

Research Article

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Seismic performance evaluation of concrete buttress dam (Dynamic linear analysis)

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Abstract: Due to a variety of reasons, including the water retained inside and the intrinsic weight of the dam itself, dam constructions have the ability to move both horizontally and vertically. If these displacements exceed a crucial limit, a dam's structural integrity is jeopardized. Concrete buttress dams in particular may be susceptible to high-frequency vibrations because of their slender structure, especially when the flow of water is involved. The Khassa Chi Dam, which is located northeast of Kirkuk City, is the subject of this study's attempt to offer an alternative since the constructed dam is an embankment dam. In this research, a concrete buttress dam design was studied as an alternative dam to the constructed one. Such designs exemplify one form of gravity dams widely implemented on diverse soil types. Finite element model (FEM) was employed to simulate the behavior of the dam. The simulation utilized DIANA FEA, which relies on governing equations. There are several steps involved in developing an accurate FEM that faithfully simulates the actual behavior of a dam and predicts its future responses. The model is evaluated in later analyses in terms of stress and displacement. In this context, RSA was conducted on the modeled buttress dam. The outcome of the displacement analysis of the buttress dam exhibited its safety across all load combinations after undergoing linear dynamic analysis. This analysis included Eigenvalue Analysis and RSA. The response remained low at seismic frequencies below 3 Hz, and the extent of displacement correlated with the frequency values.

Keywords: concrete buttress dam, Khassa chi, DIANA FEA, Eigenvalue analysis, response spectrum analysis

Abbreviation

RSA	response spectrum analysis
RHA	response history analysis
FEA	Finite Element Analysis
DIANA FEA	DIsplacement ANALyser
IRAM	implicitly restarted Arnoldi method

1 Introduction

A buttress dam is a particular type of gravity dam distinguished by its relatively thin profile. This kind of dam generally consists of a sloping face that holds the water that is being held back. It is supported by buttress walls that are carefully placed along the dam's axis: a configuration designed to maximize the use of materials. The mass of the structure and the vertical force the reservoir's water exerts against the sloped front surface are both important factors in the design's overall stability. This method offers a resource-efficient design in contrast to the traditional homogenous gravity dam [1]. Various dam categories are defined by this fundamental idea based on how the water-retaining facade is set up. These classifications include the massive head type, multiple arch type, Ambursen/flat slab type, and slender buttress dam type.

Buttress walls support the flat reinforced concrete slab that is used to build flat slab buttress dams. On top of the flat slab, these dams have a constant upstream face slope. The Norwegian-American engineer Nils F. Ambursen filed a patent for this design, which was extensively used in the first half of the twentieth century [1]. It is established that each facility possesses a distinct design lifespan determined by factors such as conditions, building type, and the facility's intended purpose, whether industrial, educational, or hydraulic. Post the designated design lifespan,

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alternative strategies are explored to ensure the continued functionality of these facilities and the services they provide.

Abbasiverki *et al.* [2] in their study investigated the nonlinear response of concrete buttress dams to high-frequency vibrations. This study considers how topography affects the consequences of amplification. The topographic amplification of seismic waves on the canyon surface is what causes the uneven stress distribution among the monoliths. When compared to low-frequency excitation, this behavior is more obvious with high-frequency excitation. Vibrations introduced across the stream cause joints to separate and the stress inside the dam to increase. As a result, the dam's security is jeopardized, especially when high-frequency stimulation is present. Marence *et al.* [3] studied the evaluation of the reliability of a gravity dam block through the integration of a Directional Adaptive Response Surface full probabilistic method and 3D coupled flow-stress finite element analysis using the DIANA FEA software. The study demonstrated that the response of this particular dam is significantly affected by the topographic amplification of high-frequency excitations. The safety of the dam was compromised as a result of cross-stream vibrations, which led to the opening of joints and an escalation of stresses. The utilization of the foundation modeling approach had a noteworthy influence on the computed response of the dam. The method which assumes zero mass yielded outcomes that were deemed untrustworthy, particularly in instances of excitations with high frequencies. The utilization of the analytical method for free-field modeling resulted in joint openings that were deemed unreliable. Hence, it is advisable to employ a precise methodology for the modeling of foundations, particularly when nonlinearity is taken into account. Løkke and Chopra [4] offered an assessment of the precision of the response spectrum analysis (RSA) method. This is achieved by conducting a comparative analysis between the outcomes of the RSA technique and those derived from the response history analysis (RHA) of the dam. The latter is modeled as a finite-element system that incorporates the interaction between the dam, water, and foundation. The RHA procedure was utilized to determine the earthquake response of a real dam to a set of 58 ground motions. These motions were carefully chosen and adjusted to align with a target spectrum that was established through a probabilistic seismic hazard analysis for the specific location of the dam. The benchmark result was obtained by determining the median of the peak responses of the dam to 58 ground motions. The estimation of the peak response was conducted through the RSA procedure, which was directly derived from the median response spectrum. The findings indicate that the RSA method is capable of providing stress

estimates with an acceptable level of accuracy for the initial stage of dam design and for assessing the safety of already existing dams. The RSA procedure has achieved a noteworthy level of accuracy, particularly given the intricate impacts of dam–water–foundation interaction and reservoir bottom absorption on the system's dynamics, as well as the numerous approximations required to formulate the procedure. Jonker *et al.* [5] in their research examined the structural analyses of the Clover Dam made during the safety review. 3-D finite element studies for thermal, static, and seismic loads were used in the structural evaluations. The review's conclusion was that the dam's gravity and buttress components don't adhere to modern design requirements. The operating limitations and the limited access to the places inside the dam where remedial work was needed made it difficult to create workable repair alternatives. In this research, an alternative buttress dam stability was investigated by testing the linear behavior of the proposed dam under different frequency seismic excitations.

The city of Kirkuk is one of the northern cities dependent on the Tigris River for its water supply because of the water imbalance during the seasons of the year, the resort has resorted to the construction of a seasonal dam on the river Khassa Chai. The purpose of constructing the dam: Providing part of the drinking water needs of the city of Kirkuk, strengthening groundwater, activating tourism in the region, and agriculture and animal husbandry.

The site of the dam at Kirkuk Governorate/on a private valley 15 km north-east of the city of Kirkuk, since the dams are one of the infrastructures with a definite age, this research suggests the construction of a concrete buttress dam on Khassa Chi. The research aims to make a comparison between the construction of an old earth dam and the proposed buttress dam on Khassa Chi by considering the displacement analysis due to different frequencies of earthquakes (Figures 1 and 2).

2 Earthquake analysis for buttress dam

Large earthquakes frequently happen near plate boundaries. However, the interior of the plates releases 95% of the seismic energy. These are referred to as intraplate earthquakes [6]. Intraplate seismology understanding has advanced during the past few decades. Intraplate earthquakes are less stationary than previously thought, suggesting a more sporadic occurrence in space. It is, however,

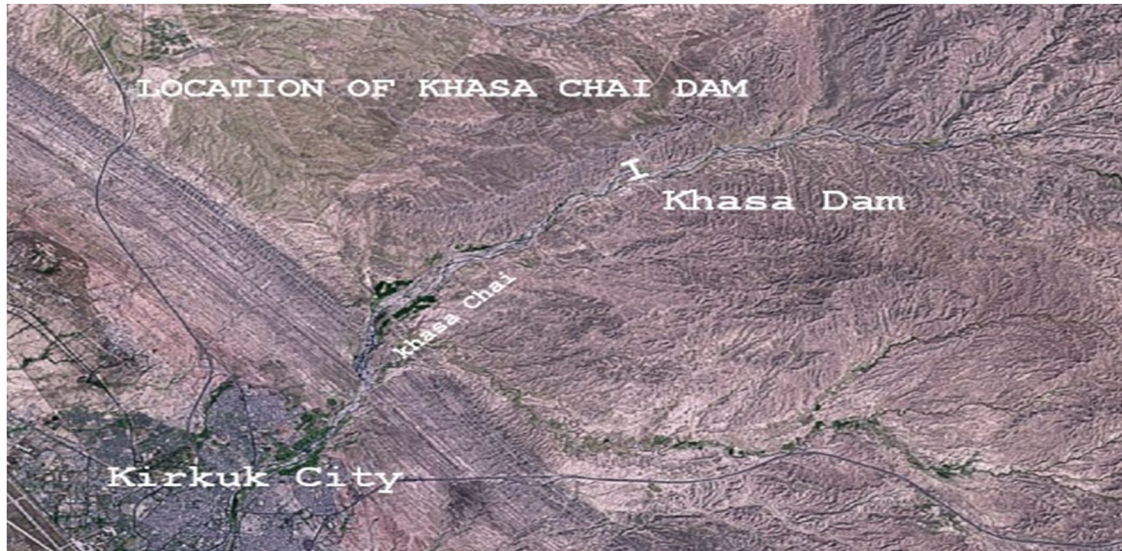


Figure 1: Location of Khassa Chi Proposed dam.



Figure 2: Khassa Chi dam currently.

impossible to forecast with any degree of accuracy when or where an earthquake will occur. Instead, probabilistic estimates – i.e., the likelihood that an earthquake of a given magnitude will occur in a particular location – are made [1]. The design earthquake motion is often provided as outcrop motions for dynamic assessments. However, the seismic input needs to be applied at the foundation's base for a direct FE analysis [2].

2.1 Eigenvalue analysis for buttress dam

An important sort of vibration analysis is frequency response analysis, which determines how a structure reacts to a vibration of a certain amplitude. Eigenvalue analysis, also known as modal analysis, is a type of vibration analysis that aims to determine a structure's natural frequencies. The primary use of Eigenvalue Analysis is to

address resonance issues that may result in mechanical damage to the structure [7].

