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A Numerical Investigation of Seismic Load Analysis for a Concrete Gravity Dam Utilizing Abaqus Software

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Abstract: - Current research efforts focus on ensuring the safety of extant dams against seismic loads and designing earthquake-resistant new dams. Dependable analytical procedures are essential for a safe design that can withstand the forces induced by earthquakes. In this research, the Pine Flat Dam in California, United States, was subjected to modal, static, and dynamic analyses utilizing the Abaqus software to evaluate the impact of hydrostatic forces and dynamic stresses on the dam, taking into account flexible foundations. Using the vertical component and the horizontal component Taft Lincoln School Tunnel earthquake data, dynamic linear and nonlinear performance for concrete gravity dam were conducted. The results obtained from the nonlinear analysis exhibited a noteworthy decrease in the highest primary stress at the heel of the concrete dam, dropping from 3.17 MPa to 1.31 MPa. Conversely, no variation was observed in the minimum principal stress. In addition, the dam's maximum principal stress has shifted from its base to its neck. The findings from these analyses can contribute to enhancing new dam designs and evaluating seismic safety for existing dams. This study highlights the importance of accurate analytical procedures in ensuring dam safety and provides significant observations regarding the behavior of concrete dams during earthquake loading.

Keywords: Concrete gravity dam, Dynamic response, Finite element method, Seismic analysis, Abaqus.

1 Introduction

Dams are essential infrastructural elements that impede the natural flow of rivers or streams to regulate water levels and establish reservoirs for many uses, encompassing water provision, power production, flood mitigation, and agricultural irrigation. Nevertheless, the structural deficiencies of these systems can result in substantial property damage and adverse environmental consequences. Due to seismic activity, which can result in cracks in the dam's structure, the probability of failure rises. Hence, it is imperative to evaluate the structural stability of dams in relation to the many forces they must withstand, including hydrodynamic forces and seismic events. Researchers have proposed various methods for studying the behaviour of dams under seismic loads. Westergaard [1] and Chopra [2] proposed early methods for analyzing rigid concrete gravity dams under seismic activity. In recent decades, extensive research has been conducted to investigate the safety of current dams against seismic loads and to develop earthquake-resistant designs for new dams [3, 4]. Numerous studies have examined dams' response to seismic loads [5, 6]. The finite element technique was subsequently implemented for the analysis of dams due to its enhanced precision and dependability in comparison to existing methodologies [7]. The stability of dams can be significantly affected by hydrodynamic forces, including water pressure and wave action. The researchers [8-10], conducted prior studies which initially proposed the concept that water lacks compressibility. However, further experiments have demonstrated the significance of water compressibility in seismic analysis, as emphasized by Chopra [2]. Numerous investigations have been undertaken by scholars, including those referenced as [11-14], to explore the use of finite element analysis inside the realm of frequency domain.

Subsequently, Sommerfeld [15] and Sharan [16] proposed less intricate boundary conditions, whilst Tsai and Lee [17]and Maity and Bhattacharya [18] created more complicated boundary conditions. The seismic reaction of a dam is strongly influenced by the soil type in which it is constructed [19]. The pioneering work conducted by Pal [20] involved the initial application of nonlinear analysis techniques to the Koyna dam. However, this study did not account for the influence of the reservoir and assumed that the dam's foundation possessed rigid characteristics. In their study, Khosravi and Heydari [21] conducted an analysis of the dam under four distinct situations. These conditions involved considering the reservoir as empty or full, and the foundation as flexible or

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rigid. The seismic response of a dam was investigated by Burman et al [6] in relation to the influence of foundation flexibility and nonlinearity. The study employed the software SAP 2000. Similarly, Reddy et al. [22] conducted comparable investigations to examine the impact of a flexible foundation. This study provides an examination of a dam construction, with a specific emphasis on the effects of rock foundation flexibility and the influence of reservoir water during seismic loading conditions. The purpose of these assessments is to provide an academic contribution to the current body of research on the safety and stability of dams under seismic loads.

2 Materials and Methods

The methodology employed for constructing a concrete gravity dam model and assessing its seismic response is exemplified using the Pine Flat Dam in California as a case study. To achieve this objective, a two-dimensional (2D) finite element (FE) model of the dam was created using Abaqus, a commercially available finite element analysis software. The accuracy of the model was validated by subjecting it to time-history dynamic analysis under seismic excitations and comparing its results with those from previous investigations [23, 24]. The study's context featured the Pine Flat Dam, a concrete gravity dam, which is visually depicted in Figure 1.

Figure 1 Pine Flat Dam view.

2.1 Abaqus Modeling of the Dam-Foundation-Reservoir System

To simulate the dam-reservoir-foundation system in this investigation, Abaqus is utilized. To precisely represent the fluid (reservoir) and the solid components (foundation and dam), a two-dimensional framework is implemented, employing CPS4R and AC2D4 elements. Four nodes, each having two degrees of flexibility, comprise the CPS4R element, which facilitates translation along the x and y axes. A pressure degree of freedom is designated to each of the four nodes of the AC2D4 component, which functions as an acoustic element. A non-reflective upstream boundary condition is enforced to enhance energy dispersion in a reservoir and maintain a constant volume in the fluid field. To mitigate the influence of Abaqus boundaries on the system's behavior, the water tank and foundation dimensions are established as being 1.5 times greater in length than the highest point of the dam [24, 25]. By applying a boundary condition, the pressure is simultaneously reduced to zero at the highest level of the reservoir and the truncated boundary. In static analysis, horizontal displacements at the dam-foundation interface are restricted, while in dynamic analysis, both horizontal and vertical motion degrees of freedom are considered. It is assumed that the fluid is compressible and inviscid, and that the foundation and dam are isotropic, elastic and homogeneous. For the finite element model, the mesh configuration is depicted in Figure 2.

