IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)
e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 6, Issue 3 (May. - Jun. 2013), PP 42-47
www.iosrjournals.org

Seismic Analysis for Safety of Dams

[1]N.P. Gahlot, [2]Dr. A.R. Gajbhiye
[1] *Sectional Engineer, Water Resources Department,Amaravati..*
[2] *Professor & Head of Civil Engineering Department, Yeshvantrao Chavan College of Engineering, Nagpur.*

Abstract: *Geo-technical engineering as a subject has developed considerably in the past four decades. There has been remarkable development in the fields of design, research and construction of dam. India is capable of designing and constructing a dam that would withstand a seismic jolt. The country needs water and electricity to provide its people good living standards. Hydropower is the solution to the country's requirements, and this can be achieved by storing water in dams.*

In the past, earthquake effects may have been treated too lightly in dam design. Are such dams safe, and how have they fared in previous earthquakes, this Paper will be limited to the some of finding about one concrete types.

What will happen to dams during severe earthquake shaking? It is obvious that at present engineers cannot answer this question with any certainty. But we are very much aware of the threat of disastrous losses of life and damage to property if dams should fail, and we are making great effort to increase our under standing of this complex topic.

This Paper deals with the case study of totaladoh Dam Situated in Vidarbha Region of Maharashtra for Seismic Analysis by I.S.Code method (Simple Beam Analysis method). This also includes future scope of analyzing the same dam for Seismic safety by very accurate method i.e. finite element method.

Keywords: *Earthquake, The finite element method, Indian Standard codes(I.S.Code), horizontal seismic coefficient (αh),Hydrostatic pressure,* Seismic analysis,

I. Introduction

As of now, there are about 23,000 large dams in the world. A large dam is defined by the International Congress on Large Dams (ICOLD) as one which has a height of 33 metres and above: Of these dams, only four, namely Koyna (India), Kremesta (Greece), Kar iba (South Africa) and Singfenkiang (China), have experienced earthquakes of a moderate magnitude of between 6.0 and 6.5 on the Richter scale within a few years of building.

Although no earthquake-related failure of a concrete dam has occurred to date, no large concrete dam with a full reservoir has ever been subjected to really severe ground shaking. Such a possibility has many groups concerned, including the Division of Safety of Dams(DSD), a California state agency responsible for assuring the safety of California dams. The Division of Safety of Dams (DSD) has the power to order an updated seismic check of a dam if new information rises or if better analysis techniques are developed by researchers. In the early" 1970s, two events led the Division of Safety of Dams (DSD) to initiate a program to perform seismic checks on all major dams under its jurisdiction. The first event was the near collapse of Lower San Fernando Dam, a large earthen dam, during the 1971 earthquake; and the second was the development of the finite element method, a tool for computerized stress analysis.

In this paper the methods of seismic Analysis of dam are discussed. A case study of Totaladoh Dam which is analyzed by simplified beam method. The Results obtained by this method will be compared by the results obtained by finite element method is the future scope of study.

II. Methods for seismic analysis of Dam

Concrete gravity dam design was, and still is, based on two-dimensional idealizations (as illustrated in the figure No.1) because gravity dams, which are generally located in wide river valleys, are long and nearly uniform in cross section. Water loading from the reservoir behind the dam seeks to overturn or slide the dam downstream; and the dam's own weight resists this action. A proper choice of dam cross section provides stability. In addition, since concrete is weak in tension and since no steel reinforcing is employed, engineers equated. The presence of tensile stresses with failure.If their computations showed tensile stress at any point, they redesigned the cross section. Stress analysis was performed by treating the dam cross section as a beam of variable thickness cantilevering from the valley floor.

