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Analysis of Protective Layer Under Overtopping Flow Using Strain Measurements

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الملخص العربى :

أثناء انشاء السدود الخرسانية والتى تعمل على حجز سعات تخزينية ضخمة للمياه مثل سد النهضة الأثيوبي والذى تم تصميمة لحجز خزان بسعة 74 مليار م^3 وبارتفاع 640 م فوق مستوى سطح البحر . فان حدوث الفيضانات فى فترات الانشاء تسمح للمياه المحتجزة امام السد بالمرور أعلى الاجزاء الخرسانية المنخفضة المنسوب او أعلى الاماكن الغير مكتملة الانشاء، ونظرا للدورة الهيدرولوجية للمياه والتى تتعرض لها الخزانات الضخمة فى دول المنبع فقد يتسبب الفيضان فى التدفق من أعلى منسوب 640 م وتتدفق المياه لتسقط على منطقة خلف السد فيحدث نحر بمنطقة خلف السد الخرساني مسببة حفرة يتزايد عمقها مع الوقت ومع تتابع التدفقات خلال بضع سنوات تتسع هذه الحفرة المتكونة فى الخلف وتقترب من اساسات السد مسببة خطر على اتزان الجسم الخرساني للسد، لذلك من الضروري عمل حماية لهذه المنطقة المتضررة. خلال هذه الدراسة تم عمل نموذج فيزيقى يمثل جسم المفيض للسد بمقياس 1:30 ونموذج معدني لقياس تاثير المياه المتدفقة من اعلى السد على منطقة خلف السد. حيث تم قياس الانفعالات الحادثة على النموذج المعدني عن طريق استخدام حساسات لقياس الانفعال مثبتة على 5 نقط بمنطقة الشد، وموزعة بكامل عرض النموذج المعدني وموصلة باسلاك توصيل لقنوات لرصد القراءات المسجلة من خلال تغير المقاومة على طريقة توصيل كوارتر ويستون بريدج، ومن هنا تم رصد التغيرات فى المقاومة المجهولة وتم رصد الانفعالات للخمس نقاط حتى منسوب 640 م، وباستخدام معادلات تيموشينكو للأسطح تم استنتاج اقصى الاجهادات وعزوم الانحناء الواقعة على النموذج المعدني. حيث يبلغ اقصى اجهاد شد حوالى 1.15 طن /سم^2 والذى يتسبب بحفرة عمقها 3.4 م بالطبيعة بالمنطقة المتضررة اسفل منسوب القاع لمنطقة خلف السد. لذلك تم اقتراح أنظمة لتخفيف هذا الاجهاد الواقع بالخلف عن طريق عمل مقارنة بين عدد من الانظمة الانشائية المقترحة لتقليل هذا التأثير وتم استنتاج ان النموذج رقم 3 والمكون من (طبقة خرسانية مثبتة على عدد 4 حوائط قاطعة). هو اكثر النماذج الانشائية اقتصاديا لمقاومة اجهادات الشد الحادثة بالمنطقة المتضررة تحت تأثير التدفق العالى للمياه بالخلف والذى لن تستطيع التربه وحدها مقاومته .

Abstract:

The Grand Ethiopian Renaissance Dam (GERD), which was designed and implemented with a full storage capacity for the regional water supply that reaches the downstream countries, has a storage capacity of 74 billion cubic meter at 640 (a.m.s.l). After the process of filling the reservoir, the storage capacity of water passes over uncompleted portions of dam body which have low crest level. The filling process of the reservoir allows water to fall from different heights depending on the operational stage. The fallen water will perform deformation on riverbed and consequently on the soil layers at the downstream side. Therefore, a protective layer is required at the downstream side to reduce any corrosion. This study aims to quantify the effects of the fallen water jet on the structural model represented the jet impact force on the protection soil layer behind the dam. A physical model was constructed in the lab for the dam spillway with different crest levels which act as sills to pass water over them. The structural model of the protective layer requires installing special equipment to measure the normal stresses and deformations occurred due to the flow impact, it is found that; the maximum deflection value is 3.4 m for prototype, the maximum normal tension stress reaches to 1.15 t/cm2 within protective layer boundary. these values are higher than the available tension strength of used material for construction of the protection layer. it leads to know the dangerous of scouring that occurs behind the dam at the impacted area of D/S side. So, a different structural solution is being mathematically investigated to mitigate the high impact of normal stresses and deformations. A different modules of structural systems are attained in order to find an efficient and suitable system for the protective layer. If there was no any formation of scour downstream dam body, the most appropriate structural system should be case No. 3 (using 4 parallel cutoff walls), which selected to mitigate effects of overtopping flow impact as much as possible. otherwise it is not the right system and there is a necessary need to reduce the reservoir water level to another safe one in the upstream reservoir level in (GERD).

