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ABSTRACT

Azadi rockfill dam is located on Zemkan River in the west of Iran. There is a deep permeable zone in the rock mass of the foundation. So, the grouting galleries are considered in two different levels to sealing the foundation successfully. The lower gallery is more critical and is studied in this paper. The critical sections that are studied are at the deepest zone of left abutment, deepest zone of right abutment and the middle part of the dam axis. In this paper, the stability conditions of the mentioned sections are studied using different methods including empirical, analytical and numerical (FDM and DEM). The groundwater condition has been considered in DEM analysis and finally, the suitable support systems are suggested based on coupled hydromechanical numerical modeling.

INTRODUCTION

Overview

The Design and analysis of stability of tunnels includes a combination of different methods. At the first stage, the preliminary study is performed using empirical methods that are related to initial support system design. These methods are the most economical methods. At the second stage, the analytical methods are used. The numerical methods and computer models are not only used for stress-strain analysis but also are prepared for initial support estimation of a special project.

Grouting and drainage galleries are common in concrete, earth and rockfill dams. Adits have been used for grouting and drainage of abutment and foundation. Adits and galleries are constructed prior to placement of the embankment as reinforced concrete structures in bedrock. Grouting galleries have been used as an expedient to allow the embankment to be constructed and grouting to be done during or following completion of earthwork. Some possible benefits from using adits and galleries in earth- and rockfill dams are (U.S. Army Corps of Engineers, 1984):

(1) Construction of the embankment can be carried out independently of the grouting schedule.

(2) The advantages of grouting with the additional weight imposed on the foundation (higher grout pressures) can be realized, while most of the objection to grouting through the embankment can be eliminated.

(3) Adits are also excellent exploratory tools that give detailed data on the nature of the rock discontinuities to be treated.

(4) Galleries and adits allow access to the foundation during reservoir filling and after it. So the additional grouting can be planned and results evaluated from direct observations.

(5) If galleries and adits are used for drainage holes, pressure can be partially relieved immediately downstream of the grout curtain.

(6) Galleries and adits can be used to build foundation instrumentation outlets. Design of the gallery and impervious section of the dam must consider that the full reservoir head will be dissipated through the core immediately above the gallery.

Design and construction of tunnels in rock require thought processes and procedures that are in many ways different from other design and construction projects, because the principal construction material is the rock mass itself rather than an engineered material. Uncertainties persist in the properties of the rock materials and in the way the rock mass and the groundwater will behave. Sound, flexible design and redundancies and safeguards must overcome these uncertainties during construction. More than for any other type of structure, the design of tunnels must involve selection or anticipation of methods of construction [Hoek et al, 1995].

Considering the variability and complexity of geologic materials and the variety of demands posed on finished underground structures, it is not surprising that standards or codes of design for tunnels are hard to find. Adding to the complexity is the fact that many aspects of rock mass behavior are not well understood and that the design of man-made components to stabilize the rock requires consideration of strain compatibility with the rock mass. Understanding rock mass response to tunnel construction is necessary to assess opening stability and opening support requirements. Several approaches of varying complexity have been developed to help the designer understand rock mass response. The methods cannot consider all aspects of rock behavior, but are useful in quantifying rock response and providing guidance in support design.

Azadi rockfill dam has about 56 million cubic meters storage capacity and 51 meter height. There is a high permeable zone in the foundation with 120 meters depth. At this condition, there are many difficulties to sealing the foundation. Therefore, it is decided to excavate four galleries in specific levels to grout the foundation. These access galleries have designed with modified horseshoe section. These galleries have 3 meters width and 3.5 meters height to carry the drilling and grouting equipments and other activities in it. The depth of critical section in left abutment, right abutment and middle of dam are 70, 40 and 58 meters, respectively. At the present paper, the stability and support system required for different parts of access and grouting galleries of Azadi rockfill dam is studied using different methods containing empirical, analytical and numerical (FDM and DEM). Because of the high depth of permeable zone in foundation of the dam, the grouting galleries have to be excavated in two different levels. The critical zones of these galleries (e.g. the end parts of galleries in right and left abutment which have lower level and higher overburden and middle of the gallery in bottom of the dam the)

Geological Aspects

Lithology, Morphology and Tectonics. The rock mass of left abutment and riverbed is made of shale-marl-limestone of Goorpy formation and Amiran Shale formation is seen in the right abutment. The valley shape is wide U-shaped. The left abutment has a uniform slope in 30 degree and is continued from level 1258 to level 1360 and the right abutment is from 1258 to 1280 with slope about 25 degree and after that, the slope is decrease to about 10 degree (Figure (1)). The axis of dam is a part of symmetric anticline which northern side of it has been erodes. The river direction is parallel to the axis of anticline (Figure (1)).

