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Back analyses of Maroon embankment dam

Analeses arrières du barrage maroon de remblai

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ABSTRACT
Maroon dam is one of the largest embankment dams in Iran, which is located in south west of the country. Because of the importance of this dam, a complete monitoring program with a regular observation has been done during and after construction.
To evaluate the stability of the dam body at present and at loading conditions which may be experienced in future, a large amount of data obtained from instrumentation system has been processed carefully and are used for back analyses with numerical method. Aim of these back analyses is to estimate strength and deformation parameters of different embankment material zones. The back analyses are performed at end of construction and after reservoir filling. For instance, changes in displacement, pore pressure and stress in specific points of dam body are compared with those histories obtained from back analyses.

RÉSUMÉ
Le barrage Marron est un des plus grands barrages de remblai en Iran qui est situé dans le sud à l'ouest du pays. En raison de l'importance de ce barrage, un programme de contrôle complet avec une observation régulière a été fait pendant et après la construction.
Pour évaluer la stabilité du corps de barrage actuellement et aux conditions de charge qui peuvent être expérimentées à l'avenir, une grande quantité de données obtenues à partir du système d'instrumentation a été traitée soigneusement et employée pour des analyses arrières avec la méthode numérique. Le but de ces derniers des analyses arrières est d'estimer des paramètres de force et de déformation de différents matériaux de remblai. Les analyses arrières sont exécutées à la fin de la construction et après le remplissage de réservoir. Pour ceci, des changements du déplacement, de la pression de pore et de l'effort dans les points spécifiques de corps de barrage sont comparés à ces histoires obtenues des analyses arrières.

1 INTRODUCTION

Maroon embankment dam has a height of 170m from the minimum elevation at the river bed. It has a central clay core which is a little inclined to the upstream. Length of the crest is 345m and the large reservoir has a volume of about 1200 million cubic meters. The dam is completely founded on a hard limestone of Asmari formation which has a thickness of about 370m at the dam site. Typical cross section of dam is shown in Figure 1.

For monitoring the behavior of dam during and after construction, some instruments have been used for identification of displacements, pore water pressure and stresses in the clay core and other zones of dam. Available as built information for interpretation and back analyses consists of following data: the construction trend curves (height against time), gradation envelopes and dry densities of different zones, moisture-density data for each layer of the clay core and the permeability of rock fill and sandy gravely shells.

