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ANALYSIS AND DESIGN OF GRAVITY DAM

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Abstract: This paper present the design and stability analysis of lower wardha dam (a concrete gravity dam situated in wardha river at varud (Baggaji) Dhanodi near arvi in wardha district). Through, the demanding years, it has been observed that failures of dams due to many factor are common. So it is the essential to analysis the dam against all its modes of failures, forces acting on it, uncontrollable disasters such as earthquake, etc. for this, the preliminary data of the dam required for design, such dimensions, base width, crest width, etc. was collected through the inspection engineer, posted at dhanodi lift irrigation office, pipri, dist. Wardha. On the basic collected data the elementary profile and practical profile of dam was estimated. Further all major and the minor force forces acting on dam were calculated, stability analysis of designed dam against all modes of failure and for various load combinations was carried out in STAAD PRO software and was checked permissible limits. Dam structures that span navigable waterways are inherently at a risk for seismic vibrations and as such they must be designed to resist these vibrations. These are very complex structures and subjected to various types of forces in nature. Evaluation of concrete gravity dam for earthquake loading must be based on appropriate criteria that reflect both the desired level of safety and choice of the design and evaluation procedures. In India, the entire country is divided into 3 seismic zones, depending upon the severity of earthquake intensity. Thus main aim of this Project analysis of high concrete gravity dam based on the U.S.B.R. recommendations in seismic zone 2 of India, for varying horizontal earthquake intensities from 0.10 g – 0.30 g with 0.05 g increment to take into account the uncertainty and severity of earthquake intensities and constant other design loads, and to analyze its stability and stress conditions using analytical 2D gravity method and finite element method. Analysis of concrete gravity dam by STAADPRO.

***Index Terms* - Component, formatting, style, styling, insert.**

I. INTRODUCTION

II. INTRODUCTION

Any structure that is constructed will undergo many forces such as wind, seismic, self-weight or forces like ice/snow etc. Among these, seismic forces are natural and as we know earthquake is a natural calamity and is so unpredictable.in order to prevent the structure from being collapse, it's very important to adopt earthquake resistant design philosophy while designing the structure. Waves which arises during seismic event carries very massive speed and when it struck with any structure it travels through foundation to the top roof resulting In-elastic deformation. There may be the possibility of collapse of whole structure or probably it will survive depending upon the design adopted but surely the structure will be costly. Sometimes damages caused by earthquake vibrations very high that goes beyond repairs works. Generally hydraulic structure like concrete gravity dam, canals and RCC multi-storeyed structures are sufficiently stiff and ductile. Concrete gravity dam is a massive structure having many forces acting on it. It's very important for the dam to survive against seismic vibrations. This paper is mainly focused on behaviour of concrete gravity dam with earthquake intensities as per U.S.B.R. recommendation. In order to study the precise behaviour of structures, finite element method plays an important role. These analyses methods can be adopted to study the structures having single degree of system or multi degree of freedom system possessing non-linear characteristics.

I. Concrete gravity dam concrete gravity dam is a solid structure which is made up of concrete or masonry. It acts as a water retaining structure and holds a large amount of water by creating a reservoir on its upstream side. That's why gravity dam is constructed across a river for retaining of water. The cross section of the gravity dam is approximately triangular in shape and having an apex at top and maximum width at bottom. There are various forces acting on the gravity dam mainly hydrostatic pressure, silt pressure, wave pressure, ice pressure, wind forces, self- weight of the dam, uplift pressure and seismic forces etc. The section of the dam is designed in such a way that it would resist all these forces acting on it from various directions under the effect of its own self weight. Gravity dams are also called as solid gravity dams because they are rigid as well as solid and no bending stresses are induced at any point on a dam structure. They are generally straight in plan the upstream face is vertical and slope of downstream face is 0.7:1. For construction , the need good foundations topography to perform better throughout in its lifetime.

II. About the software : STAAD or (STAAD.PRO) is a structural analysis and design computer program originally developed by Research Engineers International in Yorba Linda, CA. in late 2005, Research Engineer International was bought by Bentley Systems. An older version called-III for windows is used by lowa State University for educational purposes for civil and structural engineers. The commercial version STAAD. Pro is one of the most widely used structural analysis and design software. It can also make use of various forms of dynamic analysis from modal extraction to time history and response spectrum analysis.

III. Finite Element Modeling of The Dam

The dam body is modeled in STAADpro using the solid isoparametric finite element with eight nodes. Each node has three translational degrees of freedom. The stiffness matrix of the solid element is evaluated by numerical integration with eight Gauss – Legendre points. The dam is analyzed for several basic loads and loads combinations possibly met with during its service. The stresses induced are checked for all the combinations and the dimensions.

IV. Scope

A concrete gravity dam, as discussed in this paper, is a solid concrete structure so designed and shaped that its weight is sufficient to ensure stability against the effects of all imposed forces. Other types of dams exist which also maintain their stability through the principle of gravity, such as buttress and hollow gravity dams, but these are outsidethe scope of this paper. Further, discussions in this paper are limited to damson rock foundations and do not include smaller

dams generally less than 15 m high which are discussed in the Bureau of Reclamation publication “Design of SmallDams”.

