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SEISMIC RESISTANCE OF POCHAMPAD MASONRY DAM

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INTRODUCTION

Pochampad dam is a masonry gravity dam with earthen flanking bunds and is now in advance stage of construction. The dam was originally not designed for resisting earthquake forces as its site was considered to be non-seismic at the time of planning and design. Due to the recent seismic activity in the peninsular India, the seismic zoning of the country has been revised taking into account the tectonic features and also the three important earthquakes viz. Koyna earthquake (1967), Kothagudem earthquake (1969) and Broach earthquake (1970) which have been extensively investigated. As per the revised seismic zoning of India, Pochampad dam site lies on the periphery of seismic zone I and II. The significant tectonic feature is a fault oriented in NNW—SSE direction known as "Godavari Rift" and it lies about 150 Km. to 300 Km. to the east of the dam site. The recent Kothagudem (1969) earthquake which was widely felt within a radius of 650 Km. can be attributed to this tectonic feature. There is no record of actual seismic intensity experienced at the dam site during this earthquake but on the basis of the available data of known neighbouring places, the estimated seismic intensity experienced was between IV and V on Modified Mercalli Intensity scale. The Pochampad dam site lies near the periphery of the Deccan trap area in the peninsular India in which Koyna dam is also located. Due to the close distance of Godavari rift from Pochampad dam site and somewhat similar geological conditions prevailing at Pochampad and Koyna dam sites, a Koyna type shock at Pochampad can be expected though not of the same intensity. Therefore in view of the recent revision of the seismic zoning map of the country and also considering the close distance of the Pochampad dam from the fault zone, it is considered necessary to study the seismic resistance of the dam. The dam will be impounding a very big reservoir of capacity 3170 M. cu. m. having a water spread of 453 sq. km. The stability of such a dam under earthquake conditions must be ensured in view of the catastrophic consequences due to the failure of such a dam.

This paper deals with the estimation of seismic resistance of masonry dam allowing nominal tension and also permitting ultimate tensile stresses under worst earthquake conditions. Stress Analysis of the highest non-overflow and overflow monoliths of the dam under static and earthquake loads has been carried out using conventional gravity analysis. For this purpose, the distribution of earthquake forces along the height of the dam have been taken as those specified in I. S: 1893—1970[1]. The strength of masonry has been estimated knowing the strength of mortar used in the dam and other available literature. Permitting nominal tension, seismic resistance is evaluated and compared with the recommended provisions of I. S. Code[1]. Under worst earthquake conditions, the strength of the dam is also estimated permitting ultimate tensile stresses considering a Koyna type earthquake using approximate method.

It is concluded that the masonry dam is marginally safe considering earthquake

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forces permitting nominal tensile stress under design conditions. Under worst earthquake conditions by permitting ultimate tensile stress in masonry, the dam can resist an earthquake with peak ground acceleration of 16 percent of gravity which is equivalent to 25 percent of Koyna type shock. This motion can be expected at the dam site from an earthquake originating from the rift with Richter magnitude of about 7·5 with epicentre 50 km. away and a focal depth of 33 km. The earthquakes originating from the epicentre of that of Kothagudem (1969) shock will, however, cause negligible ground motion at the dam site.

NUMERICAL DATA AND STRESS ANALYSIS

Pochampad masonry dam is a straight gravity dam with vertical transverse contraction joints in between the monoliths and these joints are not to be grouted. Therefore it is analysed considering it as a cantilevered structure and each monolith is assumed to act independently of its neighbouring monoliths. For the stress analysis, two highest monoliths, viz., one nonoverflow and the other overflow, are chosen since it is known that higher monoliths have the least seismic strength and thus, are more vulnerable to damage during earthquakes[2]. Figs. 1 and 2 show the highest non-overflow and overflow