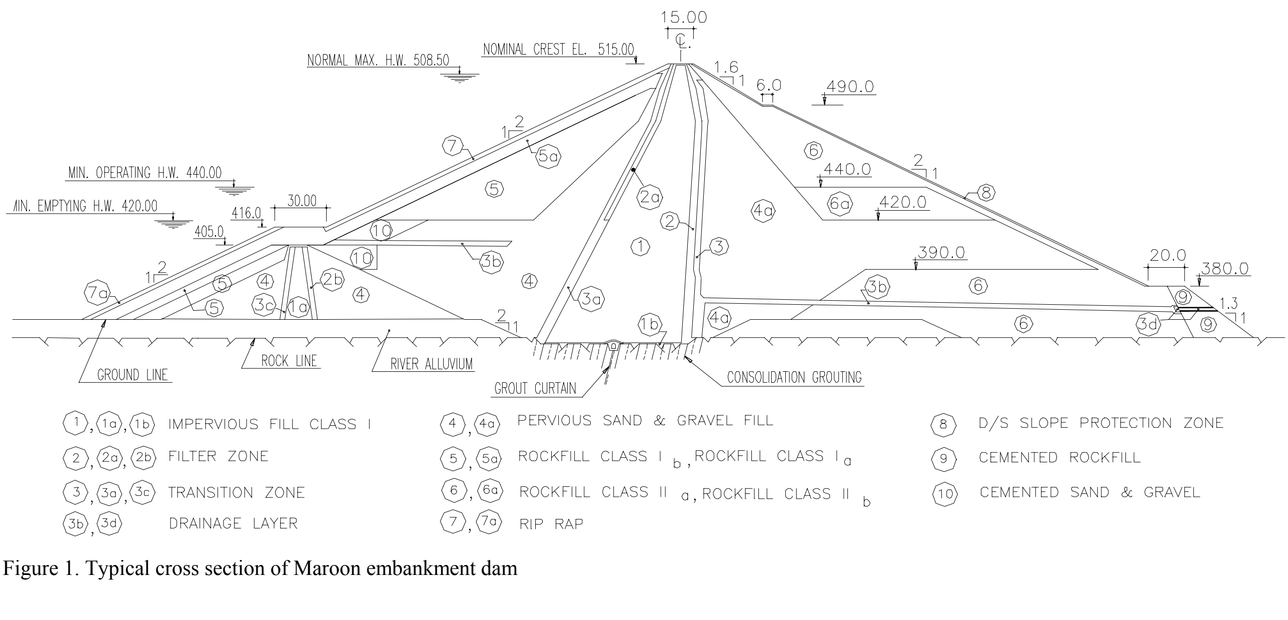

Figure 1. Typical cross section of Maroon embankment dam

Instrumentation system of the dam consists of electric piezometers, stand pipe piezometers, electric total pressure cells, inclinometers, survey stations, seepage collecting-measuring system, water level gauge and accelerographs. The instruments are installed at five cross sections. Focus of this study is mostly on the data obtained from the central and also the largest instrumentation section, which is shown in Figure 2. Inclinometers A, B, C and D and points P1 to P7 are used for comparing the results of monitoring with numerical back analyses.

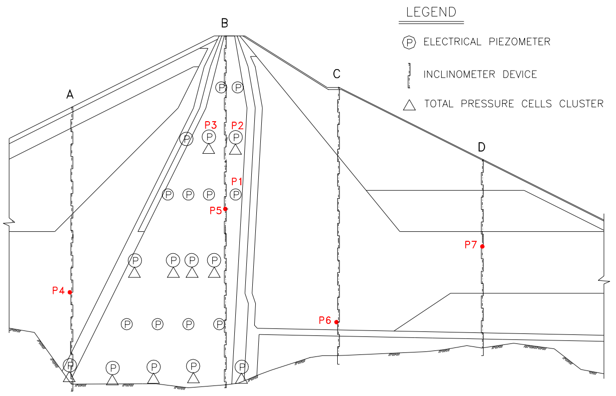

Figure 2. Location of Electric piezometers, inclinometers and pressure cells in maximum section

The interpretations and back analyses of this study are based on the instrumentation data measured during construction and impounding which covers a time period of about 2500 days from the beginning of construction. The first 970 days of this time is the construction period.

To evaluate the real stability condition of Maroon embankment dam, correct deformation and strength parameters of the materials of dam body should be calculated. For instance, back analyses are done with considering the principles of soil mechanics. Some iterative procedures are used to find these parameters for clay core, gravel and rock fill materials. For simulation of the dam behavior, finite difference code FLAC4 has been used in back analysis. In this program both flow and equilibrium equations are considered to be active during whole process of analyses from beginning to end of construction and up to 4 years after construction. In fact both end of construction and impounding are analyzed in this study. Numerical analyses are conducted for maximum section of dam. In Figure 3, finite difference mesh of the dam body is shown.

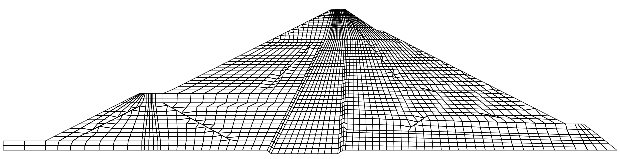

Figure 3. Mesh generation for back analyses

2 CONSTITUTIVE MODEL

Simple constitutive models have limited parameters that help to control the trials and errors process of back analyses with changing the parameters. For instance Mohr-Coulomb model is used for all parts of the dam.

Back analyses are performed and strength and deformation parameters of different zones of the dam body are calculated. In the following parts, results of analyses are discussed in three categories: pore pressure development, deformation field and stress field.

3 DEVELOPMENT OF EXCESS PORE WATER PRESSURE IN CORE MATERIAL

Data of pore pressure obtained from electric piezometers during and after construction of dam show that distribution of pore pressure in the core material is not homogenous. In fact increasing of the pore pressure in the upper part of clay core is more than lower part in such a way that maximum ratio of excess pore pressure to overburden pressure ($r_u=\Delta p/\gamma h$) in the lower and upper part of clay core are about 0.35 and 0.45, respectively. This is because lower part of the clay core was compacted with less moisture content and had less percentage of saturation. Thus different parts of the clay core have different saturation ratio, which affects the ultimate distribution of pore pressure.

Development of pore pressure depends on some parameters such as permeability of material, water bulk modulus, skeleton bulk modulus (or deformability) of clay core and drainage condition in the surrounding media. Here, identification of permeability of clay core and water bulk modulus is discussed and identification of deformation parameters, which have a great influence on settlements are discussed in the next part.

