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A SIMPLIFIED PROCEDURE FOR THE ASEISMIC DESIGN OF CONCRETE GRAVITY DAMS

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INTRODUCTION

In water resources development projects, dams are very vital structures whose safety against seismic forces has to be fully ensured by the designers. Along with critical stresses due to static loads, the dynamic stresses due to earthquake excitation constitute quite a significant part in the overall stress pattern in a dam. Stability analysis of gravity dams considering earthquake forces as equivalent static forces has become a standard practice amongst the designers. The pseudostatic approximation for the seismic forces are based on arbitrary seismic coefficients, which in turn are assessed on an estimate of the seismicity of the site. The characteristics of the structure to be analysed are however not reflected in these coefficients. The inadequacy of such arbitrary coefficients has been revealed by the experience of actual behaviour of dams during past earthquakes. The advent of spectral response method and the feasibility of complex computations with the help of high speed digital computers have made possible a rational structural analysis for dynamic loads. There is at present a keen awareness to adopt rational aseismic design procedures for analysis of such an important structure like a dam impounding huge quantum of water behind it. Such a rational analysis for dynamic loads has now necessarily become an integral part of the structural design of dams.

Several methods of dynamic analysis such as beam analysis and finite element analysis are available for carrying out dynamic analysis of dams. However such techniques are sophisticated and require computer facilities which, at present, are beyond the scope of most of the design offices in the country. Thus the project design offices are unable to finalise the designs of the section of the dam and have to seek assistance from the few institutions specialised in such sophisticated techniques and having computer facilities. The processing of the designs and finalising the section of the dam has to wait till the reports from these institutions are received. Very often, inspite of sufficient previous investigations, site conditions at a dam site change which is revealed only when the excavation for dam foundation reaches an advanced stage. A speedy recheck of the design for the dam section for modified conditions then becomes necessary. If there is a tight time schedule for construction, requiring fast tempo of works before seasonal restrictions like onset of monsoon etc., any delay in obtaining the modified reports from these institutions will considerably hamper the progress of the dam. The project design offices, though otherwise well equipped, are not effective in revising the designs under the modified site conditions, as they are unable to carry out the complex dynamic analysis. In the planning stage of a water resources development project, many alternative sections of the dam and with different materials like masonry, concrete etc., are tried for adequate safety as well as for maximum economy as the cost of the storage dam usually forms a major part of the total project. It is essential that for project sites located in seismic areas, the design offices are conversant with and adopt rational methods of aseismic design of the dams even in the planning stage itself, so that the finally adopted section of the dam may not later found to be inadequate against dynamic

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forces due to strong ground motion, thus upsetting the entire economic perspective of the project. References to specialised institutions for each such preliminary study is not practicable.

Two basic assumptions leading to assessment of seismic loads on a dam are the adoption of the characteristics of design earthquake and the value of the damping factor of the material of the dam. It is difficult to predict the exact intensity of seismic excitation in the region and the choice of a suitable design earthquake at the dam site can lead to large variation as this includes many uncertain factors like the probable magnitude and pattern of ground motion, distance from epicentre, geo-physical characteristics of the ground through which vibrations are transmitted etc. In addition, the designer has also to consider the importance of the structure, vicinity of built-up areas downstream of the dam etc. to arrive at a safe estimate of the design seismic intensity. The damping factor usually adopted for Civil Engineering structures like concrete and masonry dams varies from 5 percent to 10 percent of critical damping. Depending upon the spectral response curve adopted, even this range of damping may show considerable variations in the response of the dam section to seismic excitation. The insitu dynamic strengths of material of the dam, which ultimately decide the safety of the dam against stresses due to severe ground motion, can not also be evaluated accurately. Due to these uncertainties, suitable approximations are made for the assumptions of the earthquake characteristics, damping factor, dynamic strength of material etc.

With such approximations and limitations in 'input' data, too great a precision in evaluation of dynamic characteristics like natural period of vibration, mode shape etc. is not warranted at present, for all practical purposes. A rational but simpler procedure, which can be easily adopted in the design offices and can give results with a reasonable accuracy compared with the values obtained by more sophisticated methods of dynamic analysis like beam analysis etc. may therefore be quite acceptable for practical application. The necessity of such a simpler procedure giving reasonably accurate results and amenable for easy computations even in the field offices is keenly felt at present. The proposed simplified procedure for aseismic design of concrete and masonry gravity dams is presented to fulfill this urgent need. Once the section of the dam is finalised based on the simplified method, a detailed analysis for stress distribution and location of areas of stress concentration in the section of the dam can be carried out using the sophisticated and complex techniques. Any minor modification in profile etc. can be obtained to improve the structural conditions.