1. Introduction

Generally, the construction of concrete dam process across large river requires long time period varying between three to more than ten years, as in some African dams. During the construction time, the watershed hydrologic cycle may include one or more high flood, which allows flow overtopping the uncompleted portions or over the low concrete portions of the dam during construction stage. Dam overtopping can also occur after construction when inflow exceeds reservoir capacity or spillway discharge capacity owing to insufficient storage or release capabilities. As a result; failure of some/all dam portions, potential loss of life, a huge scour hole with severe depth endangering dam structure stability, and significant downstream damages along the river bed and its side banks are some of the possible negative impacts. For concrete dams; the failure is suddenly occurred for part/all dam body due to over stresses from overtopping or due to excessive deformations for dam foundations or abutments. Meanwhile, for an embankment dams, it happens due to overtopping or piping. As a result, instead of extending the spillways or increasing the reservoir storage and dam crest, construction of overtopping protection for the dam body, is a possible option. Overtopping protection techniques include roller-compacted concrete, conventional concrete, precast concrete blocks, gabions, turf reinforcement mats, vegetative cover, flow-through rockfill, reinforced rockfill, riprap, and a variety of geo-synthetic materials. During this study, the Grand Ethiopian Renaissance Dam (GERD) project will be used as an example for the simulation of overtopping flow during construction and study of its resulted action into downstream protective layer. The GERD project is a combination of 1.8 km long roller-compacted concrete gravity dam, a 270 m long spillway and a 5 km embankment saddle dam, required to maintain the design water level and storage, given the relatively flat topography of the dam site. GERD' location close to an international border between Ethiopia and Sudan, increases security risk of the dam. The design of spillways and overall construction will be tested during filling and operation.

2. Objectives

This research identifies the potential vulnerabilities of roller compacted concrete dams to flood overtopping. The research also, provides a method to determine the required design forces and the resulted deformation of protective layer by using strain gauge measurements technique, that can be used for protective against failure during a large flood event.

3. Methodology and Model Setup

During this study, the dimensions of the assumed protective slab layer and its anchoring system at the dam toe need to be specified based on accurate analysis for dynamic pressure distribution.

3.1 The Structural Analysis Method

By using the method of finite difference forms for deformations. Hence, we will use electrical resistance strain gauge transducers in the model of protective layer, the resulted measurements of the model will be strain values. And, in order to convert the resulted strain values to its corresponding displacements values, according to figure (1), we use the following equations that relate the axial strain and the displacement within the 2-dimensions plate.

$$\epsilon_{xx} = -z\frac{\partial^2 w}{\partial x^2} \qquad (1) \qquad , \epsilon_{yy} = -z\frac{\partial^2 w}{\partial y^2} \qquad (2)$$

in which ∂_w means the variation in displacement perpendicular to the strained element, and ∂_x or ∂_y means the variation in the length of the strained element in each dimension. By using finite difference technique as shown in figure 1, for the right-hand side' components of the above equations eqn. (1) and eqn. (2), we can get on, the following expressions for 2nd derivative of vertical displacement w either in x direction with spacing Δx or y direction with spacing Δy, as follows;

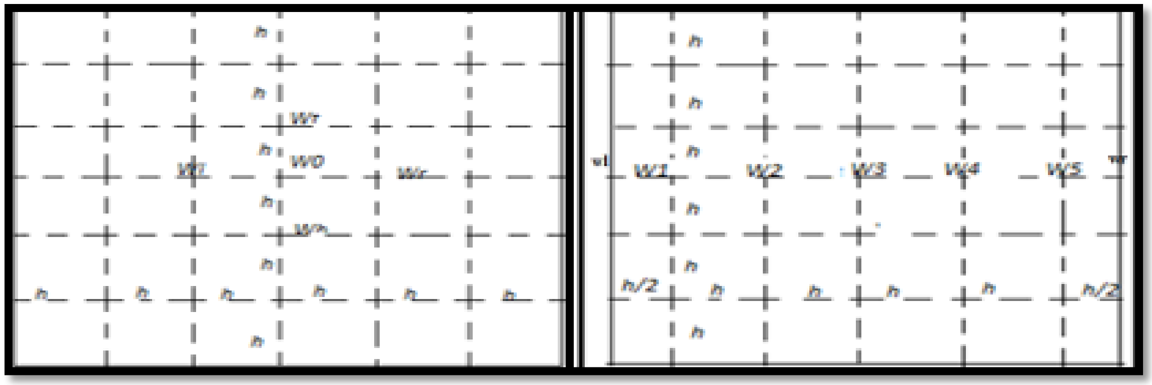

Fig.1 Uniform Mesh for Finite Difference along the structural width &The Measurement Positions of the Model