The complete design of a concrete gravity dam includes not only the determination of the most efficient and economical proportions for the water impounding structure, but also the determination of the most suitable appurtenant structures for the control and release of the impounded water consistent with the purpose or function of the project. This paper presents the basic assumptions, design considerations, methods of analysis, and procedures used by designers within the Engineering and Research Centre, Bureau of Reclamation, for the design of a gravity damand its appurtenances.

vi. classification

Gravity dams may be classified by plan as straight gravity dams and curved gravity dams, depending upon the axis alignment. The principal difference in these two classes is in the method of analysis. Whereas a straight gravity dam would be analysed by one of the gravity methods discussed in this paper, a curved gravity dam would be analysed as an arch dam structure. For statistical purposes, gravity dams areclassified with reference to their structural height. Dams up to30 m high are generally considered as low dams, dams from 30 m to 90 m high as medium-height dams, and dams over 90 m high as high dams.

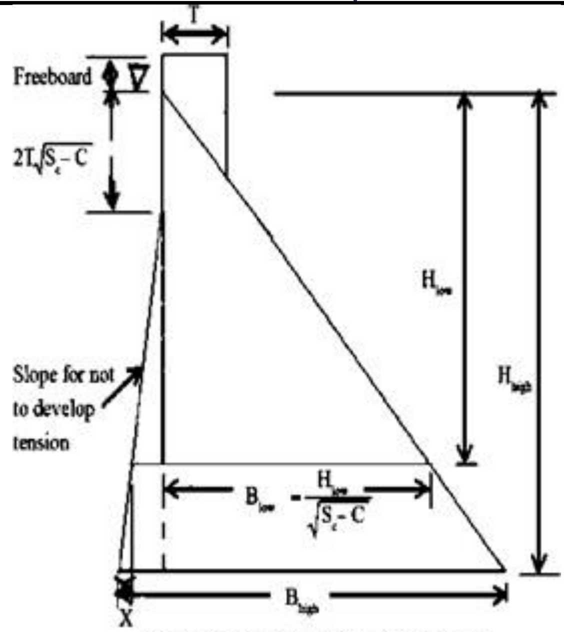

Fig.1 Elementary profile of Concrete Gravity Dam

Design Philoscophy

The Bureau of Reclamation’s philosophy of concrete dam design is founded on rational and consistent criteria which provide for safe, economical, functional, durable, and easily maintained structures. It is desirable, therefore, to establish, maintain, and update design criteria. Under special conditions, consideration can be given to deviating fromthese standards. In those situations, the designer bears the responsibility for any deviation and, therefore, should be careful to consider all ramifications. Accordingly, each of the criteria definitions in this monograph is preceded by a discussion of the underlying considerations to explain the basis of the criterion. This serves as a guide in appraising thewisdom of deviating from a particular criterion for special conditions.

The line of the upstream side of the dam or the line of the coronet of the dam if the upstream side in slanting, is considered as the orientation line for plan purposes, etc. and is known as the “Base line of the Dam” or the “Axis of the Dam”. When appropriate circumstances are on hand, such dams can be constructed up to immense heights. The ratio ofbase width to height of high gravity dams is generally less than 1:1. A typical cross-section of a high concrete gravity dam is shown in figure alongside. The upstream face may be kept throughout perpendicular or partially slanting for some of its length. A drainage passage is usually provided in orderto lessen the uplift pressure formed by the seeping water. Purposes valid to dam creation may include routing, flood damage reduction, hydroelectric power creation, fish andwildlife improvement, water superiority, water supply, and amusement. Several concrete gravity dams have been in use for more than five decades, and over this phase significant advances in the methodologies for assessment of natural phenomena hazards have caused the design-basis events for these dams to be revised upwards. Older existing dams may fail to meet revised safety criteria and structural rehabilitation to meet such criteria may be costly and difficult.

viii. Causes of failure

The incident of failures demonstrates that depending on thetype of dam, the cause of failure may be classified as:

a) Hydraulic failures; (for all types of dams)
b) Failures due to seepage.
   (i) Through foundation, (all except arch dams)
   (ii) Through body of dam (embankment dam)
c) Failures due to stresses developed within structure.

Arch dams fail instantaneously, whereas the gravity dams take some multiples of 10minutes. A study of dam failures in the world has revealed the percentage distribution ofdam breaks and its attributes cause of failure.

1. Foundation problems 40 %
2. Inadequate spillway 23 %
3. Poor construction 12 %
4. Uneven settlement 10 %
5. High pore pressure 5 %
6. Acts of war 3 %
7. Embankment slips 2 %
8. Defective materials 2 %
9. Incorrect operations 2 %
10. Earthquakes 1 %

Other surveys of dam failure have been cited by, who estimated failure rates from 2×10^{-4} to 7×10^{-4} per dam year based on these surveys.

