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STABILITY EVALUATION OF SMALL CONCRETE GRAVITY DAMS

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Abstract: The aim of this work is to ensure a structural stability of small concrete gravity dams by selecting the economic practical section that reduces the material's costs with the least acceptable factors of safety. The main parameters associated with the geometric shape; or the slope of the base of the dam, material properties (cohesion and angle of friction), the presence of passive wedge, as well as the conditions of loading with normal and maximum water heights of 30m and 33.6m, respectively, in addition to extreme condition with earthquake forces; will be the parameters presented to evaluate the structural stability for concrete gravity dams. The study of stability criteria was done on twelve virtual dam cases according to two standard methods U.S Bureau of Reclamation (USBR) and U.S Army Corps of Engineers (USACE), to obtain the height of water for safe operation and the strength of concrete required to avoid overturning and sliding of the dam. The behavior of the dam has been modeled and analyzed using analytically 2-dimensional gravity method and FEM by using 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.The profile DAM 2B with a base inclined by 6.75^o upwards toward downstream face, and width b= 25.35m was found the most optimum section for a dam required to store a volume with a height of water, h_w =30m. In this dam the value of cohesion of approximately c =200kN/m^2 was found sufficient to achieve the sliding stability for all loading combinations.

Key words: *stability, stress analysis, small concrete, gravity dam.*

تقييم استقرارية السدود الخرسانية الثقالية الصغيرة

الخلاصة: هدف البحث هو ضمان استقرارية السدود الخرسانية الثقالية باختيار المقطع الاقتصادي الذي يحقق حدا أدنى من المواد و بأقل قيم مقبولة لمعاملات الأمان . إن ميلان قاعدة السد وهو أهم عامل في الشكل الهندسي و خواص المواد : التماسك و زاوية الاحتكاك علاوة على تضمين مقاومة الوتد بالاضافة الى ظروف التحميل المختلفة الناتجة من ارتفاع المياه المتوسط 30م والأقصى 33.6م وظروف تحميل قصوى بوجود قوى الزلازل، تمثل العوامل المهمة التي تم اخذها بنظر الاعتبار في هذه الدراسة. تمت دراسة الاستقرارية لإثني عشر سدا افتراضيا اعتمادا على المعايير المعتمدة من قبل : مكتب الإستصلاح الأمريكي USBR و فيلق المهندسين الأمريكي USACE وذلك للحصول على ارتفاعات المياه المناسبة للتشغيل و قيمة مقاومة انضغاط الخرسانة المطلوبة للإنشاء وذلك لتجنب حصول إنقلاب وانزلاق السد. تمت دراسة الاجهادات بطريقتين: طريقة الجاذبية وطريقة العناصر المحددة باستخدام برنامج ABAQUS لضمان سلامة السد حيث كانت جميع الاجهادات أمينة ومقبولة من حيث الشد و الإنضغاط أيضا و لجميع السدود المدروسة. المقطع الأمثل لسد يحتجز المياه بارتفاع

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30 م و الذي تم استنتاجه من هذه الدراسة هو DAM 2B بعرض قاعدة 25.35 م وميلان بزاوية 6.75 درجة صعودا باتجاه المصب وبقيمة تماسك بحدود 200 كيلونيوتن/م^2.

1. Introduction

The stability of small concrete gravity dams against sliding and overturning, the stress distribution in the dam profile and the displacements caused by the stresses generated are the main objectives of this study. In this research, the relations associated with the selection of the economic practical profile of a small concrete gravity dam, and material properties as well as the types of loading and their usual, unusual and extreme combinations will be presented. Two standards: the U.S. Bureau of Reclamation, USBR, and the U.S. Army Corps of Engineers, USACE, are used to evaluate the structural stability and height of water required for safe operation of concrete gravity dams to avoid overturning and sliding of the dam.

Twelve (12) virtual sections of small concrete gravity dam, with a height limitation of 30m adopted for the classifications by the USBR standards, are processed to examine the structural stability and stress distribution of the dam. In this work, the influence of four (4) main parameters on the safety factors against overturning and sliding in addition to their effect on stress distribution, are performed:

a) The base width of the dam profile.
b) The inclination of the base of the dam profile.
c) The presence of passive resistance wedge at downstream face.
d) The cohesion and the angle of friction.

The stability of the concrete gravity dam is represented by the safety of the structure against the external forces, for example, the water weight and water pressure, wind pressure, uplift pressure, silt pressure, earthquake [13]. 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.

Dam stresses and displacements analyzed by using 2D-gravity and finite element methods with the aid of the ABAQUS software, regarding static and dynamic loads are found acceptable in all profiles, where it is important to prevent tensile stresses at the heel (to be less than 2.74MPa) and to avoid crushing at the toe (not more than 25MPa).

2. Optimum Section of Small Concrete Gravity Dam

In all cases the geometry of the concrete gravity dam section is assessed by choosing the optimum cross-section that taken into account all criteria of stability and stress analysis[13]. If the analytical results of selected section fail to meet the allowable limits or the stress distributions are not reasonable because of stress concentrations, modifications to satisfy the design must be made by reshaping and reanalyzing the structure. The design of a gravity dam is achieved by making successive layouts, each one being gradually developed based on the results of a stress analysis. It is difficult to examine layouts without discussing analysis and vice versa, because each operation is essential to other.

2.1. *Elementary Profile*

In this study, using limitation of low (small) dam according to U.S. Bureau of Reclamation (USBR) that the height is within 30m is used for the classification of small dam [2]. Thus, in this work, the maximum reservoir water height is assumed to be h_w = 30m. The unit weight of water and concrete are assumed to be γ_w=10 kN/m^{3}and γ_c= 24 kN/m^3, respectively.

In the absence of any force other than the forces due to water, the elementary profile will be triangular in section, with zero width at the surface water level, where water pressure is zero, and having maximum base width *b*, where the maximum water pressure acts at the base of the profile. The following procedure illustrates the way of determination of the main forces acting on elementary profile, Figure.1, these forces are:

1. Weight of the dam: $W = \frac{1}{2}\, b h_w . S_c . \gamma_w$

$$W = \frac{1}{2}\, b \times 30 \times 2.4 \times 10 = 360b\ \frac{\text{kN}}{\text{m}} \qquad \dots (1)$$

2. Water pressure: $P_w = \frac{1}{2}\gamma_w {h_w}^2$

$$P_w = \frac{1}{2} \times 10 \times 30^2 = 4500\ \frac{\text{kN}}{\text{m}} \qquad \dots (2)$$

3. Uplift pressure: $P_u = \frac{1}{2}\gamma_w b h_w$

$$P_u = \frac{1}{2} \times 10 \times b \times 30 = 150\ b\ \frac{\text{kN}}{\text{m}} \qquad \dots (3)$$

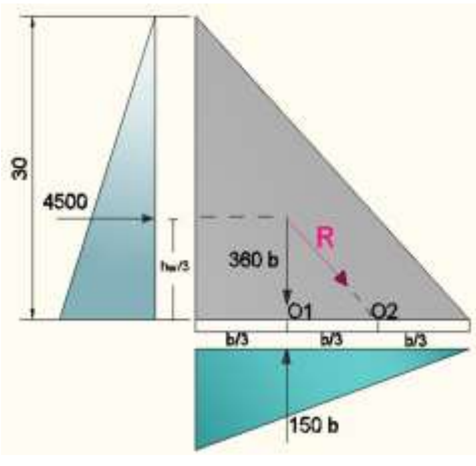


Figure1 : forces acting on elementary profile by assuming h_w=30m

Where: S_c: Specific gravity of concrete (S_c = 2.4), γ_w: Unit weight of water (γ_w= 10 kN/m^3), and h_w: Maximum reservoir water surface assuming to be 30m above the base of the dam. The two criteria that can enable computing the base width of the elementary profile are [1]:

a. *Stress Criterion*

This method assumes that there is no tension developed along the base of the dam at reservoir full condition, thus, the resultant *R* passes through the outer third point (*O2*) shown in Figure 1. Equation (4) can be used to evaluate the base width of the elementary profile by this criterion[1]: is:

$$b = \frac{h_w}{\sqrt{(S_c - 1)}},\ \text{then } b = \frac{30}{\sqrt{(2.4-1)}} = 25.35\text{m} \qquad \ldots(4)$$

b. *Stability Criterion*

For no sliding to occur, horizontal forces ΣH causing sliding should be balanced by the frictional forces $\mu\Sigma V$; where μ is the coefficient of friction, at normal cases $\mu = 0.75$; hence, equation (5) can be used to calculate the base width of elementary profile [1]:

$$b = \frac{h_w}{\mu(S_c - 1)}\ ,\ \text{then } b = \frac{30}{0.75(2.4-1)} = 28.57\text{m} \qquad \ldots(5)$$

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 [1]. Therefore, the base width will be equal to *b* = 28.57m.

