

# Optimum Design of Low Concrete Gravity Dam on Random Soil Subjected to Earthquake Excitation

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**ABSTRACT:** A 2D (Plain strain) dam – reservoir – foundation interaction is modeled using finite element method by ANSYS to find the optimum design based on the principle of fluid-structure and soil-structure interactions analyses. A low concrete gravity dam founded on random soil has been considered in this research. Al-Teeb Dam site in Amara City is the site of data collections. Earthquake records which are obtained from Badra Station on Amara City are transformed as time–history forces. The optimization process is simulated by ANSYS APDL language programming depending on the available optimization commands. The components of the optimization process are the objective function OBJ is to minimize the volume of the dam, the state variables SVs are stresses, strains, and displacements of dam, and the design variables DVs are the dimensions of the dam. The optimization process should carry out all factors of safety and stability namely "Sequential Unconstrained Minimization Technique" SUMT. In order to specify random soil parameters in the built model, the random field theory is adopted to generate random  $C - \phi$  soil. The results show that the piles or secant piles beneath the dam are necessary to improve bearing capacity factor, scaling the optimum section for economy, and to reduce seepage. It is concluded that the finite element method simulated by ANSYS is efficient with the optimization process.

**KEYWORDS:** Optimization, Gravity dam, Random soil, dam-reservoir-foundation- interaction, seismic effect

## I. INTRODUCTION

Dams are the main types of hydraulic structures that impound water or underground streams for various purposes such as hydropower and hydroelectricity, water storage and irrigations, water flow stabilization, flood prevention, land reclamation, navigation, water channel diversion as well as recreation and aquatic beautification [1].

Several researches were done to carry out the analysis of concrete gravity dam, [2] based on scaled boundary finite – element SBFEM method to develop the governing equations for the analysis of dam–reservoir interaction including the reservoir boundary absorption where the computational cost has been reduced to a great extent.

The response of a dam subjected to dynamic loading, exhibits a combined effect of the interaction among dam, reservoir and foundation systems. The analysis of dams is a complex problem due to dam–reservoir interaction. An important factor in the design of dams in seismic regions is the effect of hydrodynamic pressure exerted on the face of dam as a result of earthquake ground motions. The seismic response of a gravity dam is influenced by its interaction with reservoir. The hydrodynamic pressure acting on dam faces during earthquakes has been recognized as a main loading in the design of dams [3].

Gravity dams form a lifeline of a country economy and their failure will create huge loss of life and properties. Some of dams are in seismically active area. The dynamic analysis of a concrete gravity dam is a reasonably complex problem

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and hence its behavior under seismic actions due to earthquakes has become a matter of immense interest by the researchers[4].

A gravity dam may be constructed either of masonry or of concrete. However, now-a-days with improved methods of construction, quality control and curing, concrete is most commonly used for the construction of gravity dams[5]. Concrete gravity dam is a submerged complicated structure which maintains their stability against design loads from the geometric shape, mass and strength of the concrete [6].

In the latest decade, dynamic analysis of concrete gravity dam in time domain including dam – reservoir – foundation interaction carry out by using finite element software. [7]compared the dynamic analysis of Oued–Fodda Dam (north-western Algeria) and reservoir interaction among some finite element softwares LISA, ALGOR, ANSYS, where the main evaluation and examination were on suitable element selection for reservoir–dam interaction system and also analysis of time history for hydrodynamic pressure of floor joint and horizontal displacement of crest joint.

Since massive concrete gravity dams supporting huge water reservoir, the hydrodynamic forces resulting from the time-dependent stresses will generate at the reservoir water-dam interface. Therefore, the response of Pine Flat Dam under seismic conditions considering hydrodynamic stresses on the upstream face, varying both spatially and temporally, has been reported by[1]. They used GoeStudio 2007 to achieve the modeling. Their study illustrates a significant variation in the estimated seismic response when the hydrodynamic forces are included in the design.

It is worth mentioning that the researches in the optimization design of dam are so little. [8]presented method to define the optimal top width of gravity dam with genetic algorithm (GA). The obtained design should be most economical and the safe in which no tension is developed anywhere in the dam. He got a parametric study to perform well of the optimization goal. Also, he compared the optimization performed to a gradient-based optimization method (classic method) and he found the accuracy of the results within close proximity.

[9]developed a hybrid meta–heuristic optimization method to find the optimal shape of concrete gravity dams including dam–water–foundation rock interaction subjected to earthquake loading. This method is based on a hybrid of gravitational search algorithm (GSA) and particle swarm optimization (PSO), which is called GSA–PSO. They concluded that GSA–PSO shows the improvement in terms of computational efficiency, optimum solution, the number of function measurements and convergence history in the optimization process.

In this paper, the optimum design of concrete gravity dam including dam–reservoir–foundation interaction due to time-dependent load history will be investigated. In other hand , random soil not rock will be considered as a foundation of the dam in the analysis.

In order to achieve the aim above, the paper is organized as follows: Section II describes an ability of finite element software ANSYS to simulate dam-reservoir-foundation problem and all cavering equations related with fluid-structure, soil-structure interactions, and random soil modeling. The optimization problem is distributed in three sections, namely Sections III, VI, and V to describe the optimization process by ANSYS, applied of different loads, and procedure of running the built optimization model, respectively. The results and discussions are presented in Section IV. Finally, Section IIV presents conclusions.