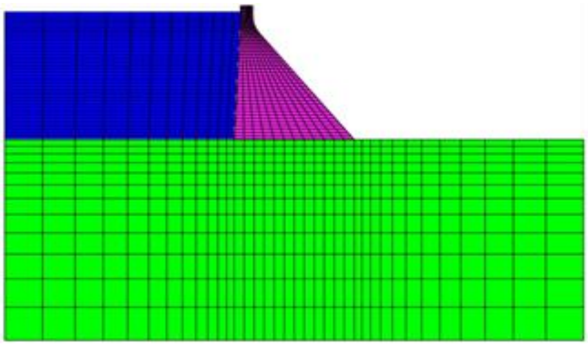


Figure 2 Abaqus Model Represents the System Consisting of the Dam, Foundation, and Reservoir.

2.2 Validating the Model with Pine Flat Dam

The pine flat dam, illustrated in Figure 1, is a concrete gravity dam with a crest length of 560.83 meters. To evaluate the precision of the constructed Abaqus model, the section in this investigation that has the greatest height, and no excess is selected, as illustrated in Figure 3. The physical attributes of the dam, reservoir, and foundation rock are summarized in Table 1. The dimensions of these components are illustrated visually in Figure 3. A cumulative sum of 972 CPS4R elements is allocated to represent foundation rock and the dam in the Abaqus model. In contrast, 504 AC2D4 elements are utilized to depict the reservoir. The following parts will undertake a comparison of the findings obtained from the generated finite element model and those recorded in previous academic publications.

Table 1 Properties of Foundation Rock, Dam and Reservoir Materials.

Parameter	Rock	Concrete	Water	Unit
Density (ρ)	2643	2430	1000	kg/$\mathbf{m^3}$
Elastic modulus (E)	22.4	22.4	-	GPa
Compressive strength (f_c')	-	28	-	MPa
Tensile strength	-	2.39	-	MPa
Bulk modulus	-	-	2.07	GPa
Poisson's ratio (ν)	0.333	0.2	-	-

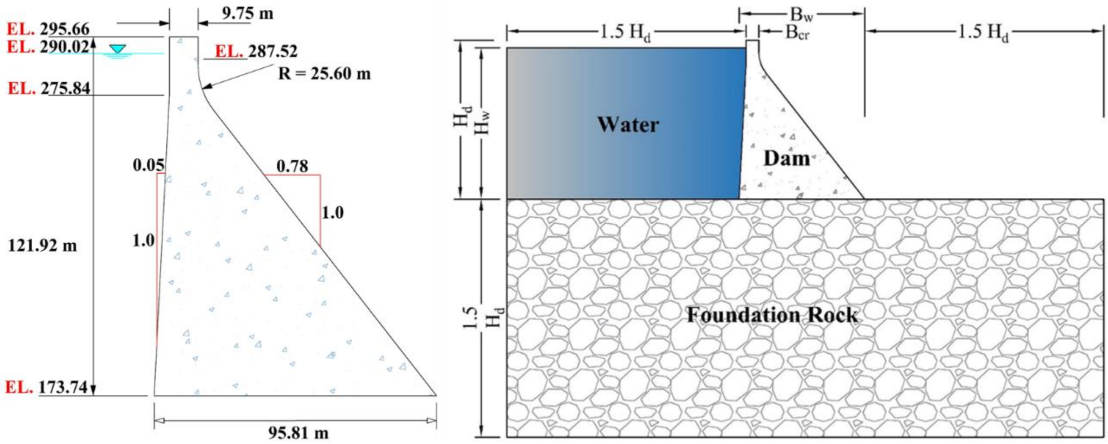
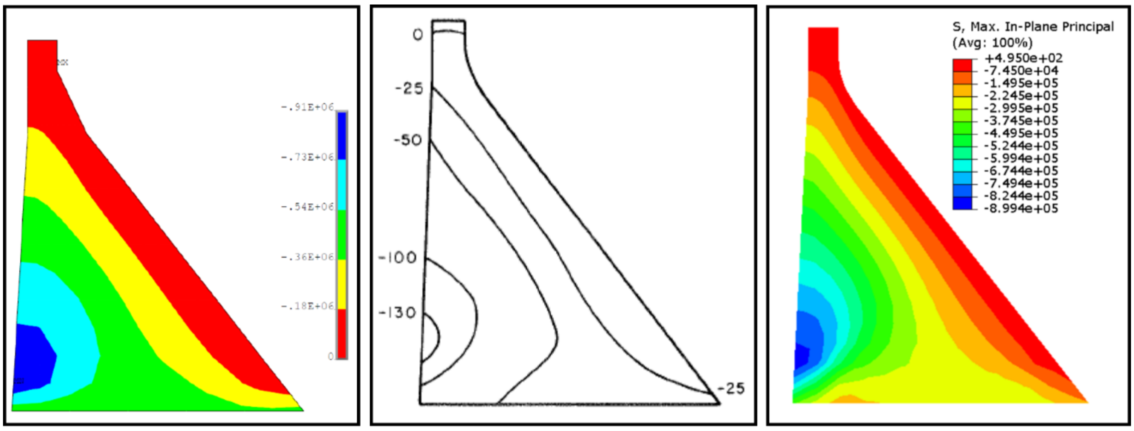


Figure 3 Pine Flat Dam's Dimensions.

3 Results and Discussion

3.1 Static Analysis

The analysis of the concrete dam, reservoir, and foundation system was conducted to account for the static gravity effect. The obtained results were compared with the findings presented in the publication by [23, 24]. The impact of the flexible foundation and the force of hydrostatic pressure that the reservoir's water exerts are both considered in this analysis. Figure illustrates the contour plots of the maximum principal stress. The maximum stress level observed in this study, which is -0.899 MPa (-130.3 psi), closely aligns with both the reference value of approximately -0.896 MPa (-130 psi) reported by Chopra and the reference value of approximately -0.91 MPa (131.9 psi) reported by Ufuk Sen. The region where the highest magnitude of primary stress is detected displays resemblances to the findings documented in the references.



Ufuk Sen (ANSYS) **Chopra and Fenves** **This Study (ABAQUS)**

Figure 4 Maximum Principal Stress Contour Plots.

