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ARTICLE

THE STABILITY ANALYSIS OF THE SLOPE OF THE EMBANKMENT DAM OF MOSHAMPA

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ABSTRACT

One of the important points in dam designs is that there are many methods to investigate the stability of embankment dams; among them, Limit Equilibrium methods have been mostly used in terms of simplicity and spending less time and costs. In this study, by numerical modeling method and using Geoslope software, it has been tried to analyze the stability of embankment dams in order to remedy the defects in the Limit Equilibrium methods. The study dam is Moshampa, which is a gravel reservoir dam with the central Natravay core. The results from the analyses with Geoslope software show that embankment dams with a clay core have an appropriate behavior in several cases in terms of stresses and seepage-caused forces and are stable in terms of statistics, as well as their implementation economically is more affordable than other types of dam.

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INTRODUCTION

Embankment dams are made of embankment materials that their behavior and properties can be changed by changes in the moisture content, changes in the stress extent, changes in the drainage position, changes in the time, or changes in other factors. This is one of the main distinctions between embankment structures and concrete and steel structures.

Due to basic need of the vast country Iran to harness and store the surface waters in order to provide water for drinking water consumption, water needed for agriculture and industry, electric power generation, control of flood and river flooding, etc; as well as the first step in the development and utilization of the country's water resources is construction of high dams.

With regard to the changes in the soil properties in contact with water and given that an embankment dam is in relation to water and its associated reservoir forces, so the stability of embankment dams has long been of interest to researchers and experts in dam engineering science.

With respect to the mentioned points and given that dams are structures that breaking and damaging them can cause irreparable life and financial damages, technical and economic considerations in the design of high dams in Iran have shown that in many cases, embankment dam with clay core is preferred over other options in dam designs and is a preferred choice in final design; so that statics show that in our country, of about 300 constructed and under-construction dams, more than 200 dams are of embankment dams type, which reveals the importance of this type of dams in many ways. Hence, controlling the stability of this type of dams seems to be an essential and serious issue. The central core in embankment dams and gravel dams has the task of sealing and storing water behind the dam. Due to the impermeability property of clay and its abundance in the nature, the central core is most often made of clay.

Structural integrity of the dam should be preserved during its performance or during events occurring in the operation time. For this purpose, the corresponding range of load and stress conditions will be discussed. Often, in all predictable events, the dam stability is provided by stresses placed on acceptable levels and the integrity of the dam core. In this study, due to the use of clay materials as the core of the dam, the stability issue of the dam slopes in the upstream and downstream will be discussed, and the dam Moshampa will be analyzed and evaluated as a case study.

In another study, it has been tried to investigate two scientific issues of the day, i.e. the slope stability and the neural network; and the slope stability coefficient of the embankment dam with the neural network was determined and compared with the outputs of the finite element Plaxis software. For training the neural network, the information obtained from 150 models of embankment dams in the finite element Plaxis software were used, and also the impact of effective parameters on the slope stability coefficient of embankment dams was determined [1].

During the investigation of a case study on Ilam dam by numerical modeling using Finite Element method and Geoslope software, the stability of embankment dams was analyzed to eliminate defects in the Limit Equilibrium method [2].

During another study, by the use of Geoslope software, the safety factors against slip and rupture on the slopes of an embankment dam with fixed specifications and geometry were investigated in two ways of Entry and Grid&Radius&exit, and it was shown that the results for the safety factor in the case Grid&Radius for the upstream and downstream slopes is about 75% more than the case Entry&Exit [3].

The stability analysis of embankment dam

In embankment dams, the core prevents water seepage. A segmentation embankment dam usually includes a central dense Natravay embankment core or an embankment core with slope toward upstream or a clay mixed core that its dimensions depend on the availability and specifications of materials in the site or close to it and will be required to prevent high seepage gradients. The core of embankment dam is run in the form of a rolled clay core or an impervious area such as a thick cover or a thin diaphragm or sheets of wood, steel, asphalt, or masonry. The following figure shows the rolled clay core of the case dam Moshampa which has been used in the analyses.