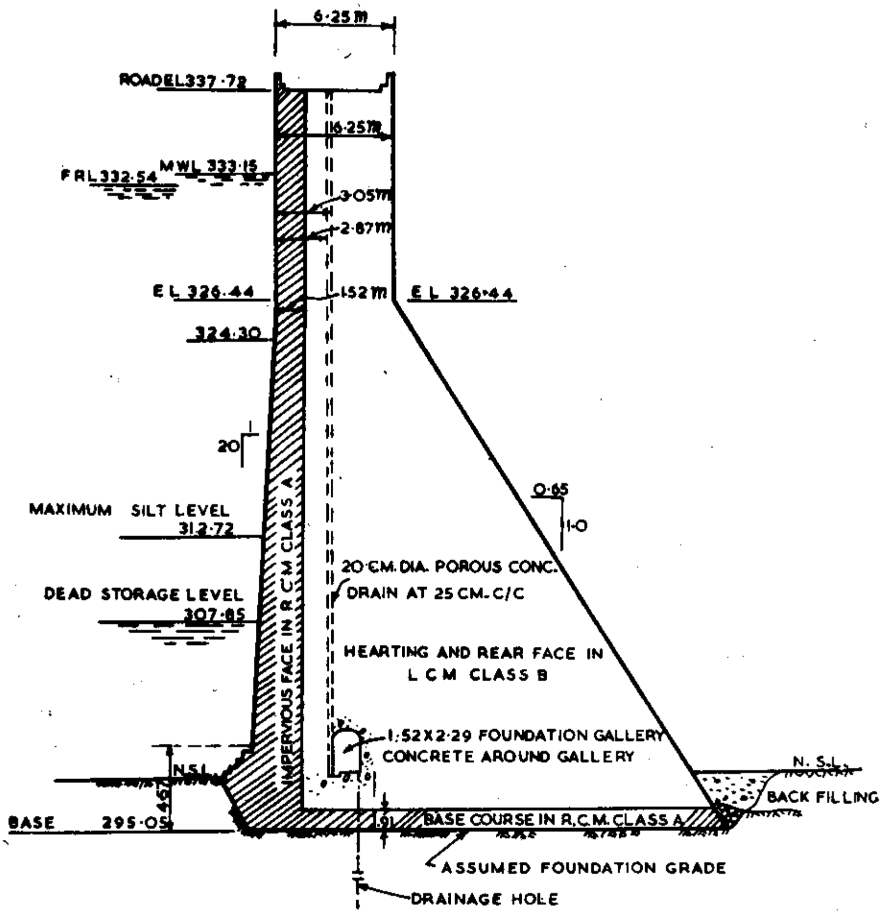

Fig. 1. Highest nonoverflow monolith of Pochampad masonry dam

monoliths of the dam which are 42.67 m and 27.4 m high respectively. The dam is being constructed of random rubble masonry in the hearting and coursed rubble stone masonry on the upstream face of nonoverflow and spillway blocks and downstream face

