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# COMPARISON OF THE STRUCTURAL STABILITY OF GRAVITY CONCRETE DAMS USING USACE AND USBR STANDARDS

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**ABSTRACT**

Safety factors play an important role in the analysis of structural stability of gravity concrete dams. In this work the study of these factorswas made according to two standard methods, USBR and USACE, which are varied in the procedure and calculation of the factors against overturning and sliding, in addition tothe difference in their acceptable limits. The results obtained from the two standards did not show substantial difference when the dambase is horizontal. To avoid the sliding phenomena, the dam base must be inclined, the cohesion at the concrete-rock contact must be raised to a value achieving the desired safety factors; or a passive wedge has to be used at the downstream face to increase the sliding resistance. The study of stability criteria was done on many virtual dam cases, to obtain the height of water for safe operation and the strength of concrete, consequently the cohesion required. The value of cohesion required by USACE is smaller than that of the USBR for the various loading conditions; also the USACE calculations permit water elevations higher than those for USBR calculations. However, the two standards use the same procedure to evaluate the stresses in the mass of the dam. The behavior of the dam has been modeled and analyzed using analytically 2-dimensional gravity method and FEM with the help of ABAQUS software package in order to ensure the safe performance of the dam. Stresses were found acceptable in all profiles, where it is important to prevent undesirable tensile stresses at the heel, and to avoid crushing at the toe.

**Keywords:** concrete, gravity, dam, stability, stress, sliding, overturning.

## INTRODUCTION

Concrete gravity dam is a solid structure built of mass concrete material; therefore, it maintains its stability against all imposed forces from geometric shape, mass and strength of concrete. The study will show the effect of various loading combinations on the stability and stress analysis of many virtual profiles of gravity dams with normal water height of 30m and maximum water height of 33.6m; so to compare the stability methods and stress analysis procedure used by two standards methods, USBR and USACE.

The section of concrete gravity dam should be chosen in such a way that it is the most economic section and fulfills all the conditions and requirements of stability.

The preliminary cross-section of the concrete gravity dam (elementary section) has a triangular shape in the beginning of the design. Many configurations will be made on this profile to achieve further structural stability. These configurations are: top width, freeboard, batter at upstream face and suitable slope of downstream face. One of the most essential and important modifications affecting the stability of the dam is the slope of concrete-rock contact surface. In some conditions it may be appropriate to include downstream passive wedge resistance as a further component of the resistance of sliding which can be mobilized. The presence of such passive wedge resistance, which will be taken into account in this project, leads to increase the weight of the dam section. The effect of cohesion and angle of friction on stability requirements will be considered in this project. Mean values of the cohesion ($c$=200 and $c$=400kN/m$^2$) is used for planes of the foundations with broken contact. The angle of friction used is considered as $\varphi = 45^{o}$.

However, in order to prevent tensile stress at the upstream face and also excessive compressive stress at the downstream face, the dam cross section is usually designed so that the resultant of acting forces falls within the third or half at all elevations of the cross section. The stability of the concrete gravity dam is represented by the safety of the structure against the external forces, for example, the self-weight and water pressure, wind pressure, uplift pressure, silt pressure, earthquake. These forces would make the dam unstable when they are large and causing an overturning, sliding, and tension effects on the dam. Analysis of the stability is generally conducted at the dam base (rock-concrete contact) and at selected planes within the dam. For this type of dam, impervious foundations with high bearing strength are essential.

A stress analysis of gravity dams is performed by two methods: gravity method and FEM using ABAQUS software, to evaluate the magnitude and distribution of stresses throughout the dam for static and dynamic load conditions. The main objectives of using F.E.M in this study are to evaluate the maximum tensile and compressive stresses and to compute the displacements of the system when the dam is subjected to usual, unusual and extreme loading conditions. The seismic response is evaluated for Ali AL-Gharbi earthquake ground acceleration data using finite element acceleration time history method.

### Study of stability

In all cases, the geometry of the concrete gravity dam section is assessed by choosing the optimum cross-section that takes into account all criteria of stability and stress analysis.

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For new projects, the first step in the design procedure for a new structure represents the layout design. This step is followed by stability and stress analysis of the structure that must be made to evaluate the magnitudes of the stability (safety) factors and the stress distributions along the structure.

**Layout design**

The section of concrete gravity dam should be chosen in such away that it is the most economic section and fulfills all the conditions and requirements of stability.

**Elementary profile:** The stability conditions required to investigate for concrete gravity dam, subjected just to its self-weight W, force resulting from water pressure $P_w$, and uplift force $P_u$ can be satisfied by a simple right-angled triangular section as shown in Figure-1, with its peak at the store water level, and which is sufficiently wide b at the base where the water pressure is maximum at the base of the profile; this triangular section called elementary profile section. The value of the water pressure at the base of this elementary profile equals $\gamma_w h_w$, where $h_w$ is the design height of water at the upstream side.

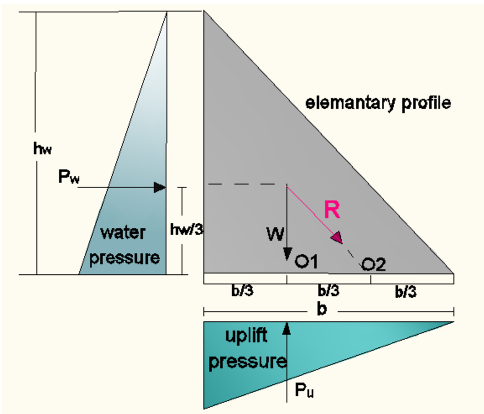


**Figure-1.** Elementary profile of concrete gravity dam.

Considering the main three forces acting on the elementary profile of gravity dam:

weight of the dam: $W = \gamma_c \times vol.$ (1)

$$W = \frac{1}{2} S_c \gamma_w\, b h_w$$

water pressure: $P_w = \frac{1}{2}\gamma_w {h_w}^2$ (2)

uplift pressure: $P_u = \frac{1}{2}\gamma_w b h_w$ (3)

where: $\gamma_w$ = unit weight of water $\gamma_w = 10\text{kN/m}^2$,
$\gamma_c$: unit weight of the concrete,
$S_c$= specific gravity of dam concrete, $S_c = 2.4$.

The base width of the elementary profile is to be found into two criteria; as follows:

**a. Stress criterion:** For reservoir full condition, and for no tension to develop, the resultant *R* must pass through the outer third point (*O2*) shown in Figure-1.

