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ARTICLE

A Comparative Study between Pseudo-static and Dynamic Analyses of Keddara Dam

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ABSTRACT

The numerical analysis of static and dynamic performance of embankment dams using the finite difference approach is a complex procedure that takes into account material behaviour, soil-dam foundation interaction, hydraulic conditions, and saturation effects. In this study, a numerical analysis using the finite difference method (FLAC 3D) is used to conduct a static and dynamic analysis of the Keddara earthen dam in the Boumerdes region of Algeria, with the goal of defining its behaviour in terms of settlement, deformation, and pore pressure variation during construction and operation. There are two steps to consider: dam construction and water filling. For the static analysis, two mathematical models are considered: the elastic model and the Mohr-Coulomb model. An actual earthquake record is used to conduct a coupled dynamic analysis, and the interaction between the fluid and solid phases is taken into account. Maximum displacement, acceleration, and pore pressure remain inconsequential for dam instability; horizontal and vertical displacements increase with distance from the dam body's base to the top.

1. Introduction

Embankment dams are major water impoundment facilities. They are most likely the earliest man-made hydraulic structures. They are dams that are built with natural materials mined locally. Earthfill or rockfill dams are the most common types. The various structure sections of an earth dam contain a variety of materials. Earth dams are generally prone to displacement during construction, following water filling, and during operation [1-3]. Earth dams are extremely vulnerable to seismic response issues, and earthquake loading can result in severe consequences ranging from economy to direct loss of life.

The ancient method and seismic criteria were used to design the old earth dam. The safety of these structures is now jeopardised due to noncompliance with current seismic safety recommendations [4,22]. Dams should operate without endangering the population in the event

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of an earthquake, and similarly under static loading. As a result, the stability of dam embankments, whether static or dynamic, remains a major concern for geotechnical engineers. Dam security is related to the strength of the dam body and foundation, permanent deformation generating cracks, slope stability, excess pore water pressure in embankments, and foundation materials [5]. According to a literature assessment, the most common causes of earth dam collapses are overtopping and internal erosion [6]. Significant progress has been made in understanding dam behaviour, particularly during seismic action. The development of numerical techniques for static and dynamic analysis was largely responsible for the progress [7].

Different methods can be used to analyse the stability of embankments and earthen dams subjected to seismic loads [23]. The methodologies have advanced greatly in recent years, and they range from simplified to detailed. Gazetas (1987) [8] compared techniques for calculating the seismic response of earthen dams subjected to seismic action and defined their advantages and limitations. Simplest approaches require few parameters for the analysis and are based on empirical correlations and simpler procedures. Solutions in decoupled linear-equivalent analysis, as well as finite element and finite difference formulations in coupled or decoupled nonlinear analysis [9,4], are detailed analyses. Simplified approaches are used to analyse small dams or to justify the use of extensive analysis for major dams. The pseudo – static method was widely utilized to assess the seismic stability of embankment dams. This method reduces the dynamic behavior to static forces. It involves replacing the seismic action with a single force and calculating the factor of safety for the sliding block that is limited by the critical failure plane. Simultaneously, the shear beam technique proved to be efficient in dynamic analysis. The approach took into account earthquake vibrations in several directions, including transverse, vertical, and longitudinal [8,10]. For additional analysis, analogous linear or non-linear analyses are used [11].

The goal of this research is to use a numerical method to examine the static and seismic response of the Keddara dam in Algeria. The analysis focuses on the effect of the mathematical model on the static analysis response of the dam during the building phase and water filling phase. The elastic model and the Mohr-Coulomb model are used in the analysis. First, the elastic model is used for the foundation and the Mohr-Coulomb model for the dam body (model (1)). Second, for both the foundation and the dam body (model (2)), the analysis is carried out using the Mohr-Coulomb model. Finite difference formulations in coupled nonlinear analysis are used for the dynamic analysis during the water filling phase. The obtained displacements, deformation, and pore pressure evolution data are discussed.

2. Description of Keddara Dam

The Keddara dam is a rockfill embankment built between 1982 to 1985. It is situated in the Boumerdes region, approximately 35 kilometers east of Algiers. The site is located in the Tellian Atlas coastal range, at 36.65° North latitude and 3.43° East longitude, near the northern edge of the Mitidjian Atlas. This dam has a capacity of 142.39 million cubic meters and a height of 108 meters. Figures 1 and 2 depict the dam's geometry and position, respectively.