2.2. *Practical Profile*

An elementary profile is only theoretical profile which needs to be modified for dependency in actual practice. Modifications should take account of providing of a limited top width, suitable freeboard, configuration of downstream slope; the slope of concrete-rock contact, providing a batter in the lower part of the upstream face.

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:

1. 1.0m for structural purpose, (including the structural bridge and the parapet)
2. 0.6m as a free board above maximum reservoir level, and
3. 2m head of water above overflow section (spillway), *H*.

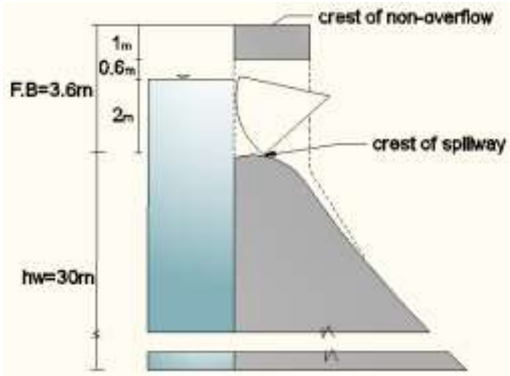


Figure 2: Freeboard configuration

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.

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

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, h_d including freeboard is considered to be:

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

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 [7].
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:

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

The vertical distance from the downstream edge of the roadway to an intersection with the sloping downstream face will be equal to 7.64m. Figure 3 shows the final output practical profile for all previous consideration, the reference, DAM 1A (shown in Table 1).

DAM 1A, is which produced from the stability criterion that is used to compute the largest base of the elementary profile (28.57m). However, to make the correct choice of the section that achieves the economic section of the dam and reduces the materials' costs with satisfying the least acceptable factors of safety; a section which is created

*Eq.6 is derived from the weir equation $Q = \frac{2}{3}\, C_d . \sqrt{2g} L H^{3/2}$ with $c_w = \frac{2}{3}\, C_d . \sqrt{2g}$ and $C_d \cong 0.75$.

from the second criterion (stress criterion) of design the elementary profile, DAM 1B, with base width of (25.35m) is taken into consideration for stability analysis. The section DAM 1C is the one with the average base width of the bases of DAM 1A and DAM 1B; of (27m) is also presented. Consequently, three practical sections were obtained, DAM 1A, DAM 1B, and DAM 1C being dams-type 1 for groups A, B, and C, respectively, as shown in Table 1 and Figure 4.

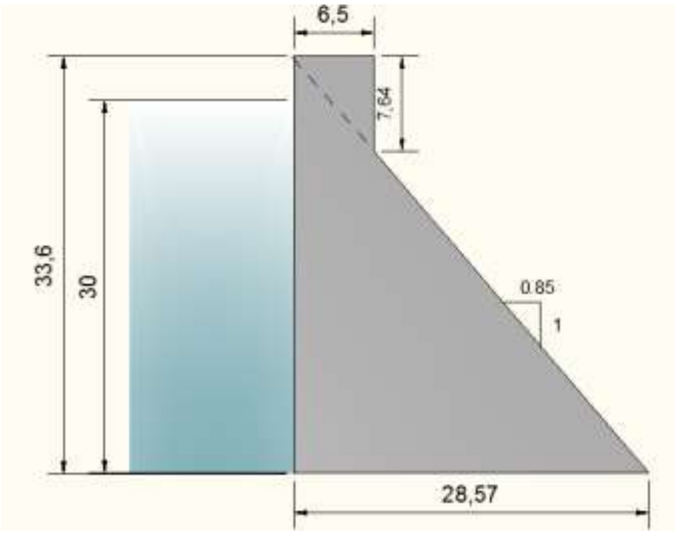


Figure 3: Practical profile at horizontal base = 28.57m, (Reference, DAM 1A)

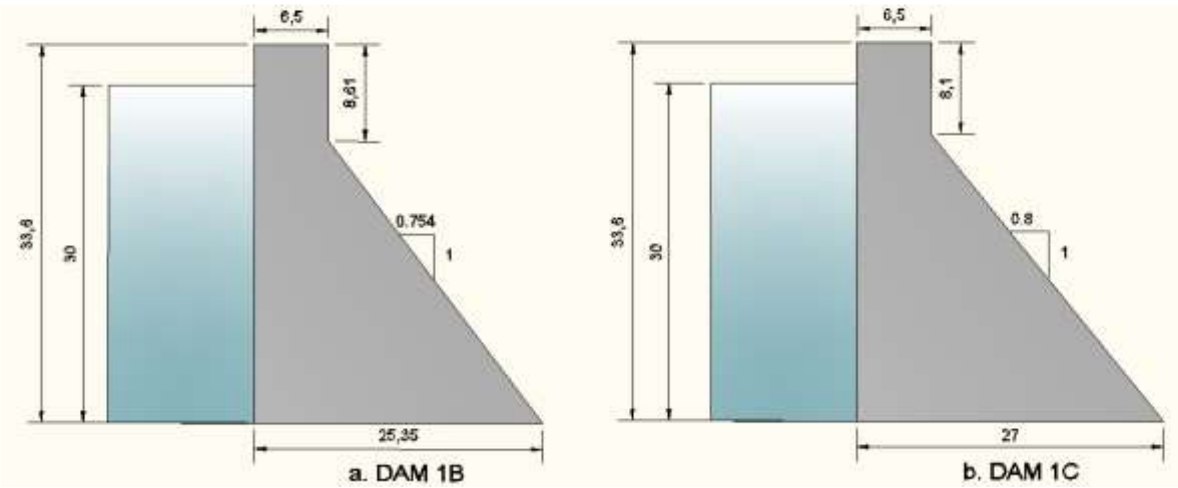


Figure 4: Practical profile of :a. DAM 1B and b.DAM 1C

d. *The 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 [6].

The incline angle α is usually used to regulate the φ 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 φ angle regulated for the geometric inclination failure surface (α) will be within +/-5% of the true factor of safety value for the inclined base, as long as the geometric term (α) is about 6 degrees, [14].

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 α is always taken as counterclockwise 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 α will be equal to 6.75^{o} (the first step of changing DAM 1B), 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 second step of changing section DAM 1B).

The same process will be performed on DAM 1A and DAM 1C with the same angle that produced from DAM 1B, i.e. $\alpha = 6.75^{o}$, to obtain the sections will be produced, DAM 2A and DAM 2C, respectively.