IX. Foundation failure

A seismic analysis must consider not only the effects ofground motions on the structure, but also their impact uponthe strength of the foundation and abutments. Two maintypes of foundation failure need to be considered:Deformation, settlement, and fault movement Liquefaction The dynamic strength of bedding planes and shear zones inthe foundation is usually lower than their static strength.During earthquakes, movements can occur along faults orother weak zones in the foundation and abutments. Thesemovements can cause a variety of problems:

1) Excessive movements can cause tensile cracking in the dam, which could possibly lead to dam failure.
2) Excessive movements can open up faults and cracks in the foundation, which may result in increased seepage and a corresponding rapid increase in uplift pressures.
3) Infiltration of water along bedding planes due to open cracks and fissures can cause reduced foundation shear strength. For gravity dams, the potential for sliding is greatest when the bedding planes are horizontal or they dip in the upstream direction.
4) Infiltration of water along bedding planes can lead to erosion of joint filler or shear zone material by piping.
5) This process can cause strength reduction as well as lead to large settlements or undermining of the dam.

x. Concrete Distress

Cracking of concrete structures may result from excessive tensile or compressive stresses, high impact loads such as barges or ice, or differential movements of foundation and abutment materials. In spillways or outlet works conveying high velocity flows, offsets in the concrete surfaces may cause cavitation. Vibration of structures by earthquake, water surges, or equipment operation may also damage concrete.

XII. Methodology and Design of High GravityDam

Check the stability of Typical section of gravity dam asshown in fig.2. For reservoir empty and full condition considering seismic forces assume reasonable value of uplift and a line of drain holes 6m downstream of the upstream face for the purpose of this check assume water level at the top of dam and no tail water. Also find principal and shear stresses at the toe & heel of dam. Take unit weight of concrete23.5kN/m³ shear strength of concrete as 1400 kN/m²and μ=0.7.The Specific Weight of Water is 9.81 kN/m³.

The allowable compressive stress 3000 kN/m² of dam material is exceeds for concrete, the dam may crush and fail by crushing. The maximum permissible tensile stress for high concrete gravity dam under worst loadings may betaken as 500 kN/m².

(i) Stability check of Concrete Gravity dam withoutconsidering seismic forces

RESERVOIR EMPTY: CASE 1

When the reservoir is empty, the various forces acting are worked out in Table 2 with reference to Fig. 2.Horizontal earthquake forces acting towards upstream are considered. Stability is examined for two sub-cases, i.e.

when vertical earthquake forces are additive to the weight of the dam, (b) when vertical earthquake forces aresubtractive to the weight of the dam. A value of 0.1g to 0.15 g is generally sufficient for high dams in seismic zones for horizontal seismic coefficien . We assume a value of 0.1g as horizontal and 0.05g for vertical seismic coefficients respectively.

Table 1. Force acting on the dam in reservoir empty case

Name of force	Designation	Magnitude force of vertical direction	Magnitude force of horizontal direction	Lever arm about toe (m)	Moment about toe in anticlockwise (KN.m)
Weight of dam	W_1 W_2 W_3	½ x 0.33 x 63.3 x 24 = 250.668 5.18x91.2= 472.416 ½ x 51.62 x 72.1 x 24 = 44661.62 $\sum V_1$ = 45384.70		56.91 54.21 34.41	14265.51 25609.67 1536806.34 $\sum M_1$ =1576681.52
Horizontal earthquake forces	P_{W1} P_{W2} P_{W3}		0.1 X W_1 = 0.1 x 250.668 0.1 X W_2 = 0.1 x 472.416 0.1 X W_3 = 0.1 x 44661.62 $\sum H$ = 5188.16	21.1 45.6 48.41	5275 21523.2 216206.81 $\sum M_2$ = 243005.01
Vertical earthquake force		$\sum V_2$ = 0.05 x $\sum V_1$ =0.05x45384.70 = 226923.5			$\sum M_2$ = 0.05 x $\sum M_1$ = 0.05 X 1576681.52 = 78834.08

Reservoir Full: Case 2

Horizontal earthquake moving towards the reservoir causing upstream acceleration and producing horizontal inertia forces towards downstream is considered, as it is the worst case for this condition. Similarly a vertical earthquake moving downward and producing forces upward, i.e. subtractive to the weight of the dam is considered. Full uplift pressure is considered. It is assumed that there is no tail water in the downstream face.

Fig. 3 shows the various forces acting on the dam in this condition. Magnitude and moment of these forces aboutthe toe are listed in Table 3. P_e is the hydrodynamic pressure, its magnitude and moment caused by it is calculatedfrom Zanger's formula (equation 7) as follow

$$Pe = 0.726 P_e H$$

Where,

$P_e = C_m . k_h . \gamma_w . H$

& $C_m = 0.735 \text{ X } \theta / 90^0 = 0.735 \text{ X } 81.9 /90 = 0.668$

$P_e = 0.668 \times 0.1 \times 10 \times 72.1 = 48.16\ KN/m^2$
$Pe = 0.726 \times 48.16 \text{ X } 72.1 = 2520.91 KN.$
$M_e = 0.412 P_e . H$
$M_e = 0.412 \times 2520.91 \times 72.1 = 74884.135\ KN.m$

