practical situations as the reservoir extends to large distances, the length of the reservoir may be taken as infinite without introducing much error.

The most widely used formulae for the determination of the hydrodynamic pressure are those given by Zangar[12]. The Zangar's formulae are based on the assumptions that the water is incompressible, only horizontal earthquake is considered, dam is assumed to behave as a rigid body, earthquake is assumed to manifest itself as a harmonic motion, and the displacements are assumed to be small. The IS code of 1966[1,2] and 1970 both specify the use of Zangar's formulae for determination of hydrodynamic pressure. The hydrodynamic pressure at any depth y below the reservoir is given by

$$P_e = C \alpha_h wh \qquad (2.1)$$

where, P_e is the hydrodynamic pressure in kg/m^2 at depth y, C the coefficient which varies with shape and depth, α_h the horizontal seismic coefficient($= 2\alpha_o$)[1,2], w the unit weight of water in kg/m^3 and h the depth of reservoir in m. For accurate determination of the value of 'C' the IS Codes[1,2] of 1966 and 1970 provide curves for various shapes of the upstream face of the dam and depths which may be used. The approximate value of 'C' may be obtained by[1,2]

$$C = \frac{C_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\,\right] \qquad (2.2)$$

where, C_m is the maximum value of c, h the depth of the reservoir in m and y the depth below the reservoir surface. A curve of the maximum value of C i.e. C_m against the inclination of the upstream face of the dam with the vertical is given in the two IS Codes[1,2] for cases of the dam where the upstream face has a constant slope. If the slope of the upstream face is not constant then for the case where the height of the vertical upstream face of the dam is equal to or greater than half the total height of the dam, then the dam is analysed as if upstream face is vertical throughout. If however, the height of the vertical portion of the upstream face of the dam is less than one half of the total height of the dam, the pressure on the sloping line connecting the point of intersection of the upstream face of the dam without the reservoir surface with the point of intersection of the upstream face of the dam with the foundation. The approximate values of the total horizontal shear and moment about the centre of gravity of a section due to hydrodynamic pressure are given by

$$V = 0.726 \ Pe \ y \qquad (2.3)$$

$$M = 0.299 \ Pe \ y^2 \qquad (2.4)$$

where V and M are the hydrodynamic shear and moment respectively at depth y below the reservoir depth.

Since the hydrodynamic pressure acts normal to the face of the dam, there shall, therefore, be a vertical component of this force if the face of the dam against which it is acting is sloping, the magnitude at any horizontal section is given by

$$W = (V_2 - V_1) \tan \theta \qquad (2.5)$$

where, W is the increase (or decrease) in vertical component of load in kg due to hydrodynamic force, V_2 the total shear in kg due to horizontal component of hydrodynamic force at the elevation of the sec-tion being considered V_1 the total shear in kg due to horizont al component of hydrodynamic force at the elevation at which the slope of the dam face commences, and θ the angle between the face of the dam and the vertical . The moment due to the vertical component of the reservoir and tail water load may be obtained by determining the level arm from the centroid of the pressure[1,2].

2.4 DEFICIENCIES OF PRESENT PROCEDURE

As discussed already the present design procedure treats earthquake forces as equivalent static forces. These methods are entirely inadequate for estimating the correct stresses caused by earthquake forces in the gravity dams. These methods disregard the properties of the structure and the nature of ground motion. The seismic coefficient methods lack any rational justification. The behaviour of Koyna dam during the earthquake of December 11, 1967 has amply proved the inadequacy of the present design procedures for aseismic design of gravity dams. Therefore, the seismic coefficient methods may be adequate for design of less important structures located in areas which are not visited by severe earthquakes . But these methods are not adequate for designing more important dam, located in seismically active areas. For these dam, detailed dynamic analysis may be resorted.

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CHAPTER - III

ANALYTICAL AND EXPERIMENTAL TECHNIQUE OF ANALYSIS FOR ESTIMATING BEHAVIOUR OF GRAVITY DAMS

3.1 INTRODUCTION

A gravity dam is composed of monoliths. These monoliths are separated by transverse contraction joints. These joints are generally neither keyed nore grouted. Therefore the external load is transmitted vertically to the foundation and no load is transmitted horisontally to the abutments. The monoliths are free to vibrate independent of one another. Thus the stability of individual monoliths will give an idea of stability of the whole structure. The two analytical methods of dynamic analysis viz the beam method and the detailed analysis using finite element method take into account the dynamic properties of structure, its damping characteristics and the nature of ground motion. Therefore, they are preferbble over the present pseudo static methods.

3.2 BEAM ANALYSIS

Here the dam is treated as vertical cantilever fixed at the base. The vibration of the dam transverse to its axis can only be taken into account in this method. It is not possible to study the effect of vertical component of ground motion on the dynamic behaviour of the dam in this method. For the design of the section of the dam it is necessary to know the forces to which it would be subjected. Knowing the forces, the stresses can easily be calculated. The steps up to getting the dynamic moments and shears will be discussed here. The material of the dam is assumed to behave linearly elastic. As the width of the gravity dam is significant in comparison to its height, shear and rotary inertia deformations are usually considered in addition to bending deformations.

The steps involved in the beam analysis are, determination of the natural frequencies and mode shapes of the dam, selecting an earthquake response spectra for calculating the earthquake response of the dam, in its various modes. (Here it is necessary to know the damping characteristics of the dam), and finally combination of the responses in the various natural modes of vibration of the dams. These steps would be discussed below.

The natural frequencies and mode shapes of the dam are determined from the case of free undamped vibrations. The equations of motion for free vibrations considering bending shear and rotary inertia effects can be written [24,25].

$$\frac{\partial}{\partial x}\left(EI \frac{\partial^2 Y_b}{\partial x^2}\right) + \mu A G \frac{\partial Y_s}{\partial x} = \rho I \frac{\partial^3 Y_b}{\partial x \, \partial t^2} \qquad (3.1)$$

$$\frac{\partial}{\partial x}\left(\mu A G \frac{\partial Y_s}{\partial x}\right) = \rho A \frac{\partial^2 Y}{\partial t^2} \qquad (3.2)$$

In the above equations Y is the total deflection, Y_b is deflection due to bending, Y_s is the deflection due to shear, X the distance measured along the height, t the time variable, E the modulus of elasticity of the dam material. ρ the mass density of the beam material, A the area of cross-section of the beam, I the moment of inertia of section about its neutral axis, μ the shape factor.

As the cross-section of the dam is non-uniform along the height and may have any arbitrary variation of slopes, a theoretical solution of equations 3.1 and 3.2 is almost impossible to obtain. Numerical techniques are used to arrive at the solution and this is most suited for use with digital computers. In order to apply the numerical techniques, it is necessary to replace the continuous system by a discrete system. This is done by dividing the dam into a convenient number of segments. The masses of the segments are lumped at the centre of the segment. These lumped masses are assumed to be connected by elastic elements having the properties of the original structure. Under free vibrations, the system vibrates harmonically . A typical discrete system is shown in Fig. 3.1 The shear, moment and deflection diagram for such a system are shown in Figure 3.2.

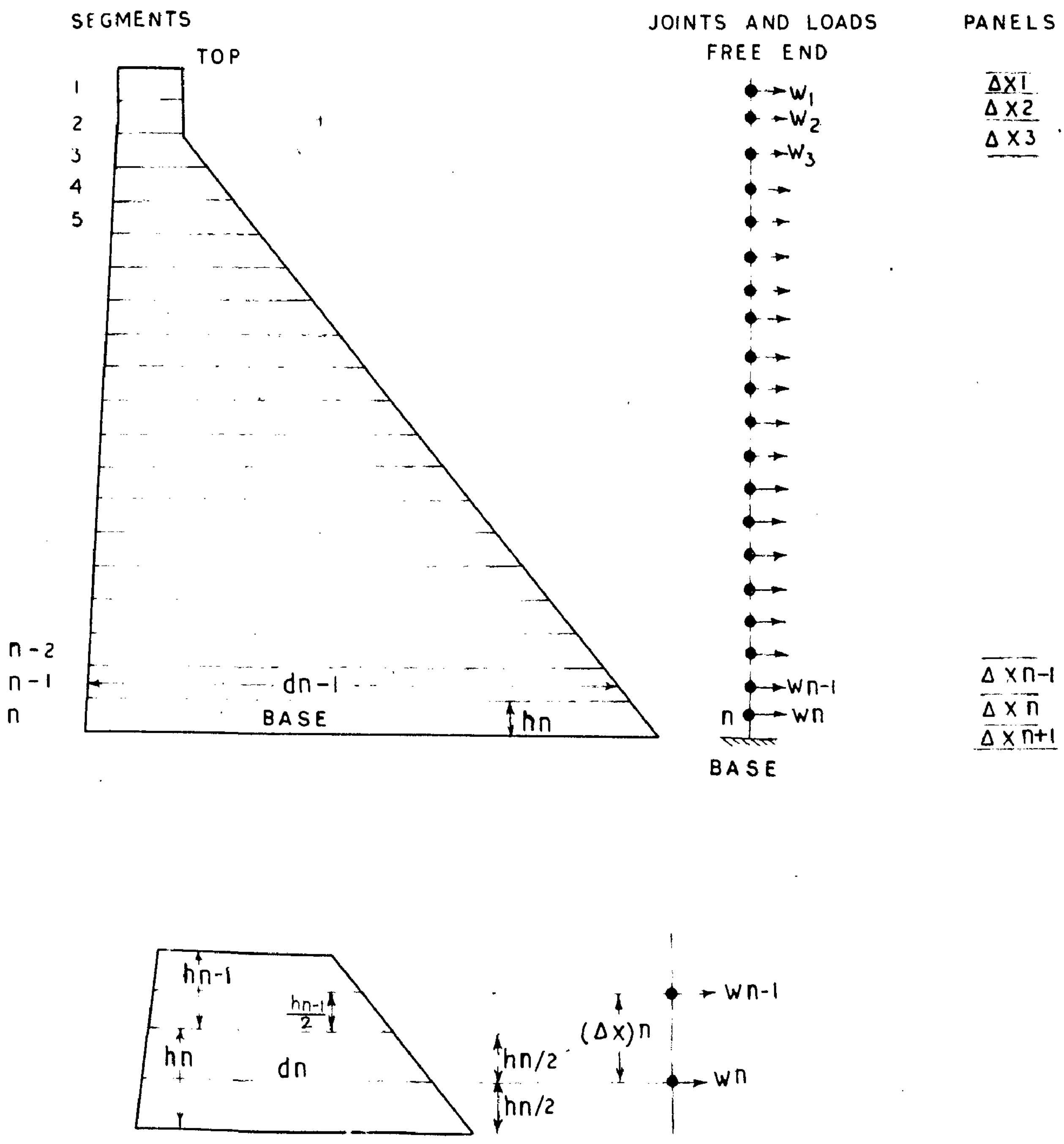


FIG. 3·1_ DAM SECTION AND DISCRETE SYSTEM

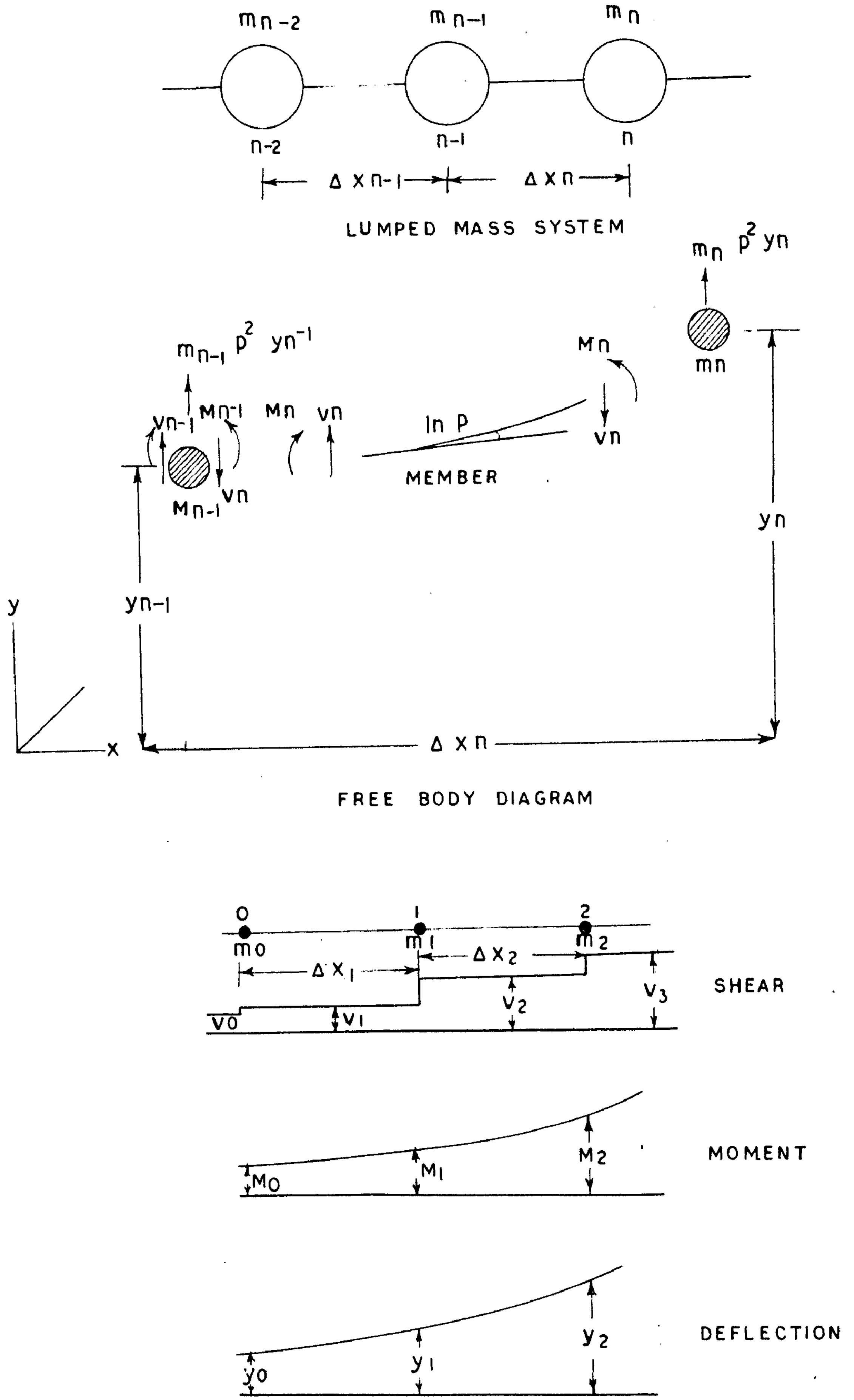


FIG.3·2_ DISCRETE SYSTEM FOR DYNAMIC ANALYSIS

For such a discrete system, if p is the natural frequency of vibration, the values of moments, shears and deflections at an nth point mass in terms of an (n-1)th point mass are given by following formulae [24].

$$V_n = V_{n-1} + m_{n-1}\, p^2\, y_{n-1} \tag{3.3}$$

$$M_n = M_{n-1} + V_n(\Delta x)_n - (\rho I)_n(\Delta x)_n p^2(\theta_b)_{n-1} \tag{3.4}$$

$$\theta_{bn} = \left(\frac{\Delta x}{EI}\right)_n \left[\frac{M_{n-1}}{2} + \frac{M_n}{2}\right] + (\theta_b)_{n-1} \tag{3.5}$$

$$y_{bn} = \left(\frac{\Delta x}{EI}\right)_n \left[\frac{M_{n-1}}{3} + \frac{M_n}{6}\right](\Delta x)_n + (\theta_b)_{n-1}(\Delta x)_n + (y_b)_{n-1} \tag{3.6}$$

$$y_{sn} = y_{sn-1} - \left(\frac{\Delta x}{\mu\, AG}\right)_n V_n \tag{3.7}$$

$$y_n = y_{bn} + y_{sn} \tag{3.8}$$

where,

V = shear force

M = Moment

θ_b = Bending slope

y = Total deflection

y_b = Bending deflection

y_s = Deflection due to shear

p = circular natural frequncy in radians per second

Δx = length of the segment

I = moment of inertia about neutral axis

ρ = mass density

β = shape factor

A = area of cross section

G = modulus of rigidity

n, n-1 = the mass point numbers

Thus, if the values of shear, moment, slope and deflection at a particular section for any frequency p are known, it is possible to find out the corresponding quantities at the next section by using equations (3.3) to (3.8). At the base, the dam is considered to be fixed. Hence the slope θ_b and deflections y_b and y_s are zero.

To arrive at the circular natural frequency of free vibrations of a dam a trial and error procedure is used. A trial value of p say p_1 is chosen. A close guess to the fundamental natural frequency can be made by using the Rayleigh's principle[25]. It is first assumed that a shear V' (it can be taken unity for convenience) and zero moment exists at the base and the shear and moment at the free end are evaluated. Let these be B_1 and B_2. It is further assumed that a moment M' (it can be taken unity for sake of convenience) and zero shear exists at the base and the shear and moment at the free end are again evaluated. Let these be B_3 and B_4. Now, if V_b and M_b are the actual shear and moment at the base, then the corresponding quantities at the free end are given by

$$V_o = \frac{B_1}{V'} V_b + \frac{B_3}{M'} M_b \qquad (3.9)$$

$$M_o = \frac{B_2}{V'} V_b + \frac{B_4}{M'} M_b \qquad (3.10)$$

Now, if the value of p is such that it coincides with one of the natural frequencies of the system, then V_o and M_o would both be equal to zero. That is the determinant

$$\Delta = \begin{vmatrix} \frac{B_1}{V_1'} & \frac{B_3}{M'} \\ \frac{B_2}{V'} & \frac{B_4}{M'} \end{vmatrix} = 0 \qquad (3.11)$$

In general, it would not be possible to guess the value of p correctly. However, various values are arbitrarily assigned to p and the value of the determinant Δ calculated. Any two successively chosen values of p, for which Δ changes sign gives one correct value of frequency to lie between them. By plotting the values of p^2 versus Δ would give a curve as shown in Figure No. 3.3

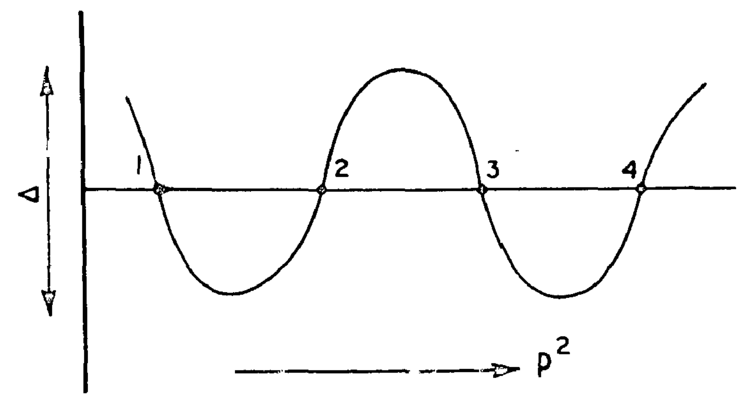


FIG. NO. 3·3

The correct values of p^2 are those which corresp ond to intersec tion of the curve with the p^2 axis. By interpolation a more correct value of the frequency is assumed and the procedure repeated till the determinant practically vanishes. Thus, a correct value of the frequency is obtained and the corresponding deflected shape of the system gives the mode shape.

Thus the various values of the natural frequencies and mode shapes of the dam are obtained. The next step is to know the responses of the dam to the earthquake excitations. These responses may be, shear, moment, deflections etc . If the precise waveform of a suitable accelerogram at the site of the dam is available, then the response of the dam can be obtained by using mode superposition technique. However, such techniques are usually more time consuming and are not generally necessary in order to get at the approximate behaviour of the dam in the beam analysis. Response in each mode is obtained by using the response spectra of the earthquakes. This is obtained more quickly as the response spectra of the past earthquakes are available in literature[26]. In the absence of the actual accelerograms at the site of the dam average spectra[27,1] can be used. Knowing the responses in each mode, the total probable response is obtained by using the root mean square criterion of combining the responses in the various modes. The responses obtained are dynamic deflections, dynamic moments, and dynamic shears. The expressions used for calculating the response in each mode are

Displacement $$Z_i^{(r)} = \phi_i^{(r)} \, v_r (Sd)_r \qquad (3.12)$$

Load $$Q_i^{(r)} = m_i \, p_r^2 \, Z_i^{(r)} \qquad (3.13)$$

Shear $$V_i^{(r)} = \sum_{j=1}^{i} Q_j \qquad (3.14)$$

Moment $$M_i = \sum_{j=1}^{i} V_j \Delta x_j \qquad (3.15)$$

and
$$v_r = \frac{\sum_{j=1}^{n} m_j \phi_j^{(r)}}{\sum_{j=1}^{n} m_j \left(\phi_j^{(r)}\right)^2} \qquad (3.16)$$

where

ϕ_j = mode shape coefficient

m_i, m_j = discrete masses

p = circular natural frequency

i is to be counted from the top of the dam

Δx_j = distance between the discrete masses

n = number of discrete masses

v_r = mode participation factor

S_d = spectral displacement

r = mode number

The total probable response is obtained by

$$AT = \sqrt{\sum_{r=1}^{k} \left[A^{(r)} \right]^2} \qquad (3.17)$$

where,

AT = Total probable response of quantities like shear, moment and displacement

$A^{(r)}$ = the response corresponding to a quantity in the rth mode

k = number of modes to be chosen.

Since it is known that it is the contribution of the first few modes that is necessary to combine the responses of all the 'n' modes of the dam. The number of modes to be combined is usually selected so that the highest frequency of the dam is of the order of 20 cycles/sec. or the shortest period is of the order of 0.05 seconds.

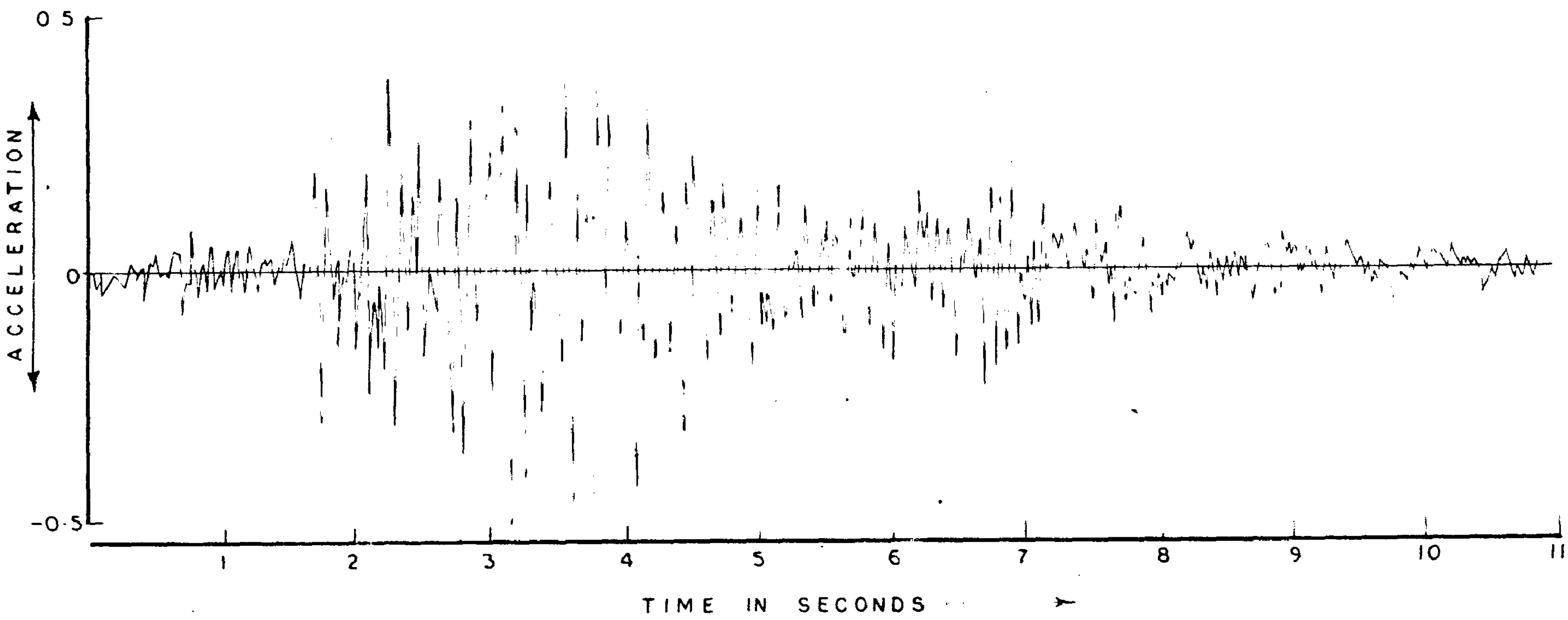
3.3 RESPONSE SPECTRUM ANALYSIS

The peak responses of structural system (assumed to be linear single degree of freedom systems) when plotted against the natural periods of vibration for various values of damping are known as response spectra. These responses may be the maximum displacement of the mass with respect to the base (termed as the spectral displacement Sd), or the maximum velocity of the mass relative to the base (termed as spectral velocity S_v), or the maximum absolute acceleration of the mass (termed as spec-tral acceleration s_a) . The response spectra are usually obtained by using numerical techniques with the help of high speed digital computer. Such techniques are well known[29,31]. For illustration the recorded accelerogram of the Koyna Earthquake of December 11, 1967 are given in Fig. 3.4 . The response spectra obtainedby numerical techniques from the above accelerogram[31] are given in Fig. 3.5 .