According to these changes, new sub-sections were produced, DAM 2A, DAM 2B and DAM 2C; they called dams-type 2 as shown in Table 1.

e. *Passive resistance wedge*

The presence of passive resistance wedge at downstream face increases sliding resistance [4]. Therefore, a wedge of rock will be considered to be adjacent to dams-type 1 to produce the new sections dams-type 3, (the third step of changing), Table 1. To compute the passive resistance force using equation:

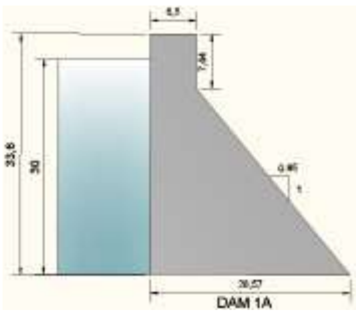
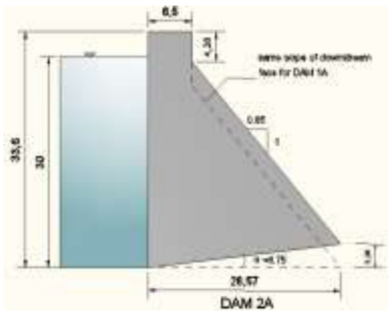
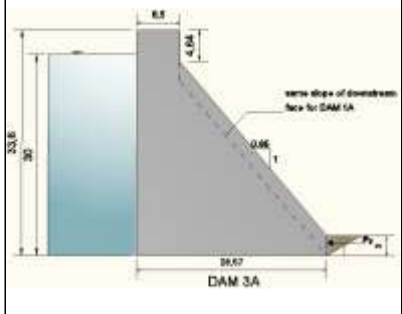
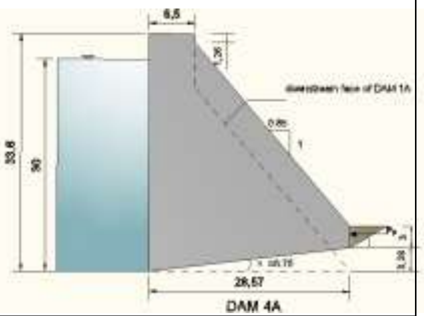
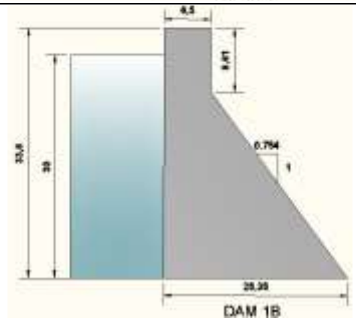
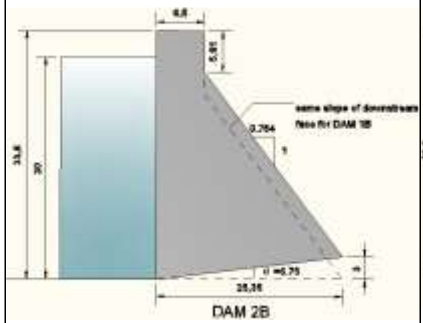
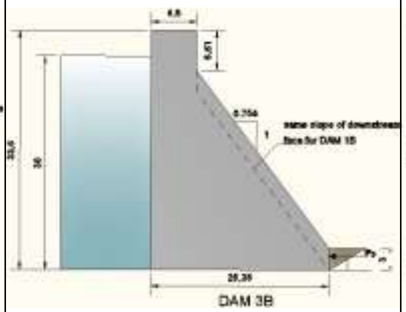
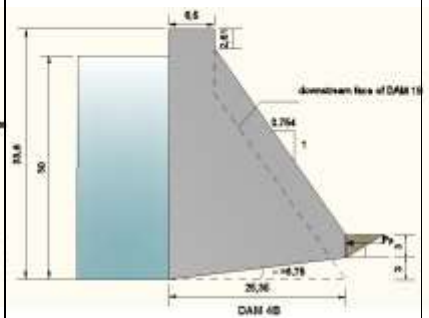
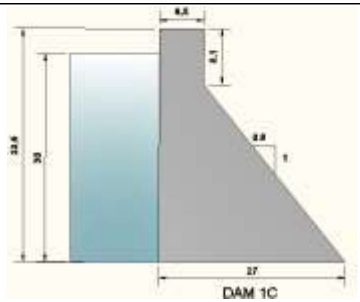
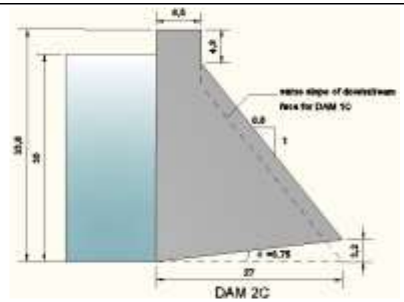
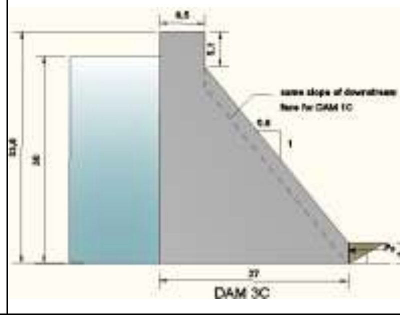
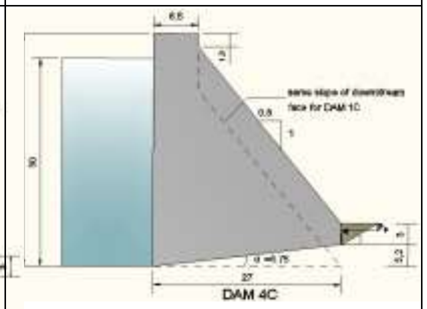
$$P_p = W_p.\tan(\varphi_p + \alpha_p) + \frac{c_p.A_p}{\cos\alpha_p\,.(1-\tan\varphi_p.\tan\alpha_p)} \qquad \ldots (9)$$

The parameters in this equation assumed in this study are: $\text{height of wedge} = 3\text{m}$, α_p: (angle of the sliding surface for wedge)=30,$\gamma_p = 20\text{kN/m}^3$, then, $W_p = \frac{1}{2} \times 20 \times 3 \times \frac{3}{tan30} \times 1 = 155.88\text{ kN}$, c_p(cohesion of passive rock wedge) =0.5MPa, φ_p (angle of friction of passive rock wedge)=30, A_p(the area of the sliding surface for wedge) =6×1=6m^2; passive resistance become:$P_p = 5466.14\,\frac{\text{kN}}{\text{m}}$.

Dams-type 4 was produced by the combination of passive resistance wedge with upward inclination of the line of the base, Table 1.

Table 1 shows all modifications and configurations obtained on dams- type 1 for all groups A, B and C.

Table 5.1 : Cases of study considered in this research work

Group / Type	(1) Horizontal base, without passive wedge	(2) Base upward with α=6.75, without passive wedge	(3) Horizontal base , with passive wedge	(4) Base upward with α=6.75, with passive wedge
(A) b= 28.57m				
(B) b= 25.35m				
(C) b= 27m				

3. Cohesion and Angle of Friction

Cohesion: For small gravity dams, due to limited area of contact, a small amount of cohesive strength can effect in a marked increase in resistance of sliding over the resistance offered by friction alone. Researchers suggest that, cohesive strength can be estimated on the basis of the age of the dam, the construction practices, and degradation of materials.

Canadian Dam association CEA (1998) [14], is noted that the cohesive strength of bonded contact joints is generally found to be twice the tensile strength, i.e. $c=2f_t$ (f_t is the tensile strength of the bonded joint). *[Lo, 1994]* showed from extensive experimental results that the average direct tensile strength of a bonded contact to be 0.92MPa. *[Lo, 1994]* reported that the least tensile strength for recovery of an intact contact during drilling was 0.18MPa (0.365MPa cohesion) [14]. Therefore, the strength of a known bonded contact that is broken through drilling should be assumed to be not more than this value. The magnitude of cohesion that will be taken into account in this study is (c=0.2MPa and c=0.4MPa) by assuming presence of weakly bonded contact.

Angle of friction: according to *"Guidelines for concrete dam"* unless the angle of friction of the sliding plane considered is well-documented by laboratory tests, the following values shall be used: 50^o for hard rocks, rough surface, 45^o for hard rocks, small roughness, 40^o for loose rocks, and 45^o for sliding planes in concrete. The angle of friction is considered as φ=45^o for the sliding planes in concrete and with contact with rock[14].

4. Stability Requirements

4.1. *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) instead of the reference section DAM 1A (b = 28.57m) to show the effect of the configuration which considered in this project; since DAM 1A satisfies the most requirements of stability. Forces acting on DAM 1B are shown in Figure 5.