OUTLINE OF DYNAMIC ANALYSIS

A gravity dam is usually constructed in monoliths separated by transverse contraction joints. These joints are usually neither keyed nor grouted for wide and flat valleys. The external load is therefore assumed to be transmitted directly to the foundation through vertical cantilever action of the dam, no load being assumed to be transmitted to the abutments. The stability of the entire dam is, therefore, expressed in terms of the stability of the individual monoliths. During ground motions, each monolith can be assumed to vibrate independently of the adjacent monoliths.

A complete and rigorous analysis of the seismic behaviour of a dam impounding a reservoir poses a complex problem. The dam and the reservoir are usually treated as two uncoupled systems. The dynamic response of the dam is determined ignoring the effect of reservoir water. The effect of reservoir water is represented as hydrodynamic pressure. Various theories (1, 2, 3, 4, 5) have been proposed for the determination of hydrodynamic pressures which essentially treat the dam as a rigid structure and the interaction effects between the reservoir and the dam are ignored. The uncoupled solutions can then be combined to obtain a complete solution of the system. Some investigators (6, 7, 8, 9, 10) have also considered the hydrodynamic pressure effect of the reservoir water as indirectly represented by an equivalent virtual mass and its effect included in the dynamic analysis itself. However, research on the importance of the interaction effects between the dam and the reservoir is currently in progress (11, 12, 13, 14).

Dynamic analysis of the dam can be carried out either by treating the dam as a one dimensional structure (beam analysis) or as two-dimensional structure (finite element analysis). Based on the analysis of several dams, (9, 10), it is noted that the earthquake forces on dams as estimated by both the methods are of the same order in magnitude and distribution. However the stresses obtained by the two methods are significantly different which is due to the nonlinear variation of stresses in the dam cross-section and is accounted for in the finite element analysis. Thus for determining the dynamic forces on the dam, either method is adequate. However, for a more precise estimation of stresses, the finite element analysis is desirable. Detailed procedures of the dynamic analysis are available in the literature (9, 10, 15, 16, 17). As pointed out earlier, the procedure is sophisticated and is not within the scope of field design offices engaged in the design of dams. Therefore, a simplified procedure is proposed for preliminary design of dams which is simple, quick and also rational. In order to establish the proposed procedure, the results of the analysis of four dams have been compared with the beam analysis and a very good agreement obtained.

PROPOSED SIMPLIFIED PROCEDURE

The proposed procedure is based on the Rayleigh method in which an assumption is made of the dynamic deflection configuration of the vibrating system. A good approximation of the dynamic deflection curve of the dam can be made by the static deflection curve under its own weight assumed to be acting horizontally. The structural shape of the cross-section of the dam and its material properties influence the response of the structure to a particular ground motion excitation. The static deflection of the dam section being based on these parameters, can be justifiably considered as a reasonable approximation of the dynamic deflection curve. This is a more rational approach than any other arbitrary assumption of a configuration which is not based on the structural shape and properties of the section under analysis.

For the purpose of analysis, the dam section is discretised by lumping the masses at a convenient number of points. About 20 to 25 segments along the height of the dam would be sufficient. Assuming the lumped weights to be acting horizontally, the deflections of the dam can be obtained using standard procedures. However, as the width of the gravity dam is also considerable compared with its height, shear deformations are also considered in addition to deflections due to bending. For numerical evaluation, the following expressions can be used to obtain shear, moment, bending slope and deflections at any point of the dam in terms of the corresponding quantities at the neighbouring point.

$$V_n = V_{n-1} + w_{n-1} \tag{1}$$

$$M_n = M_{n-1} + V_n (\Delta x)_n \tag{2}$$

$$\theta_{bn} = \left(\frac{\Delta x}{EI}\right)_n \left[\frac{M_{n-1} + M_n}{2}\right] + \theta_{bn-1} \tag{3}$$

$$y_{bn} = \left(\frac{\Delta x}{EI}\right)_n \left[\frac{M_{n-1}}{3} + \frac{M_n}{6}\right] (\Delta x)_n + \theta_{bn-1} (\Delta x)_n + y_{bn-1} \tag{4}$$

$$y_{sn} = y_{sn-1} + \left(\frac{\Delta x}{\sigma AG}\right)_n V_n \tag{5}$$

$$y_n = y_{bn} + y_{sn} \tag{6}$$

Where V is the shear force, M the moment, w the lumped weight, θ_b the bending slope, y_b the bending deflection, y_s the shear deflection, y the total deflection, Δx the distance between load points, E the modulus of elasticity, G the modulus of rigidity, I the moment of inertia, A the area of cross-section, σ the shape factor which can be taken as 5/6 for solid rectangular sections, n and $n-1$ denote the lumped mass point numbers.

