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Research Article

STATIC ANALYSIS OF SORAL EMBANKMENT DAM DURING OPERATION USING PRECISE DAM TOOL DATA AND FINITE ELEMENT METHOD

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ABSTRACT

The construction of the embankment dams involves many complex factors that make the design and implementation of these massive structures one of the most important projects in geotechnical engineering. During the construction phase and the operation of the embankment dams, various types of stresses are exposed, including the stresses caused by the dam settlement and the stresses of the necessary considerations, and the safety plan is implemented. Changing variables in terrestrial environments such as calculating the amount of water loss in the dam, controlling critical output gradients, graphical drawing of intensity, magnitude, and direction of movement of the dam body or foundation, calculating the type and dimensions of drainage systems and water was performed by software Plaxis2D and compared the exact instrument results. In this study, static analysis of soral embankment dam is performed using hardening soil model (HS), which is an elastoplastic model. The results of the research show that the maximum total of vertical stress and effective stress at the end of the construction of dam structures is equal to 1480 and 977 kPa and the maximum total of vertical stress and effective stress in the steady-state leakage stage is equal to 1150 and 887 kPa, which will occur on a rocky bed in the core area. The vertical stress curves indicate changes in the stress values in the transition from the upstream pebble shell to the core and the core to the filter and the downstream pebble shell. The reason for this is the difference between the hardness of the deformation (elastic modulus) in the core and filters and shell areas. The areas with greater hardness have less settlement than softer areas, leading to unequal settlement between the two areas. The consolidation analysis has shown that the proposed timeline is suitable for construction (20 cm thick embankment layers per day) and that the pore pressure values are such that there is no sustainability problem for the dam. In general, the results of static analysis confirm the proper behavior of the dam structure for post-construction conditions and stable leakage conditions. The displacement and stresses and the permeable water pressure are acceptable and ensure the stability of the dam.

Keywords: Static Analysis, Embankment Dam, Soral Dam, Elastoplastic Model

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INTRODUCTION

Today, dams play an important role in the development of the water industry. Iran's natural and geological conditions are seriously on the agenda of engineers and planners from the earthquake point of view. Because many dams have been built or are under construction in earthquake-prone areas, their safe design against earthquakes is of particular importance and location. The careful study of the seismic stability of embankment dams is a complex issue in the field of earth structures. The diversity of the dynamic properties of the dam body and the variety of material and thickness of the foundation can play a key role in the transmission, attenuation and amplification of earthquake waves, and the presence or absence of active faults within the dam axis, earthquake characteristics such as the distance from the epicenter to the dam, the intensity and duration of the earthquake, the type and length of waves reaching the dam and the frequency of the waves are among the factors that play an important role in the dynamic response of the dam.

In designing different dams, various applications such as providing the required water for irrigation of the plains, providing drinking water, generating hydroelectric power, flood control, etc. are considered. Depending on the needs and type of design, these goals can be considered alone or simultaneously. The type of earthen dam initially depends on the materials in which it is located or in its vicinity. In terms of the type of materials used, dams can be divided into two main groups: concrete and soil. In our country, most of the dams built are embankment dams (Ministry of Energy, 2004).

Embankment dams refers to dams that are made of natural materials in nature such as soil, stone, trenches, etc. and do not use any natural and unnatural adhesive materials in them and there should be sufficient resistance to the forces and the leakage of water from them only with the density of materials. In these dams, the dam body is heavily compacted by heavy rollers in low-thickness layers. In embankment dams, sealing is done by core (Rahimi, 2003; Vafaian 2009). Based on the statistics of the International Committee of ICOLD and the study of the factors affecting the destruction of 308 earthen dams in different parts of the world, it has been shown that uncontrolled settlement and rupture of foundation have been the main cause of 50% of failures (Najm, 2000). Major settlement embankment dams occur at the time of construction, with maximum settlement not always occurring at the largest section of the dam. Settlement dam in embankment dams is unequal, although it may be symmetrical (IK-Soo, 2011).

The examination of the research shows that settlement of embankment dams causes phenomena such as piping in the downstream and ultimately the instability of the dam. As the pore water pressure increases, the effective stress between the soil particles decreases, which reduces the shear strength of the soil and increases the risk of hydraulic rupture of the dam (Zhu, p., Leng et al., 2011). Predicting and controlling pore pressure during construction in embankment dams is

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one of the most important issues in the design and construction of such dams.
One of the problems we face during the construction of embankment dams is the arching phenomenon in the clay core of these dams. Arching has a significant effect on the behavior of core deformation. This phenomenon reduces the level of stress in the core, which increases the likelihood of hydraulic failure in the initial flooding process (Y.Jafarian et al., 2012).
During operation, the dam body withstands all possible internal and external loads. Applying these loads causes the horizontal and vertical displacements, which refers to settlement and elevation the vertical displacements in the direction of gravity accelerate and in the opposite direction, respectively. Due to the high importance of embankment dams, the use of precision instruments is inevitable in terms of displacement control (Ministry of Energy, 2004).
Designers of geotechnical structures face complex design problems; in the case of soil or rock, there are many difficulties and ambiguities in determining the actual characteristics and how they work in different conditions, because the soil mass consists of three parts: solid, liquid, gas as heterogeneous. Regarding the behavior of geotechnical structures in different stages of construction and operation, the existence of different tools for deformation and stress in different parts of the structure is necessary. Embankment dams are among the most important earthen structures, which need to be examined for various reasons, more than any other structure.
In general, embankment dams are three-dimensional, huge, heterogeneous, non- isotropic and non-elastic structures that are involved in the foundation and reservoir water. Numerical models that can consider all of the above factors will be very complex. In geotechnical issues, an advanced behavioral model is often required to model nonlinear and time-dependent behavior depending on the intended purpose. Plaxis2D software is able to consider nonlinear behavior and soil time using advanced soil models (Brinkgreve, R. B. J., 2002). Therefore, in this study, the most important issue of dam behavior in different stages along with dam construction has been investigated.