By taking the moment of the three main forces about *O2* and equating it to zero, the resultant, then, is as follows:

$$P_w.\frac{h_w}{3} - W.\frac{b}{3} + P_u.\frac{b}{3} = 0$$

$$b = \frac{h_w}{\sqrt{(S_c - 1)}} \quad (4)$$

**b. Stability or sliding criterion:** For no sliding to occur, horizontal forces, $\Sigma H$ causing sliding should be balanced by the frictional forces $\mu\Sigma V$ opposing the same. Hence:

$$P_w = \mu(W - P_u)$$

$$b = \frac{h_w}{\mu(S_c - 1)} \quad (5)$$

Based on the topography of the region of Iraq, the height of water within 30m is considered suitable height for the design of most concrete gravity dams. Then the width of base will be $b$=28.57m and $b$=25.35m for stress and stability criteria, respectively. It is observable that for satisfying the requirement of stability, the elementary profile of concrete gravity dam should have minimum base width equal to the higher of the base widths obtained from two criteria. Therefore, the suitable base width will be equal to $b$=28.57m.

**Practical profile:** An elementary profile is only theoretical profile which needs to be modified for dependency in actual practice. Modifications that will be taken into account are:

**a. Top width(T.W):** is the crest of the dam dimensioned to provide for a roadway. On the grounds presence of two side on roadway requires that the width of roadway nearly equals to 6.5m.

**b. Freeboard (F.B):** The free board in the dam should be able to avoid overtopping of the dam during maximum flood combined with waves.

For safety requirements, freeboard *F.B* is chosen to be 12% $h_w$ = 0.12×30=3.6m. This freeboard fulfills the three topics illustrated in Figure-2, as: 1m for structural purpose, (including the structural bridge and the parapet); 0.6mas a free board above maximum reservoir level and 2mhead of water above overflow section (spillway), *H*.

The head of water, *H*, above spillway section is taken by using the probable discharge *Q* of 2500m$^3$/s that satisfies records of most Iraqi dams which have similar height of the dam taken in this study. By using equation below for *H*=2m and with coefficient of discharge, $c_w$=2.2, the spillway length *L* along the dam axis, will be equal to 400m.

$$Q = c_w L H^{3/2} \quad (6)$$

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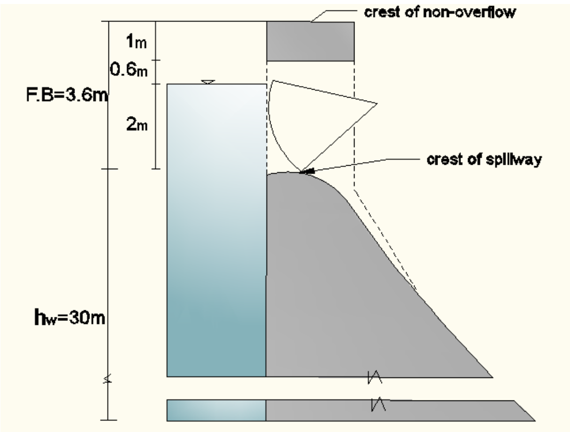


**Figure-2.** Freeboard configuration.

If the length of spillway $L$ suits with the length of the dam, it will be considered in design procedure, if not, the crest of spillway should be lowered to allow the water passing smoothly to downstream face. Hence, the total height of the dam, $\boldsymbol{h_d}$ including freeboard is considered to be:

$$h_d = h_w + F.B = 30 + 3.6 = 33.6\text{m} \quad (7)$$

**c. The upstream face:** the upstream face will usually be vertical. The *downstream face* will usually be a uniform slope starting after the curved portion of the overflow section near the crest. The slope will usually in the range of 0.7H to 1V, to 0.8H to 1V to meet stress and stability requirements at the base [28].

The downstream slope that will be taken in this work can be considered as 1 for vertical and $n$ for horizontal; where $n$ is considered to be equal to:

$$n = \frac{base\ of\ the\ dam\ (b)}{height\ of\ the\ dam\ (h_d)} = \frac{28.57}{33.6} = 0.85 \quad (8)$$

The vertical distance from the downstream edge of the roadway to an intersection with the sloping downstream face will be equal to 7.64m. The final output practical profile for all previous consideration is DAM 1A. The same procedure made on the elementary profile with base $b$=25.35m to obtain DAM 1B. These practical sections (DAM 1A and DAM 1B) are called dams-type 1, see Table-1.

**d. Inclination of concrete-rock contact:** is an important factor providing stability for the structure. Transversely, the foundation contact in practice and for more stability should be either horizontal or sloping upwards toward the downstream face. Longitudinally, the section should vary smoothly to abrupt changes so to minimize stress concentration [11].

The incline angle $\boldsymbol{\alpha}$ is usually used to regulate the $\boldsymbol{\varphi}$ angle in sliding stability spreadsheets that assume a horizontal base; to account for any overall inclination of the rock/concrete interface. However the factor of safety calculated by assuming a horizontal base with a $\boldsymbol{\varphi}$ angle regulated for the geometric inclination failure surface ($\boldsymbol{\alpha}$) will be within +/-5% of the true factor of safety value for the inclined base, as long as the geometric term ($\boldsymbol{\alpha}$) is about 6 degrees [13].

To attain more stability of a concrete gravity dam, and also to obtain the ideal section with less material and least values of factors of safety; the practice shows that geometric term $\boldsymbol{\alpha}$ is always taken as counter clockwise rotation from the horizontal contact surface. Consequently, this improvement will be applied to section DAM 1B to have the new section, DAM 2B. Assuming the rise of the toe by 3m, the resulted geometric inclination $\boldsymbol{\alpha}$ will be equal to 6.75°, with keeping the slope of the downstream face as 0.754(H):1(V). Consequently, the vertical distance from the downstream side of the crest to the point of an intersection with the downstream slope is changed from 8.61m at DAM 1B to 5.61m at DAM 2B. The same process will be performed on DAM 1A with the same angle that produced from DAM 1B, i.e. α = 6.75°, to obtain new section, DAM 2A. DAM 2A and DAM 2B are called dams-type 2.

**e. Passive resistance wedge:** In some circumstance, it may be suitable to include passive wedge resistance $\boldsymbol{P_p}$ at downstream face, as a contribution of sliding resistance. Therefore, a wedge of rock will be considered to be adjacent to dams-type 1 to produce the new sections dams-type 3 (DAM 3A and DAM 3B). To compute the passive resistance force using equation:

$$P_p = W_p.\tan(\varphi_p + \alpha_p) + \frac{c_p.A_p}{\cos\alpha_p .(1 - \tan\varphi_p.\tan\alpha_p)} \quad (9)$$

The parameters in this equation assumed in this study are:

height of wedge $= 3\text{m}$, $\alpha_p$: (angle of the sliding surface for wedge)=30, $\gamma_p = 20\text{kN/m}^3$, then, $W_p = \frac{1}{2} \times 20 \times 3 \times \frac{3}{tan30} \times 1 = 155.88$ kN, $c_p$(cohesion of passive rock wedge) =0.5MPa, $\varphi_p$(angle of friction of passive rock wedge)=30, $A_p$(the area of the sliding surface for wedge) =6×1=6m²; passive resistance become: $P_p = 5466.14\frac{\text{kN}}{\text{m}}$.