Table 2 Force acting on dam reservoir full case

Name of force	Designation	Magnitude of force in vertical (KN)	Magnitude of force in horizontal (KN)	Lever arm about toe (m)	Moment about toe anti-clockwise in (KN-m)
Weight of dam	W1 W2 W3	½ x 0.33 x 63.3 x 24=250.668. 5.18 x 91.2 = 472.416 ½ x 51.62 x 72.1x 24 =44661.62 $\sum V_1$ = 45384.70		56.91 54.21 34.41	14265.51 25609.67 1536806.34 $\sum M_1$=1576681.52
Weight of water supported on u/s slope water on d/s slope		24.9 x 0.33x 1x 10 = 8217 ½ x 63.3 x 1x 10= 3165 ½ x 0.33x 5.24x 1x 10=1646 $\sum V_1$ = 12246		56.97 57.02 1.74	4681225 1804683 150 $\sum M_2$=6486058
Uplift forces	U_1 U_2	(-)57.13 x 36 x 10 = 2056 (-) ½ x57.13x 48x10 = 137112 $\sum V_3$ = (-)13916		19.04 38.09	-39146 522259 $\sum M_3$=(-)561405

Upward vertical earthquake force 0.05W		$\sum V_4$ = (-)0.05. $\sum V_1$ = 0.05 x 4538470 = (-)22692			$\sum M_4$=(-)0.05$\sum M_1$ = 0.05 x 1576681 $\sum M_4 = (-)78834$
Horizontal hydrostatic pressure	p		(-)1/2 x88.2 x88.2x1x10 =(-)38896 ½ x 0.33x0.33x1x10 = 54 $\sum H_1 = 38950$	29.4 10	(-) 1143542 (-)54 $\sum M_4 = (-)78834$
Horizontal hydro-dynamic pressure	P_e		Calculate separately earlier $\sum H_2$= -18674		Calculate separately earlier $\sum Me = 126500$
Horizontal inertia force due to earthquake	P_{w1} P_{w2} P_{w3}		(-)0.1W_1 = 250 (-)0.1W_2 = 472 (-)0.1W_3 =4466.16 $\sum H_3 = 5188.16$	21.1 45.61 48.41	(-) 5275 (-)21523.2 (-)216206.81 $\sum M_7 = 243005.01$

Case I: When reservoir empty condition (From Table 1)

When reservoir is empty, only self-weight of the dam will be acting as force. Other forces namely water pressure and uplift will be zero. The resulting force $\sum V_1$ and resulting moment $\sum M_1$ for this case has been worked out table 1.

Position of resultant from toe :

$$X = \frac{\sum M_1}{\sum V_1} = \frac{1576681.52}{45384.70} = 34.74\text{m}$$

Its distance from centre is

$$e = \frac{b}{2} - X = \frac{57.13}{2} - 34.74\ m$$

(i.e., the resultant falls to the left of the centre)

Normal compressive stress at toe:

$$P_n = \frac{\sum V_1}{b}\left[1 + \frac{6e}{b}\right]$$
$$= \frac{45384.70}{57.13}\left[1 + \frac{6 \times 34.74}{57.13}\right]$$
$$= 279.2 \text{ KN/m}^2 \text{ compressive}$$

Normal compressive stress at heel:

$$P_n = \frac{\sum V_1}{b}\left[1 - \frac{6e}{b}\right]$$
$$= \frac{45384.70}{57.13}\left[1 - \frac{6 \times 34.74}{57.13}\right]$$
$$= 1309.60 \text{ KN/m2.}$$

Principal stress at toe:

$$\sigma_1 = P_n \sec^2 \emptyset$$

Where, $\tan \emptyset = 0.7$, and $\sec^2 \emptyset = 1.49$

$$\sigma_1 = P_n \sec^2 \emptyset$$

$$\sigma_1 = 279.2 \times 1.49$$

$$= 416.01 \text{ KN/m}^2.$$

Principal stress at heel:

$$\sigma = P_n \sec^2 \theta$$

Where, $\tan \theta = 0.1$ and $\sec^2 \theta = 1.01$

$$\sigma = P_n \sec^2 \theta$$

$$\sigma = 1309.60 \times 0.1$$

$$= 130.96 \text{ KN/m}^2.$$

Shear stress at toe:

$$\tau = P_n \tan \varphi$$

$$= 416 \times 0.7$$

$$= 291.2 \text{ KN/m}^2$$

Shear stress at heel:

$$\tau = P_n \tan \varphi$$

$$= 130.96 \times 0.1$$

$$= 13.09 \text{ KN/m}^2$$

Note that there cannot be any sliding or overturning when reservoir is empty.

Case II: When reservoir empty but vertical earthquake force acting downward.

Sometimes values of stresses at toe and heel are worked out without considering uplift as the vertical earthquake forces are maximum in this case.

Calculation of stresses: $tan\theta = 0.1$; $\sec^2 \theta$ =1.01; $tan\emptyset = 0.7$; $\sec^2 \emptyset = 1.49$

The values of vertical forces $\sum V_2$ and moments $\sum M_2$ have been worked out in table 1 where the $\sum V_2$ and $\sum M_2$ represent the sum of vertical force and sum of moment of all forces when the reservoir is empty represent the vertical earthquake force acting on dam.

$$\sum M = \sum M_1 + \sum M_2 + \sum M_3$$

$$= 1576681.52 + 243005.01 + 78834.08$$

$$= 1898520.61 \text{ KN/m}$$

Also $\sum V = \sum V_1 + \sum V_2$

$$= 45384.70 + 226923.5$$

$$= 272308.2 \text{ KN}$$

Position of the resultant from the toe is

$$X = \frac{\sum M}{\sum V} = \frac{1898520.61}{272308.2} = 6.97\text{m}$$

Position of the resultant from the centre of the base is

$$e = \frac{b}{2} - X = \frac{57.13}{2} - 6.97 = 21.59\ m > \frac{B}{6} = 9.52m, \text{ hence safe}$$

Resultant acts near the heel and slight tension will develop at toe.