MATERIALS AND METHODS

Plaxis2D Software: it step-by-step excavation and tillage with different loading and boundary conditions can be modeled using 6-node and 15-node triangular elements. In this software, the Moharre-Columbus behavioral models, the hyperbolic hardening model, the softening model (Cam-Clay model) and the Soft Soil Creep Model can be used. Also, the manufacturing and drilling process can be modeled by activating and inactivating the elements in the calculation stage of this software.
Stability Analysis with Plaxis Software: In structures such as embankments, the stability coefficient is defined as the ratio of the existing shear strength to the minimum shear strength for equilibrium. In Plaxis software, the phi-c reduction method can be used to calculate the stability factor. In this method, the general deformations does not have a physical meaning, but the partial deformations of the last stage of analysis (during failure) is used in determining the failure mechanism.

- **Finite component method**: A continuous environment is divided into a number of elements that are connected at the nodes.

The displacement values for the $\boxed{\underline{U}}$ element are defined as follows according to the node displacement v using the interpolation functions:

$$\boxed{\underline{U} = \underline{N}\underline{v}} \tag{1}$$

The interpolation functions in the matrix N are also called shape functions.

$$\boxed{\underline{\varepsilon} = \underline{L}\underline{N}\underline{v} = \underline{B}\underline{v}} \tag{2}$$

In the above equation, the $\boxed{\underline{B}}$-matrix of the interpolation matrix is the strains, which includes the minor derivatives of the interpolation functions.

$$\boxed{\int B^T \Delta\underline{\sigma} dV = \int \underline{N}^T \underline{P}^i dV + \int \underline{N}^T \underline{t}^i ds - \int \underline{B}^T \underline{\sigma}^{i-1} dV} \tag{3}$$

Equation 3 is the equilibrium equation in discrete form. The first and second sentences of the above equation represent the vector of the external forces and the last sentence is the internal forces in the previous step. The difference between the vector of external forces and internal forces must be balanced by changes in stress of $\boxed{\Delta\underline{\sigma}}$. The relationship between tension and strain at each step is generally nonlinear. As a result, it cannot be calculated directly in each step.

Behavioral model of used stiffness in analysis
When the soil is subjected to shear, its stiffness decreases and as a result, the plastic strains expand. In a particular case of a three-axis test, a hyperbolic relationship between the axial strains and the deflection stress can be assumed that Konder and then Duncan first formulated this model and Chang proposed the well-known hyperbolic model.
This behavioral model (stiffness) can completely replace the previous model because, firstly, the theory of plasticity has been used, secondly, the effects of changing the volume of the soil due to expansion have been considered, and thirdly, a yield cap has been introduced for the model.
The important point in this behavioral model is the dependence of stiffness on stress level. To consolidation loading, a model can be defined as follows:

$$\boxed{E_{oed} = E_{oed}^{ref}(\sigma / p)^m} \tag{4}$$

P is the pressure that can be used the ideal foem m=1 for certain soft soils.

- **Approximation of hyperbole with the behavioral model of stiffness**

Due to the convenience and limitations, the test load conditions are three-axis with $\boxed{\sigma_2' = \sigma_3'}$ and $\boxed{\sigma_1'}$ will be the main compressive stress. The compressive stress and strain are considered positive. In this section, it will be shown that in this model, the hypothetical stress path is expressed as the standard three-axis drained test with equation (4). First, the relevant plastic straps should be considered, and this principle is obtained from a yield function as follows:

$$\boxed{f = \bar{f} - \gamma^P} \tag{5}$$

$\boxed{\bar{f}}$ is a stress function and $\boxed{\gamma^P}$ is a function of plastic strains:

$$\boxed{\bar{f} = \frac{1}{E_{50}} \frac{q}{1 - q/p} - \frac{2q}{E_{ur}} \qquad \gamma^P = (2\varepsilon_1^P - \varepsilon_V^P) \approx -2\varepsilon_1^P} \tag{6}$$

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For hard soils, its volumetric plastic strain of $\boxed{\varepsilon_V^P}$ tends to shrink. Therefore, it is considered $\boxed{\gamma^P \approx -2\varepsilon_1^P}$ approximately.

In addition to plastic strains, the model also calculates elastic strains. Plastic strains develop in the initial load, while elastic strains are created both in the initial load and in the load/reload. The three-axis drained test path with $\boxed{\sigma_2' = \sigma_3' = cte}$ conditions and the yang elastic $\boxed{E_{ur}}$ modulus is constant, and the elastic strains are defined by the following equations:

$$\boxed{-\varepsilon_I^e = \frac{q}{E_{ur}} \qquad -\varepsilon_2^e = -\varepsilon_3^e = -v_{ur}\frac{q}{E_{ur}}} \tag{7}$$

It should be noted that this equation is not true for the early stages of loading.