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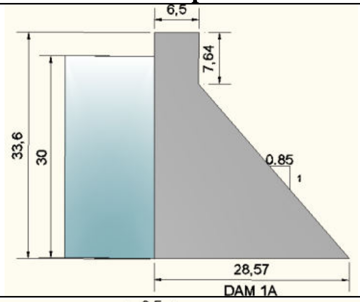
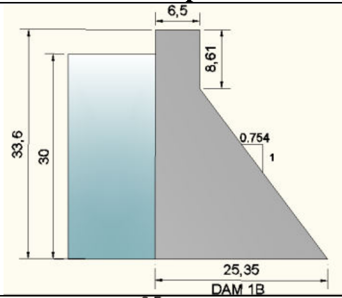
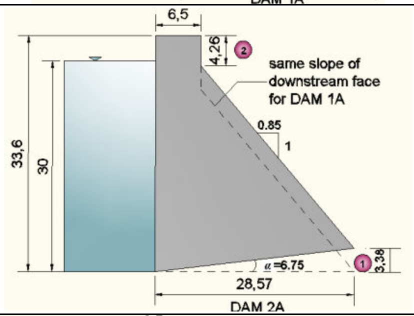
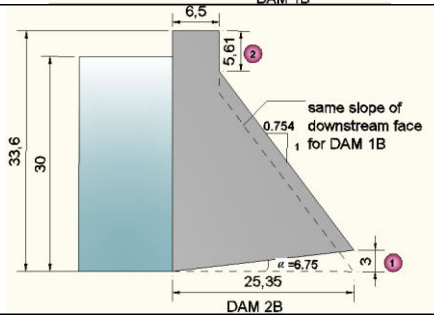
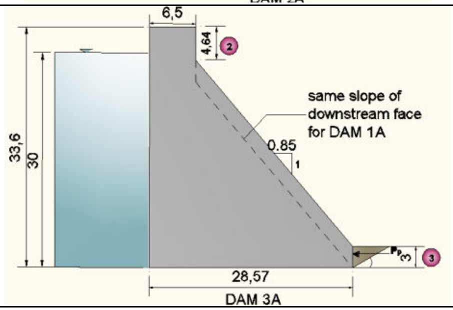
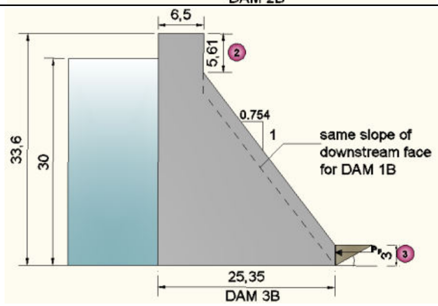
**Table-1.** Cases of study.

| | Group A | Group B |
|---|---|---|
| **Type 1** |   |   |
| **Type 2** |   |   |
| **Type 3** |   |   |

**Forces acting on concrete gravity dam**

In this project, study the stability requirement and stress analysis will be carried out on the practical profile product of DAM 1B ($b$=25.35m). Forces acting on DAM 1B are shown in Figure-3.

i. Weight of the dam: $W = \gamma_c \times vol.$ (10)

$\gamma_c$: unit weight of concrete =24kN/m$^3$, *vol.*: volume of the dam.

ii. External water pressure:

Head water force: $P_w = \frac{1}{2}\gamma_w {h_w}^2$ (11)

Tail water force: $P'_w = \frac{1}{2} \times \gamma_w \times {h'_w}^2$ (12)

$\gamma_w$: unit weight of water =10kN/m$^3$, $h_w$: height of water at upstream face=30m, $h'_w$: height of water at downstream face=3m.

iii. Internal Pressure (Uplift): $P_u = \frac{1}{2}\gamma_w b h_w$ (13)

iv. Silt Pressure: $P_s = \frac{1}{2}\gamma'_s K_A {h_s}^2$ (14)

where: $K_A = \frac{1-\sin\varphi_s}{1+\sin\varphi_s}$, $\varphi_s$: angle of shearing resistance of sediments=33°, $\gamma'_s$: effective unit weight of silt=18kN/m$^3$, $h_s$: height of accumulated silt=20m.

v. Wave force ($P_{wave}$): $P_{wave} = 2\gamma_w {h_{wave}}^2$ (15)

$h_{wave} = 0.032\sqrt{V.F} + 0.763 - 0.271\sqrt[4]{F}$ for $F$< 32km or $h_{wave} = 0.032\sqrt{V.F}$ for $F$ >32km.

$h_{wave}$ =height of waves in meters,between trough and crest,
$F$ =fetch or straight length of water expanse in km,
$V$= wind velocity in km per hour=100km/h.

vi. earthquake force**:**

a) Inertia force : $P_{eh} = W\alpha_h$ (16)

b) Hydrodynamic force: $V_{Pe} = 0.726\, p_e = c_1\alpha_h\gamma_w h_w$ (17)

and the moment of this force: $M_{pe} = 0.299\, p_e y^2$ (18)

Where, $p_e = \alpha_h\gamma_w h_w$,

$$c_1 = \frac{c_m}{2}\left[\frac{y}{h_w}\left(2-\frac{y}{h_w}\right) + \sqrt{\frac{y}{h_w}\left(2-\frac{y}{h_w}\right)}\right], c_m = 0.735\left(\frac{\theta}{90}\right)$$

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$\theta$: Angle in degrees, which the upstream face of the dam makes with the horizontal=90°.

$p_e$ = hydrodynamic earthquake pressure normal to the face,

$c_1$ = a dimensionless pressure coefficient.

$\alpha_h$ = ratio of horizontal acceleration due to earthquake and the gravitational acceleration, *i.e.*, horizontal acceleration factor=0.1.

$y$ = vertical distance from the reservoir surface to the elevation under consideration=30m

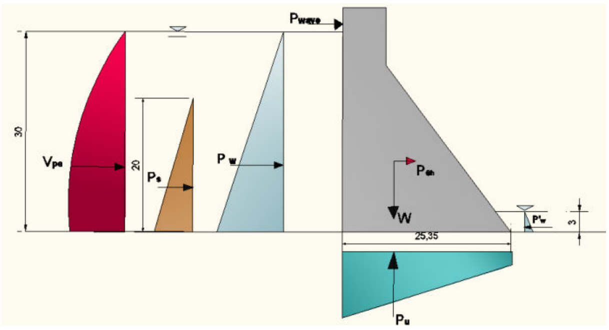


**Figure-3.** Forces acting on DAM 1B.

## Load combination

A concrete dam should be designed with regard to the most rigorous combinations of loads, which have a reasonable probability of simultaneous occurrence. For usual (normal) loads the reservoir is typically taken at the highest normal operating level ($h_w$=30m). For unusual (flood) loads, the reservoir is taken as the maximum (peak) level during the inflow design flood event ($h_w$=33.6m), and can be higher than the crest of the over flow concrete dam. For the extreme (seismic) load the reservoir level is typically taken as the usual water level.