$$\frac{\partial^2 w}{\partial x^2} = \frac{1}{\Delta x^2}(w_L - 2w_O + w_R) \quad (1') \quad , \frac{\partial^2 w}{\partial y^2} = \frac{1}{\Delta y^2}(w_T - 2w_O + w_B) \qquad (2')$$

In this simulation, the strain transducers were assumed to be, at equal divisions along x and y axes, i.e. $\Delta x = \Delta y$ = h, the first axial strain relation and the second relation is;

$$\epsilon_{xx} = -\frac{z}{h^2}(w_L - 2w_O + w_R) \qquad (3) \quad ,\epsilon_{yy} = -\frac{z}{h^2}(w_T - 2w_O + w_B) \qquad (4)$$

Where w_o is the pivot point deflection, w_L and w_R are the left and right deflection, while w_T and w_B are the top and bottom deflections. Then by using the following relation of Bending Moment M_{xx} and stress σ_{xx}, with axial deformations along X and Y axes, the following relations will be obtained. And by using eqn. (1), eqn. (2) of axial strains along x and y axes, the above equations (5) and (6) can be expressed in the following form;

$$M_{xx} = -\frac{Et^2}{6(1-\mu^2)}(\epsilon_{xx} + \mu\epsilon_{yy}) \qquad (5) \quad , \sigma_{xx} = -\frac{E}{(1-\mu^2)}(\epsilon_{xx} + \mu\epsilon_{yy}) \quad (6)$$

As the water jet will be distributed only along the middle strip of the physical model of protective layer, so the upper and lower strips' displacements values will be smaller than the middle strip values, and can be neglected, when the simulated plate has equal divisions into both perpendicular directions. The above relation is changed to the following one.

$$\sigma_{xx} = -\frac{Ez}{(1-\mu^2)h^2}(w_L - 2(1+\mu)\,w_O + w_R) \quad (7) \quad ,M_{xx} = -\frac{t^2}{6}\sigma_{xx} \qquad (8)$$

So, it is assumed that, the protective layer in the model, can be represented as a steel plate of thickness t = 5 mm and z = t/2, with boundary conditions as fixed edges along its borders at x = 0 and a, y = 0 and b, where a = b = 1 m and h= 0.2 m, for the model. Then by direct application for the previous equation of strain value eqn. (3) at each position of measurements as shown in figure 1(b), from point (1) to point (5), the following system of strain equations will be obtained;

$$\epsilon_{1x} = (0.125w_1 - 0.0625w_2)$$

$$\epsilon_{2x} = (-0.0625w_1 + 0.125w_2 - 0.0625w_3)\ ,$$

$$\epsilon_{3x} = (-0.0625w_2 + 0.125w_3 - 0.0625w_4)\ ,$$

$$\epsilon_{4x} = (-0.0625w_3 + 0.125w_4 - 0.0625w_5)\ \text{and,}$$

$$\epsilon_{5x} = (-0.0625w_4 + 0.125w_5)$$

And due to clamped boundary conditions along edges of plate, the slope of deflected shape will be equal to zero; Knowing that the deflection along clamped edges equals zero, i.e. $w_{Lx} = w_{Rx} = 0$ in Solving the above-mentioned system of strain equations from ε_{1x} to ε_{5x} by using matrix inverse method; the deformation values will be obtained at the measuring points.

3.2 Scales Representation for the Model

The Linear Dimensions; (Length, Width, height) $\lambda = L_r$, also for an elastic model subjected to static action, the deformation grows up slowly, the elastic forces are dominant and the relationship between stresses σ and strains ε, is represented by Hooke's law. Since the elastic strain, ε is dimensionless, so it will be equal in the prototype and in the physical structural model for the protective layer material i.e. $\epsilon_P = \epsilon_m$

So, $\sigma_P = E_P.\frac{\sigma_m}{E_m}$ (12)

