

Assessment and Presentation of a Treatment Method to Seepage Problems of the Alluvial Foundation of Ghordanloo Dam, NE Iran

HEDAYATI TALOUKI, HOSSEIN¹, LASHKARIPOUR, GHOLAM REZA¹,
GHAFOORI, MOHAMMAD² and SABA, ABBAS ALI³

¹Department of Geology, Ferdowsi University of Mashhad, Iran

²Department of Geology, Ferdowsi University of Mashhad

³Technician, Toos Ab Consulting Engineers Company

Email: hhossien1364@yahoo.com; lashkaripour@um.ac.ir

Abstract: The objective of this research is to assess the seepage problems in the Ghordanloo dam foundation and to present a proper method of water-proofing. The dam will be built on the Atrak river in the northeast of Iran. The abutments of this dam will be constructed on limestone and shale that belong to the lower Cretaceous. The maximum thickness of the alluvium in the dam axis is up to 60 meters. Due to the presence of large thickness of alluvium, with various permeabilities in the dam foundation, it is necessary to prevent seepage through the dam foundation. Hence, based on the results of Lofran tests the permeability of alluvium was determined. According to the results, the permeability of the alluvium was between 10^{-2} to 10^{-6} cm/s. Then, the dam body and its alluvial foundations were modeled using the SEEP/W software. Potential of water seepage, at different positions and depths of the cutoff wall, was also determined on the basis of numerical analyses. Finally, based on the amount of seepage, values for hydraulic gradient and safety factors, a proper method of water-proofing was proposed.

Keywords: Cutoff wall, Seepage, Alluvial foundations, Hydraulic gradient, Iran.

INTRODUCTION

Water stored in the dam reservoirs causes substantial changes in the net flow in the discontinuities and alluvial deposits of dam sites. This can increase the erodibility and natural permeability of sediments and rock masses (Ewert, 1997). Therefore, seepage is one of the most important phenomena that threaten the stability of dams. Hence, many researchers have studied the problem of water seepage in dam sites. Cho (2012) evaluated the effect of the changes of hydraulic gradient of soil layer on seepage by using probability analysis. Sjö Dahl et al. (2010) using the resistivity method for seepage monitoring detected internal erosion in the Røssvatn embankment dam. Malik et al. (2008) analyzed the Satpara dam in Pakistan by using the SEEP/W software and proposed a suitable method for water-proofing. Uromeihy and Barzegari (2007) analyzed the efficiency of the cutoff wall and grout curtain of the Chapar-Abad dam in Iran in order to select and assess an optimal water-proofing method. Feng and Wu (2006) by using the SEEP software investigated the impact of parameters such as soil layer thickness, soil hydraulic gradient, dam width, and cutoff wall depth

on the properties of currents flowing into dam foundation. Ghobadi et al. (2005) analyzed the problem of seepage through the right abutment of the Karun-1 dam in Iran. Nusier et al. (2002) and Malkawi and Al-Sheriadeh (2000) studied the seepage problem in the site of Kafrein dam in Jordan. Due to the importance of water leakage, this research is concerned with the assessment of the amount of seepage through the Ghordanloo dam foundation and suggestion of a treatment method for water-proofing.

The Ghordanloo dam on the Atrak river, is located 36 km northwest of Bojnord city, in north Khorasan province. The geographic coordinates of the dam site are $57^{\circ}19'$ E and $37^{\circ}37'$ N. The path to access the dam and the dam location are shown in Fig.1.

Ghordanloo dam is an earth-fill dam with central clay core that will be constructed with an height of 46 meters, the crest length 236 meter, and a reservoir capacity of 220 million cubic meters (Toosaab Co, 2009). This dam is going to be constructed with an aim to meet in part the drinking water demand of Bojnord city as well as Bojnord downtown agricultural and industrial water demand.