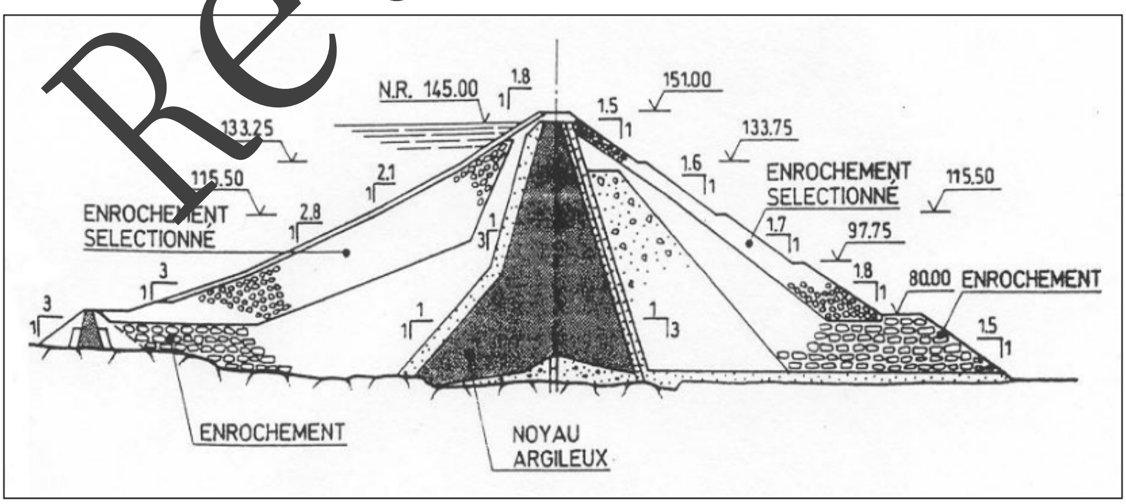

Figure 1. Geometry of Keddara dam (ANB 1987) [12].

Based on site investigations, the geotechnical parameters were obtained [12]. As shown in Table 1, and Figure 3, materials of the dam have been subdivided into six zones. The upstream part consists of limestone rockfill (Zone 1) and a transition filter (Zone 2). The downstream part consists of limestone rockfill (Zone 1), a sand filter (Zone 4) and a gravel backfill (Zone 5) and in the center clayey soil as core materials (Zone 3). The foundation of the dam is composed of stiff schist materials (Zone 6). The characteristics of materials in each zone of the dam are summarized in Table 1, where γd is the dry density, c' is the effective cohesion, υ' is the effective angle of internal friction, G is the shear modulus and K is the bulk modulus.

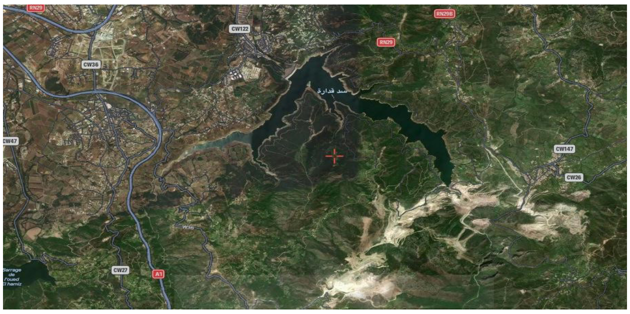

Figure 2. Location of Keddara dam on the Boudouaou River in Boumerdès Province, Algeria. (Captured by satellite pro).

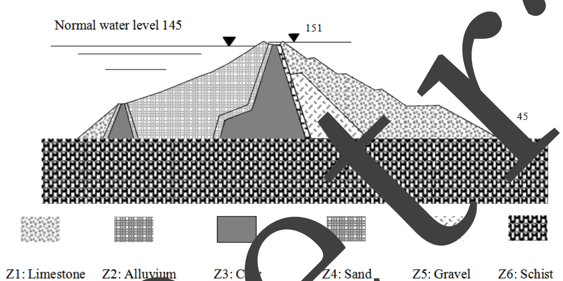

Figure 3. Different zones of the dam.

3. Numerical Modeling

The dam analysis is conducted using the finite difference program FLAC-3D, based on a continuum finite difference discretization using the Lagrangian approach. FLAC (Fast Lagrangian Analysis of Continua) [13], was developed by ITASCA group, to solve geotechnical problems. As reported in the manual program: "Every derivative in the set of governing equations is replaced directly by an algebraic expression written in terms of the field variables (e.g. stress or displacement) at discrete point in space. For dynamic analysis, it uses an explicit finite difference scheme to solve the full equation of motion using lumped grid point masses derived from the real density surrounding zone". Figure 4 shows the three-dimensional mesh used for the Keddara dam analysis consisted of 1235961 elements.