3.2 Analysis of A Modal

The dam-foundation-reservoir system's natural frequencies are determined via modal analysis and compared to the findings provided by [23, 24]. To exclude the influence of inertia and damping on the foundation and concentrate exclusively on its flexibility, it is customary in modal analysis to presume that the foundation possesses negligible mass [26]. Experimental evidence has shown that the implementation of a massless foundation does not significantly alter the vibrational properties of a dam [26]. Four different situations are utilized to conduct modal analysis on the dam-reservoir-foundation-rock system: Case 1 consists of a rigid foundation and an empty reservoir; Case 2 consists of a rigid foundation and a full reservoir; Case 3 consists of a flexible base and an empty water tank; and Case 4 consists of a flexible base and a full water tank. The initial mode structures of the four cases are depicted in Figure 5, whereas the fundamental frequencies for each are listed in Table 2. The initial four modulation frequencies and their respective structures for the Abaqus model are illustrated in Figure 6. These conditions pertain to a fully operational reservoir and a flexible foundation. Using Equation (1), one can determine the percentage difference:

$$\text{Difference}\ (\%) = \left|\frac{f_{apr} - f_{ex}}{f_{ex}}\right| \times 100 \qquad (1)$$

Where f_{ex} and f_{apr} are the frequency from the reference study and the approximations of frequencies derived from the present investigation, respectively.

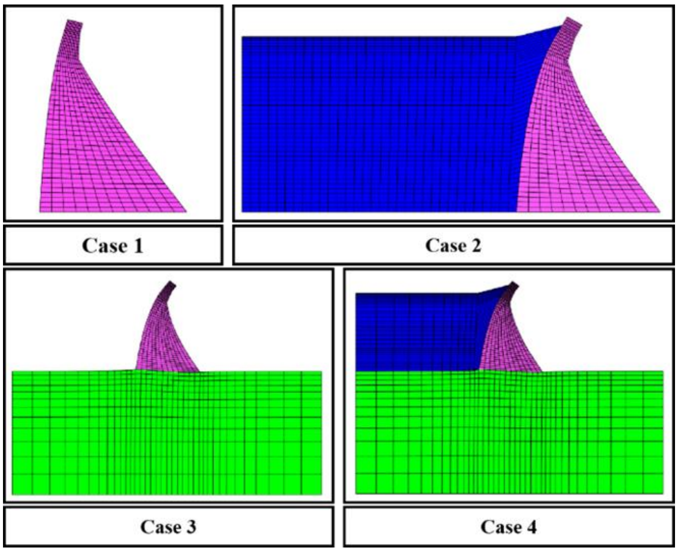


Figure 5 Mode Shapes of an Abaqus Model for Four Distinct Scenarios Utilizing Abaqus.

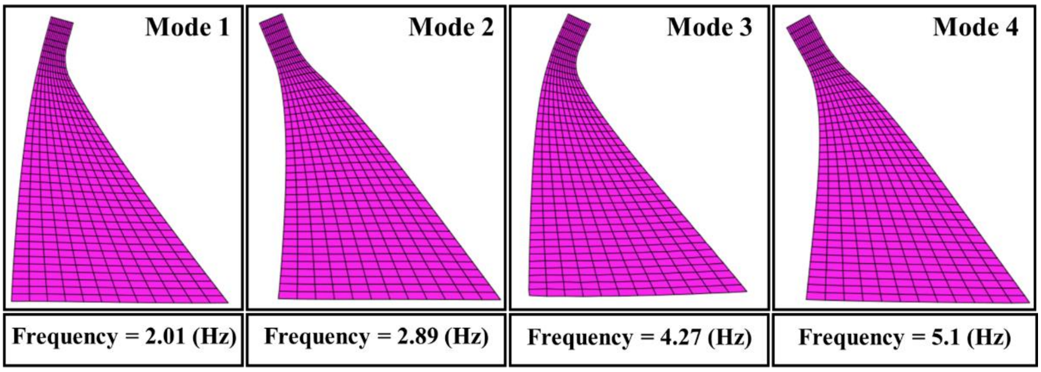


Figure 6 The Abaqus Model, Featuring a Flexible Foundation and a Full Water Tank, Exhibits its Initial Four Natural Frequencies.

Table 2 Primary Frequencies of Abaqus Model for Four Distinct Situations.

Case No.	Reservoir	Foundation	Frequency (Hz)			Difference (%) with Sen	Difference (%) with Fenves
			Fenves and Chopra	This study (Abaqus)	Ufuk Sen (Ansys)		
1	Empty	Rigid	3.14	3.15	3.16	0.3165	0.31847
2	Full	Rigid	2.54	2.53	2.56	1.1719	0.3937
3	Empty	Flexible	2.59	2.64	2.48	6.45161	1.9305
4	Full	Flexible	2.03	2.01	2.02	0.495	0.9852

3.3 Dynamic Linear Analysis

The present analysis incorporates ground motion using the without-mass foundation input method, disregarding the impact of foundation inertia. As a result, only the foundation's flexibility effects are incorporated into the system, which prevents the artificial amplification of ground motion in the free-field. Nevertheless, the massless foundation lacks dampening, hence disregarding the waste of energy facilitated by the foundation. Hence, it is imperative to employ mathematical equations to incorporate damping in order to achieve more precise outcomes [26]. The formulation proposed by reference [27] was utilised in this study. A recent discovery has revealed that employing distinct viscous damping ratios for both the dam and foundation rock can result in an excessive amount of damping for the entire system [27]. Hence, it is advisable to refrain from employing this method while examining dam-water-foundation systems. This analysis also considers the dampening impact of the materials at the bottom of the reservoir. The concrete dam is assumed to have a damping ratio of 5% ($\boldsymbol{\xi_1}$), and the parameters in Equation (2) are determined by referring to the tables in Reference [27]. In this part, a damping ratio of 14.4% was deliberately chosen to validate the created model. The results were compared with those in Ref [23], which also employed the same damping ratio as shown in Equation (2).

$$\tilde{\xi}_1 = \frac{1}{R_r}\frac{1}{(R_f)^3}\xi_1 + \xi_r + \xi_f \qquad (2)$$

$\mathbf{R_r}$ denotes the ratio by which the period lengthens as a result of the interaction between the dam and the water. $\mathbf{R_f}$ denotes the ratio by which the period lengthens as a result of the interaction between the dam and the foundation. The damping ratio $\boldsymbol{\xi_1}$ represents the damping coefficient for the dam when it is supported by a rigid foundation and the reservoir is empty. The damping of a dam supported by impounded water on a flexible foundation is denoted by the damping ratio $\tilde{\boldsymbol{\xi}}_1$. $\boldsymbol{\xi_r}$ denotes the additional damping coefficient that arises due to the interaction between the dam and the water in the reservoir. $\boldsymbol{\xi_f}$ represents the additional damping caused by the interaction between the dam and the foundation.