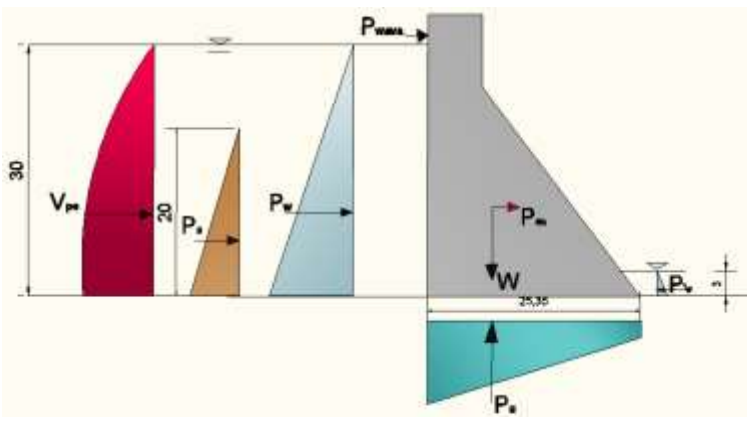


Figure 5: Forces acting on DAM 1B

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

γ_c: unit weight of concrete =24kN/m^3

vol.: volume of the dam.

*2.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)

γ_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.

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

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

where: $K_A = \frac{1-\sin\varphi_s}{1+\sin\varphi_s}$, φ_s: angle of shearing resistance of sediments=33^o, γ'_s: effective unit weight of silt=18kN/m^3, h_s: height of accumulated silt=20m (at about the end of the age of the dam $\approx 2/3h_w$).

1. *Wave force* **(**P_{wave}**)**: $P_{wave} = 2\gamma_w {h_{wave}}^2$ [1] …(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.

2. *Earthquake force***:**

a. Inertia force : $P_{eh} = W\alpha_h$ [1] (16)

b.Hydrodynamic force: $V_{Pe} = 0.726\, p_e = c_1\alpha_h\gamma_w h_w$ [1] …(17)

and the moment of this force $M_{pe} = 0.299\, p_e y^2$ [1] …(18)

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

θ : Angle in degrees, which the upstream face of the dam makes with the horizontal=90^o.

p_e= hydrodynamic earthquake pressure normal to the face,

c_1 = a dimensionless pressure coefficient.

α_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

4.2. *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. [8]

4.3. *Factors of Safety*

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.

4.3.1. *Acceptable safety factors*

USBR considered acceptable limits for sliding safety factors, as shown in Table 2 [3].

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. In the other hand USACE uses the values shown in Table 3[7].

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	$0.5f'_c$	$0.6f_c'^{2/3}$
Extreme	Within base	1.3	$0.9f'_c$	$1.5f_c'^{2/3}$

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

4.3.2. *Factor of Safety Against Overturning*

According to USBR, the factor of safety for overturning *F.O.O* is not usually tabulated within other stability factors for Bureau 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**.**

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

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; and find out whether there is a tension stresses or not. To avoid tension stresses the resultant of all forces acting on a dam should pass through the middle-third of the base of the structure, i.e. $e < b/6$

$$\text{when: Resultant location } (X') = \frac{\sum M}{\sum V}, \text{ then } e = \frac{b}{2} - X' \qquad \dots (20)$$

Carrying out the stability analysis against overturning for various loading combinations, DAM 1B possesses the following values of safety factors:
Table 4 shows the factors of safety against overturning according to USBR for three different loading conditions.

According to USACE, Table 5 shows the values of eccentricity for three conditions.

Table 4: Factors of safety against overturning of DAM 1B according to USBR

Loading condition	F.O.O *(Obtained)*	*Specification*
Usual	1.58	>1.5
Unusual	1.297	>1.5
Extreme	1.36	>1.5

Table 5: values of eccentricity of DAM 1B according to USACE

Loading condition	*Eccentricity*	*Specification*
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 30.9m instead of 33.6m for unusual loading condition, and 28.1m instead 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.

4.3.3. *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 [9]. The sliding factor of safety is the ratio of the actual frictional shear stresses to the stresses necessary to achieve equilibrium. Three methods to calculate factors against sliding: sliding resisting, shear friction and limit equilibrium method[4]. USBR uses *shear friction method* for the sliding stability.

- Without resistive wedge (dam-type 1 & dam-type 2): $F.S.S = \frac{R}{\sum H}$ … (21)

since $R = \frac{c.A}{\cos\alpha\,.(1-\tan\varphi.\tan\alpha)} + \sum V.\tan(\varphi+\alpha).$ … (22)

- With resistive wedge (dam-type 3 & dam-type 4): $F.S.S = \frac{R+P_p}{\Sigma H}$ … (23)

The *limit equilibrium method* that used by USACE [7] suggests that the factor of safety against sliding is given by:

- Without resistive wedge (dam-type 1 & dam-type 2):

- $$F.S.S = \frac{c.A+[\sum V.cos\alpha+\sum H.sin\alpha].\tan\varphi}{\sum H.\cos\alpha-\sum V.\sin\alpha} \quad \text{… (24)}$$

- With resistive wedge (dam-type 3 & dam-type 4):

$$F.S.S = \frac{\sum_{i=1}^{i=m}\frac{c_i.A_i.cos\alpha_i+\sum V_i.\tan\varphi_i}{\eta\alpha_i}}{\sum_{i=1}^{i=m}[\sum H_i - \sum V_i.\tan\alpha_i]} \quad \text{…(25)}$$

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 6.

Table 6: Factors of safety against sliding of DAM 1B according to USBR & USACE

Loading condition	*Parameters*	*Sliding factor (Obtained)*		*Specification*	
		USBR	USACE	USBR	USACE
Usual	c = 200	2.35	2.35	>3	>2
Unusual	&	1.84	1.84	>2	>1.7
Extreme	φ = 45	1.79	1.79	>1	>1.3
Usual	c = 400	3.37	3.37	>3	>2
Unusual	&	2.67	2.67	>2	>1.7
Extreme	φ = 45	2.56	2.56	>1	>1.3

Table 6, 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) 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.

4.3.4. *Safety Against Compression (Crushing) & Tension*

4.3.4.1. *Gravity Method* [1]

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 3 will be used for checking the safety against compression (Crushing) & Tension for both methods.

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:

$$\sigma_{n_D} = \frac{\sum V}{b}\left(1 + \frac{6e}{b}\right) \qquad \ldots (26)$$

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

The normal stress at the heel is: $\sigma_{n_U} = \frac{\sum V}{b}\left(1 - \frac{6e}{b}\right)$ … (27)

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.

Table 7 illustrates the normal stresses on heel and toe for DAM 1B; the results show that all stresses remain safe limits for all loading combinations.

Table 7: Normal stresses on DAM 1B

Loading condition	*Normal Stresses*	*Obtained (kN/m^2)*	*Specification (kN/m^2)*
Usual	At heel (σ_{n_U})	414.4	<7500
Unusual		586.5	<12500
Extreme		595.95	<22500
Usual	At toe (σ_{n_U})	114.96 (Compression)	0
Unusual		-93.1 (Tensile)	<5130 (Tensile)
Extreme		-66.55 (Tensile)	<12824.8 (Tensile)

4.3.4.2.Finite Element Modeling

The Finite Element Method (FEM) is a technology key 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 [10]. 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 [10]. 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 6.

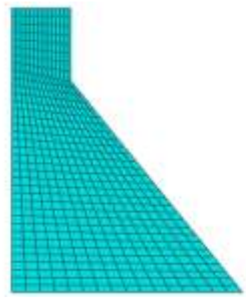


Figure 6: Finite element mesh of DAM1B

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), 33.6 m 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] [5], the tensile strength was estimated to be $f_t = 0.32{f'_c}^{2/3} = 2.736$MPa[5]. When f'_c is compressive strength of concrete and it was assumed as 25MPa in this project[2], Table 8.

Table 8: Concrete properties of DAM 1B and 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 subjected 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[12], Figure 7, are applied to all nodes at the base of the dam.