### II. FINITE ELEMENT MODELING OF DAM-RESERVOIR-FOUNDATION BY ANSYS

The dam–reservoir–foundation interaction based on principle of fluid–structure and soil–structure interaction [10]. The procedure for the dynamic analysis of dam–reservoir–foundation interaction based on the coupling equation of fluid–structure–soil interaction.

#### a- Acoustic fluid fundamentals

In acoustical fluid-structure interaction problems, the structural dynamic equation needs to be considered along with the Navier-Stokes equations of fluid momentum and the flow continuity equation. The acoustic wave equation given below is based on the following [11]:

1. The fluid is compressible (density change due to pressure variations)
2. The fluid is inviscid (no viscous dissipation).
3. There is no mean flow of the fluid.
4. The mean density and pressure are uniform throughout the fluid.

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$$\frac{1}{c^2} \frac{\partial^2 P}{\partial t^2} - \nabla^2 P = 0 \dots \dots \dots (1)$$

Where  $C$  is the speed of pressure wave;  $P$  is the acoustic hydrodynamic pressure;  $t$  is time and  $\nabla^2$  is the two – dimensional Laplace operator.

To solve wave equation , Fig. 1 show dam-reservoir system with some boundary conditions may be imposed on the fluid domain as follows[12]:-

$$P = 0 \text{ at } y = h, \quad u_y = 0 \text{ at } y = 0, \quad \ddot{u}_x = ag \cos\left(\frac{2\pi t}{T}\right) \text{ at } x = 0, \text{ and } P = 0 \text{ when } x \rightarrow \infty$$



Fig.1 Rigid dam-infinite reservoir system

The hydrodynamic pressure for the above mentioned boundary conditions is as follows:

$$p(x, y, t) = \frac{8a\rho gh_w}{\pi^2} \cos\left(\frac{2\pi t}{T}\right) \sum_{n=1,3,5,\dots}^{\infty} \left( \frac{1}{n^2 C_n} e^{-q_n} \sin\left(\frac{n\pi(h_w-y)}{2h_w}\right) \right) \dots \dots \dots (2)$$

Where:

$$q_n = \frac{n\pi C_n x}{2h_w} \dots \dots \dots (3)$$

$$C_n = \sqrt{1 - \frac{16\rho h_w^2}{n^2 K T^2}} = \sqrt{1 - \frac{16h_w^2}{n^2 C^2 T^2}} \dots \dots \dots (4)$$

The maximum pressure at  $x = 0$  occurs when  $t=0, T, 2T, \dots$  and can be written as:

$$p_{max}(x, y, t) = \frac{8a\rho gh_w}{\pi^2} \sum_{n=1,3,5,\dots}^{\infty} \left( \frac{1}{n^2 C_n} e^{-q_n} \sin\left(\frac{n\pi}{2}\right) \right) \dots \dots \dots (5)$$

While the pressure at  $y = 0$  also occurs when  $t=0, T, 2T, \dots$  and can be written as:

$$p_{max}(x, y, t) = \frac{8a\rho gh_w}{\pi^2} \sum_{n=1,3,5,\dots}^{\infty} \left( \frac{1}{n^2 C_n} \sin\left(\frac{n\pi(h_w-y)}{2h_w}\right) \right) \dots \dots \dots (6)$$

In this research  $n = 1$  and  $T = 2\pi$  have been taken.

The maximum pressure at a given time on the upstream face ( $x = 0$ ) happens at the bottom of the reservoir. The shape of the pressure diagram is such that the curve has a horizontal tangent at the top and a vertical tangent at the bottom.

The coupled fluid–structure interaction could be written as:

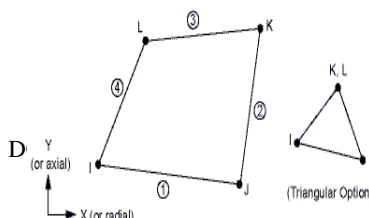
$$\begin{bmatrix} [M_e] & [0] \\ [[M^{fs}] & [M_e^p] \end{bmatrix} \begin{Bmatrix} \{\ddot{U}_e\} \\ \{\ddot{P}_e\} \end{Bmatrix} + \begin{bmatrix} [C_e] & [0] \\ [0] & [C_e^p] \end{bmatrix} \begin{Bmatrix} \{\dot{U}_e\} \\ \{\dot{P}_e\} \end{Bmatrix} + \begin{bmatrix} [K_e] & [K^{fs}] \\ [0] & [K_e^p] \end{bmatrix} \begin{Bmatrix} \{U_e\} \\ \{P_e\} \end{Bmatrix} = \begin{Bmatrix} \{F_e\} \\ \{0\} \end{Bmatrix} \dots \dots (7)$$

It could be written easy as [13]:

$$M_f \ddot{P}_e + C_f \dot{P}_e + K_f P_e + \rho_\omega Q^T (\ddot{u}_e + \ddot{u}_g) = 0 \dots \dots \dots (8)$$

Where  $M_f$ ,  $C_f$ , and  $K_f$  are the fluid mass, damping and stiffness matrices, respectively,  $P_e$ ,  $\ddot{u}_e$  and  $\ddot{u}_g$  are the nodal pressure, relative nodal acceleration and nodal ground acceleration vectors, respectively. The term  $\rho_\omega Q^T$  is also often referred to as coupling matrix.

In ANSYS (APDL language programming), a four-node FLUID 29 element shown in Fig. 2, is used to discretize the coupled fluid–structure interaction represented by Eq. (7).