## Stability requirements and stress analysis

In this project, the study of stability criteria is made according to two standard methods, US Bureau of Reclamation, and USBR and US Army corps of engineering, USACE.

## Acceptable safety factors

USBR considered acceptable limits of safety factors against sliding, as shown in Table-2.

**Table-2.** Recommended shear friction safety factors in USBR guidelines.

| Sliding plane | Usual loading condition | Unusual loading condition | Extreme loading condition |
|---|---|---|---|
| Dam concrete/ base interface | 3.0 | 2.0 | 1.0 |
| Foundation | 4.0 | 2.7 | 1.3 |

The maximum allowable compressive stress in the concrete should be not greater than the specified compressive strength divided by 3 for the usual loading combinations. The maximum allowable compressive stress for the unusual loading combinations should be not exceeding specified compressive strength divided by 2. The allowable compressive stress for the extreme condition should be not greater than the specified compressive strength.

USACE considers the acceptable limits of safety factors of stability that shown in Table-3.

**Table-3.** Stability and stress criteria according to USACE.

| Load condition | Resultant location at base | Minimum sliding F.S.S | Concrete stresses | |
|---|---|---|---|---|
| | | | Compressive | Tensile |
| Usual | Middle 1/3 | 2.0 | $0.3f'_c$ | 0 |
| Unusual | Middle 1/2 | 1.7 | $0.5f'_c$ | $0.6f'^{2/3}_c$ |
| Extreme | Within base | 1.3 | $0.9f'_c$ | $1.5f'^{2/3}_c$ |

Note: $f'_c$ is 1-year unconfined compressive strength of concrete.

## Factor of safety against overturning

The factor of safety for overturning $F.O.O$ is not usually tabulated within other stability factors for USBR dams, but may be calculated if required by dividing the total resisting moments by the total moments tending to cause overturning about the downstream toe.

$$F.O.O = \frac{\sum Resisting\ moments\ (\Sigma M_R)}{\sum overturning\ moments\ (\Sigma M_O)} > 1.5 \quad (19)$$

According to USACE, the overturning stability is calculated by applying all vertical forces, $\sum V$ and the lateral forces for each loading condition to the dam, followed by, summing moments $\sum M$ caused by the resulting forces about toe to calculate the resultant location, in other wordthe eccentricity $e$ should be less than $b/6$; If the resultant acting on the dam at any of its sections passes outside middle third of the base of the dam, the dam shall rotate and overturn about the toe.

$$Resaltant\ location\ (X') = \frac{\sum M}{\sum V} \quad (20)$$

when $\sum M = \sum M_R - \sum M_O$

$$e = \frac{b}{2} - X' \quad (21)$$

## Factor of safety against sliding

Sliding along the dam-rock interface is the most common failure mode for concrete gravity dams and study proves that the strength of concrete is key factor in the design of concrete gravity dams. The sliding factor of safety is the ratio of the actual frictional shear stresses to the stresses necessary to achieve equilibrium.

One of the main causes of uncertainty in the analysis of gravity dam stability is the amount of cohesive bond present at the dam/foundation interface. For gravity

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dams, due to available area of contact, amount of cohesive, *c* strength can result in a marked increase in sliding resistance over the resistance offered by friction alone (represented by angle of friction, $\varphi$). In this thesis mean values of the cohesion, ($c$= 200 and 400kN/m$^2$) are used for planes of the rock-concrete contact in dam foundation and $\varphi$=45$^o$.

Three different methods have been developed to assess the safety against plane sliding: sliding resistance method shear friction method andlimit equilibrium method.

USBR uses shear friction method for the sliding stability. The shear-friction is based on the calculation of a safety factor against sliding; this safety factor will be found by dividing the horizontal force available to resist the horizontal loads (sliding resistance, *SR*) by the actual horizontal forces those are causing the displacement. Its general form is as follows:

**a. Without resistive wedge**

$$F.S.S = \frac{SR}{\sum H} \tag{22}$$

$$SR = \frac{c.A}{\cos\alpha.(1-\tan\varphi.\tan\alpha)} + \sum V.\tan(\varphi+\alpha) \tag{23}$$

For horizontal plane α=0, $F.S.S = \frac{c.A+\sum V.\tan\varphi}{\sum H}$ (24)

$\sum H$ : Summation of actual horizontal forces, causing displacement
SR: maximum sliding resistance
ΣV = summation of vertical forces (including reduction from uplift forces)
*A* = area of potential failure plane.
*c* = cohesion.
$\varphi$ = angle of internal friction.
α = angle between inclined sliding plane developed and the horizontal (positive for upwards sliding). (It must be recognized, here, that the angle **α** represents the possible angle of failure irrespective that this plane is concrete-rock contact or any plane tithing the body of the dam).

**b. With resistive wedge**

A passive resistance in equation (9) may be utilized as a contribution for sliding resistance, the assumption for that is to increase stability against sliding. Then the factor of safety against sliding will be:

$$F.S.S = \frac{SR+P_p}{\Sigma H} \tag{25}$$

The limit equilibrium method is the method used by USACE in order to assess the sliding stability as shown in Table 2.3. In addition to this, for usual loading, it is required that the resultant of forces acting on the dam should fall within the middle third of the dam foundation contact area to maintain the compressive stresses in the concrete. For unusual loading conditions, the resultant must remain within the middle half and for extreme loading resultant must fall within the base. [28]

**a. Without resistive wedge**

The limit equilibrium method that used by US Army Corps of engineers suggests that the factor of safety against sliding is given by:

$$F.S.S = \frac{c.A+[\sum V.\cos\alpha+\sum H.\sin\alpha].\tan\varphi}{\sum H.\cos\alpha-\sum V.\sin\alpha} \tag{26}$$

For horizontal plane, equation of *F.S.S* using limit equilibrium method is similar to the equation of shear friction method, i.e. equation (24)

**b. With resistive wedge**

When resisting wedge assumed to be found adjacent with dam (multi wedges) the factor against sliding will be found using equation (27).

Then, $$F.S.S = \frac{\sum_{i=1}^{i=m}\frac{c_i.A_i.\cos\alpha_i+\sum V_i.\tan\varphi_i}{\eta\alpha_i}}{\sum_{i=1}^{i=m}[\sum H_i - \sum V_i.\tan\alpha_i]} \tag{27}$$

where:
*i*= the subscript associated with planar segments along the critical potential failure surface.
*m*= the number of wedges in the failure mechanism or number of planes making up the critical potential failure surface.

The factor $\boldsymbol{\eta_{\alpha_i}}$can be determined with equation below:

$$\eta_{\alpha_i} = \frac{1-\frac{\tan\varphi_i.\tan\alpha_i}{F.S.S}}{1+tan^2\alpha_i} \tag{28}$$

An initial estimate of *F.S.S* is used to obtain $\eta_{\alpha_i}$ is equal to 4.