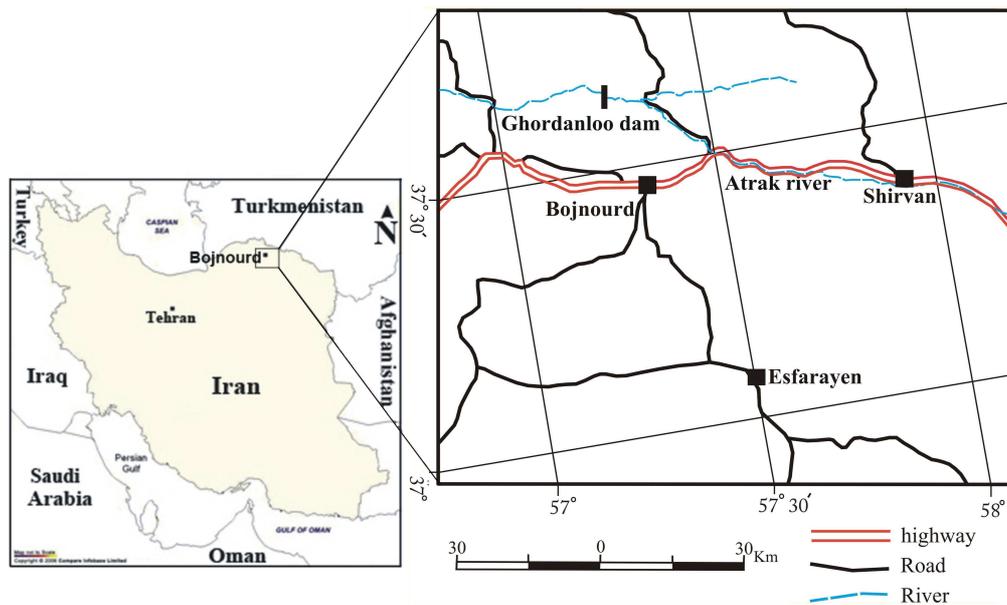


Fig1. Geographic coordinates and path to access of the dam site.

SITE GEOLOGY

Information on the regional geology of the area has been given by Afshar-Harb (1984) and the Toossab Engineering Company (2009). The geological studies confirm that the valley of the dam site is affected by morphotectonic activities. The valley has an asymmetric U-shape and with the slope dip between 27° to 40° in the right abutment and between 50° to 78° in the left abutment. Tectonic forces have greatly affected the dam site and have created many faults. Local faults located in this site are basically right-lateral and left-lateral strike-slip faults with a northwest-northeast strike. These faults have led to the formation of a narrow and rather deep valley (with a width of approximately 90 meters). Figures 2 and 3 show the geological map and cross section (along the A-B path, see fig 2) of the dam, respectively. In stratigraphic terms, the dam site comprises of Tirgan and Sarcheshme formations and Quaternary deposits.

Tirgan Formation (KTr)

The lithology of Tirgan Formation is oolitic limestone with thin layers of marly limestone and marl. Because of the high roughness and hardness of the layers, the Tirgan limestone can be easily identified from the shale and marly deposits of Sarcheshmah Formation. Age of this formation is between Barremian and Aptian and forms most of the abutments and the bed rock of this dam site.

Sarcheshmah Formation (KSr)

The lithology of Sarcheshmah Formation is composed

of gray-dark gray marly shale with lime layers, which are highly weathered and fragmented. This formation belongs to Aptian age and covers the upper part of the abutments (higher than the dam crest) and outcrops up to the middle part of the reservoir.

Quaternary Deposits (Qt)

These deposits include river bed deposits, alluvial terraces and the talus.

The river bed deposits are mainly composed of fine grained material and hardly has coarse grained material.

Alluvial terraces deposits are scattered along the river banks and all over the valley. The central parts of the strait mainly include fine grained material, and abutment surroundings are of coarse aggregates.

Talus material is mainly the result of erosion of shale and crushed limestone zones, and are spreaded on the hillside and foot of the abutments. These materials mostly include coarse and angular particles ranging from boulders to rock blocks. On the other hand, these materials are slightly thick and are located on older lithostratigraphic units. These deposits are the most recent sedimentary units in the dam site.

SITE ENGINEERING GEOLOGY

Twenty nine boreholes with maximum depths of 120 meters and total depth of 2050.90 meters (754.45 meters into the alluvium and 1296.45 meters into rocks) were drilled, in order to evaluate the engineering and geological