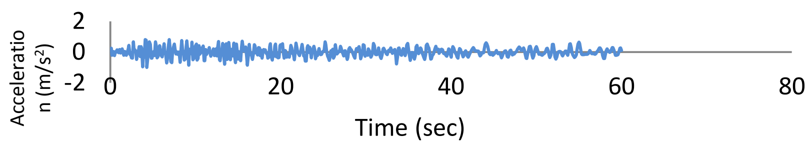


Figure 7: Acceleration – time records of earthquake hit Ali Al-Gharbi [12]

5. Verification of Study

Many small dams were designed in Iraq by official governmental centers. So to ensure that the stability methods used in this research work are considered dependable, one of these dams was taken to verify the methods used to achieve acceptable safety factors. Chem Kanny Maran dam was used as a proof for these stability methods. Moreover, Baozhusi dam (in China) was used to ensure accuracy of the FEM used to compute the stresses in various cases of dam sections for multiple loading combinations and comparing the results with the original results of this dam found in [Alsuleimanagha, Z, Liang, J, 2012] [10]. The results obtained showed close agreement.

6. Results and Discussion

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.

The stability and stress analysis indicate that there are some important factors affecting the structural stability of small concrete gravity dam; among those appear the base width of the dam, the inclination of the base toward the downstream side, the existence of resisting passive wedge, the cohesion of the dam material, the angle of friction of the failure plane, and others. Hereby the effect of the four main factors will be summarized in the figures 8-13 below.

6.1. *The Effect of the Base Width*

As the length of the base width increases then the dam will be more stable; it is true that DAM 1A with base width 28.57m is more stable than DAM 1C with average base, which, in turn, is more stable than DAM 1B with 25.35m base width. This effect appear on factors against overturning and sliding as shown in figure 8 and figure 9, respectively.

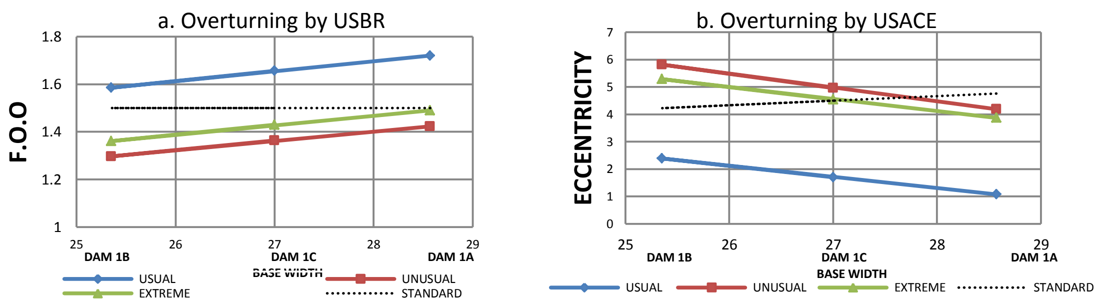


Figure 8: Effect of increasing the base width of DAM 1B on overturning according to:a.USBR &b.USACE

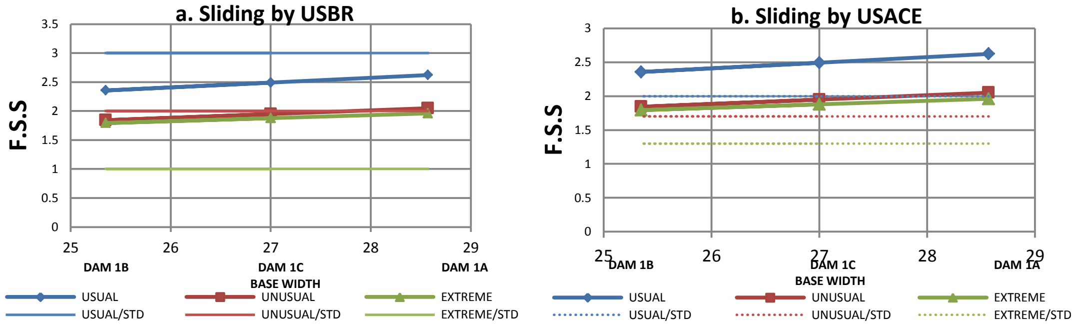


Figure 9: Effect of increasing the base width of DAM 1B on sliding according to a.USBR &b.USACE

6.2. *Effect of Slope of the Base on the Stability of Small Concrete Gravity Dam*

Figure 10 shows three loading condition, the upward inclination (counterclockwise 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, which in turn has more stability and safety factors (overturning and sliding) from the case of downward inclination (clockwise rotation) around the heel, DAM 2B/I, which is the lowest point of upstream face of the dam; for two standard, USBR and USACE.

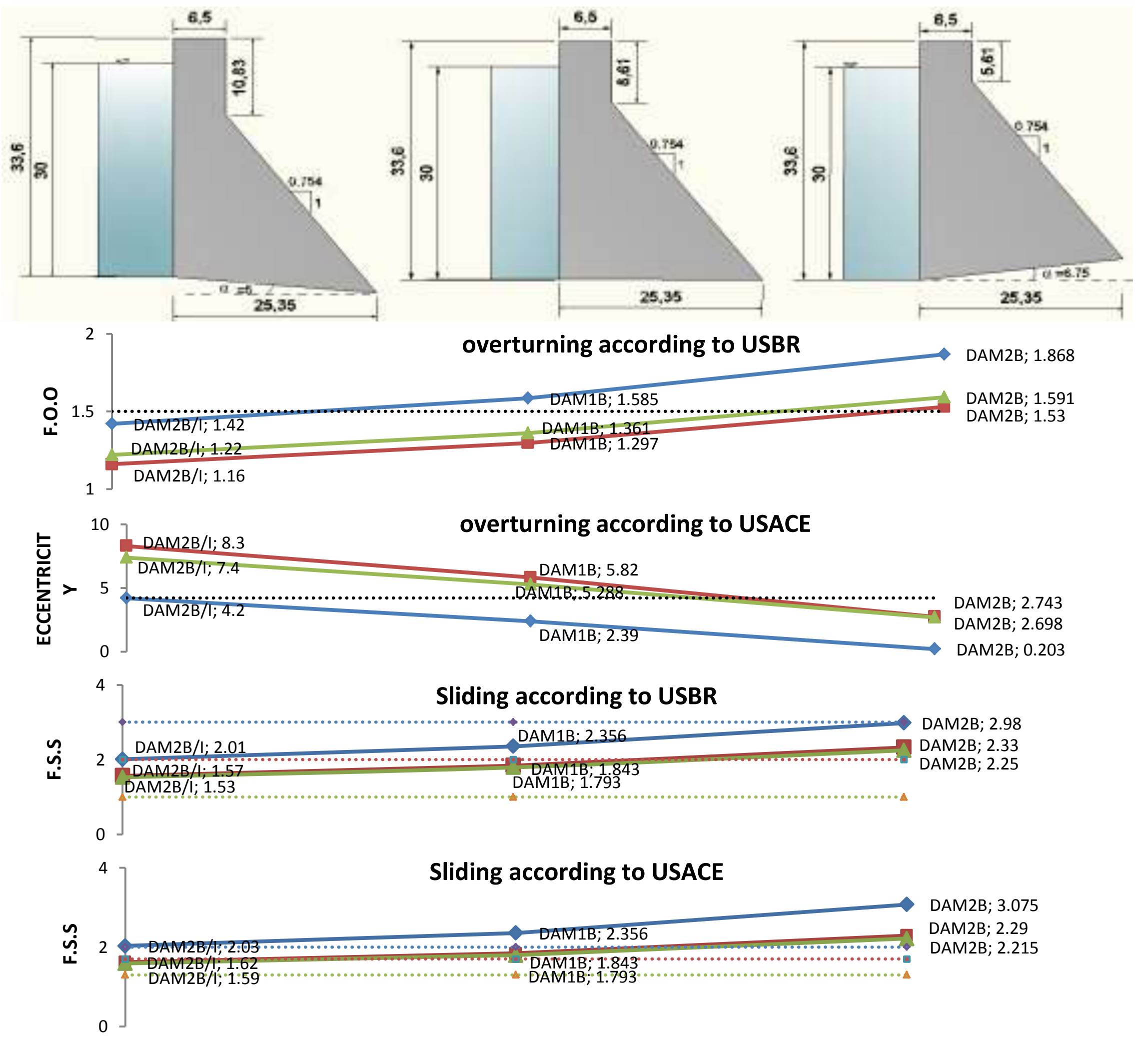


Figure 10: Effect of inclined the base of DAM 1B on safety factors according USBR and USACE