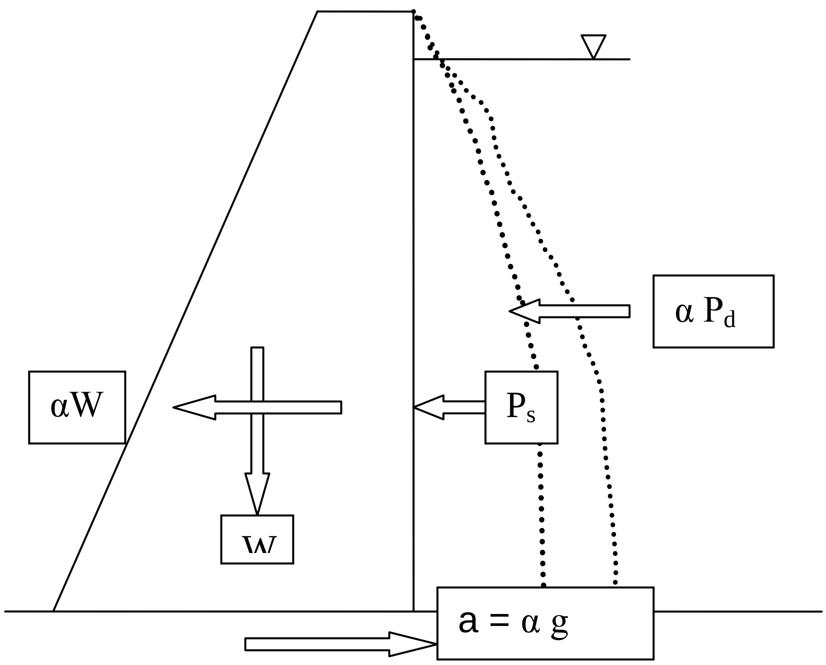


Fig. No. 1 :Design of a concrete gravity dam (above) is based on a two-dimensional idealization. The design loads include the dam weight W, the static water force P, and the ground acceleration α (given as a fraction α of the gravitational acceleration g). The ground acceleration produces an inertia force α Won the dam and an additional water force α P_d (where P_d is the water force caused by a unit acceleration of the water into the reservoir).

Arch dams are built in narrow canyons, and they are true three-dimensional structures. They resist the water load by combining cantilever bending from the canyon floor with arch thrusting to the abutments. Usually their proportions are much thinner than those of concrete gravity dams. Tensile stress was again avoided in the design, but engineers found stress analysis much more complicated. An iterative relaxation method applied to independent arch and cantilever sections was developed, which produced many rooms full of engineers grinding out stress calculations. Which appeared to be a stiff structure, would move rigidly with the ground. Thus, if the ground accelerates at a fraction, designated α (alpha), of gravity, an inertial force of magnitude α times the dam weight is created and acts on the dam in the downstream direction. Moreover, additional water pressure is generated, proportional to the acceleration of the dam into the reservoir if water incompressibility is assumed. This feature was recognized in 1933, and it has been included in dam design ever since. Typical values of α were 0.05 to 0.15, and inclusion of earthquake effects stilI allowed the no-tension criterion to be observed in the design.

Early design procedures were obviously great simplifications of reality. Dams are not really rigid; they are flexible structures that vibrate on their own when excited by ground motion. Stress analysis methods were approximate, and the maximum ground accelerations used were only fractions of what could occur. Vertical and cross-stream components of ground motion were neglected. But the pertinent question is, of course: How have dams designed by the methods described performed during past earthquakes? And the answer is: Fairly well,

III. Seismic analysis of Dam by Finite element Method.

The finite element method transforms the governing differential equations (the equations of solid mechanics in the case of a dam) to a matrix equation that is solved on the computer. The structure to be analyzed is meshed into elements (see figure No.2), which are connected at nodal points. Associated with these nodes are displacement degrees of freedom, which become the unknowns of the matrix equation. Solution of the matrix equation yields the structure displacements, from which the stresses are easily computed. As long as the governing differential equations are linear, the finite element method produces remarkable solutions. Nonlinearities, however, are much more difficult to handle. An example of nonlinearity in dam behavior is the formation of cracks or opening of built-in joints due to the presence of tensile stresses. Even today, finite element techniques have not progressed to the point where this type of non linearity can be handled.