3.1 *Permeability of clay core*

Reduction rate of pore pressure after construction is very low and this causes to very low coefficient of permeability for clay core (Figure 4). For initial estimation of permeability of clay core, Terzaghi's one dimensional consolidation theory (1943) is used:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \tag{1}$$

$$c_v = \frac{k}{\gamma_w m_v} \tag{2}$$

where u is excess pore pressure, z is length in direction of dissipation, c_v is coefficient of consolidation, k is permeability of soil, γ_ω is unit weight of water and m_v is coefficient of volume compressibility of soil.

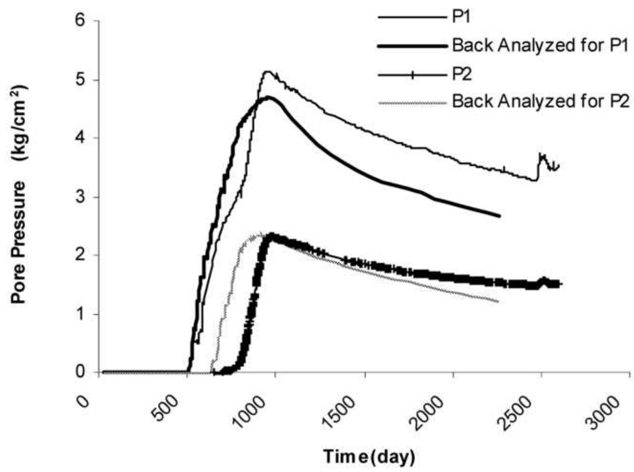

Figure 4. Changes in the excess pore pressure during and after dam construction

Pore pressure time histories obtained from piezometers show that pore pressure in the clay core increases until end of construction and then decreases. Some of these curves are shown in Figure 4. In each curve, reduction of pore pressure in time t after the end of construction is known. Thus degree of consolidation can be fined in each time after the end of construction. From these degrees of consolidation, non-dimensional time factor (T_v) is determined. With considering the probable length of drainage path (H) of each electric piezometer during time t, coefficient of consolidation and permeability of clay core are determined from equation 3 and 2, respectively :

$$T_v = \frac{c_v t}{H^2} \tag{3}$$

With previous equations, permeability of clay core on each location of electric piezometers is calculated and is in the range of 10^{-9} to 10^{-8} cm/s. Thus in the first numerical back analysis, permeability of clay core is used in this domain. With comparing the results of pore pressures obtained from numerical analyses and those obtained from observation, average permeability of clay core is reached to 5×10^{-8} cm/s. Results of back analyses for some points are shown in Figure 4. It is obvious that results are very close to real pore pressure curves.

3.2 *Water bulk modulus*

In the numerical models, water bulk modulus has great effect on ultimate pore pressure developed in the clay core during construction of the dam. As it mentioned before, lower parts of clay core was compacted with lower moisture content and was unsaturated. Due to this situation, bulk modulus of water should be very lower than pure water (water without any air content). The inclusion of even 1% air in the soil is sufficient to significantly decrease the water bulk (Fredlund and Rahardjo, 1993). Hence different water bulk modulus should be applied in lower and upper part of clay core. These changes are used for simulation of unsaturated behavior of soils especially in lower part of the clay core. With some analyses and doing trials and errors, water bulk modulus of lower and upper part of clay core are obtained and are 3.5MPa and 100Mpa, respectively.

Distribution of pore pressure in the clay core after impounding is shown in Figure 5. It is shown that with above parameters, pore pressures from analyses are very close to exact distribution from observation.

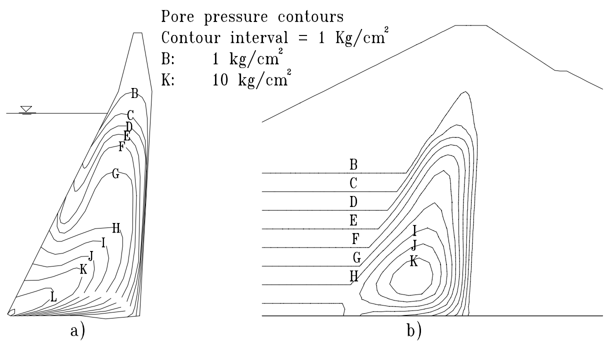

Figure 5. Distribution of pore pressure a)From observation and b) From back analyses

4 DISTRIBUTION OF DISPLACEMENTS

Data from inclinometers are used for identification of deformability parameters of dam body. In the maximum section of dam, inclinometers are placed in 4 locations, in the upstream shell, clay core and downstream shell which are shown in Figure 2. Elastic modulus, cohesion and friction angle of the embankment materials are obtained from comparing the back analyses results with the monitoring data of inclinometers. Deformability parameters are changed in such a way that displacement curves of points in the numerical analyses reach to exact settlements measured by inclinometers. Finally the deformability parameters in whole part of dam are calculated and are shown in Table 1.