Finite element (FE) analysis has become a primary tool in the structural evaluation of concrete dams by analyzing the intrinsic value of the structure; a set of determinants is defined as the stiffness matrix to determine the coordinates of the structural model which are the locations and directions of the point forces and displacements; the mass matrix for locating forces on nodes and include Execute Eigenvalue Analysis for set IRAM or FEASR as solution method in this research select the (IRAM). The size of the subspace in this method is chosen empirically. A poor choice of this size could lead to the non-convergence of the method [8] and definition of solver type as well as number of eigenfrequencies and boundary of frequency.

2.2 Response spectrum analysis

The Response Spectrum Analysis technique is utilized to approximate the structural response to brief, stochastic, transient dynamic occurrences. Instances of such occurrences include seismic activities and tremors. The absence of precise information regarding the load's temporal evolution poses a challenge for conducting an analysis that accounts for time-dependency. The event's brief duration precludes it from being classified as an ergodic or stationary process, thereby rendering a random response methodology inapplicable [9].

The methodology of the response spectrum technique relies on a distinctive form of mode superposition. The concept entails furnishing an input that establishes a threshold for the maximum level of excitation that an eigenmode, characterized by a specific natural frequency and damping, can experience as a result of an event of this nature.

2.3 Steps of analysis

The definitions of the elements, including the material and geometrical qualities assigned to them, are checked during model evaluation. Moreover, the element assembly, or ordering of the element and nodal variables in the system set of degrees of freedom, is defined. The model evaluation can be configured with the following parameters:

- An extended test of the aspect ratio and geometry of elements;
- Employing a set of material safety factors;

- Nodal normal of shell elements averaged;
- A choice for assessing reinforcements;
- Tolerance for constructing vector-based variable directions;
- Automated tie-downs.

2.3.1 Analysis of structural Eigenvalues

The natural or Eigen frequencies and corresponding shape modes of a structure are identified using an eigenvalue analysis; Diana offers several types for carrying out an eigenvalue analysis: standard eigenvalue analysis, free vibration analysis, linearized stability analysis, heat flow analysis, fluid–structure interaction analysis, unsupported system with shift, and nonlinear system with shift [10].

2.3.2 The free vibration equations

To be resolved in the mode superposition approach, which are expressed as:

$$K\varnothing = \omega^2 M\varnothing, \quad (1)$$

where K is the matrix of symmetric stiffness, M is The finite element mass matrix, ω is the angular speed, and eigenvector is the mode shape vector that corresponds to the mode $\varnothing$.

2.3.3 The Execute Eigenvalue analysis

A user-specified number of Eigen pairs is calculated by the Arnoldi method-based eigenvalue analysis. It is possible to calculate up to n Eigen pairs, where n is the dimension of the system matrix [11].

According to the method of eigenvalue analysis used, the resulting eigenvalues are output in ascending order of absolute value: Standard eigenproblem, Free vibration, and Stability analysis [12].

2.3.4 Analysis of response spectrum

Response Spectrum Analysis is a way to figure out how a system will respond linearly when it is exposed to a base excitation spectrum. The Response Spectrum Method, as described by Gupta [13], is used to build Diana. Maximum forces, [11] which can be calculated from maximum relative displacements, are of special interest during the design process. Reaction Spectrum Analysis outputs are typically presented as multimodal summaries [14].

The algorithm in DIANA FEA works under the assumption that damping has a negligible contribution. In most cases, damping is already factored into the response spectrum indirectly.

Alternatively, sections of the critical damping factor C_{crit} with specifically determined modal damping ratios C_i or computed modal damping ratios based on strain energy are tool that can be combined with the actual excitement spectrum.

$$M_{ii}(t) + K_u(t) = -Mi\ddot{U}_{su}(t). \quad (2)$$

$M_{ii}(t)$ is the inertia force, $K_u(t)$ is the stiffness force, $Mi\ddot{U}_{su}(t)$ is the excitation force, and the negative sign in excitation force refers to the acceleration exerted in a direction opposite the axis. There is a working direction in the base acceleration spectrum, i.e., a base acceleration value $\ddot{U}_{su}$, the base excitation system of equations can then be expressed. where, i represents the degree of freedom's contribution to the excitation's operating direction. The internal set of forces are the forces acting on the structure:

$$f_k = Ku(t), \quad (3)$$

and the pseudo-inertia forces

$$f_m = M(\ddot{u}(t) + i\ddot{u}_{su}(t)) = Ma(t). \quad (4)$$

Equilibrium of the forces gives $f_m = -f_k$ which results in

$$a = \omega^2 u, \quad (5)$$

where a is the acceleration force, ω is the angular speed, and u is the displacement.

The spectral acceleration SA is the absolute pseudo-acceleration's maximum value

$$S_A(\omega) = \max|a(t)| = \omega^2 S_D(\omega), \quad (6)$$

where the spectral displacement SD is

$$S_D(\omega) = \max|u(t)|. \quad (7)$$

3 Load and load combinations [14]

3.1 Dead load

In the numerical model, gravity loads are typically categorized as body forces. With this process, the density and the gravitational acceleration could be calculated. Hence, the gravitational forces are determined by the modeled structure's volume.

$$F = \sum V \cdot \gamma, \quad (8)$$

Here, V is the structure volume (m^3), γ : Unit weight of material (kg/m^3).

3.2 Hydrostatic water pressure

Is typically the main static external force at work on major dams? It is important to take into account hydrostatic pressure both upstream and downstream by combining the least advantageous two [15]. Equation (9) is used for calculating the hydrostatic pressure:

$$P_w = \rho_w \cdot g \cdot y, \quad (9)$$

where ρ_w is the water density, and y is the water depth (m).

3.3 Uplift pressure

The uplift forces depend on the type of foundation at the bottom of the dam, where the uplift forces are taken into consideration if the foundation type is of type Mat foundation, and the intended forces are neglected in the case of the type of foundation continuously isolated [16]. In this research, foundation type isolated was assumed, and thus, the uplift forces could be neglected [16]; equation can be used to compute the uplift pressure distribution [17] (Figure 3):

$$P_u(x) = \rho_w g \left(H - \frac{H-h}{L} x \right), 0 \leq x \leq L, \quad (10)$$

where $P_u(x)$ is the uplift pressure at position x , where L is the front plate thickness (L_{fp}) or total width (L_{tot}); H is the upstream water head, and h is the downstream water head.

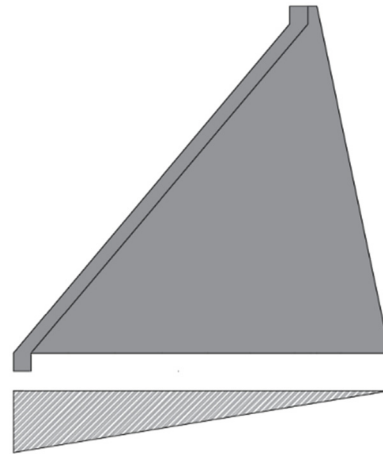


Figure 3: Uplift pressure under dam.

3.4 Ice load

A shift in the amount of an already-developed ice cover at a reservoir causes the ice load on dams to change. The volume of the ice grows as the temperature rises, and *vice versa*. The ice cover will crack as the volume decreases, and those cracks are immediately filled with ice-cold water. The ice cover will be pushed by a rise in temperature, resulting in horizontal pressure from the ice cover to the surroundings. The thickness of the ice cover, the length of the hot weather, and the speed at which the temperature is rising all affect how much pressure is present [18].

The amount of the ice load can generally be considered with a range of 50–200 kN/m. The thickness of the ice cover can consequently be considered to be between (0.6 and 1) m. The thickness of the ice cover is expected to have a triangular distribution of the ice pressure, with the design water level having the highest pressure [17].

3.5 Earthquake loads

Typically, outcrop motions are used to represent the design earthquake motions for dynamic studies. However, the seismic input needs to be applied at the foundation's base for a direct FE analysis [2].

After taking into account the accelerations that may be anticipated at each project site, as defined by the geology of the site, proximity to major faults, and earthquake history of the region, as indicated by seismic records available, earthquake loadings should be chosen. For projects without comprehensive seismicity investigations, the likelihood zone can be determined using seismic risk maps. For buildings in zones 0 and 1, an earthquake analysis is not necessary unless site analyses reveal the existence of faults that could cause structural damage in the event of an earthquake or previous earthquake epicenters are found nearby. The magnitude of the earthquake forces can be determined using the following techniques: Seismic Coefficient Method, Pseudo Dynamic Method, and Dynamic Methods [19].

3.6 Wind pressures

If a dam superstructure is carrying big crest gates, a wind pressure of 30 pounds per square foot in any direction should be used in the stability analysis.

4 Geometry and material characteristics of a concrete buttress dam

The concrete buttress dam that this research investigated was chosen due to its low cost and suitability for all soil types. Mass concrete's strength and durability are affected by numerous variables. The concrete must be strong enough to safely withstand the design loads throughout the structure's lifetime. The concrete must be resistant to the effects of weathering (freeze–thaw cycles), chemical activity, and erosion. The concrete's strength and durability must be uniform throughout the construction, as the weakest point will determine its structural sufficiency [2].

The design of the dam was based on the data available in the site investigations of the dam site of Khassa Chi [20].

The design criteria for buttress dams are very similar to those for gravity dam structures. The key difference is the incorporation of the buttress thickness, represented by the letter “*t*,” which takes on the responsibility of supporting the additional load resulting from the size of the dam. This area includes the total of the separate space between succeeding buttresses (shown as ‘*x*’) and the actual buttress thickness (expressed as ‘*t*’).

