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Behaviour of an Earth Dam Using Finite Element Modelling with Different Soil Constitutive Models: Case Study of El Hma Dam, Ben Arous, Tunisia

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Abstract. The present paper deals with the analysis of the behaviour of an earth dam at the end of construction, during the first filling of the reservoir and at long term by using Finite Element Method. In this analysis, El Hma dam, which located in Tunisia, has been selected as a case study. Three constitutive laws including elastic, Mohr-Coulomb and Drucker-Prager, were assumed to present the material characteristics of the dam and its foundation. Attention will be given to the study of predicted deformation, stresses and pore pressure distributions. The interpretations of the predicted results lead to evaluate the dam behaviour.

Keywords: earth dam, Finite Element Modelling, Constitutive models.

1. Introduction

Prediction of the deformations of an earth dam has a significant effect on its performance and safety. Significant movements and settlements of the crest and the body of the dam can occur during various stages of the construction of the dam, at the end of construction, the first filling of the reservoir and during the operation of the dam.

The first filling of the reservoir is the most important stage in the earth dam construction because the effect of wetting. Increasing of water level in reservoir involves the deacrease in geotechnical property values and the Young modulus of the material in the submerged sections of the structure. Once the filling of the reservoir is completed, the dam undergoes long-term deformations [1]. The weight of the embankment and the pressure of the reservoir water involve the fill material to settle resulting in a vertical movement of the structure. The reservoir water pressure also causes permanent horizontal deformation perpendicular to the longitudinal axis of embankment. However large permanent deformations could occur due to reservoir drawdown [2].

On the hand, in embankment dams the progress of filling of the reservoir will develop a considerable pore pressure within the core of the dam. The increasing of water level in the reservoir may double the pore pressure in the core due to seeping the water within the dam which may lead to excess pore pressure in the core ending to hydraulic fracturing of the dam.

Today, the finite element method has become established as a useful tool to model the deformation of earth dams. The concept of the finite element method, which is used to analyze expected displacements, strains, stresses and pore water pressures in the structure associated with different loading and boundary conditions, is extensively described by Zienkiewicz [3]. To perform the finite element analysis of the dam, the selection of the material model is essential [4]. Recently, efforts has been devoted toward the development of more sophisticated and refined constitutive models, which resemble the behaviour of real engineering materials more closely [5]. More constitutive models associated with various degrees of sophistication and complexity, have been reported in the literature [6]. A new generation of programs and codes can now be run comfortably on a personal computer.

In this paper, emphasis is given on practical applications of the finite element method to analyze the deformation of an earth dam. El Hma dam, which located in Tunisia, has been selected as case of study. The presented results are limited to only one cross-section in the middle of the dam. Three constitutive models including Elastic, Mohr Coulomb and Drucker-Prager, were assumed to present the construction material characteristics of the dam and its foundation, have been considered.

Finite element analyses were conducted using widely used commercial finite element software package ABAQUS [7], which is developed by Hibbitt, Karlsson and Sorensen, Inc. It is a general-purpose commercial finite element software, capable of performing linear and non-linear analyses. Moreover, it has many built-in material models for many types of analyses in its material library.

However the most critical problem in predicting deformations of earth dams by using finite element modelling is to obtain characteristics of the fill materials. The difficulty in determining material characteristics is the main cause of uncertainty in modeling of deformations. Results of properly monitoring schemes may be used to enhance the predicted model. Due to the uncertainty of the model parameters [8], the monitored deformations of El Hma dam should be performed to improve the model [9]. The monitoring of the deformation of the dam will be discussed in another communication.

2. Material Models

The material models used for the analyses are plasticity models and the elasticity model. These material models are available in the ABAQUS [7] materials library and they can be used with the plane strain continuum type elements.

3. Elasticity Model

The elasticity model, either linear elastic or porous elastic model, demonstrates the ability of the constitutive law to simulate the non-linearity behavior of materials due to revertible strains. In this model the linear relationship between stress and strain is the simplest link implying a constant proportionality between general stress increments and strain increments [10]. The full link between stresses and strains can be written as a compliance relationship:

$$\sigma_{ij} = D_{ijkl} . \varepsilon_{kl}$$

This expression describes Hooke's Law of elasticity. If isotropic marerial behaviour is independent of the direction of the solicitation; only two independent constants subsist in the last expression:

$$\sigma_{ij} = \lambda . (\sum_k \varepsilon_{kk}) . \delta_{ij} + 2\mu\varepsilon_{ij}$$

Where: λ and are lamé's constants, and δ_{ij} is the kronecker tensor. The elasticity model has been implemented numerically in ABAQUS 6.4 [7].