For calculations of various responses of the dam due to ground motion if a recorded accelerogram is avail-able for the site in case of past earthquake, the same can be used. But such situation is rare. In such a case the average spec tra obtained by Housner[27] and adopted by the IS Codes[1,2,58] can be used. But, these spectra would not be useful for epicentral regions[33] . Suffic-ient data however is not available so as to facilitate recommendation of a set of standard spectra for epicentral regions. The average spectra as given by Housner can be used only for the site which is about 20 miles or more from the epicentre[33]. The values of damping can be determined experimentally.

3.4 DETAILD ANALYSIS USING FINITE ELEMENT METHOD

The stresses are calculated using bending theory in case of beam analysis which assumes a linear



(a) HORIZONTAL COMPONENT TRANSVERSE TO DAM AXIS (KOYNA EARTHQUAKE)

FIG. 3.4 _ ACCELEROGRAM RECORDED AT BLOCK I-A OF KOYNA DAM ON DEC. 11, 1967

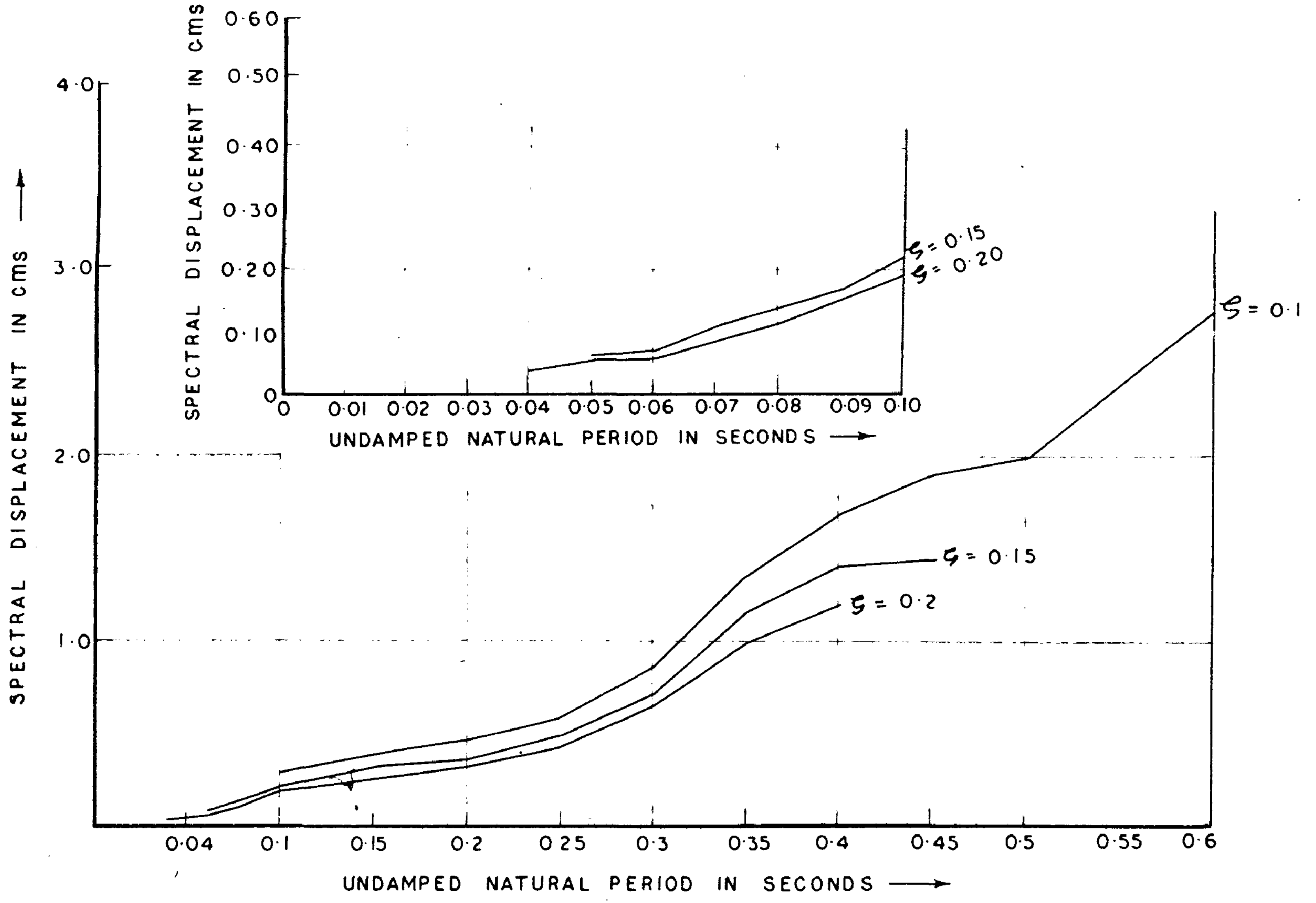


FIG. 3·5_ DISPLACEMENT SPECTRA DUE TO KOYNA EARTHQUAKE OF 11-12-67
TRANSVERSE COMPONENT

variation of stresses across the width of the dam. But, in actual practice the stress distribution is different than linear because the cross sectional dimensions of the dam are significant in comparison to its height. Therefore the beam analysis gives only the approximate distribution of stresses in the dam. But, to get an accurate representation of stresses in the dam it is necessary to analyse it necessarily as a two dimensional structure. This is done by using finite element method.

The detailed dynamic analysis using finite element method is a well known technique[21,25,34,35,36,38,40]. In this analysis first the dam structure is reduced to a discrete system by dividing the dam cross sec-tion into a number of finite elements. The finite elements may be either triangular, rectangular or quadrilateral. Several finite element models are available for analysing the systems representing plane behaviour[41,42]. Generally, for greater accuracy, a finer mesh which in turn requires a large number of finite elements is used. The more the finite elements the more will be the degrees of freedom of the system. This requires a large computational time and memory of the computer. Two types of finite element idealisation of homogeneous earth dam one with rectangular elements predominant and the other with only triangular elements[43] were studied. It was shown that the use of combination of rectangular elements in finite element idealisation is better in comparison to the use of triangular elements alone, as for the same accuracy, the number of degrees of freedom in the former case is much less than those in the latter case. This ultimately results in saving of memory space and computer time. As regards the coarseness or fineness of finite element mesh, it may be stated here that for evaluation of forces a relatively coarser mesh would be sufficient[43]. But, for the evaluation of stresses a very fine mesh is necessary[44].

In this method the vertical as well as the horizontal component of the ground motion can be taken into account. The equation of motion for free undamped vibrations is solved to get the natural frequencies and mode shapes. Knowing these quantities the response due to particular ground motion can be obtained by using the mode superposition method[35,40].

3.5 DYNAMIC TESTING AND ITS DEFICIENCIES

The dynamic behaviour of the concrete gravity dam can also be studied experimentally using a shake table. Free vibration tests are carried out to determine the fundamental natural period of vibration and mode shapes. Forced vibration tests are carried out to determine the acceleration response and to find the weak zones of the section. So far, the dynamic testing for the models of the non-overflow section of Kolkowadi dam (masonry)[50] and Koyna dam[51] have been carried out.

The theory of model analysis is presented. The pertinent variables involved in the model testing are -

L Linear dimension of the dam (L)

H Height of the dam (L)

y Displacement of the dam (L)

w Weight density of dam material (FL^{-3})

E Modulus of elasticity of dam material (FL^{-2})

a Acceleration applied to the dam (LT^{-2})

g Acceleration due to gravity (LT^{-2})

t Time period of dam (T)

P Force applied to the dam (F)

σ Stresses in the dam (FL^{-2})

The dimensions of the various variables are indicated in the brackets. Their physical quantities can be grouped into dimensionless terms and the prediction equation can be written as

$$\frac{Y}{L} = f\left(\frac{H}{L}, \frac{a}{g}, \frac{gt^2}{L}, \frac{wL}{E}, \frac{\sigma}{E}, \frac{P}{EL^2}\right) \qquad (3.18)$$

For a true structural model, for studying any quantity of interest, the remaining dimensionless terms must be the same for model and prototype. If this condition is satisfied, it will be possible to predict the behaviour of the prototype from the observed, behaviour of the model. However, it is practically difficult to make the dimensionless term $\frac{wL}{E}$ same for the model and the prototype which requires that

$$\left(\frac{wL}{E}\right)_{model} = \left(\frac{wL}{E}\right)_{prototype} \qquad (3.19)$$

or

$$w_m = w_p \frac{E_m}{E_p} \cdot \frac{Lp}{L_m} \qquad (3.20)$$

or

$$w_m = w_p \frac{E_m}{E_p} \quad q \qquad (3.21)$$

Where m denotes the quantity pertaining to model and p denotes the quantity pertaining to prototype, q is the scale ratio and equals L_p/L_m. Since the models are usually made smaller than the prototype the condition given by the equation 3.51 requires that the material of the model should have low elastic modulus and high density. This condition is usually not practicable with a large scale ratio. Hence by making observations on models, it is not possible to completely predict the prototype behaviour. However, even if the density condition is violated, it is feasible to study the dynamic characteristics such as the natural period of vibration, mode shapes and acceleration pattern. Further if the model material has low strength, it is possible to vibrate the model to such an extent that cracks are produced in the model which will be indicative of the weak sections of the model. Thus vibration tests are usually carried out on geometrically similar models with a limited objective.

For the dynamic testing of the model of the dam a shake table is used. The size of the shake table used for Kolkewadi dam[50] model testing was 2.0 m x 1.0 m. Keeping in view the size of the shake table a scale ratio is chosen. In case of Kolkewadi dam it was 100. The models are prepared of the mixture of plaster of paris and sand[50,51]. This mixture has the qualities of quick setting, low modulus of elasticity and small tensile strength. The proportions used in making models for Kolkewadi dam were, Plaster of Paris : sand : water as 1:10:3 . The steady state motion to the shake table is imparted with the help of an oscillator driven by a motor. The speed of the motor can be varied by a speed control unit. The accelerations of the model are picked up by an electronic amplifier and recorded on the pen recorder.

The free vibration tests are carried out to determine the fundamental natural period of vibration and mode shape. Acceleration pickups are fixed at selected points along the height of the dam. The model is excited by giving an initial displacement in the transverse direction and the resulting accelerations in the models are recorded on pen recorders. From the records so obtained the natural period of vibration can be obtained. The mode shape is obtained by normalising the accelerations with respect to the accelerations at the top of the model.

For computing the natural period of the prototype monolith from observed value on the model, the value of longitudinal wave velocity, $\sqrt{E/\rho}$, in the material is required. Here E is the modulus of elasticity of the material and ρ is the mass density of the material. The longitudinal wave velocity for the dam material can be assumed at 3048 m/sec. (10,000 ft/sec) as has been assumed for Kolkewadi dam, Koyna dam(50,51). For the model material, the longitudinal wave velocity can be worked out experimentally by vibrating vertical prismatic cantilevers.

The relationship between the natural period of the prototype and the model is as follows :

$$T_p = \frac{\sqrt{E_m / \rho_m}}{\sqrt{E_p / \rho_p}} \cdot q \cdot T_m \qquad (3.22)$$

where T_p is the natural period of vibration of prototype dam and T_m is that of the model, q is the model scale ratio. The computed natural periods of vibration for Koyna and Kolkewadi dam show good agreement with the values of the theoretically calculated periods(50,51). Similarly the fundamental mode shape obtained by theoretical analysis shows a good comparison(50).

Steady state forced vibration tests are performed to study the acceleration response and to find out the weak zones of the section. Continuous records of the shake table accelerations and the accelerations at the top of the model are obtained. The speed of the oscillators is increased till the first crack occurs. After the appearance of the first crack, the shaking is continued till further cracks develop and overtoppling of the cracked portion occurs. A comparison of the acceleration at the top of the model at first crack as obtained experimentally with the theoretically predicted acceleration at the top of the model shows good agreement between the two[50].

A comparison of the model before and after the steady state forced vibrations tests show that the cracks occurred at the neck where the downstream slope changes abruptly[50,51]. Cracking was expected at this level as the stresses calculated by theoretical techniques were higher than the tensile strength of the material of the dam. Thus, the dynamic testing is an effective method to check the adequacy of the theoretical techniques of vibration analysis of dam.

However, there are limitations in the application of the dynamic testing to predict the behaviour of the prototype from observations on models. It will be clear from the discussion that follows that it will not be possible, in prac tice, to satisfy all the requirements of similitude. Due to impossibility of simulating all modelling conditions and also due to difficulties in reproducing earthquake type of excitations in the laboratory, experimental studies give results which are basically qualitative in nature, however , the behaviour of the model observed experimentally can be compared with its own theoretical behaviour. If the theoretical and experimental behaviour of the model itself compare favourably, the adequacy of the theoretical methods of analysis gets proved.

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CHAPTER - IV

STUDY OF INDIAN STANDARD CODE PROVISIONS AND COMPARISON WITH DYNAMIC ANALYSIS

4.1 GENERAL

The Indian Standard code for Earthquake Resistant Design of structures is under constant revision (1966, 1970, 1974). The dynamic moments and shears calculated on the basis of recommendations of these codes when compared with the dynamic moments and shears as obtained from dynamic analysis would definitely indicate whether these revisions are rational or otherwise. A simplified procedure for dynamic analysis is developed after carrying out dynamic analysis of a number of practical profiles of gravity dams subjected to three strongest components of recorded ground motion[55]. Average curves for determining the fundamental natural period of dam, dynamic moments and shears along the height of the dam are proposed from which knowing the geometry of the dam profile and the material properties and using selected response spectra these quantities can be easily calculated. The dynamic moments and shear as per the IS code provisions and as per dynamic analysis are compared with each other.

The IS code of 1974 specifies use of two approaches to calculate the dynamic moments and shears viz, the seismic coefficient method for dams upto 100 m height and the response spectrum approach for dams over 100 m height.

In case of seismic coefficient method the basic seismic coefficient α_o is specified for each of the seismic zones in which the country is divided. The value of horizontal seismic coefficient α_h may be taken as

$$\alpha_h = 1.5 \ \beta \ I \ \alpha_o \qquad (4.1)$$

where β is a coefficient depending upon soil-foundation system may be taken as 1 for dams. I is a coefficient depending upon the importance of the structure may be taken as 2 for dams. Thus for dams the horizontal seismic coefficient at top of dam works out to

$$\alpha_h = \beta \ \alpha_o \qquad (4.1a)$$

The value of horizontal seismic coefficient at bottom is zero a linear variation from top to bottom is proposed.

In the response spectrum method the IS code of 1974 specifies that the horizontal seismic coefficient may be taken as

$$\alpha_h = \beta \ I \ F_o \frac{S_a}{g} \qquad (4.2)$$

$$= 2 \ F_o \frac{S_a}{g} \qquad (4.2a)$$

where β and I have same meaning as explained earlier F_o is the seismic zone factor for average acceleration spectra and is specified for each of the seismic zones S_a/g is the average acceleration coefficient and can be read from the acceleration spectra given in IS code (1974) for appropriate natural period and damping of the structure.

The calculations of dynamic shear and moment can be done easily in case of seismic coefficient method. For the Response spectrum method the IS code of 1974 specifies the following formula for calculating the fundamental period of vibration of the dam :

$$T = 5.55 \frac{H^2}{D} \sqrt{\frac{W_m}{g\ E_m}} \qquad (4.3)$$

where H is the height of the dam in meter, D is the base width of dam m, E_m the modulus of elasticity of the dam material, Kg/m^2, W_m the unit weight of the dam material kg/m^3, and g is the accelor tion due to gravity in m/sec^2. Using the period as calculated by equation 4.3 above and a damping of 5 % the design horizontal seismic coefficient α_h can be calculated from equation (4.2a).

The base shear and moment (V_B and M_B) respectively can be calculated by following formulae

$$V_B = 0.6\ W\ \alpha_h \qquad (4.4)$$

$$M_B = 0.9\ W\ \bar{H}\ \alpha_h \qquad (4.5)$$

where W is the total weight of the dam, in kg, $\bar{H}$ the height of centre of gravity of the dam above the base in m, and d_h as already defined.

For any horizontal section at depth y , below the top of the dam the shear V_y and the bending moment M_y may be obtained as follows .

$$V_y = C_v' \quad V_B \qquad (4.6)$$

$$M_y = C_m' \quad M_B \qquad (4.7)$$

The values of C_v' and C_m' are given in a plot corresponding to various y/H ratios.

In the simplified[55] procedure for dynamic analysis certain graphs are given from which knowing the material properties of the dam and the details about the cross-section of the dam the fundamental natural period of vibration, can be calculated. Knowing the fundamental period of vibration of the dam, using the selected acceleration spectra the S_a/g can be read from same corresponding to specified damping. Further from the average curves given for the dynamic moment and shear coefficients at the bottom (C_{mb} and C_{vb}) can be read for the dam height and total slope. Further from the average distribution curves for moments and shear the value of non dimensional moment and shear coefficients $\alpha(x)$ and $\beta(x)$ can be read from these , the values of moments and shears at any point x above the base of the dam can be calculated as follows:

$$M(m) = \alpha(m)\, C_{mb} \,.\, W.H. \frac{(S_a)}{g} \qquad (4.8)$$

$$V(m) = \beta(m) \,.\, C_{vb} \,.\, W. \frac{S_a}{g} \qquad (4.9)$$

where W is the total weight of the dam , H is the total height of the dam S_a/g is the acceleration coefficient in the fundamental mode of vibration of the dam.

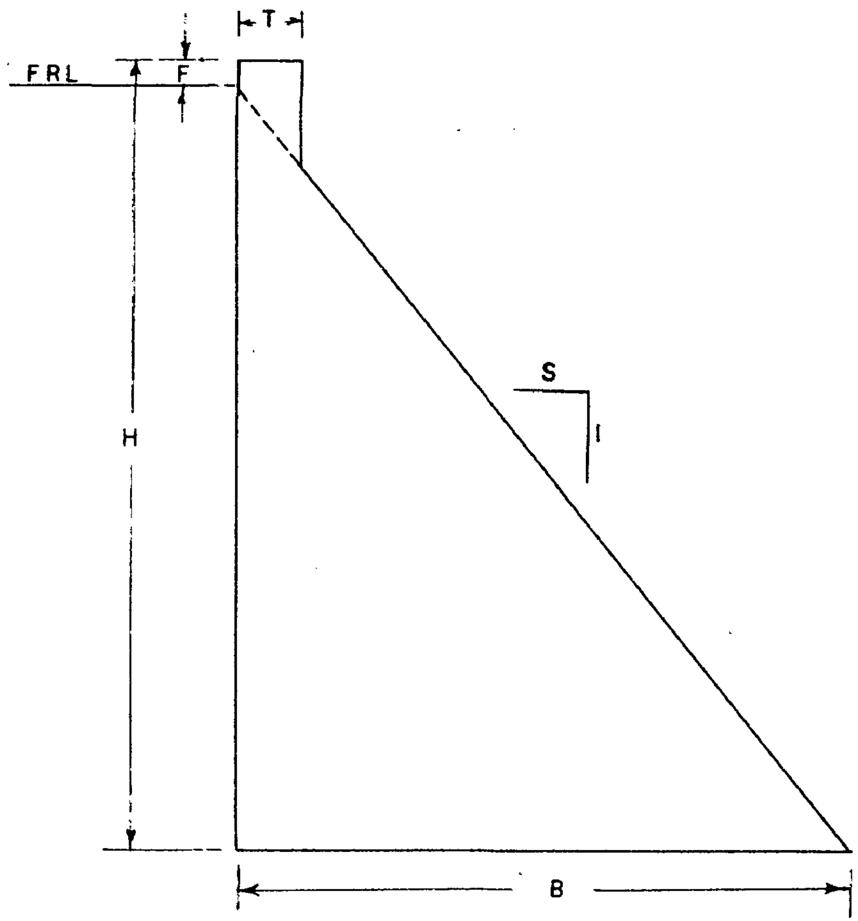
4.2 SELECTION OF DAM SECTIONS

For the sake of comparison of the dynamic moment and shear distribution as per the provisions of the IS codes revised from time to time (1966, 1970 , 1974) four dam cross-sections with upstream face vertical and downstream face with a slope of 0.8 :1 and height 50 m, 100m , 150 m and 200 m are selected as shown in Fig. 4.1. The dynamic moment and shear coefficients at various point below the dam are calculated. These values are expressed in non-dimensional form. The values of dynamic moments and shears at any point can be expressed in terms of following in case of IS codes of 1966 and 1970.

$$M_y = \alpha(y) \,.\, W.\bar{H}\, \alpha_0 \qquad (4.10)$$

$$V_y = \beta(y) \,.\, W.\, \alpha_0 \qquad (4.11)$$

The values of moment and shear can be expressed in case of IS code of 1974 where the response spectrum approach is proposed and in case of dynamic analysis-



DIMENSIONS OF PARAMETERS

SECTION NO.	H METRES	S	T METRES	F METRES	B METRES
1	50	0·8	8·0	2·0	38·40
2	100	0·8	8·0	3·0	77·60
3	150	0·8	8·0	4·5	116·40
4	200	0·8	8·0	5·0	156·00

FIG. 4·1_ DAM PROFILES CHOSEN FOR COMPARISON OF VARIOUS CODE PROVISIONS

$$M_y = \alpha(y) \; W \bar{H} \; \frac{S_a}{g} \qquad (4.12)$$

$$V_y = \beta(y) \; W \; \frac{S_a}{g} \qquad (4.13)$$

4.3 CASE I DAM HEIGHT 50 m

The dam section has been divided into 10 segments as shown in Fig. 4.2 The dynamic moment and shear coefficients are plotted against the corresponding points along the height of the dam as per the provisions of IS Codes 1966 and 1970 vide Fig. 4.3 The dynamic shears as per the IS Code of 1974 would be the same as given by provisions of IS code of 1970 as the provisions in both the cases are exactly same. The dynamic shear coefficient in case of dynamic analysis can be calculated after determining the fundamental natural period of vibration of dam, the calculations of the same are presented in Table 4.9. The values of base shears coefficients (β_{x}) are read from the average curves proposed in dynamic analysis(55).

The relation between the dynamic shear coefficient $\beta(x)$ as read from curves of dynamic analysis and $\beta(y)$ as proposed here is as follows :

$$\beta(y) = \beta(x) \; . \; C_y b . \; 2 \qquad (4.14)$$

$$= 2 \times 0.5175 \; \beta(x)$$

$$= 1.035 \; \beta(x)$$

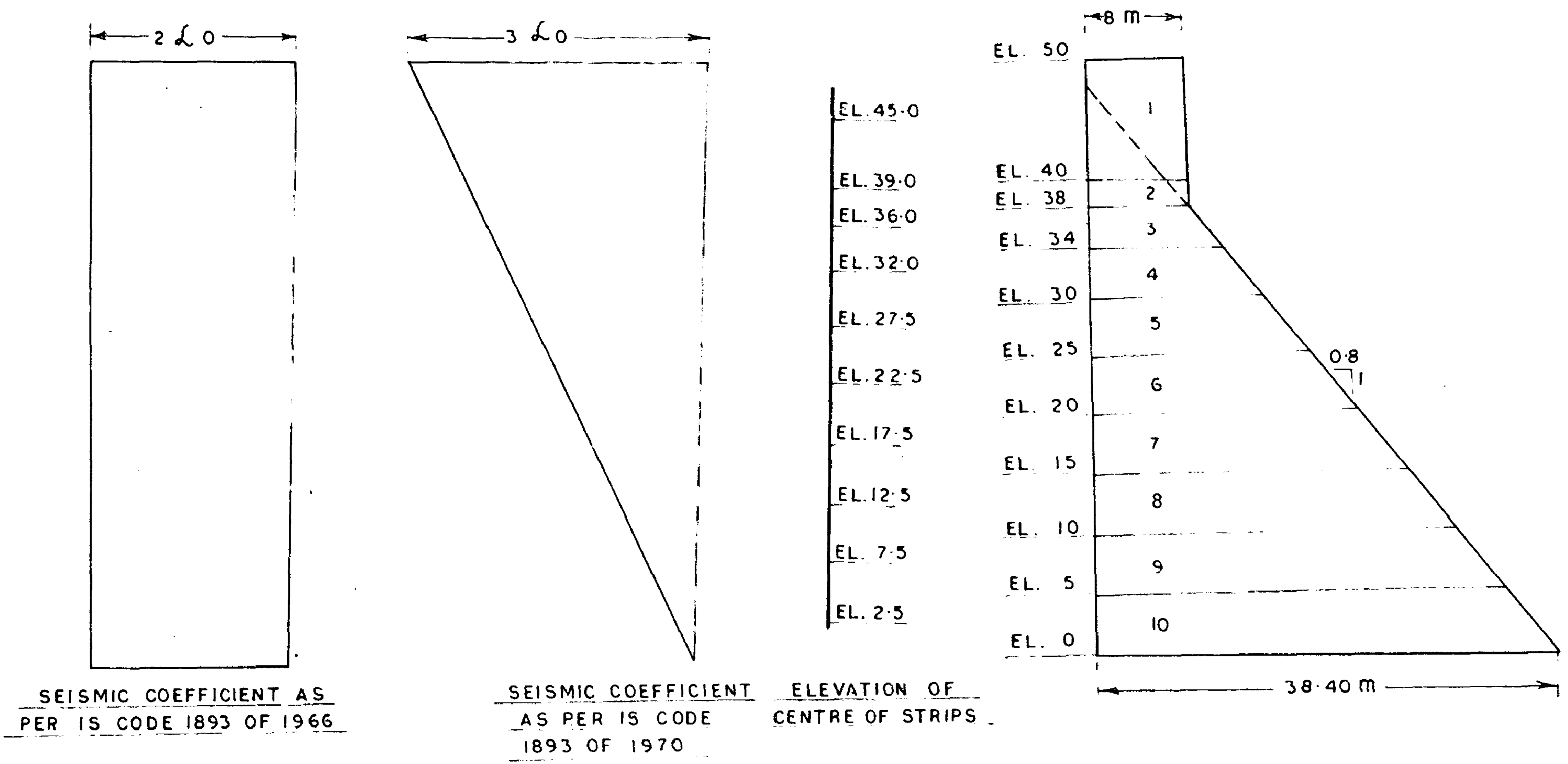


FIG. 4·2 _ DAM PROFILE 50m HEIGHT.