Normal compressive stress at toe:

$$P_n = \frac{\sum V}{b}\left[1 + \frac{6e}{b}\right]$$

$$= \frac{272308.2}{57.13}\left[1 + \frac{6 \times 21.59}{57.13}\right]$$

$$= 1557 \text{ KN/m}^2, \text{ which is} \leq 3000 \text{ KN/m}^2 \text{ (safe)}$$

Normal compressive stress at heel:

$$P_n = \frac{\sum V}{b}\left[1 - \frac{6e}{b}\right]$$

$$= \frac{272308.2}{57.13}\left[1 - \frac{6 \times 21.59}{57.13}\right]$$

$$= -60 \text{ KN/m}^2 \text{ Which is} \leq 420 \text{ KN/m}^2 \text{ (safe)}$$

Principal stress at toe:

$$\sigma_1 = P_n \sec^2 \emptyset$$

Where, $\tan \emptyset = 0.33/31.65 = 0.1$, and $\sec^2 \emptyset = 1.49$

$$\sigma_1 = P_n \sec^2 \emptyset$$

$$\sigma_1 = 1577 \times 1.49$$

= 2349.73 KN/m^2; which is ≤ 3000 KN/m^2 (safe)

Principal stress at heel:

$$\sigma = P_n \, sec^2 \, \theta$$

Where, tan θ = 0.1 and $sec^2 \, \theta$ = 1.01

$$\sigma = P_n \, sec^2 \, \theta$$

σ = -60 x 1.01

= -60.6 KN/m^2. Which is ≤ 420 KN/m^2 (safe)

Shear stress at toe:

$$\tau = P_n \tan \varphi$$

= 1577 x 0.7

= 1103.9KN/m^2

Shear stress at heel:

$$\tau = P_n \tan \varphi$$

= -60x 0.1 =-6 KN/m^2

Case III: When reservoir empty but vertical earthquake force acting Upward.

$\sum M = \sum M_1 + \sum M_2 + \sum M_3$

= 1576681.52 + 243005.01-78834.08

= 17408525 KN/m

Also $\sum V = \sum V_1 - \sum V_2$

= 45384.70 -226923.5

= 43115 KN

Position of resultant from toe :

$$X = \frac{\sum M}{\sum V} = \frac{17408525}{43115} = 9.6\text{m}$$

Its distance from centre is

$$e = \frac{b}{2} - X = \frac{57.13}{2} - 9.6 = 18.97 \, m > \frac{B}{6} = 9.52m\text{, hence safe}$$

[-ve sign shows that resultant lies near the heel and therefore, tension will develop at toe.]

Normal compressive stress at toe:

$$P_n = \frac{\sum V}{b}\left[1 + \frac{6e}{b}\right]$$

$$= \frac{43115}{57.13}\left[1 + \frac{6 \times 18.52}{57.13}\right]$$

= 2258.2 KN/m^2, which is ≤ 3000 KN/m^2 (safe)

Normal compressive stress at heel:

$$P_n = \frac{\sum V}{b}\left[1 - \frac{6e}{b}\right]$$

$$= \frac{43115}{57.13}\left[1 - \frac{6 \times 18.52}{57.13}\right]$$

= -74.8 KN/m^2 Which is ≤ 420 KN/m^2 (safe)

Principal stress at toe:

$$\sigma_1 = P_n \, sec^2 \, \emptyset$$

Where, tan $\emptyset$ = 0.33/31.65= 0.1, and $sec^2 \, \emptyset = 1.49$

$$\sigma_1 = P_n \, sec^2 \, \emptyset$$

σ_1 = 2258.2 x 1.49

= 111.45 KN/m^2; which is ≤ 3000 KN/m^2 (safe)

Principal stress at heel:

$$\sigma = P_n \, sec^2 \, \theta$$

Where, tan θ = 0.1 and $sec^2 \, \theta$ = 1.01

$$\sigma = P_n \, sec^2 \, \theta$$

σ = -74.8 x 1.01

= -75.54KN/m^2. Which is ≤ 420 KN/m^2 (safe)

Shear stress at toe:

$$\tau = P_n \tan \varphi$$

= 2258.2 x 0.7

= 1580.74KN/m^2

Shear stress at heel:

$$\tau = P_n \tan\varphi$$
$$= -74.8 \times 0.1 = -7.48 \text{ KN/m}^2$$

When reservoir is full (From table 2)

Horizontal earthquake moving towards the reservoir causing upstream acceleration, and thus producing horizontal forces towards downstream is considered, as it is the worst case for this condition. Similarly, a vertical earthquake moving downward and thus, producing forces upwards, i.e subtractive to the weight of the dam is considered. the uplift coefficient C is taken as equal to 0.6, as given in the equation, and thus uplift pressure diagram as shown in fig.1.2 (c), is developed.