In the deflection phase of the three-axis test, the total axial strain is equal to the sum of the elastic strain components and one plastic component obtained from Equation 8:

$$\boxed{-\varepsilon_1 = -\varepsilon_1^e - \varepsilon_1^P \approx \frac{1}{2E_{50}}\frac{q}{1-q/q_a}} \tag{8}$$

This equation is without considering the volumetric plastic strain, that is, when $\boxed{\varepsilon_V^P}$ = 0, these volumetric plastic strains are not zero, but for hard soils, the volumetric changes of plastic are negligible compared to axial strains. Therefore, it is clear that the behavioral model of soil stiffness is a hyperbolic curve under triple-axis test loading conditions.

Modeling

General features of the model

The earthen dam with a clay core is geologically located on a rock bed that has low permeability. Details of the geometry of the dam are as follows:

Dam height: 43 meters

Crown width: 7 meters

Dam body material parameters for HS model

The soil geotechnical variables for modeling of hardening soil model are presented in Table (1).

Table 1: Specifications of dam body materials

Parameter	Filter	Core	Sand shell	Masonry shell
$C(kN/m^2)$	5	43	5	**5**
ϕ	35	18	37	**42**
ψ	5	0	7	**5**
M	0.26	0.73	0.34	**0.33**
$E_{50}^{ref}(MPa)$	23	6.2	74.4	**30.44**
$E_{oed}^{ref}(MPa)$	23	6.2	74.4	**30.44**
$E_{ur}^{ref}(MPa)$	78	18	383	**148**
υ_{ur}	0.2	0.2	0.2	**0.2**
$P^{ref}(MPa)$	500	100	800	**500**
K_0^{NC}	0.43	0.58	0.4	**0.34**
R_f	0.9	0.9	0.9	**0.9**
$\gamma(kN/m^3)$	19	19.5	20	**20**
$\gamma_{sat}(kN/m^3)$	**20**	**20**	**21**	**21**

RESULTS

Model evaluation

In general, in the analysis of any problem by numerical models, the most important part is to introduce the parameters of the model, finding the correct and real parameters of the model and introducing them is the main part of numerical modeling other than how to element and define the boundary conditions of the problem and their support conditions, each of which has a significant role in the analysis results. In elastoplastic analyzes, the plastic behavior of the soil is obtained by calibration or matching of the data.

- **Calibration of dam materials with HS elastoplastic model**

Static three-axis experiments were performed on different materials. In this section, a sample of three-axis cylinders with a diameter of 3.8 cm and a height of 7.6 cm with 1935 knots and 223 elements is considered. This sample is subjected to a comprehensive pressure of 100, 400, 700 and 900 kN/m² and then applied to the deflection stress to material rupture.
After executing the program and plotting the deviation strain – axial stress curves for the CU conditions, the results are first obtained in the software output by Jpeg format, and is then output from the data format and compared separately with the lab curves. The calibration results of the model core materials are shown in Figure 1. The best curve for choosing the right angle of expansion is the volume strain-axial strain curve, which is due to the fact that these curves are not available, this parameter was selected using Bolton's suggestions. In addition, the deviation strain – axial stress curves are only for the available core materials, for which a comparison has been made.

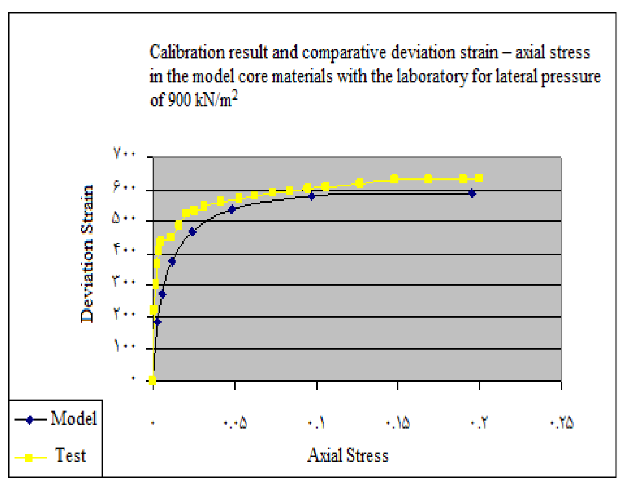


Figure 1: Calibration result and comparative deviation strain – axial stress in the model core materials with the laboratory for lateral pressure of 900 kN/m²

Soral Dam Finite Component Model

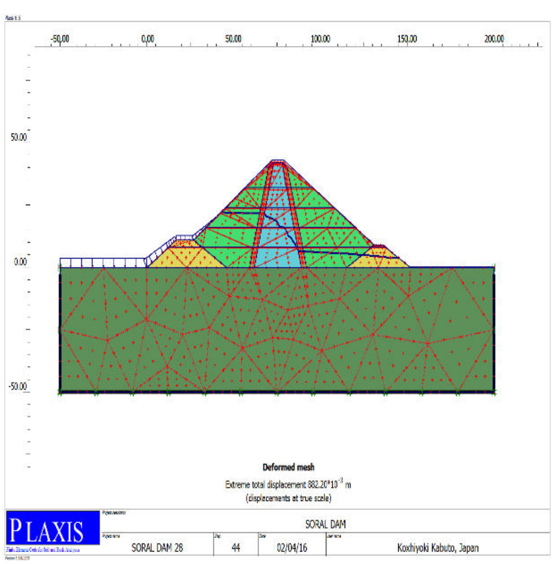
A two-dimensional element network consisting of 1723 nodes and 203 fifteen-node triangular elements is considered for modeling.