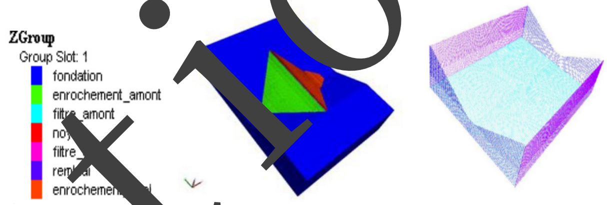

Figure 4. 3D Modeling of Keddara dam and Boundary Condition

The construction of the dam embankment is modeled layer by layer with a same thickness, starting from the upstream side to the downstream side, and up to the crest (151 NGA). The first embankment layer is modeled after foundation analysis (equilibrium state). The appropriate boundary conditions in modeling and grid were selected according to the FLAC3D manual program guidelines. Boundary conditions are imposed along the lateral borders and the base of the model, such as the displacements are zero, and the nodes on the foundation external surface are fixed on the three directions X, Y, Z.

Mohr-Coulomb plasticity model is used for sandy

Table 1. Geotechnical Soils Properties [15]

Zones		Materials	γ_d [kN/m^3]	c' [kPa]	υ'[°]	K [MPa]	G [MPa]
Zone 1	Rockfill	Selected limestone	25	0	45	9.6×10^6	5.8×10^6
Zone 2	Transition filter	Alluviums	18	50	13	260×10^3	120×10^3
Zone 3	Core	Colluvial clay	20	55	14	253.5×10^3	117×10^3
Zone 4	Drain	Sand	18	0	35	433.3×10^3	200×10^3
Zone 5	Backfill	Gravel materials	18	0	40	274.67×10^3	160×10^3
Zone 6	Foundation	Schist	28	0	45	2.67×10^7	1.6×10^7

and clayey materials that yield when subjected to shear loading. The criterion depends on minor and major principal stresses as presented in Equation (1).

$$F(\sigma'_{ij}) = |\sigma'_1 - \sigma'_3| - (\sigma'_1 + \sigma'_3)sin\varphi' - 2c'cos\varphi' = 0. \quad (1)$$

σ'_1 and σ'_3 are the principal stresses and υ is the angle of internal friction.

Mechanical parameters of Mohr-Coulomb criterion are: E (Young's Modulus), υ (Poisson ratio), c (cohesion), υ (angle of internal friction) et ψ (dilatancy angle). Mohr-Coulomb model parameters are obtained from laboratory tests. C and φ are calculated in the Mohr plane (σ, τ) using the stress states at failure.

The shear strength equation for saturated soils is expressed as a linear function of effective stress as:

$$r = c' + (\sigma - u)tg\ \varphi'. \quad (2)$$

c' is the effective cohesion, υ' is the effective angle of internal friction, σ is the total normal stress in the plan of failure, and *u* is the pore water pressure.

In FLAC 3D, the Mohr-Coulomb model is the conventional model used to represent shear failure in soils. The model is expressed in terms of the principal stresses (s_1, s_2, s_3), which are the three components of the generalized stress vector of this model. The components of the generalized strain vector are the principal strains e_1, e_2 and e_3. The increment expression of Hooke"s law in terms of the generalized stress and stress increments has the form [13,24,25]:

$$\begin{cases} d\sigma_1 = \alpha_1 d\varepsilon_1 + \alpha_2(d\varepsilon_2 + d\varepsilon_3) \\ d\sigma_2 = \alpha_1 d\varepsilon_2 + \alpha_2(d\varepsilon_1 + d\varepsilon_3) \\ d\sigma_3 = \alpha_1 d\varepsilon_3 + \alpha_2(d\varepsilon_1 + d\varepsilon_2) \end{cases} \quad (3)$$

where $\alpha_1 = K + 4G/3$, $\alpha_2 = K - 2G/3$. a_1 and a_2 are the material constants defined in terms of the shear modulus G and bulk modulus K.

For rockfill dams, it is recommended to consider an average shear modulus, function of the depth of the dam from the crest [illegible,8]:

$$G(z) = G_b \left(\frac{Z}{H}\right)^m \quad (4)$$

G_b is the average shear modulus at the base. According to Gazetas (1987) [illegible] m is taken equal to 2/3. Noting that different formulas were used to determine the shear modulus according to the type and density of materials [15,16,24].

In the construction stage, the analysis is conducted using the elastic model for the foundation and Mohr-Coulomb model for the dam body (model (1)). Therefore, the linear elastic law (Hooke's law) is used considering the mechanical properties; Young's Modulus E and Poisson ratio υ. Due to the nature of the body dam materials, the elastoplastic Mohr-Coulomb model is used. In this model, the behavior is assumed first elastic and then plastic. In addition to the elastic deformation defined by Hooke's law, the elastoplastic models integrate a permanent plastic deformation.

4. Dynamic Analysis

The research is carried out with the 2003 Boumerdes earthquake record in order to examine the dam response to real earthquake action. The Boumerdes earthquake, with a magnitude of Mw = 6.8 and an intensity of I = X, killed around 2000 people and injured 11,000 others [17]. According to the Algerian Research Center of Astronomy, Astrophysics, and Geophysics, the epicentre was located around 20 kilometres from the Keddara dam at 36.91° N and 3.58° E, at a depth of 8 kilometres to 10 kilometres (CRAAG).