The Rayleigh damping coefficients that are implemented on the system are denoted as α and β, representing stiffness-proportional damping and mass-proportional damping, respectively. Equations (3) and (4) are used to calculate these coefficients [28]:

$$\alpha = \xi \frac{2\omega_i\omega_j}{\omega_i+\omega_j} \quad (3)$$

$$\beta = \xi \frac{2}{\omega_i+\omega_j} \quad (4)$$

The fundamental frequencies of the dam and foundation system are represented by ω_j and ω_i, when j and i correspond to the j^{th} and i^{th} frequencies, accordingly. The Rayleigh dampening parameters are computed in this investigation by utilizing both the initial and fifth essential frequency ranges.

In order to consider the gravitational force acting on the dam, an acceleration of 9.81 m/s^2 in the vertical direction was incorporated into the system. At the onset of the time history analysis, the acceleration was established as an initial condition and persisted consistently during the earthquake investigation. Put simply, the dam's own weight was treated as a fixed starting condition for the investigation of its behaviour over time.

3.3.1 Ground Motion

In order to make a comparison with the data obtained from [23, 24], we have chosen the Kern County earthquake that occurred on 21 July 1952 and was recorded at Taft Lincoln School Tunnel [29]. The earthquake was simulated by applying the free-field ground acceleration, incorporating both the vertical component and the horizontal component. Figure 7 and Figure 8 depict the two components, which were implemented using the Abaqus software programme. It must consider the disparity in elevation between the nodes located at the base and summit of the dam. Figure 9 depicts the displacement-time graph of the lowest point (heel) and highest position (crest) at the base of the dam, where the acceleration of the ground motion is applied. The gap in the outcomes is negligible and may be ignored.

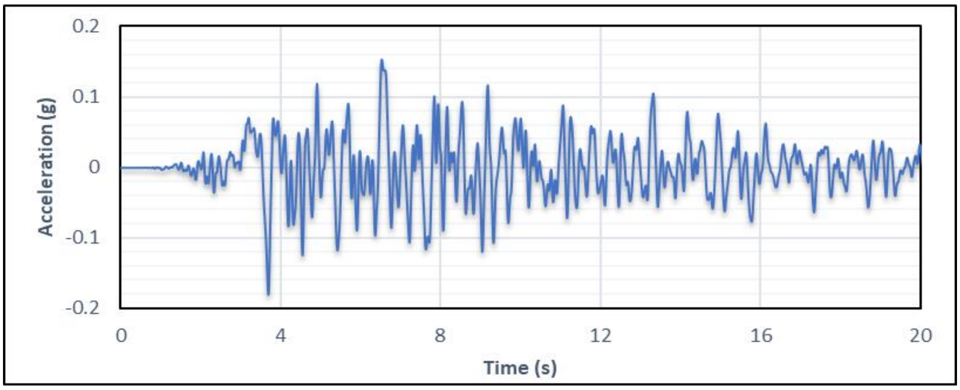


Figure 7 Unscaled Horizontal Component of the Taft Earthquake.

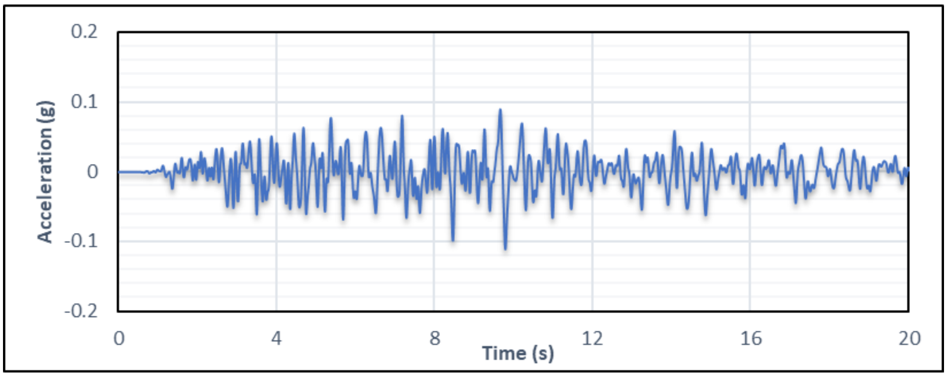


Figure 8 Taft Earthquake's Vertical Component, Unscaled.

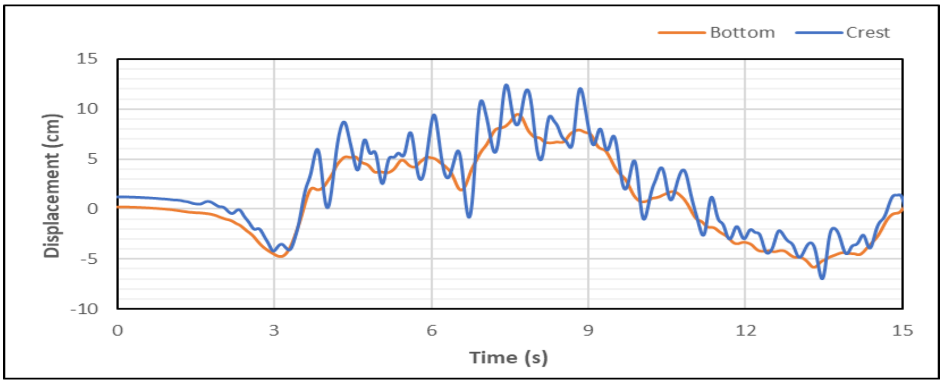


Figure 9 Temporal Variation of Displacement for the Heel and Crest Nodes of the Dam.