**Safety against compression (crushing) & tension**

**Gravity method**

Safety against crushing and tension is similar in the way of procedure according in both standard methods discussed above, USBR and USACE. The comparable stress values are so close to each other. Table 2.3 will be used for checking the safety against compression (Crushing) & Tension for both methods.

**a. Safety against compression**

A dam may fail by the failure of its materials, *i.e.*, the compressive stresses produced may exceed the allowable stresses, and the dam material may get crushed. The vertical normal stress distribution at the toe is given by:

$$\sigma_{n_D} = \frac{\Sigma V}{b}\left(1+\frac{6e}{b}\right) \tag{29}$$

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The reference compressive strength in this study is taken as 25MPa for comparing the resulting stress in the structure.

**b. Safety against tension**

The normal stress at the heel is: $\sigma_{n_U} = \frac{\Sigma V}{b}\left(1 - \frac{6e}{b}\right)$ (30)

It is evident that if e > b/6, the normal stress of the heel will be tensile. No tension should be allowable at any point of the dam under any condition. For no tension to develop, the eccentricity must be less than b/6. In other words, the resultant should always lie within the middle third, Figure-4.

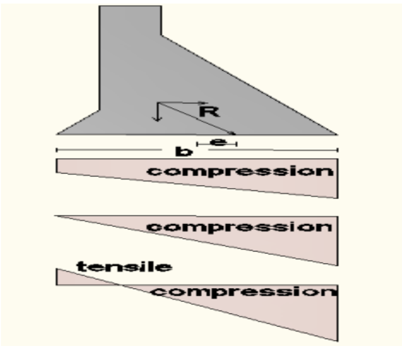


**Figure-4.** Normal stresses on the base of concrete gravity dam.

**Finite Element modeling**

The Finite Element Method (FEM) is a key technology in the modeling of advanced engineering systems. It's a numerical and an approximation method for determining responses (stress, strain, deformation, etc.) of a body under external loads. Its results will depend upon element type, mesh size, and mesh configuration.

A three-dimensional problem can be rearranged (simplified) if it can be treated as a two dimensional (2D) solid. The dam was considered as a 2D solid, where one coordinate (z-axis) was ignored. According to the geometry of the dam, the nature of loading on the dam makes the dam problem as plane strain problem; therefore, it is analyzed as plane strain problem using ABAQUS software. The finite element meshes used in the analysis of the DAM 1B section consist of 646 nodes and 592 elements, first order, reduced-integration plane strain elements (CPE4R), Figure-5.

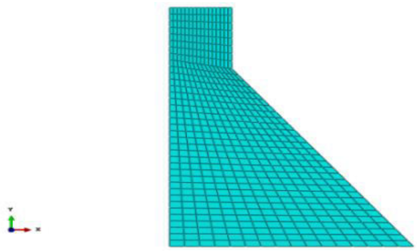

**Figure-5.** Finite element mesh of DAM 1B.

DAM 1B is 33.6m high and 25.35m wide at the base of the solid section. The upstream wall is straight and vertical, and the downstream face with slope of 0.754H: 1V. The depth of the water at the upstream of the dam was 30 meters for usual condition and extreme condition (when Ali AL-Gharbi earthquake applied), 33.6m at flood condition (unusual condition). For the purpose of this study and to make agreement with the practice in dam construction which requires that dams must be founded on very strong sound bed-rock, i.e. the foundation is rigid. The materials of DAM 1B section are assumed to be homogeneous, isotropic and linear elastic material. According to [ACI 207.1R-96, for mass concrete], the tensile strength was estimated to be $f_t = 0.32{f'_c}^{2/3} = 2.736$MPa [3]. When $f'_c$ is compressive strength of concrete and it was assumed as 25MPa [18] in this project, Table-4.

**Table-4.** Concrete properties of all assumed dams.

| Property | Concrete | Unit |
|---|---|---|
| Density | 2400 | Kg/m$^3$ |
| Elastic modulus | 30000 | MPa |
| Poisson's ratio | 0.18 | - |
| Allowable Compression strength | 25 | MPa |
| Allowable tensile strength | 2.736 | MPa |

The dam was subject to different loads which include: gravity load due to self-weight of the dam, hydrostatic pressure, silt pressure, uplift pressure, seismic load and hydrodynamic pressure. In this project, finite element analysis by using ABAQUS program, was carried out to the same dam section used in two-dimensional gravity method, DAM 1B, and for three loading combinations, usual, unusual, and extreme; to investigate the stresses and deformations under the expected design loads. For dynamic loading condition, the transverse ground accelerations of Ali AL-Gharbi, Figure-6, are applied to all nodes at the base of the dam.

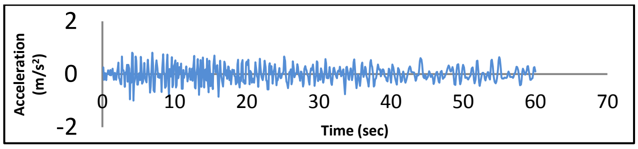


**Figure-6.** Acceleration - time records of earthquake hit Ali Al-Gharbi.

## RESULTS AND DISCUSSIONS

The same calculations of DAM 1B will be performed on all virtual sections shown in Table.1 to study the factors that affecting on stability requirements and stress analysis according to both USBR and USACE standards.

Carrying out the stability analysis against overturning for various loading combinations, DAM 1B possesses the following values of safety factors:

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**Results of overturning factors**

Table-5 shows the factors of safety against overturning according to USBR for three different loading conditions.

**Table-5.** Factors of safety against overturning of DAM 1B according to USBR.

| Loading condition | F.O.O (Obtained) | Standard |
|---|---|---|
| Usual | 1.58 | >1.5 |
| Unusual | 1.297 | >1.5 |
| Extreme | 1.36 | >1.5 |

According to USACE, Table 6 shows the values of eccentricity for three loading conditions.

**Table-6.** Values of eccentricity of DAM 1B according to USACE.

| Loading condition | Eccentricity | Standard |
|---|---|---|
| Usual | e = 2.39 | e < 4.225 |
| Unusual | e = 5.82 | e < 4.225 |
| Extreme | e = 5.288 | e < 4.225 |

For both standard, USBR and USACE, DAM 1B is accepted for overturning safety for usual loading combination and fails for unusual and extreme loading combination. According to USBR, in order to achieve safety against overturning for DAM 1B for unusual and extreme loading conditions, the level of water should be dropped to suitable elevation, which achieves a safety factor of overturning equal to 1.5 ($\frac{\Sigma M_R}{\Sigma M_o} = 1.5$). Therefore, the water height should be at level the 30.9minstead of 33.6m for unusual loading condition, and 28.1minstead of 30m for extreme loading condition.