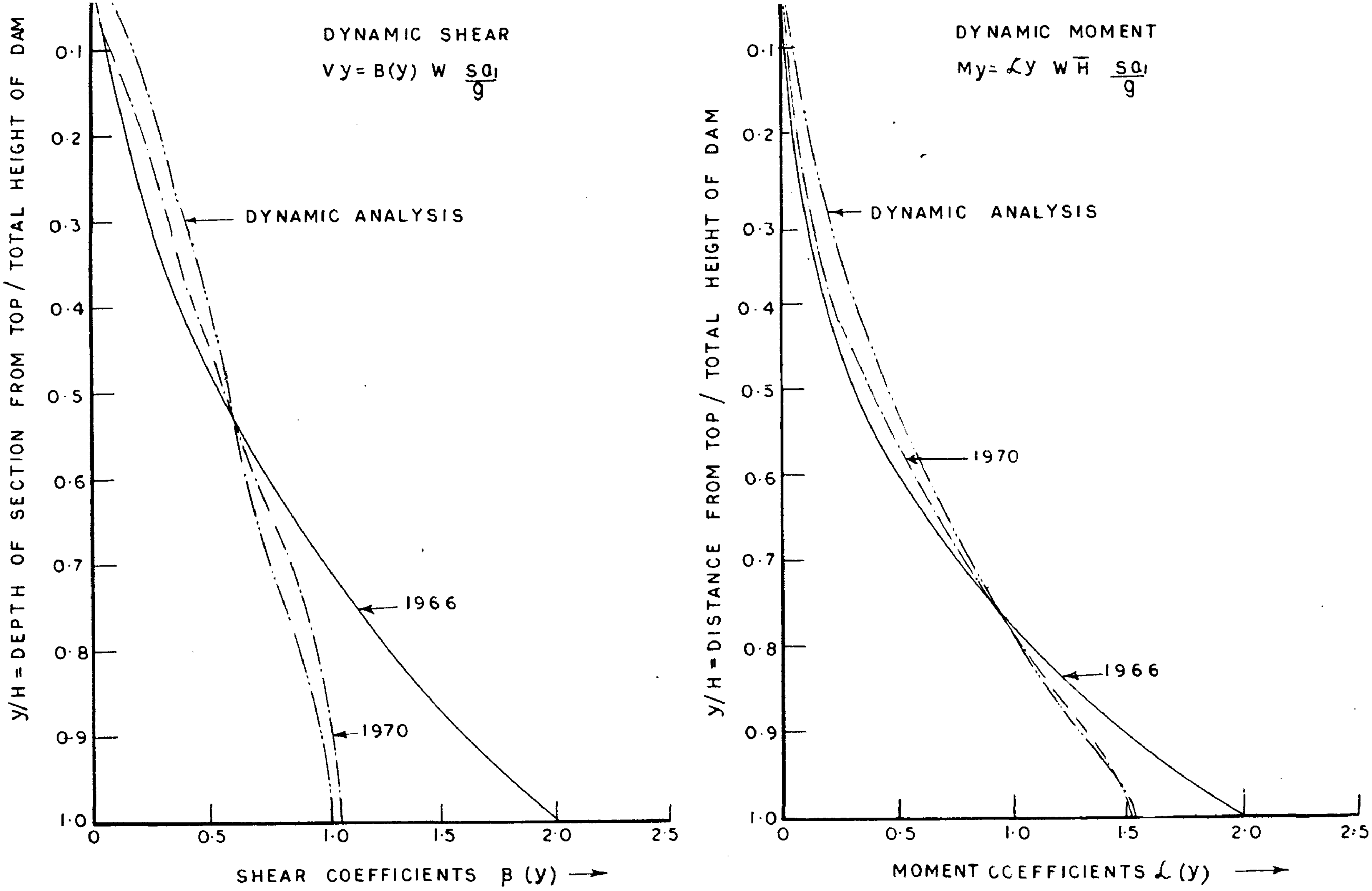


FIG. 4·3 _ SHOWING SHEAR AND MOMENT COEFFICIENT FOR A 50m HIGH DAM AS PER DIFFERENT IS CODES

The dynamic shear coefficients $\beta(y)$ are plotted in Fig.4.3. The values $C_y b$ are given vide Table 4.11 for all the cases.

Similarly the dynamic m moment coefficients in case of IS code of 1966 and 1970 are obtained and plotted in Fig. 4.3. The dynamic moment coefficients in case of IS code of 1974 would be the same as given by IS code of 1970. The value of dynamic moment coefficient $\alpha(y)$ would be related to the value $\alpha(y)$ as read from average curves in dyanmic analysis (55) as follows

$$\alpha(y) = \alpha(n)\ 2.\ C_{mb}.\ \frac{H}{\bar{H}} \qquad (4.15)$$

$$= \alpha(n)\ 2.\ 0.2725\ \frac{50}{17.60}$$

$$= 1.53\ \alpha(n)$$

The values of dynamic moment coefficients $(\alpha(y)$ are plotted in Figure 4.3. The value of C_{mb} are given in Table 4.11 for all the cases.

4.4 <u>Case II DAM HEIGHT 100 m</u>

The dam is divided into 11 segments as shown in Figure 4.4 . The dynamic shear coefficients calculations as per the IS Code provisions of 1966 and 1970 are given in table 4.1 and 4.2 respectively. The relation between the dynamic shear coefficient C_v' as given by IS Code of 1974 and the dynamic shear coefficient $\beta(y)$ is

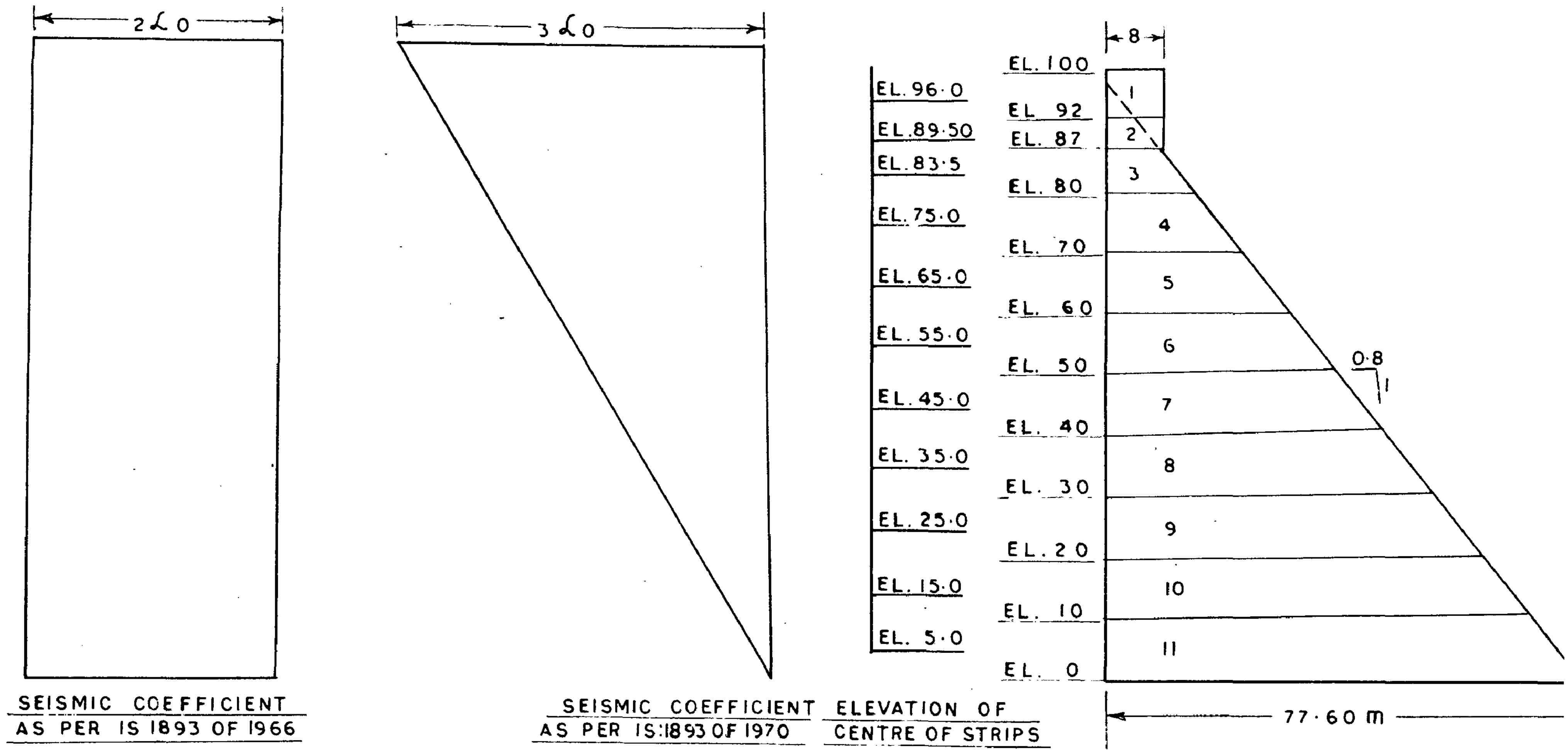


FIG. 4·4 _ DAM PROFILE 100 m HEIGHT

$$\beta(y) = 0.6 \times 2 \times C_v'$$

$$= 1.2\, C_v' \qquad (4.16)$$

The values of C_v' are read from curve given in IS code of 1974. The corresponding values of $\beta(y)$ are calculated vide table 4.3.

The dynamic shear coefficients $\beta(y)$ and those read from average curves in dynamic analysis are related as from equation 4.14

$$\beta(y) = 2 \times 0.5375\ \beta(x)$$

$$= 1.075\ \beta(x) \qquad (4.17)$$

These values are given in Table 4.4.

Similarly the values of dynamic moment coefficients are calculated as per IS codes of 1966 and 1970 and are presented in Tables 4.5 and 4.6 respectively. The relation between dynamic moment coefficient $\alpha(y)$ and the one C_m' as read from curves given in IS code of 1974 is as follows

$$\alpha(y) = 2 \times 0.9 \times C_m'$$

$$= 1.8\, C_m' \qquad (4.18)$$

The values of C_m' are read from curve in IS Code of 1974 and corresponding values of $\alpha(y)$ are calculated vide Table 4.7.

The value of dynamic moment coefficient $\alpha(y)$ is related to $\alpha(x)$ in case of dynamic analysis by equation 4.15 which works out to

$$\alpha(y) = 2 \times 0.275 \times \frac{100}{33.6} \times \alpha(x)$$

$$= 1.635\ \alpha(x) \qquad (4.19)$$

These values are given in Table 4.8.

The values of dynamic moment coefficients as per the various IS Code provisions and as per dynamic analysis are plotted in Fig. 4.5.

4.5 CASE III DAM HEIGHT 150 m

In this case the dam profile is divided into 12 segments vide Fig. 4.6.

The dynamic shear coefficient $\beta(y)$ and that given by dynamic analysis $\beta(x)$ are related in this case using equation 4.14 as follows .

$$\beta(y) = 2 \cdot 0.525 \times \beta(x)$$

$$= 1.05\ \beta(x) \qquad (4.20)$$

The dynamic moment coefficients as per the various IS codes (1966, 1970 , 1974) are plotted in Fig.4.7. The dynamic moment coefficient $\alpha(y)$ and $\alpha(x)$ read from curves in dynamic analysis are related by equation 4.15

$$\alpha(y) = 2 \times 0.275 \times \frac{150}{48.4} \times \alpha(x)$$

$$= 1.709\ \alpha(x) \qquad (4.21)$$

The values of $\alpha(x)$ are read from curves given in dynamic analysis. The values of dynamic moment and shear coefficients

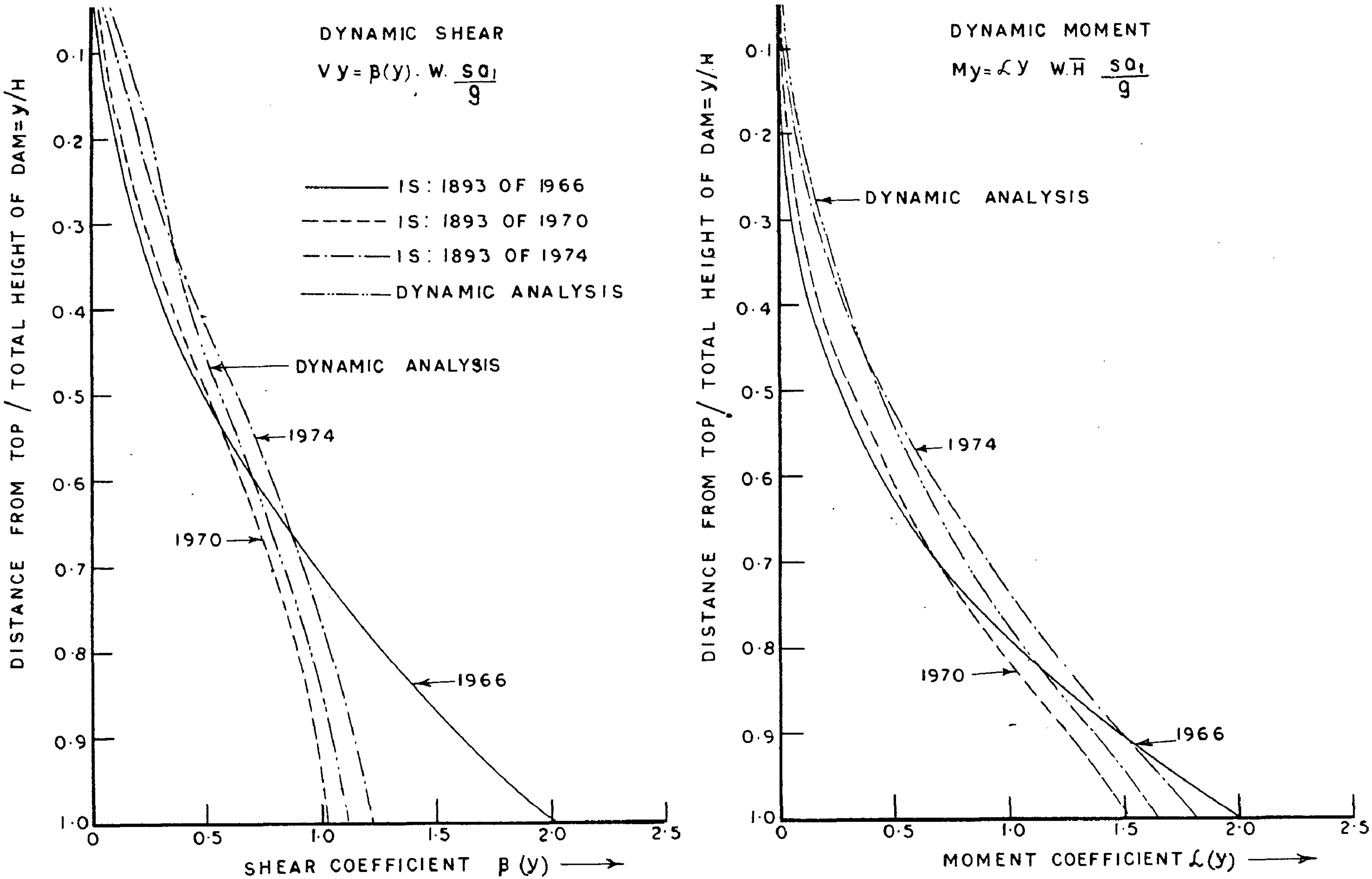


FIG. 4·5 — SHOWING SHEAR AND MOMENT COEFFICIENTS FOR A 100 m HIGH DAM AS PER DIFFERENT IS CODES.

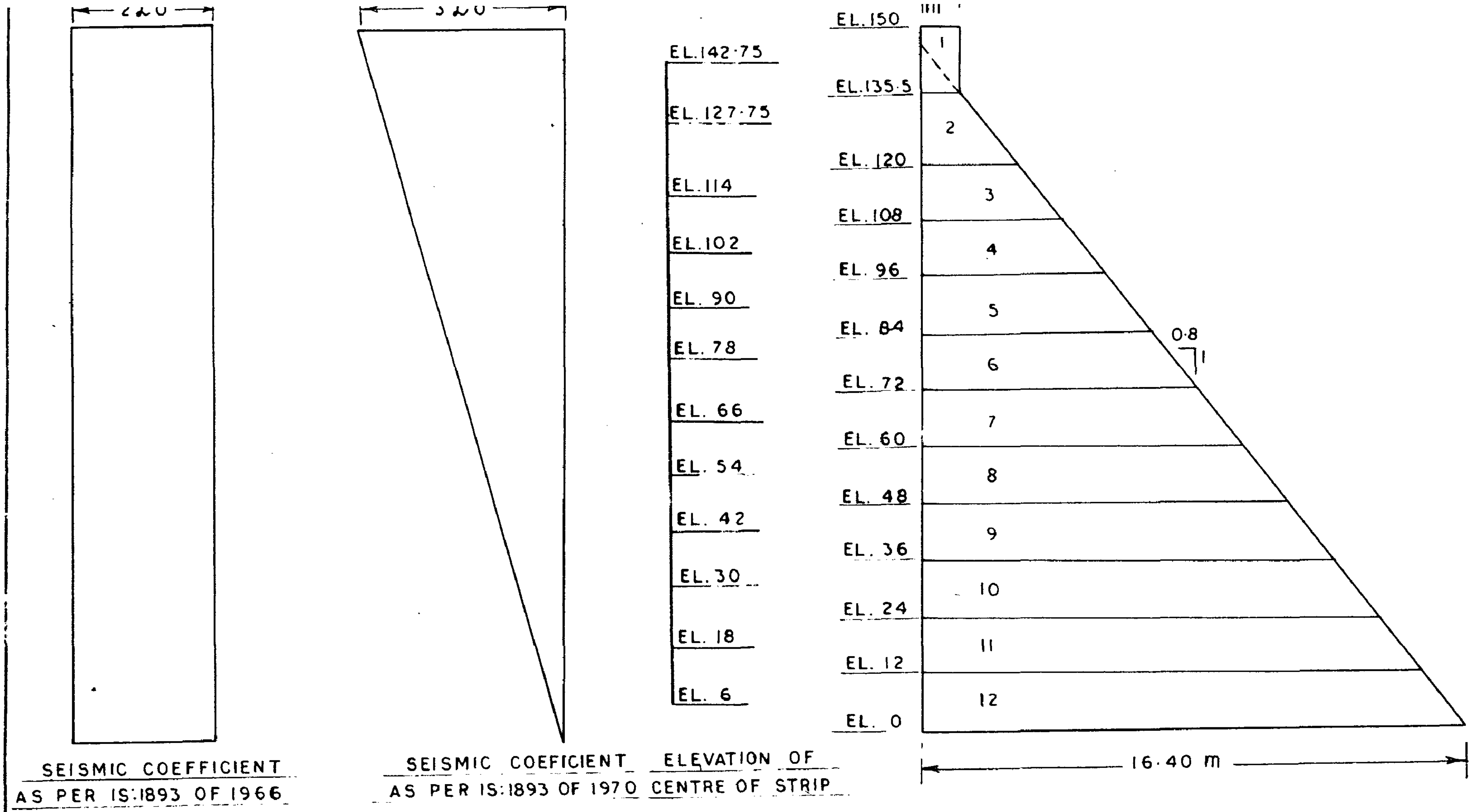


FIG. 4·6_ DAM PROFILE 150 m HEIGHT

are plotted in Fig. 4.7.

4.6 CASE IV DAM HEIGHT 200 m

For division of dam profile plane vide Fig. 4.8. The dynamic shear coefficients $\beta(y)$ can be calculated in case of dynamic analysis using equation 4.14 as follow

$$\beta(y) = 2 \times 0.56 \times \beta(x)$$

$$= 1.12 \ \beta(x) \qquad (4.22)$$

The dynamic moment coefficients $\alpha(y)$ can be calcul ted by using equation 4.19 as

$$\alpha(y) = 2 \times 0.28 \times \frac{200}{65.75} \times \alpha(x)$$

$$= 1.705 \ \alpha(x) \qquad (4.23)$$

The values of shear and moment coefficients are plotted in figure 4.9.

The figures 4.3, 4.5 , 4.7 and 4.9 would give the qualitative comparision between the dynamic shears and moments as per various IS code provisions and as per the dynamic analysis. But, to compare these coefficients in a quantitative manner it is necessary to bring them to a common basis. Here it is proposed to compare the base moment and shear coefficients on the basis of α_0 the basic seismic coefficient for the region. The values of base moment and shear coefficients are already expressed in

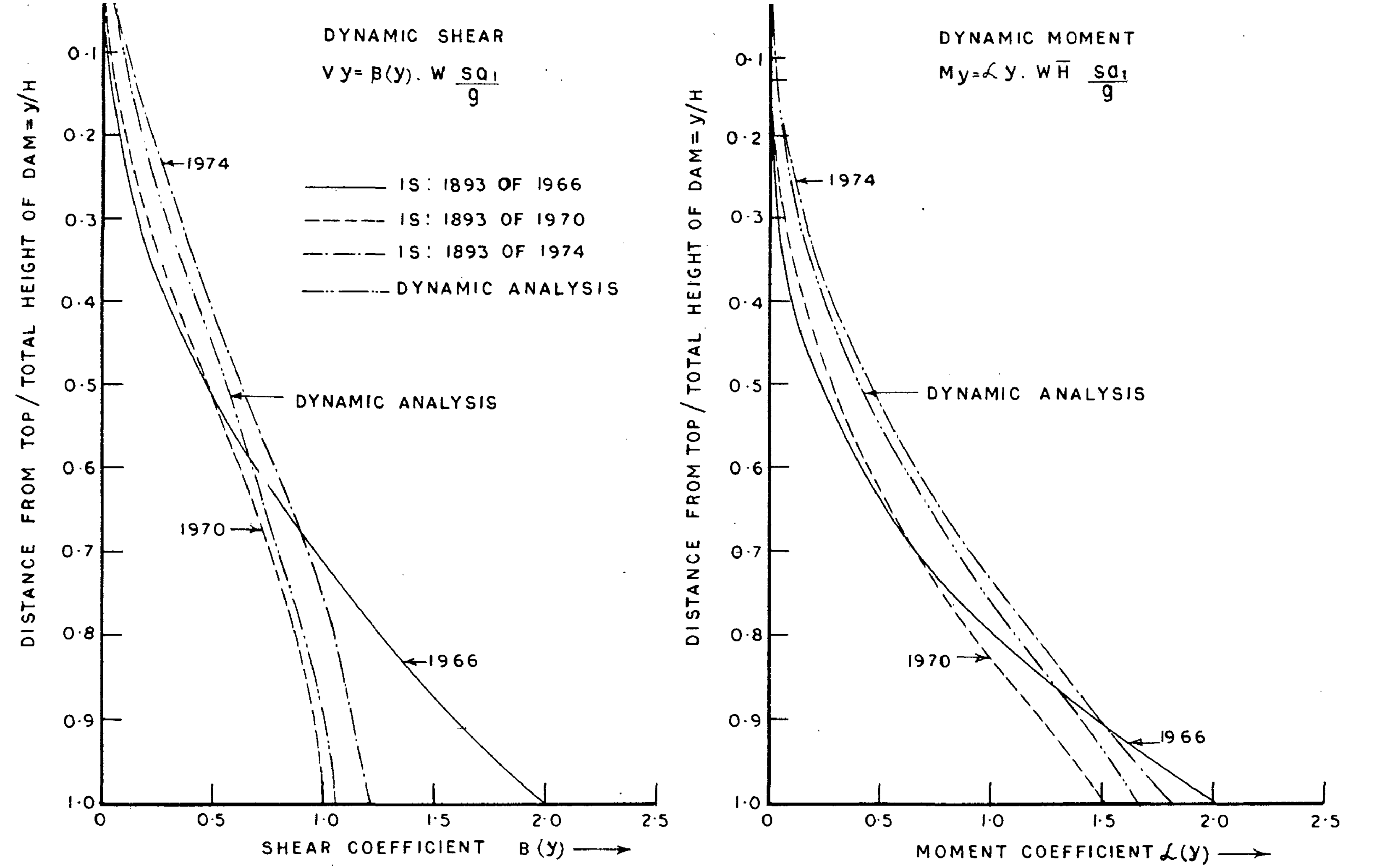


FIG. 4·7_ SHOWING SHEAR AND MOMENT COEFFICIENTS FOR A 150m HIGH DAM AS PER DIFFERENT IS CODES.