3.3 Simulation of The Design Flood

As the engineering scale for the length dimension is L_r =30, For the prototype, the design flood is the maximum one that can be occurred during the construction period and it is equal to 14700 m^3/s. and the allowed crest width for passing this discharge during construction is equal to 270 m, i.e. 54.4 m^3/s/m' or 43.55 m^3/s/0.8 m' and corresponding to about 9.02 m from the assumed flow depth over spillway. It is equal to 265 liter/s/m' and water depth of 30 cm for the linear scale model.

$$q_p = \frac{2}{3}\gamma bh\sqrt{2gh} \qquad (m^3/s/m') \qquad (13)$$

3.3.1 Assumptions for Simulation of Protective Layer

To get a suitable engineering similitude for the protective layer, it is needed to realize the situation of the produced scour hole due to overtopping flow as follows;

1. The scour may be represented approximately as circular or square shape of side length equal to crest width of the dam' spillway during overtopping.
2. Scour depth may be reached to a huge depth, so the contact between the protective layer and the underlining fill can be neglected that means the fill isn't supports for the protective layer.

3. Protective layer is used as stilling basin that has vertical walls along its edges, so the protective layer has clamped edges with these walls.

$$x^2 = 4hh_o \tag{14}$$

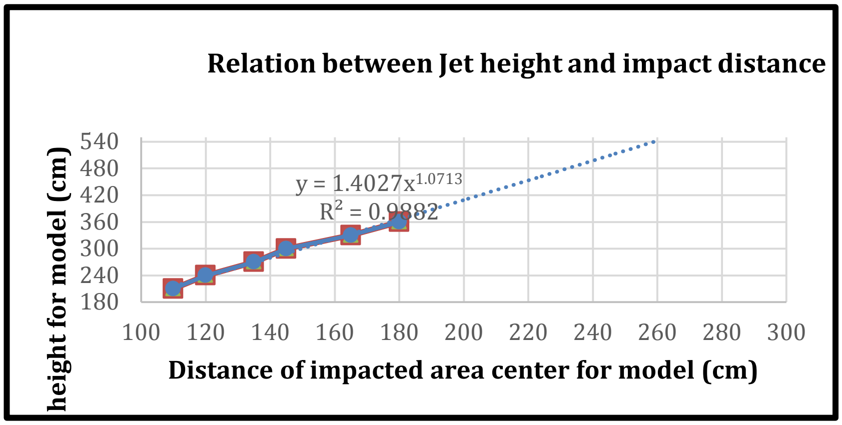

Fig.2 Relation Between Jet Height and Its Max. Impacted Area

As shown in figure (2), it can be deduced that the eq.14 is the most suitable to represent the relation between the dam height and the distance of the centered jet on impacted area. During this study and due to limited space of experimental area of laboratory and limited capacity of available data logger system, it is suggested to use movable model of area dimension (1 m ×1 m) which can be moved into one direction perpendicular to the dam body in order to cover the scaled model area for the layer.

3.4 The Model of Protective Layer

As a plate (1 m × 1 m) in both directions with thickness 5 mm, clamped edges along its 4 perimeter and welded vertically with steel plate. This model is directly supported on a two parallel rails (140 cm width and 400 cm long) with horizontal steel bracings. The rails and bracings made up of built up standard steel angles sections, to transfer the impact load over the model of the protective layer, along its lower connecting surface with the top surface of flooring concrete slab of the laboratory hall, as shown in figure (3). Also, the hydraulic physical model of the dam' spillway with different crest levels and protective layer, as shown in figure (4).