For USACE, like USBR, DAM 1B fails in unusual and extreme loading conditions. To avoid this type of failure, the height of water must satisfy the rule that the resultant of all forces shall intersects the base of the dam within the middle third, must be calculated. In other ward, this height of water must achieve that *e* should be less or equal *b/*6 which is equal to 4.225m. The maximum height of water for unusual loading combination is 32.05m, and 28.8m for extreme loading combination. This meansthat USACE allows water levels higher than USBR method.

**Results of sliding factors**

Because of the base of DAM 1B is horizontal, the same results of sliding factor appear for both standard, USBR and USACE, as shown in Table-7.

**Table-7.** Factors of safety against sliding of DAM 1B according to USBR & USACE.

| Loading condition | Parameters | Sliding factor (Obtained) | | Standard | |
|---|---|---|---|---|---|
| | | USBR | USACE | USBR | USACE |
| Usual | $c = 200$ & $\varphi = 45$ | 2.35 | 2.35 | >3 | >2 |
| Unusual | | 1.84 | 1.84 | >2 | >1.7 |
| Extreme | | 1.79 | 1.79 | >1 | >1.3 |
| Usual | $c = 400$ & $\varphi = 45$ | 3.37 | 3.37 | >3 | >2 |
| Unusual | | 2.67 | 2.67 | >2 | >1.7 |
| Extreme | | 2.56 | 2.56 | >1 | >1.3 |

This Table, again, yields the notice, that according to USBR, DAM 1B fails in sliding for usual and unusual loading conditions when bond of the concrete-rock contact is moderately weak ($c$ =200kN/m$^2$). So as to avoid the sliding, the cohesion must be increased for no less than 328kN/m$^2$ (then $f'c$ will be about 6.56MPa where $c$=0. 05$f'c$ [13]) to achieve *F.S.S* equal to 3 for usual loading condition; and 239kN/m$^2$ ($f'c$=4.78MPa) for unusual loading condition. However, in USACE, DAM 1B achieve the requirements of overturning safety for all loading conditions.

**Results of compression and tension**

Table-8 illustrates the normal stresses on heel and toe for DAM 1B; the results show that all stresses remain safe limits for all loading combinations.

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**Table-8.** Normal stresses on DAM 1B.

| **Loading condition** | **Normal stresses** | **Obtained** (kN/m$^2$) | **Standard** (kN/m$^2$) |
|---|---|---|---|
| Usual | At heel ($\sigma_{nU}$) | 414.4 | <7500 |
| Unusual | | 586.5 | <12500 |
| Extreme | | 595.95 | <22500 |
| Usual | At toe ($\sigma_{nU}$) | 114.96 (Compression) | 0 |
| Unusual | | -93.1 (Tensile) | <5130 (Tensile) |
| Extreme | | -66.55 (Tensile) | <12824.8(Tensile) |

**Results of parameters affecting on stability factors**

a) Slope of the base of the dam (dams-type 2), Figure-7 shows the effect of the slop of the base of the dam on stability factors for three loading condition. The results show that the upward inclination (counter clockwise rotation) of the line of the base around an axis passing through the heel, DAM 2B give more stability from the normal case of horizontal base, DAM 1B for two standard, USBR and USACE.

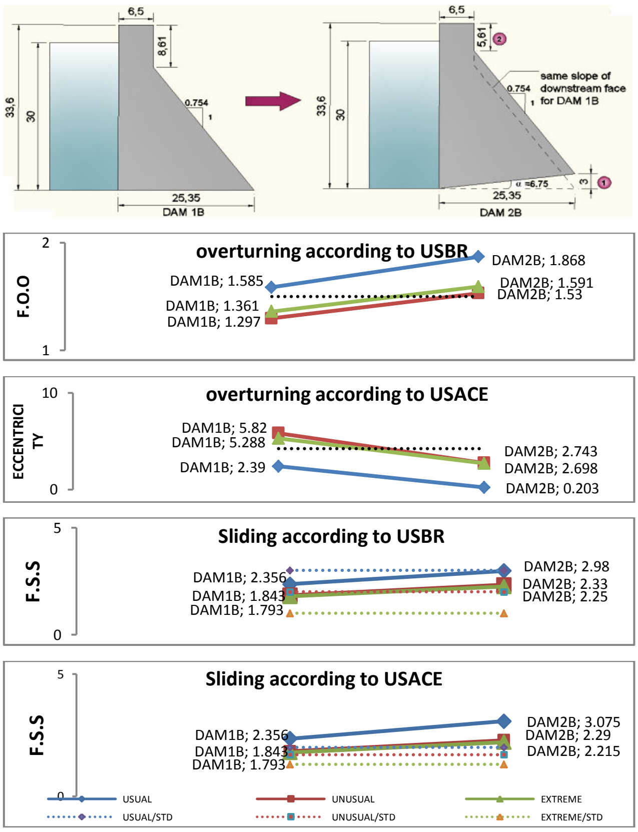


**Figure-7.** Affect of inclined the base of DAM 1B on safety factors according USBR and USACE.

b) The effect of the passive wedge on the values of *F.S.S* is that this wedge will increase the *F.S.S* more rapidly from the case of the dams-type 1 as compared with the section dams- type 3by 54%, 56%, and 50.5% for USBR for the load combinations usual, unusual, and extreme, respectively; and about 44.5%, 46% and 40% for USACE standards. Figure-8 describes this effect when usual loading condition are applied with $c$ = 200kN/m$^2$ and $\varphi$=45.

c) The effect of the cohesion on the values of *F.S.S* is that the higher value of cohesion will increase the *F.S.S* more rapidly from the case with low values, for example at DAM 1A; for USBR standards the increments of *F.S.S* were about 43.5%, 45.5%, and 43% for the load combinations usual, unusual, and extreme, respectively, when increasing the cohesion from 200kN/m$^2$ to 400kN/m$^2$, Figure-9. Approximately, the same ratios were obtained for USACE standards, Figure-10.