At the top of the dam, the shear and moment are zero. Starting from the free end, the shears and moments at the various points are obtained step by step using Eqs. 1 and 2. Knowing the shears and moments, the deflections of the dam can be worked out. At the base of the dam, the bending slope, θ_b, and the deflections, y_b, y_s and y are zero as the dam is assumed to be rigidly fixed. Starting from the base, the slopes and deflections at the various points are obtained step by step using Eqs. 3 to 6. Knowing the deflections, the dynamic characteristics and the dynamic responses can be obtained using the following expressions.

$$p^2 = \frac{g \sum_{i=1}^{N} w_i y_i}{\sum_{i=1}^{N} w_i y_i^2} \tag{7}$$

$$T = \frac{2\pi}{p} \tag{8}$$

$$\phi_n = \frac{y_n}{y_T} \tag{9}$$

$$\gamma = \frac{\sum_{i=1}^{N} m_i \phi_i}{\sum_{i=1}^{N} m_i \phi_i^2} \tag{10}$$

$$Z_n = \gamma \phi_n Sd \tag{11}$$

$$X_n = p^2 Z_n \tag{12}$$

$$Q_n = \frac{w_n X_n}{g} \tag{13}$$

$$V'_n = V'_{n-1} + Q_{n-1} \tag{14}$$

$$M'_n = M'_{n-1} + V'_n (\Delta x)_n \tag{15}$$

Where p is the circular natural frequency in rad./sec., w the lumped weight, y the static deflection, g the acceleration due to gravity, T the natural period of vibration of the dam, ϕ the normalised dynamic deflection shape, y_T the deflection of the dam at the top, γ the participation factor, m the lumped mass, Z the dynamic displacement, Sd the spectral displacement, X the acceleration, Q the dynamic load, V_n' the dynamic shear, M_n' the dynamic moment, Δx the distance between load points and N the total number of lumped masses.

A suitable choice of the design earthquake and its response spectrum curve is essential for calculating the dynamic responses of the dam. Two parameters namely the natural period of vibration and the damping are needed to obtain the spectral values from the response spectrum. The natural period of vibration has already been computed and damping for concrete and masonry dams can be taken between 5 to 10 per cent of critical damping. The dynamic loads, shears and moments can be obtained using Eqs. 13 to 15.

The dynamic moments and shears can also be conveniently expressed as.

$$M'_n = \alpha_n \, WH \tag{16}$$

$$V'_n = \beta_n \, W \tag{17}$$

Where M' is the dynamic moment, V' the dynamic shear, W the total weight of the dam, H the total height of the dam, β_n the non-dimensional coefficient for dynamic shear, α_n the non-dimensional coefficient for dynamic moment and n denotes the point under consideration.

Once the dynamic shears and dynamic moments throughout the height of the dam are determined, further computation of dynamic stresses can be done using standard procedures. Appendix-I fully illustrates the method of computation by the proposed simplified procedure for a typical gravity dam.

ANALYSIS OF SOME EXISTING DAMS

Dynamic analysis has been carried out both by beam analysis and the proposed simplified procedure for the highest non-overflow sections of four existing gravity dams, namely Kolkewadi dam (64.1 m), Sholayar dam (93.0 m), Koyna dam (103.0 m) and Pine flat dam (129.0 m). Their heights range from 64 m to 129 m which covers the range of most of the gravity dams in the country. Further, these dams differ in materials of construction, upstream and downstream slopes thus covering a wide range of materials and profiles usually adopted for gravity dams. Figs. 1 to 4 show the cross-sections of the dams considered for analysis. The characteristics of these dams are presented in Table 1. By the proposed simplified procedure, the dynamic characteristics as well as the dynamic responses of the dams have been evaluated. These results have been compared with those obtained by dynamic analysis of the dam treating it as a beam in which the responses have been obtained using mode superposition method. Damping has been taken as equal to 5 per cent of critical damping. The response spectrum of Koyna earthquake of Dec. 11, 1967 (Transverse Component) as shown in Fig. 5 has been considered for evaluating the seismic responses (18).

Table 2 presents a comparison of the results obtained by the two methods. The natural periods of vibration, the dynamic base moments and the dynamic base shears have been compared. It is noted from the table that the natural period of vibration obtained by the proposed procedure is very close to that obtained by dynamic