As a result, it carries the weight of (*x* + *t*) meters of dam extension within a meter's length of the buttress section. This is different from gravity dam segment lengths, which are often measured in units of meters. In order to take into account this situation, it is cleverly possible to change the density of water's unit weight by using a surcharge factor, designated by the letter “*S*.” The form of this surcharge factor is $S = (x + t)/t$. As a result, $w^*(x + t)/t$ is the effective unit weight of water [21].

The bases of the dam were designed as the first stage in the design stage by relying on the sliding criterion and the stress criterion. The value of the largest base was taken to ensure the safety aspect and by relying on the available hydrological data. The study revealed that under normal operating conditions, the water height measures 55 m with an inclination angle of 50° [21]. This measurement corresponds to the initial section of the buttress. Subsequently, an additional section of the buttress was incorporated, measuring 1.5 m in width. The design of the final section of the buttress, with a width of 12 m, was based on the inclination of the initial section. Table 1 shows details [21].

$$\text{Slender ratio} = \frac{\text{Height of buttress}}{\text{Thickness of buttress}} = 12 \text{ to } 15, \quad (11)$$

Table 1: The recommended buttress spacing depended on dam height

Mean Dam height in m	The recommended buttress spacing in m
<15	4.5
15–30	4.5–7.5
30–45	7.5–12
>45	12–15

Bold value represents the maximum and minimum limits.

$$\text{Thickness of buttress} = \frac{58}{12} = 4.5 \text{ m,}$$

$$\text{Massiveness factor} = \frac{\text{Spacing of buttress}}{\text{Thickness of buttress}} = \frac{12}{4.5} \quad (12) \\ = 2.67 \text{ (2.5 to 3).}$$

By adding 3 m as free board the final dam height has been reached, employment DIANA FEA program for designing the dam, giving properties for each material, as will be shown in Table 2. All other design considerations were taken into consideration as selecting the slenderness ratio is the ratio of the height of the buttress to the thickness of the buttress, A lower slenderness ratio implies a wider base compared to the height, resulting in a more squat or stubby dam shape. This configuration offers

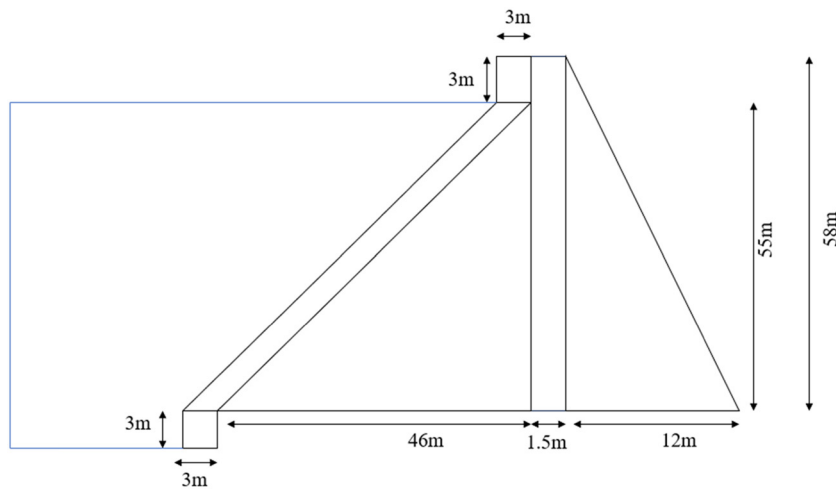
inherent stability and resistance against overturning due to the greater base width. Lower slenderness ratios are often preferred in areas with high seismic activity to enhance the dam's resistance to seismic forces. massiveness factor and plotting suitable master curve. The height of the dam is 58 and 55 m is max water level, based on represents the highest level of water ever observed in Khassa Chai natural conditions, to which a freeboard of 3 meters is added, and the freeboard significance stems from the fact that a sufficient freeboard reduces the likelihood of an uncontrolled release of water or tailings on the dam, preventing social and environmental effects upstream of the impoundment [22] (Figures 4 and 5).

A dam is divided into a front side, which is a flat slab with a thickness of 3 meters, with a slope of 50 degrees. The angle of inclination is from 40 to 50 degrees, where the lower the angle, the better, as it will better resist forces coming from water from upstream [21].

The back side of the dam consists of three parts of the buttress, the first part is 46 m wide and 55 m high, which is located below the front slab, and the second Which is the insulation wall part is 1.5 m wide, and 58 m high, in which is located drainage gallery, and the third part is represented by a width of 12 m at the base and a height of 58 meters and all pillars are 4.5 meters thick, The foundation

Table 2: Material properties for concrete dam and foundation and reservoir model

Properties material	Density (kg/m ³)	Young's modulus (N/m ²)	Poisson's ratio	Mass density specification	Material class
Dam	2,400	3.2×10^{10}	0.2	—	Concrete and masonry
Foundation	0	8×10^9	0.2	Saturated density	Soil and rock
Reservoir	1,000	—	—	—	—

**Figure 4:** Geometry of concrete buttress dam case study.

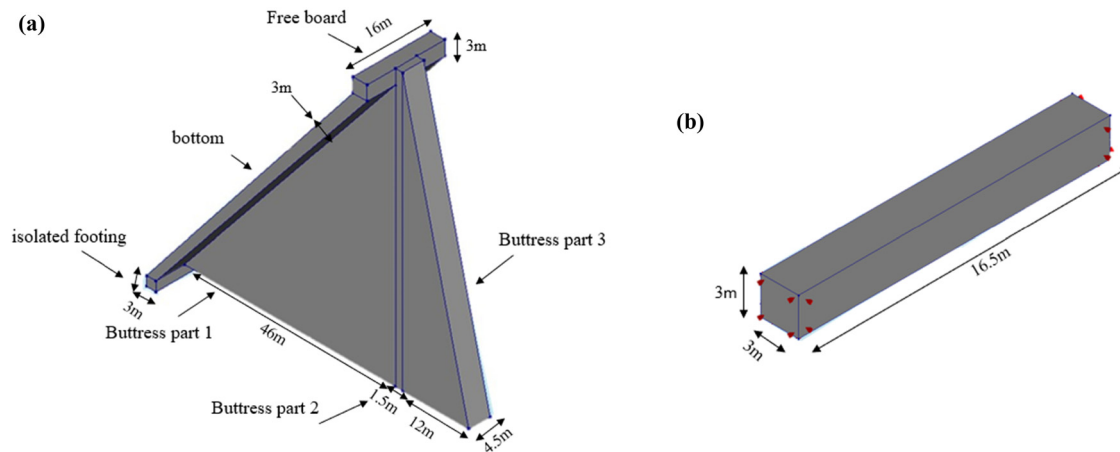


Figure 5: (a) Dam Model, (b) foundation model.

of the dam is 3 meters deep and 3 meters wide, which extends below the slab along the length of the dam. The concrete material has young's modulus = 3.2×10^{10} N/m², density = 2,400 kg/m³, and Poisson's ratio = 0.2.

4.1 Properties of foundation

The Khassa Chi valley traverses the northwest plunge of the Qadir Karam syncline in the project area. The Bakhtiari series has a dip direction that is virtually perpendicular to the valley; therefore, the bed slopes downwards from left to right banks. The dip angle is minor, ranging from 5 to 8 degrees.

The Khassa Chi River bed is situated in close proximity to the boundary between the comparatively pliable stratum of the left bank and the relatively dense stratum of the right bank, where the latter is presumed to overlay the stratum that is visible on the left bank.

The abutment on the left bank primarily consists of reddish clayey-silty marls, which are typically overlaid by clayey weathering products. Several thin layers of loosely cemented soft sandstone are present within the marls.

The predominant geological composition on the right bank consists of silty and sandy clay. The series under consideration exhibits a significant occurrence of beds composed of conglomerates that are well-cemented locally. Conglomerates are commonly observed as vast lenses.

The Khassa Chi valley is characterized by a prominent relief, primarily composed of conglomerates that constitute a significant portion of the series, situated above the crest elevation of the proposed dam. The site exhibits various instances of erosion and alluvial deposition. The

upper terrace is characterized by the presence of thin layers of gravel and sand located at the apex of the left bank abutment. Conglomerates that are well cemented are situated on pre-existing formations in the vicinity of the slope that intersects the abutment of the left bank in the lower region.

The foundation on which the dam rests was chosen to be continuous strip footing, and in this case, the effect of the uplift forces resulting from groundwater behind and below the dam is neglected due to its low value [16], where the concrete part is under the front slab only and is 3 meters wide and 3 meters deep, and the foundation remains for the bottom of the buttress parts of the earth materials. The Khassa Chai valley is 300–350 m wide; it contains cobbles, gravel, and sand which are 3.5 to 5.0 meters thick.

4.2 Finite element modeling

The majority of the study was devoted to investigating and comprehending the modeling approach for buttress dam Analysis and all of its possible applications in DIANA.