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Fig. 2 FLUID29Element Geometry[11]

**b- Soil-Structure interaction**

Modeling and analysis of dynamic soil structure interaction during earthquakes have gone through direct method (Global Method) where the soil and structure are included within the same model and analyzed in a single step [10]. The discretized structural dynamic equation including the dam and foundation subject to ground motion can be formulated using the finite–element approach as [13]:

$$M_s \ddot{u}_e + C_s \dot{u}_e + K_s u_e = -M_s \ddot{u}_g + Q p_e \dots\dots\dots(9)$$

Where  $M_s$ ,  $C_s$ , and  $K_s$  are the structural mass, damping and stiffness matrices, respectively,  $u_e$  is the nodal displacement vector with respect to ground and the term  $Q p_e$  represents the nodal force vector associated with the hydrodynamic pressure produced by the reservoir.

A four–nodes PLANE 42 element (structural 2D solids) plain strain, shown in Fig. 3 which available in ANSYS is used for both dam body and soil foundation modeling.

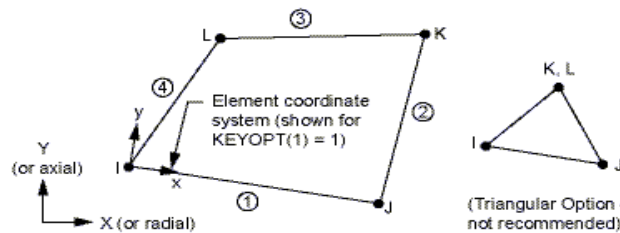


Fig. 3 PLANE42 Element Geometry [11]

**c- The Coupled Fluid-Structure-soilEquation**

The complete finite–element discretized equations for dam-water-foundation interaction problem could be written from both Eqs. (8 and 9) in an assembled form as:

$$\begin{bmatrix} M_s & 0 \\ M_{fs} & M_f \end{bmatrix} \begin{pmatrix} \ddot{U}_e \\ \ddot{P}_e \end{pmatrix} + \begin{bmatrix} C_s & 0 \\ 0 & C_f \end{bmatrix} \begin{pmatrix} \dot{U}_e \\ \dot{P}_e \end{pmatrix} + \begin{bmatrix} K_s & K_{fs} \\ 0 & K_f \end{bmatrix} \begin{pmatrix} U_e \\ P_e \end{pmatrix} = \begin{pmatrix} -M_s \ddot{U}_g \\ -M_{fs} \ddot{U}_g \end{pmatrix} \dots\dots\dots(10)$$

Where  $K_{fs} = -Q$  and  $-M_{fs} = \rho_w Q_T$

Eq.(10) expresses a second order linear differential equation having unsymmetrical matrices and may be solved by means of direct integration methods. In general, the dynamic equilibrium equation of systems modeled in finite elements can be expressed as:

$$M_c \ddot{u}_c + C_c \dot{u}_c + K_c u_c = F(t) \dots\dots\dots(11)$$

Where  $M_c$ ,  $C_c$ ,  $K_c$ , and  $F(t)$  are the structural mass, damping, stiffness matrices and dynamic load vector, respectively.

**d- Model of Random Soil Properties**

The properties of random soil ( $C$  and  $\phi$ ) could be modeled using random field theory [14]. The soil cohesion,  $C$ , is assumed to be lognormally distributed with mean  $\mu_c$ , standard deviation  $\sigma_c$ , and spatial correlation length  $\theta_{lnc}$ . The lognormal distribution is selected because it is commonly used to represent non-negative soil properties and since it has a simple relationship with the normal. A lognormally distributed random field is obtained from a normally distributed random field,  $G_{lnc}(\tilde{x})$ , having zero mean, unit variance, and spatial correlation length  $\theta_{lnc}$  through the transformation

$$c(\tilde{x}) = \exp\{\mu_{lnc} + \sigma_{lnc} G_{lnc}(\tilde{x})\} \dots\dots\dots(12)$$

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Where  $\tilde{x}$  is the spatial position at which  $C$  is desired. The parameters  $\mu_{lnc}$  and  $\sigma_{lnc}$  are obtained from the specified cohesion mean and variance using the lognormal distribution transformations

$$\sigma_{lnc}^2 = \ln \left( 1 + \frac{\sigma_c^2}{\mu_c^2} \right) \dots\dots\dots(13)$$

$$\mu_{lnc} = \ln \mu_c - \frac{1}{2} \sigma_{lnc}^2 \dots\dots\dots(14)$$

The friction angle,  $\phi$  is assumed to be bounded both above and below, so neither normal nor lognormal distributions are appropriate. A beta distribution is often used for bounded random variables. Unfortunately, a beta distributed random field has a very complex joint distribution and simulation is cumbersome and numerically difficult. To keep things simple, a bounded distribution is selected which resembles a beta distribution but which arises as a simple transformation of a standard normal random field,  $G_\phi(\tilde{x})$  according to

$$\phi_{min} + \frac{1}{2}(\phi_{max} - \phi_{min}) \left\{ 1 + \tanh \left( \frac{s G_\phi(\tilde{x})}{2\pi} \right) \right\} \dots\dots\dots(15)$$