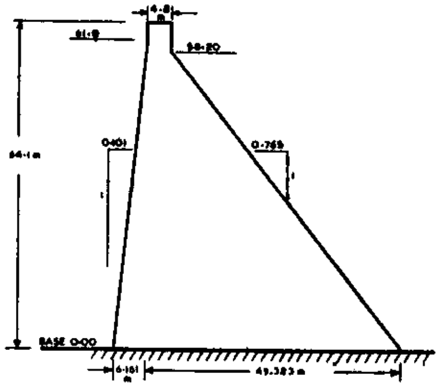

FIG 1 CROSS-SECTION OF KOLKEWADI DAM

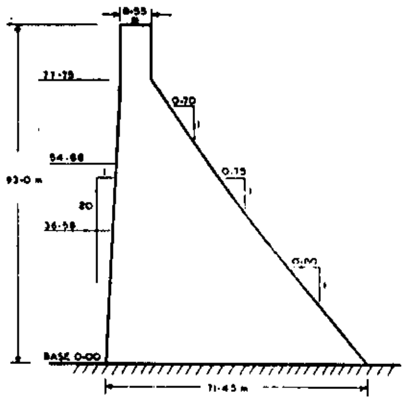

FIG. 2 CROSS-SECTION OF SHOLAYAR DAM

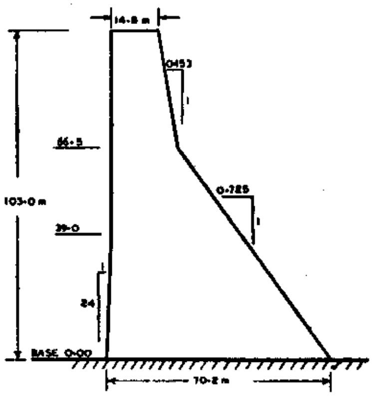

FIG. 3 CROSS-SECTION OF KOYNA DAM

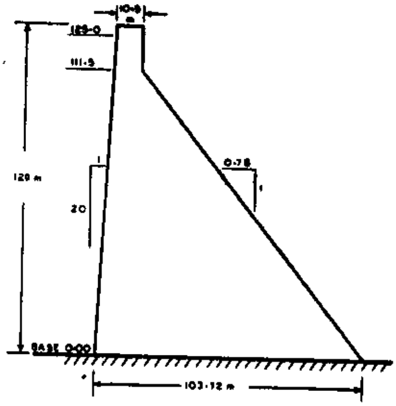

FIG. 4 CROSS-SECTION OF PINE FLAT DAM

TABLE I*

CHARACTERISTICS OF DAMS ANALYSED

S. No.	Dam	Height	Ratio of Base Width to Height of Dam	Dam Material and Properties
1.	Kolkewadi	64.1 m	0.862	Masonry $\rho = 2.40$ t/m^3 $E = 2.28 \times 10^6$ t/m^2 $\nu = 0.2$
2.	Sholayar	93.0 m	0.770	Masonry $\rho = 2.33$ t/m^3 $E = 2.13 \times 10^6$ t/m^2 $\nu = 0.2$
3.	Koyna	103.0 m	0.682	Rubble Concrete $\rho = 2.65$ t/m^3 $E = 2.50 \times 10^6$ t/m^2 $\nu = 0.2$
4.	Pine Flat	129.0 m	0.800	Concrete $\rho = 2.40$ t/m^3 $E = 2.20 \times 10^6$ t/m^2 $\nu = 0.15$

*ρ—Weight density of dam material; E—Modulus of Elasticity; ν—Poisson's ratio.

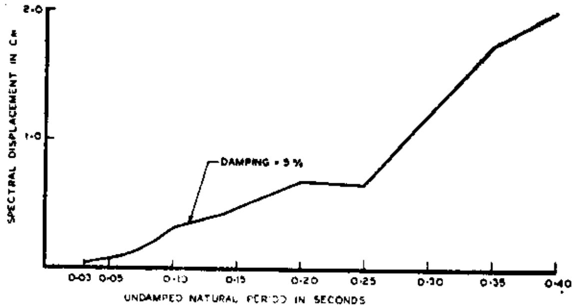

FIG 5 DISPLACEMENT SPECTRUM DUE TO KOYNA EARTHQUAKE OF DEC 11, 1967 - TRANSVERSE COMPONENT

TABLE 2*

COMPARISON OF RESULTS BY PROPOSED PROCEDURE AND DYNAMIC ANALYSIS

S. No.	Dam	Height	PROPOSED PROCEDURE			DYNAMIC ANALYSIS		
			T	M_b'	V_b'	T	M_b'	V_b'
1.	Kolkewadi	64.1 m	0.138	0.268 WH	0.500 W	0.142	0.250 WH	0.466 W
2.	Sholayar	93.0 m	0.230	0.162 WH	0.340 W	0.237	0.173 WH	0.498 W
3.	Koyna	103.0 m	0.328	0.202 WH	0.350 W	0.337	0.185 WH	0.438 W
4.	Pine Flat	129.0 m	0.295	0.174 WH	0.320 W	0.304	0.169 WH	0.364 W

* T—Natural period of vibration in sec; M_b'—Base moment; V_b'—Base shear; W—Weight of dam; H—Height of dam.

analysis of the dam. As is expected, the values of the natural period obtained by the proposed procedure are on the lower side to those obtained by dynamic analysis and the error is not more than 3%. The table also shows that the dynamic base moments obtained by the proposed procedure are very close to those obtained by dynamic analysis. The values obtained by the proposed procedure are generally on the conservative side and the maximum difference is of the order of 9%. Similarly the dynamic base shears obtained by the proposed procedure are reasonable compared to those obtained by dynamic analysis although the difference in this case is somewhat large than in the previous case. For checking the design of the dam, moments are more critical as they induce tensile stresses and these can be obtained quite accurately by the proposed procedure.