4. Drucker Prager Elasto-Plasticity Model

The use of Drucker Prager criterion for soil modelling has been extensively reported and fully described in literature ([11]). Within this framework, the Drucker-Prager yield criterion under plane strain conditions which have been considered to model the embankment-foundation system materials.

Drucker-Prager criterion has been successfully adopted in analysis of geomaterials [5]. It is considered as a generalization of the von Mises criterion for cohesionless soil, taking into account the first invariant of stress tensor J_1 and the second invariant of deviatoric stress tensor J_2. In principal stress space, the criterion is given as:

$$F(\sigma_{ij}) = \sqrt{J_2(\sigma_{ij})} + \alpha J_1(\sigma_{ij}) - k \leq 0$$

J_1 represents the trace of stress tensor $(J_1 = \sigma_1 + \sigma_2 + \sigma_3)$, and k are material parameters related to the soil friction angle and the cohesion and σ_1, σ_2 and σ_3 being the principal stresses of a stress tensor.

$$J_2 = \frac{1}{6}\left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\right]$$

In the principal stress space $(\sigma_1, \sigma_2, \sigma_3)$, the failure mechanism is represented by a cylindrical cone-shape surface and having as axis (hydrostatic axis) a straight line of equation:

$$\sigma_1 = \sigma_2 = \sigma_3 .$$

- If $\alpha = 0$ the Drucker-Prager criterion can be reduced to von Mises type criterion. Therefore, the cylindrical cone shape surface becomes a cylinder.
- If $\alpha > 0$, the plastic strain is associated with an increasing of volume, i.e. dilatancy
- Under plane strain conditions, displacements perpendicular to the cross section are assumed to be zero, the yielding criterion matches Mohr Coulomb criterion

The plastic potential, which controls the soil dilatancy, is defined as:

$$G(\sigma_{ij}) = \sqrt{J_2(\sigma_{ij})} + \beta J_1(\sigma_{ij}) + \text{Constant}$$

Where, β is a parameter of behaviour law.

If the associative flow rule is adopted, parameters and β are equal. The Drucker-Prager model can be reduced to Von Mises type criterion by letting = $\beta = 0$.

The associated elasticity is the Hooke's linear elasticity. The Drucker-Prager criterion embraces in total 5 parameters: E, v, k, α, β.

The expression of the criterion leads to the following remark: in compression and for the case of sand, friction angles are limited to small values. However, many experimental results prove the contrary. It is obvious that the Drucker-Prager criterion does not adapted to the modelling of sands.

Generally, a correspondence can be established between parameters α, β and k of the Drucker- Prager envelope and , ψ and c of the Mohr-Coulomb envelope. For axis metrical triaxial conditions, $(\sigma_2 = \sigma_3)$, the correspondence between both criteria leads to the following relationships (Chen W.-F and A. F. Saleeb, (1982)) :

$$\alpha = \frac{2\sin\varphi}{\sqrt{3}(3-\sin\varphi)}$$

$$\beta = \frac{2\sin\psi}{\sqrt{3}(3-\sin\psi)}$$

$$k = \frac{6c\cos\varphi}{\sqrt{3}(3-\sin\varphi)}$$

Under plane strain conditions, ($\varepsilon_2 = 0$, for example), the hypothesis of the associative flow rule and the correspondence between criteria leads to the following relationships [10]:

$$\alpha = \frac{tg\varphi}{\sqrt{9+12tg^2\varphi}}$$

$$\beta = \frac{tg\varphi}{\sqrt{9+12tg^2\varphi}}$$

$$k = \frac{3c}{\sqrt{9+12tg^2\varphi}}$$

5. Mohr-Coulomb Model

The Mohr Coulomb yield criterion has a long history of usage in classical soil shear strength characteristics in terms of the Mohr-Coulomb cohesion c and friction ϕ. This criterion is used for cohesionless soil and for cohesive soil at long term [12]. Tresca criterion, which is considered as a particular case of Mohr- Coulomb criterion, is used for cohesive soil at short term.