The various forces acting in this case are:

i) Hydrostatic pressure P and P·
ii) Hydrodynamic pressure Pe (Pe' is neglected as it is very small and neglection is on conservative side.)
iii) Uplift forces U1 and U2.
iv) Weight of the dam, W_1, W_2 and W_3.
v) Horizontal inertial earthquake forces acting towards downstream, equal to $0.1W_1$, $0.1W_2$, $0.1W_3$ at c.gs. of these weights W_1, W_2, and W_3 respectively.
vi) A vertical force equal to 0.05W or ($0.05\sum V_1$) acting upward.

Calculation of Pe

Pe and the moment due to this hydrodynamic force is calculated, and then all the forces and their moments are tabulated in table 1.2(b).

Calculation of Pe from Zanger's formulas

Pe = 0.726 pe.H

Where pe = Cm. Kh. Yw . H

And $\text{Cm} = 0.735\,\frac{\theta}{90^0}$

Since the u/s inclined face is extended for more than half the depth, the overall slope up to the whole height may be taken.

$$\therefore \tan\theta = \frac{72.1}{5.18} = 13.91$$

$$\theta = 13.91$$

$$\therefore C_m = 0.735 \times \frac{\theta}{90^0}$$

$$= 0.735 \times \frac{81.9}{90}$$
$$= 0.668$$

Pe = 0.668 x 0.1 x 10 x72.1 = 48.16
Pe = 0.726 x 48.16 x 72.1 = 2520.91 KN
Me = 0.412. Pe. H = 0.412 x 2520.91 x 72.1= 74884.135kn.m

Case I: Reservoir full with all forces including uplift.

The values of vertical forces $\sum$ V and moments $\sum M$ have been worked out in table 1 where the $\sum$ V and $\sum M$ represent the sum of vertical force and sum of moment of all forces when the reservoir is empty represent the vertical earthquake force acting on dam.

$$\sum M = 157668.52 + 6486058 - 561405 - 78834 - 1143596 - 1265500 - 243005.01$$
$$= 1989184.91 KN/m$$

$$\sum V = 4538470 + 12246 - 13916 - 22692$$
$$= 4514108 KN$$

Position of the resultant from the toe is

$$X = \frac{\sum M}{\sum V} = \frac{1989184.9}{4514108} = 0.44 m$$

Position of the resultant from the centre of the base is

$$e = \frac{b}{2} - X = \frac{57.13}{2} - 0.44 = 28.13\ m > \frac{B}{6} = 9.52m, \text{ hence safe}$$

The resultant is nearer the toe and tension is developed at the heel

Normal compressive stress at toe:

$$P_n = \frac{\sum V}{b}\left[1 + \frac{6e}{b}\right]$$
$$= \frac{4514108}{57.13}\left[1 + \frac{6 \times 28.13}{57.13}\right]$$
$$= 312.4 \text{ KN/m}^2, \text{ which is} \leq 3000 \text{ KN/m}^2 \text{ (safe)}$$

Normal compressive stress at heel:

$$P_n = \frac{\sum V}{b}\left[1 - \frac{6e}{b}\right]$$
$$= \frac{4514108}{57.13}\left[1 - \frac{6 \times 28.13}{57.13}\right]$$
$$= -1544.2 \text{ KN/m}^2 \text{ Which is} \leq 420 \text{ KN/m}^2 \text{ (safe)}$$

Principal stress at toe:

$\sigma_1 = P_n\, sec^2\, \emptyset$

Where, tan $\emptyset$ = 0.33/31.65= 0.1, and $sec^2\, \emptyset = 1.49$

$$\sigma_1 = P_n\, sec^2\, \emptyset$$
$$\sigma_1 = 312.4 \text{ x } 1.49$$
$$= 465.48 \text{ KN/m}^2; \text{ which is} \leq 3000 \text{ KN/m}^2 \text{ (safe)}$$

Principal stress at heel:

$$\sigma = P_n\, sec^2\, \theta$$

Where, tan θ = 0.1 and $sec^2\, \theta$ = 1.01

$$\sigma = P_n\, sec^2\, \theta$$
$$\sigma = -1544.2 \text{x } 1.01$$
$$= -155.9 \text{ KN/m}^2. \text{ Which is} \leq 420 \text{ KN/m}^2 \text{ (safe)}$$

Shear stress at toe:

$$\tau = P_n \tan \varphi$$
$$= 312.4 \text{ x } 0.7$$
$$= 218.4 \text{KN/m}^2$$

Shear stress at heel:

$$\tau = P_n \tan \varphi$$
$$= - (1544.2 - 894.67) \text{ x } 0.1$$
$$= - 64 \text{ KN/m}^2$$

Factor of safety against overturning

$$= \frac{\sum M_+}{\sum M_-} = \frac{2619262.8}{1681041.6} = 1.55; \text{ Which is} > 1.5 \text{ (Hence, safe)}$$

Factor of safety against sliding

$$= \frac{u \sum V_3}{\sum H} = \frac{0.70 \times 13916}{41587.2} = 0.23; \text{ which is} > 2 \text{(Hence, unsafe)}$$