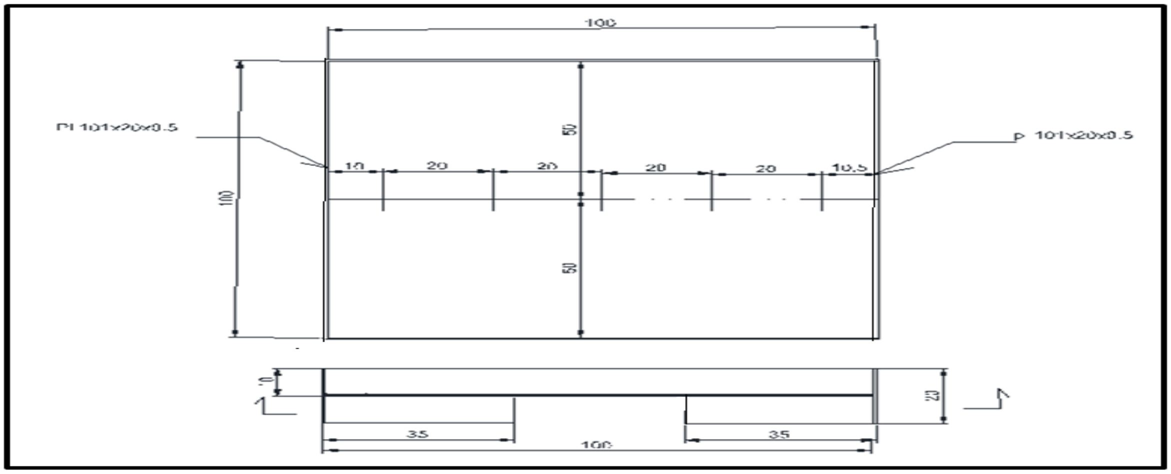

Fig.3 Dimensions of the structural model and Measurements Positions

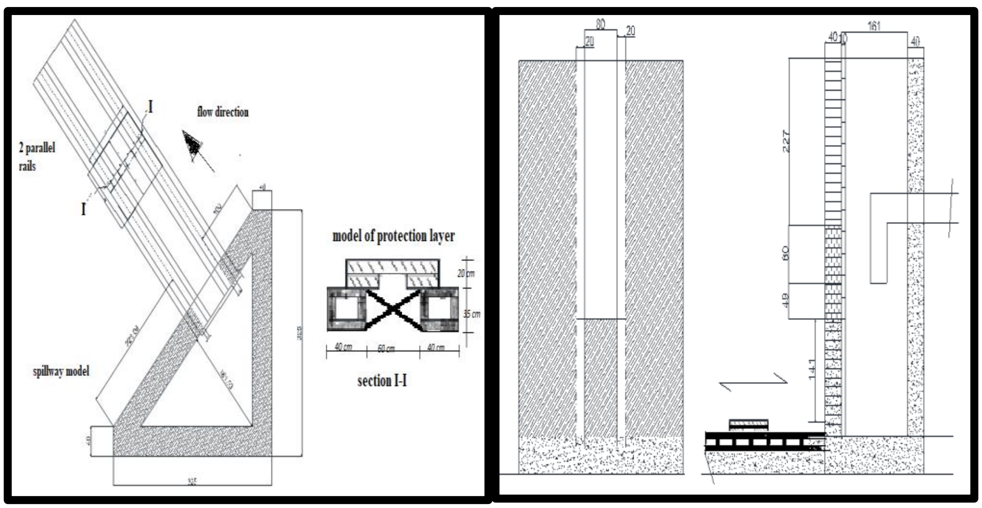

Fig.4 The hydraulic physical model of spillway. shows plan arrangement & section for the physical model (dam' spillway and protective layer)

3.5 The Strain Gauge Requirements for Measuring Flow Impact

It is very important that the strain gauge be properly mounted onto the test specimen so that the strain is accurately transferred from the test specimen, though the adhesive and strain gauge backing, to the foil itself. Manufacturers of strain gauges are the best source of information on proper mounting of strain gauges. A fundamental parameter of the strain gauge is its sensitivity to strain, expressed quantitatively as the gauge factor (GF). Gauge factor is defined as the ratio of fractional change in electrical resistance to the fractional change in length (strain): The Gauge Factor for metallic strain gauges is typically around 2.

3.5.1 Strain Measuring Device

TC-32K is a compact hand measurements device that had been used during this study, it is produced by Tokyo Sokki Kenkyuio company. It provides easy connection with lead wires and banana plugs and speedy measurements. After sensor mode selection, it is easy to set the requirements for measurements as; coefficient, initial values of the scanned channels and record their data. By using a dedicated switch box “CSW-5A”, auto measurements of the 5 points become available. Switch box type “CSW-5A” is the dedicated switch box for 5 measurement points scanning.

3.5.2 Construction of the Strain Transducers

To detect the errors of lead wire resistance changes due to electrical resistance in the strain gauge such as; Temperature-induced resistance, some forms of the Wheatstone bridge were used. The strain transducers, used herein, were built up of a Quarter Wheatstone bridge. The configuration of this strain transducer is shown in Figure 3. Each strain transducer consists of One electrical resistance strain gauge, cemented in lower side (subject to tension) of the tested model at the same location of the desired layout of the strain gauges.

3.5.3 Electrical Circuit for The Model

The main wiring system for the steel model of the protective layer downstream. the model was instrumented by Five strain transducers of Quarter bending Wheatstone bridge circuit. The terminals of the wiring system were connected via a data logger by using the strain gage accessories "terminal modules box”, which feeds the data logger system. This circuit enabled the strain transducers to be connected together with the data logger which could scan the developed strain acted on all the strain transducers at each time step and store its measurements values into its internal memory then feeds the values to the computer in digital form during data processing.