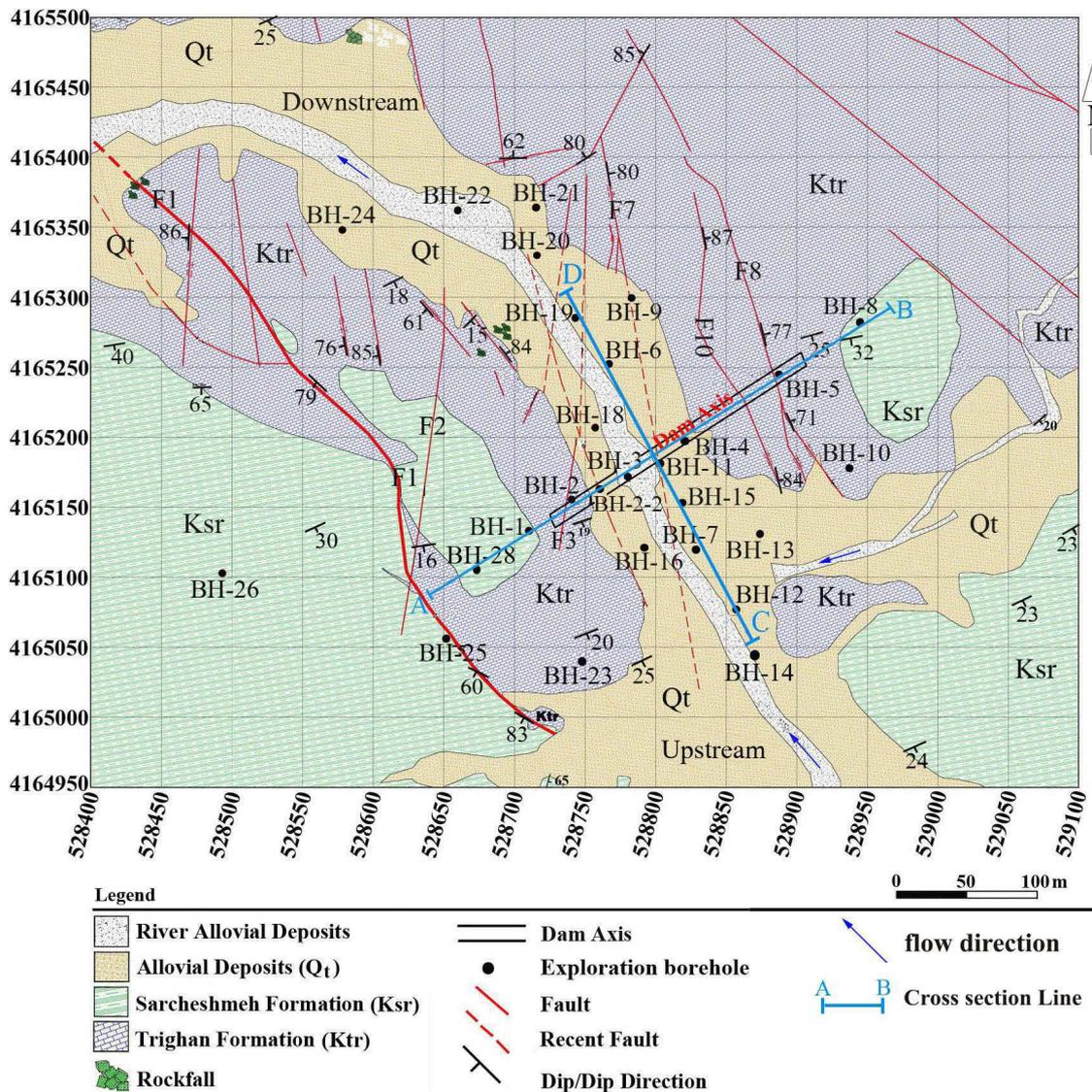


Fig. 2. Geological plan map of the dam area.

characteristics of the site (Figs. 2 and 3). Overall, 442 in-situ permeability tests (216 Lofran tests and 226 Lugeon tests) were performed on the alluvium and rock mass in the site (Table 1). According to the geotechnical and excavation studies the maximum thickness of the soil in the dam valley is 60.50 meters. Based on the results of the laboratory tests, the soils in the dam foundation can be divided into two major groups.

CL and CL-ML Unit

According to the field investigation, this sedimentary unit covers a large part of the dam foundation. These sediments are resulted from the erosion of shale and marls spread over the reservoir and catchment of the dam. Based on sieve analysis, hydrometry test, and Atterberg limits

test these soils are classified as lean clay with sand and low plasticity (CL) and silty clay with low plasticity (CL-ML) by Unified Soil Classification System (ASTM D 2487, 2006; Das, 2008). The saturation density of this unit is 20.60 KN/m².

GC-GM unit

This unit is classified as silty clayey gravel with sand (GC-GM) by Unified Soil Classification System (ASTM D 2487, 2006; Das, 2008). Although these materials belong to the group of coarse-grained soils, their strength and permeability are mostly dependent on their fine-grained contents as they have high amount of silt and clay. The saturation density of this unit is 21.00 KN/m².