Table 1. Material properties of the dam body obtained from back analyses

Zone \ Properties	γ_d g/cm³	k cm/s	E Kg/cm²	ν	C Kg/cm²	φ (deg)	ψ (deg)
Clay Core	1.71	5E-8~5E-9	120~140	0.32	0.5	24~26	0
Filter and Draiange	2.1	1.E-04	400	0.25	0	35	2
Upstream Gravelly Shell	2.17	0.017	1200~1400	0.22	0	35~37	4
Upstream Rock fill	2	0.76	200~400	0.25	0	36~38	5
Downstream Gravelly Shell	2.17	0.017	800~1200	0.25	0	34~36	4
Downstream Rock fill	2	0.76	600~750	0.25	0	33~35	5

In Figure 6 time histories of settlement from numerical back analyses at 4 points in the location of inclinometers are compared with observed data. It can be seen that changes in the displacements are very close to observed data especially during construction.

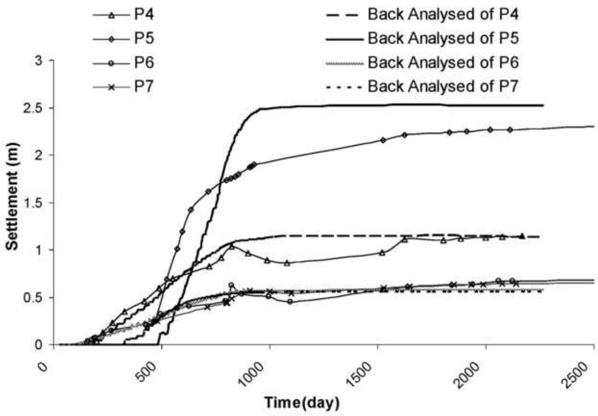

Figure 6. Time histories of settlement from observation and back analyses in 4 points of embankment dam

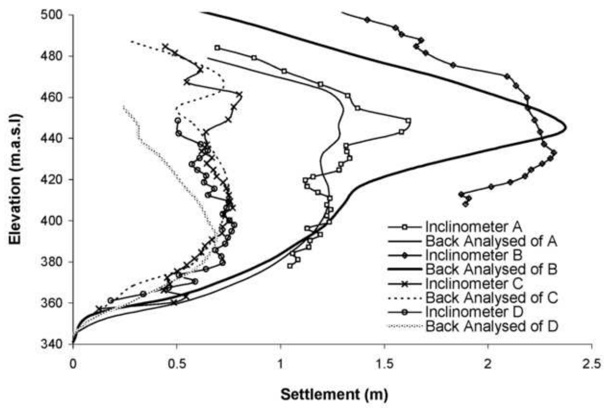

Figure 7. Profiles of settlement in direction of inclinometers, 4 years after the end of construction

Profiles of settlement from numerical back analyses are compared with those from observation data, which is shown in Figure 7. It is shown that in the upstream inclinometer, the settlement curve has two maximum points: one in the sandy gravel material and the other in rock fill material which cause to two different deformability parameters for rock fill and sandy gravel material. Maximum settlement of upstream inclinometer at the end of construction and 4 years after that are 1 m and 1.6 m, respectively. In the inclinometer which is located in the clay core only one maximum settlement occurs, which is approximately in the mid height of clay core and is about 2.25m. Also for inclinometer C in the downstream of the dam, the settlement curve has two maximum points because of two different materials in the path of the inclinometer. Maximum settlement 4 years after construction is 0.8m. In the inclinometer D at downstream of dam body only one peak of displacement is seen and is about 0.75m.