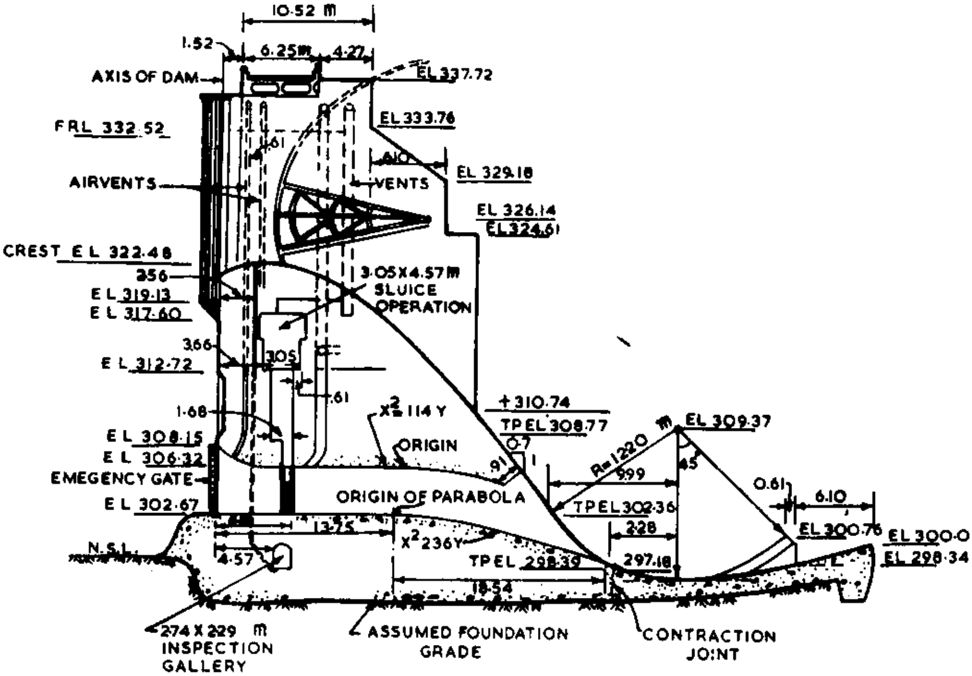

Fig. 2. Highest overflow monolith of Pochampad masonry dam

of nonoverflow blocks only. In the hearting portion of the dam, a lean cement mortar (class B mortar) is being used. For a thickness of 1.5 m from the upstream face and 0.9 m above the foundation, a rich cement mortar (Class A mortar) is used. The specifications for the mortar being used in the dam are as given in Table. 1.

TABLE 1

COMPRESSIVE STRENGTH OF MORTAR

Sl. No.	Type of mortar	Mix proportion by weight	Minimum compressive strength	
			28 days	1 year
1	Class A	1 : 2.75	140.6 kg/cm². (2000 p.s.i.)	175.8 kg/cm² (2500 p.s.i.)
2	Class B	1 : 4 (20% Surkhi admixture)	105.4 kg/cm² (1500 p.s.i.)	140.6 kg/cm² (2000 p.s.i.)

In the spillway blocks, 0.9 m glacis concreting is provided on the downstream face and suitable dowel bars are embedded to have proper bond between masonry and glacis concrete.

Stress analysis of the highest monoliths using gravity analysis has been carried out under static and earthquake forces. Principal stresses at upstream and downstream face of the dam at selected elevations for reservoir full and reservoir empty conditions have been determined. Internal stress distribution is not determined since the maximum stresses occur at the faces and these are needed for the estimation of seismic resistance. Following forces are considered in the stress analysis:

1. Weight of dam
2. Reservoir water pressure
3. Uplift pressure
4. Forces due to waves on non-overflow monolith
5. Equivalent weight of superstructure on overflow monolith.
6. Silt pressure upto dead storage level
7. Earthquake forces.

Following data is taken for the stability analysis.

Unit weight of masonry $=2.4$ tonne/m^3
Unit weight of water $=1.0$ tonne/m^3
Unit weight of submerged silt $=0.895$ tonne/m^3
Angle of repose of submerged silt $=30°$
Allowable shear strength of masonry $=280$ tonne/m^2
Modulus of elasticity of masonry $=1.4\times10^6$ tonne/m^2
Coefficient of internal friction $=0.7$
Weight of spillway gate $=100$ tonne each gate
Weight of spillway bridge $=16.6$ tonne per m
Fetch of reservoir $=32$ km.

Uplift forces occur as internal pressures in pores, cracks and seams in the dam foundation. Drains are provided in the dam to reduce the magnitude and to change the distribution of uplift pressures. In the present study, the uplift pressure has been taken equal to 100% water pressure at the upstream face reducing linearly to 33.3% at the line of drainage holes and further reducing linearly to zero at the downstream face.