6.3. *The Existence of Resisting Passive Wedge*

Figure 11 indicate the fact that when checking the stability against overturning and sliding of DAM 1B in both specification USBR and USACE for various loading combination when using a resisting passive wedge. The figure shows that a limited increment of factors against overturning, this increment is due to increasing of the weight of the dam not by the presence of the passive wedge. On the other hand, the existence of this wedge increase the sliding factor by about 58% for USBR standard, and about 47.5% for USACE standard. The effect of the existence of passive wedge is more clear when combining the effect of inclined base and using resisting passive wedge, Figure 12 indicate the effect of this combination of configuration the passive wedge and the inclined base on the stability against overturning and sliding in both standards USBR and USACE for various loading combinations of DAM 1B.

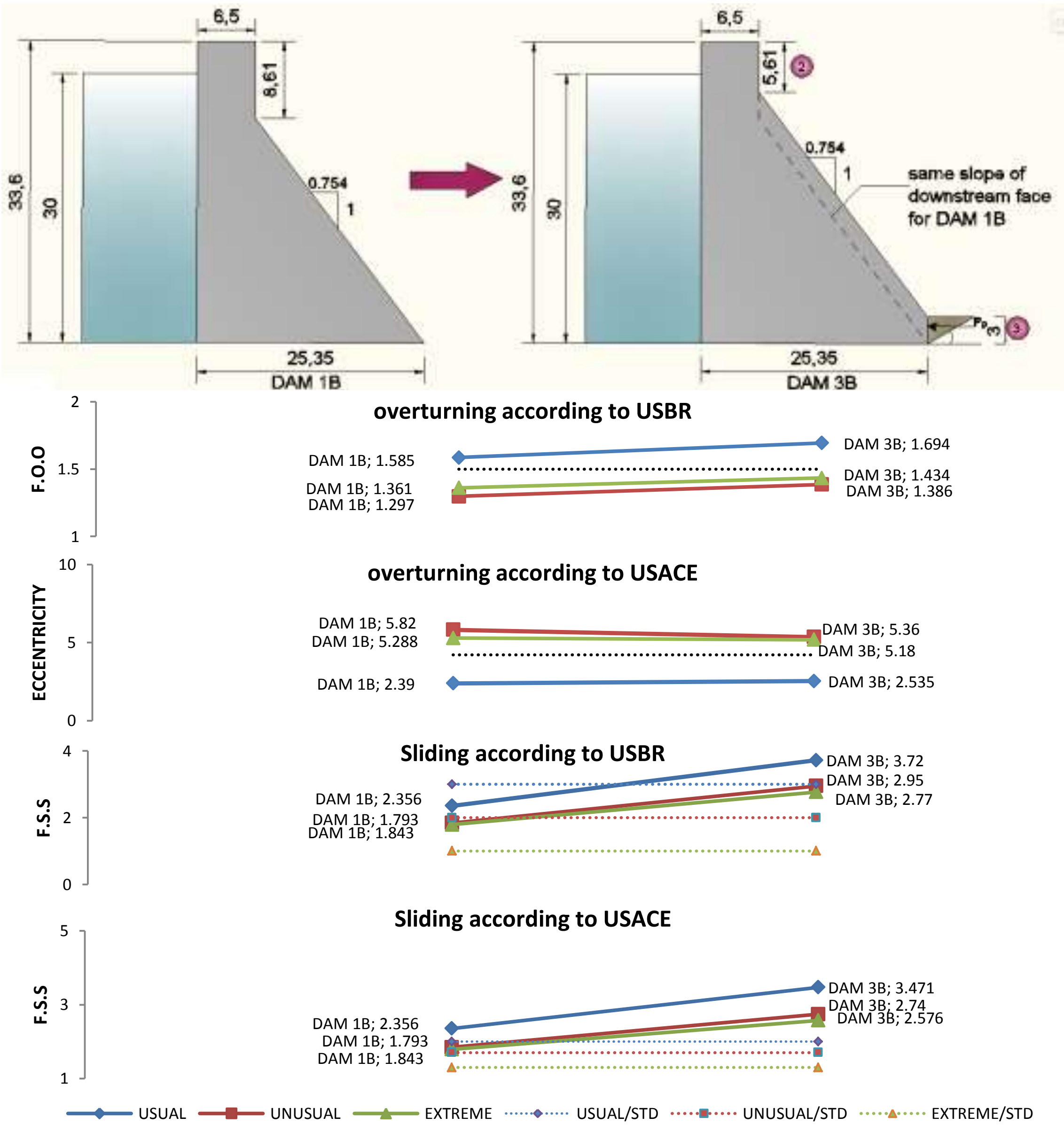


Figure 11: Effect of the presence of passive wedge adjacent to DAM 1B on safety factors according USBR and USACE