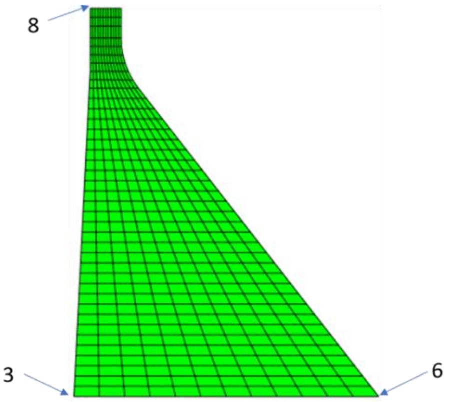


Figure 10 Nodal Point Coordinates at the Dam's Toe (6), Heel (3), and Crest (8).

3.3.2 Implications of Dynamic Analysis

A comparison is made between the findings obtained from the dynamic evaluation and the displacement-time history that is referenced [23, 24]. These comparisons are presented in Figure 11 through Figure 16. In Figure 17, (a) a contour plot is shown that displays the maximum principal stress at 6.99 seconds. This is where the highest value of the principal stress is observed. Furthermore, Figure 17 (b) exhibits a contour plot illustrating the least principal stress at 6.72 seconds. It is worth noting that the largest value of the minimum principal stress occurs at nodal point 3. Figure 18 (a) shows a graphical representation of the maximal primary stress, with the highest value recorded at 6.99 seconds. In the same way, the contoured representation of the minimal primary stress is illustrated in Figure 18 (b), revealing that it reaches its greatest value at nodal point 6, precisely at 6.99 seconds. To determine the percentage of error (%), the initial five instances of the displacement-time history diagrams are scrutinized for their maximum and minimum peak values. For each nodal point, the sums of the absolute values of the vertical and horizontal displacement components are presented in **Error! Reference source not found.**. The error percentage is determined by utilizing Equation (1).

Table 3 Percentage Error of Displacement-Time History Diagrams.

Nodal Point	**Horizontal Displacement**			**Vertical Displacement**		
	This study (cm)	**Fenves and Chopra (cm)**	**Error (%)**	**This study (cm)**	**Fenves and Chopra (cm)**	**Error (%)**
8	33.23	29.52	12.56	7.2	8.30	13.25
3	1.71	1.74	1.72	2.64	2.18	21.1
6	1.88	2.01	6.46	1.3	1.09	19.27

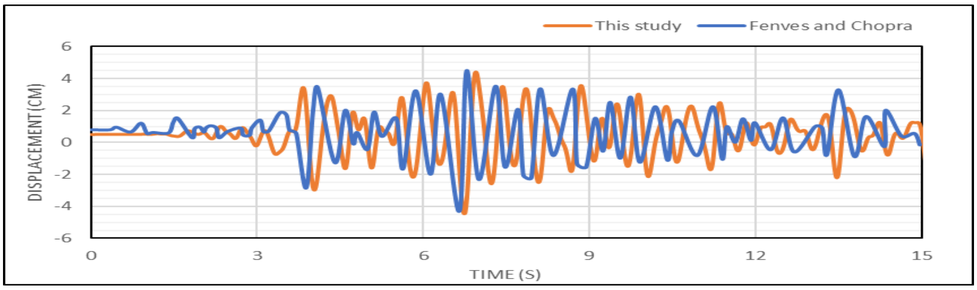


Figure 11 Crest Horizontal Displacement (Nodal Point 8)

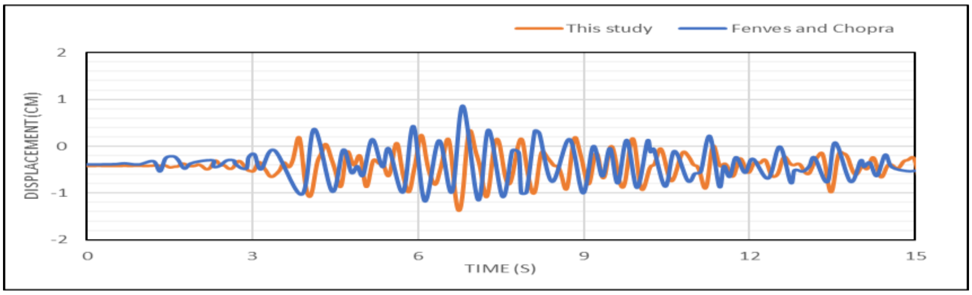


Figure 12 Crest Vertical Displacement (Nodal Point 8)

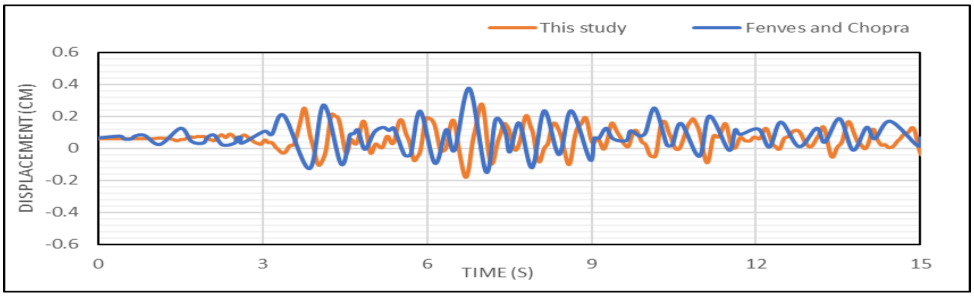


Figure 13 Heel Horizontal Displacement (Nodal Point 3)