3.6 The Model Setup

Setting up the physical model and measurement device; As the 5-transducers connections setting up between the model and data logger were finished, it is moved to downstream of the spillway and settled on fixed guide rail perpendicular to spillway direction. This arrangement allows us to move away the model along this rail as the jet height increase. The 5-transducers connections free ends were connected directly to connecting wire, of same diameter and resistance as the terminals of the strain gauge and with suitable length to enable us from following the test process and measurements. Then the terminals of the connecting wires were connected to data logger via Switch box “CSW-5A. The connecting wire length reaches to about 8 m length to keep the physical model and the guide rail always in touch during the test. Which, during running the model, the impact flow velocity and its accompanying force, try to push the physical model along the rail in the flow direction but this force is resisted by friction of a coarse steel sheet surface plate with the model. the connecting wire become in lateral movement along its length, due to the vibrations resulted from this impact force and its resistance. This movement results elongation or shrinkage for the wire components, then additional tension or compression strains. To eliminate this effect, the measurements were collected for complete model, firstly, without operation of spillway and without cable extension, then secondly, with cable extension. The readings for first case were lower than the second case. The percentage between these reading sets were used to get the reading of strain gauge measurements due to operation only without the additional readings due to elongation.

4. Results and Discussion

From the previous study of physical simulation for the dam' spillway overtopping, the downstream bed is exposed to the direct impact due to overtopping flow jet at different heights. It is important especially in high dams as GERD. Jet could lead to a scour hole formation into downstream layer, either normal bed material or protective slab as stilling basin. When it continuous for decades it will cause a catastrophic failure in stability of dam. so, finding a solution during that time of construction will be late, and it will need a huge cost for different studying attempts to design and implement the protective layer in time. as a conclusion for the resulted normal tension stresses for prototype, it is found that; its values are higher than the available tension strength of used material for construction of the protective layer, whichever it. Which it reaches to (1.15 t/cm^2) within protective layer boundary. Also, the maximum deflection value reaches to 0.113 m for the model and about 3.4 m for prototype, which means total damaged of the protective layer under this effect. So, it's necessary to looking for; the resulted pressure distribution of the impacted water jet with the protective layer, and a suitable structural system of protective layer, that can endure these stresses and to reduce the chances of a scour hole formation. It is clearly concluded that designers must continue to develop the required engineering design details to ensure a performance of the system with a suitable overtopping protective under impingement jet with fluctuating pressure, the anchors are necessary to ensure the concrete protective slab remains firmly attached to the foundation. Figure (5), shows the distribution of bending moment along X-Axis (parallel to spillway crest), for the case of overtopping flow corresponding to maximum height of the prototype spillway crest 643 (a.m.s.l). The best fitting relation of it, is obtained as following:

$$\mathrm{M_{MAX}} = 0.0003\mathrm{x}4 - 0.0005\mathrm{x}3 + 0.0004\mathrm{x}2 - 9\mathrm{E}\text{-}05\mathrm{x} + 3\mathrm{E}\text{-}06 \qquad (15)$$

M is moment value, and x is the distance along X-axis of protective layer. From the principles of theory of structures, Dakhakhni [3], the relation between applied force, pressure and the resulted internal forces of structure element as following;

The second derivative of the above fitting relation eqn. 15, gives the following;

$$\mathrm{p} = \frac{\partial^2 \mathrm{M}}{\partial \mathrm{x}^2} = -0.0036\mathrm{x}2 + 0.003\mathrm{x} - 0.0008 \qquad (16)$$

Application of eqn. 16 for different values for x between zero and protective layer width, represents the pressure distribution along the layer width, and it is shown graphically in figure (5). The maximum pressure is located within the central part and decreasing towards the edges from two sides. From this figure (5), it can be seen that the maximum predicted value for resulted pressure due to impact of water jet of the prototype can be calculated after taking its simulation scale into consideration, as; the average applied pressure value is 5 t/m^2 for the direct impacted zone (middle third portion) of the protective layer will be applied as direct uninform distributed pressure for different structural system of protective layer, to compare internal forces between these cases.