A standard procedure using the finite element method was developed for computing the (linear) response of concrete dams. The sketch (Fig. No.2) illustrates this procedure. A finite element model is constructed of the dam and of a portion of the foundation region that extends out to an artificial boundary where earthquake motions are applied. The motion specified by the engineer is actually the free-field motion (that is, the motion that would occur at the dam-foundation interface if the dam were not present), and the engineer must back-calculate what motion to apply at the foundation boundary. Since the foundation boundary produces wave reflections that contaminate the computed dam response after a short time, the foundation mesh is usually assumed to be mass less. The alternative is to place the foundation boundary far away from the dam, but this results in a large, expensive to solve matrix equation. The water is included in the analysis by an added-mass approach. An appropriate volume of water is assigned to move with each horizontal, nodal degree of freedom at the upstream dam face. Treating the water in this manner neglects water compressibility (which can be important for deep reservoirs) and ignores the additional pressures generated by the vertical and cross-stream components of earthquake ground motion along the reservoir boundary.

Obviously, the most pressing research need is for computational techniques that accurately model the cracking behavior, but little progress has been made to date. Recently, however, some headway has been reported on improved modeling of the dam foundation and of the water in the reservoir. The artificial foundation boundary can be replaced by mathematical transmitting boundaries, which reflect only a small fraction of an incident wave. For the water,We have developed finite element models (Fig. No.3) that includes water compressibility and the additional pressures generated. by vertical and cross-stream motions of the reservoir boundaries. Both of these effects have been shown to influence the earthquake response of concrete dams significantly.

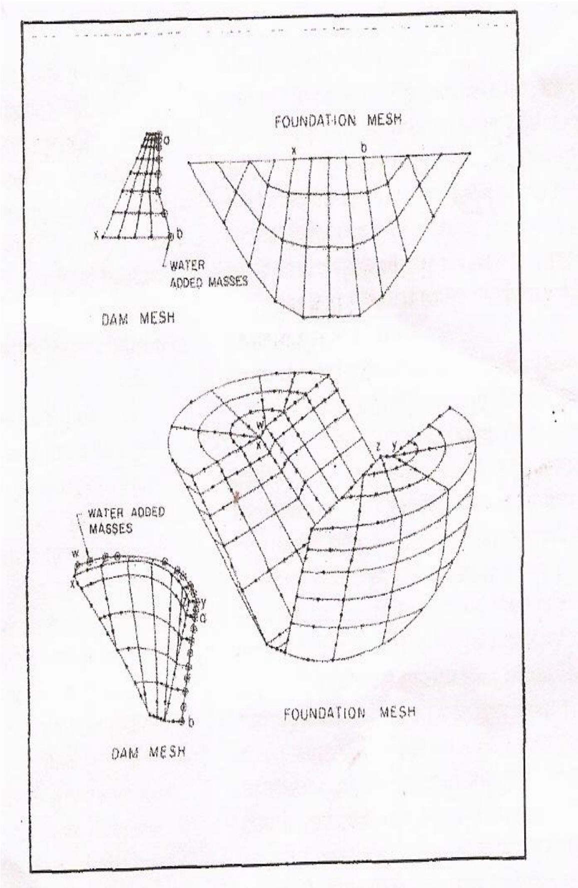
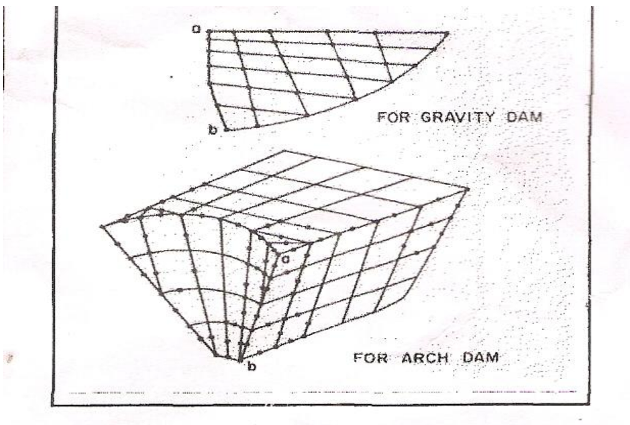