Figs. 6 to 9 show the distribution of the dynamic moment and dynamic shear coefficients along the height of the dam for the four dams considered. It is noted from

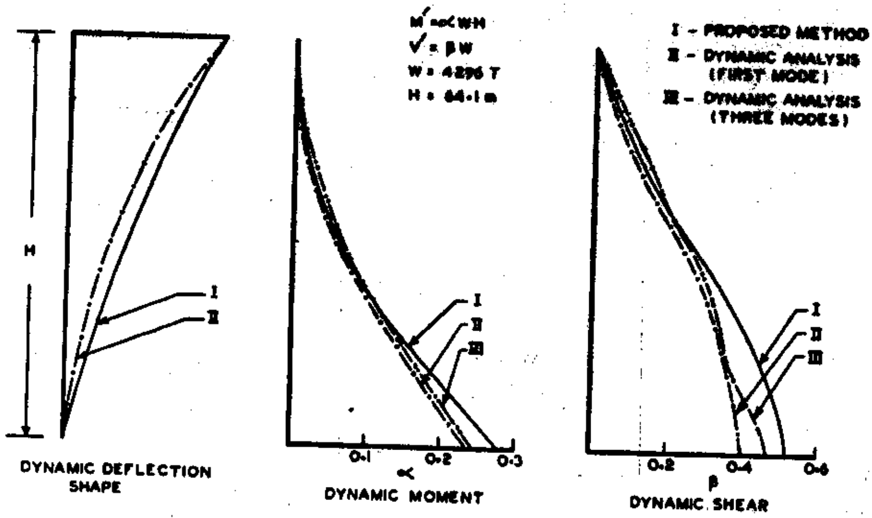

FIG. 6 DYNAMIC DEFLECTION SHAPE, DYNAMIC MOMENT AND DYNAMIC SHEAR DIAGRAMS FOR KOLKEWADI DAM

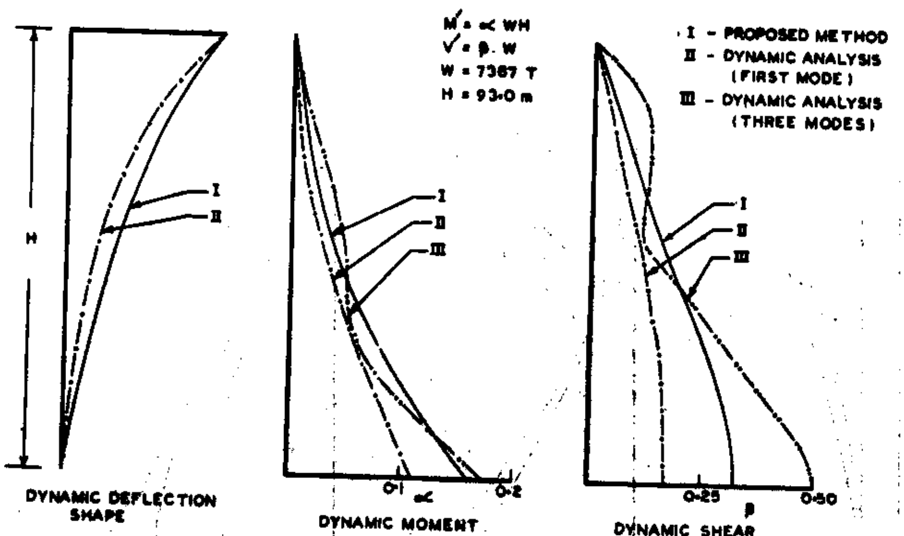

FIG. 7 DYNAMIC DEFLECTION SHAPE, DYNAMIC MOMENT AND DYNAMIC SHEAR DIAGRAMS FOR SHOLAYAR DAM

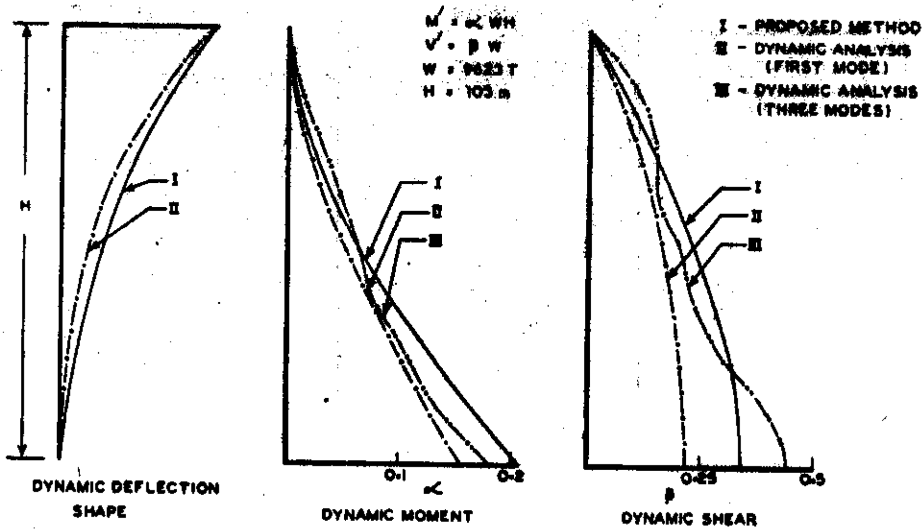

FIG. 8 DYNAMIC DEFLECTION SHAPE, DYNAMIC MOMENT AND DYNAMIC SHEAR DIAGRAMS FOR KOYNA DAM

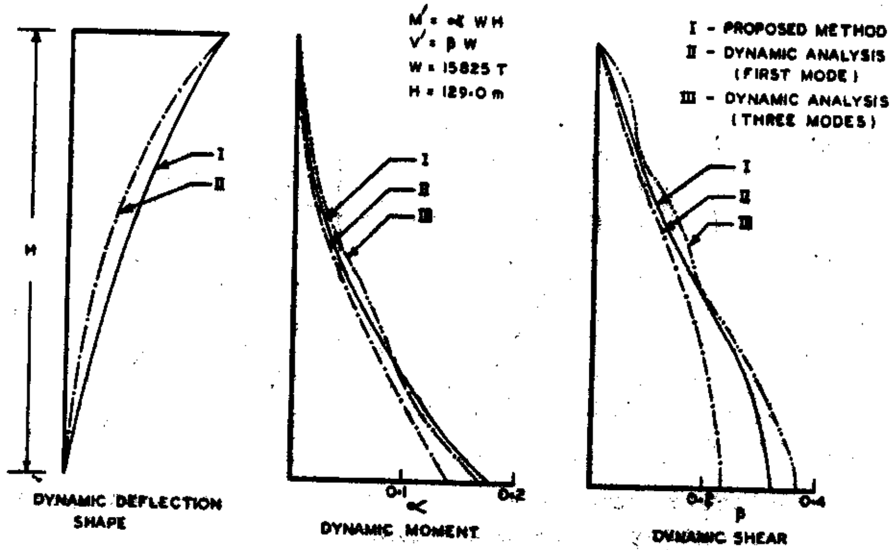

FIG. 9 DYNAMIC DEFLECTION SHAPE, DYNAMIC MOMENT AND DYNAMIC SHEAR DIAGRAMS FOR PINE FLAT DAM

the figures that the dynamic moments obtained by the proposed procedure are quite close to those obtained by dynamic analysis at all sections of the dam. Similarly the dynamic shears obtained by the proposed procedure are close to those obtained by dynamic analysis at all sections but the difference in this case is somewhat large than in the case of moments as has also been noted earlier, but this is not critical for design.

From the preceeding discussion, it is noted that the proposed procedure gives results which are quite close in comparison to those obtained by dynamic analysis of the dams. The range of accuracy obtained in the results by the proposed procedure is quite acceptable for actual designs, particularly in the context of many uncertainties involved in the basic assumptions of characteristics of ground motion, damping factor and dynamic strength of materials etc. The proposed procedure considers the characteristics of the structure as well as the characteristics of the earthquake, requires only manual calculations and is quite simple to use. Thus the proposed procedure can be readily adopted in the design offices as a reasonably accurate and simple method of preliminary design of concrete and masonry gravity dams.

CONCLUSIONS

A simplified procedure has been proposed for the aseismic design of concrete and masonry gravity dams. The procedure is adequate for estimating the dynamic response of the dams. The method is simple, quick and can be readily adopted in the design offices for preliminary design of dams.