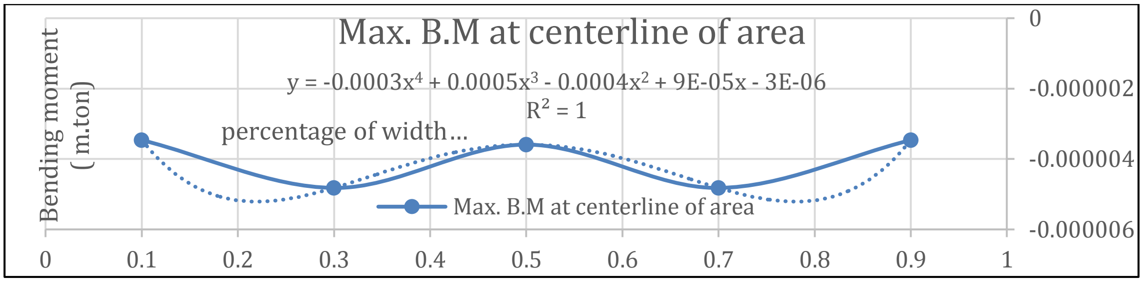

Fig.5 Resulted Pressure Distribution Based on Simulation at Centerline of Protective Area

4.1 Different structural systems for protective layer

The high-tension normal stresses as a result of Positive or negative bending moment, due to relatively high Jet head, on the metal model significantly declared that the material must sustain the effects of these stresses on the protective layer. The protection layer during this study, is represented by raft concrete slab with simulated horizontal dimensions 24 m x 24 m, according to linear scale ratio 1:30, and takes into account the reduction of elasticity due to shrinkage and creep stresses which occurs so modulus of elasticity will be E=140 ton /cm^2. it will be considered as a parameter to be able to compare the outcomes of internal forces between different structural system cases, as it's required for design in prototype. Different structural systems will be assumed hereinafter to be used in design of protective layer' foundations as; grid of piles, different alignments of cutoff walls, or a combination between them. Which, using piles or more than two cutoff walls transversely or longitudinally beneath the protection layer as foundation, will be as a method to reduce the seepage force and uplift pressure acting on the protective layer's lower surface. Like these arrangements will maintain the balance between the impingement force of fallen water Jet and uplift pressure and overcome on the collapsing may occurs on the protection layer and the provide support against erosion tendency into the toe of dam. All assumptions and expected dimensions of the additional component with concrete slab (Cutoff walls), as follow; the cutoff walls can be added to concrete slab and works as a one structural system. The Cutoff walls dimensions may be basically assumed on the minimum cross section allowable to withstand the impact load on concrete slab, it is studied as a preliminary step to cover impacted area from high deformations occurs, which must to be under researches in future. Minimal seepage occurs when the cutoff wall is installed at the heel of the protective layer with a sufficient penetration depth to get the concrete slab firmly attached with the lower layers, so it must be located at the edges of slab as first step in design. The effect of two or more cutoff walls has not been previously considered. There is usually the need for two or more cutoff walls in the design of hydraulic structures; the upstream cutoff wall is used to control the seepage rate and the downstream cutoff wall is used to control the exit hydraulic gradient. Additional cutoff walls can be employed where necessary. Therefore, this study investigates the effect of the cutoff walls as a structural support beneath the concrete slab, the location of cutoff walls, the number of cutoff walls, and their distance from each other to reduce the internal forces inside concrete slab. Therefore, the design of the protective layer (concrete slab) was assumed as an element with dimensions (24 m x 24 m) with minimum thickness equals 10% of span of concrete slab = 2.4 m; it will be under test with the additional cutoff walls to resist the maximum bending moment with a sufficient thickness. So, the element of slab with cutoff walls could use to help designers to cover all length of crest that was actually impacted and still an impossible scaled model in laboratory because of the large length of spillway crest. Also, the cutoff wall thickness is approximately assumed 10% of the cutoff wall depth. also, when the penetration depth of cutoff wall increases, the stresses and displacements of the union area will increase, while the hydraulic gradient and seepage at the union area decrease. So, the appropriate depth is between (0 to 12m) to reduce the vertical and horizontal stresses per meter on impervious layer. According to **(**N. Sartipi,F. Salmasi ,J. Abraham,A. H. Dalir), the depth of cutoff walls in m (d_i); if there are two or three cutoff walls; it will be ranged ($5 \leq d_i \leq 28$), and the distance between cutoff walls (L_i) ($0.50\ d_i \leq L_i \leq 2\ d_i$), In this study the selected depth of cutoff walls equals 6 m. also, the location of cutoff walls or distance between it, is governed by the above equation. So, the minimum distance is selected to be 4 m between each other, and between piles. **Case one:** reinforced concrete raft layer 2.4 m thickness rigidly jointed with two parallel cutoff walls, which are parallel to the dam body and perpendicular on the flow direction about 4 m from edges of foundation. it is assumed to be of cross section 6 m depth, 60 cm width and deepening of 24 m. **Case two:** reinforced concrete raft layer 2.4 m thickness rigidly jointed with three parallel cutoff walls, which are parallel to the dam and perpendicular on the flow direction spaced 4 m, 8 m and 20 m from the toe of dam body. It is presumed that the cross section is the same 6 m depth, 60 cm width and deepening a length of 24 m. **Case three:** reinforced concrete raft layer 2.4 m thickness rigidly jointed with four parallel cutoff walls, which are parallel to the dam and perpendicular on the flow direction spaced 4 m, 8 m, 12 m, and 20 m from the downstream toe of dam foundation. It is presumed that the cross section is the same 6 m depth, 60 cm width and deepening a length of 24 m. **Case four:** reinforced concrete raft layer 2.4 m thickness rigidly jointed with grid of piles with depth of 15 m, 120 cm diameter and spacing 4 m in each direction. **Case five:** using reinforced concrete raft layer with depth 2.4 m, 24 m length in each direction. The model results for different cases are shown in figure (6) as following.