The Mohr-Coulomb criterion is composed of two straight lines in the Mohr (τ, σ) plane where τ and σ are the shear and normal stresses on the failure plane. The mathematical expression of these two straight lines can be expressed as:

$$F(\sigma_{ij}) = \sigma_1 - \sigma_3 - (\sigma_1 + \sigma_3)\sin\varphi - 2c\cos\varphi \le 0$$

They are inclined of an angle ϕ regarding the normal stress axes. With σ_1 and σ_3 being the maximum and minimum principal stresses, respectively

$(\sigma_1 \leq \sigma_2 \leq \sigma_3)$ and c and ϕ□ are the angle of friction and the cohesion of the soil materials, respectively.

Mohr-Coulomb model can be reduced to Tresca type criterion by letting $\varphi = 0$. In principal stress space $(\sigma_1, \sigma_2, \sigma_3)$, the criteria presented defined by the function F is a pyramid associated with hexagonal cross-section and having as axis a straight line of equation:

$$\sigma_1 = \sigma_2 = \sigma_3$$

When $\varphi = 0$, this pyramid transforms into a circular cylinder.

The plastic potential can be expressed in terms of minimum and maximum principal stresses as:

$$G(\sigma_{ij}) = \sigma_1 - \sigma_3 + (\sigma_1 + \sigma_3)\sin\psi + \text{constant}$$

Where, ψ is the dilatancy angle. For $\psi =$ this is corresponds to the associative flow rule. Mohr-Coulomb model involves five parameters, namely Young's modulus, E, Poisson's ratio, ν, the cohesion, *c*, the friction angle, ϕ, and the dilatancy angle, ψ.

The Drucker-Prager model does not suffer from the non-smooth corner regions that generally affect Mohr-Coulomb-type soil models. A number of investigators prefer Drucker-Prager plasticity model due to the relative smoothness of the yield surface, the corresponding lack of sharp corners, the ability to model ductile tensile failure, and the coupling between shear failure compressive plasticity.

In addition, it is more realistic in that the Drucker-Prager model predicts a saturation of soil strength with increasing effective confining stresses. However, the classical Mohr-Coulomb model unrealistically predicts no saturation of shear strength with increasing effective normal confining stresses [4].

6. Boundary Conditions of a Free Surface Flow through Earth Dam

As the water flows, the soil in the dam undergoes volume changes in response to changes in total stress. Volume changes can generate pore-water pressures and alter the transient flow regime within the embankment dam [13]. The solution to these problems requires that soil behaviour be analyzed by incorporating the effects of the transient flow of the pore-fluid through the voids, and therefore requires that a two phase continuum formulation be available for porous media [14].

A procedure was incorporated in the ABAQUS computer program for performing finite element analysis on the behaviour of a dam including water seepage. ABAQUS has been coded based on a procedure that couples stress equilibrium (mechanical behaviour) and water flow (hydraulic behaviour) using the Theory of Consolidation for soil layers. Generally, flow of water through both saturated and unsaturated soil follows Darcy's law [13]. However, boundary conditions associated with the complexity of domains are not specified or prescribed, for example free-surface boundary. Therefore, the resolution of the flow equation is sometimes not straightforward. A water flow associated with free surface is characterized by a free surface limiting water flow to its top boundary. The seepage through an embankment dam is an example of unconfined flow bounded at the upper surface by a phreatic surface. The relative position of the free surface is an internal variation of the element to which hydraulic and mechanical properties depend on positions of saturated or unsaturated zones, respectively.

7. El Hma Earth Dam

El Hma dam is an embankment dam built across El Hma River located at Morneg town in the district of Ben Arous, Tunisia. The dam controls a hill slope reservoir of about 123 km^2 and receives an annual average discharge of about 7.8 Mm^3. Its maximum operating capacity amounts to 12 Mm^3.

The main objectives of the realization of El Hma dam are to:

- Recharge the Morneg aquifer at the downstream side of the dam;
- Minimize damage caused by flooding and preserve the discharge of high flood seasons (flood control);
- Meet the requirements of irrigation supply to Morneg region, Tunisia (over 760 hectares of land).