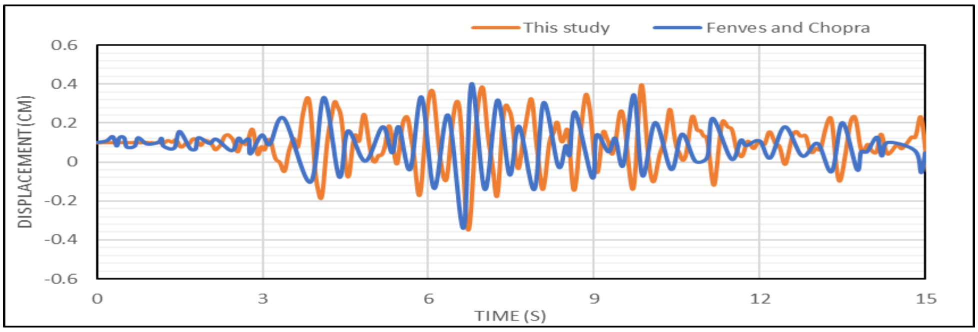


Figure 14 Heel Vertical Displacement (Nodal Point 3)

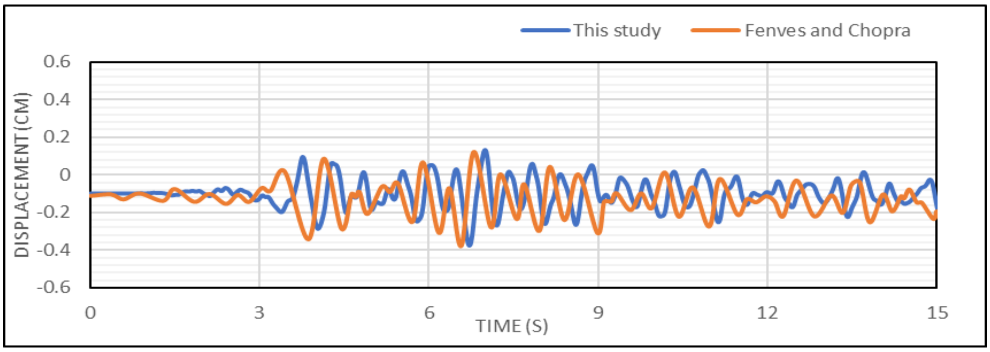


Figure 15 Toe Horizontal Displacement (Nodal Point 6)

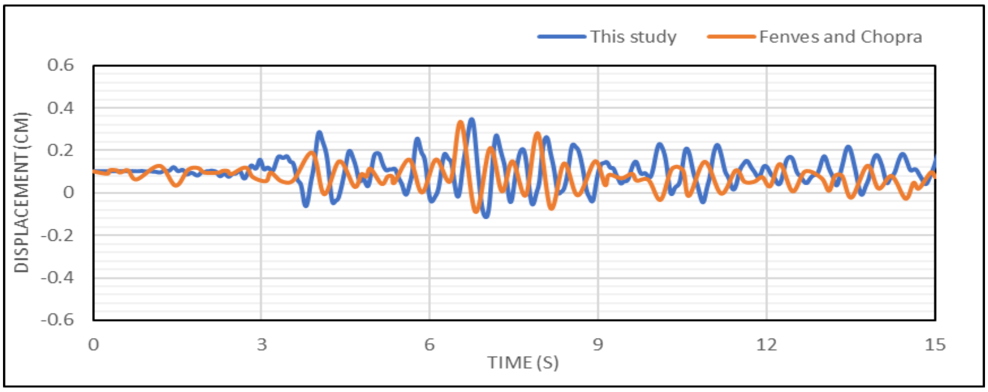


Figure 16 Toe Vertical Displacement (Nodal Point 6)

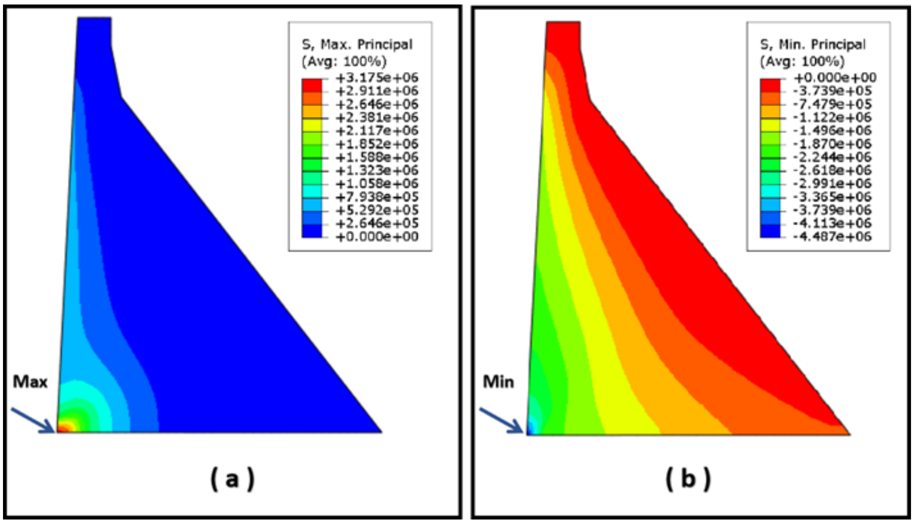


Figure 17 (a) Plot Contour of Highest Primary Stress at time 6.99 s, and (b) Plot Contour of Smallest Primary Stress at time 6.72 s (in Pa).