Wind blowing over the reservoir causes a drag on water surface. The effect of this drag is to pull the top surface along the direction of wind and thus ripples and waves are formed which cause additional pressure on the dam section. The following formula is used to determine the wave height.

$$h_w=0.17\sqrt{VF} \qquad \ldots(1)$$

where

F is Fetch in miles, V the wind velocity in miles per hour and h_w the height of wave in feet.

The pressure intensity, P_w, induced by the waves is given by

$$P_w=2.4\, w\, h_w \qquad \ldots(2)$$

and the total pressure acting in the horizontal direction is given by

$$P=2.0\, w\, h_w^2 \qquad \ldots(3)$$

Where w is the density of water. The centroid of the pressure diagram is taken at 3/8 h_w above the reservoir level.

Earthquake forces depend upon the characteristics of the ground motion and also on the properties of the dam section and can be estimated by a dynamic analysis[3,4,5]. For this, a knowledge of the precise wave-form of the ground motion record is needed. Since no recorded ground motion data are available at the dam site, for the purpose of estimation of seismic resistance, the distribution of earthquake forces have been taken as those specified in Indian Standard Code[1]. The variation of seismic coefficient along the height of the dam is shown in fig. 3. Horizontal and vertical seismic forces are taken

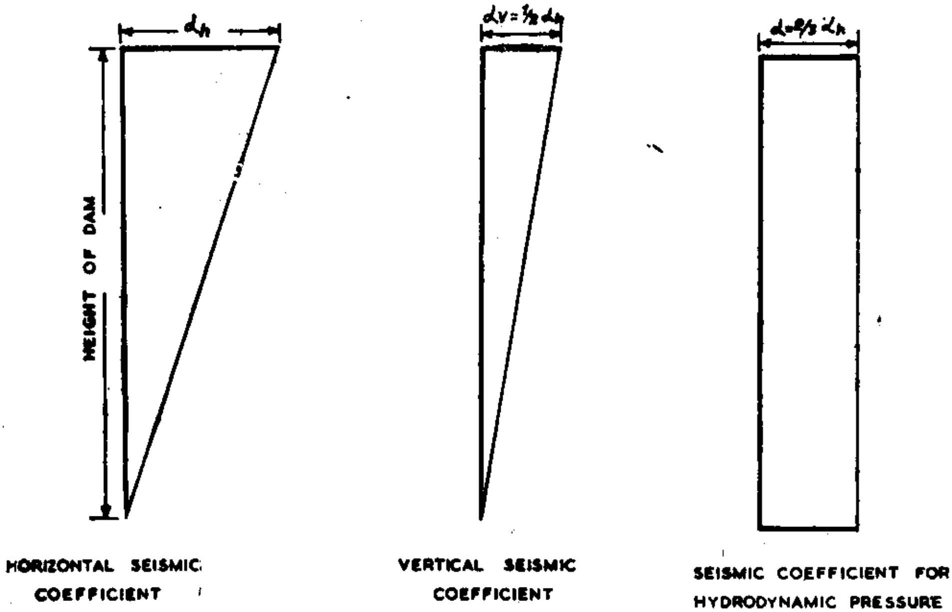

Fig. 3. Indian standard design seismic coefficient distribution for dams

into account and acting simultaneously. The vertical seismic coefficient is taken as half of the horizontal coefficient. The hydrodynamic pressure has been taken as recommended in the I.S. Code [1].

Assuming a linear variation of vertical normal stress, stress analysis of the monoliths has been carried out using gravity analysis. Vertical normal stresses at upstream and downstream face have been calculated. From these, the principal stresses at the faces are obtained using the following expressions :

$$\left.\begin{aligned} \sigma_{1U} &= \sigma_{ZU} \sec^2 \phi_u - p_u \tan^2 \phi_u \\ \sigma_{2U} &= P_U \end{aligned}\right\} \quad \ldots(4)$$

$$\left.\begin{aligned} \sigma_{1D} &= \sigma_{ZD} \sec^2 \phi_D - p_D \tan^2 \phi_D \\ \sigma_{2D} &= p_D \end{aligned}\right\} \quad \ldots(5)$$

Where σ_1 and σ_2 are the major and minor principal stresses, σ_z is the vertical normal stress on horizontal plane, p the normal water and silt pressure, ϕ the angle between the face of the dam and the vertical and the subscripts U and D indicate values on upstream and downstream face respectively.