Where  $\phi_{min}$  and  $\phi_{max}$  are the minimum and maximum friction angles, respectively, and  $s$  is a scale factor which governs the friction angle variability between its bounds. Fig.4 shows how the distribution of  $\phi$  (normalized to the interval [0,1]) changes as  $s$  changes, going from an almost uniform distribution at  $s = 5$  to a very normal looking distribution for smaller  $s$ . In all cases, the distribution is symmetric so that the midpoint between  $\phi_{min}$  and  $\phi_{max}$  is the mean. Values of  $s$  greater than about 5 lead to a U-shape distribution (higher at the boundaries), which is not deemed realistic. Thus varying  $s$  between about 0.1 and 5.0 leads to wide range in the stochastic behavior of  $\phi$ . In this research  $s$  is taken equal to 5.0

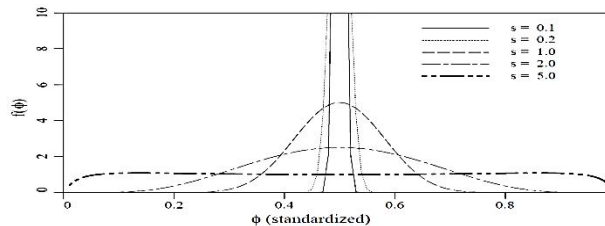


Fig.4 Bounded distribution of friction angle normalized to the interval [0,1] [14]

In this research the material properties of dam, reservoir, and foundation could be summarized in Table 1a, b, and c where represent the properties of concrete, water, and soil foundation, respectively:

Table 1a Dam Material properties

Properties Item	Density $\rho$ (Kg/m <sup>3</sup> )	Modulus of elasticity $E_s$ (Gpa)	Poisson's ratio $\nu$	Note
concrete	2400	25	0.2	Homogeneous, isotropic, and elastic properties of mass concrete are assumed.

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Table 1b Reservoir Material properties

Properties Item	Density $\rho$ (Kg/m <sup>3</sup> )	Bulk Modulus of elasticity $E_s$ (Gpa)	Sonic velocity (m/s)	Note
water	1000	2.02	1440	Inviscid and compressible.

Table 1c Foundation Material properties [15]

Properties Item	Density $\rho$ (Kg/m <sup>3</sup> )	Modulus of elasticity $E_s$ (Mpa)	Poisson's ratio $\nu$	Cohesive of soil (C) (Kpa)	Angle of internal friction $\phi^\circ$	Note
soil	1599	250	0.3	21.34	32.15	Tacked as a Random Soil

### III. MODELING OF OPTIMIZATION PROBLEM BY ANSYS

The scheme of the practical section of dam with Somerfield-type radiation boundary condition of reservoir and foundation that will be optimized by ANSYS can be given in Fig. 5 [16],[9] and [17].

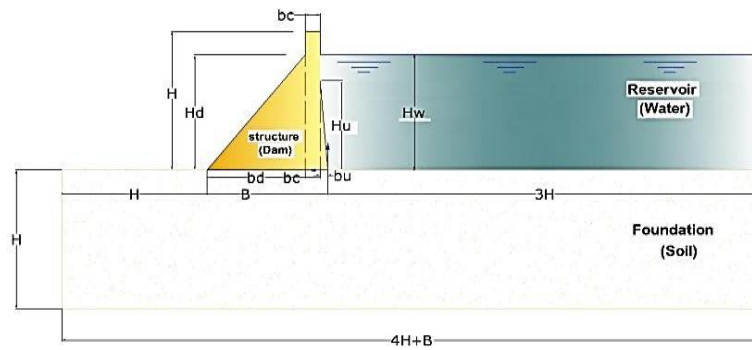


Fig. 5 Scheme of optimized dam model by ANSYS

Where  $H$  is the total height,  $H_w$  is the water height in the reservoir,  $H_u$  is the upstream dam face slope height,  $H_d$  is the downstream dam back slope height,  $B$  is the total dam base width,  $b_u$  is the upstream dam face slope width,  $b_c$  is the dam crest width, and  $b_d$  is the downstream dam back slope width.

The optimization problem is to minimize the cost, therefore the adopted objective function implies on minimizing the volume of dam, and this implies the minimization of the dam section area, as given in Eq.(12)

$$\text{Min } A = \frac{b_u \cdot H_u}{2} + b_c \cdot H + \frac{b_d \cdot H_d}{2} \dots \dots \dots (12)$$

Where each one of  $b_u, b_c, b_d, H, H_w, H_u, H_d$  is a design variable, while stress, strain, and displacement of the dam are considered as state variables.

The factors of safety that should be realized for stability and safety after optimization process will be finished could be given as [18], [19], and [20]:

1. Against sliding,  $FOS_S = \frac{\text{Stabilizing Force}}{\text{Driving Force}} = \frac{\sum F_V}{\sum F_H} > 1.5 \dots \dots \dots (13)$

2. Against Overturning,  $FOS_{ovt} = \frac{\text{Resistance Moment}}{\text{Overturning Moment}} = \frac{\sum M_R}{\sum M_o} > 2 \dots \dots \dots (14)$

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$$3. \text{ Bearing capacity of soil, } FOS_{BC} = \frac{q_{ult}}{q_{max}} = \frac{CN_c + qN_q + \frac{1}{2}\gamma \bar{B} N_\gamma}{\frac{\Sigma V}{B} \left(1 \mp \frac{6e}{B}\right)} \geq 3 \dots \dots \dots (15)$$

Where  $CN_c + qN_q + \frac{1}{2}\gamma \bar{B} N_\gamma$  is Terzaghi's bearing capacity, and  $\bar{B} = B - e$  and  $e$  is the eccentricity.