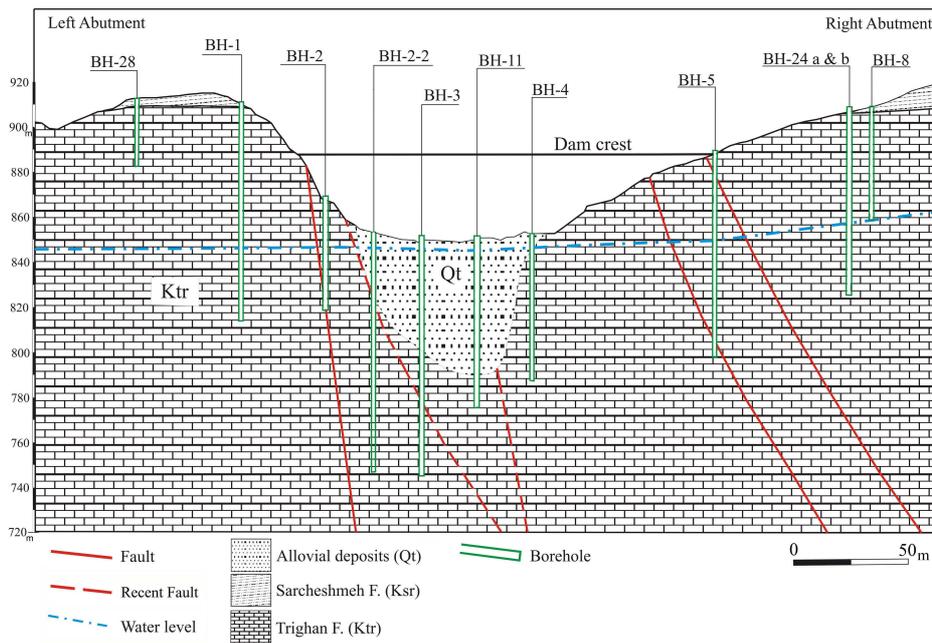


Fig. 3. Cross-section of the dam axis along the A-B path (see fig 2)

ASSESSMENT OF THE PERMEABILITY OF SEDIMENTS

Soils have variable degrees of permeability. Krahn (2009) stated that seepage through soils is highly controlled by their permeability. Therefore, accurate assessment of

seepage is dependent on accurate assessment of the permeability of sediments. On the other hand, in order to prevent piping, it is necessary to control seepage and guarantee the stability of the dam (Sherard, 1968).

Hence, at the beginning of this phase the extent of spread

Table 1. Excavated Borehole details and characters in the Dam area

Boreholes	Depth of drilling (m)			In-situ Permeability Test		Borehole location
	Total	Alluvial	Rock mass	Lofran	Lugeon	
BH 1	96.80	0	96.80	-	18	Left Abutment (Dam axis)
BH 2	50	-	50	-	9	Left Abutment (Dam axis)
BH 2-2	106.20	28	78.20	-	14	Left Abutment (Dam axis)
BH 3	104.6	49.20	55.40	14	10	Dam axis
BH 4	65.15	8	57.15	3	11	Dam axis
BH 5	91.40	-	91.40	-	17	Riverbed (Dam axis)
BH 6	70.20	56.50	13.70	16	3	Riverbed (Downstream)
BH 7	73.90	50	23.90	16	4	Riverbed (Upstream)
BH 8	50.80	-	50.80	-	10	Right abutment (Dam axis)
BH 9	47.75	37.1	10.65	12	2	Riverbed (Downstream)
BH 10	31.10	-	31.10	-	6	Right abutment
BH 11	75.4	60.50	14.90	19	1	Riverbed (Dam axis)
BH 12	91.2	51.5	39.7	17	7	Riverbed (Upstream)
BH 13	74	42.50	31.50	12	6	Riverbed (Upstream)
BH 15	72.50	55	17.50	16	2	Riverbed (Upstream)
BH 16	67	39	28	11	5	Riverbed (Upstream)
BH 18	83	20	63	5	12	Riverbed (Downstream)
BH 19	82	58.15	23.85	16	4	Riverbed (Downstream)
BH 20	85	63	22	19	4	Riverbed (Downstream)
BH 21	40	9	31	2	6	Riverbed (Downstream)
BH 22	85	59	26	18	5	Riverbed (Downstream)
BH 23	55	-	55	0	10	Left abutment (Diversion tunnel)
BH 24	50	22	28	4	5	Left abutment (Diversion tunnel)
BH 24a	75	-	75	-	14	Right abutment (Grouting Site)
BH 24b	65	-	65	-	13	Right abutment (Grouting Site)
BH 25	60	-	60	-	10	Left abutment (Over fall tunnel)
BH 26	120	-	120	-	13	Left abutment (Over fall tunnel)
BH 27	53	43	10	16	1	Left abutment (Diversion tunnel)
BH 28	30	-	30	-	4	Left Abutment

of the substrata (CL, CL-ML and GC-GM units) was determined precisely based on the information obtained from excavated boreholes logs. Figure 4 shows the cross section of sediments in the dam foundation and along the dam axis. Figure 5 also shows the cross section of sediments along the C-D path (see Fig. 2).