The stresses have been calculated at four sections in the dam and the final results are given in Tables 2 and 3. The stresses due to earthquake are determined in terms of α_h, the horizontal seismic coefficient at the top of the dam.

PROPERTIES OF DAM MASONRY

In order to estimate the seismic resistance of the dam, the strength of dam masonry is required. No tests have been carried out on masonry at the project site. Therefore, the strength of masonry is estimated knowing the compressive strength of mortar used in the dam. Compressive strength tests on mortar using 15.24 cm (6") cubes are carried out and the specified strengths are given in Table 1. It can be seen from the table that the compressive strength of rich mortar (class A mortar) is 140.6 kg/cm^2 (2000 p.s.i.) and 175.8 kg/cm^2 (2500 p.s.i.) at 28 days and one year respectively. The strength of lean mortar (class B mortar) is 105.4 kg/cm^2 (1500 p.s.i.) and 140.6 kg/cm^2 (2000 p.s.i.) at 28 days and one year respectively.

Studies carried out at Hirakud project[6,7] on the compressive strength of masonry and mortar indicate that 180 days compressive strength of masonry is about 108 percent to 132 percent of that of mortar. Therefore, in the present study, it is assumed that the compressive strength of masonry is atleast equal to the strength of lean mortar (class B mortar). Hence the compressive strength of dam masonry is taken as 140.6 kg/cm^2 (2000 p.s.i.)

At Koyna project, studies have been carried out to know the relationship between tensile and compressive strengths of masonry. These studies [6, 8] reveal that one year tensile strength of masonry varies from 7.0 percent to about 8.3 percent (say 7.5 percent average) of its compressive strength. Therefore, in the present study, the tensile strength of masonry is taken as 7.5 percent of its compressive strength. Thus, the tensile strength of dam masonry is estimated as 10.5 kg/cm^2 (150 p.s.i.)

Under dynamic loading conditions, the structure is subjected to high rates of strain. Under these conditions, the dynamic strength of the material is known to be larger than the static strength[13]. However, sufficient data is not available about the dynamic increase factor for masonry under rapid strain rates. Thus the tensile strength of masonry under dynamic conditions is taken the same as under static conditions, that is, 10.5 kg/cm^2 (150 p.s.i.)

ESTIMATION OF SEISMIC STRENGTH

Knowing the strength of masonry and using the results of stress analysis presented in Tables 2 and 3, the seismic resistance of the dam can be evaluated. Under design conditions, a nominal tensile stress of about 1.0 percent of the ultimate compressive strength or 10.0 percent of the ultimate tensile strength is generally allowed in the dam design. The ultimate compressive strength of dam masonry has been estimated earlier as 1406 tonne/m^2 (2000 p.s.i.) and the ultimate tensile strength as 105 tonne/m^2 (150 p.s.i.). Therefore a nominal tension of 10.5 tonne/m^2 (15 p.s.i.) has been allowed under nominal earthquake conditions. As per ASCE–USCOLD practice [9] also, a nominal tension of 10.5 tonne/m^2 (15 p.s.i.) is allowed in dams.