The embankment consists of a thick impervious core, impervious blanket, upstream shoulder and downstream shoulder. The construction materials used for the embankment are:

- The core consists of brown compacted clay, protected upstream and downstream by granular filters. These filters protect the core dam and are considered as transition zones between the core and upstream and downstream shoulders.
- Upstream shoulder consists of mixed material (clay, brown silt and materials taken from necessary excavations of conglomerates of unified structure and from the spillway)
- Downstream shoulder consists of cobbly and clayey gravels, and is considered as permeable. A berm width of about 7 m is located at downstream slope, which may provide additional stability to both the embankment and the foundation. The berm is covered with a 0.50 m thick rock fill layer lying on a transition layer having a thickness of about 0.25 m.
- Impervious blanket of about 1.50 m thick exists over all the downstream shoulder in order to protect the foundation against the risk of potential piping.

Average slopes of the dam are made of 1V: 3 H and 1V: 3.5 H upstream and downstream faces respectively. Figure (1) shows an example of a general schematic cross-section of El Hma dam. In table (1), the main dimensions of the Al Hma dam are summarized.

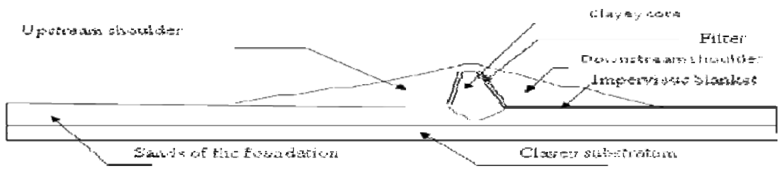

Fig. (1). General schematic cross-section of El Hma dam ([15]).

Table (1). El Hma dam geometry [13].

Dam length (m)	1200
Embankment height from the ground level (m)	26
Embankment width at its base (m)	196
Crest width (m)	6
Core width at the top/bottom (m)	4/5

The reservoir and the foundation area are infilled with sediments of Quaternary age. A sandy shallow soil layers are overlying an impervious clayey substratum. The thickness of the sandy layer is varying between 13 m at the upstream side and 7 m at downstream side, and mainly consists of sand and cobbles associated with clayey interbedded layers. The clayey substratum has an approximate thickness of about 9 m at the upstream side to about 13 m at the downstream side.

8. Finite Element Analyses

Figure (2) shows the finite element idealization considered for the embankment-foundation system. The embankment and foundation soils have been discretized into four nodded-quadrilateral elements. A total of 395 elements and 596 nodes were used. ABAQUS automatically meshes the geometry according to the given element size.

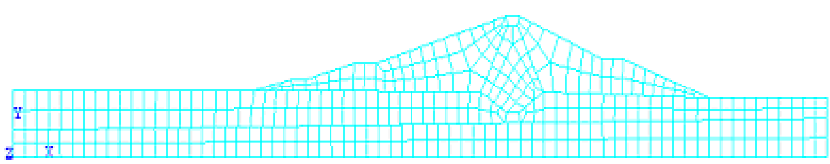

Fig. (2). Two-dimensional finite element mesh proposed for embankment-foundation system.

Zero vertical and horizontal displacements are specified at the substratum contact. However, the foundation is free to move in the vertical direction and fixed in horizontal direction at the left and right sides. However, the coupled hydro-mechanical model will be limited to the embankment including few elements of the upstream shoulder and the core, and the dam foundation.

In the horizontal direction, the model site was extended from the dam surfaces to about 101.5 meters and 50 meters from left and right sides, respectively.

Then the total width of the foundation is about 345.42 meters. The dam foundation was modelled to the depth of 21 meters below the ground level.

At the end of construction of the dam a significant pore pressure development is expected either in the embankment or foundation during construction of the embankment [12]. The embankment is constructed in layers with soils at or above their optimum moisture content that undergo internal consolidation because of the weight of the overlying layers. Embankment layers may become saturated during construction as a result of consolidation of the layers: there is drainage of the water from the soil during construction resulting in the development of significant pore pressures. The end of construction behavior modelling is analyzed in dry condition. The shear strength parameters are considered as based on geomechanical classification and laboratory tests, odometer and triaxial cells, performed on a dry material [16]. Geotechnical parameters of the embankment and foundation materials used for the modelling are summarized in table (2).