If the soil is replaced by a pile to bear the coming loads, in this case

$$q_{ult} = q_p + q_f \dots \dots \dots (15-a)$$

Where:  $q_{ult}$  = ultimate bearing capacity of pile (KN/m<sup>2</sup>),  $q_p$  = theoretical bearing capacity for tip of foundation, or end bearing (KN/m<sup>2</sup>),  $q_f$  = theoretical bearing capacity due to shaft friction, or adhesion between foundation shaft and soil (KN/m<sup>2</sup>). Also,

$$q_p = \frac{Q_p}{A_p}, \quad q_p = \gamma h_p N_q \text{ (for cohesionless and silt soil) }, \quad q_p = 9C \text{ (for cohesive soils) } \dots (15-b)$$

Where:  $Q_p$  is the theoretical bearing capacity for tip of foundation, or end bearing (KN),  $A_p$  is the cross-section area (m<sup>2</sup>), in which  $A_p = \frac{\pi}{4} \times D^2$ ,  $D$  is the diameter of a pile (m),  $\gamma$  is the unit weight of soil (KN/m<sup>3</sup>),  $h_p$  is the effective depth of pile (m),  $N_q$  is the bearing capacity factor for soil,  $C$  is the cohesion of soil (KN/m<sup>2</sup>). While,

$$q_f = \frac{Q_f}{A_f} \dots \dots \dots (15-c)$$

Where:  $Q_f$  is the theoretical bearing capacity due to shaft friction, or adhesion between foundation shaft and soil (KN),  $A_f$  is the surface area of a pile that the adhesion with a soil may be happened (m<sup>2</sup>), in which  $A_f = P \times L$ ,  $P$  is the perimeter of pile cross-section (m) and  $P = \pi D$  for circular pile or  $P = 4D$  for square pile;  $L$  is the effective length of pile (m). In other hand,  $q_f = KS \tan \lambda$  for cohesion less soils (KN/m<sup>2</sup>),  $q_f = C_A + KS \tan \lambda$  for soil silts (KN/m<sup>2</sup>), and  $q_f = a S_u$  for cohesive soils (KN/m<sup>2</sup>). Where:  $S_u$  is a cohesion factor (KN/m<sup>2</sup>), Also:

$$a = 1 - 0.1(S_{uc})^2 \text{ for } S_{uc} < 48 \text{ KN/m}^2 \text{ cohesion factor,}$$

$$a = \frac{1}{S_{uc}} [0.9 + 0.3(S_{uc} - 1)] \text{ for } S_{uc} > 48 \text{ KN/m}^2$$

$$S_{uc} = 2C = \text{unconfined compressive strength (KN/m}^2)$$

$C_A$  is the adhesion, so  $C_A = C$  for rough concrete, rusty steel, corrugated metal; while  $0.8C < C_A < C$  for smooth concrete; in other hand, it may be  $0.5C < C_A < 0.9C$  for clean steel.

$\lambda$  is the external friction angle of soil and wall contact (degree)

$S = gD$  it is the effective overburden pressure (KN/m<sup>2</sup>)

$K$  = lateral earth pressure coefficient for piles.

$$4. \text{ Exit gradient (seepage), } FOS_{EG} = \frac{I_c}{I_e} = \frac{G-1/1+e}{H/B} \geq 1 \dots \dots \dots (16)$$

Where  $G$  is the specific gravity of soil and is equal to 2.7, and  $e$  is the void ratio and equal to 0.60 [15]

5. Shear stress of soil should don't exceed allowable shear stress of soil. Therefore, in this paper

$$\tau = q_{max} * \tan \beta \dots \dots \dots (17)$$

Where  $\beta$  is the inclination of upstream face of dam with vertical axes, and allowable shear stress of soil is equal to 0.79 Mpa.

6. The crack section should be avoided. The resultant force must pass through middle third of the dam width i.e.  $e < \frac{B}{6}$

7. Liquefaction process on the foundation when the soil is sand and saturated must be prevented.

In this paper, the selected site of the optimized dam may be on east region of Iraq with Iran boundary. The topographic of this region is shown in Fig.6.

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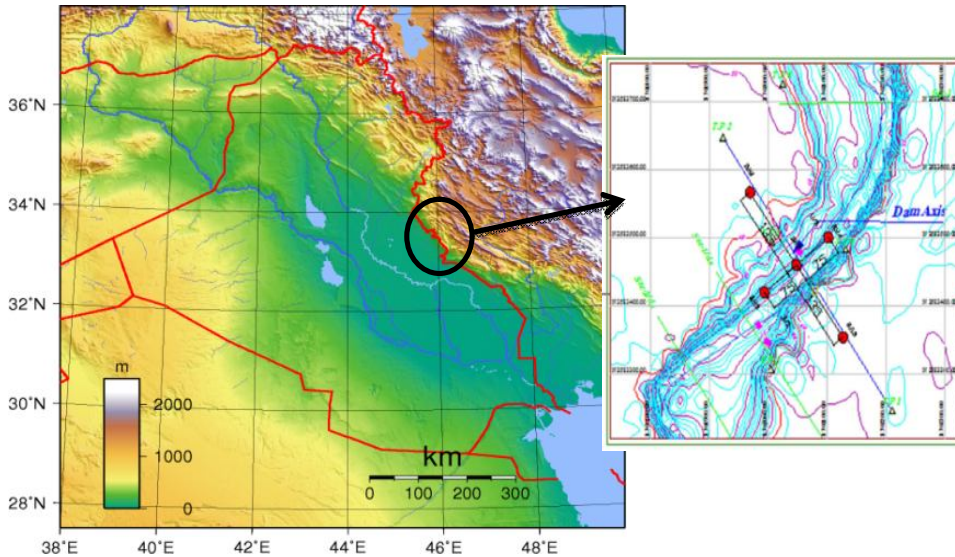


Fig. 6 Topographic of Iraq and site region of study [15]

Therefore, the height of water in the reservoir  $H_w$  for this region is found to be 15m and the practical section that is used roughly in design could be given according to the variables ratio in Table 2[21].