The seismic resistance of the dam has been evaluated permitting nominal tension and also by permitting ultimate tensile stress under worst earthquake conditions and the results are presented in tables 2 and 3. It is noted from table 2 that by permitting nominal tension, the minimum seismic resistance of the non-overflow monolith in terms of α_h, the horizontal seismic coefficient at the top of the dam, is 0.061 under reservoir full condition and 0.093 under reservoir empty (dead storage level) condition. Similarly from Table 3, it is noted that by permitting nominal tension, the minimum seismic resistance of the overflow monolith is 0.172 under reservoir full condition and 0.201 under reservior

TABLE 2*

PRINCIPAL STRESSES AND SEISMIC STRENGTH OF NONOVERFLOW MONOLITH

Elevation	Stress	RESERVOIR FULL CONDITION				RESERVOIR EMPTY CONDITION			
		PRINCIPAL STRESSES		SEISMIC STRENGTH		PRINCIPAL STRESSES		SEISMIC STRENGTH	
		Upstream Face	Downstream Face	Permitting nominal tension	Permitting ultimate tension	Upstream Face	Downstream Face	Permitting nominal tension	Permitting ultimate tension
326.44	σ_1	$14.55 \mp 157.50\alpha_h$	$34.60 \pm 134.30\alpha_h$	0.159	0.757	$27.10 \pm 137.33\alpha_h$	$27.10 \mp 137.33\alpha_h$	0.274	0.906
	σ_2	$7.42 \mp 7.57\alpha_h$	0.0			0.0	0.0		
315.05	σ_1	$20.11 \mp 162.43\alpha_h$	$48.20 \pm 199.15\alpha_h$	0.188	0.772	$61.30 \pm 124.03\alpha_h$	$3.06 \mp 144.8\alpha_h$	0.093	0.747
	σ_2	$18.81 \mp 13.94\alpha_h$	0.0			0.0	0.0		
305.05	σ_1	$15.74 \mp 171.97\alpha_h$	$76.30 \pm 213.38\alpha_h$	0.152	0.702	$78.39 \pm 120.70\alpha_h$	$3.74 \mp 140.5\alpha_h$	0.101	0.776
	σ_2	$31.57 \mp 16.96\alpha_h$	0.0			$3.81 \pm 3.1\alpha_h$	0.0		
295.05	σ_1	$0.00 \mp 170.77\alpha_h$	$122.50 \pm 216.22\alpha_h$	0.061	0.615	$85.17 \pm 114.21\alpha_h$	$17.67 \mp 135.5\alpha_h$	0.207	0.905
	σ_2	$45.15 \mp 17.85\alpha_h$	0.0			$17.40 \pm 6.1\alpha_h$	0.0		

* Stresses are presented in tonne/m^2. Compressive stresses have been marked as positive and tensile stresses as negative.

TABLE 3*

PRINCIPAL STRESSES AND SEISMIC STRENGTH OF OVER-FLOW MONOLITH

Elevation	Stress	RESERVOIR FULL CONDITION				RESERVOIR EMPTY CONDITION			
		PRINCIPAL STRESSES		SEISMIC STRENGTH		PRINCIPAL STRESSES		SEISMIC STRENGTH	
		Upstream Face	Downstream Face	Permitting nominal tension	Permitting ultimate tension	Upstream Face	Downstream Face	Permitting nominal tension	Permitting ultimate tension
315.67	σ_1	$3.11 \mp 79.20\alpha_h$	$138.80 \pm 272.0\alpha_h$	0.172	1.365	$27.50 \pm 52.80\alpha_h$	$61.20 \mp 154.0\alpha_h$	0.466	1.080
	σ_2	$16.86 \mp 13.99\alpha_h$	0.0			0.0	0.0		
308.87	σ_1	$13.30 \mp 116.50\alpha_h$	$52.30 \pm 138.0\alpha_h$	0.203	1.015	$51.60 \pm 77.50\alpha_h$	$11.90 \mp 81.30\alpha_h$	0.276	1.440
	σ_2	$23.66 \mp 16.36\alpha_h$	0.0			0.0	0.0		
301.96	σ_1	$17.65 \mp 137.25\alpha_h$	$71.50 \pm 168.15\alpha_h$	0.205	0.892	$62.70 \pm 96.0\alpha_h$	$11.25 \mp 106.5\alpha_h$	0.204	1.090
	σ_2	$34.44 \mp 17.79\alpha_h$	0.0			$8.0 \pm 4.84\alpha_h$	0.0		
295.05	σ_1	$17.90 \mp 150.52\alpha_h$	$91.34 \pm 191.05\alpha_h$	0.189	0.816	$81.0 \pm 104.0\alpha_h$	$14.0 \mp 122.0\alpha_h$	0.201	0.975
	σ_2	$43.83 \mp 18.24\alpha_h$	0.0			$17.40 \pm 6.25\alpha_h$	0.0		