Table (2). Geotechnical parameters.

		Embankment soil			Foundation soil	
Parameter	**Core**	**Upstream shoulder**	**Downstream shoulder**	**Filter**	**Sand and cobbles with clayey interbedded layers**	**Clayey substratum**
C(kPa)	72	50	0	0	0	70
ϕ (°)	21	23	48	36	46	19
E(MPa)	17.5	19	80	40	75	17
	0.32	0.3	0.25	0.27	0.23	0.33
ψ (°)	21	23	48	36	46	19
E	0.578	0.525	0.42	0.47	0.44	0.59
K(m/s)	10^{-9}	10^{-8}	10^{-3}	10^{-6}	2.10^{-3}	10^{-8}

In table (2), ϕ is the soil friction angle, c is the cohesion, E is the Young's modulus, ν is the Poisson's ratio, ψ is the dilatancy angle, e is the void index, and k is the permeability.

The analysis during the reservoir filling is performed considering two effects associated with pressure of water and wetting. The buoyancy forces that correspond to the water level of the reservoir are also accounted for. These forces are evaluated in the conditions corresponding to the minimum and maximum water levels in the reservoir. Then the difference between them was gradually applied to the slope during a time span coinciding with the average time required by filling. During the filling of the reservoir, the values of geotechnical parameters decrease through the embankment as well as the Young modulus because of the wetting effect [15].

The long-term analysis is performed, under assumptions neither variations associated to the periodic variations of the level of the reservoir nor seismic loading, are accounted for. The maximum storage reservoir level is considered that is maintained long enough to produce a steady-state seepage condition. In the long-term behavior analysis all materials are considered fully drained (effective) associated with wet material conditions. The values of geotechnical parameters and elastic parameters (Young modulus and Poison’s ratio) of the construction materials decrease [15].

Subsequently and due to the change of the embankment volume on the dam foundation, several analyses need to be performed in order to predict the material behaviours at the embankment-foundation interface.

9. Prediction of Settlement

The finite element analysis is performed to predict the vertical displacement of El Hma dam at the end of construction, during first impounding and at long term. Results of analyses illustrating the evolution of settlements at the embankment-foundation interface for the elastic, Drucker-Prager [20] and for Mohr-Coulomb criteria at the end of construction, during first reservoir filling and at long term have been plotted in figures (3 to 5).

A comparison between different criteria used at the embankment-foundation interface for the three different states: end of construction, during first impounding and at long term have been depicted in figures (6 to 8).

At the end of construction of the dam, the calculated settlement underneath the crest, more precisely at the embankment-foundation interface, reached approximately 0.56 m for the elastic model, 0.70 m for Drucker-Prager criterion and 1.04 m for Mohr-Coulomb criterion. After impounding, the calculated settlement under the crest, especially at the embankment-foundation interface, increases to reach approximately 0.70 m for the elastic model, 0.80 m for Drucker-Prager criterion and 1.70 m for Mohr-Coulomb criterion. At long term, these calculated settlements continue to increase and reach approximately 0.80 m for the elastic model, 1.30 m for Drucker-Prager criterion and 2.00 m for Mohr-Coulomb criterion.

Fig. (3). Settlement at the interface embankment-foundation-Elastic model

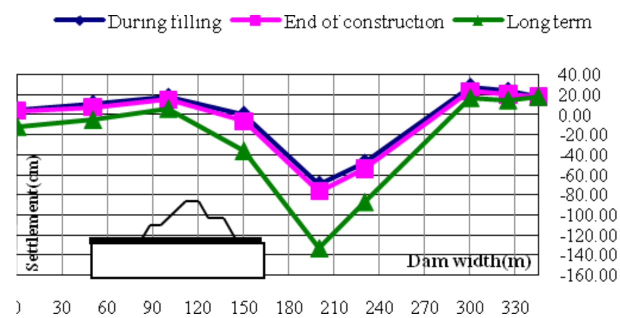

Fig. (4). Settlement at the interface embankment-foundation- Drucker-Prager model