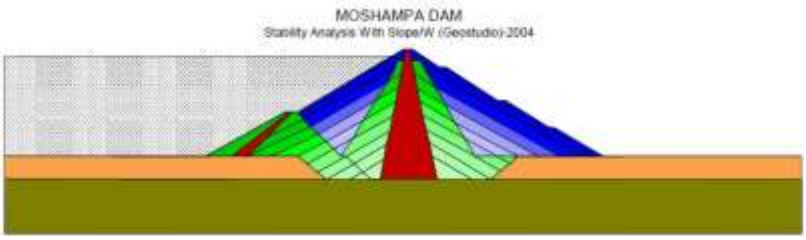


Fig. 1: The clay core of Moshampa dam.

The core in the place of embankment dam can be built in three forms and positions as follows:

1. Vertically in the middle or center of the core;
2. Slightly steeped towards the upstream of the dam;
3. Quite steeped towards the upstream of the dam.

If the downstream of the core has the slope 1V:0:5H or its slope is more towards the upstream, then the core is considered as moderately slanting core. The core is considered as slanting core if the shell of the downstream and the core have a self-stable slope at about 1V:1.25H. Usually, this amount of slope is used in the gravel dams in which the downstream sell, firstly, is made independently, and after a while, the filter and the core are run on the upstream. The upstream slope of the core is always selected more softened than that of the downstream because the water level increases as the water depth increases, and the core width should be increased.

There are different methods to investigate the static stability of the embankment dams. Some of these methods are: 1) limit equilibrium; 2) limit analysis (lower and upper limits of the plasticity theory); and 3) Finite Element method. The methods 1 and 3 are very common to determine the slope stability of ranges of an embankment dam. The Finite Element method is appropriate for the determination of distribution of stresses and strains within the levee and its foundation. These stresses and strains provide a complete feature of the behavior of the dam and offer the possibility to calculate the safety factor by comparing the shear strength on the surface in which the most vulnerable surface can be moved. Anyway, over the last few decades, the development of the Finite Element method combined with the increasing abilities of the strong computers allows designers of the embankment dams to analyze stresses and strains accurately and with real assumptions. The accuracy of the results, in fact, is only subject to the reliability of the input data on soil characteristics of the construction site.

In the analysis by the Finite Element method, the minimum and maximum principle stresses are calculated throughout the body and the foundation of the dam, and the shear stresses are determined from the results of the principle stresses. The resulted shear stresses are compared with the existent shear strength throughout the body and its foundation and then conclude the safety factor in the face with fracture. The Finite Element method for the first time was suggested by Clough & Woodwar for Geotechnical problems in 1967, but application of this method for analyzing the large embankment structures such as embankment dams was introduced by Dancan in 1996. In 1976, Clough and Zienkiewicz offered too general reports for the static analysis of the embankment dams. In 1976, Kullhaxy & Gurtowski studied the phenomenon of load transfer and Hydraulic Fracturing in the segmentation dams and they found that the phenomenon of load transfer in the embankment segmentation dams occur because of changing hardness in the adjacent areas. Keskin et al (2004) have investigated the effect of the Natravay core thickness and the seismic coefficient value 1 on the stability of both upstream and downstream slope of Kizilaca dam. Their aim has been to determine the final choice of the slopes of the Natravay core from the perspective of the optimal stability of the outer slopes of the dam, to construct economically the Natravay core and to determine whether or not withdraw the alluvium below the dam.
Based on the studies conducted by Bob & Mermel (1968) after the phenomenon of overtopping, the most important factor (over 30%) for the failure of embankment dams stems from the piping phenomenon. Cracks caused by the negative effective stresses from the impact of forces acting on the body of the dam

are one of the causes of this phenomenon, which can be estimated by the Finite Element method. So understanding the distribution of stress and deformation in the dam body to analyze cracks and development of them is inevitable.