To analyze the flat strain (the length of the dam to the crown height is about 5.6 and the dam is located on a uniform rock foundation with a high elastic modulus), a two-dimensional model of the middle section of the dam, which has the highest elevation of the stone foundation, has been used. The foundation of the dam is modeled up to a depth of 40 meters, and a range of 50 meters above and below is also modeled. To ensure that the dimensions mentioned for the foundation are sufficient, a analysis was performed with the dimensions of the larger foundation (the foundation of the dam is modeled up to a depth of 60 meters and a range of 70 meters above and below is modeled). The results show a very good agreement between the displacements and stresses in the analysis. For elementation, the dam is divided into 5 layers in height to minimize improper elements in the model. This model includes 283 elements and 626 nodes. In this two-dimensional element network, 15-node three-dimensional elements are used. The extent of the impact of the number of elements and nodes on the stresses and strains has been investigated, which indicates that this number of elements and nodes is sufficient.

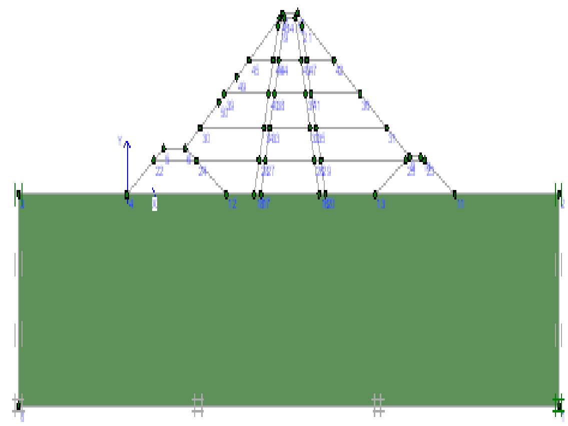
- **Simulation of the dam in static mode**

In this section, two different models for the Plaxis2D program were defined. In the first model, the overall structure of the dam is modeled for long-term behavior of the dam in a drained manner to calculate the amount of stress, deformations, and flow of the outlet water, and determine the same potential curves and reliability factor.

In the second model, the construction stage of the dam structure is simulated in 5 stages (layers). In the analysis of consolidation, the construction time of the dam equal to 20 cm of embankment per day is considered. The conditions for the construction of the first and seventh stages are shown in Figure 2.



(A)



(B)

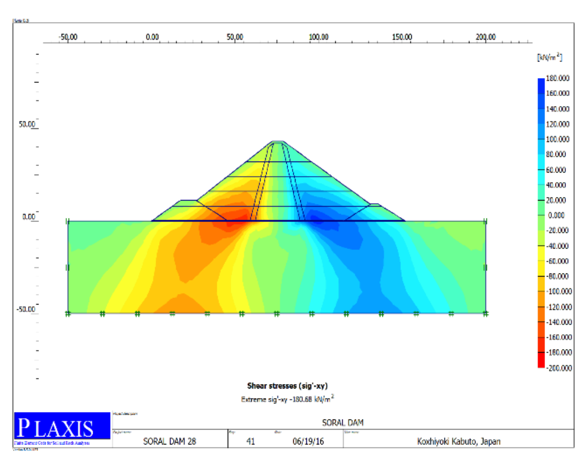
Figure 2: a) The first stage of land modeling, b) The seventh stage, applying water pressure to the model

Static analysis

Static analysis was performed nonlinearly using the HS elastoplastic model, taking into account the foundation to a depth of 40 m and despite the water force. Loading is done in 5 steps of 8 meters and attempts have been made to create the conditions for the construction of the modeled steps. The water pressure is then applied to the model. The obtained stress distribution of this method is real because the dam construction process models static analysis and also the effect of water in the dam reservoir is modeled on the dam body and is effective in the distribution and size of static stresses.

Distribution of stresses

The results of stress-strain analysis at the end of the construction of the dam structure are shown as horizontal and vertical stress curves in Figures 3a and b. In these Figures, the maximum vertical total stress and effective stress is equal to 1480 and 977 kPa and will occur on a rock bed in the core area. The results of stress-strain analysis in the steady-state leakage phase are shown as horizontal and vertical co-stress curves in Figure 3c and d. In these Figures, the maximum vertical total stress and effective stress is equal to 1150 and 887 kPa and will occur on a rock bed in the core area. The shear stresses increase on the sides of the clay core and near the filters, and this concentration increases the shear stresses from the crown of the dam to the bottom. The maximum concentration of shear stresses is at the junction of the core to the floor rock and the clay core and the filters.