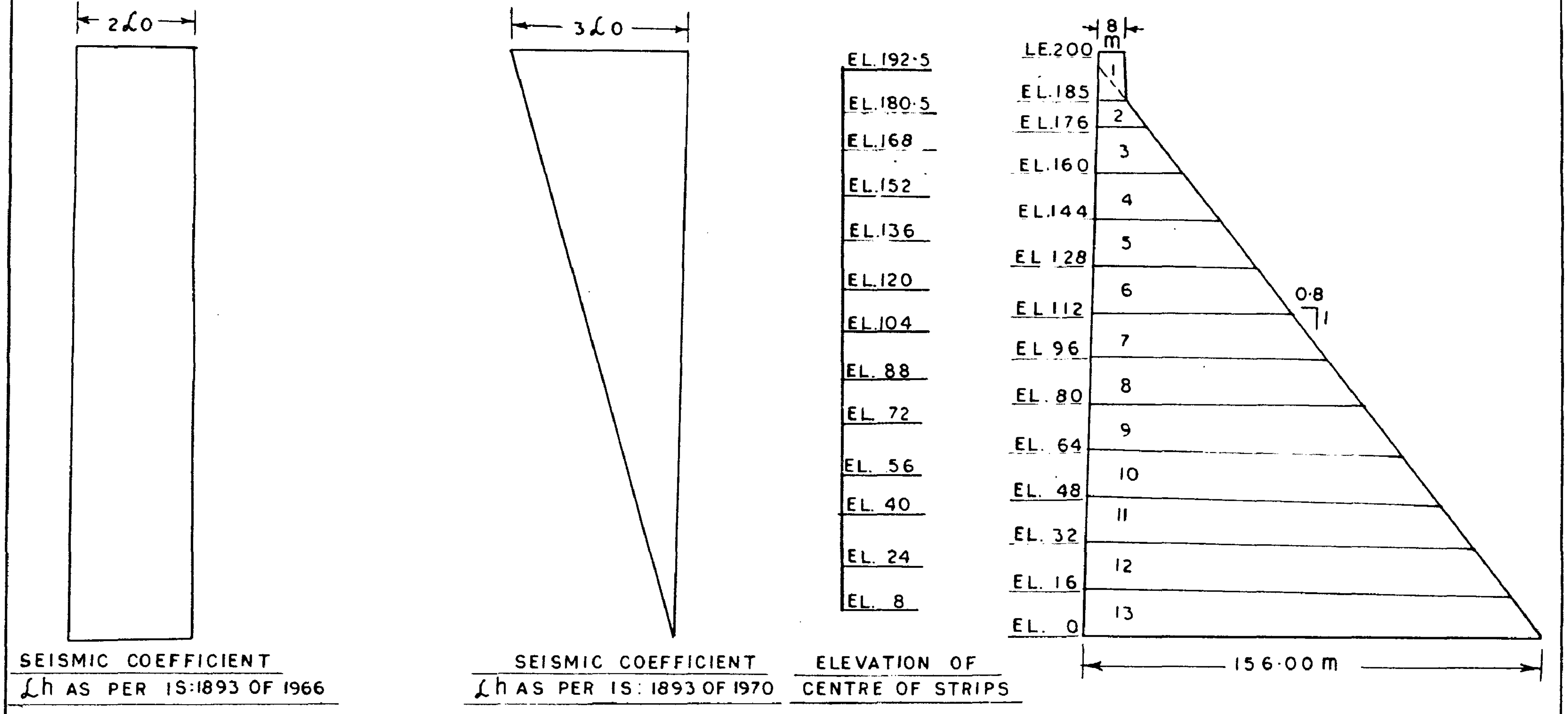


FIG. 4·8_ DAM PROFILE 200 m HEIGHT.

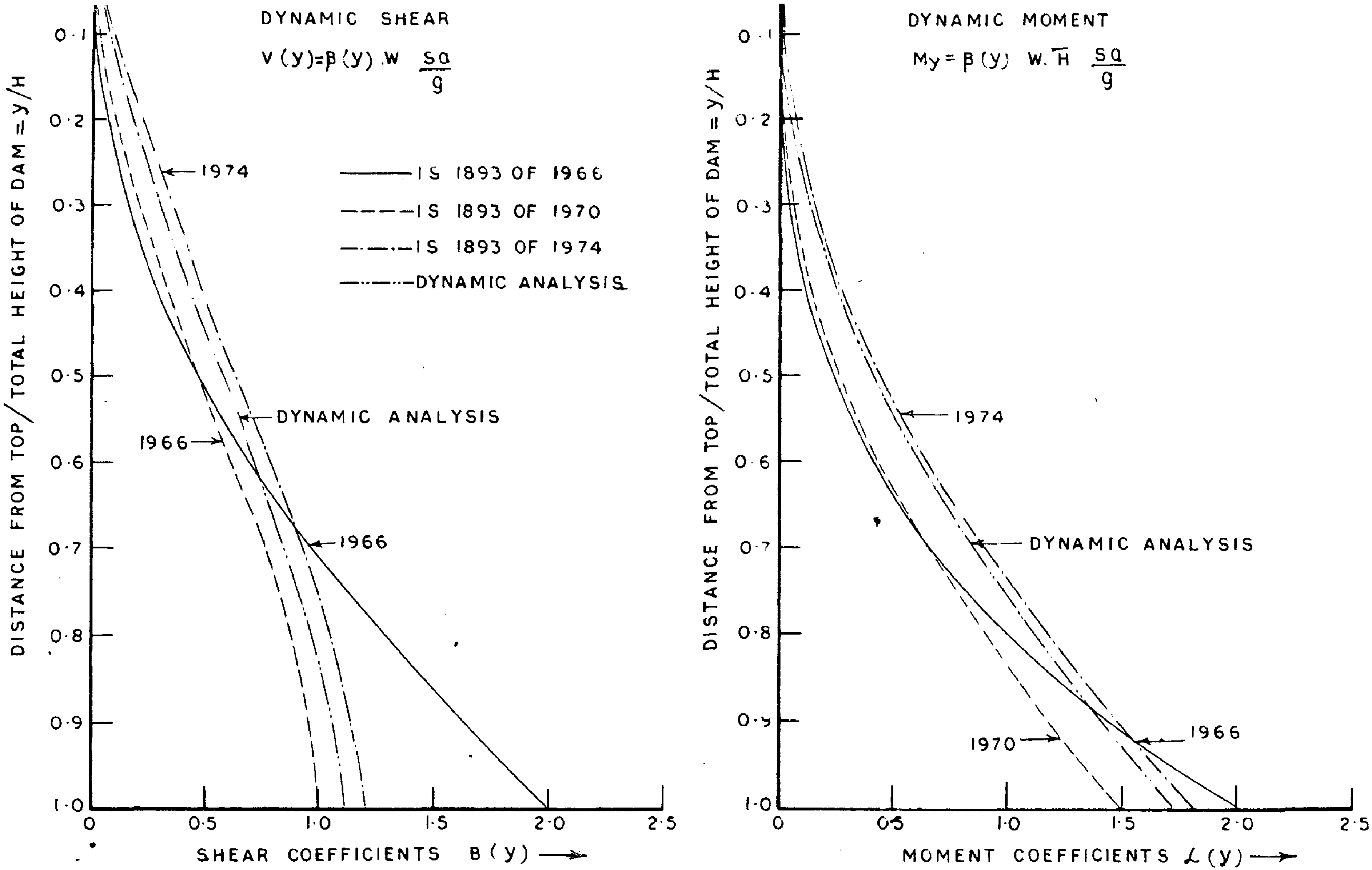


FIG. 4·9 _ SHOWING SHEAR AND MOMENT COEFFICIENTS FOR A 200 m HIGH DAM AS PER VARIOUS IS CODES.

terms of IS codes of 1966 and 1970. The values of base moments and shear coefficients in case of IS Code of 1974 and dynamic analysis would be expressed in terms of α_0. The period, of vibration of the dam in its fundamental mode in case of IS code of 1974 and those in case of dynamic analysis are given in table 4.9 and 4.11 respectively. The table 4.10 gives the various relations obtained in case of four cases considered for quantitative comparison of base moment and shear coefficients. The values of base moment and shear coefficient in four cases as per the provisions of various IS Codes and dynamic analysis are given in Table 4.12 below :

TABLE 4.12 DYNAMIC MOMENT AND SHEAR COEFFICIENTS AT BASE OF DAM FOR QUANTITATIVE COMPARISON

Sl. No.	Case	Base moment coefficient as per				Base shear coefficient as per			
		IS: 1893			Dynamic Analysis	IS: 1893			Dynamic Analysis
		1966	1970	1974		1966	1970	1974	
1	I	2.00	1.525	1.525	1.301	2.00	1.068	1.068	0.88
2	II	2.00	1.520	1.710	1.570	2.00	1.009	1.140	1.031
3	III	2.00	1.520	1.715	1.633	2.00	0.99	1.162	1.01
4.	IV	2.00	1.435	1.530	1.490	2.00	0.9875	1.020	0.98

4.7 OBSERVATIONS

It is observed from a perusal of figures 4.3, 4.5, 4.7

and 4.9 that the distribution of the dynamic moments and shears as per the IS code of 1966 is entirely different from those given by later codes (viz. 1970 and 1974) and dynamic analysis. Therefore the provisions of IS code of 1966 which specify a uniform seismic coefficient along the entire height of the dam are entirely inadequate for the calculation of moments and shears in the dam. The distribution curves for moments and shears as per the IS code of 1970 are definitely an improvement over the earlier code. But still the distribution does not follow same trend as given by that of dynamic analysis. The distribution curves as per IS code of 1974 follow very nearly the same trend as given by use of dynamic analysis. Therefore qualitatively, the IS code of 1974 gives the moment and shear distribution along the height of the dam in a more realistic manner and can be used with much more confidence for preliminary design of major dams located in seismically active areas . It is also further observed that the Response Spectrum Approach specified by IS code of 1974 for dams over 100 m height gives a better distribution of dynamic moments and shears which is more near to the realistic distribution as per the dynamic analysis. The two distributions tally very well as the height of dam increases. Thus the differentiation between the approaches to the seismic design of dams for low heights (below 100 m) where the IS Code (1974) specifies use of seismic coefficient method and for higher heights (above 100 m) where the Response spectrum is proposed, is realistic. There was however no such differentiation in the earlier IS Codes of 1966 and 1970.

The quantitative comparison of moments and shears at the base of the dam as per the various codes and as per dynamic analysis can be observed from Table 4.12 . It is seen that the dynamic moment and shear coefficients at the base are the highest in all the cases considered as per the IS code of 1966. The provisions of IS code 1966 overestimate the dynamic shears and moments at the base. The percentage of overestimation (over the dynamic analysis) in case of dynamic moment is 54, 27, 22 and 34 in Case I, II, III and IV respectively. Similarly the percentage of over estimation in the dynamic shears in corresponding cases is 127, 94, 98 and 104. Thus it can be seen that the provisions of IS Code of 1966 err very much on the higher side in both dynamic moment and shear at the base. They are therefore entirely inadequate. The IS Code provisions of 1970 are definately an improvement over the 1966 code as the difference between the shear and moment coefficients in this case is very small as compared to dynamic analysis. In case I the moments and shear at the base as per the provision of 1970 code are on higher side than those as given by dynamic analysis by approximately 17 per cent and 21 per cent respec tively. In rest of the three cases however these are a little on the lower side than those given by dynamic analysis. Thus the IS Code of 1970 which specifies a triangular variation of seismic coefficient along the height of the dam is an improvement in the right direction over the earlier code.

The provisions of IS Code 1974 are still a better improvement over the 1970 code provision. The values of dynamic moment and shear at base as given by the IS Code of 1974 are on lower side than the actual in case of case I. But, in all other cases they are a little on the higher side than those given by dynamic analysis. But, this overestimation can be tolerated because, there are so many uncertain factors on which the calculations of dynamic moments and shears due to earthquake is based. Therefore a code which gives a value of dynamic moment and shear coefficient on a little higher side would definitely be quite all right. The percentage of overestimation in dynamic moment and shear is of 9 and 12 per cent respectively in case 1. But these percentages are 3 and 4 respec-tively in case IV.

4.8 HYDRODYNAMIC PRESSURE

Due to the horizontal acceleration of the foundation there is an instantaneous hydrodynamic pressure exerted against the dam in addition to hydrostatic forces. The various IS codes (1966, 1970 , and 1974) specify Zangar's formulae for computing hydrodynamic pressures . It is proposed to verify the approximate formulae given to calculate the hydrodynamic shear and moment at any depth y below the reservoir as given by equations 2.3 and 2.4 in Chapter II. It is felt that the shear and moment coefficients of 0.726 and 0.299 respectively as given in IS codes may not be constant over the entire depth of the reservoir.

The pressure diagram of the hydrodynamic pressure P_e would be given in Figure 4.10 below.

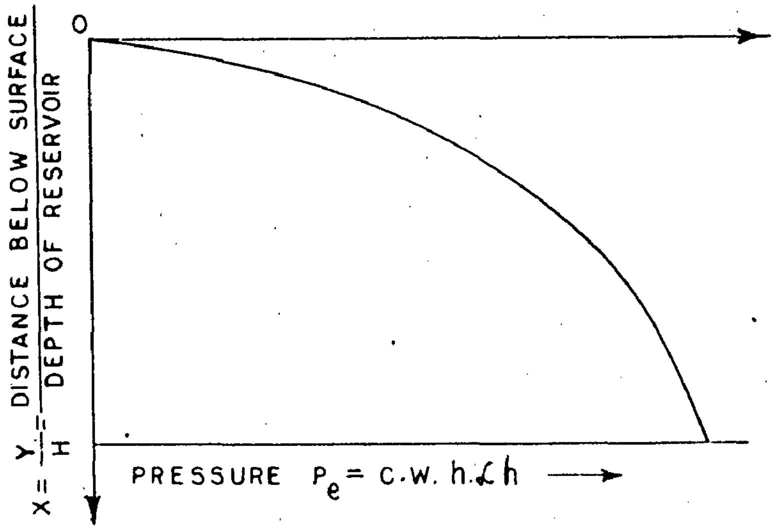


FIG. 4·10

The area under the pressure diagram at a particular level would give the shear V_e at that level and the moment of this area about the centre of gravity of the horizontal cross-section of the dam at that level would give the moment M_e. It is proposed to evaluate the area over a range of ratio $x = y/h$ and calculate the hy- dynamic shear coefficient $\beta_H(y)$. The other factors such as w, h, α_h are constants and therefore can be ignored for calculations of shear and moment coefficients.

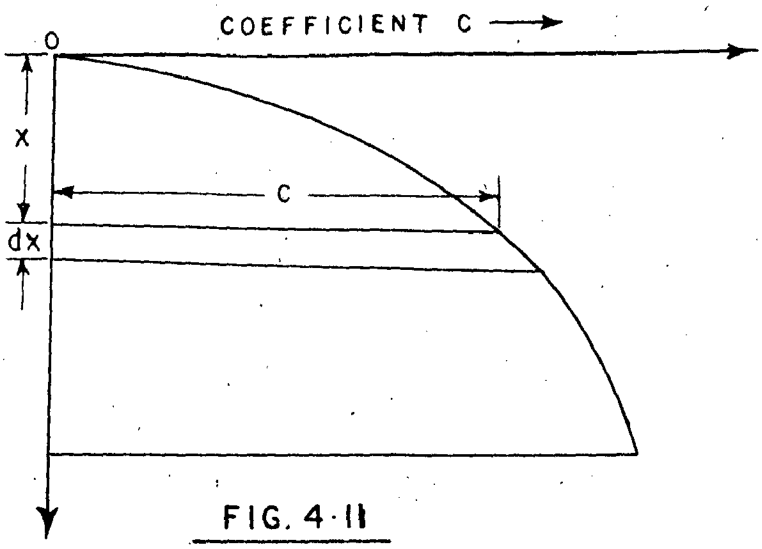


FIG. 4·11

The area of the strip at a distance x from top of water surface and depth dx would be Cdx as shown in Figure 4.11 above. The area upto x below top of water surface is given by

$$\text{Area} = \int_0^x C\,dx \qquad (4.24)$$

Substituting value of C using equation 2.2

$$\text{Area} = \int_0^x \frac{C_m}{2}\left[x(2-x) + \sqrt{x\,(2-x)}\right] dx$$

$$= \frac{C_m}{2}\left[\int_0^x 2x\,dx - \int_0^x x^2\,dx + \int_0^x \sqrt{2x - x^2}\,dx\right]$$

$$= \frac{C_m}{2}\left[x^2 - \frac{x^3}{3} + \int_0^x \sqrt{1 + (1-2x+x^2)}\,dx\right] \qquad (4.25)$$

Now putting $1-x = \sin\theta$, $\therefore$ $dx = -\cos\theta \, d\theta$

$$I = \int_0^x \sqrt{1-(1-2x+x^2)} \, dx = -\int_{\pi/2}^{\theta} \sqrt{1-\sin^2\theta} \, \cos\theta \, d\theta$$

$$= -\int_{\pi/2}^{\theta} \cos^2\theta \, d\theta$$

$$= -\int_{\pi/2}^{\theta} \frac{1}{2} (1+\cos 2\theta) \, d\theta$$

$$= \left[-\theta/2 - \frac{\sin 2\theta}{4} \right]_{\pi/2} + C$$

Putting $\theta = \pi/2$ when $x = 0$, area $= 0$ the value of C comes to $+\pi/4$

$$\therefore \text{ Integral} = \left[-\frac{\theta}{2} - \frac{\sin\theta \cos\theta}{2} + \frac{\pi}{4} \right] \qquad (4.25a)$$

$$= \left[-\sin^{-1}(1-x) - \frac{(1-x)x\sqrt{1-(1-x)^2}}{2} + \frac{\pi}{4} \right]$$

Finally the Area under the C curve is given by

$$\text{Area} = \frac{C_m}{2} \left[x^2 - \frac{x^3}{3} - \sin^{-1}(1-x) - \frac{(1-x)\sqrt{1-(1-x)^2}}{2} + \frac{\pi}{4} \right]$$

(4.26)

The values of area at various values of $x = y/h$ are calculated and given in Table 4.13 . The values of C at various depths and corresponding $P_e y$ are given in Table 4.12. The value of hydrodynamic shear coefficient $\beta_H(y)$ are obtained dividing the area under the curve upto the level under consideration by the factor $P_e . y$ at that level. These values of hydrodynamic shear coefficients $\beta_H(y)$ are plotted vide, Fig. 4.12.

Similarly the moment of the area about the water surface would be calculated. The moment of the elemental area Cdx about the top of water surface is given by

$$\text{Moment} = \int_0^x C\, dx \,.\, x \qquad (4.27)$$

Substituting value of C from equation 2.2

$$\text{Moment} = \int_0^x \frac{C_m}{2} \left[x(2-x) + \sqrt{x\,(2-x)}\right] x\, dx$$

$$= \frac{C_m}{2} \left[\int_0^x 2x^2\, dx - \int_0^x x^3\, dx - \int_0^x x \sqrt{x(2-x)}\, dx \right]$$

$$= \frac{C_m}{2} \left[\frac{2}{3} x^3 - \frac{x^4}{4} - \int_0^x x \sqrt{x(2-x)}\, dx \right] \qquad (4.27a)$$

Now

$$I = \int_0^x x \sqrt{2x - x^2} = \int_0^x x \sqrt{1-1+2x-x^2}\; dx$$

$$= \int_0^x x \sqrt{1 - (1-x)^2}\, dx$$

Let $1-x = \sin\theta \quad \therefore dx = -\cos\theta\, d\theta$

$$x = 1 - \sin\theta$$

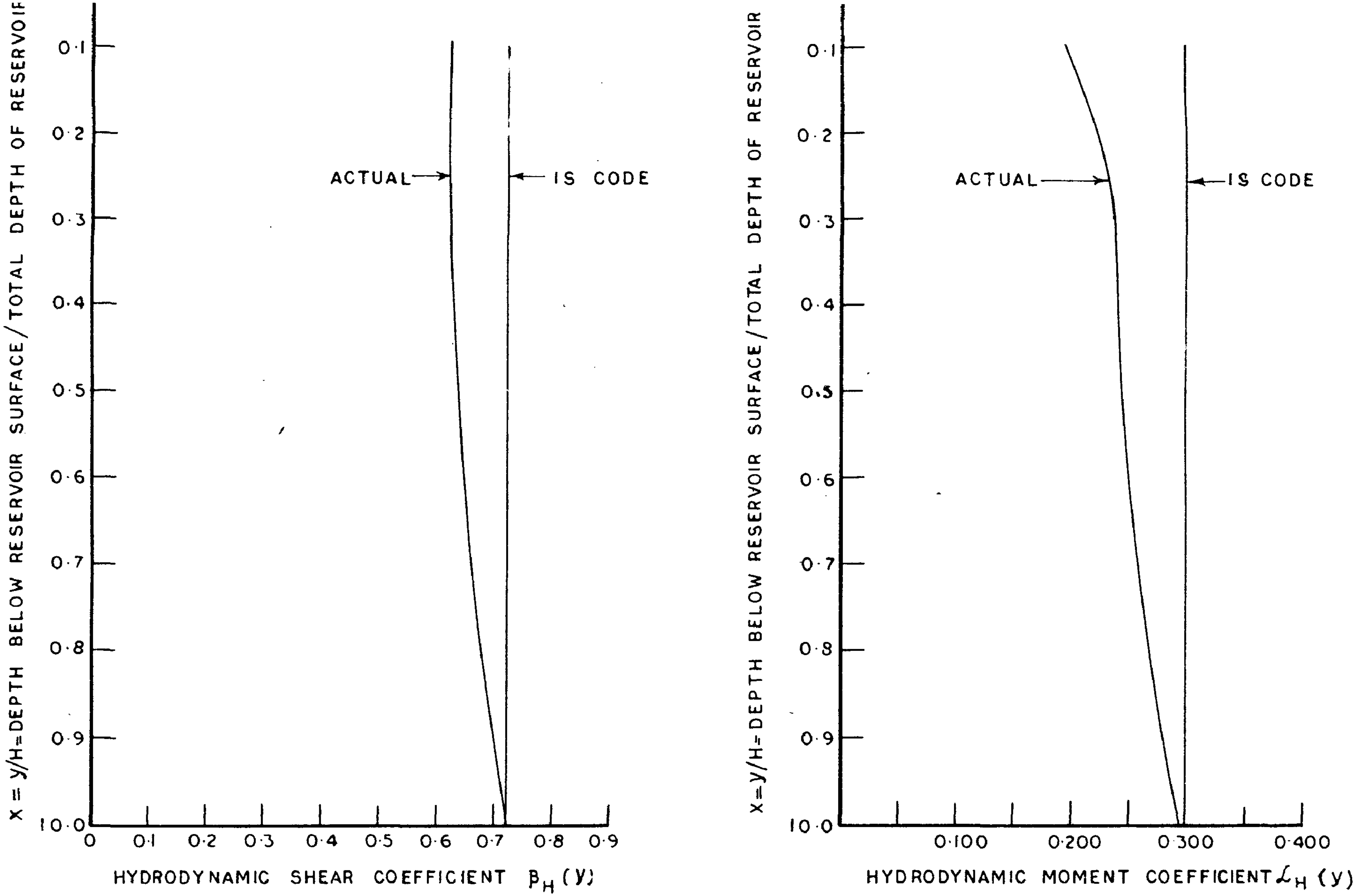


FIG. 4·12_MOMENT AND SHEAR DISTRIBUTION CURVES FOR HYDRODYNAMIC FORCES

$$I = -\int_{\pi/2}^{\theta} (1-\sin\theta)\sqrt{1-\sin^2\theta}\;\cos\theta\,d\theta$$

$$= -\int_{\pi/2}^{\theta} (1-\sin\theta)\cos\theta\;.\cos\theta\,d\theta$$

$$= -\int_{\pi/2}^{\theta} (1-\sin\theta)\cos^2\theta\,d\theta$$

$$= -\int_{\pi/2}^{\theta} \cos^2\theta\,d\theta + \int_{\pi/2}^{\theta} \sin\theta\cos\theta\,.\cos\theta\,d\theta$$

$$I = -\frac{1}{2}\int_{\pi/2}^{\theta} 2\cos^2\theta\,d\theta + \frac{1}{2}\int_{\pi/2}^{\theta}\sin 2\theta\;\cos\theta\,d\theta$$

$$= -\frac{1}{2}\int_{\pi/2}^{\theta} (1+\cos 2\theta)\,d\theta + \frac{1}{2}\int_{\pi/2}^{\theta}\frac{1}{2}\left[\sin\theta + \sin 3\theta\right]d\theta$$

$$= -\frac{1}{2}\left[\theta + \frac{\sin 2\theta}{2}\right] + \frac{1}{4}\left[-\cos\theta\right] + \frac{1}{4}\left[-\frac{\cos 3\theta}{3}\right] + C$$

(4.28a)

at $x = 0$, $\theta = \pi/2$ and moment $= 0$ hence value of constant C is $+\pi/4$. Finally the moment of area about the water surface is given by

$$\text{Moment} = \frac{C_m}{2}\left[\frac{2x^3}{3} - \frac{x^4}{4} - \frac{\theta}{2} - \frac{\sin\theta\cos\theta}{2} - \frac{\cos\theta}{4} - \frac{\cos 3\theta}{12} + \frac{\pi}{4}\right] \tag{4.29}$$

$$= \frac{C_m}{2}\left[\frac{2x^3}{3} - \frac{x^4}{4} - \sin^{-1}(1-x) - (1-x)\sqrt{1-(1-x)^2} - \frac{\sqrt{1-(1-x)^2}}{4} - \frac{\cos 3\theta}{12} + \frac{\pi}{4}\right] \tag{4.29a}$$

The values of the moments are tabulated in Table 4.14. These momen s divided by the area at corresponding levels would give the distance of C.G. of area ($\bar{y}$) from the water surface. Thus the distance of C.G. of the area from the level under consideratio ($\bar{\bar{x}} = x-\bar{y}$) can be calculated and the corresponding moment of area about the base of the section can be calculated. This moment divided by $P_o y^2$ would give the hydrodynamic moment coefficient $\alpha_H(y)$. These are plotted in Fig. 4.12.