APPENDIX I

The method of computation of dynamic response by the proposed procedure has been illustrated by applying it for the analysis of Kolkewadi dam for which the results have already been presented earlier. The dam cross-section, material properties and response spectrum of the earthquake has also been discussed earlier. The 64.1 m high dam has been converted into a discrete system of 22 segments. The dead weight has been assumed to be acting at the centre of the segment. This does not result in any appreciable error compared to the case when the dead weight is concentrated at the centre of gravity of the segment. Considering these weights to be acting horizontally on a cantilever fixed at the base and free at the top, the static shears and moments can be obtained using Eqs. 1 and 2 respectively and these are tabulated in Table 3. Knowing the static shears and moments, the bending slope, bending deflection, shear deflection and total deflection can be obtained using Eqs. 3 to 6 starting from the fixed end (base of the dam). Knowing the deflection of the dam, the normalised deflection shape can be obtained by dividing the deflection at the point with the deflection at the top of the dam. The natural frequency and natural period of vibration can be obtained using Eqs. 7 and 8. These calculations are tabulated in Table 4. From the table

$$\sum w_i y_i = 1199.60\, t \text{ cm}$$

$$\sum w_i y_i^2 = 568.00\, t \text{ cm}^2$$

Using Eq. 7

$$p^2 = \frac{g \sum w_i y_i}{\sum w_i y_i^2} = \frac{981 \times 1199.60}{568.00} = 2072.1$$

TABLE 3

CALCULATION OF STATIC MOMENTS AND SHEARS

Load Point No.	Height of Segment (m)	Weight of segment w(ton)	Distance between load points Δx(m)	Static shear V(t)	Static moment M(tm)
Free end (Top)					
1	3.10	35.70	1.55	0.00	0.00
2	2.80	33.15	2.95	35.70	105.31
3	1.20	16.10	2.00	68.85	243.01
4	3.00	53.30	2.10	84.95	421.41
5	3.00	72.10	3.00	138.25	836.16
6	3.00	90.70	3.00	210.35	1467.21
7	3.00	109.50	3.00	301.05	2370.36
8	3.00	128.00	3.00	410.55	3602.01
9	3.00	147.00	3.00	538.55	5217.66
10	3.00	165.50	3.00	685.55	7274.31
11	3.00	184.20	3.00	851.05	9827.46
12	3.00	203.00	3.00	1035.25	12933.21
13	3.00	221.80	3.00	1238.25	16647.96
14	3.00	240.50	3.00	1460.05	21028.11
15	3.00	259.00	3.00	1700.55	26129.76
16	3.00	277.50	3.00	1959.55	32008.41
17	3.00	296.50	3.00	2237.05	38719.56
18	3.00	315.00	3.00	2533.55	46320.21
19	3.00	333.50	3.00	2848.55	54865.86
20	3.00	352.50	3.00	3182.05	64412.01
21	3.00	371.00	3.00	3534.55	75015.66
22	3.00	390.00	3.00	3905.55	86732.31
Built in end (Base)		0.00	1.50	4295.55	93175.63

TABLE 4

CALCULATION OF STATIC DEFLECTIONS AND NATURAL PERIOD OF VIBRATION

Load point No.	Distance between load points $\Delta x\,(m)$	Moment of Inertia $I(m^4)$	Bending slope $\theta_b\,(10^{-6})$	Bending Deflection $y_b\,(10^{-6})\,m$	Shear Deflection $y_s\,(10^{-6})m$	Total static deflection $y(10^{-6})\,m$	Normalised deflection ϕ	Weight of segment $w\,(t)$	$W_i\,y_i$ $(t\ cm)$	$W_i\,y_i^2$ $(t\ cm^2)$
Built in end	1.50	13240.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
22	3.00	11370.0	4.45	3.41	131.80	135.21	0.013	390.00	5.273	0.071
21	3.00	9796.0	13.90	31.31	420.00	451.31	0.042	371.00	16.744	0.756
20	3.00	8318.0	23.30	87.46	694.80	782.26	0.072	352.50	27.575	2.157
19	3.00	6998.0	32.78	171.96	955.90	1127.86	0.105	333.50	37.614	4.242
18	3.00	5820.0	42.35	285.05	1203.50	1488.55	0.138	315.00	46.889	6.980
17	3.00	4793.0	52.10	427.00	1437.70	1864.70	0.173	296.50	55.288	10.310
16	3.00	3888.0	61.82	598.33	1658.40	2256.73	0.209	277.50	62.624	14.133
15	3.00	3103.0	71.63	799.06	1865.90	2664.96	0.247	259.00	69.022	18.394
14	3.00	2435.0	81.63	1029.51	2059.80	3089.31	0.286	240.00	74.143	22.905
13	3.00	1868.0	91.85	1290.30	2240.40	3530.70	0.327	221.80	78.311	27.649
12	3.00	1400.0	102.30	1582.15	2407.70	3989.85	0.369	203.00	80.994	32.315
11	3.00	1014.0	113.00	1905.85	2562.10	4467.95	0.414	184.20	82.300	36.771
10	3.00	708.0	124.12	2262.35	2703.40	4965.75	0.460	165.50	82.183	40.810
9	3.00	471.0	135.74	2653.06	2831.70	5484.76	0.507	147.00	80.626	44.221
8	3.00	293.0	148.06	3079.90	2947.10	6027.00	0.558	128.00	77.146	46.496
7	3.00	167.0	161.48	3545.58	3050.20	6595.78	0.611	109.50	72.224	47.637
6	3.00	83.9	176.60	4054.47	3141.30	7195.77	0.666	90.70	65.266	46.964
5	3.00	34.0	194.65	4613.77	3221.20	7834.97	0.725	72.10	56.490	44.260
4	2.10	14.6	218.95	5238.22	3292.20	8530.42	0.790	53.30	45.467	38.785
3	2.00	10.0	239.90	5722.12	3331.70	9053.82	0.838	16.10	14.577	13.198
2	2.95	9.2	255.20	6219.44	3367.10	9586.54	0.887	33.15	31.780	30.465
1	1.55	9.2	262.70	6986.44	3395.10	10381.54	0.960	35.70	37.062	38.476
Free end	—	—	262.70	7393.44	3395.10	10788.54	1.000	0.00	0.000	0.000
									Σ 1199.598	567.996