(A)

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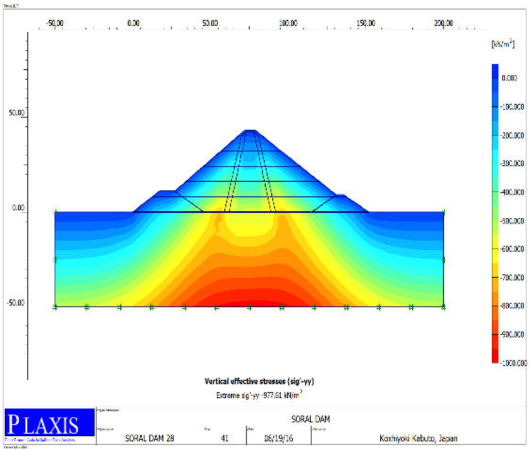

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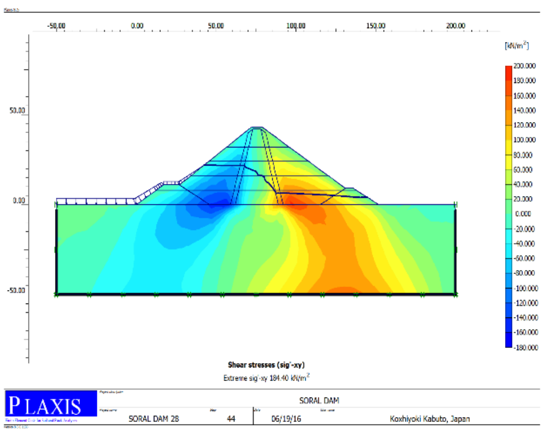

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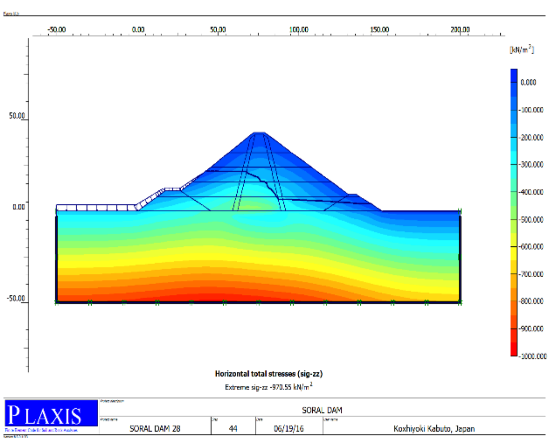


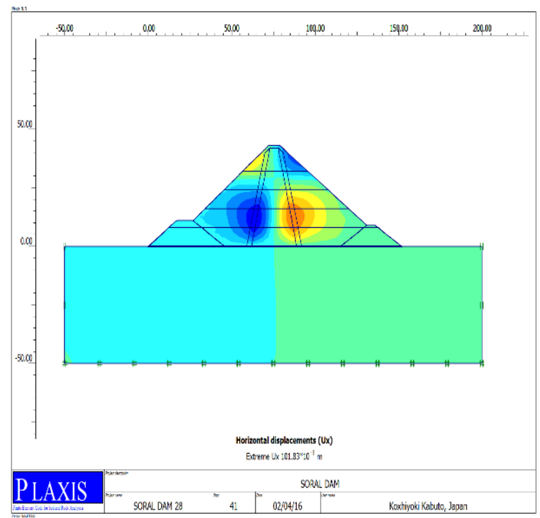
(D)

Figure 3: a) Distribution of effective vertical stresses at the end of the manufacturing stage; b) Distribution of effective shear stresses at the end of the manufacturing stage; c) Distribution of total horizontal stresses in the stable leakage stage; d) Distribution of effective shear stresses in the stable leakage stage

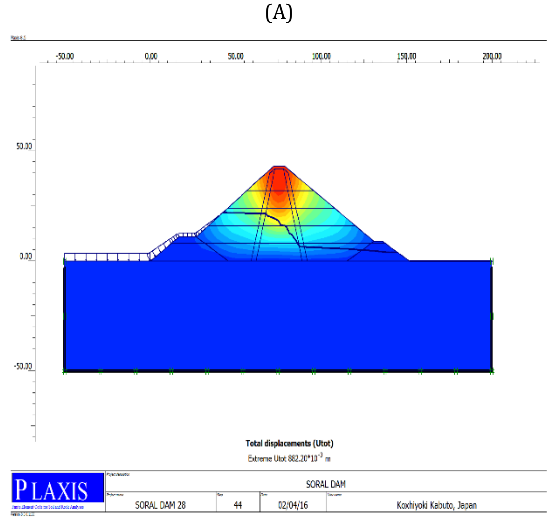
Distribution of displacements

According to the results, due to the relatively large deformation of the shell and the filter, most of the deposits occurred in the clay core. The results of the program analysis in determining the displacement of the dam body in the steady-state leakage state are shown in Figure 4a.

The maximum settlement occurred at the end of the construction phase of the dam structure at one-third of the core height of the dam crown, which is approximately 88 cm. The settlement amount is reduced by moving towards the gable. The distribution of horizontal displacements at the end of the construction phase of the dam structure is shown in Figure 4b. By referring to this Figure, the maximum displacement has occurred around the middle height of the dam and is about 10 cm. In general, the horizontal displacement at the end of the dam construction is upwards and downwards and the horizontal displacements of the dam at the end of the construction of the structure have relatively small values and indicate its stable condition in these conditions. In steady-state leakage, the total horizontal displacement distribution is downstream and reaches 0.11 m.