* Stresses are presented in tonne/m^2. Compressive stresses have been marked as positive and tensile stresses as negative.

empty condition. As per the revised sesimic zoning map of India[1], the pochampad dam lies on the periphery of zone I and II. Aoccrding to the Indian Standard Code provisions, the design seismic coefficient for dams in zone II is 0.06. The minimum seismic strength of the dam in terms of the seismic coefficient is 0.061 and thus the dam can be considered marginally safe under the condition when nominal tension is permitted.

It is also noted from Tables 2 and 3 that by permitting ultimate tensile strength of the material, the minimum seismic strength of the non-overflow monolith in terms of seismic coefficient at top of dam is 0.62 and 0.75 for reservoir full and reservoir empty condition. Similarly for the overflow monolith, the minimum seismic strength under reservoir full and reservoir empty conditions is 0.82 and 0.98. The accelerations which will be acting at the top of the dam during a Koyna type shock can be estimated using an approximate method. Based on the analysis of several dams[3], the fundamental natural period of vibration of the dam can be obtained using the expression

$$T=2.0 \frac{H}{r_b} \frac{H}{V_L} \qquad \ldots(6)$$

where T is the fundamental natural period of vibration, H the height of dam, r_b the radius of gyration at the base of the dam, V_L is the longitudinal wave propagation velocity of dam material and is given by

$$V_L = \sqrt{\frac{Eg}{w_c}} \qquad \ldots(7)$$

were E is the modulus of elasticity of dam material, g the acceleration due to gravity and w_c the weight density of dam material.

Knowing the natural period of the dam, the spectral acceleration in the first mode can be obtained using response spectrum of the earthquake[10]. For the range of natural periods of interest, and damping equal to 10% of critical damping, the spectral displacement due to Koyna earthquake is given in Table 4.

TABLE 4

SPECTRAL DISPLACEMENT DUE TO KOYNA EARTHQUAKE

Sl. No.	Natural Period in Second	Spectral displacement in cm
1.	0.05	0.082
2.	0.07	0.132
3.	0.10	0.215
4.	0.15	0.523
5.	0.20	0.986

The spectral acceleration can be obtained using the expression:

$$S_a = p^2 S_d \qquad \ldots(8)$$

where S_a is the spectral acceleration, S_d the spectral displacement, p the natural frequency and is equal to $2\pi/_T$.

Based on the analysis of several dams[8], the horizontal acceleration at the top of the dam can be approximately taken equal to 2.5 times the spectral acceleration in the fundamental mode. Then a strength factor has been evaluated which gives the ratio of seismic resistance of the dam to the seismic coefficient expected due to Koyna shock. This factor gives approximately the fraction of peak ground acceleration of Koyna shock that the dam can withstand.

Knowing the properties of the dam section and using Eqs. 6, 7 and 8, following results are obtained for the non-overflow and overflow sections due to Koyna shock.