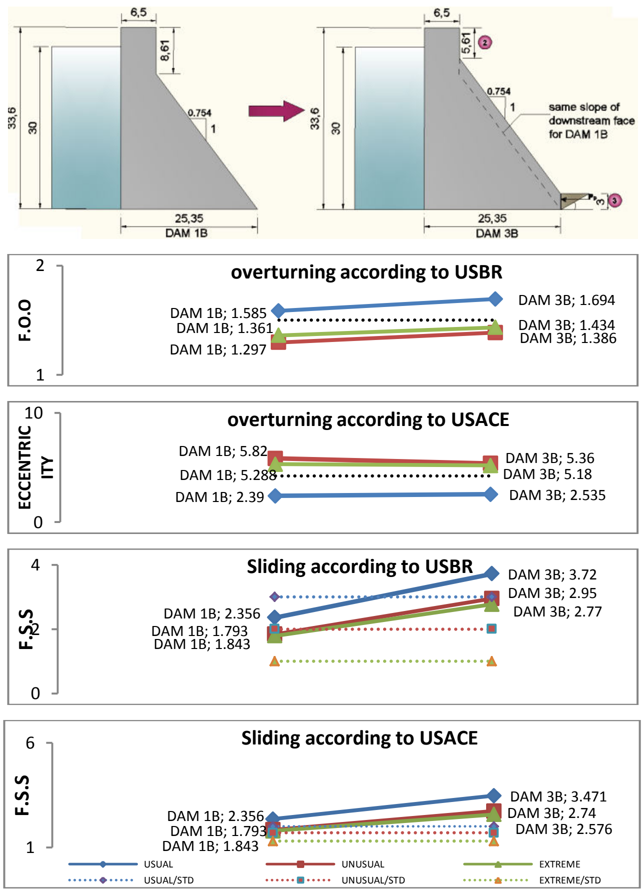


**Figure-8.** Effect of the presence of passive wedge adjacent to DAM 1B on safety factors according USBR and USACE.

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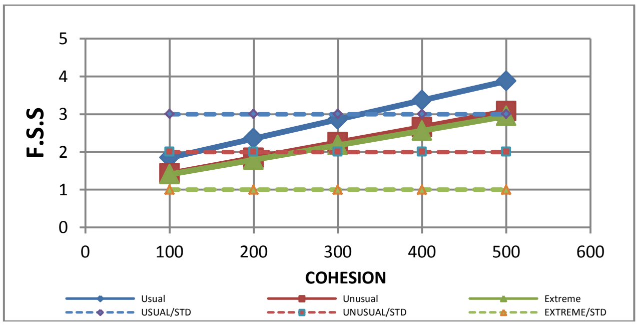


**Figure-9.** Effect of cohesion on F.S.S of DAM 1B USBR.

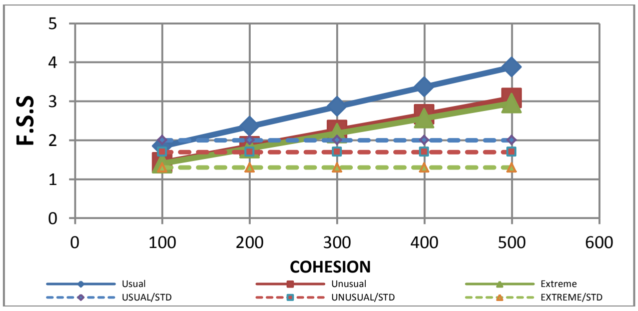


**Figure-10.** Effect of cohesion on F.S.S of DAM 1B USACE.

### Results from ABAQUS software

The aim of the FEM is to determine the responses of the structure concentrating on the maximum tension and compression stresses and the displacements, based on the characteristic of the structure and the nature of the earthquake. The Figures 11 to 16 below show the result of stress analysis for DAM 1B for three loading conditions.

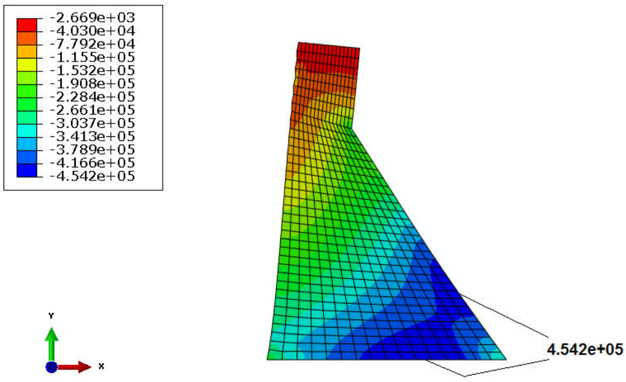


**Figure-11.** Maximum compression stresses in DAM 1B for usual loading condition.

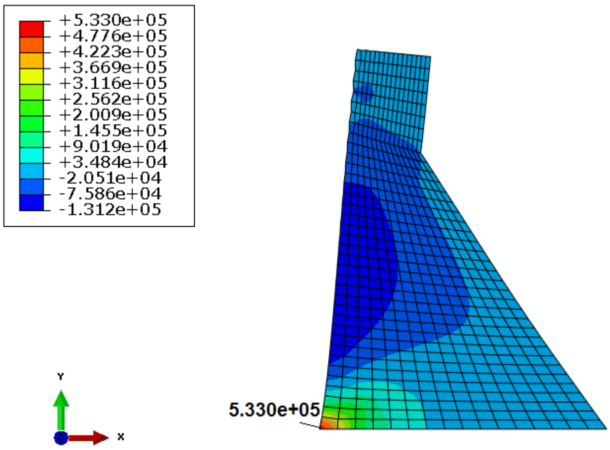


**Figure-12.** Maximum tensile stresses in DAM 1B for usual loading condition

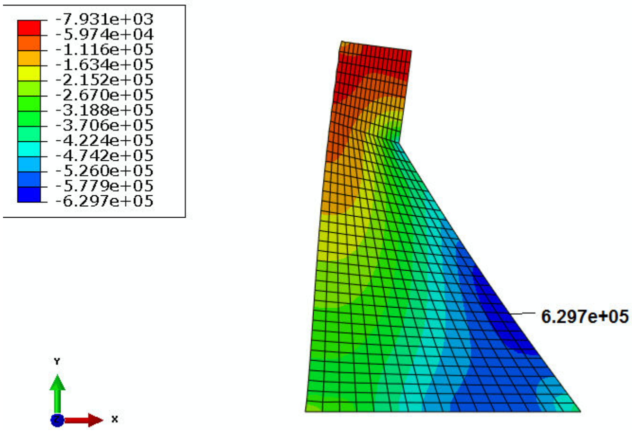


**Figure-13.** Maximum compression stresses in DAM 1B for unusual loading condition.

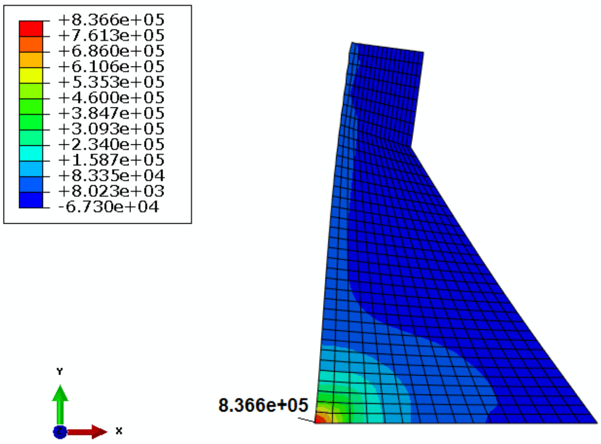


**Figure-14.** Maximum tensile stresses in DAM 1B for unusual loading condition.

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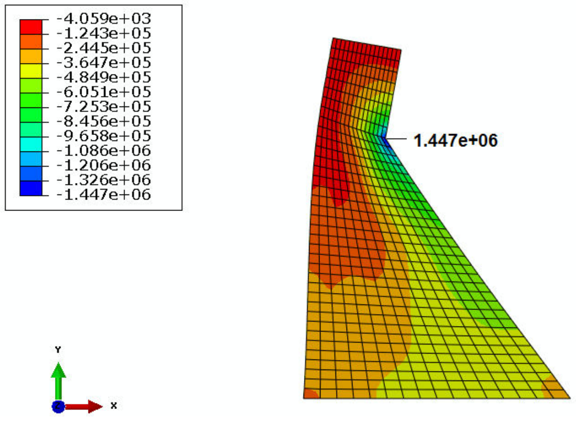