(A)



(B)

Figure 4: a) Distribution of total displacement in steady-state leakage mode, b) Distribution of horizontal displacement at the end of structural construction

Also, in the study of the dam rupture mechanism at the end of the dam construction, the dam reliability coefficient is 1.55 and the slope reliability coefficient in the case of stable leakage is 1.451. The results show that the stability of the dam slopes is easily ensured in the two end-of-building modes (in total stress analysis) and stable leakage in static mode.

Soral body precision instruments

Due to the need to study the behavior of the section with the maximum height and also the sections located in areas with high slope of bedrock (supports) and other considerations, 5 sections and a plan to select and different tools with specifications according to the tables attached (sphygmomanometer cells, electric piezometers, deflection gauges, stoppage gauges, etc.) are installed in each of the sections and plans related to them. The installed tools on these sections and the plan of the dam body and related facilities are located.

The total list and number of installed tools are as described in Table 2.

Table 2- Vibrating wire piezometer

Row	Tool name	Installation location	Installation level	Location	Installation Date	The depth of the excavated borehole
1	VWP-1	Clay core	2015	Upstairs	1387/06/23	0
2	VWP-2	Clay core		Downstream	1387/06/23	
3	VWP-3	Clay core	2005	Upstairs	1386/06/31	0
4	VWP-4	Clay core	2005	Middle part	1386/06/31	0
5	VWP-5	Clay core	2005	Downstream	1386/06/31	0
6	VWP-6	Clay core	1989.5	Upstairs	1385/07/14	0
7	VWP-7	Clay core	1989.5	Middle part	1385/07/14	0
8	VWP-8	Clay core	1989.5	Downstream	1385/07/14	0

In numerical studies in the Plaxis calculation module, it is possible to select a series of points as history point. In this code, there are no more than 16 possible points due to the limitations in the software. The first series of points are used as registration points for displacement and the second series of points for recording stress changes. In each of these points, it is only possible to select 8 points.

Pressure changes were made by changing the river water level. In a two-year process where the dam has been drained, due to the incomplete drainage of the layers, the water pressure of the holes caused by the construction is initially up to 15% different from the software. Precision instruments were damaged on 30 March 2019, and no data was available. It is not possible to fully adapt the days available in the software and the instrumentation, so linear mediation has been used to adapt. The PLAXISFLOW module software is not active in the software due to the broken software, and the software does not have the ability to accurately simulate the water level with time, but with these limited facilities, water pressure values have been estimated with high accuracy in predicting leakage values.

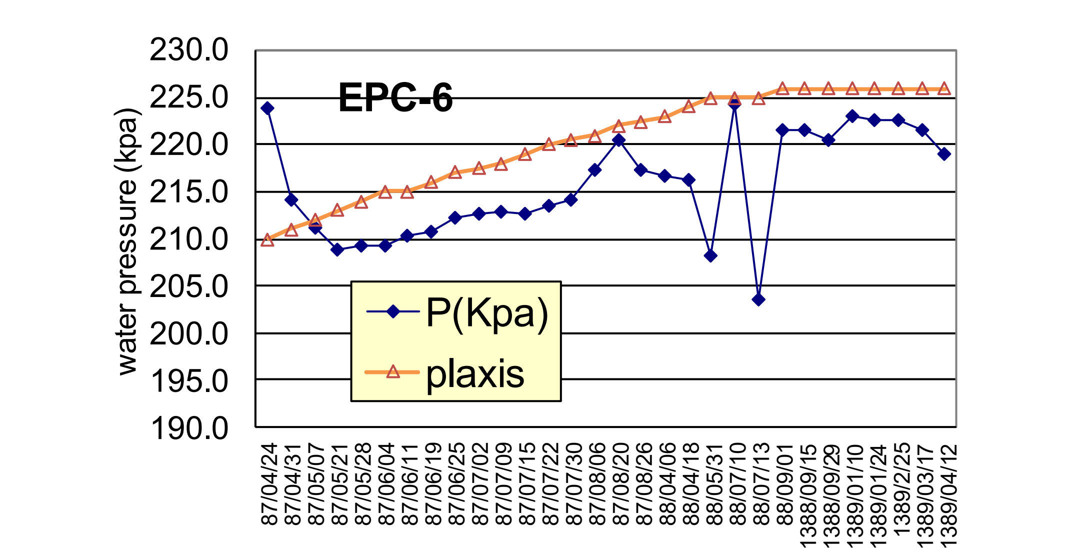


Figure 5. Pore water pressure changes in the sixth piezometer

The results show that at the end of the dewatering, the amount of water pressure in the cavities is constant and reaches 200 kPa. The maximum difference in results is 5%, which is reasonable and acceptable. In Figure (5), the reason for the fluctuations may be due to reading errors or problems in the above tool, which returns to normal after the course and is well adapted to the software results.

Comparison of settlement results of instrumentation and finite element code

Figure (6) shows the settlement results in the dam body compared to Plaxis.