(a) Non-overflow Section :

Natural period of vibration	=0.187 sec.
Spectral displacement	=0.87 cm
Spectral acceleration	=0.996 g
Acceleration at top of dam	=2.49 g
Seismic Coefficient at top of dam	=2.49
Minimum seismic resistance of dam for reservoir full condition	=0.62
Minimum seismic resistance of dam for reservoir empty condition	=0.75
Strength factor for reservoir full condition	=0.25
Strength factor for reservoir empty condition	=0.3

(b) Overflow Section :

Natural period of vibration	=0.07 sec.
Spectral displacement	=0.13 cm
Spectral acceleration	=1.08 g
Acceleration at top of dam	=2.7 g
Seismic coefficient at top of dam	=2.7
Minimum seismic resistance of dam for reservoir full condition	=0.82
Minimum seismic resistance of dam for reservoir empty condition	=0.98
Strength factor for reservoir full condition	=0.3
Strength factor for reservoir empty condition	=0.36

It can be seen that minimum value of strength factor is 0.25 and occurs for reservoir full condition for non-overflow section. For the overflow section, the minimum value of strength factor is 0.3. Thus it can be concluded that the dam as a whole will be able to withstand a shock with peak ground acceleration equal to 25% of Koyna type shock. The peak recorded ground acceleration during Koyna earthquake of Dec. 11, 1967 is 0.63 g. Therefore Pochampad masonry dam can withstand a Koyna type ground motion with peak acceleration of 0.16 g under worst earthquake conditions permitting ultimate tensile stress.

The ground motion at a site during an earthquake depends upon several factors such as the magnitude of the earthquake, distance from the epicentre, depth of focus, local geology and the properties of the intervening strata. An approximate estimate of the peak ground acceleration during an earthquake can be made using the following expression.[12]

$$\frac{a}{g} = \frac{2.925\left(\frac{10}{h_f}\right)^{M-5}}{1+4.5\left(\frac{10}{h_f}\right)^{M-5}} \; e^{-0.26}\left(\frac{D}{h_f}\right)^{3/2} \qquad \ldots(9)$$

where a is the peak ground acceleration, g the acceleration due to gravity, M the magnitude of the earthquake, h_f the focal depth in miles, D the distance of the site from the epicentre in miles.

The Kothagudem earthquake (1969)[11] had a magnitude of 6.0 with epicentre about 275 km from the dam site and a focal depth of 33 km. The peak ground acceleration expected at the Pochampad dam site due to Kothagudem earthquake can be estimated using Eq. 9 and this works out as 0.001 g which is quite negligible. However, the nearest point of the rift from the dam site is at a distance of about 50 Kms. Movements have taken place in the past along this rift and it is likely that earthquakes may originate from this rift system in future also. To cause a ground motion of 0.16 g at the pochamped dam site, the magnitude of earthquake originating from the rift system at a distance of 50 Km. and focal depth of 33 Km can be estimated using Eq. 9 and this works out as 7.65. Therefore by permitting ultimate tensile stress, the Pochampad dam can resist an earthquake of magnitude of the order, of 7.5 originating from the rift having focal depth of 33 Km and with epicentre 50 Km away which is the nearest distance of the rift from the dam site.

CONCLUSIONS

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

1. Permitting a nominal tension of 10.5 tonne/m² (15 p.s i.) the minimum seismic resistance of the dam in terms of seismic coefficient at the top of the dam is 0.061 for the non-overflow monolith and 0.172 for the overflow monolith. The recommended design seismic coefficient for the dam as per Indian standard Code is 0.06. Therefore the masonry dam as a whole can be considered to be marginally safe.

2. Under worst earthquake conditions by permitting ultimate tensile stresses, the non-overflow monolith has a minimum seismic resistance of 0.62 and it can resist about 25 percent of Koyna type shock. Similarly the overflow monolith has a minimum seismic resistance of 0.82 and it can resist about 30 percent of Koyna type shock. Thus the masonry dam as a whole can resist a shock with peak ground acceleration of 0.16 g, that is, about 25 percent of Koyna type earthquake. This ground motion can be expected at the dam site from an earthquake of magnitude of the order of 7.5 originating at a distance of 50 Km which is the nearest distance of the dam site from the rift and having a focal depth of 33 Km.

3. The earthquake of magnitude 6.0 originating from the rift with epicentre at kothagudem causes negligible ground motion at the dam site.

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