The phenomenon of stress transfer or arching

One of the important and dangerous issues in the embankment dams is the arching phenomenon inside the dam body. This phenomenon consists of leaning the core (stress transfer) due to being softer than the stiff shell which reduces the vertical pressure on the core; (or vice versa) this phenomenon occurs due to the differences in the core density relative to the shell (or vice versa). The coefficient of arching within the core of embankment dams is obtained by the equation (1):

(1) $$\text{Arching coefficient} = \frac{\sigma_v}{y.h}$$

In which, σ_v is the total vertical pressure within the core (Kap), y is the specific gravity of the core (KN/m3), and h is the embankment height (m).
If stresses at each elevation value reduce to values smaller than water pressure at the same values because of arching, then it might result in Hydraulic Fracturing or formation of cracks caused by high water pressure.

Fracturing

Fracturing of embankment dams can be caused by the slope instability of the downstream and upstream of the dam. Typically, slipping occurs as a result of the loss of shear strength across weak planes or inside soils. With the release of uncontrolled waters hold behind the dam, it becomes clear that a small amount of fracturing has been occurred, and any abnormal appearance of the soil shear strength that is in contrary to the basic functions of the waters behind the dam properly represents the fracture, which can be a result of the arching phenomenon.

Comparing the Stability Analysis in the Finite Element Method and in the Limit Equilibrium Method
The main unknowns in the stability analysis of the roof slope in the Limit Equilibrium method is the normal stress at the bottom of each piece. The safety factor for each piece in the limit equilibrium method is same, and each piece has a balance of forces. The advantage of using the stresses resulted from the Finite Element method is that the safety factor is independent for each piece and finally calculates the total safety factor. In this case, the safety factor against the slipping is obtained by equation (2):

$$F.S = \frac{\sum_1 \tau_r \Delta L_1}{\sum_1 \tau_0 \Delta L_1} \qquad (2)$$

In which, is the shear strength of each part of the slipping surface, is the length of the slipping surface component, and is the shear stress in the i-th component surface. The value of is determined by the use of the stress analysis of each element and also is obtained by considering Mohr-Coulomb law and by the use of equation (3):

$$\tau_r = C + (\sigma_n - u)\tan\phi \qquad (3)$$

In which, u is the pore water pressure. The local concentration of the shear stresses could not certainly be achieved in the arrangement of the limit equilibrium, in which the normal stress of the bottom is basically caused by the weight of the piece, and this is one of the concentrations in the Limit Equilibrium method. The Finite Element Analysis is a method that, in addition to the safety factor against fracturing, investigates deformations as well, but the Limit Equilibrium methods achieve only the safety factor against fracturing. The accuracy of the embankment dams design depends not only on the prediction of the safety factors of slopes but also on the prediction of displacements. So, displacements can be restricted within the limits of acceptable that for this end, the use of analytical methods based on the stress-strain relations of materials is suggested.

Geoslope software

Geoslope software have the potential to enter the material properties close to the field conditions, and unlike other software, is able to analyze the semi-saturated environment. Program SIGMA/W is a Finite Element software product that can be used for the analysis of stress and deformation of embankment structures. Also, its comprehensive formulation helps to analyze both simple and too complicated issues. Material properties in SIGMA/W can be exploited by the use of effective stress parameters for the analysis of drained soils or by total stress parameters for the analysis of non-drained soils.

Pore water pressure during construction of dam

In materials indicated by the total stress parameters, changes in the non-drained water pressure can be determined from changes in the total stress using the Skempton pore water pressure parameters A and B. By specifying the material properties as total stress parameters, the options of the pore water pressure parameters A and B exist and the program SIGMA/W determines changes in the pore water pressure based on these parameters. The equation investigation for changes in the pore water pressure has been shown in equation (4) by the use of parameters A and B:

$$\Delta u = \beta(\frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3}) + \alpha\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2} \quad (4)$$

In which, A=α and B=β. The pore water pressure parameters A and B in the program SIGMA/W are defined using functions. The parameter B is defined as a function of the negative or positive pore water pressure which helps for the analysis of the saturated and unsaturated soil conditions. In this study, using the above-mentioned parameters A and B, the pore water pressures during the construction of the dam are estimated in the effect of the stress changes.