Table 2 Variables Ratio After[21]

Variables Ratio	Range	Values Adopted		
		Max.	Ava.	Min.
$B/H$	0.75-0.85	0.75	0.80	0.85
$H_w/H$	0.85-0.95	0.85	0.90	0.95
$H_u/H$	0.50-0.70	0.50	0.60	0.70
$H_d/H$	0.80-0.90	0.80	0.85	0.90
$b_u/B$	0.063-0.088	0.063	0.0755	0.088
$b_c/B$	0.093-0.15	0.093	0.1215	0.15
$b_d/B$	0.788-0.84	0.788	0.80	0.84

The average variables ratio in Table (2) is tacked as an initial value for the model in optimization problem. Then, the design variables (DV<sub>s</sub>) are set to the following constraints:

$$\begin{aligned}
 (H_{min} = H_w/0.95) \leq H \leq (H_{max} = H_w/0.85) \\
 (H_{u min} = 0.50H) \leq H_u \leq (H_{u max} = 0.70H) \\
 (H_{d min} = 0.80H) \leq H_d \leq (H_{d max} = 0.90H) \\
 (b_{u min} = 0.063B) \leq b_u \leq (b_{u max} = 0.088B) \dots \dots \dots (18) \\
 (b_{c min} = 0.093B) \leq b_c \leq (b_{c max} = 0.15B) \\
 (b_{d min} = 0.84B) \leq b_d \leq (b_{d max} = 0.788B) \\
 B = b_u + b_c + b_d
 \end{aligned}$$



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## IV. LOADS OF OPTIMIZATION PROBLEM

In the design of concrete gravity dam, it is essential to determine the loads required in the stability and stress analyses which are weight of dam , water pressure (static and dynamic) ,uplift forces , earthquake forces , and seismic load .The forces of wind waves, silt, and Ice are ignored in this research.Fig. 7 shows these forces . The seismic load is concerned according to the records of earthquake stations placed on the region of study. Therefore, the records of Badra station on Amara City [22], is transformed as time-history force as shown in Fig.8

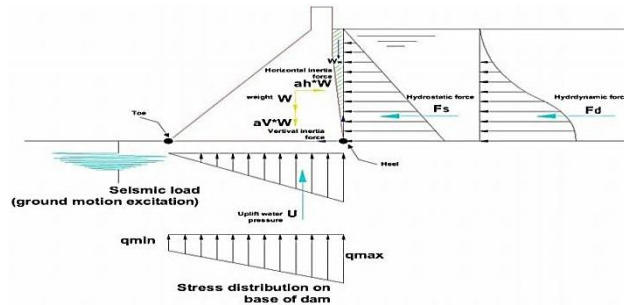


Fig. 7 Loads on optimization dam model

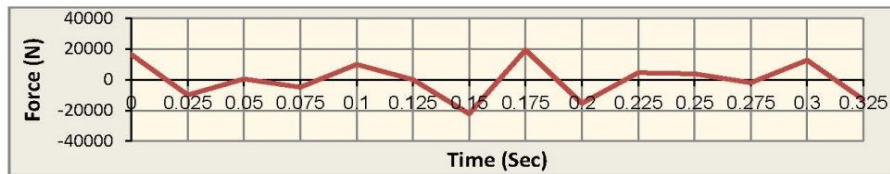


Fig. 8 Earthquake time-history force

## V. RUNNING OF OPTIMIZATION PROBLEM BY ANSYS

The scheme of ANSYS APDL steps of the modeling of the optimization process including dam–reservoir–foundation interaction under seismic effect could be shown in Fig. 9

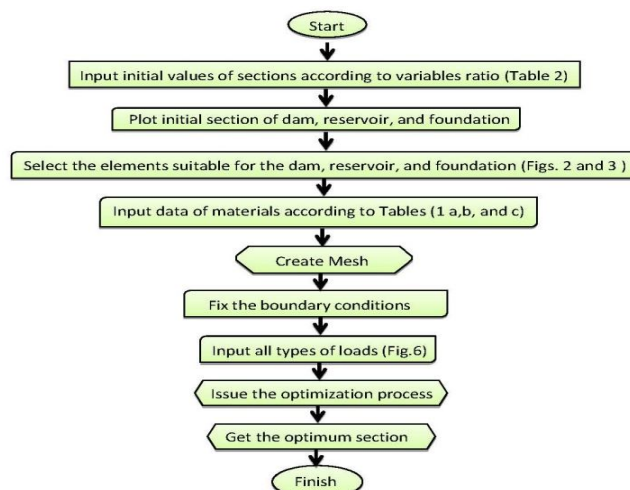


Fig 9 Scheme of ANSYS APDL modeling

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The 2D finite element model of the optimization problem using ANSYS APDL is given in Fig. 10. It is shown from the figure the discretization of model and the interaction among dam-reservoir-foundation interaction, the fixed boundary, and applied loads.