4.9 OBSERVATIONS

The plot of hydrodynamic shear and moment coefficient against the various depths below the top of the reservoir surface as presented in Fig. 4.12 shows clearly that the hydrodynamic shear and moment coefficients $\beta_H(y)$ and $\alpha_H(y)$ respectively are not constant over the entire depth of the reservoir as contemplated in the IS code provisions (1966, 1970, 1974). The value of hydrodynamic shear coefficient at the bottom of the reservoir is 0.726 which is equal to the value given by IS Code. But, at a point which is 1/10th of reservoir depth below the water s-urface the value of hydrodynamic shear coefficient is 0.6275, as per theoretical analysis. Thus at this point the hydrodynamic shear given by the IS code formula over-estimates the value by about 15 per cent over the correct value. This overestimation is near about 14 per cent at a point which is at a depth of 40 per cent of total depth below the reservoir. It is therefore suggested that we may use the hydrodynamic shear distribution as proposed in the fig. 4.12 for accurate evaluation of the hydrodynamic shear.

In case of hydrodynamic moment the percentage of overestimation of the same as given by the IS code formula is a low as 2.2 per cent at bottom of reservoir and as high as 54 per cent at depth of 10 per cent of total depth below the reservoir water surface. Thus the variation is pronounced in case of hydrodynamic moments. Therefore for correct evaluation of hydrodynamic moments the distribution curve given in Fig. 4.12 is proposed.

- -

Sl. No.	Segment No.	Distance y of bottom of segment from top of dam. m	Ratio of distance of bottom of segment from top to total height of dam y/H	Mean width of segment W_m m	Height of segment Δh m	Weight of segment $2.4\ W_m \times \Delta h$ tonne	Dynamic shear due to wt. of each segment $= wt. \times 2\alpha_o$ $= Col\ 7 \times 2\alpha_o$ tonne	Dynamic shear Ve at bottom of segment $\sum Col.\ 8$ α_o	Dynamic shear coeff. $\frac{Ve}{W \alpha_o}$ $= \frac{Col.9}{9190\ \alpha_o}$	Remarks
1	2	3	4	5	6	7	8	9	10	11
1										
2	1	8	0.08	8	8	153.5	307	307	0.0334	
3	2	13	0.13	8	5	96	192	499	0.0544	
4	3	20	0.20	10.8	7	181.5	363	862	0.0940	
5	4	30	0.30	17.6	10	422.5	845	1707	0.1860	
6	5	40	0.40	25.6	10	615	1230	2937	0.3200	
7	6	50	0.50	33.6	10	807.5	1615	4552	0.4950	
8	7	60	0.60	41.6	10	1000	2000	6552	0.7130	
9	8	70	0.70	49.6	10	1190	2380	8932	0.9700	
10	9	80	0.80	57.6	10	1380	2760	11692	1.2700	
11	10	90	0.90	65.6	10	1575	3150	14842	1.6190	
12	11	100	1.00	73.6	10	1769	3538	18380	2.0000	

Total weight of Dam W = 9190 t

Sl. No.	Segment No.	Distance y of ~~of botton~~ of botton of segment from top of Dam	Ratio of distance y to the total height of dam $\frac{col.3}{100}$	Weight of segment vide col.7 of Table.4.1	Value of seismic coef α_h at the centre of the segment. $\times\alpha_0$	Dynamic shear due to wt. of individual segment col5 x 6 x α_o	Dynamic shear Vo at bottom of segment $\sum$ Col. 7 x α_o	Dynamic shear coef $\frac{Vo}{W \alpha_o}$ $\frac{Col.8}{9190 \alpha_o}$
1	2	3	1	5	6	7	8	9
1	-	-	-	-	-	-	-	-
2	1	8	0.08	153.5	2.880	442	442	0.0439
3	2	13	0.13	96	2.685	258	700	0.0770
4	3	20	0.20	181.5	2.505	455	1155	0.1257
5	4	30	0.30	422.5	2.250	950	2105	0.2310
6	5	40	0.40	615	1.950	1200	3305	0.3620
7	6	50	0.50	807.5	1.650	1331	4636	0.5050
8	7	60	0.60	1000	1.350	1350	5986	0.6520
9	8	70	0.70	1190	1.05	1250	7236	0.7380
10	9	80	0.80	1380	0.75	1035	8272	0.9000
11	10	90	0.90	1575	0.45	709	8981	0.9750
12	11	100	1.00	1769	0.15	285	9246	1.0090

Sl. no.	Ratio of Distance x from bottom of dam to total height of dam x/H	Ratio of distance y from top of dam to total height of dam $\frac{y}{H} = 1 - \frac{x}{H}$	Dynamic shear coeff. read from figure in IS:1893 of 1970 C'_v	Dynamic shear coeff $\beta(y) = 1.2 C'_v$ = 1.2 x Col.4.	Rema
1	2	3	4	5	
1	1	0	0	0	
2	0.92	0.08	0.060	0.072	
3	0.87	0.13	0.115	0.1380	
4	0.8	0.2	0.185	0.2220	
5	0.7	0.3	0.290	0.3480	
6	0.6	0.4	0.400	0.4800	
7	0.5	0.5	0.520	0.6250	
8	0.4	0.6	0.650	0.7800	
9	0.3	0.7	0.780	0.9360	
10	0.2	0.8	0.870	1.0450	
11	0.1	0.9	0.940	1.129	
12	0	1.0	1.000	1.2000	

Sl. No.	Ratio of distance x from bottom of dam to total height of dam x/H	Ratio of distance y from top of dam to total height of dam $\frac{y}{H} = 1 - \frac{x}{H}$	Dynamic shear coeff. β(x) read from av. curve in dynamic analysis (Ref. 55)	Dynamic shear coeff. β(y) = 1.075 β(x) = 1.075 Col.4	Remarks
1	2	3	4	5	6
1	1.0	0	0	0	
2	0.9	0.1	0.140	0.1505	
3	0.8	0.2	0.235	0.2600	
4	0.7	0.3	0.325	0.3390	
5	0.6	0.4	0.415	0.4460	
6	0.5	0.5	0.525	0.5650	
7	0.4	0.6	0.640	0.6890	
8	0.3	0.7	0.770	0.8280	
9	0.2	0.8	0.885	0.9525	
10	0.1	0.9	0.970	1.0410	
11	0	1.00	1.000	1.0750	

Sl No.	Ratio of distance from bottom to total height of dam x/H	Ratio of distance from top to total height of dam $\frac{y}{H} = 1 - \frac{x}{H}$	Dynamic moment M_y = $x \alpha_o$	Dynamic moment coeff. $\alpha(y) = \frac{M_y}{wH\bar{\alpha}_o} = \frac{M_y}{308178\ \alpha_o}$	Remai
1	2	3	4	5	
1	1	0	0	0	
2	0.92	0.08	1228	9.004	
3	0.87	0.13	3244	0.010	
4	0.80	0.20	8006	0.026	
5	0.70	0.30	20852	0.068	
6	0.60	0.40	44072	0.144	
7	0.50	0.50	81516	0.264	
8	0.40	0.60	137036	0.444	
9	0.30	0.70	214456	0.696	
10	0.20	0.80	317576	1.030	
11	0.10	0.90	450246	1.460	
12	0	1.00	616356	2.000	

TABLE 4.6 DYNAMIC MOMENT AS PER IS: 1893 OF 1970 FOR 100M HIGH DAM

Sl. No.	Ratio of distance from top to total height y/H	Dynamic M_y fm $x\,\alpha_o$	dynamic moment coefficient $= \frac{m_y}{WH\,\alpha_o} = \frac{M_y}{308178 \times \alpha_o}$		Remarks
1	2	3	4		
1	0	0	0		
2	0.08	1,792	0.0058		
3	0.13	4,677	0.01515		
4	0.20	11,211	0.0363		
5	0.30	27,631	0.0895		
6	0.40	54,861	0.1780		
7	0.50	94,641	0.3060		
8	0.60	157,721	0.4790		
9	0.70	213,801	0.6910		
10	0.80	291,306	0.9440		
11	0.90	377,526	1.2210		
12	1.00	468,616	1.5200		

TABLE 4.7 DYNAMIC MOMENT FOR 100M HIGH DAM AS PER IS: 1893 OF 1974

Sl. No.	Distance from bottom to total heigh ratio x/H	Distance from top to total height ratio $\frac{y}{H} = (1 - \frac{x}{H})$	Dynamic moment coeff. C_m read from Figure in IS:1893 of 1974	Dynamic Moment coefficient $\alpha(y) = 1.8$ cm.
1	2	3	4	
1	1.0	0.000	0.000	0.000
2	0.92	0.08	0.005	0.009
3	0.87	0.13	0.015	0.027
4	0.80	0.2	0.04	0.072
5	0.70	0.3	0.09	0.162
6	0.6	0.4	0.159	0.279
7	0.5	0.5	0.250	0.450
8	0.4	0.6	0.360	0.648
9	0.3	0.7	0.509	0.910
10	0.2	0.8	0.665	1.199
11	0.1	0.9	0.820	1.479
12	0	1.0	1.000	1.800

TABLE 4.8 DYNAMIC MOMENT FOR 100M HIGH DAM AS PER DYNAMIC ANALYSIS

Sl. No.	Distance from top bottom to total height ratio x/H	distance from top to total height ratio $\frac{y}{H} = (1 - \frac{x}{H})$	Dynamic moment coeff. $\alpha(x)$ read from average curves in dynamic analysis (55)	Dynamic moment coefficient $\alpha(y) = 1.635\ \alpha(y)$
1	2	3	4	5
1	1.0	0	0	0
2	0.9	0.1	0.02	0.0327
3	0.8	0.2	0.055	0.0900
4	0.7	0.3	0.105	0.1718
5	0.6	0.4	0.175	0.286
6	0.5	0.5	0.260	0.425
7	0.4	0.6	0.370	0.609
8	0.3	0.7	0.500	0.8175
9	0.2	0.8	0.650	1.061
10	0.1	0.9	0.830	1.359
11	0	1.00	1.000	1.635

TABLE 4.9 FUNDAMENTAL PERIOD OF VIBRATION OF DAM BY IS CODE OF 1974

Sl. No.	CASE	Height of Dam H m	Base width of Dam B m	Unit weight of dam W_m kg/m^3	Modulus of elasticity of dam material kg/m^2	Acceleration due to gravity g m/sec^2	Fundamental period of vibration of Dam T seconds	Value of S_a/g read from average spectra	Value of $2\frac{F_o S_a}{g} = 2 \times 0.25 \frac{S_a}{g} = 0.5 S_a/g$	Ratio of $\frac{2 F_o S_a}{g} / \alpha_o = \frac{Col. 10}{0.05}$
1	2	3	4	5	6	7	8	9	10	11
1	I	50	38.40	2400	2200×10^6	9.80665	not necessary			
2	II	100	77.60	-do-	-do-	-do	0.238	0.19	0.095	1.9
3	III	150	116.40	-do-	-do-	-do-	0.357	0.19	0.095	1.9
4	IV	200	156.00	-do-	-do-	-do-	0.476	0.17	0.085	1.7

TABLE 4.10 QUANTITATIVE COMPARISON OF DYNAMIC MOMENTS AND SHEARS

Sl. No.	CASE	DYNAMIC ANALYSIS					IS CODE OF 1974				
		Natural period of vibration T_1	S_a/g read from average spectra	$\frac{2F_o S_a}{g}$ read $0.5 \frac{S_a}{g}$	Shear coeff $\beta(y)$ $C_{vb} \times \frac{2F_o S_a}{\alpha_o}$ $\beta(x)$	Moment coeff $C_{mb} \times \frac{2F_o S_a}{g \alpha_o} \frac{H}{B}$ $\times \alpha(x)$	Natural period of vibration of Dam T_1	S_a/g read from curve	$\frac{2F_o S_a}{g}$	Shear coeff. $\beta(y)$ $0.6 \times \frac{2F_o S_a}{g} \frac{1}{\alpha_o}$ $\times C'_v$	Moment Coe $\alpha(y)$ $0.9 \times \frac{2F_o S_a}{g \alpha_o}$ $\times C'_m$
1	2	3	4	5	6	7	8	9	10	11	12
1	I	0.1415	0.17	0.085	0.88β(x)	1.301α(x)					
2	II	0.2415	0.192	0.096	1.031β(x)	1.57 α(x)	0.238	0.19	0.095	1.14 C'_v	1.71 C'_m
3	III	0.3440	0.192	0.096	1.01 β(x)	1.633 α(x)	0.357	0.19	0.095	1.162 C'_v	1.715 C'_m
4	IV	0.4530	0.175	0.0875	0.98 β (x)	1.49 α(x)	0.476	0.17	0.085	1.02 C'_v	1.53 C'_m

TABLE 4.11 CALCULATIONS OF FUNDAMENTAL PERIOD OF VIBRATION OF DAM AS PER DYNAMIC ANALYSIS AND BASE MOMENT AND SHEAR COEFFICIENTS

Sl No.	CASE	Height of dam H	Modulus of elasticity of dam E kg/m²	Density of dam material ρ kg/m³	Acceleration due to gravity g m/sec²	Longitudinal wave velocity $V_L = \sqrt{\frac{Eg}{\rho}}$ m/sec.	Base width B m	Radius of gyration of base $r_o = \frac{B}{\sqrt{12}}$ m	Period coeff. read from curve CT_1	Fundamental period $T_1 = C_{T_1} \times \frac{H}{V_L} \times \frac{H}{r_o}$ sec.	Base shear coeff. Cvb read from Fig.	Base moment Coef read from Fig.	Distance of C.G. from bottom $\bar{h}$
1	2	3	[illegible]	5	6	7	8	9	10	11	12	13	14
1	I	50	2200x10⁶	2400	9.80665	3000	33.40	11.10	1.885	0.1415	0.5175	0.2725	17.
2	II	100	-do-	-do-	-do-	-do-	77.60	22.40	1.620	0.2415	0.5375	0.2750	[illegible]
3	III	150	-do-	-do-	-do-	-do-	116.40	33.60	1.540	0.3440	0.5250	0.2750	[illegible]
4	IV	200	-do-	-do-	-do-	-do-	156.00	45.00	1.525	0.4530	0.5600	0.2300	[illegible]

TABLE 4.12 CALCULATIONS FOR COEFFICIENT

l. o.	x $\frac{y}{h}$	$2 - \frac{y}{x}$ $= 2 - x$	$\frac{y}{x}(2 - \frac{y}{x})$	$\sqrt{\frac{y}{x}(2 - \frac{y}{x})}$	$\frac{y}{h}(2 - \frac{y}{h}) + \sqrt{\frac{y}{h}(2 - y/h)}$ Col. 4 + col. 5	$C = \frac{C_m}{2} [$ Col
1	2	3	4	5	6	
	0	2	0	0	0	
	0.1	1.9	0.19	0.435	0.625	
	0.2	1.8	0.36	0.600	0.960	
	0.3	1.7	0.51	0.714	0.224	
	0.4	1.6	0.64	0.800	1.440	
	0.5	1.5	0.75	0.865	1.615	
	0.6	1.4	0.84	0.915	1.755	
	0.7	1.3	0.91	0.954	1.864	
	0.8	1.2	0.96	0.930	1.940	
	0.9	1.1	0.99	0.995	1.985	
	1.0	1.0	1.0	1.0	2.00	

TABLE 4.13 HYDRODYNAMIC SHEAR COE

$x = \frac{y}{h}$	x^2	$-\frac{x^3}{3}$	$\sin\theta = 1 - x$	θ in deg.	θ in rad.	$-\frac{\theta}{2}$	$\cos\theta$	$-\sin\theta\cos$ Col.5 x 9
2	3	4	5	6	7	8	9	10
0	0	0	1	90	1.571	-0.7855	0	0
.1	.01	-.0003	0.9	64.17	1.12	-.5600	.436	-.39
.2	.04	-.0027	0.8	53.13	0.926	-.4630	.600	-.48
.3	.09	-.009	0.7	44.43	0.775	-.3875	.714	-.50
.4	.16	-.0213	0.6	36.83	0.644	-0.3220	.800	-.48
.5	.25	-.0417	0.5	30.00	0.523	-.2615	.865	-.43
.6	.36	-.072	0.4	23.51	.410	-.2050	.916	-.36
.7	.49	-.1143	0.3	17.47	.305	-.1525	.954	-.28
.8	.64	-.1706	0.2	11.55	.2015	-.1007	.980	-.19
0.9	.81	-.2430	0.1	5.75	.1002	-.0500	0.985	-.09
1.0	1.0	-.3333	0	0	0	0	1.00	0

TABLE 4.14 HYDRODYNAMIC MOMENT COEFFICIENT

:	$x = \frac{y}{h}$	$\frac{2}{3} x^3$	$-\frac{x^4}{4}$	$\sin\theta = 1-x$	θ in deg.	θ in rad.	$-\theta/2$	$\cos\theta$	$+\frac{\sin\theta\cos\theta}{2}$ $\frac{\text{Col.}5\times9}{2}$	$-\frac{\cos\theta}{4}$ (-)
	2	3	4	5	6	7	8	9	10	11
	0	0	0	1.0	90	1.571	-0.7855	0	0	0
	0.1	0.00067	-.00002	0.9	64.17	1.120	-0.5600	0.436	-.1960	-.1088
	0.2	0.00533	-.0004	0.8	53.13	.926	-0.4630	0.600	-.2400	-.1500
	0.3	0.01800	-.00203	0.7	44.43	.775	-0.3875	0.714	-.2500	-.1785
	0.4	0.04267	-.00640	0.6	36.87	.644	-0.322	0.8	-2400	-.2000
	0.5	0.08333	-.01562	0.5	30.00	.523	-0.2615	0.865	-.21625	-.216
	0.6	0.14400	-.03240	0.4	23.51	.410	-0.[illegible]	0.916	-.183	-.229
	0.7	0.22867	-.06001	0.3	17.47	0.305	-.1525	0.954	-.143	-.238
	0.8	0.34132	-.1024	0.2	11.55	.2015	-.1007	0.980	-.098	-.245
	0.9	0.486	-.1640	0.1	5.75	.1002	-.0500	0.985	-0.0492	-.249
	1.0	0.6667	-0.25	0	0	0	0	1.000	0	-.250

Note $P_e = C\ wh\ \alpha h$

$$P_e\, y^2 = C\ wh\ .\ \alpha h\ .\ (y/h)^2\ h^2$$

$$= C\ wh^3 (\alpha h)\ (y/h)^2$$

$P_e y^2$ = constant x C x^2 where, w, h

M_e = Moment of Pe about base of s

= This also contains w, h^3 αh

$$\alpha_H = \frac{M_e}{P_e\ y^2}$$ will give hydrod

CHAPTER - V

SIGNIFICANT RESULTS

5.1 GENERAL

Several dams have been analysed by various investigators using the techniques of dynamic analysis. As a result of these studies a lot of useful information about the behaviour of a concrete gravity dam subjected to earthquake is available now. These significant results are presented below which show the effect of various parameters on the earthquake behaviour of the dam.

5.2 EFFECT OF RESERVOIR WATER

The effect of reservoir water can be seen by analysing the structure with reservoir water and without it. The effect of reservoir water on natural period of vibration, dynamic displacements and accelerations, dynamic stresses, and total stresses as obtained in studies is as below. The reservoir water is taken as an added virtual mass.

The natural periods of vibration of the dam in various modes with and without the reservoir water have been calculated by both the beam method and the finite element analysis for Koyna dam[36]. They are reproduced in Table 5.1

TABLE 5.1 NATURAL PERIODS OF VIBRATIONS IN SECONDS

Sl No.	Mode No.	Beam Analysis		Finite Element Analysis	
		Without reservoir water	With Reservoir water	Without Reservoir water	With Reservoir water
1	2	3	4	5	6
1	1	0.340	0.389	0.355	0.401
2	2	0.132	0.156	0.135	0.162
3	3	0.069	0.082	0.102	0.107
4	4	-	-	0.071	-

It may be observed from the Table 5.1 above that there is an increase in the natural periods of vibration of the dam due to presence of reservoir water in all the modes of vibration in both the methods of analysis. The increase is less in higher modes than the first mode.

The dynamic displacements both vertical and horizontal are found to increase due to reservoir water all along the height of the dam[36].

The dynamic accelerations (vertical) also increase due to presence of reservoir water all along the height of dam[36]. The horizontal accelerations however, increase near the top of the dam but decrease elsewhere[36]

The effect of reservoir water on the dynamic stresses can be observed from table 5.2 below[36].

TABLE 5.2. COMPARISON OF DYNAMIC STRESSES

Method of Analysis	Condition	Base		El 39.00 m		El° 66.50m	
		up stream	down stream	up stream	down stream	up stream	down stream
Beam Method	Without water	±17.8	∓17.8	±17.6	∓17.6	±30.1	∓30.1
	with water	±24.1	∓24.1	±26.4	∓26.4	±37.3	∓37.3
Finite Element Method	Without water	±26.7	∓8.2	±24.2	∓13.6	±28.6	∓22.9
	With water	±39.7	∓10.4	±33.4	∓33.4	±39.1	∓32.4

Note- Stresses are in kg/cm^2

° d/s slope changes at this level.

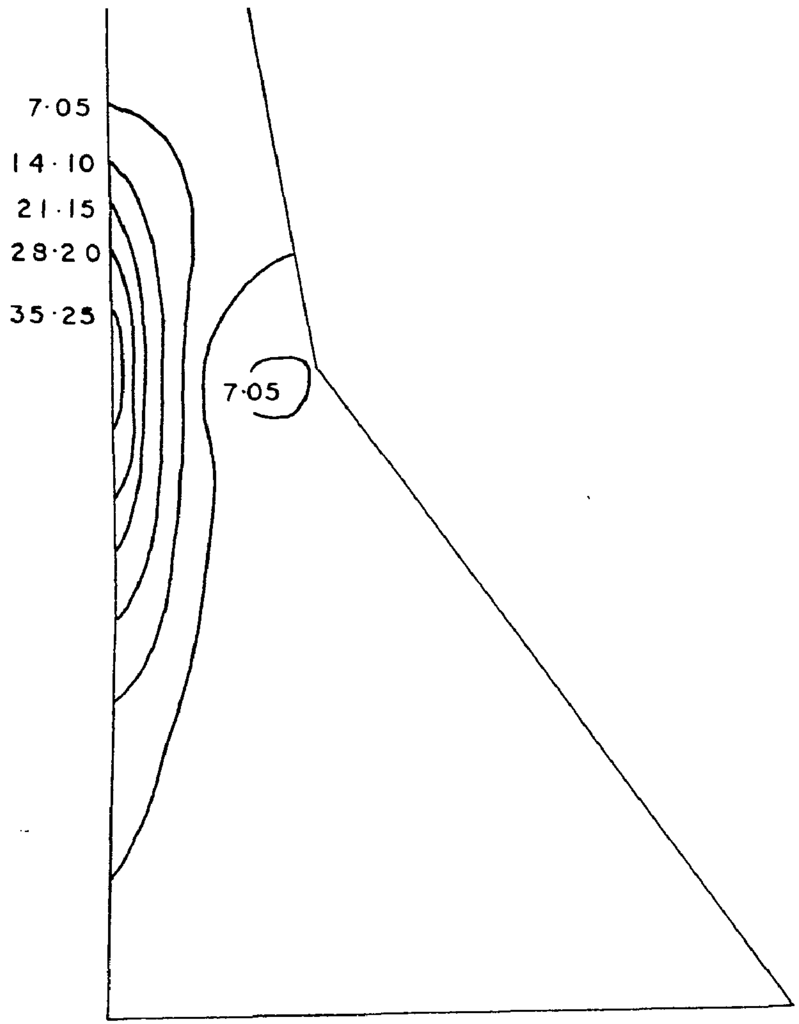
It may be noted from table 5.2 above that the effect of reservoir water is to increase the dynamic stresses. The increase in vertical normal dynamic stresses ranges from 25 per cent to 50 per cent due to reservoir water similar effect is observed in case of principal stresses also(36).