The Boumerdes earthquake had a peak acceleration of 0.202 g and a length of around [illegible]0 seconds. Figure 5 depicts the record. The frequency of the primary peak is 13.3 Hz, as illustrated in Figure 5. The Mohr-Coulomb creteri on is used to undertake the dam's elastoplastic analysis for both the dam core and the foundation (model (2)). The analysis performed is a comprehensive coupled analysis that takes into account effective stresses, an anisotropic fluid model for the core, and an isotropic model for the dam's other materials. The interaction between the fluid and solid phases is considered in the study. The main objective is to investigate the influence of water filling on the dam response to real earthquake, and to compare the numerical results to the real behavior of the dam to the 2003 earthquake.

Except for the foundation, the water flow in the dam is taken into account when doing a coupled analysis. As a result, the core permeability is taken to be k1 =1.10~12 m/s, k2 =1.10~12 m/s, and k3 =1.10~13 m/s in the directions x, y, and z, with the porosity n=0.8.

5. Results and Discussions

5.1 Vertical Displacement

The numerical analysis allows for the measurement of vertical dam displacement during construction and after water filling. According to the two models (1) and (2), vertical displacement in the dam at the conclusion of the construction stage and before water filling is shown in Figures 6 and 7. The dam core is where the most settlement is induced (Zone 3). Despite the comparable distribution of settlements across models (1) and (2), model (l) (Elastic for the foundation and Mohr-Coulomb for the dam body) has a bigger induced settlement of 9.44 cm compared to 8.86 cm for model (2). (Mohr-Coulomb for the foundation and the dam). However, according to Figure 8, the displacement variation with the dam axis shows an increase of displacement at the top with 22 cm and 20 cm respectively for the models (1) and (2). Figure 8(a)-(b) shows that the induced settlements variation, according to models (1) and (2), with the dam axis, in stage of construction and after water filling, is no significant. A slight variation is noted in the middle (h/H = 0.5) and the top (h/H = 1) of the dam.

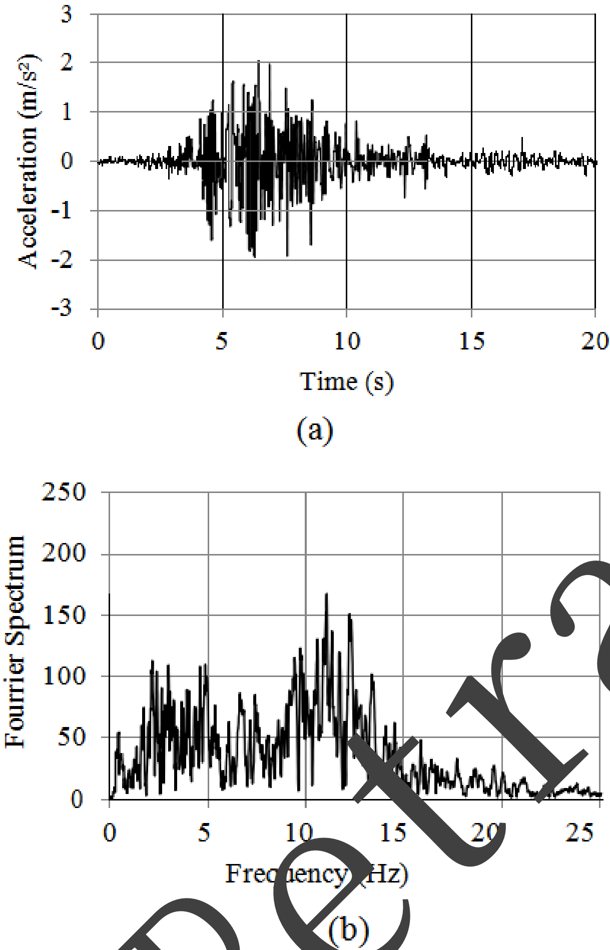

Figure 5. (a) Boumerdes Earthquake Record (b) Fourrier Spectra

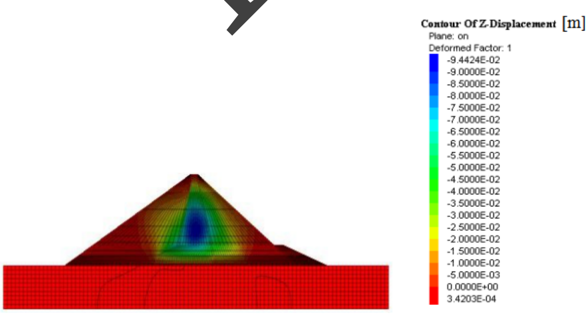

Figure 6. Settlement [m] of the Dam According to Model (1)