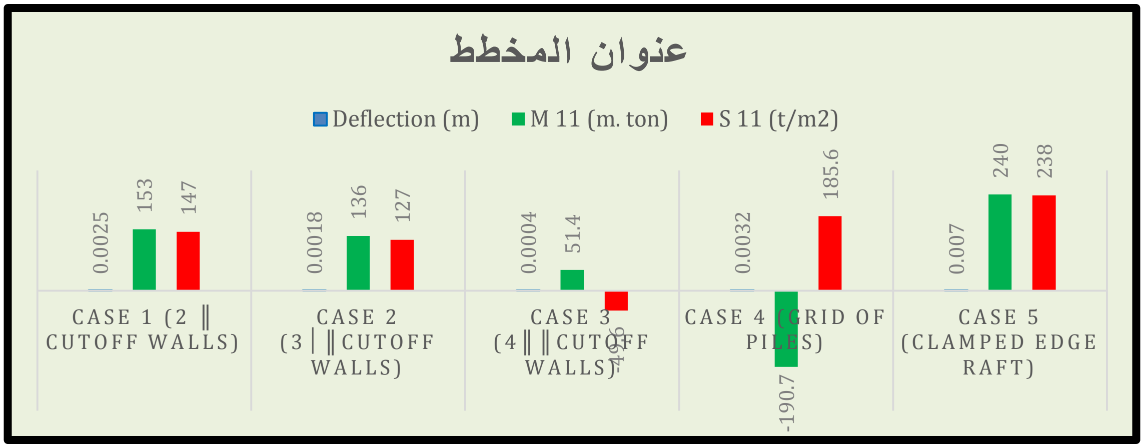

Fig.6 Variation of Deflection and Internal Forces with Structural System of Protective Layer

5. CONCLUSIONS

-From the previous figure (6), it can be drawn the following points: The proposed used system of the strain measurements during this study proved its capability to deduce different internal forces and deformations for the simulated model of the protection layer under any loading conditions for dam' spillway overtopping flow.

-Under the maximum reservoir water level, the maximum resulted normal stress (tension or compression) is higher than the tension strength for any available material. so, it is necessary to look for other high strength material or change the structural system for the protection layer to the suitable system with the resulted stress, in case of dam safety regulations are provided with this max. water level. Otherwise, the reservoir water level must be decreased to that one provides safety for the protection layer.

-Among the different assumed cases of structural systems for protection layer, case No. 3 (4 parallel cutoff walls), reached to the minimum deflection and internal forces within the protection layer. The internal forces are suitable with tension strength of the available contents of reinforced concrete.

-Results of case 1 (using 2 parallel cutoff walls) is almost as that of case 2 (using 3 parallel cutoff walls).

-Results of internal forces and deflection are decreases as the number of cutoff walls are increases, from 2 walls, to three walls, to four walls.

-Case No. 5 (raft without any additional structure system) for protection layer, which its results are not necessary to be provided using available concrete contents or additives. the resulted tension stress is about 25 kg/cm^2, which can be achieved by using concrete F_{cu}= 250 kg/cm^2.

So, in case of completing the GERD main dam with the proposed the highest possible level of water in the reservoir at (643 a.m.s.l), the ordinary concrete raft with marginal side walls are not suitable at all to provide safety for dam body. In this case, the protection layer must be as case No. 3 (using 4 parallel cutoff walls).

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