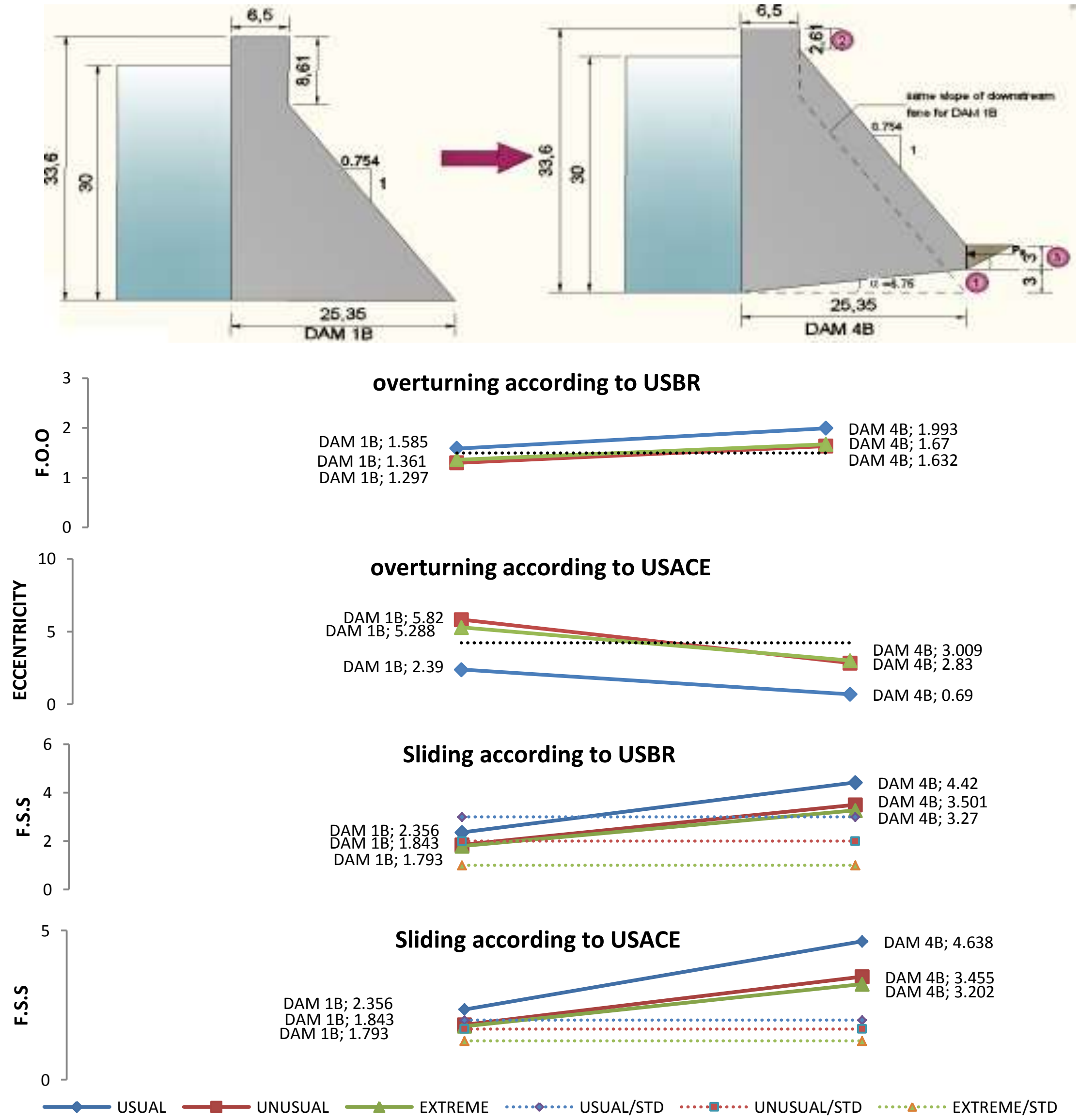


Figure 12: Effect of inclined the base of DAM 1B on safety factors according USBR and USACE

6.4. *The Effect of Cohesion on the Stability*

The contribution from cohesion can be included in the calculation of the factor of safety against sliding. 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. At DAM 1B, for USBR the increment of *F.S.S* is 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.

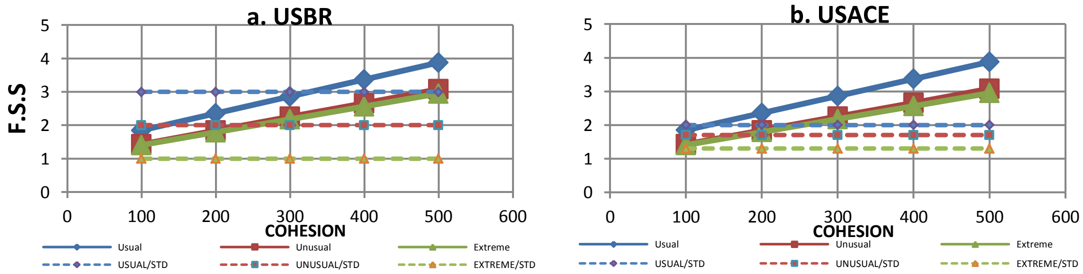
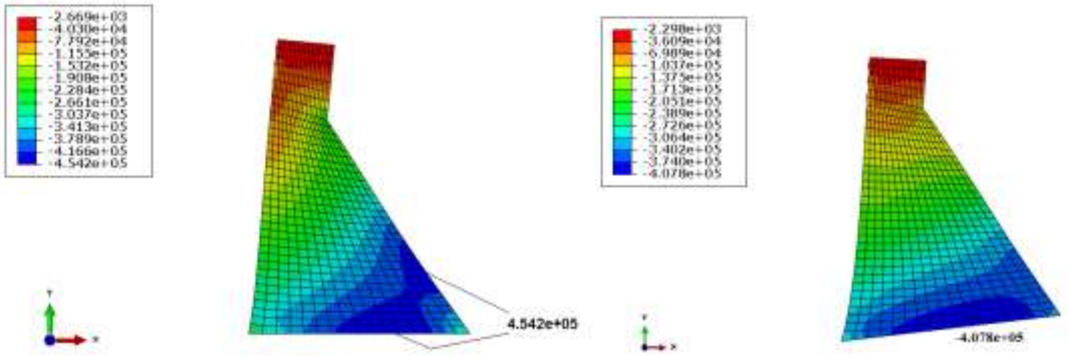


Figure 13: Effect of cohesion on F.S.S of DAM 1B according to a. USBR &b. USACE

6.5. *Results from ABAQUS software*

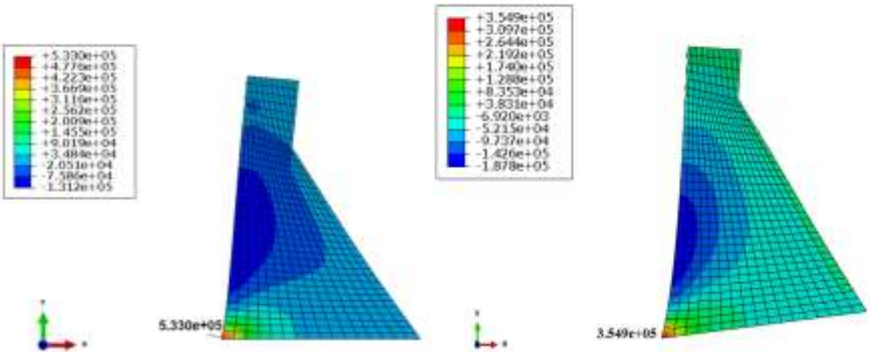
The aim of the FEM is to determine the responses of the structure concentrating on the maximum tension and compression stresses and the displacements, based on the characteristic of the structure and the nature of the earthquake.

Figures below show the result of stresses analysis for DAM 1B and DAM 2B to display the effect of the inclination on stress distribution for three loading conditions.



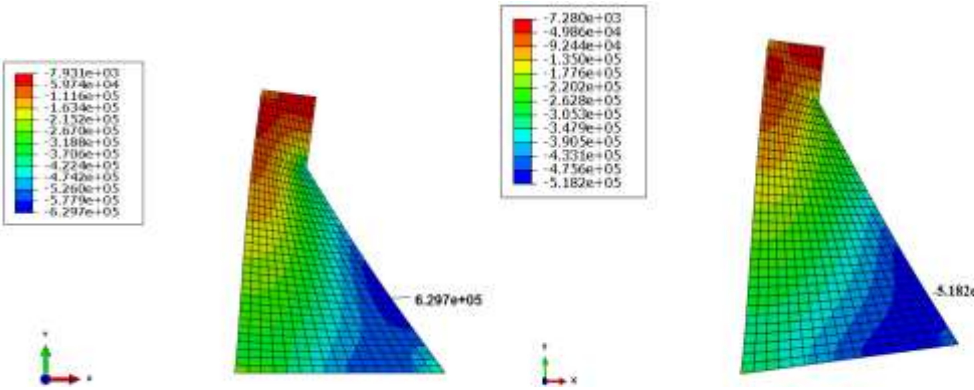
a. DAM 1B b. DAM 2B

Figure 14: Maximum compression stresses in DAM 1B and DAM 2B for usual loading condition



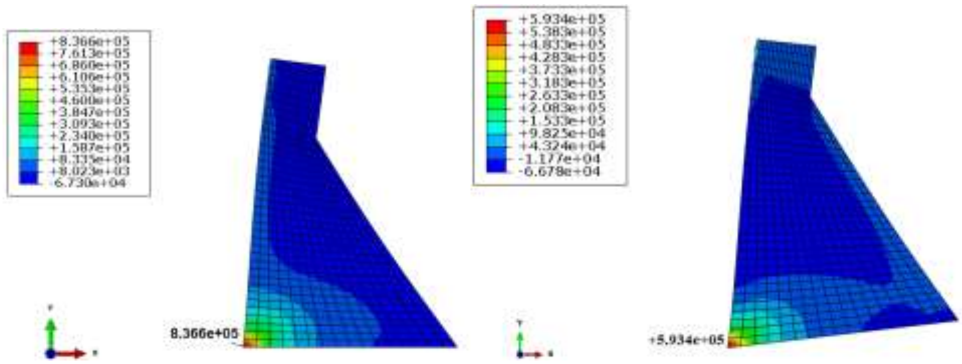
a. DAM 1B b. DAM 2B

Figure 15: Maximum tensile stresses in a. DAM 1B and b. DAM 2B for usual loading condition