Diana has several options for modeling structures such as a Rectangle system, coordinate system, and Relative Coordinate System; in this research, a coordinate system was used to model the dam structure [23], The system of equations to be solved for problems involving linear elasticity is:

$$K_u = f, \quad (13)$$

where K is the stiffness matrix of system, u is a vector representing the unknown nodal degrees of freedom, such as rotation and displacement, and f is the nodal force

vector that corresponds to the degrees of freedom u ; each part of the dam was modeled DIANA program using the selected coordinates as shown:

$$\begin{array}{rcl}
 & \begin{matrix} 0 & 6 & 0 \\ 44.5 & 6 & 0 \\ 44.5 & 6 & 55 \end{matrix} & \begin{matrix} 44.5 & 6 & 0 \\ 46 & 6 & 0 \\ 46 & 6 & 58 \\ 44.5 & 6 & 58 \end{matrix} \\
 \text{Buttress part 1} = & & \text{Buttress part 2} = \\
 & \begin{matrix} 46 & 6 & 0 \\ 60 & 6 & 0 \\ 46 & 6 & 58 \end{matrix} & \begin{matrix} 0 & 0 & 0 \\ 0 & 0 & -2 \\ -2 & 0 & -2 \\ -2 & 0 & 0 \end{matrix} \\
 \text{Buttress part 3} = & & \text{Roof} = \\
 & \begin{matrix} 0 & 0 & 0 \\ 44.5 & 0 & 55 \\ 41.5 & 0 & 55 \\ -2 & 0 & 0 \end{matrix} & \begin{matrix} 44.5 & 0 & 55 \\ 41.5 & 0 & 58 \\ 41.5 & 0 & 58 \\ 41.5 & 0 & 55 \end{matrix} \\
 \text{Bottom} = & & \text{Top} =
 \end{array}$$

Figure 6 depicts the dam–reservoir–foundation system's finite element model for this study:

After modeling a dam, a water level in a reservoir is given, and then, the properties of each of the materials are determined as shown in Table 1. After applying mesh to it, the number of elements was determined as 177,081 elements as follows:

- Isolated Footing (68 elements)
- Reservoir (164,305 elements)
- Slab (2,431 elements)
- Fluid–Structure element (1,224 element)
- Buttress 1 (6,280 elements)
- Buttress 2 (580 elements)
- Buttress 3 (2,040 elements)

4.3 Boundary conditions

4.3.1 Constraints

The exterior faces of the base that lie in the YZ plane are constrained in the X direction, Similarly, we impose Y -axis constraints on the foundation's outer faces aligned with the XZ plane, and Z -axis constraints on the foundation's bottom face.

The reservoir-free surfaces' head potential must be zeroed out. Because of this, we employ a fixed head potential on the reservoir's upper and lower faces. The following sections provide the results of the concrete buttress dam analyses.

A set of hypotheses was identified

- The distance of the near field reservoir has been determined at 150 m.
- the type of foundation is isolated footing and uplift pressure is ignored.

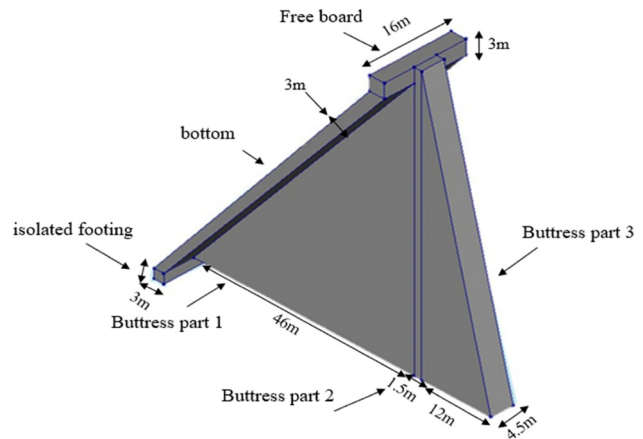


Figure 6: FEM for dam case study.

- the connection between the front slab and the buttress is rigid connection.

To begin, the mechanical properties of the FE models are analyzed by an eigenvalue analysis; in the first static step, loads of gravity and hydrostatic pressure are applied, and then, in the second step, nonlinear dynamic analyses are done (Figures 7 and 8):

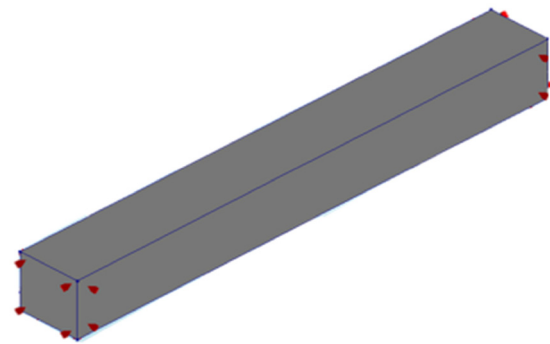


Figure 7: Constraints of buttress dam foundation.

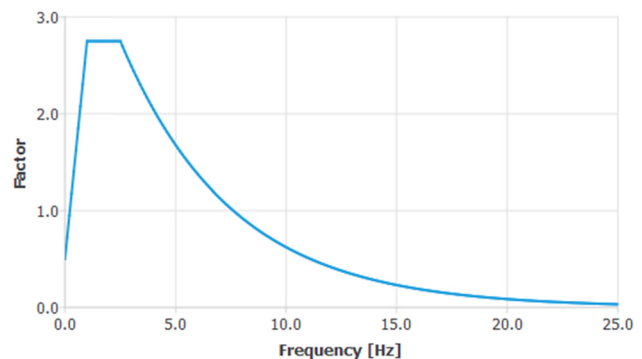


Figure 8: Definition of a frequency function for the combined load and earthquake (a response spectrum).

5 Results and discussion

5.1 Structural response analysis without reservoir

5.1.1 Eigenvalue analysis

A color map gradient from cool colors to warm colors to represent the level of displacement. Typically, warm colors such as red or orange are used to represent higher values, indicating areas of potential concern or higher displacement, while cool colors such as red and green refer to lower values. A preliminary design of the buttress dam with initial dimensions is done by using the DIANA FEA program, for load combination design (dead load only) with max. displacement +1 and min. displacement -1 and without considered reservoir (Figures 9–11).

5.1.2 Response spectrum analysis

The analysis is being performed to obtain a quick estimate of the maximum displacement, the SRSS method is recommended in the previous study as a better choice. The range is specified displacement 0.2 as the maximum and -0.2 as a minimum in analysis displacement in Superposition type SRSS for Response Spectrum Analysis in DIANA FEA; the SRSS method takes into account the uncertainty in the

results of the modal analyses. In other words, the SRSS method assumes that there is a chance that the actual maximum displacement could be slightly less than 0.2 and a chance that it could be slightly more than 0.2. The SRSS method then calculates a displacement that is likely to be exceeded only 50% of the time. This is a conservative approach that is often used in engineering design. It ensures that the structure is designed to withstand the most likely maximum displacement, while also taking into account the possibility of a slightly larger displacement. From the observation of figures, there appears a group of figures affected by vertical and horizontal forces; for example, the first figure of DtXH refers to the displacement of a point on a structure due to the horizontal component of a load, DtYL refers to the displacement of a point on a structure due to the vertical component of a load (Figure 12).

5.2 Structural response analysis with reservoir

5.2.1 Eigenvalue analysis

Range of displacements is (1, -1) and with an application with combination 2 for the eigenvalue and response spectrum analyses with reservoir (Deadweight and Hydrostatic Pressure load sets (Figures 13–15)).

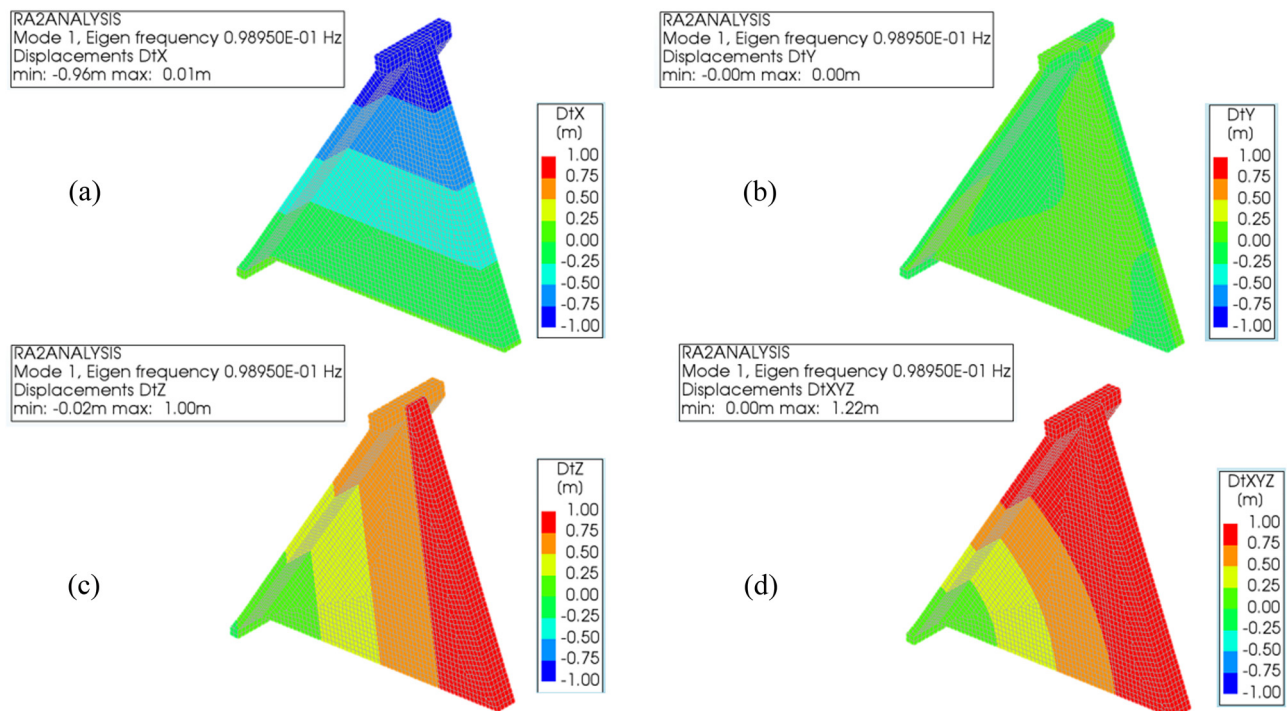


Figure 9: Mode 1 with Eigen frequency 0.98950×10^{-1} Hz: (a) displacement in the x direction, (b) displacement in the y direction, (c) displacement in the z direction, and (d) displacement in the xyz direction.