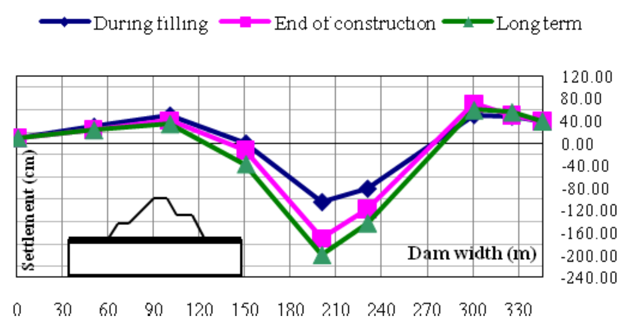

Fig. (5). Settlement at the interface embankment-foundation- Mohr-Coulomb model

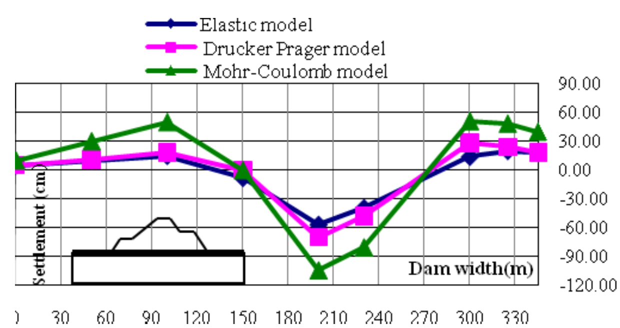

Fig. (6). Settlement at the interface embankment-foundation at end of construction - comparison between different criteria.

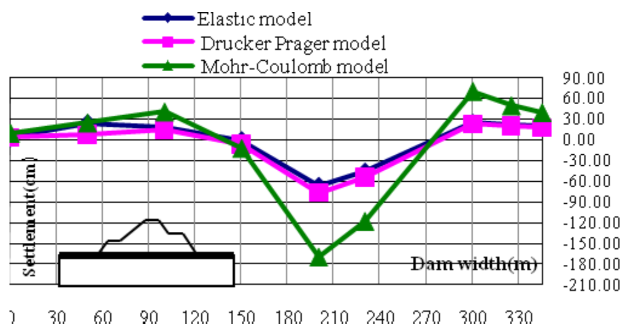

Fig. (7). Settlement at the interface embankment-foundation during filling-comparison between different criteria.

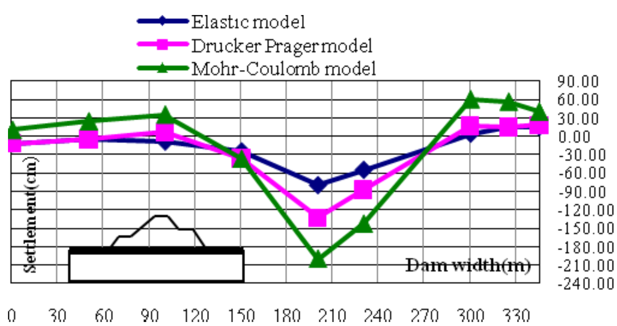

Fig. (8). Settlement at the interface embankment-foundation at long term -comparison between different criteria.

At the end of construction, these calculated settlements under the embankment are associated with significant swellings at the upstream and downstream toes of the dam. Then, these swellings decrease, due to soil consolidation, and reach small values at long term for all three criteria at the upstream toe level of the structure.

The decreasing of vertical displacement at the upstream toe of the dam is caused by the effects of pressure of water and effect of wetting. Similar observation can be demonstrated for the downstream toe of the dam that at long term the soil swelling stabilizes. However these displacements remain more significant compared to those occurred at the upstream toe of the dam.

The magnitude of settlements is due to the following combination:

- the consolidation corresponding to an increasing of effective stresses

- modifications of effective stresses due to variations of volumetric weight of foundation and embankment soils and variations of pore pressures associated with settlements
- the development of significant lateral strains.

10. Prediction of Horizontal Displacements

Distributions of horizontal displacements within the dam depend strongly on the consolidation state under the embankment. Horizontal displacement predictions of the dam and the downstream berm for three criteria; elastic, Drucker-Prager and for Mohr-Coulomb at the end of construction, during first reservoir impounding and at long term have been plotted in figures (9 to 11). When plotting displacement versus the height, the heterogeneity of materials across the dam from the berm level to foundation layers has been considered.