The relations governing on the stability analysis and slope/w software

The static analysis in this study has been done based on the application of the matric suction. In the stability analysis in the Limit Equilibrium method by the help of SLOPE/W, there are only two ways to apply the matric suction in the shear strength of unsaturated soil and the estimation of the safety factors; one of them is the determination of materials suction friction angle ( ) which does not have a high accuracy due to considering it as fixed in the depth by the software. The second way is the definition of moisture curve of materials for the estimation of the expected suction from this curve which acts much more accurate the previous method (Krahn, 2014). Thus, in this study, according to the equation (5) (Vanapalli et al., 1996), the moisture curve of materials has been used to calculate the shear strength of the unsaturated soil and finally to estimate the safety factors:

$$s = c' + (\sigma_n - u_a)\tan\phi' + (u_a - u_w)\left[\left(\frac{\theta_w - \theta_r}{\theta_s - \theta_r}\right)\right]\tan\phi' \quad (5)$$

In which, c' and are adhesion and effective internal friction angle, respectively; is the value of irreducible moisture; and is the moisture value at different suctions. The safety factors in the stability analysis by the Limit Equilibrium method as advised by Mahdavi et al. (2010) have been obtained based on the Morgenstern Method (Morgenstern, 1963). Because due to considering the synchronous effect of all the forces, it usually concludes a lower safety factor and has a less error percentage than other methods. The properties of the intended materials as the stuff of the homogeneous dam body was selected based on the unsaturated soil hydraulic database (UNSODA), and for this purpose, the soil sample with the code 2360 was selected from this database as clay soil with low permeability and was used in the modeling. The physical specifications, the average of the shear strength parameters, as well as the gradation curve of materials are presented in [Tables 1 and 2] and [Fig. 2].

Table 1: Specifications of the selected sample (Anon, 1996)

Sample number	Soil kind	The soil special dry weight (gr/cm^3)	The special weight of the soil solid grains (gr/cm^3)	The volumetric water content in the saturated mode	Saturated hydraulic conductivity (cm/day)
2360	clay	1.73	2.573	0.492	5

Table 2: Specifications of the selected sample (Anon, 1996)

Soil kind	Optimal moisture (%)	Adhesion in the compressed mode (C_0) (KN/m^2)	Adhesion in the saturated mode (C_{sat}) (KN/m^2)	Internal friction angle (degree)
Sticky clay soil	17.3	91	14	28

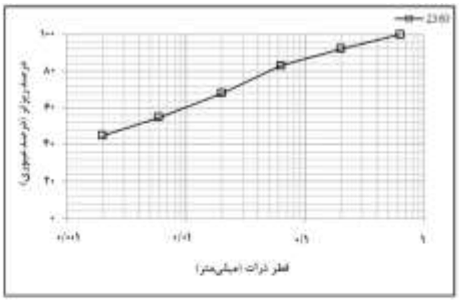
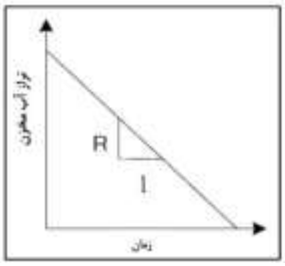


Fig. 2: Grained curve of the selected sample.