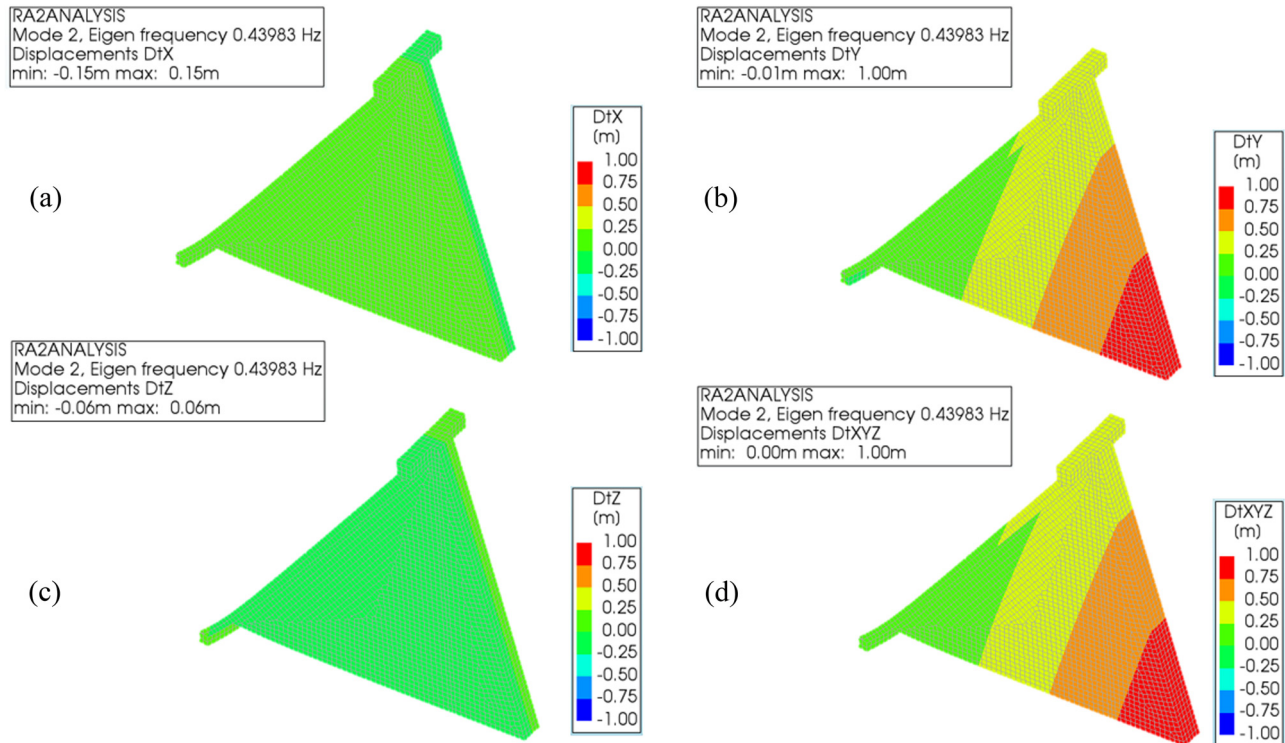


Figure 10: Mode 2 with Eigen frequency 0.43983 Hz: (a) displacement in the x direction, (b) displacement in the y direction, (c) displacement in the z direction, and (d) displacement in the xyz direction.

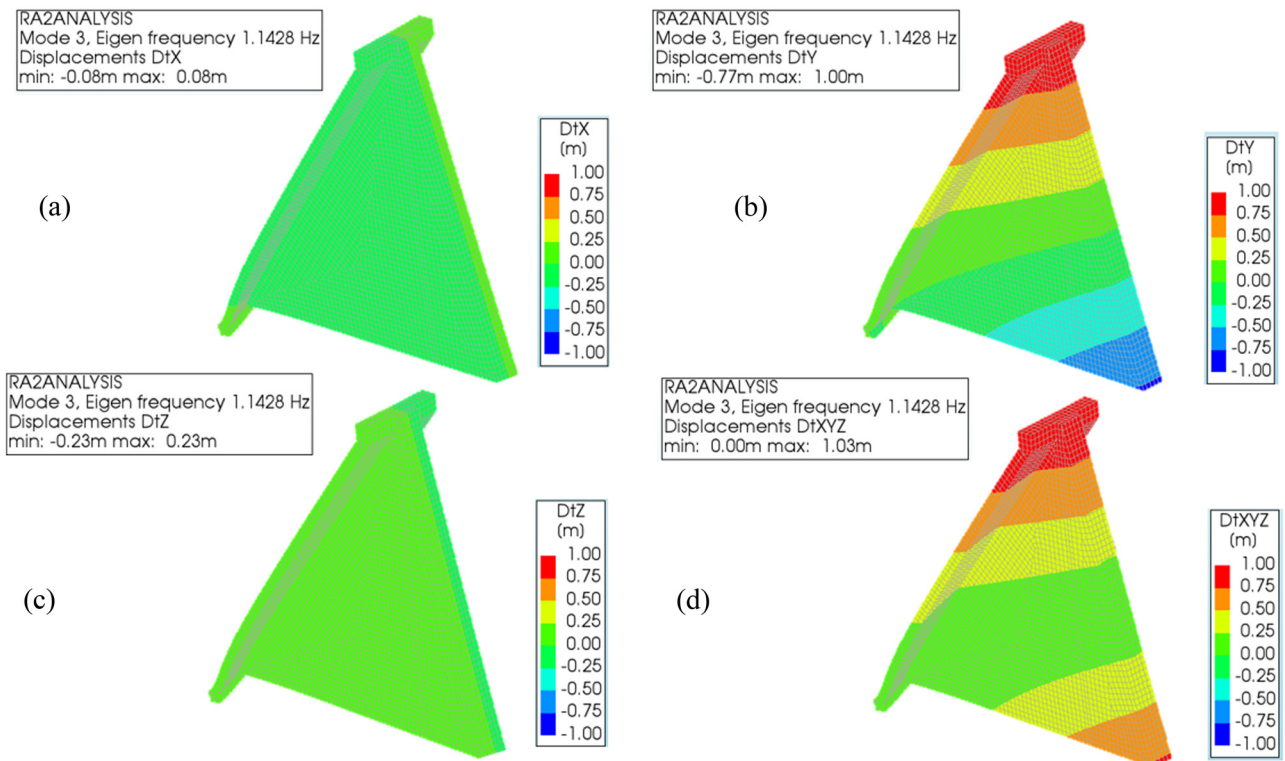


Figure 11: Mode 3 with Eigen frequency 1.1428 Hz: (a) displacement in the x direction, (b) displacement in the y direction, (c) displacement in the z direction, and (d) displacement in the xyz direction.

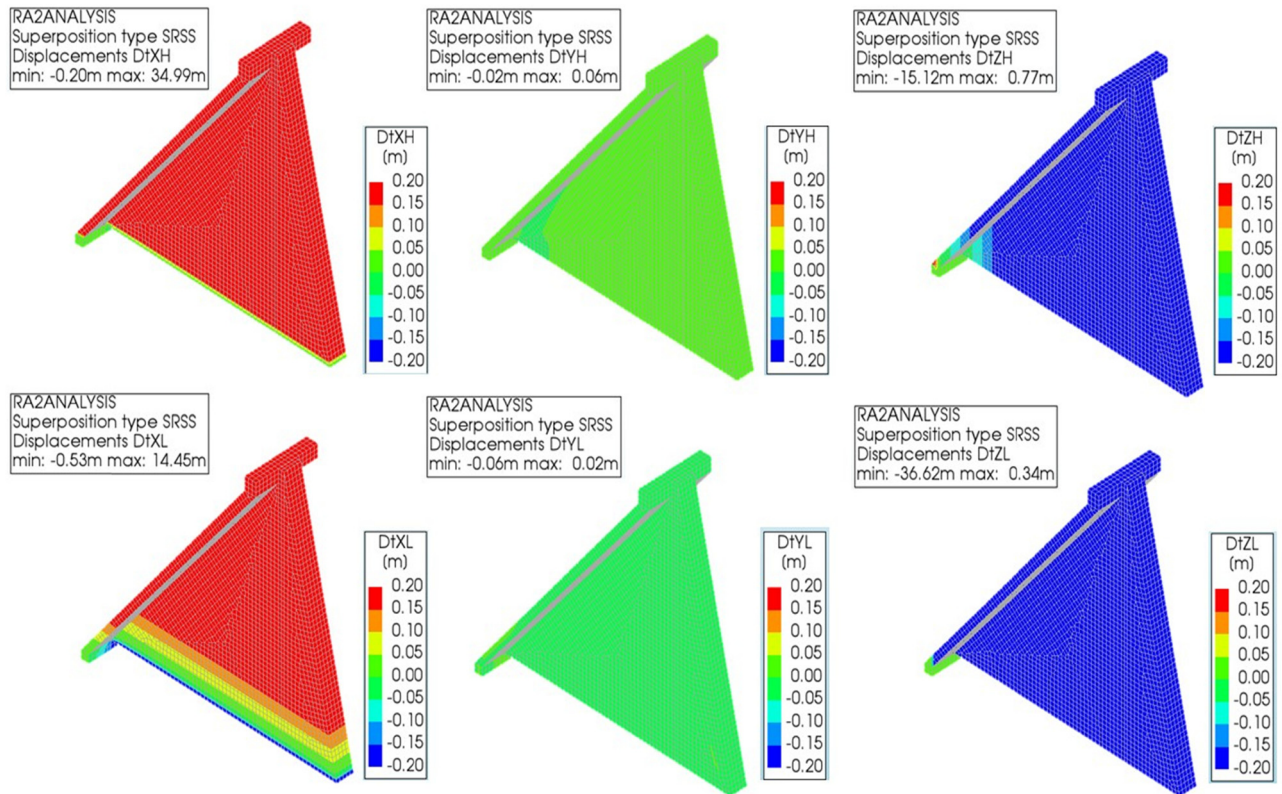


Figure 12: Superposition type SRSS for RSA.