Shear friction factor

$$\text{S.F.F} = \frac{u \sum V_3 + b.c}{\sum H} = \frac{0.70 \times 13916 + 57.13 \times 1}{41587.2} = 3.25 \text{ ; which is} > 3 \text{ (Hence, safe)}$$

Safety against sliding according IS6512-1984:

Taking F_φ =1.5 and F_c =3.6 for load combination B,

$$F = \frac{\frac{u \sum V}{F\varphi} + \frac{cb}{F_C}}{\sum H} = \frac{\frac{0.7 \times 13916}{1.5} + \frac{2200 \times 57.13}{3.6}}{41587.2} = 0.99 \cong 1.0 \text{ ;Which is} > 1.0 \text{(Hence, slightly safe)}$$

Case II: Reservoir full with all forces without uplift:

Sometimes values of stresses at toe and heel are worked out without considering uplift as the vertical forces are maximum in this case.

Calculation of stresses: $\tan \theta = 0.1$; $sec^2\theta = 1.01$; $\tan\emptyset = 0.7$; $\text{Sec}^2\emptyset = 1.49$

The value of vertical forces $\sum V_2$ and moments $\sum M_2$ have been worked out in table where the $\sum V_2$ and $\sum M_2$ represent the sum of vertical forces and sum of moments of all forces when the reservoir is full but when uplift is not acting.

$$\sum M = 157668.52 + 6486058 - 78834 - 1143596 - 1265500 - 243005.01$$

= 3912791.51KN/m

$\sum V$ = 4538470 + 12246 – 22692

= 4528024KN

Position of the resultant from the toe is

$$\text{X} = \frac{\sum M}{\sum V} = \frac{3912791.51}{4528024} = 0.86\text{m}$$

Position of the resultant from the centre of the base is

$$e = \frac{b}{2} - X = \frac{57.13}{2} - 0.86 = 27.70\ m > \frac{B}{6} = 9.52m, \text{ hence safe}$$

Normal compressive stress at toe:

$$P_n = \frac{\sum V}{b}\left[1 + \frac{6e}{b}\right]$$

$$= \frac{4528024}{57.13}\left[1 + \frac{6 \times 27.70}{57.13}\right]$$

= 309.82 KN/m^2, which is $\leq$ 3000 KN/m^2 (safe)

Normal compressive stress at heel:

$$P_n = \frac{\sum V}{b}\left[1 - \frac{6e}{b}\right]$$

$$= \frac{4528024}{57.13}\left[1 - \frac{6 \times 27.70}{57.13}\right]$$

= -151.32KN/m^2 Which is $\leq$ 420 KN/m^2 (safe)

Principal stress at toe:

$\sigma_1 = P_n\, sec^2\, \emptyset$

Where, tan $\emptyset$ = 0.33/31.65= 0.1, and $sec^2\, \emptyset = 1.49$

$\sigma_1 = P_n\, sec^2\, \emptyset$

σ_1 = 309.82 x 1.49

= 461.63 KN/m^2; which is $\leq$ 3000 KN/m^2 (safe)

Principal stress at heel:

$$\sigma = P_n \sec^2 \theta$$

Where, $\tan\theta$ = 0.1 and $\sec^2\theta$ = 1.01

$$\sigma = P_n \sec^2 \theta$$

σ =-151.32 x 1.01

= -152.83 KN/m^2. Which is $\leq$ 420 KN/m^2 (safe)

Shear stress at toe:

$$\tau = P_n \tan\varphi$$

= 309.82 x 0.7

= 216.87KN/m^2

Shear stress at heel:

$$\tau = P_n \tan\varphi$$

= - (151.32 -894.67) x 0.1

=- 74.34 KN/m^2

Stability check of dam by considering seismic forces

For worst condition consider that:

a) Horizontal earthquake acceleration acts upstream.
b) Vertical earthquake acceleration acts downwards.

Hydrodynamic pressure due to water caused by earthquake can be found out from Zanger's formula. Since the slope is upto middle depth, approximately value of θ Can be found out by joining heel to the upstream edge,

Calculation of Pe from Zanger's formulas

Pe = 0.726 pe.H

Where pe = Cm. Kh. Yw . H

And Cm = 0.735 $\frac{\theta}{90^0}$

$\therefore \tan\theta = \frac{0.33}{91.2} = 0.003$

$\theta = 1.7183$

$\therefore C_m$ = 0.735 x $\frac{\theta}{90^0}$

= 0.735 x $\frac{1.7183}{90}$

= 0.7209

Pe = 0.7209 x 0.1 x 10 x91.2 = 65.74 KN/m^2

Pe = 0.7209 x 65.74 x 91.2 = 4322.15 KN

Me = 0.412. Pe. H = 0.412 x 4322.15 x 91.2= 126500KN/m^2

$$\sum M = 157668.52 + 6486058 - 561405 - 78834 - 1143596 - 1265500 - 243005.01$$