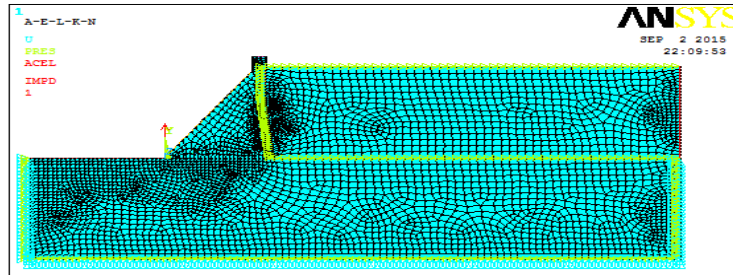


Fig. 10 Finite element discretization model of optimization process

In this research, the individual-accumulative optimization process is developed. This process aims to carry out each constraint individually through the optimization process. Therefore, the modified sections are provided to reach the final optimum section.

## VI. RESULTS AND DISCUSSIONS

After the optimization process is conducted, the initial, modified, and optimum sections of dam are given in Table 3. As noticed in this table, the feasible sections and modified feasible sections represent the feasible or alternative sections obtained from the optimization process.

Table 3 Initial and Optimum Dimensions

Sections	Dimensions (m)									Notes
	$h_w$	H	$h_d$	$h_u$	B	$b_u$	$b_c$	$b_d$	Area	
Initial section	15.00	16.66	14.16	10.00	13.34	1.06	1.62	10.66	107.80	Fail section.
Feasible section 1	14.12	16.92	14.05	10.92	14.08	0.90	1.71	11.47	114.40	Pass section except of sliding, overturning, and bearing capacity factor.
Feasible section 2	14.12	16.92	14.05	10.12	18.36	1.17	2.23	14.95	149.21	Pass section except of bearing capacity factor.
Modified Feasible section 2	14.12	16.92	14.05	10.12	18.36	1.17	2.23	14.95	149.21	Pass section with piles Case 1
Modified Feasible section 1 (Optimal)	14.12	16.92	14.05	10.92	14.08	0.90	1.71	11.47	114.40	Pass section with piles Case 2

Also, the optimal variables ratio that obtained after optimization process is finished are given in Table 4. These ratios will be depended on the design of dam to give more economic section.

Table 4 Variables Ratio

Variables Ratio	$h_w/H$	B/H	$h_u/H$	$h_d/H$	$b_u/B$	$b_c/B$	$b_d/B$
Initial	0.90	0.80	0.60	0.85	0.0799	0.1214	0.80
Optimal	0.83	0.72	0.83	0.65	0.0639	0.1216	0.81

In other hand, R.C. piles are considered in the optimization process in two cases. The first one; case-1; the piles serve as a barrier to reduce seepage beneath a dam, and as a structure to improve bearing capacity that sufficient to bear the dam. The optimum details of number, type and dimensions of these piles are given Table 5.

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Table 5 Details of piles for case-1

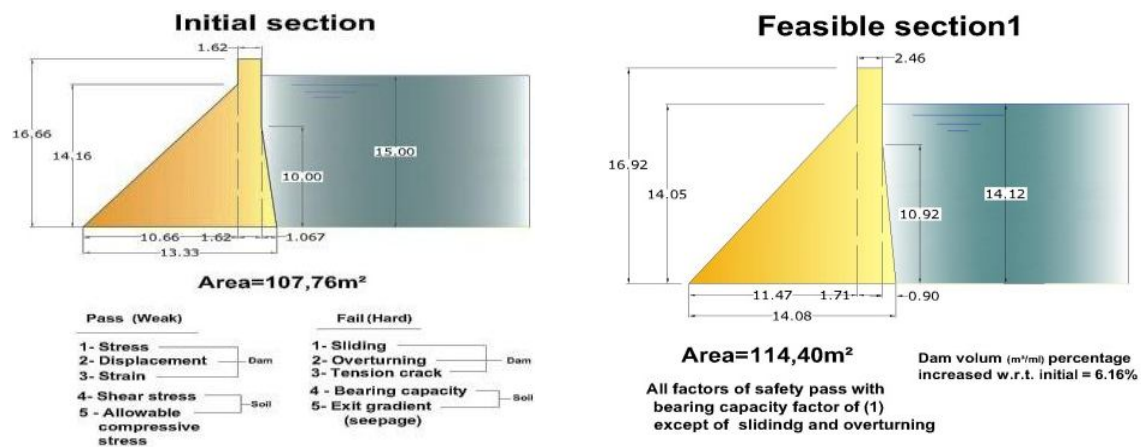
D <sub>P</sub> (m)	l <sub>P</sub> (m)	No.	l <sub>P</sub> / D <sub>P</sub>	l <sub>P</sub> / B	Type of pile	FOS)bearing capacity
1.00	2.53	1	2.53	0.13804	Secant Pile	3.000
0.90	2.85	1	3.166	0.15518	Secant Pile	3.000
0.80	3.22	1	4.025	0.17550	Secant Pile	3.000
0.70	3.67	1	5.24	0.2000	Secant Pile	3.000
0.60	4.22	1	7.033	0.23010	Secant Pile	3.000
0.50	4.92	1	9.84	0.26820	Secant Pile	3.000
0.40	5.85	1	14.625	0.31870	Secant Pile	3.000
0.30	7.17	1	23.90	0.39059	Separate Pile	3.000

The second; case-2; where the piles serve as well mentioned in case-1 as a key beneath a dam to overcome both sliding and overturning. All details related these piles which may be as separate piles with space in between them or may be as secant piles are given in Table 6.