Based on the field investigation and excavated boreholes, it can be said that coarse-grained soils around the dam are mainly placed between fine-grained soils as intermediary layers. As we move along the dam reservoir, the amount of fine-grained soils is reduced and the amount of coarse-grained soils increases (Fig 5). In order to assess the permeability of the foundation strata, Lofran constant-head test and falling-head test were carried out. At some sections, excessive water loss was encountered and the tests failed. So, the results of the Lofran tests were used to assess permeability of the sub-strata. In Figure 6 the range of permeability values and the trend of permeability variations at each borehole are presented. As seen from this diagram, the dam foundation has usually a permeability of 10^{-2} to 10^{-6} cm/s. In addition, the trend of permeability variations at the dam foundation does not follow a certain rule and zones with high levels of permeability can be seen at great depths.

Terzaghi and Peck (1967) stated that in a stratified soil with different permeability coefficients, it is necessary to determine an equivalent permeability coefficient. Therefore, in order to perform a more precise assessment of seepage, the equivalent permeability value of the dam foundation was determined based on relations 1, 2, and 3 (Das, 2008) and according to the results of Lofran tests (Fig 6), and on the description of materials in boreholes logs. Table 2 shows the values obtained for the equivalent permeability of the soil foundation, cutoff wall, crest and core of the dam, which is used in the modeling.

$$K_h = \frac{H_1 K_1 + H_2 K_2 + H_3 K_3 + \dots + H_n K_n}{H} \quad (1)$$

Table 2. Equivalent permeability for each section of the dam and foundation

Zone	Permeability (cm/s)
Soil foundation	8.18×10^{-4}
Crest	1×10^{-3}
Core	1×10^{-7}
Cutoff walls	1×10^{-8}

$$K_v = \frac{H}{\left(\frac{H_1}{K_1}\right) + \left(\frac{H_2}{K_2}\right) + \left(\frac{H_3}{K_3}\right) + \dots + \left(\frac{H_n}{K_n}\right)} \quad (2)$$

$$K_{av} = \sqrt{K_v + K_h} \quad (3)$$

where K_n : Hydraulic conductivities of the individual layers. H_n : Thickness of the individual layers. K_h : Average hydraulic conductivity in the horizontal direction. K_v : Average hydraulic conductivity in the vertical direction. H : Total thickness of the layers. K_{av} : Equivalent permeability hydraulic conductivity in the soil.

NUMERICAL ANALYSIS OF SEEPAGE USING THE SEEP/W SOFTWARE

Different methods such as excavation of alluvium, grout curtain, cutoff wall, and clay blanket are available for waterproofing the dams sites. The amount of seepage, hydraulic gradient, safety factor and cost are the important factors in selecting a treatment method. Due to the large thickness of the sediments, expanded reservoir of the dam, and the costs, it seems employing cutoff wall is the best way to waterproof this dam. Hence, based on the estimated amount of seepage, hydraulic gradient, and safety factor the efficiency and optimal depth of the cutoff wall were assessed. To achieve this goal, the SEEP/W software and the finite element method were put to use. The SEEP/W software is the finite element software that can mathematically model the seepage by using the numerical analysis method (Krahn, 2009).

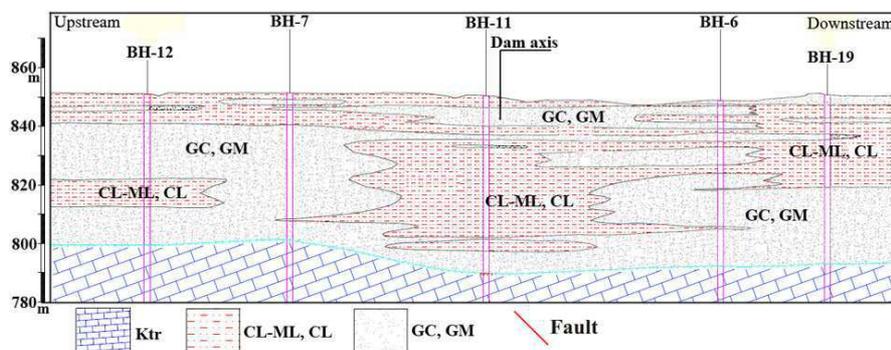


Fig. 4. Cross-section of the soil deposits in the dam foundation.