5.3 EFFECT OF VERTICAL GROUND MOTION

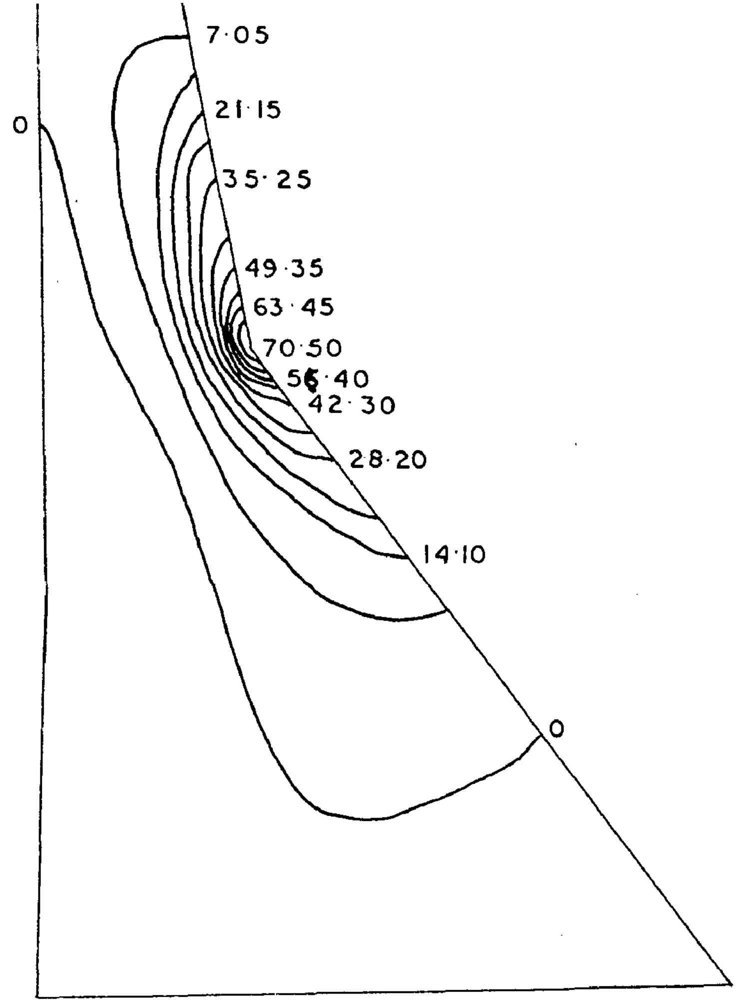
It can be studied by determining the response of dam due to horizontal component of ground motion alone as well as the response under the simultaneous action of horizontal and vertical components. Several investigators have studied this aspect.

The dynamic displacements (both vertical and horizontal) and dynamic horizontal accelerations calculated with horizontal and vertical components of ground motion acting simultaneously and with only horizontal component of ground motion are very close to each other showing that the vertical component of ground motion does not affect the displacement and acceleration pattern in the gravity dam[36]. The dynamic stresses due to simultaneous action of horizontal and vertical component of ground motion and those due to action of horizontal component of ground motion alone show practically no variation which gas again goes to prove that the vertical component of ground motion does not play an important part in determining the stress pattern in case of gravity dams.

These observations are more or less supported in a recent study[39] carried out on the behaviour of Koyna dam during earthquake of December 11, 1967. It is found that the inclusion of vertical component of ground motion does not cause a noticeable change in the maximum displacements, it causes an increase of 5 per cent in the horizontal accelerations and about 30 per cent increase in vertical accelerations at the crest. Figures 5.1 and 5.2[39] show the maximum principal stress contours

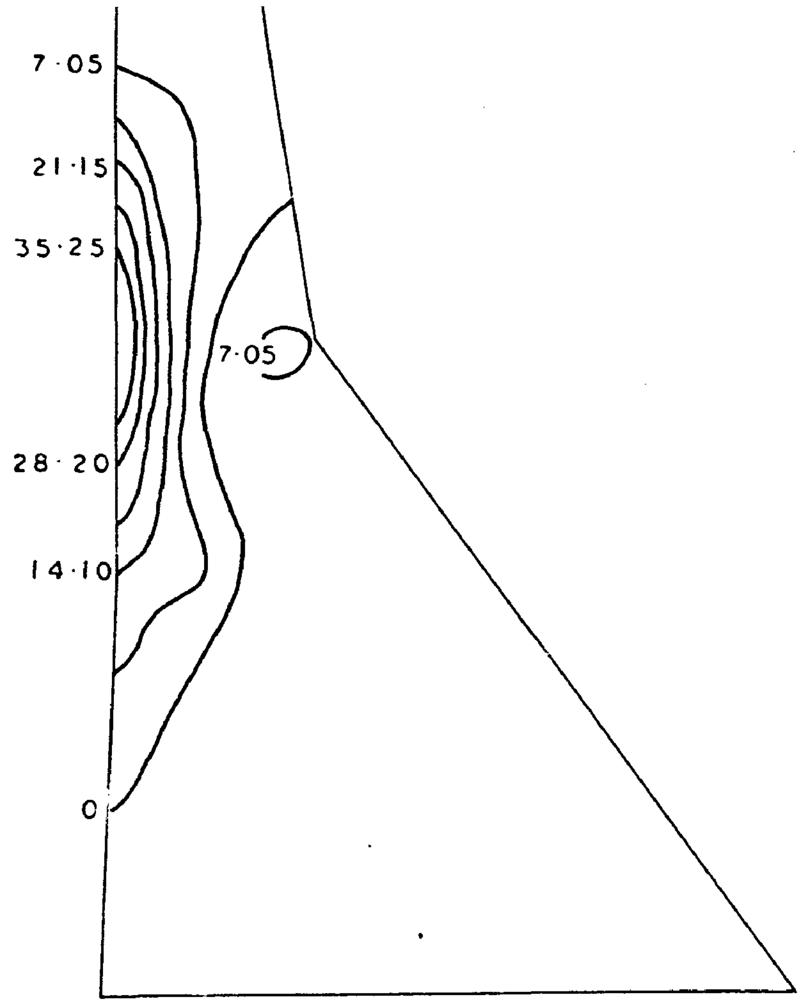


MAXIMUM PRINCIPAL STRESSES IN kg/cm^2 AT 4·425 SECS

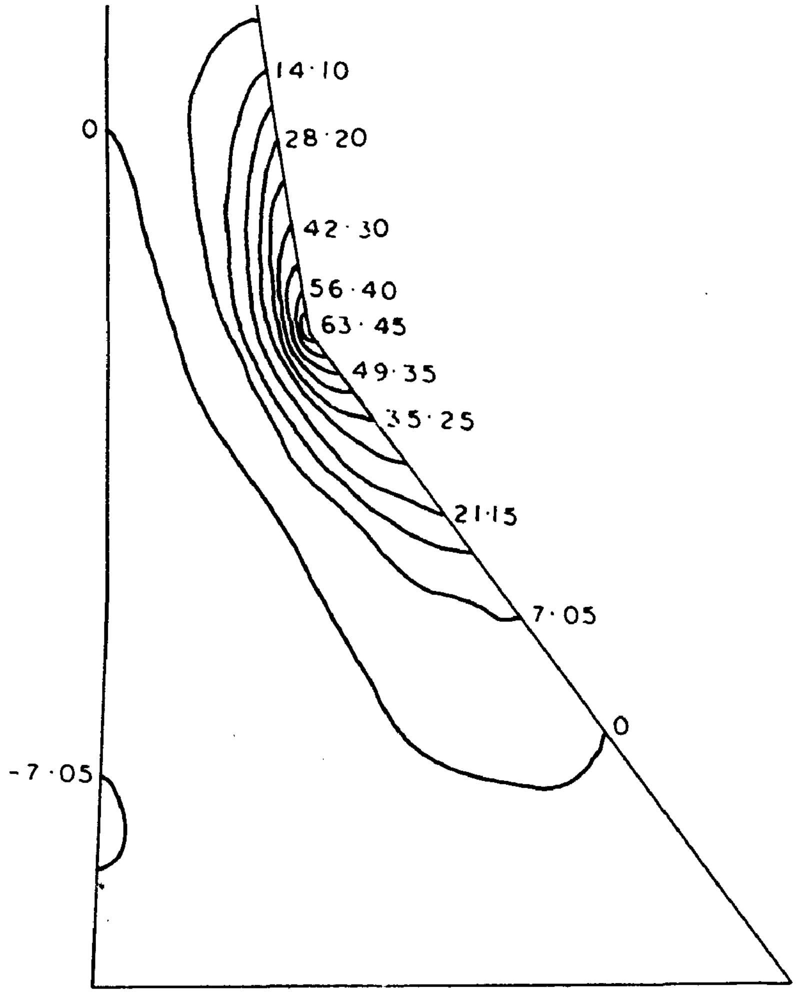


MAXIMUM PRINCIPAL STRESSES IN kg/cm^2 AT 4·25 SECS

FIG. 5·1 _ CRITICAL STRESSES IN NON-OVERFLOW MONOLITHS OF KOYNA DAM DUE TO TRANSVERSE AND VERTICAL COMPONENTS OF KOYNA EARTHQUAKE



MAXIMUM PRINCIPAL STRESSES IN kg/cm^2 AT 4·425 SECS



MAXIMUM PRINCIPAL STRESSES IN kg/cm^2 AT 4·60 SECS

FIG. 5·2 _ CRITICAL STRESSES IN NON-OVERFLOW MONOLITSH OF KOYNA DAM DUE TO TRANSVERSE COMPONENT OF KOYNA EARTHQUAKE in NOF section of Koyna dam due to simultaneous action of transverse and vertical components of Koyna earthquake and that of the transverse component alone. A comparison of these two figures shows that the larger tensile stress changes little near the upstream face but increases by about 7.05 kg/cm^2 near the downstream face due to inclusion of vertical component . As indicated by the relatively smaller contours for the higher stresses near the upstream face, the inclusion of vertical component of ground motion appears to result in a decrease in tensile stresses near the upstream face in the upper part . Although the tensile stress has increased by about 7.05 kg/cm^2 to more than 70.50 kg/cm^2 near the point where the downstream face changes abruptly. The changes away from the zone of stress concentration appear to be small.

In a recent study[16] it has been observed that the vertical component of ground motion causes an appreciable increase approximately 40 per cent in crest displacement and 40 per cent to 70 per cent increase in the tensile stresses if the effects of interaction between the reservoir and the dam are included in the analysis. If however the interaction effects are neglected the vertical component of ground motion has little influence on the response of the dam.

5.4 IMPORTANCE OF LIGHT WEIGHT STRUCTURAL SYSTEM AT TOP

Dynamic analysis of stresses in the concrete gravity dams during earthquakes demonstrates that accelerations at the top are maximum and thus the more critical tensile stresses occur in the upper parts of the dam[39]. The extra concrete near the crest of the dam to support roadway etc. is found to be responsible for this. This problem was studied by some investigators[39,40,52].

The responses of the Pine Flat dam and its structural section (a triangular section with zero width at the maximum waterlevel) were calculated[39]. The two sections are shown in Figures 5.3 . The periods of vibration of first four modes of the Pine flat dam were 0.256, 0.125, 0.092 , and 0.072 seconds and those of its structural section were 0.22, 0.099, 0.087 and 0.060 seconds respectively. This shows that there is a decrease in the periods of vibration in case of structural section for all the modes. The shapes of the first two modes of vibration of the Pine Flat dam and its structural section are presented in Fig.5.4. It is apparent that the added mass at crest affects the mode shapes considerably in the upper parts of the dams, resulting in significant increase in the displacement gradients especially for higher modes. A comparison of the principal stresses over the Pine Flat dam and its structural section at two selected instants of time as shown in Fig. 5.5 and

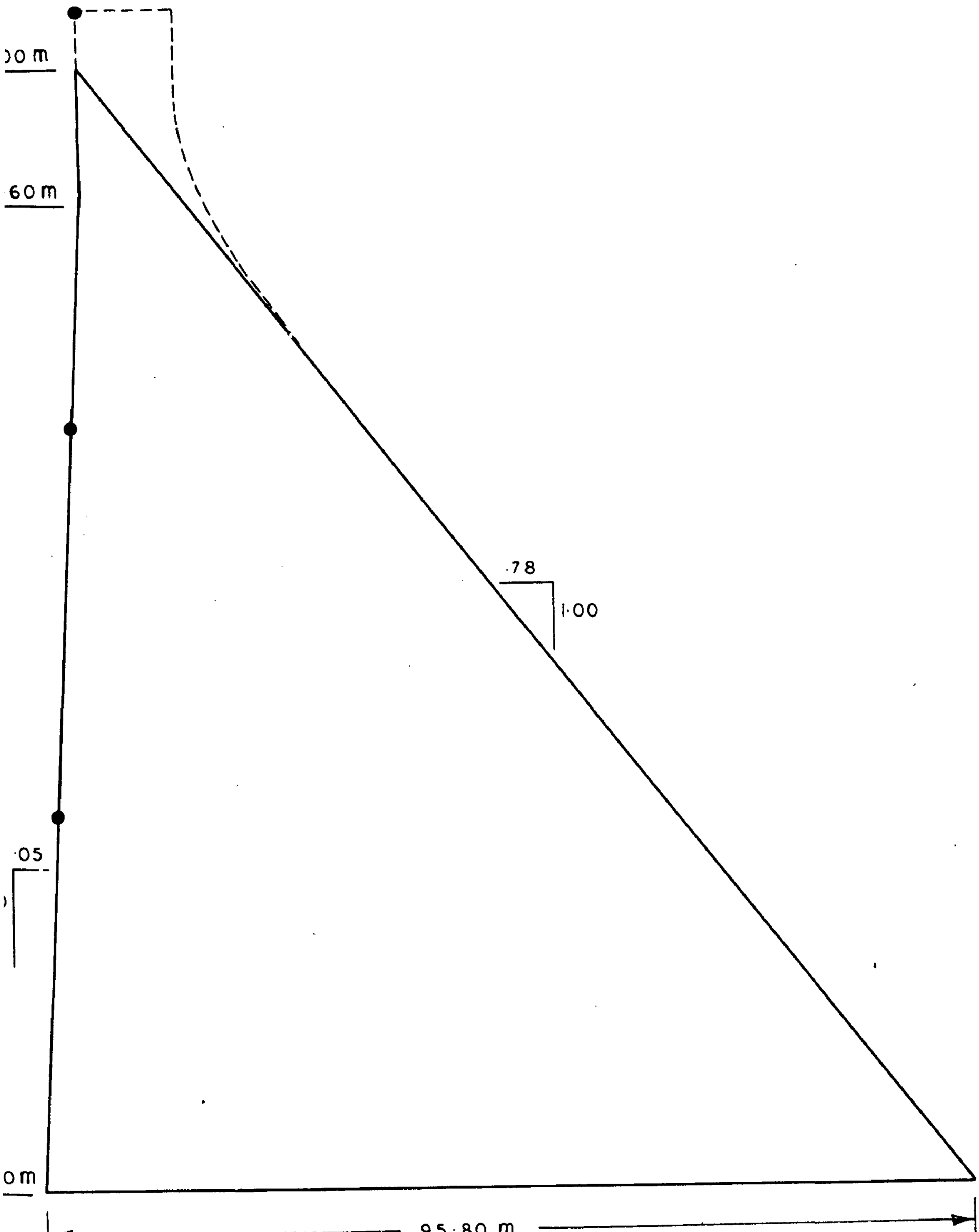


FIG. 5·3 _ PINE FLAT DAM STRUCTURAL SECTION

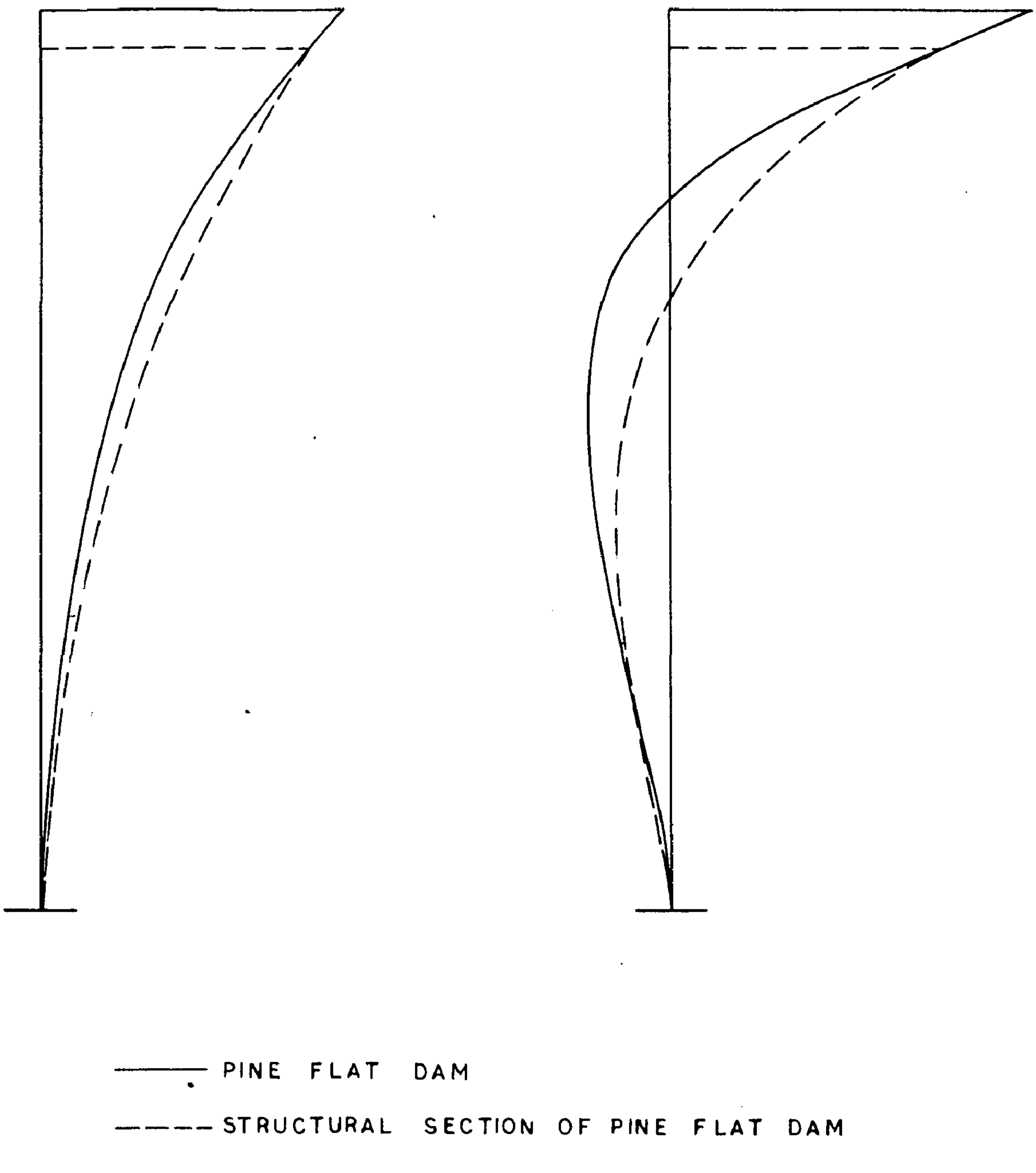


FIG. 5·4 _ MODE SHAPES OF PINE FLAT DAM

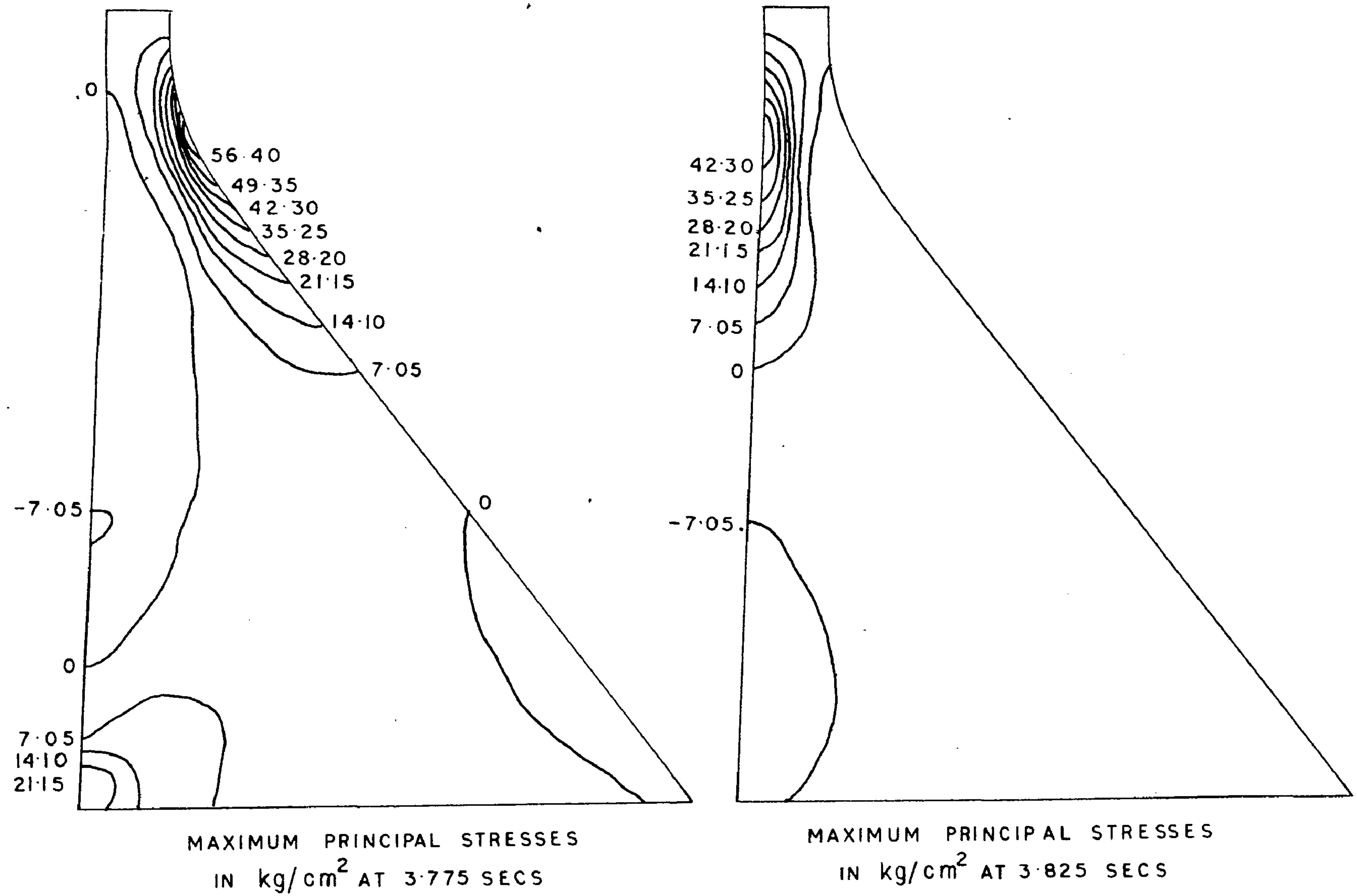
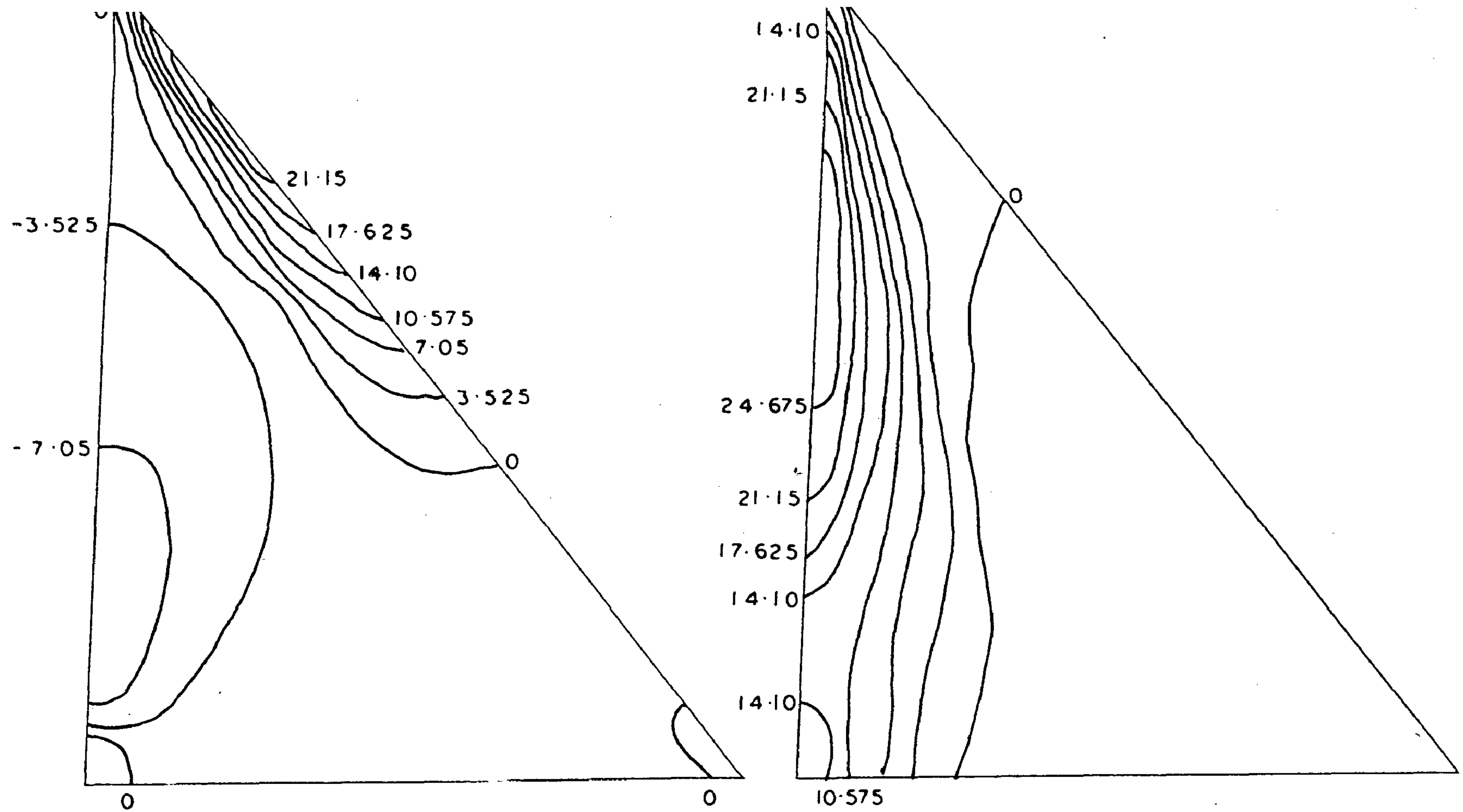