Fig. No. 2 : Shows an illustration of how improved earthquake safety evaluations of dams have been made possible through use of finite use of finite element analysis. A common procedure employs a dam mesh, a mesh of finite region of foundation, an added masses on the upstream dam face to represent the water.



fraction of an incident wave. For the water, I

Fig. No. 3 : Finite element model of the water in the reservoir accurately represents its effect on the dam response to earthquake shaking, these model includes water compressibility and additional pressure generated by vertical and cross stream motions of reservoir boundaries

IV. Objectives of Study :

Objectives of study is to estimate the stresses induced due to earthquake shock on the dam by I.S.Code method and compare these with the results which will be obtained for the same dam by the seismic analysis carried out by Finite element method.

V. Case Study - Analysis of Totaladoh Dam by I.S.Code method

The Totaladoh dam is a masonry type Gravity Dam constructed in the year 1982-83 on the Pench river near village totaladoh, Nagpur district in Maharashtra(India).It is an interstate Hydro cum Irrigation project of Maharashtra and Madhyapradesh. The salient feature of Dam is as under.

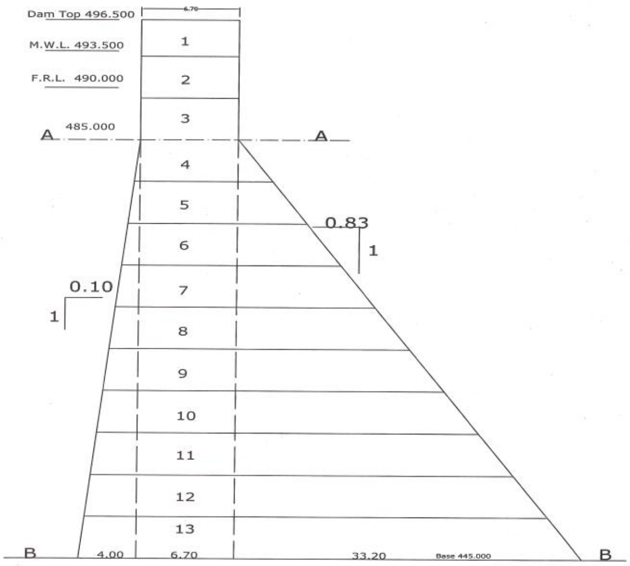
Controlling Levels :
(I) Crest level of Spillway – 482.000 m.
(II) M.D.D.L. - 464.000 m.

Length of Earthen dam – 2381 m.
Length of Masonry dam – 680 m.
Total Height of Dam – 51.50 m.

(III)	F.R.L.	- 490.000 m.	Width at Top	- 6.70 m.
(IV)	T.B.L.	- 495.500 m.	Width at Base	- 43.90 m.
(V)	M.W.L.	- 493.000 m.	Upstream slope	- 0.10 : 1.00
(VI)	Gross storage	- 1241 Mm^3	Downstream slope	- 0.83 : 1.00
(VII)	Live storage	- 1091 Mm^3	Density of Masonry	- 23.3 KN/m^3

Seismic Analysis Of Totaladoh Dam by I.S.Code method :

For the study dam section is discredited into 13 segments. Wight of each segment is assumed concentrated at the centre of segment. considering the dam to be mathematically modeled as a cantilever fixes at base and free at top and assumed the dead weight of segments acting horizontally through the centre of respective segments.



CROSS SECTION OF TOTALADOH DAM - NON OVERFLOW SECTION

A) Calculations of Stresses due to horizontal component of earthquake.