As can be seen from the figures, these displacements reach their maximum values beneath the downstream berm of the dam body. Additionally, the slope changes once new material is crossed. At long term, horizontal displacements increase over the time to reach their maximum values about 0.62 m for Mohr-Coulomb criterion, about 0.45 m for Drucker-Prager criterion and about 0.32 m for elastic criterion.

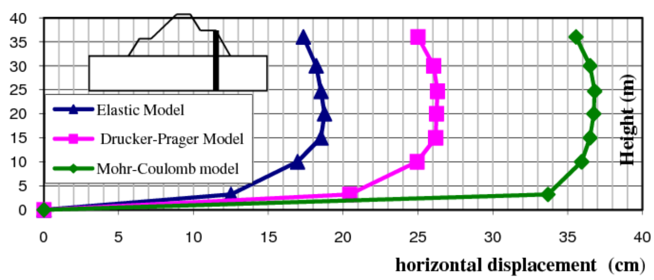

Fig. (9). Horizontal displacement of the downstream shoulder at the end of construction.

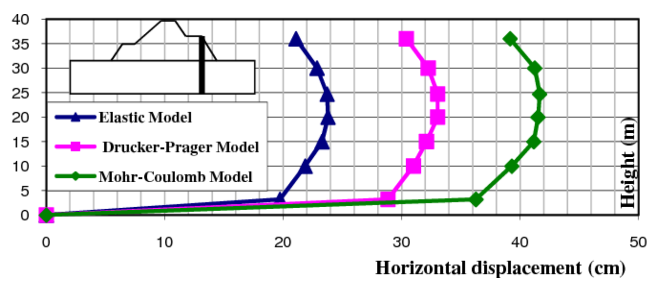

Fig. (10). Horizontal displacement of the downstream shoulder during filling.

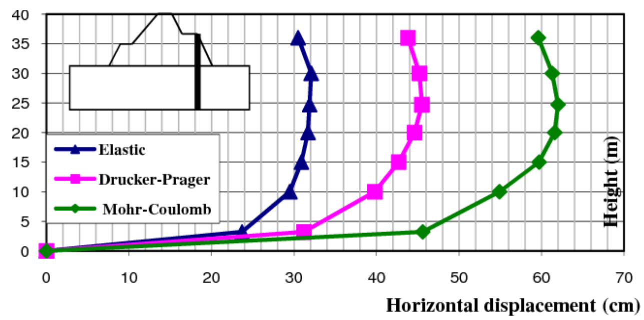

Fig. (11). Horizontal displacement of the downstream shoulder at long term.

11. Prediction of Stress State and Pore Pressure

ABAQUS is also used to conduct effective stress analysis. First, the nodal pore pressures are evaluated through a finite element seepage calculation adopting the same mesh used also for the stress analyses. Then, the nodal forces equivalent, in the finite element sense, to the calculated pore pressures are determined and introduced as external loads in the stress analysis.

The development of pore pressure in the central zone of the core is quite large comparing with that in the other two sides of the clayey core. Moreover, as can be seen that the pore pressure increases under the upstream and downstream toes of the dam for Mohr-Coulomb criterion, while under the crest of the dam the maximum pore pressure is developed for elastic and Drucker-Prager criteria.

Superposed stress diagrams for different criteria at long term have been plotted in figures (13 to 14). The maximum stress under the crest of the dam is a tensile stress for three criteria. Adversely, in the upstream and downstream toes of the dam the stresses are not higher. However, it should be noted, therefore, that in Figure (13), the estimated stress from 2D modelling of about 2.5 MPa under the middle of the dam is not realistic. The estimated stress from the three-dimensional model is of about 9.50 MPa. More investigation, mainly the results from automated monitoring surveys, should be conducted in order to verify and to enhance the Finite Element model.

Obviously, in figure 14 the behaviour of the dam at long term is totally compressive in term of effective stresses. This gives a realistic idea of the dam behaviour. The concentration of compressive effective stresses under the crest reaches about 2.4 MPa for the elastic criterion, about 2.5 MPa for the Mohr-Coulomb criterion and about 3.6 MPa for the Druger-Prager criterion.