FIG. 5·5 _CRITICAL STRESSES IN PINE FLAT DAM DUE TO KOYNA EARTHQUAKE



MAXIMUM PRINCIPAL STRESSES IN kg/cm^2 AT 2·925 SECS

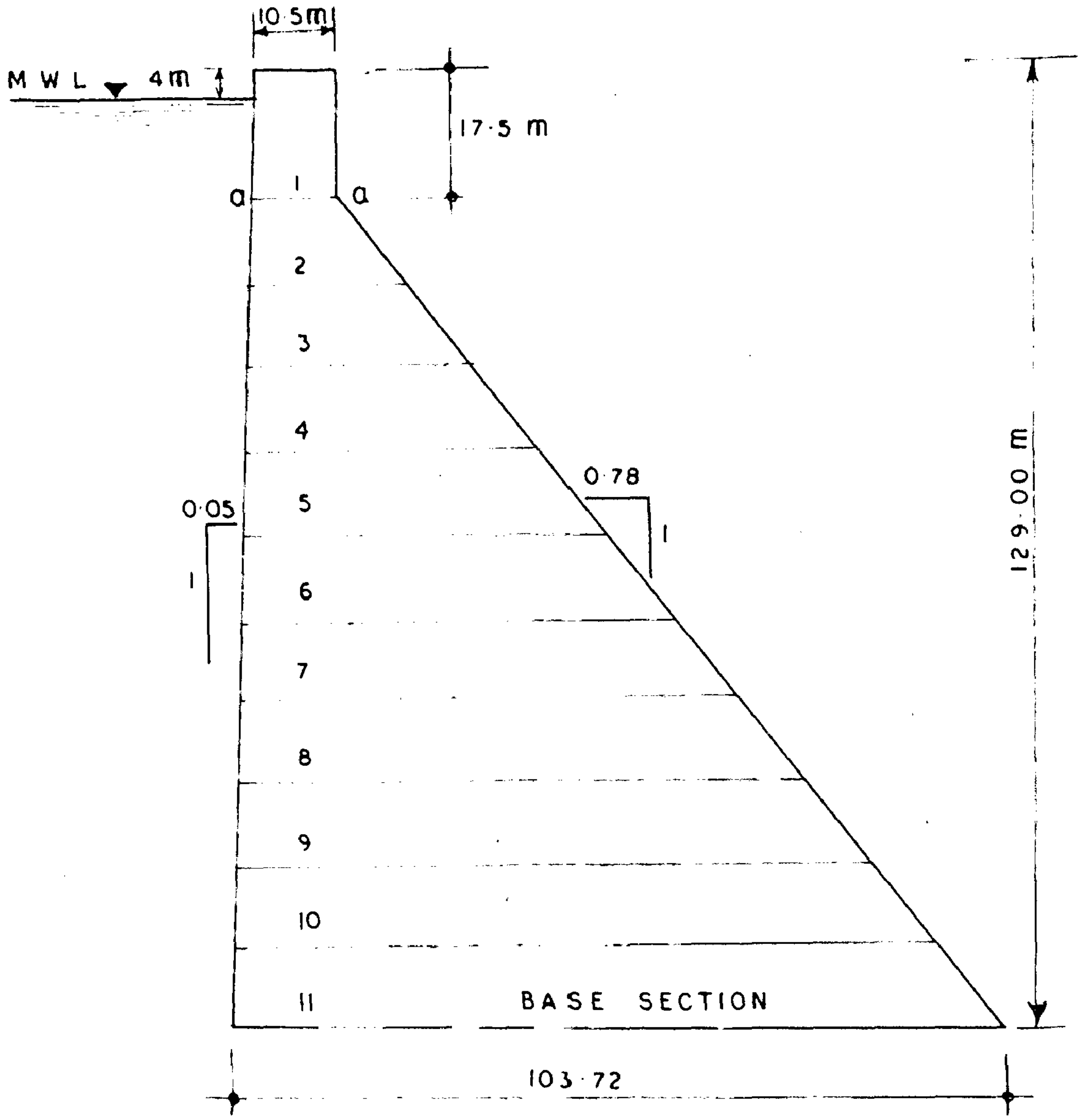
MAXIMUM PRINCIPAL STRESSES IN kg/cm^2 AT 3·675 SECS

FIG. 5·6_CRITICAL STRESSES IN PINE FLAT DAM STRUCTURAL SECTION DUE TO KOYNA EARTHQUAKE

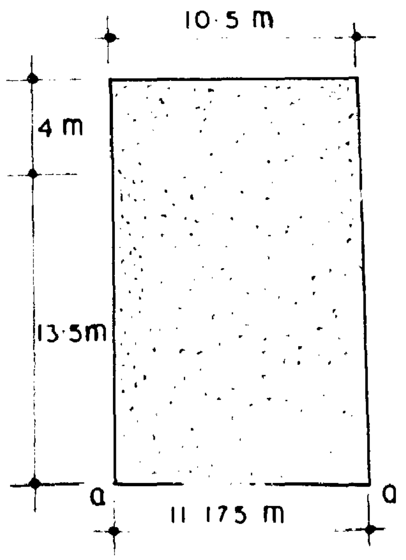
5.6 indicates that the maximum tensile stress on the downstream face is about 21.15 kg /cm^2 and that on the upstream face is a bout 24.675 kg/cm^2 for the structural sec tion (Fig. 5.6) while the corresponding values in case of Pine Flat dam are 56.40 kg/cm^2 and 42.30 kg/cm^2 respec tively (Fig. 5.5). It is apparent that much smaller tensile stresses develop in the structural section they being reduced to about 40 percent at the downstream face and about 60 per cent at the upstream face. It is evident from the results discussed above that the additional material near the crest of the dam causes considerable increase in earthquake stresses especially near the top.

This aspect of reducing the mass at the crest of a dam by developing light weight structural systems to support the roadway and perform other necessary functions was studied further(52). The dynamic analysis was carried out by the beam method of analysis. The bas-ic section of the dam used for the study was the Pine Flat dam of california slightly modified as shown in Fig. 5.7 . Seven different light weight structural systems as shown in Fig. 5.7 and 5.8 were considered.

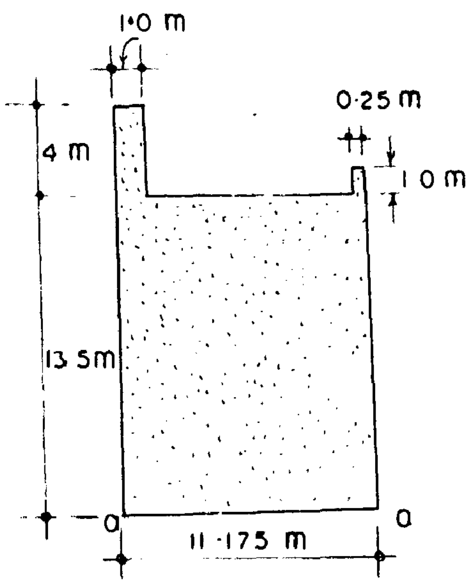
The static principal stresses remain more or less the same for all the cases considered for both reservoir



MODIFIED PINE FLAT DAM



a-CASE-1



b-CASE-2

FIG. 5·7

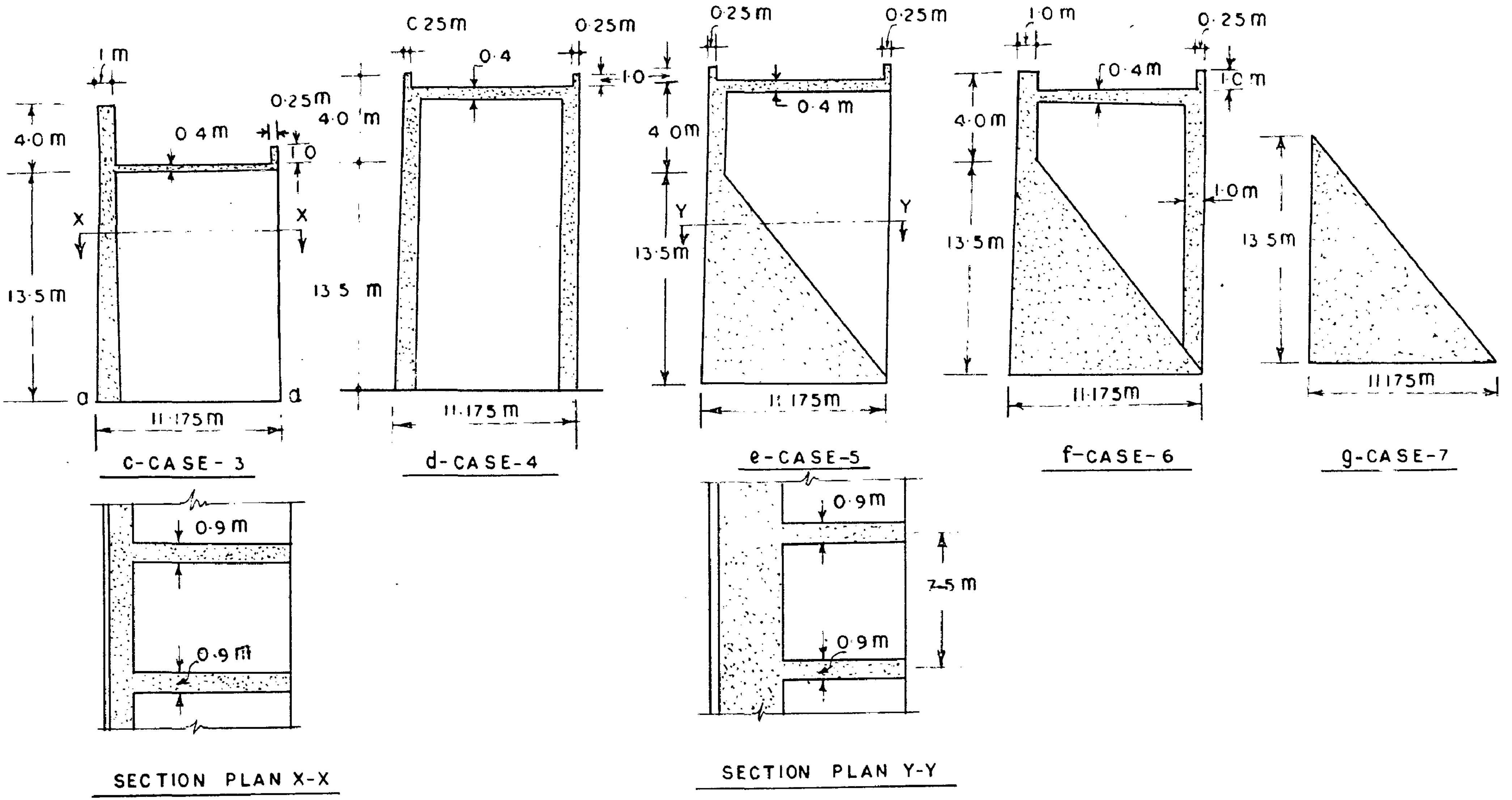


FIG. 5·8_ CASE OF STRUCTURAL SYSTEMS AT TOP OF DAM full and empty conditions vide Table 5.3 below.

TABLE 5.3 STATIC PRINCIPAL STRESSES kg/cm^2

(A) RESERVOIR FULL CONDITION

Section	Case 1		case 3		case 7	
	u/s	d/s	u/s	d/s	u/s	d/s
1.	0.407	10.310	-1.060	2.481	0.070	3.123
3	1.839	8.363	-1.074	9.213	-0.260	8.831
5	1.109	13.636	-1.022	14.702	-0.4657	14.377
7	0.820	19.226	-1.077	20.171	-0.657	19.908
9	0.353	24.836	-1.180	25.635	-0.844	25.434
11	-0.1849	30.433	-1.310	31.145	-1.029	30.958

(B) RESERVOIR EMPTY CONDITION

Section	Case 1		Case 3		Case 7	
	u/s	d/s	u/s	d/s	u/s	d/s
1	3.572	7.265	2.1047	-0.564	3.235	0.078
3	10.216	0.314	7.303	1.1643	8.117	0.782
5	14.698	0.583	12.567	1.649	13.123	1.325
7	19.373	1.170	17.724	2.119	18.144	1.852
9	24.171	1.777	22.032	2.593	23.169	2.375
11	29.040	2.3709	27.913	3.082	28.196	2.896

Note = + compressive stress and - Tensile stress.

The modal deflections are found to be increased at and near the top when the crest weigh is reduced as compared to the solid crest vide figures 5.9 and 5.10 . The total dynamic deflections are increased at and near the top in other cases as compared to case I for reservoir full condition. For reservoir empty condition the deflection all the cases are reduced at and near the top as compared to case 1.

The dynamic shear and moment diagrams are given in Figures 5.11 and 5.12 for empty reservoir and full reservoir conditions respectively It can be observed from these figures that the dynamic shears are reduced at and near the top but they are found to be increased at and near the base as compared to case 1 for reservoir full condition (Fig. 5.12) For reservoir empty condition also (Fig. 5.11) the shears are reduced at and near the top as compared to case 1. The reduction now is large and occurs upto greater distance measured from the top as compared to reservoir full condition. At and near the base the largest shear occurs in case 1 as compared to other cases (Fig. 5.11).

The dynamic moments are reduced at and near the top as compared to case 1, while at and near the bottom they are found to be somewhat increased for reservoir full condition (Fig. 5.11). For reservoir empty condition however the moments are reduced throughout the height of the dam as compared to case 1 (Fig. 5.11).

The (combined) compressive principal stresses at the upstream face are more or less the same for reservoir full condition (Fig. 5.13a) . The tensile principal stresses are mainly due to the dynamic effect of earthquake. A considerable