Table 6 Details of piles for case-2

D <sub>P</sub> (m)	l <sub>P</sub> (m)	No.	l <sub>P</sub> / D <sub>P</sub>	l <sub>P</sub> / B	Type of pile	FOS)bearing capacity
1.00	3.40	1	3.40	0.24156	Secant Pile	3.000
0.90	3.79	1	4.21	0.26923	Secant Pile	3.000
0.80	4.25	1	5.31	0.30182	Secant Pile	3.000
0.70	4.80	1	6.85	0.34098	Secant Pile	3.000
0.60	5.47	1	9.11	0.38857	Secant Pile	3.000
0.50	6.31	1	12.62	0.44824	Secant Pile	3.000
0.40	7.43	1	18.57	0.52781	Secant Pile	3.000
0.30	9.01	1	30.03	0.6400	Separate Pile	3.000

The individual - accumulative optimization process suggested by authors to include each constraint individually in the optimization process. This lead to construct multiple loops in order to get the optimal solution in final of optimization process. Consequently, the initial, feasible, modified, and optimal sections details could be shown in Fig. 11.



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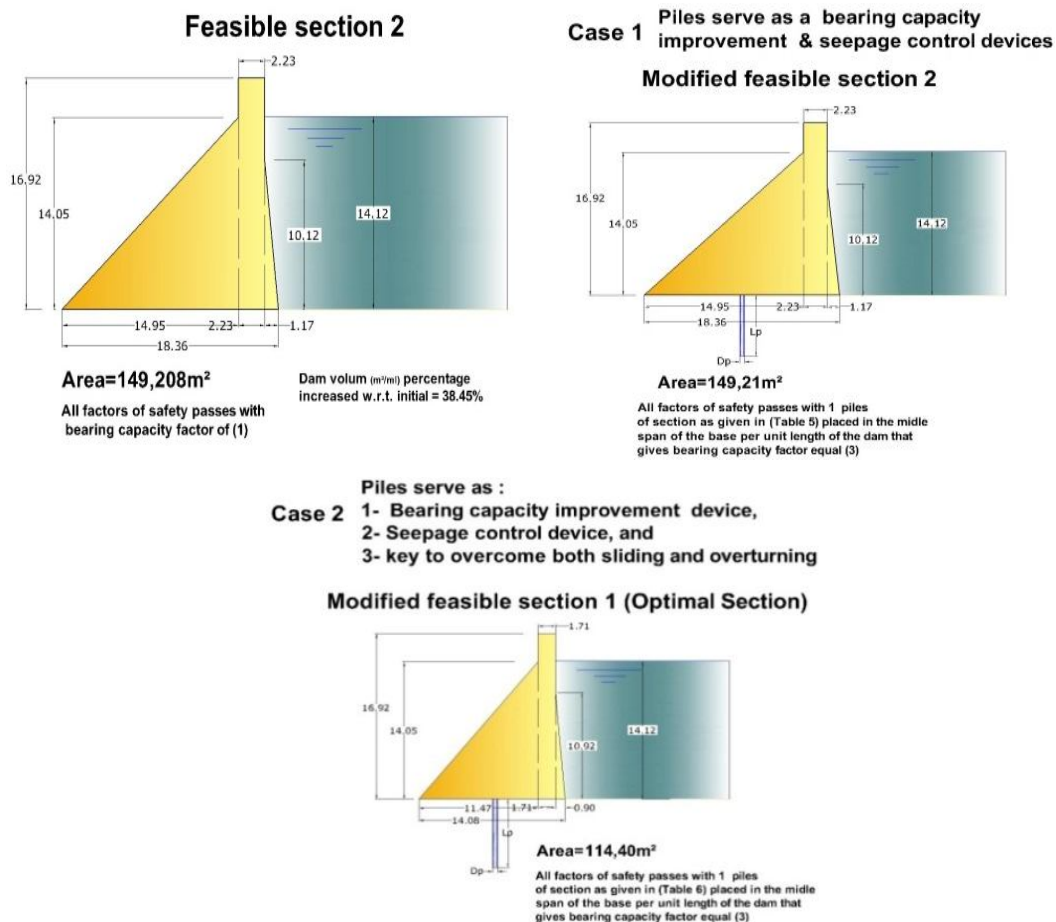


Fig.11 Optimization Process

## VII. CONCLUSIONS

The section of concrete gravity dam should be chosen in such a way that it is the most economic section and satisfies all the conditions and requirements of stability and safety. It is found from the results the following:-

1. The ANSYS APDL is efficient tool to simulate dam-reservoir-foundation interaction problem and optimization process.
2. The optimization process is suitable technique to get the optimum section.
3. The individual - accumulative optimization process gives reasonable procedure to carry out the factors of safety individually.
4. The initial section of dam that is provided according to initial variables ratios is not representing the optimum section because of falling of factors of safeties.
5. The optimum section of a dam on random soil is obtained with piles serve in the same time as a bearing capacity improvement device, a seepage control device, and a key underneath a dam in order to overcome both sliding and overturning.

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