Determination of the parameters under investigation in the analysis for the seepage and stability of the reservoir rapid drawdown

The rapid drawdown occurs when the water level in the vicinity of the slope falls so sharply that the soil inside the slope does not have enough time to drain and the fryatic level inside the slope could not appropriately follow the final level of the reservoir water and stays for a while in its original location

(Griffiths & Lane, 2000). One of the most important factors in the phenomenon of the rapid drawdown is drawdown ratio and drawdown rate. The drawdown ratio is expressed as L/H, where as shown in Figure 3, L is the value of drop in the reservoir level due to the drawdown and H is the balance of normal water level in the reservoir. Also, according to Figure4, the drawdown rate means the dropped balance of the reservoir water as compared to the time, which is shown by R, and its unit is centimeters per day (cm.day) or meters per day (m.day) (Berilgen, 2007). America Development Organization (Anon, 1987) has suggested the critical drawdown rate as equivalent to 0.5 feet (15 cm) per day. To investigate the effect of the drawdown rate on all the analyses, four rates 0.15, 0.5, 1, and 2 were used.



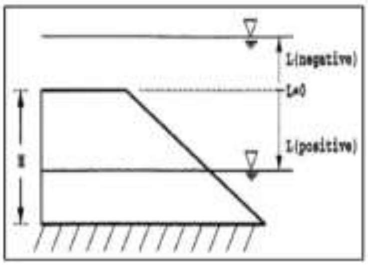


Fig. 4: Drawdown rate (Berilgen, 2007). **Fig. 3:** Drawdown ratio (Griffiths & Lane,2000).

Soil water characteristic curve was estimated by the use of four methods existing in seep/w software. In addition to them, based on the real data UNSODA, a total of five kinds of soil water characteristic curves were obtained. Also, the permeability functions were estimated based on the three methods available in seep/w software and by the default of the above characteristic curve. The statistic parameters of Root Mean Square Error (RMSE) and the indicator Nash-Sutcliffe (NSE) were calculated to determine the curve by the Best Estimation in relation to the real data UNSODA (KBE) and the Worst Estimation in relation to the real data UNSODA (KWE) and were used in its modeling according to [Table 3]. In addition to the selection of the above permeability functions, the use of linear estimation (Ksat) (Linear Estimation: in zero mutric suction, K=Ksal and in mutric suction=-100kpa, K=Ksal/100), which is equal to the saturated hydraulic conductivity and is independent of mutric suction and the function obtained from the real data (KUNSODA), generally has led to the application of five kinds of hydraulic conductivity function in all the analyses. An example of hydraulic specifications of materials in order to use in the modeling has been provided in [Table 3]. In addition to the stability analysis, the soil water functions indicated in the following table has been used in Transient State Seepage Analysis (rapid drawdown of the reservoir) as well.

Table 3: Hydraulic conductivity and sticky soil water functions in order to use in the modeling

Hydraulic conductivity functions	Soil water functions	Symptoms
K_{BE}	θ_{BE}	$K_{BE}\ (\theta_{BE})$
K_{WE}	θ_{WE}	$K_{WE}\ (\theta_{WE})$
K_{LE}	θ_{BE}	$K_{LE}\ (\theta_{BE})$
K_{LE}	θ_{LE}	$K_{LE}\ (\theta_{LE})$
K_{LE}	θ_{sat}	$K_{LE}\ (\theta_{sat})$
K_{UNSODA}	θ_{UNSODA}	$K_{UNSODA}\ (\theta_{UNSODA})$
K_{sat}	θ_{sat}	$K_{sat}\ (\theta_{sat})$

Designation of the optimal dimensions of the homogeneous dam level

In this study, the optimal geometric dimensions have been calculated based on the genetic algorithm provided by Montasery et al. (2010) located on the impermeable foundation, which almost is in accordance with the existing design criteria; and comparing to the method proposed by American Development Organization, it suggests smaller dimensions, but it has similar results when comparing to the values proposed by Sherard. The above algorithm has been proposed to design the optimal dimensions of heterogeneous embankment dams. Thus in this study, to determine the optimal dimensions of homogeneous embankment dams, the same stuff of core and shell materials are selected, and the optimal dimensions are calculated based on this. Data necessary to determine the optimal dimensions of the homogeneous embankment dams section are: dam height (H), dam crest width (b), the water level of the dam upstream (h), the effective adhesion and angle of materials internal friction, the maximum dry density and permeability density of materials.