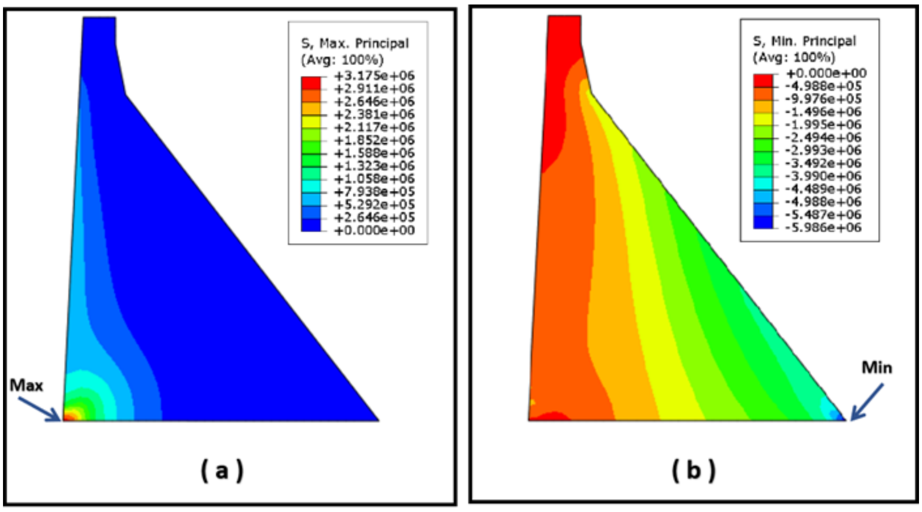


Figure 18 (a) Plot Contour of Highest Primary Stress at time 6.99 s, and (b) Plot Contour of Smallest Primary Stress at time 6.99 s (in Pa).

3.4 Dynamic Nonlinear Analysis

Based on the linear analysis findings, seismic activity with a peak ground acceleration of 0.18 g leads to tensile strains of approximately 3.17 MPa in the dam's foundation. This exceeds the specified tensile strength of 2.39 MPa, as indicated in Table 1, considering a compressive strength of 28 MPa. Consequently, these elevated stress levels can potentially cause structural damage, such as the formation of cracks and displacement of the dam's foundation. Furthermore, it is unlikely that the cohesion between the dam and the foundation can withstand such high stress levels. To account for the nonlinearity observed in the system, the Abaqus program is utilized to simulate contact elements between the dam and foundation. This modeling approach incorporates the Coulomb-Mohr friction equation, where the contacting surfaces can resist shear stress until a specific threshold is exceeded, triggering relative sliding motion. The shear stress capacity is precisely defined by Equation (5):

$$\tau_{\mathrm{lim}}(t) = \sigma_n(t) \tan\varphi + c \qquad (5)$$

The variables $\sigma_n(t)$ and $\tau_{\mathrm{lim}}(t)$ denote the normal stress and shear capacity, respectively, at a specific time t. The friction angle is denoted by the parameter φ, whereas the cohesion of the interface between the dam and the foundation rock is represented by c. A surface sliding phenomenon arises when the shear stress surpasses the capacity. In a "standard contact relationship" is represented the contact surface's behaviour in the normal direction. Normal pressure can be transmitted between interfacing surfaces prior to the occurrence of opening in this form of contact. Otherwise, when the tensile stress exceeds the tensile strength of the material, these contact elements will begin to open, despite their ability to transmit normal compressive stress.

In order to validate the accuracy of the contact model and juxtapose its findings with those of Reference [30], a dynamic evaluation is conducted on the Abaqus madel utilizing the information presented in Table 4. In the dam-water-foundation rock system, just the horizontal component of the Taft Earthquake, scaled to 0.4 times the Peak Ground Acceleration (PGA), is applied. Both the dam and the foundation rock are subject to a Rayleigh damping coefficient of 5%. The modulus of elasticity of the foundation rock is 0.25 times that of the dam. Figure 19 depicts the dam's sliding displacement response.

Table 4 Material Attributes of the Abaqus Model with the Base Sliding.

Material characteristic	Fenves & Chavez
The foundation rock's modulus of elasticity (GPa)	5.6
the dam's modulus of elasticity (GPa)	22.4
Coefficient of friction	1
Cohesion (MPa)	0

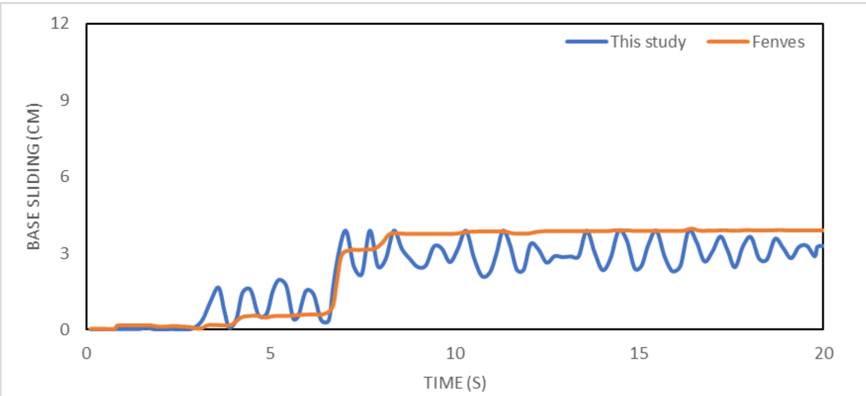


Figure 19 Abaqus Model Response with Base Sliding.

In order to assess the principal stresses in relation to the outcomes of the linear evaluation depicted in Figure 17 and Figure 18, a nonlinear dynamic analysis was conducted utilizing the data from Table 1 regarding the material properties. The vertical and horizontal unscaled components of the Taft Earthquake were applied concurrently to the Abaqus system comprising the dam, water, and foundation. The resulting principal contoured representations are shown in Figure 20 and Figure 21. The analysis revealed that the highest principal stress at node 3 (heel) significantly decreased, going from 3.17 MPa to 1.31 MPa. A significant difference was not observed in the lowest principal stress, however. Additionally, it was observed that the maximal principal stress had transitioned from the dam's base to its neck.