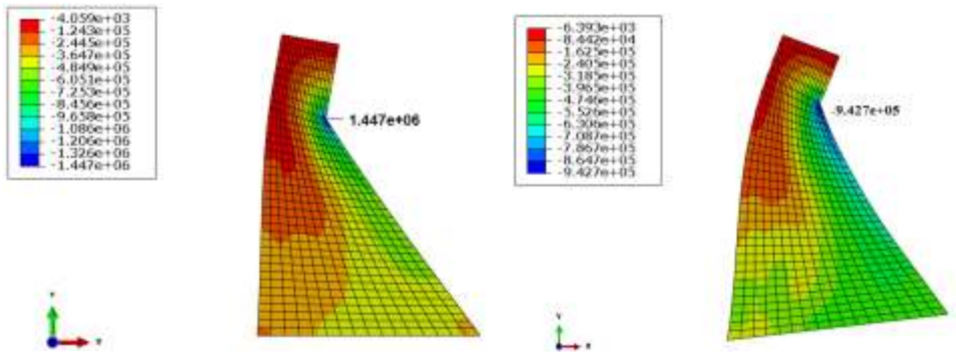
a. DAM 1B b. DAM 2B

Figure 16: Maximum compression stresses ina. DAM 1B and b. DAM 2B for unusual loading condition



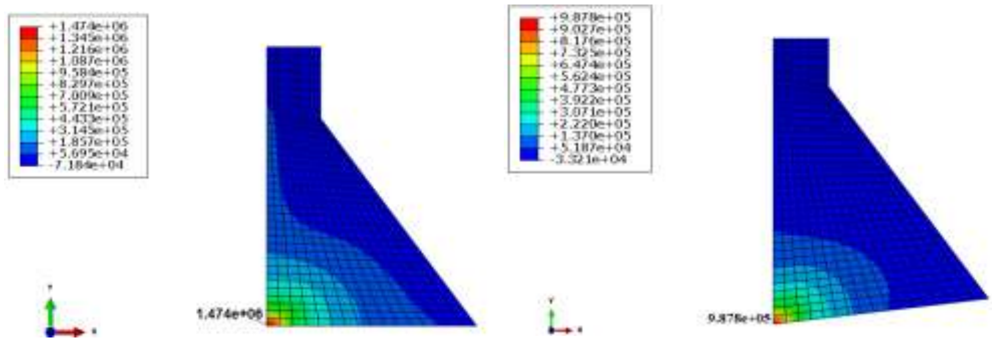
a. DAM 1B b. DAM 2B

Figure 17: Maximum tensile stresses in a. DAM 1B and b. DAM 2B for unusual loading condition



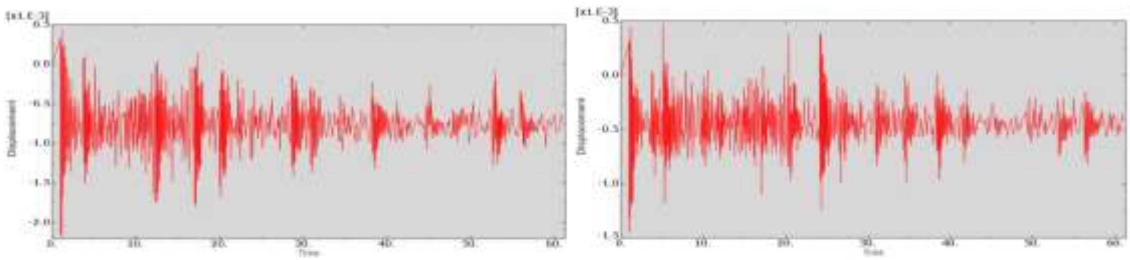
a. DAM 1B b. DAM 2B

Figure 18: Maximum compression stresses in a. DAM 1B and b. DAM 2B for extreme loading condition



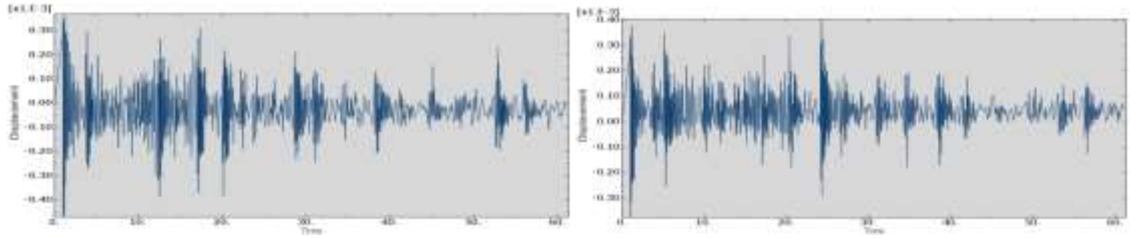
a. DAM 1B b. DAM 2B

Figure 19: Maximum tensile stresses in a. DAM 1B and b. DAM 2B for extreme loading condition



a. DAM 1B b. DAM 2B

Figure 20: Horizontal crest displacement of a. DAM1B b. DAM2B related to ground displacement of extreme condition



a. DAM 1B b. DAM 2B

Figure 21: Vertical crest displacement of a. DAM 1B and b. DAM 2B related to ground displacement of extreme condition

For three loading condition, usual, unusual, extreme; the maximum compression stress for both DAM 1B and DAM 2B does not exceed the allowable compressive strength of the selected concrete which is 25MPa.

The highest value of the tensile stress for DAM 1B and DAM 2B was occurred at the heel of the dam; this value is acceptable, since it is less than 2.74MPa that given in ($f_t = 0.32{f'_c}^{2/3} = 0.32 \times 25^{2/3} = 2.74$ 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.

The results obtained for DAM 1B and DAM 2B show that DAM 2B is with better stability than DAM 1B, since DAM 2B satisfies all stability requirement for the same loading condition and shows less stress values in both tension and compression. One of the negative marks is that DAM 2B weighs more than DAM 1A by about 4%.

7. Conclusions

1- Many conclusions are withdrawn from this study; the main among those 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 permit 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.

2- 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.

3- Dams-type 2 with upward inclination of the line of the base around an axis passing through the heel by 6.75^o (counterclockwise rotation), give more stability (for overturning and sliding) from Dams-type 1 with horizontal base, which in turn have more stability from the dams in case of downward inclination around the heel (clockwise rotation).

4- Also Dams-type 2 show less stress values in both tension and compression. When applying USBR standards, the percentages of increase in *F.O.O* by comparing Dams-type 1 with Dams-type 2 were about 18.5% for usual condition, 18% for unusual, and 18.7% for extreme loading combination, respectively. The same conclusion is true when applying USACE standards. The results illustrate the same fact when computing the factors of safety against sliding in both standards by applying USBR and USACE for Dams-types 1, 2, groups *A*, *B*, and *C*. The percentages of increase to prefer Dams-type 1 on Dams-type 2 were about 27.7% for usual loading condition, 27.8 % for unusual condition, and 26.6% for extreme loading condition. These percentages were for USBR standards, while, for USACE standards the percentages were about 35%, 28%, and 26%, for usual, unusual, extreme loading combinations, respectively.

5- 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.

6- Dams-type 4 was produced by the combination of passive resistance wedge with upward inclination of the line of the base, where the stability against overturning and sliding increases with largest ratios. This increment is about 27% for overturning stability and about 85% for sliding stability.

7- 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 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. Approximately, the same ratios were obtained for USACE standards.

8- 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 condition. The stresses obtained in Dams-type 2 are less than the stresses obtained in Dams-type 1 with various loading combinations. All the stresses computed were within acceptable limits.

9- By using acceleration- time records of Ali-Al-Gharbi earthquake, the motion of the upstream crest relative to the lowest heel point at the upstream side was found insignificant and about 1.75mm in the horizontal direction.

10- The profile DAM 2B with a base inclined by 6.75^o upwards toward downstream face, and width b = 25.35m, was found the most optimum section for a dam required to store a volume with a height of water, h_w =30m. In this dam the value of cohesion of approximately c =200kN/m^2 was found sufficient to achieve the sliding stability for all loading combinations.

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