RESULTS

Dynamic analysis of the dam moshampa

In this section, dynamic analyses of the dam Moshampa on critical section are examined. The history of input acceleration, the geometry of model, material properties and the method to determine displacements are provided and finally, residue displacements are calculated.

Analysis using equivalent linear method

In the Equivalent Linear Method, three parameters are taken into consideration: maximum shear modulus Gmax, changes in the modulus reduction ratio G/Gmax, and damping with the shear strain. The solution is completely linear elastic, but by some times of trails and errors the best shear modulus and the intended damping for the equilibrium equation are calculated.

Determination of displacements the slipping block newmark

In the Limit Equilibrium method, the safety factors less than 1 in the quasi-static conditions represent the movement of the slipping surface under investigation. In these circumstances, in fact, the earthquake force is more than the resistance of the considered slope, and the difference between these quantities leads to applying the force on the slipping block and finally displaces the block. This idea was proposed by Newmark for the first time. The difference between the induced acceleration and the critical acceleration ky results in the speed in the sipping block, and finally, displaces the block. In fact, during an earthquake, the slipping mass moves across the plane from the moment that the safety factor is less than 1, and stops at the time and location in which the stress from earthquake is less than the resistance of the slipping surface, and the balance is reestablished. Based on the relative displacement quantity of the slipping surface that may be in the range of a few centimeters to several meters, the stability and instability of the dam is investigated.

The slope stability of the dam body is estimated by the use of a computer program GEO-SLPOE/W-2004. In this program is based on the methods Bishop and Spencer, and is able to estimate the safety factor on the fracture levels in the circular, wedge or any arbitrary form by the use of body segmentation methods. In this program, the fracture level related to the least safety factor is obtained, which is compared with the allowed safety factor under development by authorities.

Seismic input on dynamic analyses

The conducted dynamic analyses are based on the earthquake history in MCL conditions. The earthquake acceleration history and acceleration response spectra with 5% damping has been obtained. The history of input acceleration has been obtained by reforming the acceleration history of water station. In addition, to have more reliability and control in the stability calculations based on the recommendations of the Southern California Earthquake Center Regulations “DMG 117 Guidelines”, for seismic coefficient K=0.15 with a minimum of safety factor S.F=1.1 and finally for an earthquake with a magnitude M=7.4 and amax=0.43g, seismic coefficient K=0.20 (which is equal to the maximum accelerations of designation of the seismic countries such as “Turkey, Korea, and Japan” as well) with a minimum of safety factor S.F=1.00 was done as well. The history of the crest acceleration is as in the [Fig. 5].

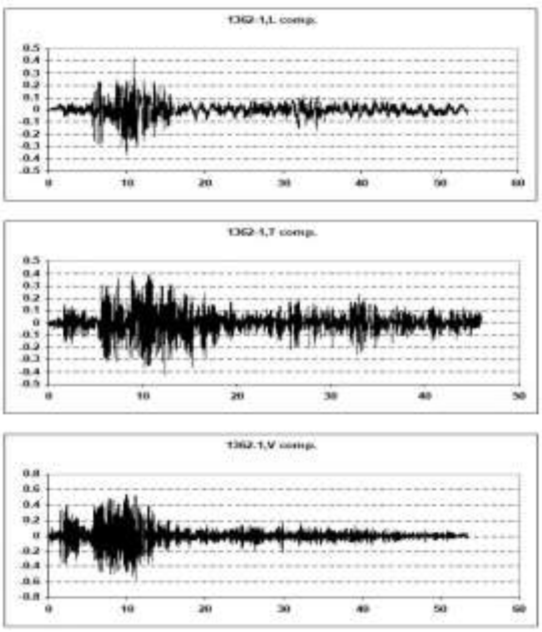


Fig. 5: The acceleration of the reformed water map for MCL level.