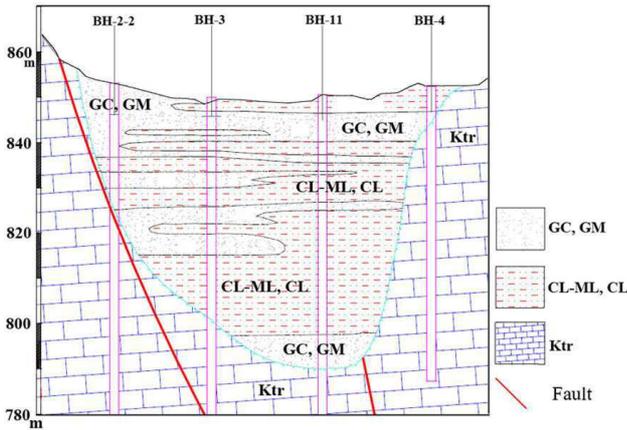


Fig. 5. Cross-section of the soil deposits along the C-D path (see fig 2).

In order to assess the amount of seepage and the optimal treatment method, the dam and its alluvial foundation were modeled by the software (Figure 7). Based on the information obtained from boreholes, the thickness of the alluvium is 60 meters along the dam axis and is 52 meters in the upstream. Water level in the dam reservoir is assumed to be equal to the maximum level (41 meters). In order to simplify the complex situation in the dam foundation, some hypotheses were used. It was assumed that underground water flows is in a steady state in saturated environments

and follows the Darcy’s law. The soil was homogeneous and saturated and the degree of permeability of sediments in the dam reservoir was equal to the degree of permeability at the dam foundation. During the analyses the cutoff wall was assumed to be made of plastic concrete with a thickness of 1 meter. Four geotechnical layers were considered and permeability coefficient of the defined layers was determined (Table 2). After putting the required information into the software, two dimensional (2D) meshing was carried out by using foursquare elements (Fig. 9a). 42778 nodes and 42202 elements were created for modeling. Finally, the amount of seepage through the dam foundation and its hydraulic gradient were analyzed in the following four situations:

- a In natural conditions and without using any treatment method.
- b Model □: When the cutoff wall is located beneath the dam core.
- c Model □□: When the bed of the upstream crest is waterproofed by using a clay blanket (similar to the one used for the materials used in dam core) with a thickness of 4 meters. In this case, the cutoff wall is located in upstream toe.
- d Cutoff walls were simulated for various depths of 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 52, 55, and 60 meters for model □ and □□.