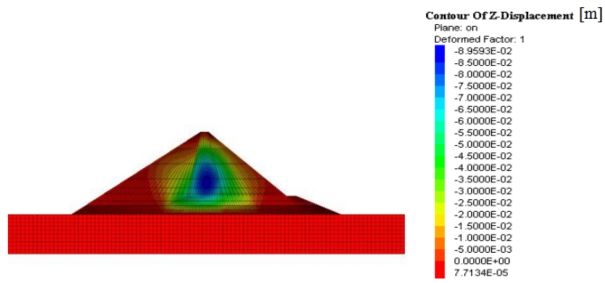

Figure 7. Settlement [m] of the Dam According to Model (2)

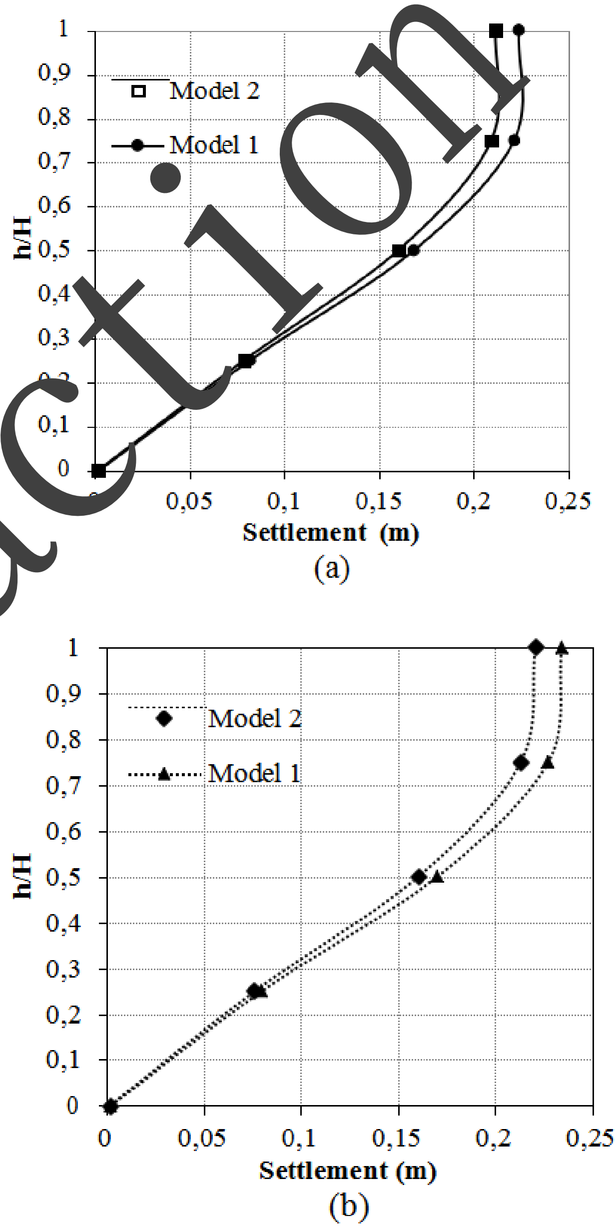

Figure 8. Comparison of settlements with the dam axis according to model (1) and Model (2) (a) before water filling, (b) after water filling.

5.2 Comparison with Settlements Recorded on Site

The dam filling water began by the end of 1985. Using dam tools instrumentation for data monitoring consisted of magnetic inclinometers/taseometers (listed from 1 DZ to 12 DZ as shown in the legend of Figure 9), fixed at the top of the dam, on the core and the downstream and upstream recharge, settlements and deformation of the structure were recorded. The crest settlement histories, between 1987 and 2004, are presented in Figure 9(a)-(b) [18]. The cumulative settlements in the period have reached 450 mm in the middle of the backfill, with a rate of around 15 mm/year. It should be noted a differential settlement of around 150 mm between the upstream and downstream marks of the right bank profile. The cumulative settlement has reached about 6 mm per m of backfill since 1987.