Materials Properties in Dynamic Analyses

As mentioned previously, the dynamic analyses have been done by the Equivalent Linear Method. It was noted that usually, there are three basic parameters as follows:

The maximum shear modulus of materials
Shear modulus reduction with increased shear strain
Increased damping with increased shear strain
All above parameters are obtained from the special dynamic experiments. Also, some relations offered by different researchers can be used that in these studies this way has been applied.

Table 4: The maximum shear modulus for the dam body

Material kind	Area	Maximum shear modulus (Gmax)	Reference
Clay core	1	$3270\,\frac{(2.97-e)^2}{1+e}(\sigma_0')^{0.5}$	Hardin & Black (1968)
Filter and drainer	2	$220K_{2\max}(\sigma_0')^{0.5}$	Seed & Edriss (1970)
Corner-rounded sell stone	3	$8400\,\frac{(2.17-e)^2}{1+e}(\sigma_0')^{0.55}$	Kokusho & Esashi (1981)
Rockfill	4	$\frac{13000\times(2.17-e)^2}{1+e}(\sigma_0')^{0.55}$	Kokusho & Esashi (1981)

Table 4: the values of critical accelerations and residue displacements in the upstream and downstream slipping surfaces

Slope	Slipping block	Critical acceleration coefficient (Ky)	Horizontal displacement of residue (cm)	Vertical displacement (cm)
Upstream	SU1	0/19	44	21
	SU2	0/24	83	40
	SU3	0/25	107	51
	SU4	0/27	145	69
Downstream	SD1	0/29	0	0
	SD2	0/43	9	5
	SD3	0/45	23	12
	SD4	0/54	24	12

Slope Stability Analyses of Slopes of Moshampa Dam Body

1. Critical sections
2. Load mode
3. The mode of construction ending
4. The mode of stable seepage
5. The mode of half-full reservoir
6. The mode of rapid water drop in reservoir

Table 6: The allowed intended safety factors in slope stability analyses

Mode	Load mode	Load mode	Minimum of allowed safety factor
1	End of construction	1/4	For the upstream and downstream slope
2	Half-full reservoir	1/5	For the upstream slope
3	Rapid water drop in reservoir from the normal balance	1/2	For the upstream slope
4	Stable seepage from the normal balance	1/5	For the downstream slope
5	Mode 1 with earthquake loading	1/15	Seismic coefficient equal to 0.1g
6	Mode 1 with earthquake loading	1/10	Seismic coefficient equal to 0.12g
7	Mode 1 with earthquake loading	1/00	Seismic coefficient equal to 0.15g
8	Modes 2 and 4 with earthquake loading	1/15	Seismic coefficient equal to 0.12g
9	Modes 2 and 4 with earthquake loading	1/10	Seismic coefficient equal to 0.15g
10	Modes 2 and 4 with earthquake loading	1/00	Seismic coefficient equal to 0.2g

Overall Conclusion by the Limit Equilibrium Method

From the results of the slope stability analyses conducted on the critical sections of the dam in different loading modes and in static and quasi-static conditions, it can be concluded that, in general, the stability of much of the dam, which is located in riverbed and alluvial foundation, determines the slopes of the dam body. According to the analyses results, the stability of the dam slopes on this location in static conditions has been well provided; and in the quasi-static conditions, the stability of the dam slopes are well provided as well.

Static analyses and seepage analyses in the finite element method by the plaxis software

The results of the seepage analyses are expressed to determine the seepage amount of the body and foundation of the dam and the flow gradients and, in general, to investigate the stability against the seepage forces.

The Aim of Seepage Calculations

1. To determine the equi-potential lines, water pressure, and current
2. To calculate the output rate of the body and foundation;
3. To calculate the hydraulic gradients in the dam core and current focus points.