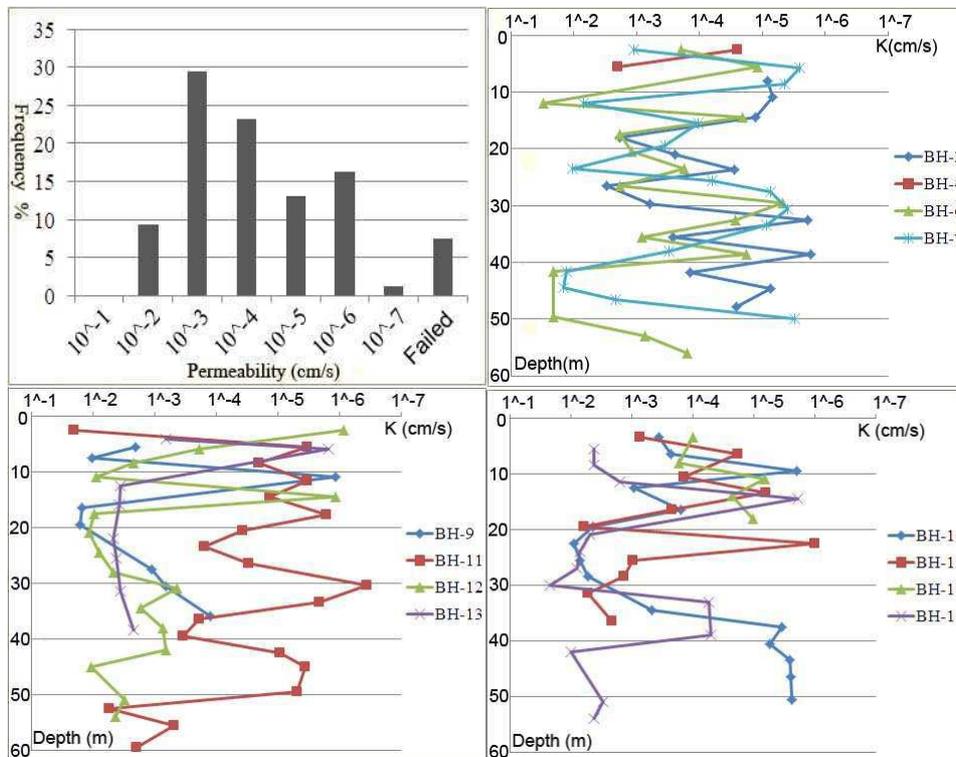


Fig. 6. Graph of frequency and changes in soil permeability

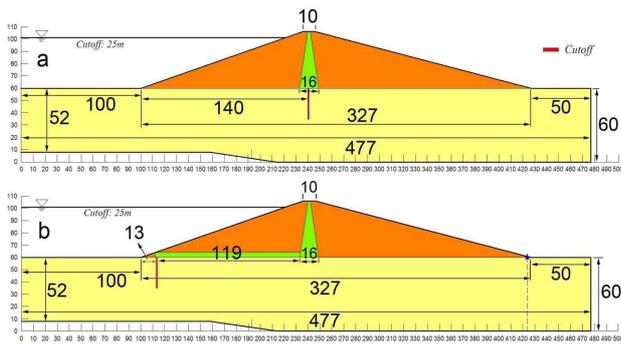


Fig. 7. Dam and the foundation model in software, a: model □, b: model □□ (in m).

Table 3 shows the assessed values of seepage in different states. Based on the analyses it can be said that in natural conditions (Fig.9b), when no treatment method is employed, the annual amount of seepage is 2.018 million cubic meters, the safety factors is 0.96, and the hydraulic gradient is 1.10. By repeating the simulation 25 times, the amount of seepage through the dam foundation in different situations and at different depths of cutoff wall was analyzed (Table 3).

Based on the performed analyses it is known that by using clay blanket beneath the upstream crest, without cutoff

Table 3. The results of the assessment for dam foundation seepage

Cutoff depth (m)	Model	Amount of seepage (m ³ /year)	Hydraulic gradient	Safety factor
State A	□	2.018 × 10 ⁶	1.10	0.96
State B	□□	1.339 × 10 ⁶	0.92	1.15
5	□	1.997 × 10 ⁶	1.07	0.99
	□□	1.322 × 10 ⁶	0.90	1.17
10	□	1.952 × 10 ⁶	1.05	1
	□□	1.296 × 10 ⁶	0.87	1.21
15	□	1.896 × 10 ⁶	0.98	1.08
	□□	1.264 × 10 ⁶	0.86	1.23
20	□	1.832 × 10 ⁶	0.98	1.10
	□□	1.232 × 10 ⁶	0.84	1.26
25	□	1.762 × 10 ⁶	0.95	1.11
	□□	1.192 × 10 ⁶	0.82	1.29
30	□	1.685 × 10 ⁶	0.90	1.18
	□□	1.147 × 10 ⁶	0.78	1.36
35	□	1.602 × 10 ⁶	0.85	1.25
	□□	1.093 × 10 ⁶	0.74	1.43
40	□	1.488 × 10 ⁶	0.79	1.34
	□□	1.026 × 10 ⁶	0.69	1.53
45	□	1.397 × 10 ⁶	0.75	1.41
	□□	9.368 × 10 ⁵	0.64	1.65
50	□	1.269 × 10 ⁶	0.68	1.55
	□□	7.654 × 10 ⁵	0.52	2.03
55	□	1.009 × 10 ⁶	0.57	1.86
52	□□	3.98 × 10 ⁴	0.025	42.4
60	□	1.2 × 10 ⁴	0.01	106

State A: In natural conditions and without using any treatment method, State B: The bed of the upstream crust is water-proofed by using a clay blanket (without cutoff wall)

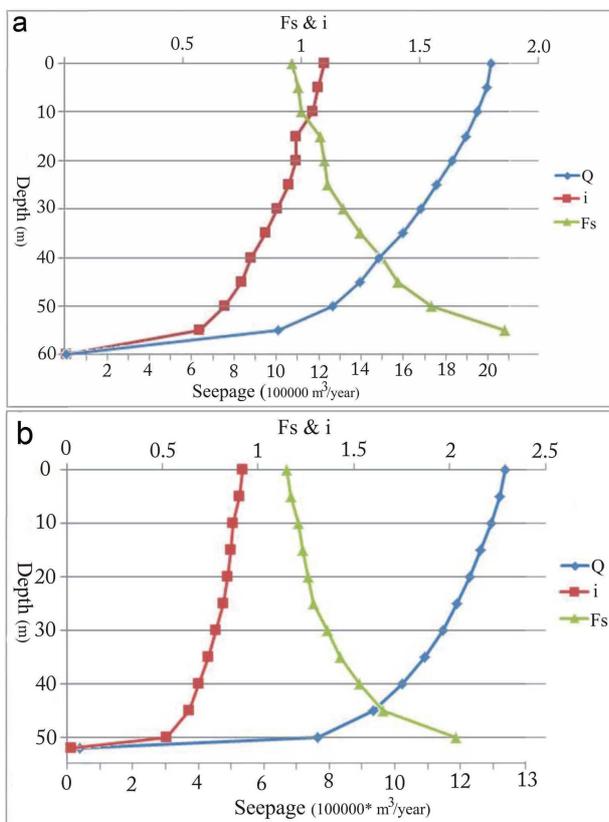


Fig. 8. The amount of seepage (Q), hydraulic gradient (i) and safety factors (Fs) Changes vs cutoff depth, a: model □, b: model □□