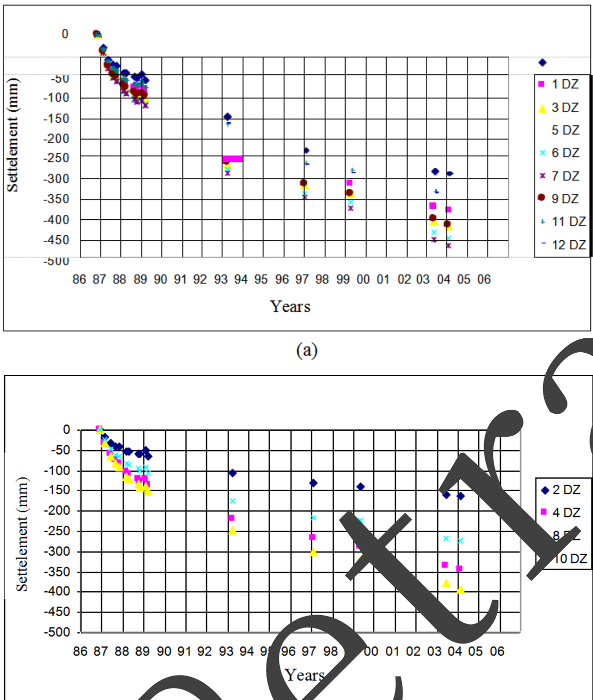

Figure 9. Settlement at the Crest of the dam from 1987 to 2004; (a)Upstream Mark (b) Downstream Mark (ANB 2006)

5.3 Deformations

Considering the two stages, before and after water filling, and using the two models (1) and (2), the dam response in terms of deformation is presented in Figure 10 and Figure 11. It shows an increase in deformation with the dam axis. The deformation variation in XY and YZ directions seems to be low, with an insignificant increase value at the top of the dam. Deformation is negligible in the other directions of the structure.

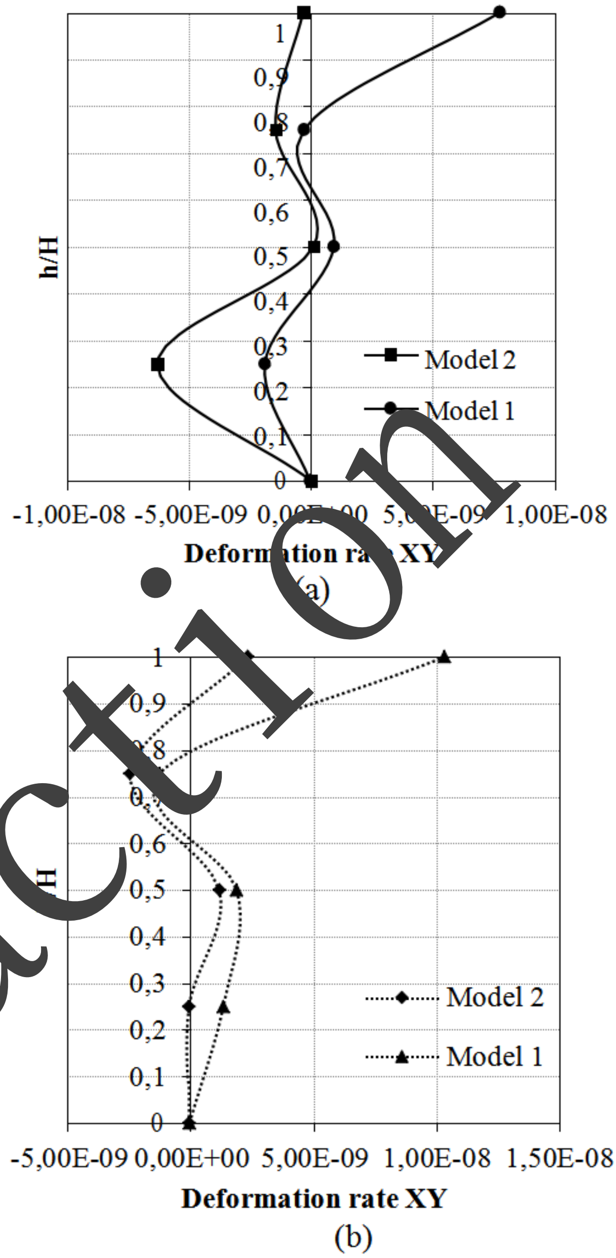

Figure 10. Dam deformation considering Model (1) and (2) in XY Direction

5.4 Pores Pressure Variation

The variation in pore pressures with the dam axis (base, middle and top) in the dam core and the upstream rockfill, accordingly to the models (1) and (2) (Figure 12(a)-(b)). Pore pressure is more important at the base of the dam body and decreases with the dam axis. We can see identical values of pressure in the dam core and the rockfill for model (1). However, for model (2), the pore pressure varies according to the nature of the material (core or rockfill).