Sections under Examination in the Seepage Analyses

According to the longitudinal profile and cross-sections of the dam, it is observed that there is an alluvial foundation with depth ~22m and length ~120m from the riverbed bellow on the dam axis, which is exploited to seal the foundation from bellow the core. To gain an estimation of the output rate and the gradients available in the foundation, the rock foundation section with depth 200m has been modeled.

Table 7: Permeability characteristics of the materials of the body of the dam Moshampa

Material kind	Clay core	Filter	Drainer	Upstream and downstream shell	Rockfill in the exterior shell of the upstream and downstream
Permeability coefficient (cm/s)	$K_x = 1\times10^{-6}$ $K_y = 1\times10^{-7}$	$K = 1\times10^{-4}$	$K = 0.5$	$K = 1\times10^{-3}$	$K = 0.1$

Table 8: The results of seepage analysis in section 1 (in the riverbed) of Moshampa dam with the sealing curtain in different depths

Depth of the sealing curtain	Maximum gradients in the clay core	Average gradient in the foundation	Maximum gradient in downstream output of the core	Lifting force in downstream	Total output rate	Total output rate	Output rate ( )
0	1/7	1	0/2	1/5	274	8650000	$1/37\times10^{-3}$
60	1/7	0/7	0/1	1	220	6940000	$1/1\times10^{-3}$
80	1/7	0/6	0/1	0/9	195	6160000	976×10^{-6}
100	1/7	0/4	0/8	0/7	173	5450000	868×10^{-6}
120	1/7	0/3	0/6	0/6	154	4850000	768×10^{-6}
140	1/7	0/3	0/6	0/5	134	4230000	670×10^{-6}

Table 9: The results of seepage analysis in section 2 (section in the flanks) of Moshampa dam with the sealing curtain in different depths

Depth of the sealing curtain	Maximum gradients in the clay core	Average gradient in the foundation	Maximum gradient in downstream output of the core	Lifting force in downstream	Total output rate	Total output rate	Output rate ( )
0	1/4	1	0/2	2/2	330	10500000	$1/65\times10^{-3}$
70	1/4	0/5	0/15	0/9	170	5250000	833×10^{-6}
90	1/4	0/4	0/1	0/7	132	4160000	660×10^{-6}
120	1/4	0/3	0/05	0/5	108	3400000	540×10^{-6}

140	1/4	0/2	0/05	0/4	97	3050000	485×10^{-6}

CONCLUSION AND SUGGESTIONS

With regard to the complicated permeability texture in the foundation of Moshampa, two different sections have been considered to analyze the seepage and to determine the output rate and the hydraulic gradients in the core and downstream of the body. To obtain the total output rate with respect to the permeability texture of the foundation, the length of sections 1 and 2 has been considered equal to 200m. For section 1 (section in the riverbed) in the case with and without sealing curtain with depths 60, 80, 100, 120, and 140, the seepage analyses have been done. For section 2 (section in the flanks), the sealing curtain with depth 70, 90, 120, and 140 has been analyzed as well.
The output rate from the total body and the foundation resulted from all section in the case without the sealing curtain has been about 20000000m3/year, which is significantly reduced by embedding the sealing curtain with different depths. Also, value of the average hydraulic gradient of the lifting force in the foundation and the downstream foundation is reduced significantly by embedding the sealing curtain. With regard to the permeable texture of the foundation rock and that the permeable texture implies in the lack of a rock with a very low permeability at depths of up to 120m and by embedding a sealing curtain with depth 120m, the hydraulic gradients are in the allowable mode; and based on the analysis results, the deepness of the sealing curtain to more than 200m does not have a significant impact on the output rate reduction, and with respect to this issue that sealing injection for most dams constructed in the world that do not have a foundation with a very low permeability at low depths usually is in the water height, in this project, the depth of sealing curtain is suggested equal to 120m. In this case, with respect to the analyses, the total output rate at about 260Ls is reduced equal to 8200000, which is equivalent to ~1% of the reservoir volume that is in permitted level.

CONFLICT OF INTEREST
There is no conflict of interest.

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None

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