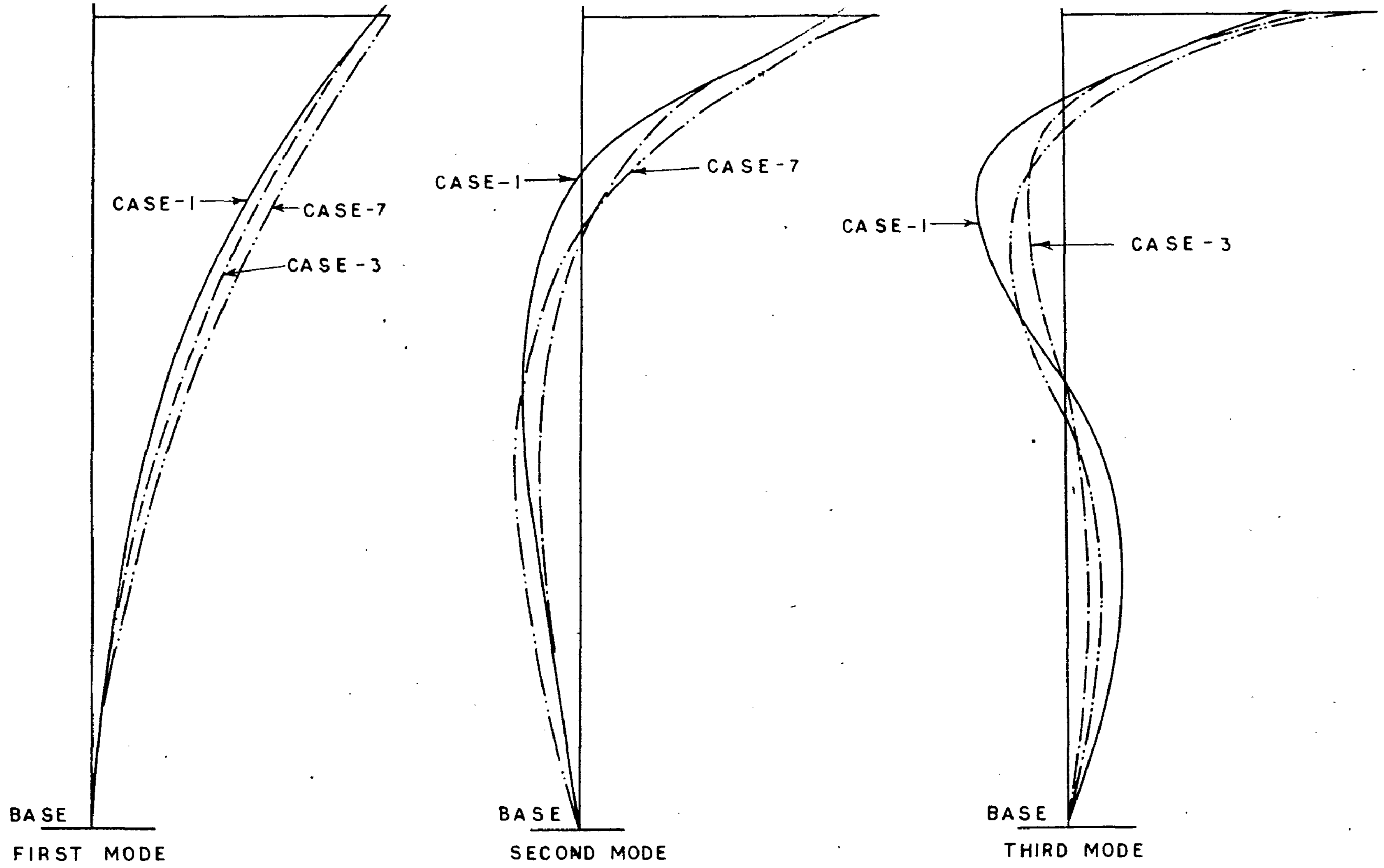


FIG. 5·9_MODE SHAPES FOR RESERVOIR FULL CONDITION

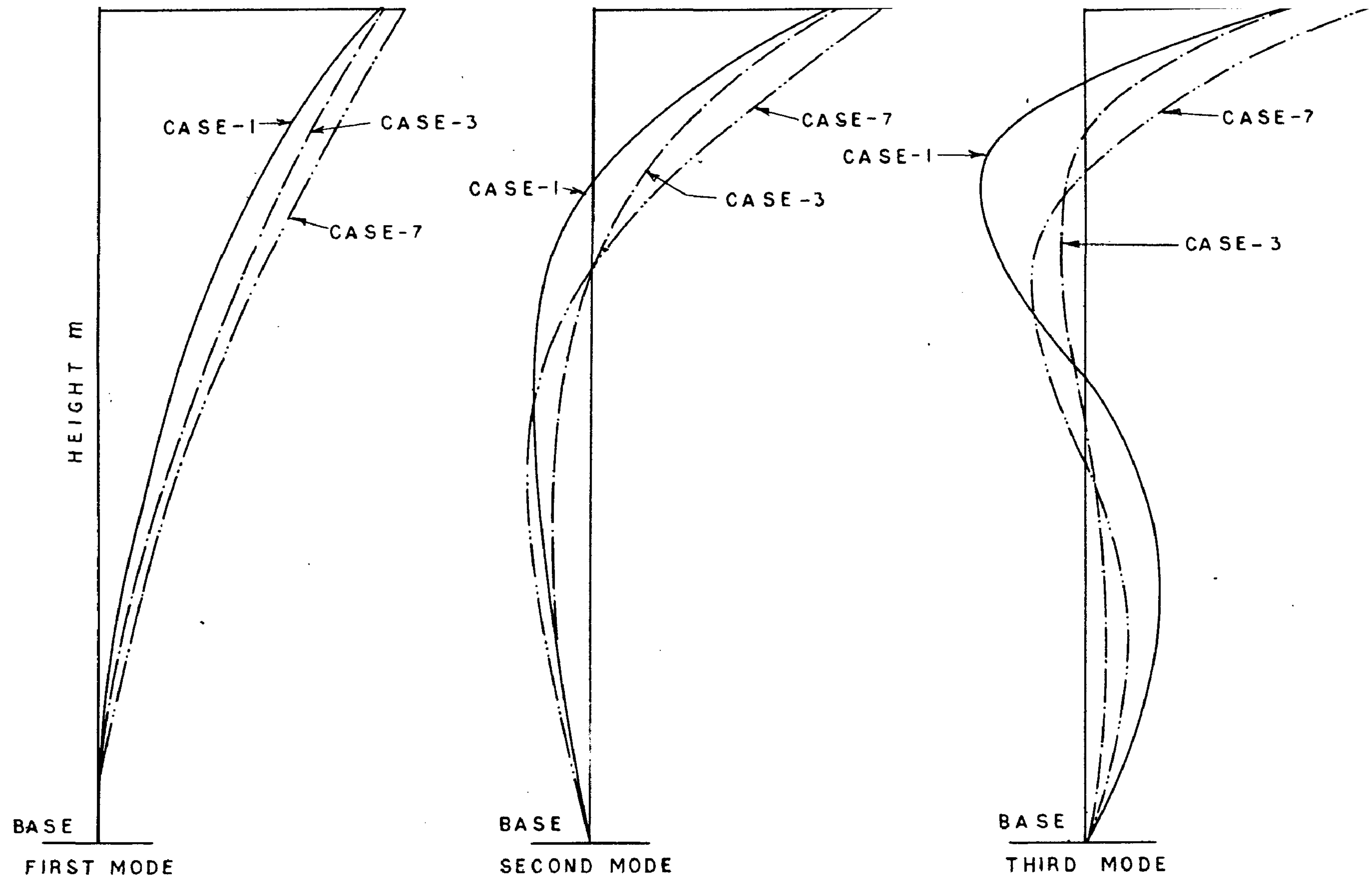


FIG. 5·10 — MODE SHAPES FOR EMPTY RESERVOIR CONDITION

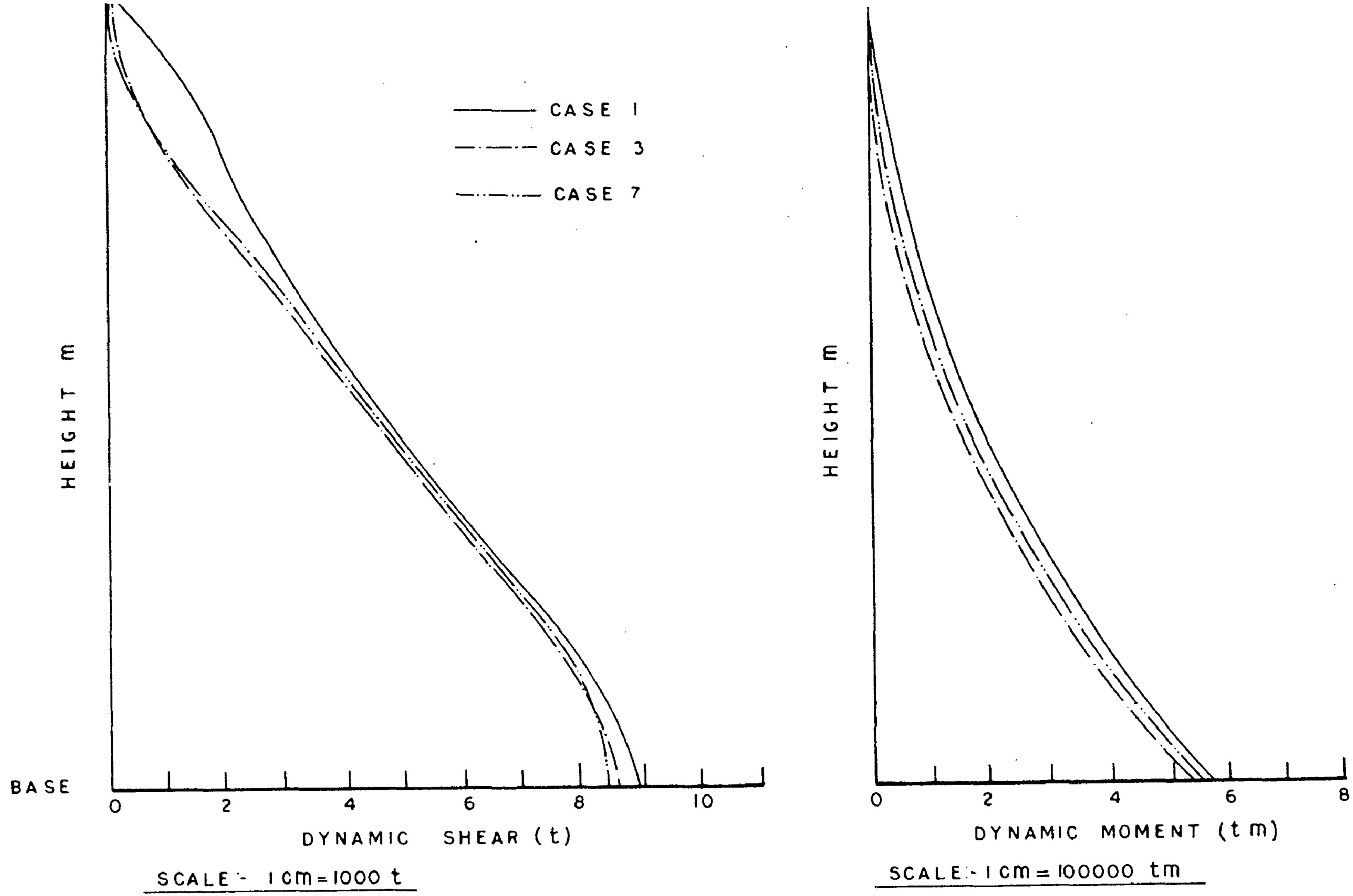
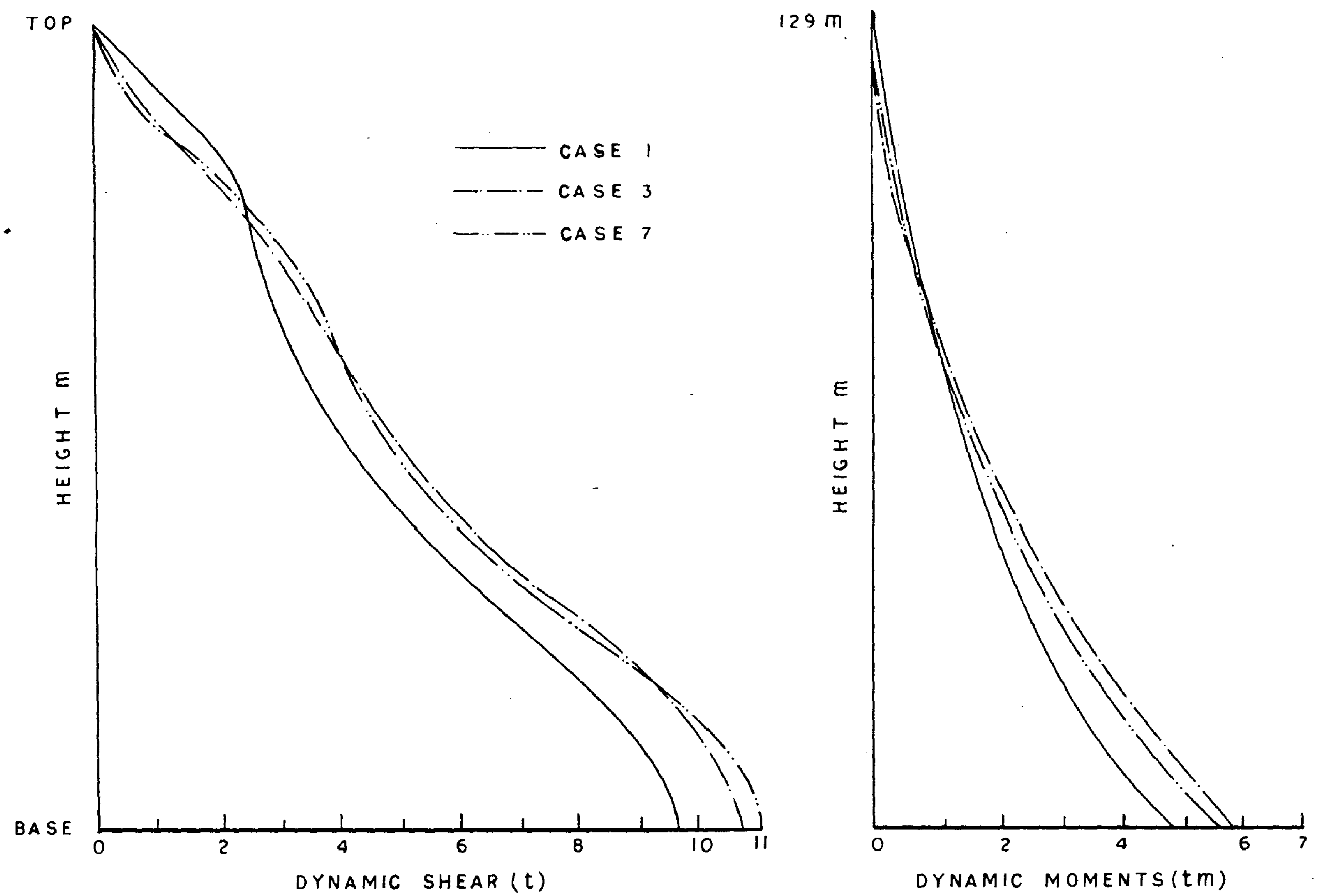


FIG. 5·11_DYNAMIC SHEAR AND MOMENT FOR VARIOUS CASES DUE TO KOYNA EARTHQUAKE FOR EMPTY RESERVOIR CONDITION



SCALE :- 1cm = 1000 t

SCALE :- 1cm = 100000 tm

FIG. 5·12 _ DYNAMIC SHEAR AND MOMENT FOR VARIOUS CASES DUE TO KOYNA EARTH QUAKE FOR FULL RESERVOIR CONDITION

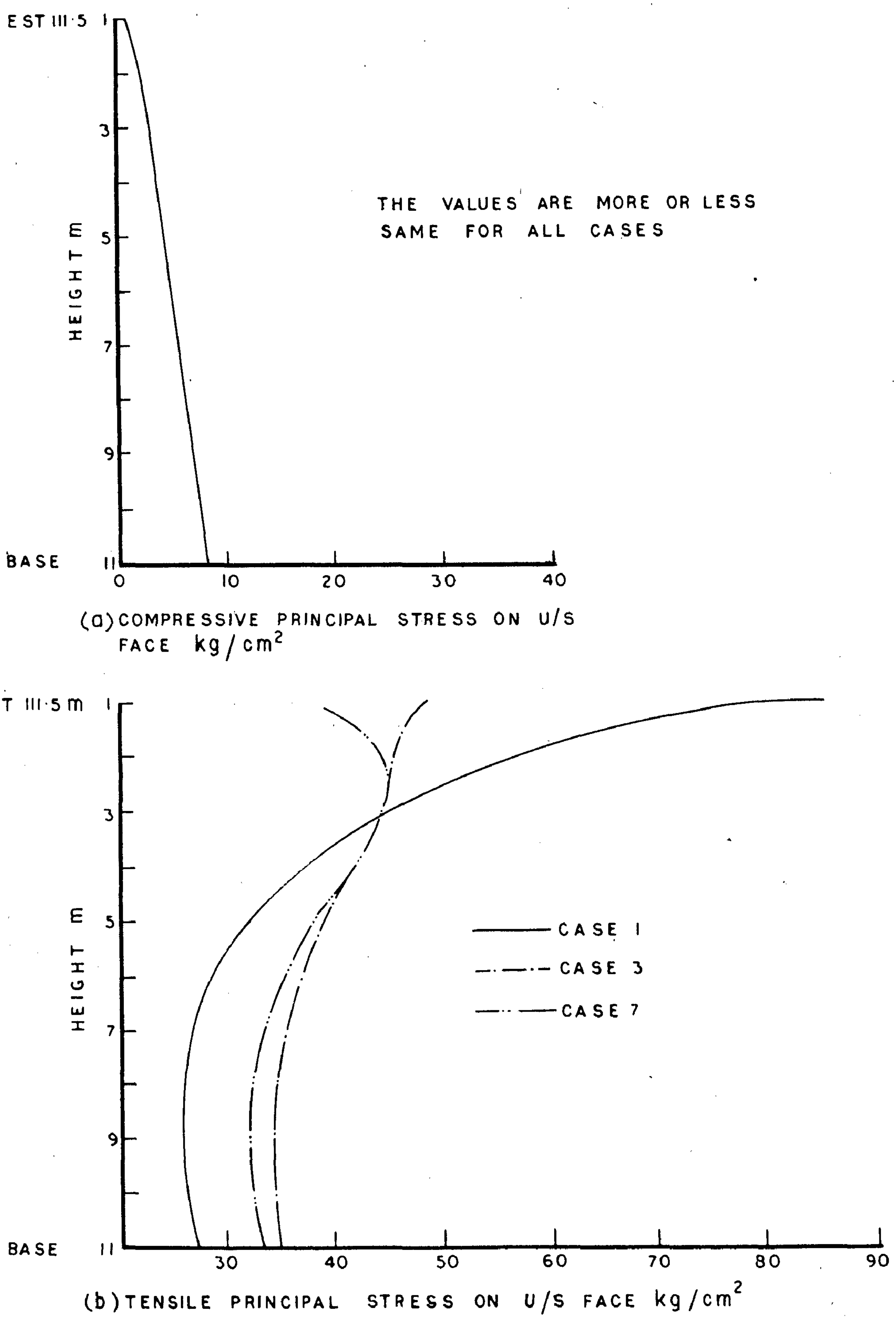


FIG. 5·13 — COMBINED PRINCIPAL STRESSES FOR VARIOUS CASES DUE TO KOYNA EARTH QUAKE FOR FULL RESERVOIR CONDITION.

reduction in tensile stresses near the crest is seen in other cases as compared to case -1 (5.13b).

For empty reservoir condition, the tensile principal stresses at the downstream face are mainly due to earthquake dynamic effect (Fig.5.14b) . The maximum compressive princip-al stresses are developed at the upstream face (Fig. 5.14a). Near the crest the compressive principal stresses are considerably smaller in other cases as compared to case 1(Fig. 5.14a) The same variation also holds good for tensile stresses at the downstream fact (5.14b). The differences in the stresses at the upstream and downstream faces at the lower sections in all the cases considered are considerably smaller as compared to the differences near the crest.

Thus from the analysis[52] presented above it may be seen that it is worthwhile to replace the top of the dam by a light weight structural system.

5.5 IMPORTANCE OF AVOIDING ABRUPT CHANGES IN SLOPES

The analysis of Koyna dam NOF monolith indicates large tensile stresses around EL 56.50 m where the down-stream slope changes abruptly and at the heel of the dam[36]. This indicates that it would be desirable to avoid abrupt changes in the slope of the face of the dam. This fact is also confirmed by the earlier study[39] in which it was found that there is a concentration of stresses near the

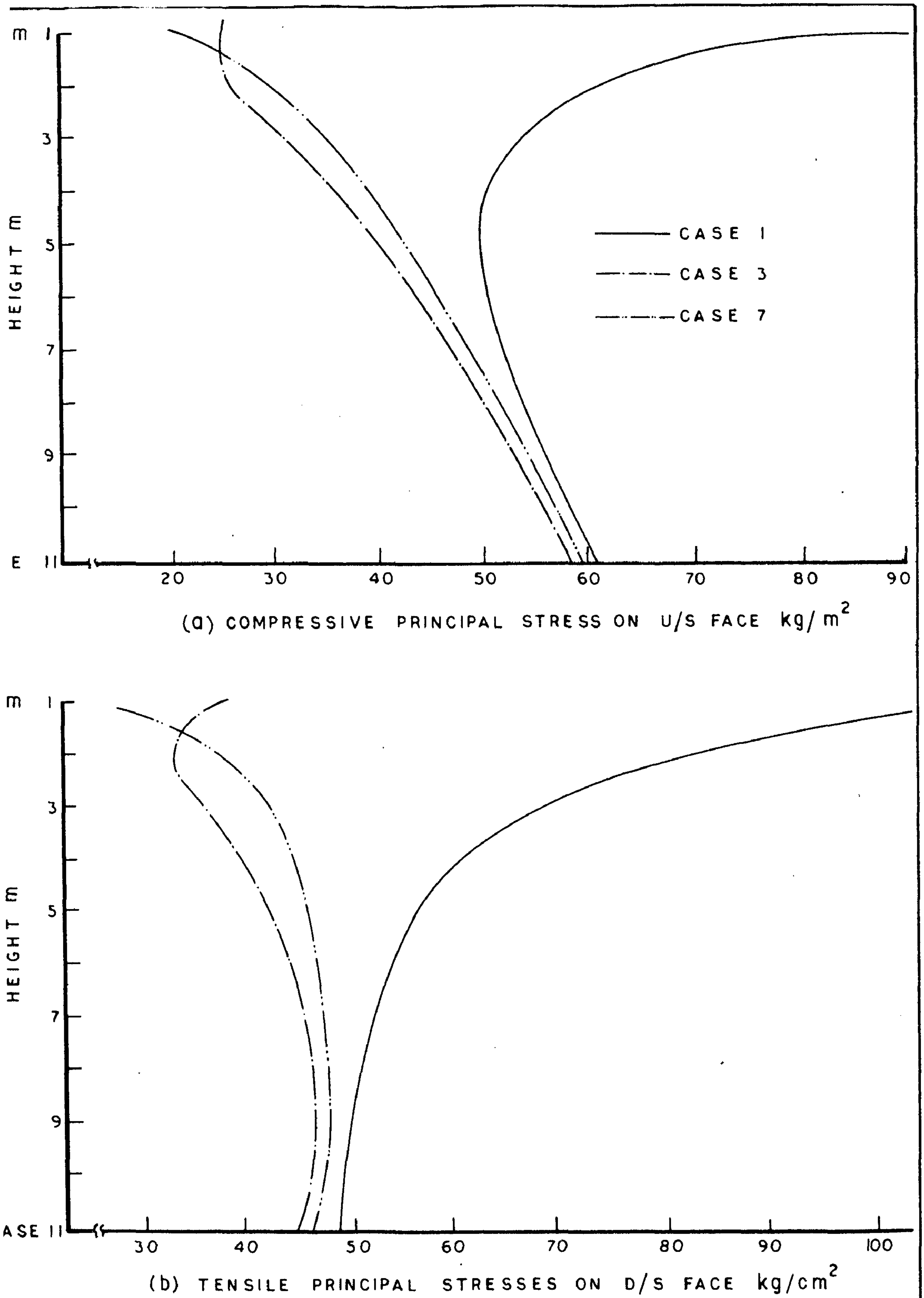


FIG. 5·14 _ COMBINED PRINCIPAL STRESSES FOR VARIOUS CASES DUE TO KOYNA EARTHQUAKE FOR EMPTY RESERVOIR CONDITION

RL 66.50 m where the dam slope changes abruptly and maximum tensile stresses occur at this point. The dynamic analysis in case of Kālkovadi dam[50] also confirms this point. Therefore it is better to avoid any abrupt changes in the downstream slope of the dam. If at all there is any change in slope proper transitions or fillets may be provided at the junction of the vertical portion of the dam and the downstream slope of the dam.

5.6 EFFECT OF SLOPING UPSTREAM FACE OF DAM

The conventional profile of gravity dam has the upstream face vertical or nearly vertical. Adequacy of such a profile from the point of view of its response to dynamic loading such as earthquake is a problem. The stresses due to an earthquake are of an alternating nature. Therefore it is sometimes considered desirable to have some slope on the upstream face also. This can be investigated by comparing the response of an original dam and a modified dam obtained by removing concrete from the downstream face and adding it on the upstream face, so that the total volume of concrete remains the same. It will be observed that the dynamic response in the two sections when obtained by using beam theory would be the same. Thus it was concluded[56] that the conventional profile would be adequate under earthquake condition also.

Arya and Agrawal[58] studied this point further. The responses of the dam with downstream slope 0.85 :1 and the upstream face vertical with those for the dam with downstream slope 0.85:1 and upstream slope 0.0625 :1 were studied for three heights of dam (100 m , 150 m and 200m). A typical case for a 200 m dam profile is given in Fig. 5.15, from which it would be clear that there is a reduction in the tensile stress in the case where the upstream face is sloping (0.0625:1) The same trend is seen in all other heights of dam. It is therefore concluded that the upstream face of the dam may be kept sloping instead of vertical.

5.7 ADEQUACY OF THEORETICAL TECHNIQUES

The adequacy of theoretical techniques in predicting the behaviour of a gravity dam during an earthquake can be seen by comparing the actual behaviour of a dam during an earthquake and its theoretically predicted behaviour. The Koyna earthquake of Dec 11, 1967 provided an excellent opportunity for study because there a moderately high (103 m) gravity dam was subjected to a severe earthquake whose epicentre was very close to the dam. During this earthquake the higher non over flow monoliths suffered severe distress (cracking) whereas the non over flow monoliths had escaped serious damage. The dynamic analysis of the NOF and OF sections of Koyna dam was carried out by both the methods[36].

The maximum tensile stress in the region around EL 66.5 m is of the order of 33.6 kg/cm^2 on the upstream face and is about 25.0 kg/cm^2 on the downstream face[36]. If the dynamic tensile strength of concrete[53] is assumed

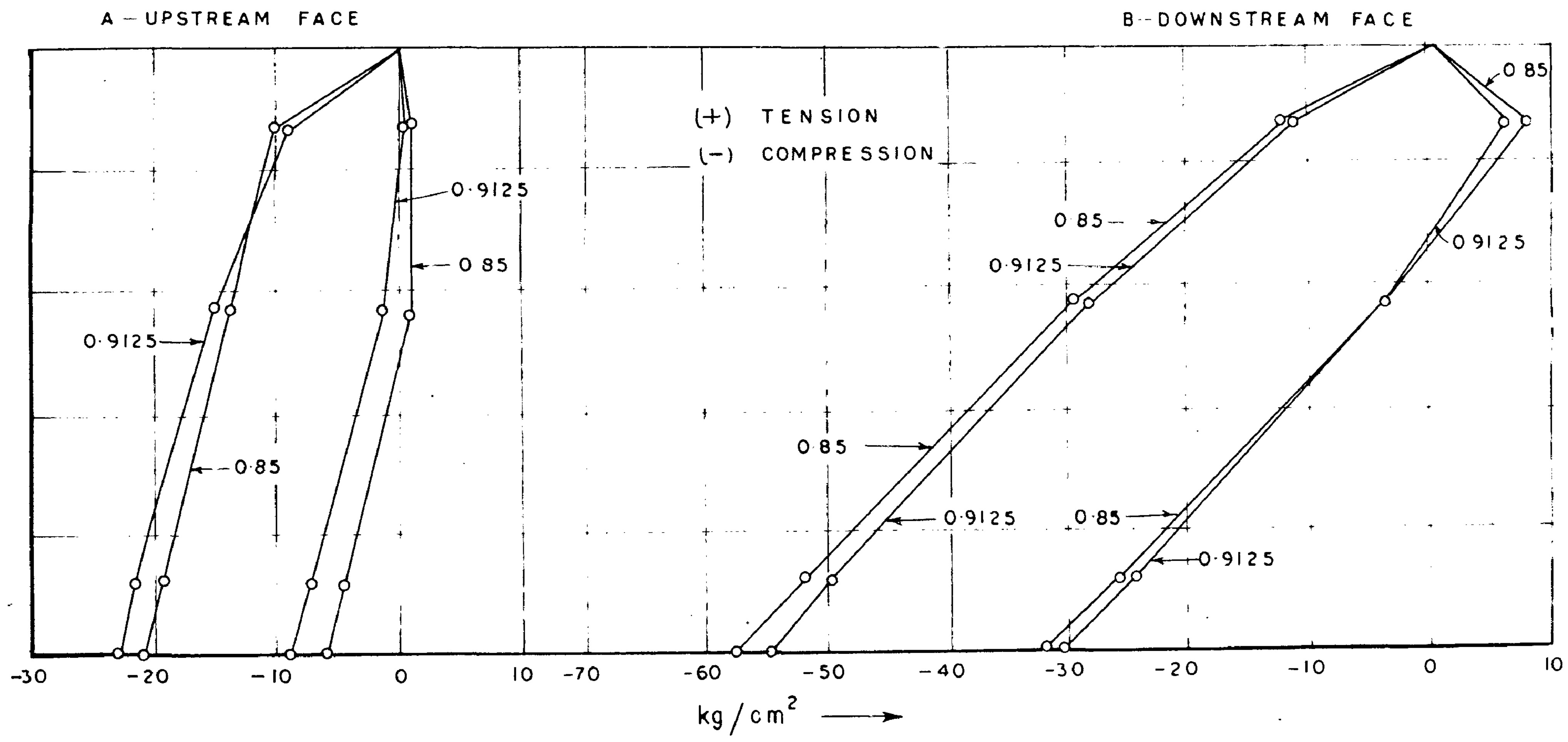


FIG. 5·15 — VARIATION OF PRINCIPAL STRESSES IN 200 m DAM FOR LOADING CONDITION OF FULL RESERVOIR PLUS EARTHQUAKE to be about 30 kg/cm^2, it would be quite reasonable to expect cracking in the dam. The observed behaviour of the koyna dam shows th t there was an approximately horizontal crack at EL 66.5 on the downstream side of the NOF monoliths. The existance of horizontal cracks is indicated in the region EL 60.00 m to EL 73.00 m especially around EL 66.50 m on upstream side. This indicates that structural cracking of the dam has occurred in the region around EL 66.50 m both on the upstream and downstream which is as it should be expected from the theoretically calculated behaviour of the dam. Thus the theoretical techniques get proved.

The model of the highest NOF section of Koyna dam was tested on shake table. The natural period of vibration of dam in its first mode was experimentally calculated at 0.338 seconds which as found from theoretical analysis by beam method and finite element method was 0.340 seconds and 0.355 seconds respectively. Thus the theoretical analysis once again gets proved. Steady state forced vibration tests on the model of NOF section were carried out. The cracks in the model appeared at the neck where the downstream slope changes abruptly which was expected by theoretical techniques and proved by the actual behaviour of the dam during earthquake of Dec 11, 1967. Thus the theoretical techniques are adequate in predicting the behaviour of gravity dam during earthquake. The experimental technique should be used to verify the theoretically calculated behaviour.

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CHAPTER VI

CONCLUSIONS

The present design procedure which is based on an equivalent seismic coefficient is inadequate in the seismic design of concrete gravity dams. The IS code of 1974 is definitely an improvement in the right direction over the earlier codes (1966, 1970) as the dynamic moments and shears calculated on the basis of this code are very near to those obtained by dynamic analysis. Therefore the IS code of 1974 can be used for preliminary design of major dams. For final design however, detailed dynamic analysis should be carried out. The dynamic testing should be used to verify the results of the theoretical analysis.

The hydrodynamic moment and shear formulae as specified by IS code (1966, 1970 and 1974) are not correct. The hydrodynamic moments as given by the formula in IS code overestimates the value by as high as 54 per cent at 10 per cent depth from water surface to as low as 2.2 per cent at bottom of reservoir. The corresponding overestimation in case of hydrodynamic shear is 15 per cent at 10 per cent depth to 0 per cent at bottom of reservoir. The correct distribution curve for hydrodynamic moment and shear given in Fig. 4.12 should be used.

The effect of vertical ground motion is negligible if the hydrodynamic interaction effects are neglected. If the interaction effects are included the vertical ground motion causes an appreciable increase of about 40 per cent in crest displacement and 40 to 70 per cent in the critical tensile stresses.

The structures like elevated towers at the top of the dam should be avoided. The top of the dam should be provided with a light weight structural system in order to reduce the tensile stresses in the upper region.

Abrupt changes in the slope of the dam should be avoided by providing proper transitions or fillets so that concentration of tensile stresses is avoided.

The upstream face of the dam may be kept sloping instead of vertical so as to reduce the tensile stresses at top.

The theoretical techniques are adequate in predicting the behaviour of dams subjected to earthquake. The experimental technique should be used to verify the results of the theoretical analysis.

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