TABLE 5

CALCULATION OF DYNAMIC LOADS, SHEARS AND MOMENTS

Load point No.	Dynamic Load Qn(t)	Dynamic shear V'n(t)	Dynamic moment M'n(tm)	Dynamic moment coefficient α_n	Dynamic shear coefficient β_n
Free end					
1	66.15	0.00	0.00	0.000	0.000
2	56.75	66.15	195.14	0.001	0.015
3	26.04	122.90	440.94	0.002	0.029
4	81.27	148.94	753.71	0.003	0.035
5	100.88	230.21	1444.34	0.005	0.054
6	116.58	331.09	2437.61	0.009	0.077
7	129.13	447.67	3780.62	0.014	0.104
8	137.85	576.80	5511.02	0.020	0.134
9	143.84	714.65	7654.97	0.028	0.166
10	146.93	858.49	10230.44	0.037	0.200
11	147.18	1005.42	13246.70	0.048	0.234
12	144.57	1152.60	16704.50	0.061	0.268
13	139.98	1297.17	20596.01	0.075	0.302
14	132.48	1437.15	24907.46	0.090	0.335
15	123.47	1569.63	29616.35	0.108	0.365
16	111.94	1693.10	34695.65	0.126	0.394
17	99.00	1805.04	40110.77	0.146	0.420
18	83.90	1904.04	45822.89	0.166	0.443
19	67.58	1987.94	51786.71	0.188	0.463
20	48.98	2055.52	57953.27	0.210	0.478
21	30.07	2104.50	64266.77	0.233	0.490
22	9.79	2134.57	70670.48	0.257	0.497
Built in end	0.00	2144.36	73887.02	0.268	0.500

$$p=\sqrt{2072.1}=45.52 \text{ rad/sec}$$

$$T=\frac{2\pi}{p}=\frac{2\pi}{45.52}=0.138 \text{ sec}$$

Value of T in the first mode by dynamic analysis $=0.142$ sec.

Using Eq. 10

$$\gamma=\frac{\sum_{i=1}^{N} m_i\phi_i}{\sum_{i=1}^{N} m_i\phi_i^2}=y_T\frac{\sum_{i=1}^{N} w_i y_i}{\sum_{i=1}^{N} w_i y_i^2}$$

$$\gamma=\frac{1.08\times 1199.60}{568.00}=2.28$$

Corresponding value of the participation factor in the first mode by dynamic analysis is 2.31.

Taking damping equal to 5% of critical damping and for natural period of 0.138 sec, the spectral displacement Sd for Koyna earthquake of Dec. 11, 1967 can be obtained using spectrum curve at figure 5 and is equal to 0.40 cms.

Dynamic force

$$Qn=w_n\frac{p^2\gamma\phi_n\, Sd}{g}$$

$$Qn=\frac{2072.1\times 2.28\times 0.40}{981}\,\phi_n w_n$$

$$Qn=1.93\,\phi_n w_n$$

Knowing the dynamic load at each point, dynamic shears and moments can be obtained using Eqs. 14 and 15 and can be expressed in a more convenient form as given by Eqs. 16 and 17. The calculations are tabulated in Table 5.

The dynamic moments and shears at the base of the dam by the proposed procedure are 0.268 WH and 0.500 W. The corresponding values obtained by the dynamic analysis are 0.250 WH and 0.466 W respectively. The distribution of moments and shears along the height of the dam is presented in Fig. 6.

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