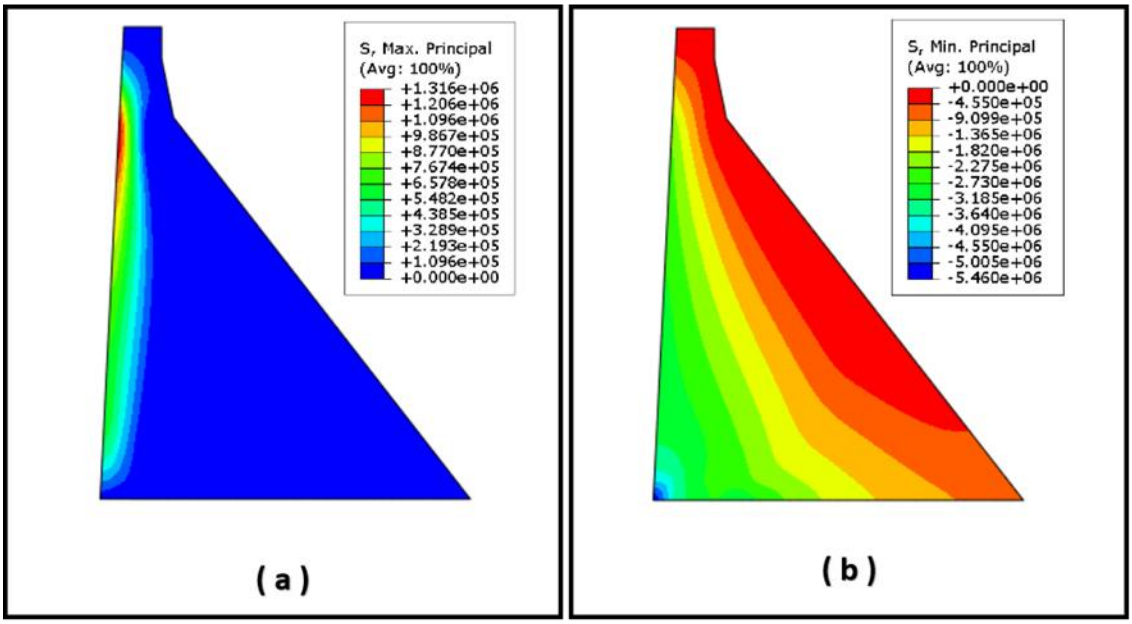


Figure 20 (a) Contour Plot of the Highest Principal Stress at 6.95 s, and (b) Contour Plot of the Lowest Principal Stress at 5.85 s (both in Pa).

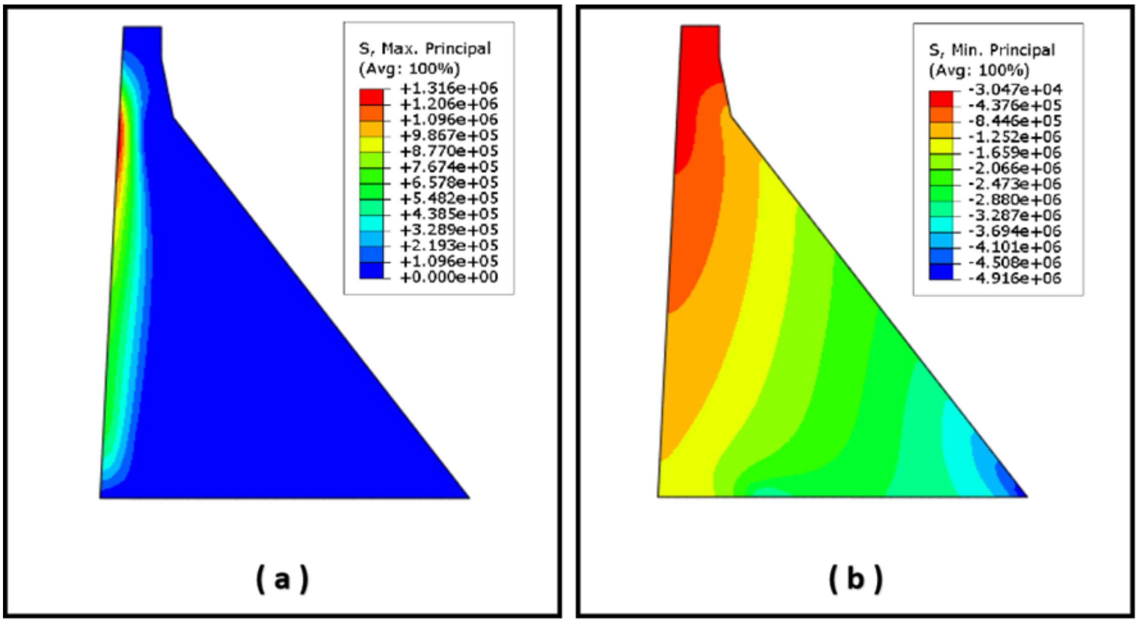


Figure 21 (a) Contour Plot of the Highest Principal Stress at 6.95 s, and (b) Contour Plot of the Lowest Principal Stress at 7.03 s (both in Pa).

4 Conclusion

This paper presents an exhaustive Abaqus -based numerical analysis of the seismic load analysis of the Pine Flat Dam in California, United States. The study integrates modal, static, and dynamic analyses while considering flexible foundations and hydrostatic forces. Using earthquake data, the dam was subjected to dynamic linear and nonlinear analyses, allowing for a comparison of results to improve the design of new dams and evaluate the seismic safety of existing dams.

The dynamic linear analysis yielded a maximal principal stress value of 3.17 MPa, while the dynamic nonlinear analysis produced a value of 1.31 MPa. Although no substantial variation was observed in the lowest principal stress, it was noteworthy that the highest principal stress had transitioned from the base to the neck of the dam. These findings highlight the significance of precise analytical procedures in ensuring the safety of dams and offer valuable insights into the behaviour regarding concrete gravity dams when subjected to seismic forces.

In addition, the study emphasizes the significance of reservoir water effects, and foundation flexibility in dam safety analysis. By addressing these factors, the paper contributes to the existing corpus of knowledge regarding the safety and stability of dams under seismic loads. The research findings highlight the need for a comprehensive understanding of these factors in order to effectively design and evaluate the structural resistance to seismic activity.

In conclusion, this study substantially improves our knowledge of the seismic load analysis of the Pine Flat Dam. Our ability to design safer dams and evaluate the seismic safety of existing dams is bolstered by the exhaustive numerical analysis, which incorporates a variety of analytical techniques and considers crucial factors. The paper's findings provide significant insights into the behaviour of concrete gravity dams subjected to seismic loads, which have implications for the broader field of dam engineering.

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