As per I.S. 1893:1984 & 2002(Part I) Provision

As the height of dam is less than 100 m. the analysis is to be done by the seismic coefficient method. The horizontal seismic coefficient αh can be calculated as

Horizontal seismic coefficient αh $= \beta I \alpha o$

Where β – coefficient depending upon soil foundation system, for Dams β = 1.00

I – importance factor for dams I = 3.00

αo - as Totaladoh project lies in zone No. II

(latest map in fifth revision of I.S. code) hence αo=0.02

$\therefore$ αh = 1.00 x 3.00 x 0.02 = 0.06

Weight of dam per metre length = 25374.87 KN.

As per I.S.Code 1893 P.no.42

At the top of dam horizontal seismic coefficient shall be taken as 0.75 αh = 1.5x0.06 = 0.09

And reduces linearly zero at base.

$$\therefore \hbar = \frac{\{[6.7x51.5x(51.5/2)]+[1/2x4x51.5x(40/3)]+[1/2x33.2x40x(1/3x40)]\}}{\{(7.6x51.5) + (1/2\ x4x40) + (1/2x33.20x40)\}}$$

$\therefore \hbar =$ 15.70 m.

At section BB

Base shear = V**BB =** 0.60W αh = 0.6 x 25374.87x0.06 = 913.50 KN.

Base Moment = M**BB =** 0.9W $\hbar$ αh = 0.9x 25374.87x15.70x0.06 = 21521 KNm.

Stress at Base = ρ**BB** =(±6 M/T^2) = ± 6 x 21521.81/$(43.90)^2$ = ±67.00 KN/m^2

At section AA

y/h = 11.5/51.5 = 0.223, C'_V=0.20, C'_m=0.09

Shear at AA= V**AA =** C'_V *V**BB** = 0.20x 913.50 = 182.70 KN.

Moment at AA= M**AA =** C'_m M**BB** = 0.09 x 21521.81 = 1936.15 KNm.

Stress at Base = ρ**AA** =(±6 M/T^2) = ± 6 x 1936.15/$(6.70)^2$ = ±258.79 KN/m^2

B) Calculations of Stresses due to vertical component of earthquake.

Load point	Weight of segment	αv= 0.75* αh	W* αv	Lever arm	Moment	Stress Calculations
At section AA						
1	546.39	0.045	24.59	3.35	82.37	X1 = 250.53/74.79 =3.35 m.
2	624.44	0.041	26.19	3.35	87.74	ρmax = W* αv/B(1± 6e/B)
3	624.44	0.038	24.01	3.35	80.43	= 74.19/6.70 (1+0)
Total	1795.27		74.79		250.53	ρmax = ± 11.16 KN/m^2
Section at Base -BB						
4	797.79	0.035	27.88	5.21	145.28	ρmin = 74.19/6.70 (1+0)
5	1144.50	0.031	36.00	7.07	254.53	ρmin = ∓ 11.16 KN/m^2
6	1491.20	0.028	41.70	8.93	372.34	X2 =4555.96/428.09 =12.89 m.
7	1837.90	0.024	44.97	10.79	485.19	X = (74.79*3.55+353.31*12.89)/
8	2184.61	0.021	45.81	12.65	579.54	(74.79+353.31) = 11.25 m.
9	2531.31	0.017	44.24	14.51	641.87	e = b/2 – X2 = 43.90/2-11.25
10	2878.02	0.014	40.24	16.37	658.67	∴ e = 10.70 m.
11	3224.72	0.010	33.81	18.23	616.40	ρmax = W* αv/B(1± 6e/B)
12	3571.42	0.007	24.97	20.09	501.65	=428.09/43.90(1+6*107/43.9)
13	3918.13	0.003	13.69	21.95	300.59	ρmax = ± 24.01 KN/m^2
Total	23579		353.31	20.16	4555.96	ρmin = W* αv/B(1± 6e/B)
			428.09			=428.09/43.90(1-6*107/43.9) ρmin = ∓ 4.50 KN/ m^2