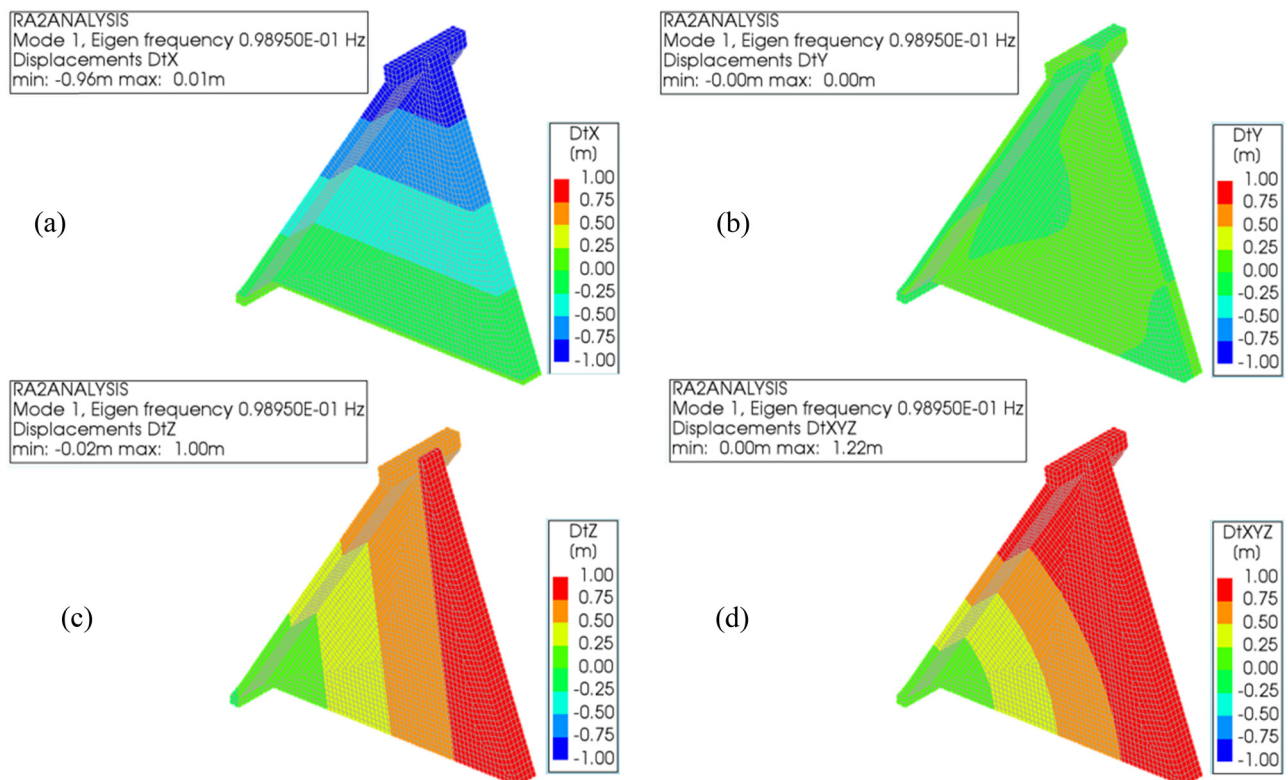


Figure 13: Mode 1 with Elgen frequency 0.98950×10^{-1} Hz: (a) displacement in the x direction, (b) displacement in the y direction, (c) displacement in the z direction, and (d) displacement in the xyz direction.

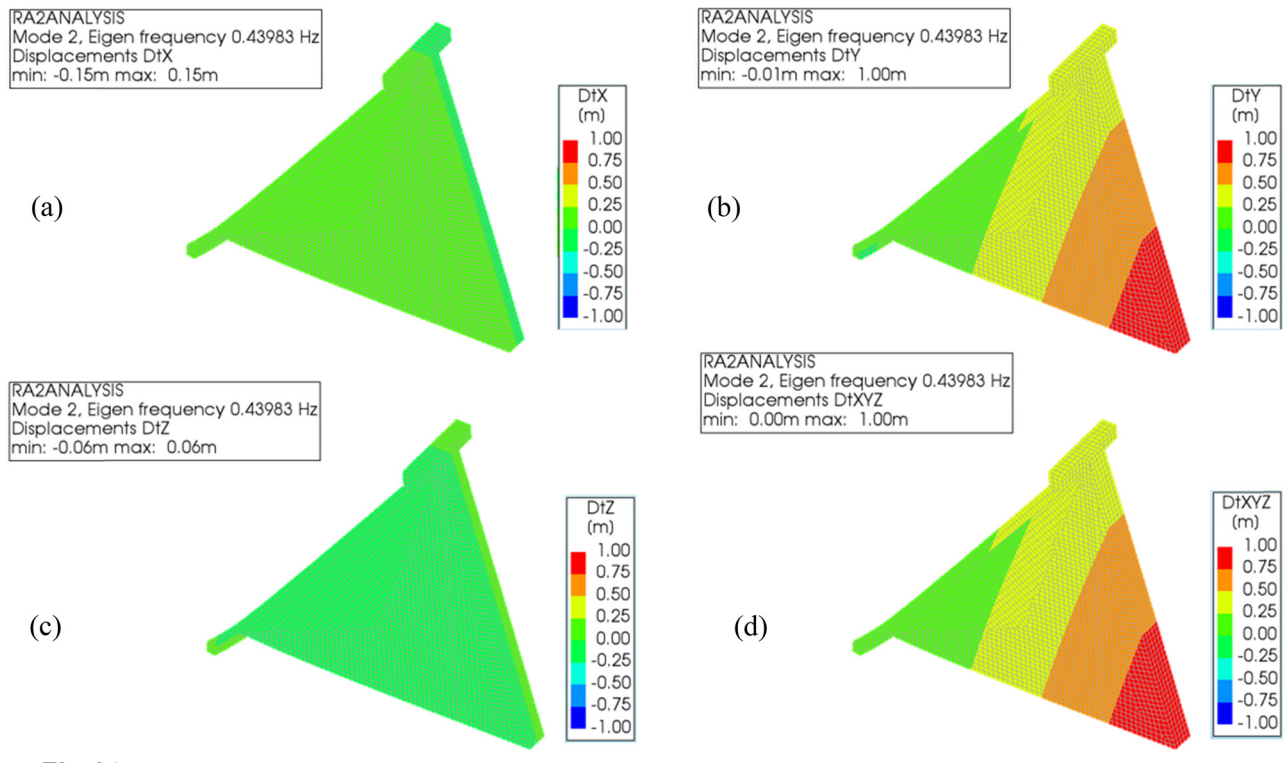


Figure 14: Mode 2 with Elgen frequency 0.43983 Hz: (a) displacement in the x direction, (b) displacement in the y direction, (c) displacement in the z direction, and (d) displacement in the xyz direction.