wall (Fig. 9e), the amount of seepage can be reduced by 36% (Table 3, state B). The reason is that by using clay blankets the length of the flow path is increased and thus the amount of seepage and hydraulic gradient is decreased. When clay blanket is used in dam reservoir, it may be subject to failure in case of fast drawdown of the reservoir water level uplift and in this case the downward load on the blanket is reduced before the uplift pressures can decrease themselves (Sowers, 1962).

In addition, using clay blanket in dams with vast reservoirs is costly. But, since in Model □□ clay blanket is placed beneath the crest, the risk of uplift failure is eliminated. According to Table 3 and Fig.8, the optimal depth of the cutoff wall is equal to 40 meters and it will be constructed in the upstream toe (Fig. 9f). Consequently the amount of seepage is reduced by 50% and hydraulic gradient and safety factors will be equal to 0.69 and 1.53, respectively (Fig.8b).

CONCLUSIONS

In this paper, by using engineering and geological properties of the Ghordanloo dam an optimal method was introduced for dam foundation water-proofing prior to construction. The maximum thickness of the sediments at the dam foundation is 60.50 meter. Seepage and hydraulic

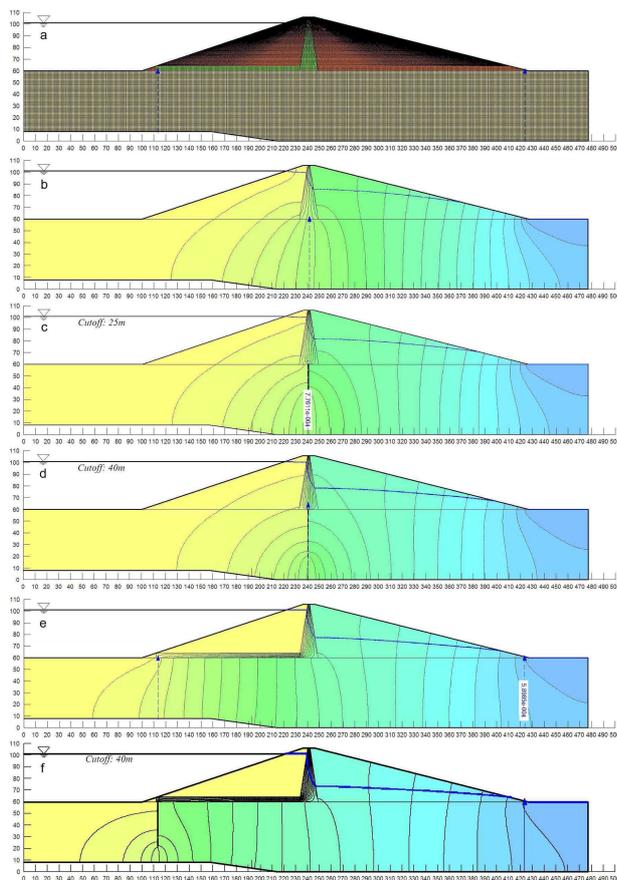


Fig. 9. Results of seepage analysis, **a:** deformed mesh for model □, **b:** In natural conditions (State A), **c:** model □ with cutoff depth 25m, **d:** model □ with cutoff depth 40m, **e:** model □□ (state B), **f:** model □□ with cutoff depth 40m.

gradient are the two parameters that contribute to the selection of a proper water-proofing method. The results of the analyses obtained by the SEEP/W software were used in determining the optimal depth and location for water-proofing the dam foundation. Based on the aforementioned studies it was concluded that placing a cutoff wall with a depth of 40 meters in the dam upstream toe (Model □□) is the most appropriate method for controlling seepage, reducing hydraulic gradient, and increasing safety factors. When clay blanket is used in dam reservoir, it may be subjected to failure by uplift in case of fast drawdown of the reservoir water level. But, since in Model □□ clay blanket is placed beneath the crest, the risk of failure by uplift is eliminated. On the other hand, clay blankets add to the length of flow paths and consequently lead to a reduction in the amount of seepage and hydraulic gradient. Therefore, using clay blanket beneath the upstream crest of dams and placing cutoff walls is a proper way for resisting seepage and increasing safety factors of dams (especially the ones built on alluvial deposits with high thicknesses).

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