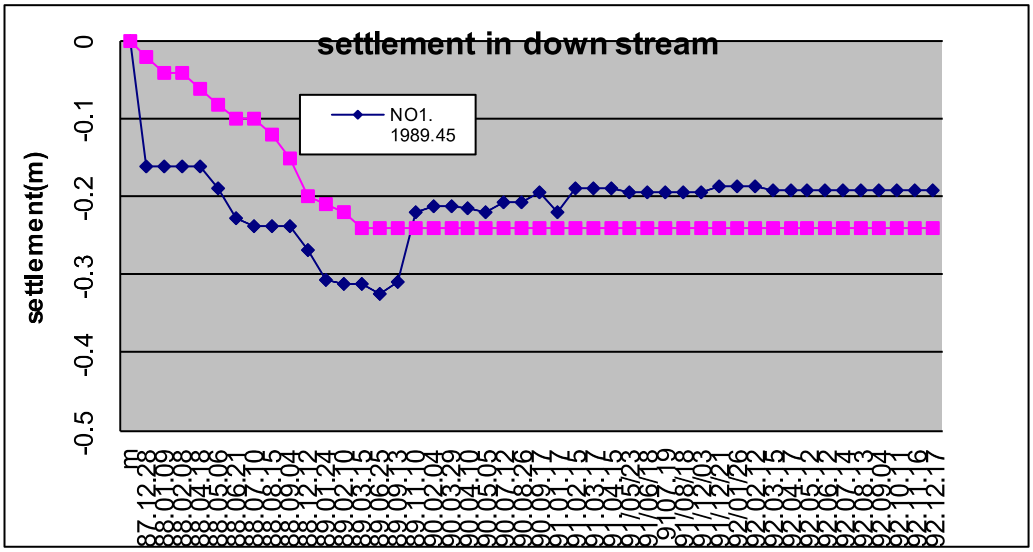


Figure 6: Comparison of settlement results of instrumentation and PLaxis.

Due to the lack of information about the river's water level, the rise and fall of the river has caused dilation in the body of the dam. Insufficient information causes uniform pressure distribution in the dam body by the software, but this pressure distribution is different from reality, which is one of the reasons for the incompatibility of these two cases.
Due to the method of dewatering and construction, it is not possible to simulate this process in the software, so in some parts of the diagram, the instrumentation is part of the horizontal diagram, but in the software, because the settlement calculations are step by step, and in each step a new session is calculated, the settlement in the software always has a mild slope.

DISCUSSION AND CONCLUSION

In this study, static analysis of Soral embankment dam using hardening soil (HS) model, which is an elastoplastic model, is obtained. This model is able to consider two types of shear stiffness and compressive stiffness. The eleven HS model parameters were carefully calculated, which this shows the result of the calibration of the materials of the dam body.
The two-dimensional element network uses 15-node trihedral elements. Dimensional analysis was performed to evaluate the adequacy of the model dimensions. To ensure that the dimensions mentioned for the foundation are sufficient, an analysis was performed with the dimensions of the larger foundation (the foundation of the dam is modeled up to a depth of 60 meters and a range of 70 meters above and below is modeled). The results show a very good agreement between the displacements and stresses in the analysis.
In order to perform the analysis and simulation of the dam in static mode in the body of the dam, an attempt has been made to create the conditions for the construction of the modeled stage. In the modeling, the construction stage of the dam structure is simulated in 5 stages (layers). In the analysis of consolidation, the construction time of the dam is considered equal to 20 cm of embankment per day. The water pressure is then applied to the model. In this part of the modeling, the overall structure of the dam for long-term dam behavior is assessed as drainage. To calculate the amount of stress, deformations, flow rate of the output water, determine the same potential curves, and reliability coefficient of sloping construction stages have been modeled in the long-time. Finally, the water pressure of the pore is compared with the results of the instrument.
The results of stress-strain analysis at the end of the construction of the dam structure and the stable leakage state are shown as horizontal and vertical stress curves and in the end of the construction of the dam structure, the maximum vertical total stress and effective stress is equal to 1480 and 977 kPa and in stable leakage state, the maximum vertical total stress and effective stress is equal to 1150 and 887 kPa will occur on a rock bed in the core area. Vertical co-stress curves indicate changes in stress values in the transition from the top rocking shell to the core and from the core location to the filter and the bottom shell. The reason for this is the difference between deformation stiffness (elastic modulus) in the core and filters and shell areas. The areas with greater stiffness have less settlement and leading to unequal settlement between the two areas.
The displacement of the dam body has been investigated both at the end of the structural construction and in the case of stable leakage. It can be seen that due to the relatively large deformation modulus of the shell and the filter, most of the settlements have occurred in the clay core, which is natural and logical. Also, the settlement of the core at the axis of the dam is much longer than the junction of the filters, which indicates the occurrence of the arching phenomenon.
The maximum settlement occurred at the end of the construction phase of the dam structure at one-third of the core height of the dam crown, which is approximately 88 cm. The amount of settlement is reduced by moving towards the sloping. The distribution of horizontal displacements at the end of the construction phase of the dam structure indicates that the greatest displacement occurred around the mid-height of the dam and is about 10 cm. In general, the horizontal displacement at the end of the dam construction is upstream and downstream. In general, the horizontal displacements of the dam at the end of the construction of the structure have relatively small values and indicate its stable condition in these conditions. In steady-state leakage, the total horizontal displacement distribution is downward and reaches 0.11 m.
The maximum pore water pressure varies from 0 to 200 kPa in stable leakage mode of core. At levels above 20 m, the water pressure is zero and corresponds well with the results of the instrument. In addition, no additional water pressure is created in the upstream shell area, and in this regard, there is a consistency with reading the instrumentation. Following the pore water pressure at -6 and -300 kPa, the results of the instrumentation show that moving towards the crown due to the elimination of the pore water pressure reduces its values. In the area of the downstream shell and the pore water pressure filter is zero.
The total stress analysis and the stability of the dam using the calculated stress field have shown that the up and down slopes at the end of the construction have a sufficient reliability against slip and rupture and the stability of the slopes is ensured in this condition.
The reliability of the dam at the end of the construction is 1.55. The results of effective stress analysis have shown that up and down slopes