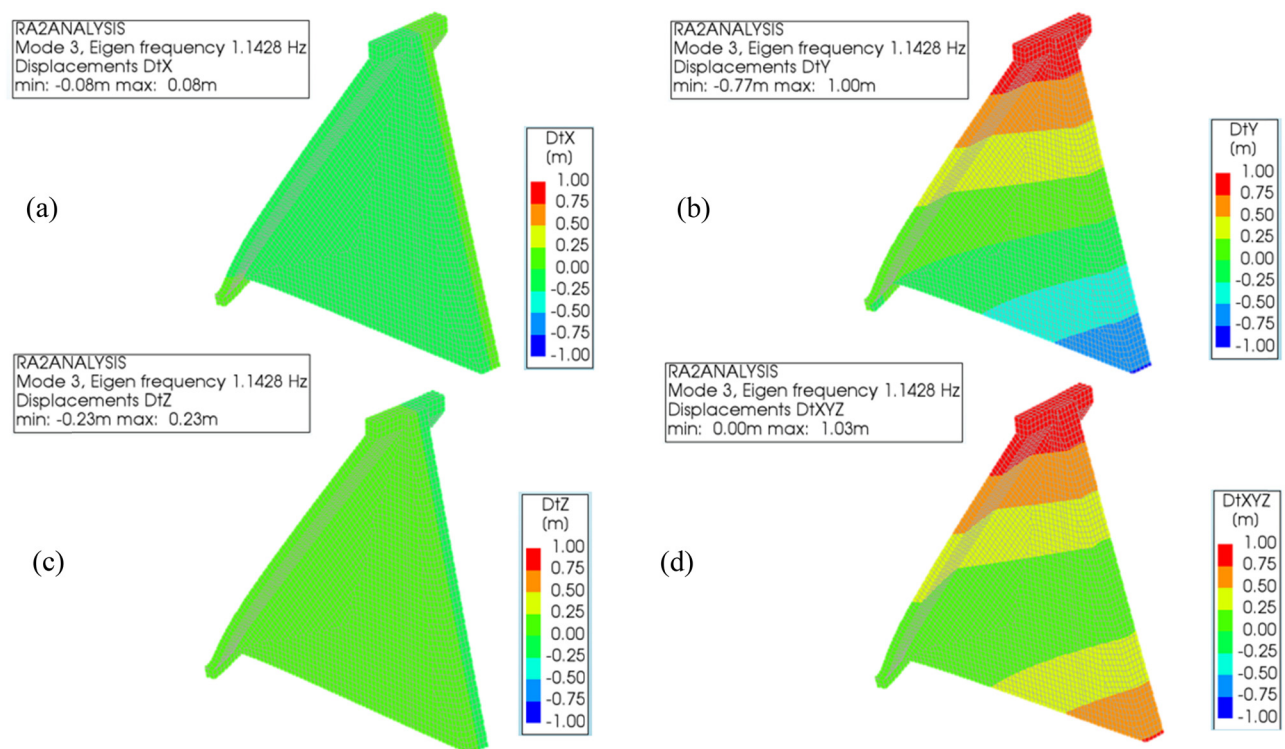


Figure 15: Mode 3 with Elgen frequency 1.1428 Hz: (a) displacement in the x direction, (b) displacement in the y direction, (c) displacement in the z direction, and (d) displacement in the xyz direction.

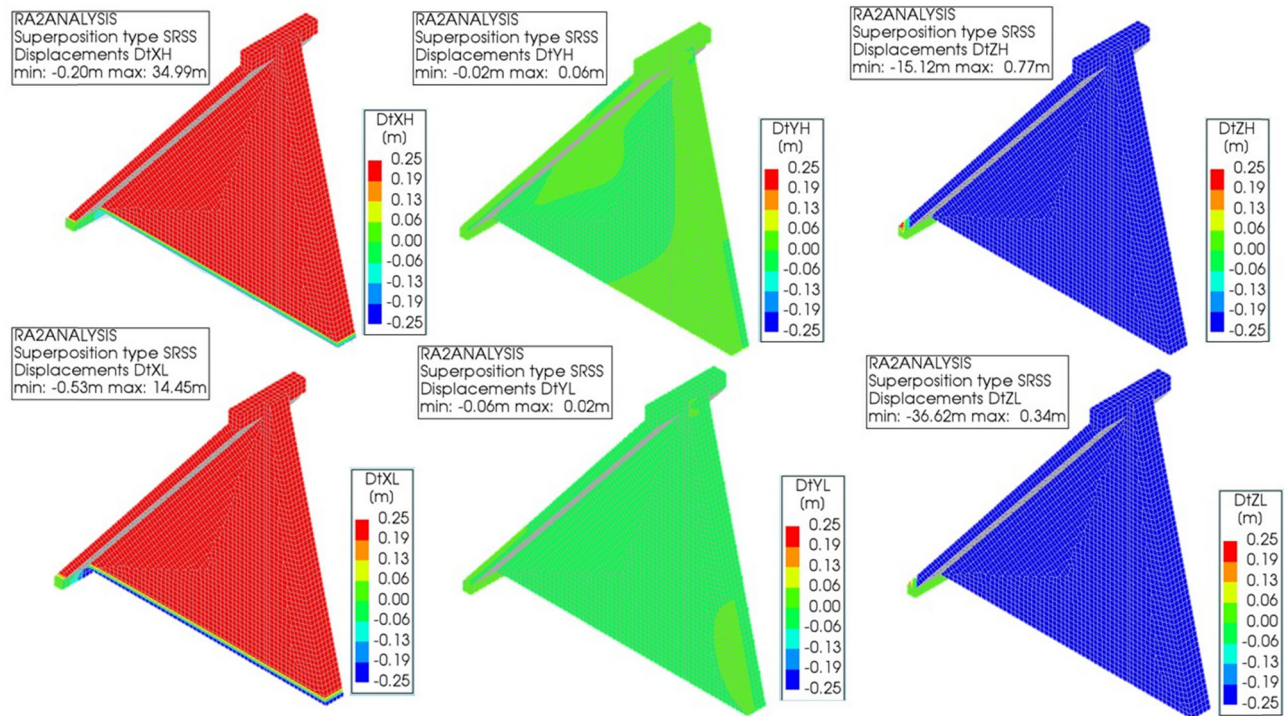


Figure 16: Superposition-type SRSS for Response Spectrum Analysis.

5.2.2 Response spectrum analysis

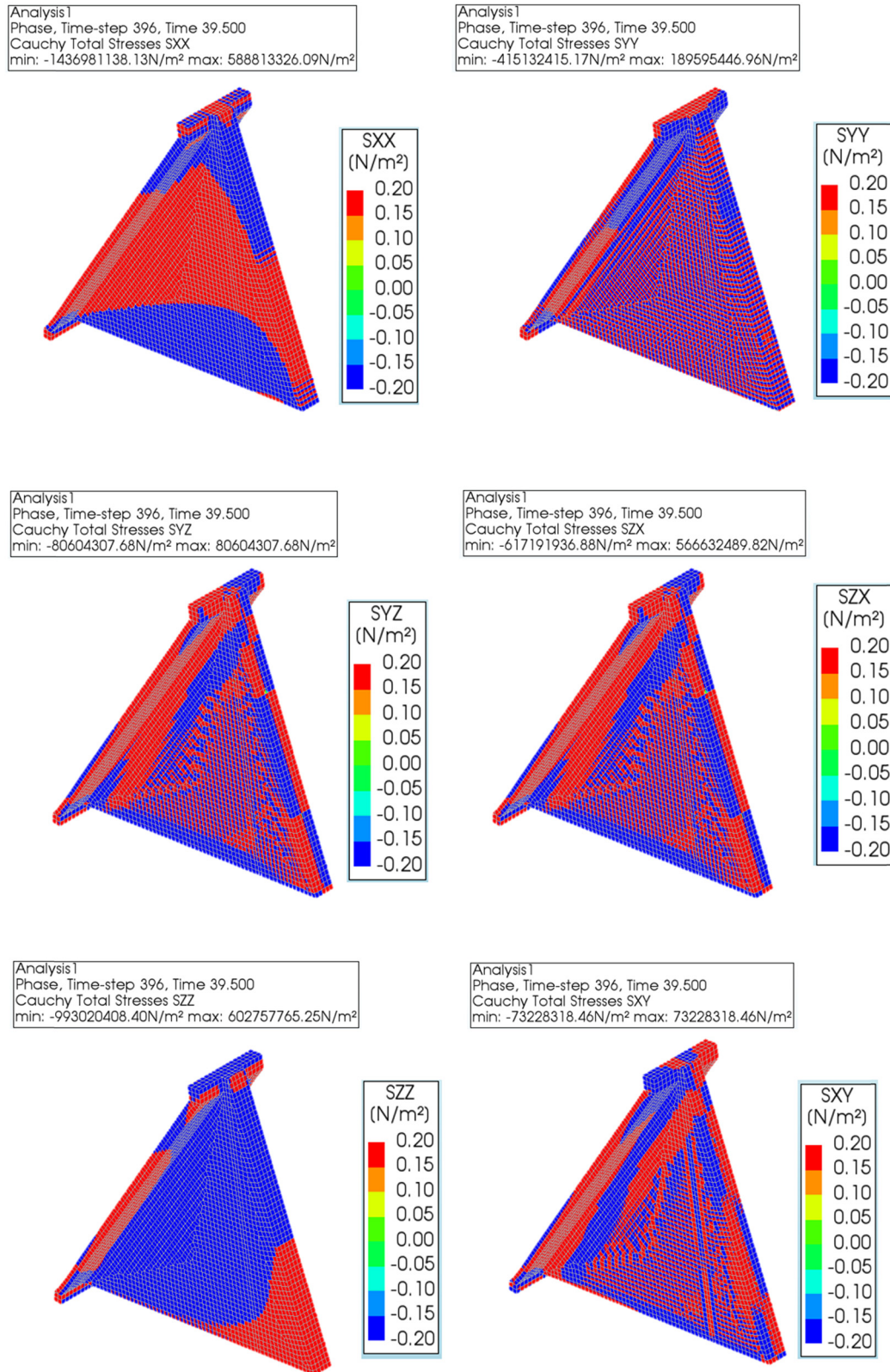
The range is specified displacement 0.25 as the maximum and -0.25 as a minimum in analysis displacement in Superposition-type SRSS for Response Spectrum Analysis in DIANA FEA (Figure 16).

5.3 Stress

Figure 17 shows 6 Modes, which appear as stress for each of them in response to the force applied to the dam; in Figures 18–23, the first mode represents the normal stress component in the x -direction. It indicates the stress acting along the horizontal axis of the dam, Figure 17 shows the dam response to the model in the form of stress for node 3,539, which shares a group of elements. The amount of stress to which a node from each element is exposed varies, as the highest stress to which a node of an element is exposed appears from 7,737, and this element is shared with the upstream slab for this reason the stress to the x -direction becomes high on this node. The second mode shows the normal stress component in the y -direction. It indicates the stress acting along the vertical axis of the dam. The response of

this dam to stress is shown in Figure 11, and the highest value of stress is also shown in the same previous element.