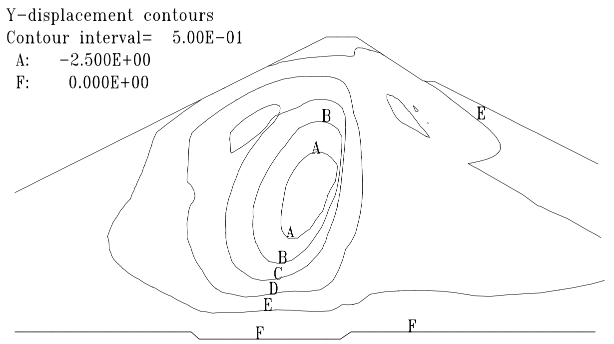

Figure 8. Pattern of settlements from back analyses, 4 years after the end of construction

It can be seen that settlement of the rock fill material in upstream and downstream shell are a little more than sandy gravel material. To model these differences, rock fill material should have more deformability which cause to low elastic and strength parameters. Settlement in the rock fill after construction is still increasing, but rate of increasing is very low. Pattern of settlement from back analyses at 4 years after end of construction is shown in Figure 8.

5 STRESSES IN CLAY CORE

Due to low elastic modulus of clay core and development of pore water pressure in it, reduction in stress is seen in the clay core in comparison to other materials of dam body. This phenomenon is called Arching. Measured stresses from pressure cells show that value of arching (ratio of developed vertical stress to overburden pressure (γh)) at clay core is between 0.45-0.6 which is in allowable range. Results of back analyses show that maximum value of arching in clay core is between 0.5-0.55. Thus results of back analyses are in good agreement with real measurements. Pattern of vertical stresses from back analyses is shown in Figure 9, which shows a very sharp reduction in the stress in the clay core.

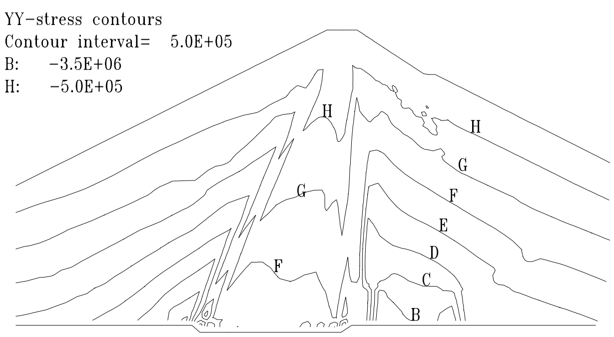

Figure 9. Pattern of vertical stresses from back analyses

Another way for proving the correctness of strength parameters of clay core is using effective stress path at those points in which variation of both total stress and pore pressure are recorded at the same time. In Figure 10, effective stress path of points P2 and P3 in the clay core are shown. It is shown that real stress path of these points at the end of loading tends to failure criterion from back analyses (c = 0.5 Kg/cm2 and $\phi = 26^{o}$). This Figure shows that strength parameters of clay core were defined in a proper manner.

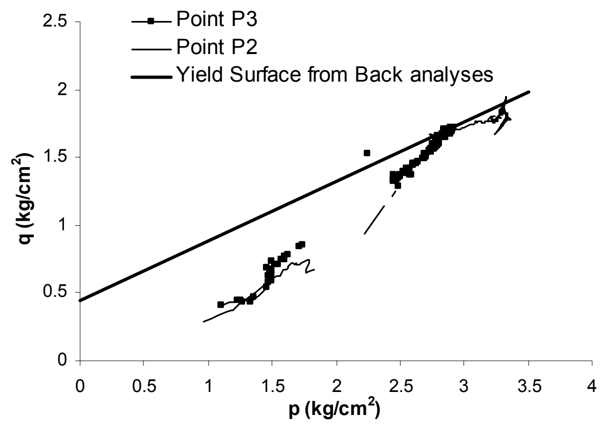

Figure 10. Comparison of real effective stress path with failure criterion from back analyses

6 CONCLUSION

Data of instrumentation are used for evaluation of deformation and strength parameters of materials of a large embankment dam. Data of electrical piezometers show that the dissipation rate of pore pressure in clay core is very small. With this notification, back analyses results give very low permeability for clay core.

The strength and deformation parameters of dam body are estimated from comparison between the results of instrumentation and back analyses. It is shown that elastic parameters of rock fill materials in both upstream and downstream shells of the dam are lower than sandy gravel material. Also in back analyses, clay core should have appropriate cohesion (in the effective stress analyses) to capture the observation data.

Effective stress path of some specific points in clay core material is obtained from instrumentation data and is used for estimation of strength parameters. These parameters are in good agreement with the same parameters, which are obtained from back analyses.

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