= 1989184.91KN/m

$\sum V$ = 4538470 + 12246 – 13916 – 22692

= 4514108KN

Position of the resultant from the toe is

$$X = \frac{\sum M}{\sum V} = \frac{1989184.9}{4514108} = 0.44\text{m}$$

Position of the resultant from the centre of the base is

$$e = \frac{b}{2} - X = \frac{57.13}{2} - 0.44 = 28.13\ m > \frac{B}{6} = 9.52m, \text{ hence safe}$$

The resultant is nearer the toe and tension is developed at the heel

Normal compressive stress at toe:

$$P_n = \frac{\sum V}{b}\left[1 + \frac{6e}{b}\right]$$
$$= \frac{4514108}{57.13}\left[1 + \frac{6 \times 28.13}{57.13}\right]$$
$$= 312.4\ \text{KN/m}^2, \text{ which is} \leq 3000\ \text{KN/m}^2\ \text{(safe)}$$

Normal compressive stress at heel:

$$P_n = \frac{\sum V}{b}\left[1 - \frac{6e}{b}\right]$$
$$= \frac{4514108}{57.13}\left[1 - \frac{6 \times 28.13}{57.13}\right]$$
$$= -1544.2\ \text{KN/m}^2\ \text{Which is} \leq 420\ \text{KN/m}^2\ \text{(safe)}$$

Principal stress at toe:

$$\sigma_1 = P_n\, sec^2\, \emptyset$$

Where, tan $\emptyset$ = 0.33/31.65= 0.1, and $sec^2\,\emptyset = 1.49$

$$\sigma_1 = P_n\, sec^2\, \emptyset$$
$$\sigma_1 = 312.4 \times 1.49$$
$$= 465.48\ \text{KN/m}^2;\ \text{which is} \leq 3000\ \text{KN/m}^2\ \text{(safe)}$$

Principal stress at heel:

$$\sigma = P_n\, sec^2\, \theta$$

Where, tan θ = 0.1 and $sec^2\,\theta = 1.01$

$$\sigma = P_n\, sec^2\, \theta$$
$$\sigma = -1544.2 \times 1.01$$
$$= -155.9\ \text{KN/m}^2.\ \text{Which is} \leq 420\ \text{KN/m}^2\ \text{(safe)}$$

Shear stress at toe:

$$\tau = P_n \tan\varphi$$
$$= 312.4 \times 0.7$$
$$= 218.4\text{KN/m}^2$$

Shear stress at heel:

$$\tau = P_n \tan\varphi$$
$$= -(1544.2 - 894.67) \times 0.1$$
$$= -64\ \text{KN/m}^2$$

Calculation of factor of safety

Factor of safety against overturning

$$= \frac{\sum M_+}{\sum M_-} = \frac{2619262.8}{243005.01} = 1.0;\ \text{Which is} < 1.5\ \text{(Hence, unsafe)}$$

Factor of safety against sliding

$$= \frac{u \sum V_3}{\sum H} = \frac{0.70 \times 22692.35}{41587.2} = 0.38;\ \text{which is} < 1.5\text{(Hence, unsafe)}$$

Shear friction factor

$$\text{S.F.F} = \frac{u \sum V_3 + b.c}{\sum H} = \frac{0.70 \times 22692.35 + 57.13 \times 1400}{51881.6} = 1.84;\ \text{which is} < 3\ \text{(Hence, safe)}$$

Safety against sliding according IS6512-1984:

Taking F_φ =1.2 and F_c =2.7 for load combination E

$$F = \frac{\frac{u \sum V}{F_\varphi} + \frac{cb}{F_c}}{\sum H} = \frac{\frac{0.7 \times 22692.35}{1.2} + \frac{1400 \times 57.13}{2.7}}{51881.6} = 0.59;\ \text{Which is} > 1\text{(Hence, unsafe)}$$

Observation and Results:

Table 1.1 Results for Maximum stresses

(Stability check for Concrete Gravity Dam with considering all Forces Manually)

Sr.No	Stress at toe		Stress at heel	
	Max. Principal (KN/m^2)	Max. Shear (KN/m^2)	Max. Principal (KN/m^2)	Max. Shear (KN/m^2)
1.	**Case I: Reservoir Empty Condition** 416.01	291.2	130.96	13.09
2	**Case II: Reservoir Full With No Uplift** 461.63	216.87	-152.83	74.34
3	**Case III: Reservoir Full with Uplift(considering seismic forces)** 465.48	218.4	-155.9	64

Table2: Results and Observation

Comparative results are tabulated for problem analysed with and without seismic forces manually

Sr. No	Action/ consideration	Magnitude	Effect	Check	Comment
In absence of Seismic forces					
1	F.O.S.against overturning	1.55	Overturning	safe	stable
2	F.O.S.agai nstsliding	0.23	Sliding	unsafe	unstable
3	Shear friction factor	3.25	Shearing	safe	stable
4	Safety again stsliding As per IS 6512-1984	1.0	Sliding	Slightly safe	stable
Seismic forces effect					
1	F.O.S.agai nst overturnin g	1.0	Overturnin g	unsafe	unstable
2	F.O.S.against sliding	0.38	Sliding	unsafe	unstable
3	Shear friction factor	1.84	Shearing	safe	stable
4	Safety again stsliding As per IS 6512-1984	0.82	Sliding	unsafe	unstable

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