C) Calculations of stresses due to Hydro dynamic pressure of horizontal Earthquake acceleration

y	h	y/h	Cm	Cs	ρe	Vy	My	Stress
At section AA								Cs= Cm/2{y/h(2-y/h) +√ y/h(2-y/h)}
8.50	48.50	0.175	0.70	0.26	11.30	69.76	244.11	αh= 0.06 x1.50 = 0.09
Section at Base -BB								stress are given by. ρ^{AA} = W/T ± 6M/T^2 where W = 0
48.50	48.50	1.00	0.70	0.70	30.55	1075.70	21486.51	= 0 ± 6 x 244.11/(6.70)^2 ρ^{AA} = ± 32.62 KN/m^2 ρ^{BB} = W/T ± 6M/T^2 where W = 0 =0 ± 6 x 21486.50/(43.90)^2 ρ^{BB} = ± 66.89 KN/m^2

D) Calculations of stresses due to Hydro dynamic pressure of Vertical Earthquake acceleration

As per I.S. Code 1893-1984 (Clause 7.3.1.2 P.no.44)

For Seismic coefficient method

At top of dam it would be 0.75 times the value of αh and reduces linearly zero at base.

Therefore in this case, as we are superimposing the stresses due to various conditions separately. The stresses due to vertical component are obtained by multiplying the stresses due to hydrostatic pressure by a factor ± αv i.e. ± 0.03

At section AA

Stresses due to hydrostatic pressure = ± 27.84 KN/m^2

(Calculated as in Static analysis)

∴ Stresses due to vertical component of earthquake= ± 27.84 x 0.03 = ± 0.83 KN/m^2

At section BB(Base)

Stresses due to hydrostatic pressure = ± 22.35 KN/m^2

(Calculated as in Static analysis)

∴ Stresses due to vertical component of earthquake= ± 22.35 x 0.03 = ± 0.67 KN/m^2

VI. Results :

The results obtained of the seismic analysis of existing Totaladoh dam by I.S.Code method are expressed in the table below.

Results showing Stresses at section AA & BB by I.S. Code method (KN/m^2)

Loading Condition	Section AA		Section BB	
Dynamic condition	Upstream	Downstrean	Upstream	Downstrean
a) Horizontal Inertia	±258.79	∓ 258.79	±67.00	∓ 67.00
b) Vertical Inertia	± 11.16	∓ 11.16	±24.01	∓ 4.50
Hydrodynamic Pressure				

c) Horizontal component	± 32.62	∓ 32.62	± 66.89	∓ 66.89
d) Vertical component	± 0.83	∓ 0.83	± 0.67	∓ 0.67
TOTAL	± 303.40	∓ 303.40	± 155.57	∓ 136.06

VII. Conclusions :

As the design of this dam was done at 30 years ago that time the earthquake that may have been treated too lightly and used a very bold method for analysis.The finite element method is too complicated for analysing such structure due to contains of large calculation, but now a day such analysis is possible due to availability of various computer programs. The major work is the seismic analysis of dam by finite element method.

References:

[1]. H.D. Sharma (1981) " concrete Dams – Criteria for Design." University of Roorkee. Metropolitian publication New delhi.
[2]. Dr. D.S Saini (1983), "Dams and Earthquake". Department of civil engineering, University of Roorkee.
[3]. Bureau of Indian Standards Publication IS 1893-1984. Criteria for Earthquake Resistant design of structures.
[4]. Engineering Seismology, Institution of Engineers (India),Kolkata publication journal, Vol. 83, Aug 2002.
[5]. Dams and earthquake Safety. (By John.F. Hall)
[6]. Dissertation Report on Seismic study of gravity dams for earthquake hazard mitigation and rehabilitation there of (N.P.Gahlot)
[7]. Investigation of Liquefaction Failure in Earthen Dam during Bhuj Earthquake (www.nicee.org)
[8]. Earthquake Tips (C.V.R. MURTHY, Dept. of Civil Engg. IIT Kanpur)