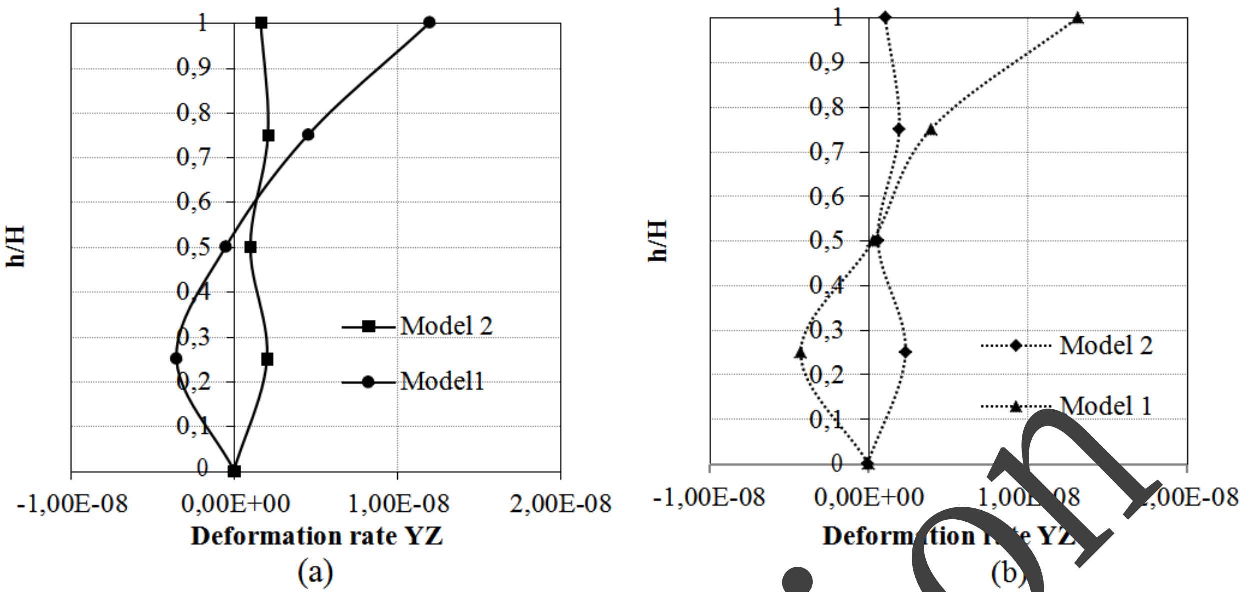

Figure 11. Dam deformation considering Model (1) and (2) in YZ direction

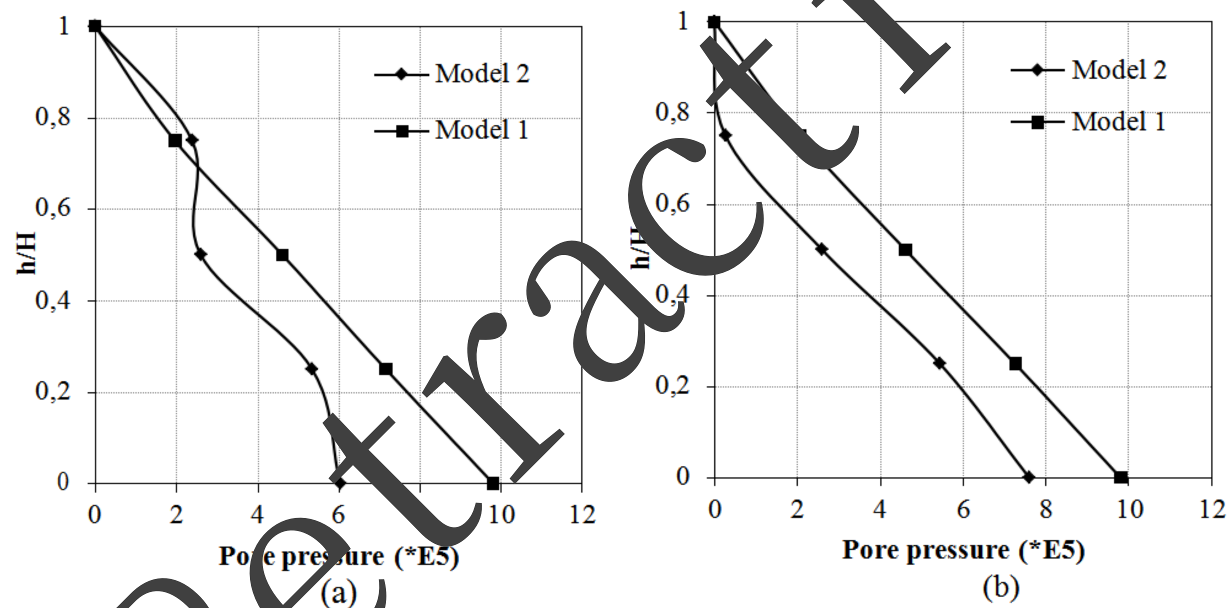

Figure 12. Pores pressure variation in the dam for the two Models (a) Core (b) Upstream rockfill

5.5 Dynamic Analysis Response

The coupled analysis concerns the response of the dam to the Boumerdes 2003 earthquake record. Geotechnical properties are summarized in Table 1. The foundation is assumed to be stiff with a Young's Modulus E= 4.10^7 MPa and the permeability and porosity of the core are equal to 1.10^{-12} m/s and n=0.8.