Stress in the third mode represents the normal stress component in the z -direction. It indicates the stress acting perpendicular to the surface of the dam. The stress of this model is represented in Figure 19; the data indicate that the highest stress comes from element 10,371, which is a common element between the node and freeboard. fourth mode appears the shear stress component in the xy -plane. It indicates the stress acting parallel to the base of the dam in the horizontal plane. data in Figure 20 refer to the maximum stress that came from 7,747 and this element is common with isolated footing. The fifth mode appears the shear stress component in the zy -plane. It indicates the stress acting parallel to the surface of the dam but in a different vertical plane. Data in Figure 21 refer to the maximum stress that came from 7,747, and this element is common with the upstream slab and refers to stress coming from hydrostatic pressure in the reservoir; final mode represents the shear stress component in the xz plane. It indicates the stress acting parallel to the side of the dam but in the vertical plane. Data in Figure 21 refer to the maximum stress that came from 7,737, and this element is common with the upstream slab and refers to stress coming from the abutment.

**Figure 17:** Element stress for all directions.

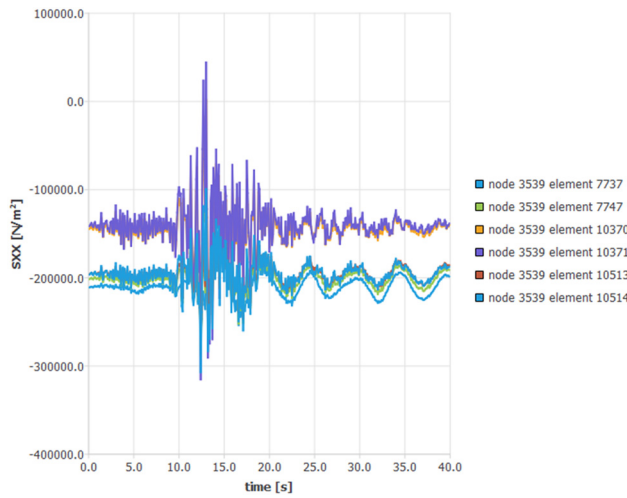


Figure 18: The normal stress component in the x-direction.

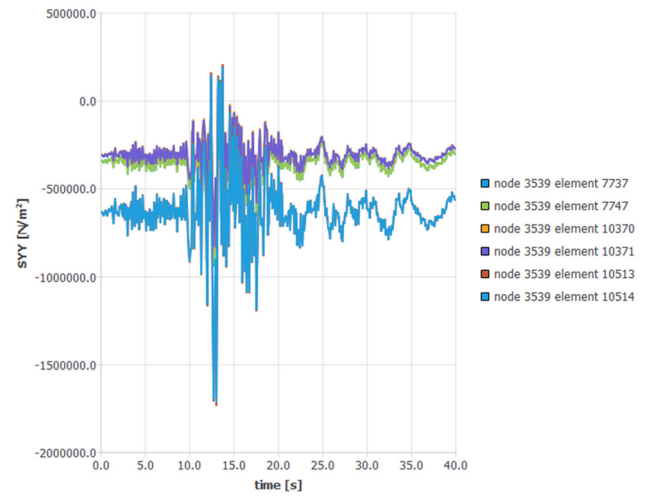


Figure 21: The normal stress component in the y-direction.

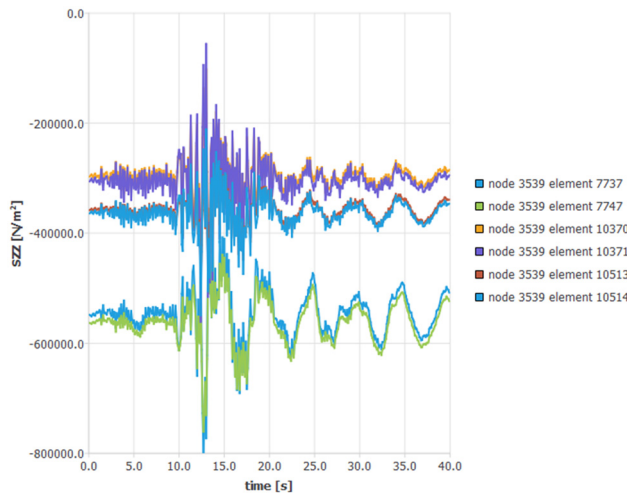


Figure 19: The normal stress component in the z-direction.

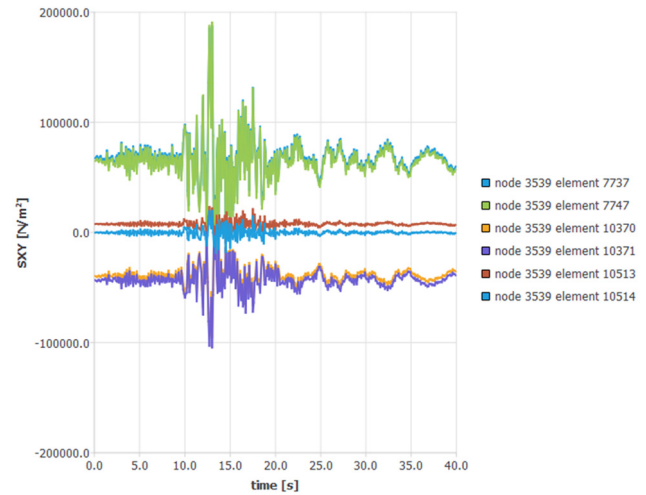


Figure 22: The shear stress component in the xy-plane.

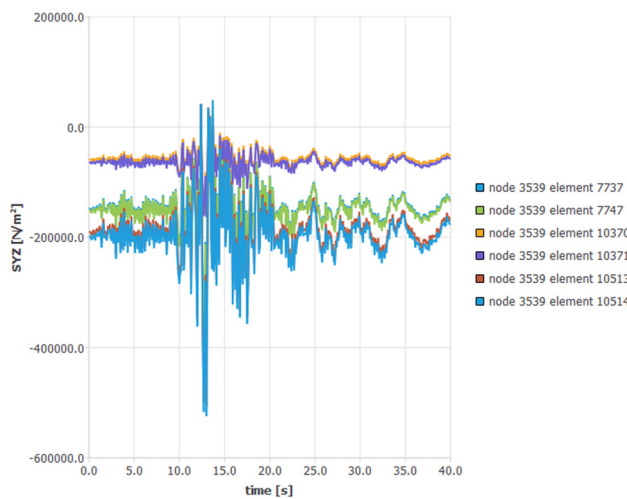


Figure 20: The shear stress component in the zy-plane.

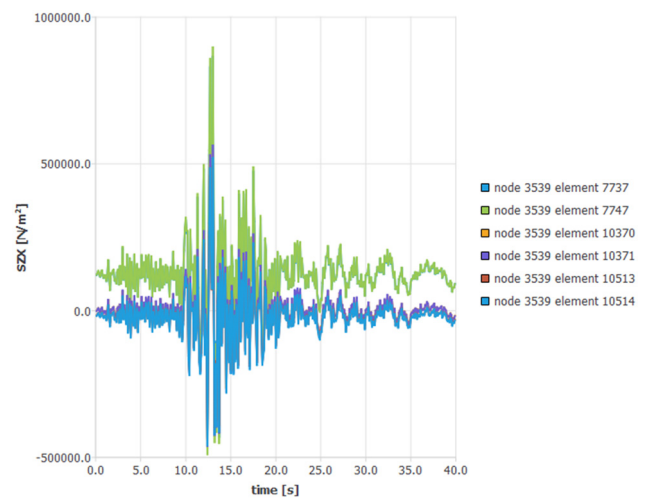


Figure 23: The shear stress component in the xz-plane.

6 Conclusions

The aim of this research article is to investigate a buttress concrete dam as an alternative to the Khassa Chai dam, especially the current one, which is considered one of the vital dams in the city of Kirkuk, using the Diana engineering program by subjected a dam under different seismic frequencies that ranged from $(0.8710 \times 10^{-1} - 13.412)$ Hz, Where 10 mods were made with different seismic frequencies by using eigenvalue analysis, it was determined that a typical buttress has natural frequencies that are less than 3.7 Hz, which is a rather high frequency, due to the slenderness of the concrete buttresses, Since the city of Kirkuk is far from a seismic line, 3.7 Hz is considered very high for this region.

The design of this dam was based on the assumption that the foundation of a dam is a continuous strip footing, which means neglecting the lifting force resulting from the bottom of the dam due to the presence of groundwater, in addition to the movement of water from the upstream to the downstream.

The dynamic linear analysis included two methods: Eigenvalue analysis and Response spectrum analysis for evaluating dam model. Linear eigenvalue analyses are limited in their ability to account for nonlinear behavior; thus, the modeling of contraction joints as hinges is necessary. The natural frequencies associated with the free vibration modes of the monoliths in the buttress dam are relatively high.

To implement the Eigenvalue analysis method, the (IRAM) was chosen to determine the range of convergence in the values of the frequencies of specific 10 models. Despite the convergence of the applied frequency values, the results show a difference in the response of the dam for each of the randomly applied values of the frequencies, where the response spectrum appears in the results of the analysis that the response of the dam is low in the low frequencies, while the response of the dam increases in the high frequencies, which are non-Realistic frequencies in the Kirkuk area where the dam is located.

To execute damping of the dam, the RSA was the chosen (Implicitly included in the excitation spectrum) method for damping for reducing or preventing oscillation resulting from the frequency of seismic. The SRSS method was chosen in a Modal superposition. This approach is applicable in cases where all modes can be regarded as decoupled, meaning that each mode possesses a frequency that is sufficiently distant from its adjacent modes to conclude that those modes do not interfere with one another. The results ensured the safety of the dam from all load combinations applied to the suggested dam.

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