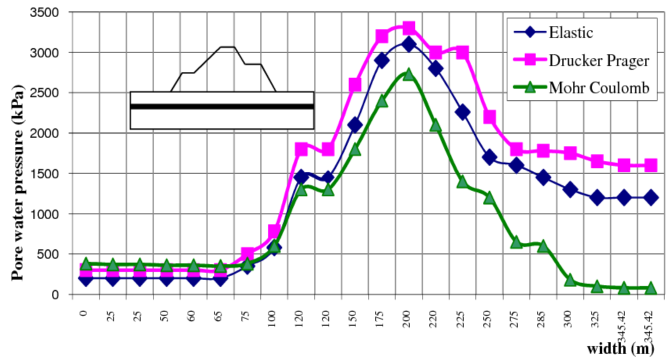

Fig. (12). Pore water pressure in the middle of the foundation at long term.

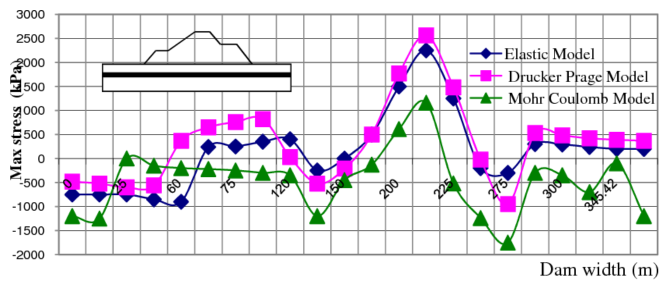

Fig. (13). Maximum stress in the principal plane at the middle of the dam foundation using different behavior models at long term.

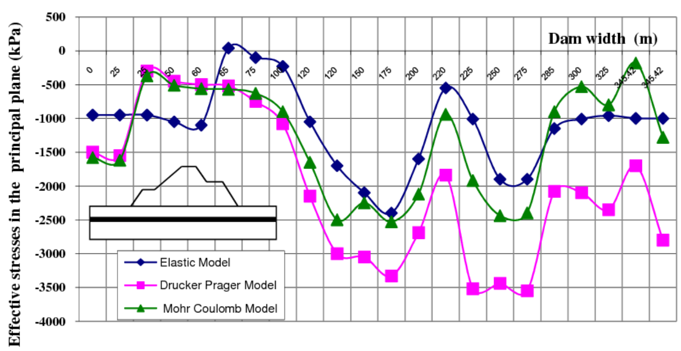

Fig. (14). Effective stress calculated in the principal plane at the middle of the dam foundation by different behaviour models at long term.

12. Conclusion and Recommendations

The results of two-dimensional numerical modelling with ABAQUS software and its constituent models led to results highlighting the potential benefits of using this approach to predict the behaviour of El Hma earth dam**.**

El Hma embankment dam was used as a case study to apply and compare different constitutive models. The behaviour criteria that will be judged the more suitable is the one that gives the closer deformation values to the observed deformation. However, more elaborated constitutive model should be used in attempting to reach a more realistic final state of stress.

However, he most critical problem encountered in modeling the deformations is obtain in-situ characteristics of construction soil material, which is the main source of uncertainty in modeling the deformations. The selection of material model in order to predict the behaviour of earth structure is most important when dealing with finite element modelling.

The long-term prediction of settlement of the El Hma earth dam, a simple creep model should be introduced in order to predict long-term time-dependent fill materials behaviour and the evaluation of model parameters. For future analysis a concepts from viscoplasticity and constitutive model that describes the soil creep behaviour should be incorporated.

Further research must be devoted to the development of integrated monitoring systems to increase the reliability of El Hma dam. The mathematical modeling of deformations by using finite element method should be integrated in the design and analysis of monitoring surveys. The location of the sensors or the observed targets must include points where maximum deformations are expected. As depicted in Figure 13, the estimated stress from 2D modelling of about 2.5 MPa under the middle of the dam is not realistic. The automated monitoring surveys should be conducted in the middle of the dam in order to verify and to enhance the Finite Element model.

13. Acknowledgments

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14. References

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السلوك الميكانيكي لسد أرضي باستعمال طريقة المحاكاة العناصر المحدودة مع إدراج قوانين مختلفة لسلوك التربة المستعملة : منطقة الدراسة سد الحمى بن عروس تونس

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