The seismic loading induces a maximum displacement in the seismic excitation X-direction at the dam core and downstream rockfill, which attains 10.4 cm (Figure 13). We can see the displacement variation with the dam axis (base, middle and top) under the maximum of the seismic excitation. The displacement increases with the distance from the base of the dam body with an approximative stabilization in the middle and the top of the dam. Figure 14 shows the maximum vertical displacement under the earthquake, which correspond to a very slight increase at the top of the dam, but remains insignificant [5,19,20]. It is worth noting that the permanent static vertical displacement is not considered in dynamic response. Figure 15 shows the induced acceleration in the dam under the seismic excitation. It can be observed a quasi-stabilization of the acceleration from the base to the top of the dam, except at the middle of the core where it increases. Figure 16 provides the pore pressure evolution in the dam under seismic loading. It can be observed that the seismic excitation induces an increase in the pore pressure in the base of the dam body, with an excess pore pressure ratio of 1.14 compared the static state [5,21]. This increase remains not significant to produce liquefaction or instability of the dam. The impermeable clay core has oriented to the upstream face, which indicates more stability.

Finally, it was observed that the core clay affected the dam behaviour under earthquake effect. It was noted that the dissipation of the pore pressure is fast during and after construction due to increasing of stresses and settlement at the end of construction.

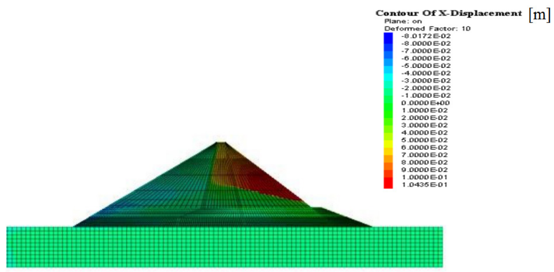

Figure 13. Dam displacement in the X direction under seismic loading

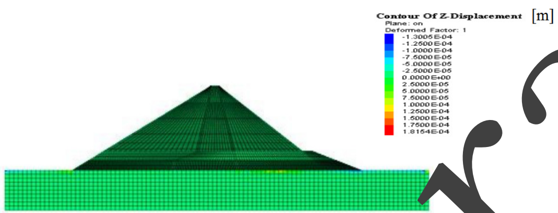

Figure 14. Dam displacement in the Z direction under seismic loading

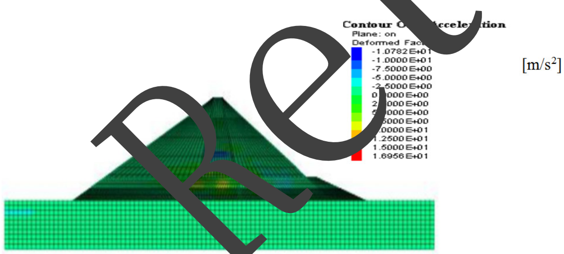

Figure 15. Dam acceleration [m/s²] in the X direction under seismic loading

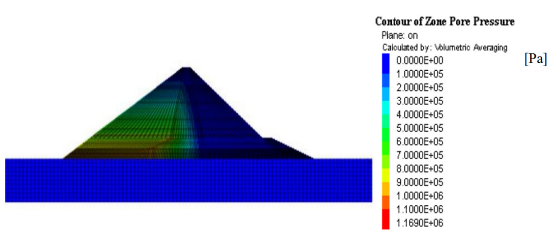

Figure 16. Pore pressure variation [Pa] in the dam under seismic loading

6. Conclusions

This paper presents the static and seismic examinations of earth dam utilizing finite difference method. Mathematical examination of the Keddara dam, situated in Boumerdes district (Algeria), is directed utilizing FLAC 3D program, with the target to characterize its static and dynamic way of behaving, as far as settlement, distortion and pore pressure variety during its construction and operation .

- The dam's response is described using an elastic model and an elastoplastic constitutive model with the Mohr-Coulomb failure criterion.
- Static study demonstrated that the largest induced settlement is found in the dam core.
- The displacement variation along the dam axis demonstrates an increase in displacement at the dam's top. In the case of vertical displacement, the calculation model has no effect, whereas deformation and pore pressure do. Monitoring data reveals that the dam's static settlement has been completed with the consolidation time.
- Furthermore, pore pressure is greater near the dam body's base and decreases with dam axis.
- The three-dimensional seismic analysis of Keddara dam was conducted using Boumerdes earthquake (2003) record.
- Horizontal and vertical displacements rise with distance from the dam body's base to the top; maximum displacement, acceleration, and pore pressure remain insignificant for dam instability.

Conflict of Interest

There is no conflict of interest.

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