**Figure-15.** Maximum compression stresses in DAM 1B for extreme loading condition.

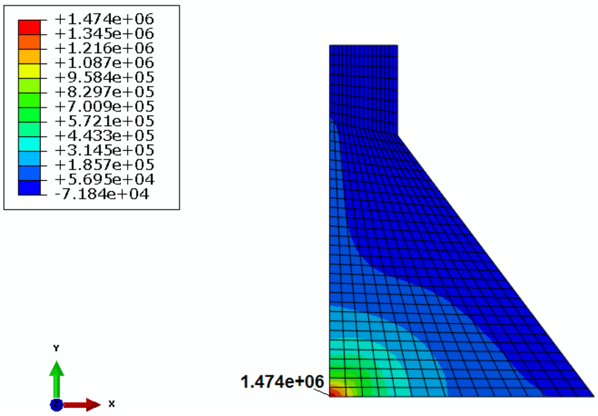


**Figure-16.** Maximum tensile stresses in DAM 1B for extreme loading condition.

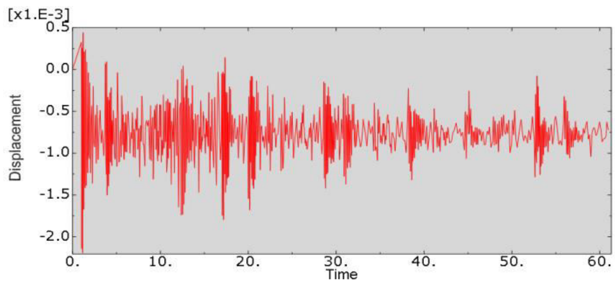


**Figure-17.** Horizontal crest displacement of DAM 1B related to ground displacement of extreme condition.

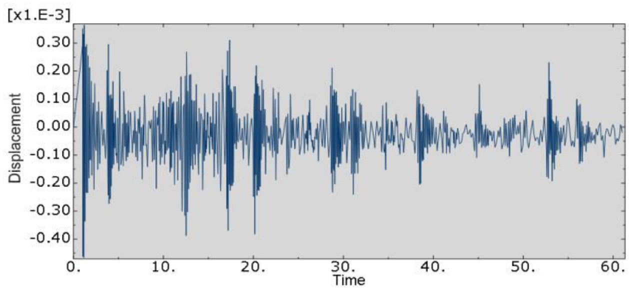


**Figure-18.** Vertical crest displacement of DAM 1B related to ground displacement of extreme condition.

For three loading condition, usual, unusual, extreme; the maximum compression stress for both DAM 1B does not exceed the allowable compressive strength of the selected concrete which is 25MPa.

The highest value of the tensile stress for DAM 1B was occurred at the heel of the dam; this value is acceptable, since it is less than 2.736MPa that given in ($f_t = 0.32{f'_c}^{2/3} = 0.32 \times 25^{2/3} = 2.736$ MPa). The positive values represent the tensile stresses, while the negative values represent the compressive stresses. According to the extreme loading integrated displacement results, the maximum horizontal displacement of crest related to ground displacement towards the downstream was about 2.25mm, and maximum vertical displacement was about 0.45mm.

## CONCLUSIONS

Based on the results of this study, the following conclusions have been drawn:

- It is observable that for satisfying the requirements of stability, the elementary profile of concrete gravity dam should have minimum base width equal to the higher of the base widths obtained from two criteria; stress criteria and stability criteria. Therefore, the base width for the reference study case in this project was equal to ***b*** =28.57m, which obtained from stability criteria. To make the perfect choice of section that achieves all the requirements of stability with lowest cost and less material, the section produced from stress criteria, is taken into account in this project with***b***= 25.35m. In both cases the normal height of water just prior to the dam profile is taken as 30m.

- For horizontal plane without passive resistance, *F.S.S* in shear friction method,by using USBR standards, is similar to ***F.S.S*** in limit equilibrium method by using **USACE** standards.

- The main conclusion offeredby this study is that when evaluating the stability against overturning, the USACE calculations for eccentricity, - in which the resultant of all forces shall intersect the base of the dam within the middle third, or($e<b/6$) - those calculations allow water elevations higher than those of USBR calculations for *F.O.O*. As a result and to avoid the phenomena of overturning during the operation of the dam in unexpected (unusual and extreme) loading conditions; the height of water was to be slightly lowered from the levels at 33.6m and 30.0m, respectively; to achieve the *F.O.O* of 1.5 for USBR.

- For USACE calculations the value of cohesion, ***c*** =200kN/m$^2$ is found sufficient to achieve sliding resistance for all groups and types in various loading combinations: usual, unusual, and extreme.

- According to USBR standards, the cohesion at the concrete-rock contact must be raised to a suitable value to achieve the value of *F.S.S* within acceptable limits. This value of cohesion is related directly to the compressive strength of concrete. The required magnitude of cohesion to achieve sliding stability in usual loading combination is more than that in unusual loading, while there is no such failure noticed for extreme combination. Examples are: $c$ = 328 and 239kN/m$^2$ for usual and unusual loading conditions, respectively, for DAM 1B; and $c$ =204kN/m$^2$ for DAM 2B in usual loading combination.

- The presence of passive resistance wedge at the downstream face increases sliding resistance with adequate ratio. This fact is true when comparing Dams-type 1 with Dams-type 3. The results show that for USBR standards the existence of passive wedge increases the sliding factor by about 54%, 56% and 50.5% for usual, unusual, and extreme loading conditions, respectively; and about 44.5% , 46% and 40% for usual, unusual and extreme loading conditions for USACE standards.

- The results indicate that the stability against overturning and sliding in usual loading combination was more than those of the unusual and extreme loading combinations for both standards USBR and USACE.

- The main objectives of using F.E.M in this study are to evaluate the maximum tension and compression stresses and to compute the displacements of the system when the dam is subjected to usual, unusual and extreme loading conditions. The stresses obtained in Dams-type 2 are less than the stresses obtained in Dams-type 1 with various loading combinations. Stresses were found acceptable in all profiles, where it is important to prevent undesirable tensile stresses at the heel, and to avoid crushing at the toe.

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