in steady-state leakage mode have a good reliability against slip and rupture and provide slope stability in this condition. The slope reliability coefficient in steady-state leakage mode is 1.451 using the calculation results.

The consolidation analysis has shown that the proposed time plan is suitable for construction (embankment layers 20 cm thick per day) and the values of pore pressure are such that there is no problem in terms of sustainability for the dam. In general, the results of static analysis confirm the proper behavior of the dam structure for post-construction conditions and stable leakage conditions. The displacements and stresses and the permeable water pressure are acceptable and ensure the stability of the dam.

In a numerical study, Khamsi and Mir Ghasemi have modeled using the finite element method of dam behavior during construction and the first catchment. Then the hydraulic cracking of the core of this dam has been investigated using several methods. In the next step, the hydraulic failure at the core of Glabar Dam in Zanjan province has been studied using the relationships that have provided a correct prediction of the cracking of the Hittite Jute dam. To do this, pore water-stress coupling analysis was performed for the dam. Because of these assessments, it became clear that the Komak Panah and Ghanbari relations provide a correct prediction of the fragmentation of the core of the Hittite Jute Dam. Also, the occurrence of hydraulic failure in Glabar Dam was considered unlikely.

In the study of Shamsai and Mousavi, the failure parameters of embankment dams (failure width, failure slope, failure time, and maximum output flow rate) have been studied and evaluated using data related to 142 broken earth dams in the world. According to research, changing the geotechnical conditions in determining the time of shedding of the pipe created in the body of the dam, in the BREACH erosion model achieves better results compared to the actual broken dams in the world. Based on the obtained results, residential areas and important facilities at the bottom of the above-mentioned dam are at risk of flood failure [18].

In the study of Ghanbari et al. (2012), a quasi-static method for analyzing soil dams has been investigated by considering the effect of foundation (Case study: Masjed Soleiman Dam). The results show that paying attention to the simultaneous modeling of the body and foundation plays an important role in accurately predicting the seismic behavior of earthen dams. Accordingly, considering the real difficulty of the foundation and avoiding the rigid foundation assumption in the analysis will reduce the response to the acceleration of the dam. This result is consistent with the studies of Papalou and Bielak (2004).

In the paper of Kazemi et al. (2015), uncertainty related to the type of numerical analysis in calculating the dynamic deformation of soil slopes has been investigated. The calculation of the dynamic deformation in the earthen slopes caused by the earthquake has always been of interest to engineers and designers. The two main issues that should be considered in the design of permanent earth slopes, including earthen dams, backpacks, natural slopes, are the issue of stability of slopes and the control of deformations. It should be noted that even if a soil slopes are controlled in terms of stability, excessive deformation in these slopes during operation could reduce the efficiency of those slopes and cause problems. Therefore, it is important to calculate the deformations created in the slopes caused by the earthquake load. The calculation of the dynamic deformations of slopes always has uncertainty, including uncertainty in determining the severity of a possible earthquake, determining the physical characteristics of the soil, the used numerical method in determining these deformations, etc. The main purpose of this paper is to investigate the uncertainty related to the type of used numerical analysis in determining the created deformations in these slopes by FLAC2D and FLAC3D software. Finally, it can be concluded that the two-dimensional analysis of deformations is larger.

The different methods of limit equilibrium of static, dynamic stability and permanent leakage analysis of Zamkan earthen dam in Kermanshah province have been studied by Hazari and Ghobadian (2015). The results of various analyzes show that Zamkan dam is stable in static and dynamic mode, but for permanent leakage mode, the reliability coefficient obtained in some methods such as Janbo method, especially for downstream slope, should be less than the minimum allowable reliability coefficient. In general, the obtained results by different methods in static and dynamic mode are more convergent than in permanent leak mode. It is recommended that methods such as Morgenstern-Price or Spencer's method be used to analyze and determine the reliability coefficients of earthen slopes so that the obtained reliability coefficients in real conditions can easily meet the need for earthen slope stability.

Given the progress of studies in this area, the development of the earthquake effect should be considered as two components in seismic evaluation, despite a behavioral model. The influence of factors such as the height of the dam, the type of selected behavioral model for dam materials and the type of earthquake operation are important and determining factors in the complex behavior of the dam against earthquakes. In addition, the use of artificial neural networks to detect behavioral patterns consistent with dam behavior in numerical modeling, and to perform back-to-back analyzes can be suggested.

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