Description of Discontinuity. Discontinuities are the most important factor, which affect on rock mass behavior. The characteristics of discontinuities in the abutments are shown in table (1). The other parameters are spacing (0.2-0.6 meter), extension (greater than 20 meter), JRC (2.3), aperture (0.5-2.5 millimeter), weathering (unweathered to slightly weathered) and filling (calcite, marl, pyrite, organic material).

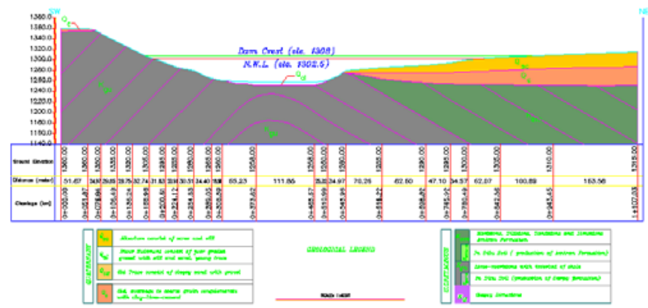

Fig. 1. Geological map of dam axis

Table 1. Characteristics of discontinuities

	Type of Discontinuity	Orientation: Dip Direction (Deg.)	Orientation: Dip (Deg.)
Right Abutment	Joint Set 1	312	68
	Joint Set 2	125	78
	Bedding	50	45
Left Abutment	Joint Set 1	312	68
	Joint Set 2	125	78
	Bedding	225	43

GEOTECHNICAL PARAMETERS

Intact Rock Properties

The laboratory tests including uniaxial, triaxial and Brazilian and index tests were done on intact rock samples. The results are listed in Table (2).

Table 2. Results of laboratory test on intact rock samples

E (GPa)	ν	γ (kg/m^2)	σ_c (MPa)	σ_t (MPa)	C (MPa)	ϕ (Deg.)	m*
6.7	0.2	2600	30	2.8	7	31	8

* Material constant in Hoek - Brown criteria [Hoek, 1992]

Discontinuities Properties

Based on available equations and laboratory tests on discontinuities, the strength and deformability properties of discontinuities are according to table (3).

Table 3. Properties of discontinuities

K_n (GPa/m)	K_s (GPa/m)	C (MPa)	ϕ (Deg.)	σ_t (MPa)
5.0	9.0	2	21	0.0

Rock Mass Properties

Reliable estimates of the strength and deformation characteristics of rock masses are required for almost any form of analysis used for the design of slopes, foundations and underground excavations. Laboratory tests provide a quantitative assessment of the properties of intact rock specimens. Laboratory tests do not represent the properties of the rock mass in situ, which are affected by joints, bedding planes, and other flaws that are not present in the laboratory specimens. In addition, mechanisms of behavior tested in the laboratory do not always represent the mechanisms of behavior experienced in situ. There are different methods to aware about rock mass properties such as rock mass classification systems (the cheapest method) including Terzaghi's method, RMR, RSR, Q, RMi, … to complex in situ tests (the most expensive method).
It was also recognized that the available rock mass classification systems are no longer adequate as a vehicle for relating the laboratory failure criterion to geological observations in the field, particularly for very weak rock masses. This resulted in the introduction of the Geological Strength Index (GSI) by Hoek, Wood and Shah (1992), Hoek (1994) and Hoek, Kaiser and Bawden (1995). In GSI classification system, is started from the properties of intact rock and then introduced factors to reduce these properties on the basis of the characteristics of joints in a rock mass to capture the properties of rock mass. GSI method is compatible for computer simulating of rock structures. For Azadi rock fill dam, the GSI value is equal to 42. Based on the GSI value, the rock mass properties of foundation are calculated and listed in table (4).

Table 4. Rock mass properties

E (GPa)	ν	σ_c (MPa)	σ_t (MPa)	C (MPa)	ϕ (Deg.)	m*	s**
2.6	0.25	0.58	0.026	0.94	21	0.5	0.0004

* and ** are material constant in Hoek- Brown criteria. [Hoek, 1994]

EMPERICAL METHODS

RSR System

The RSR classification is a quantitative method for description of rock mass quality, which is applied to predict the appropriate support system. The parameters A, B and C in RSR classification are equal to 15, 23 and 15, respectively. So the RSR value is equal to 53 and the suggested support system is as following [U.S. Army Corps of Engineers, 1984]:
(a) Shotcrete with 60 mm thickness.
(b) Rockbolt with 2.5 cm diameter and 1.5 m spacing.
The first alternative is more suitable based on geological conditions.

RMR System

According to the RMR_{1989} [Hoek et al, 1980], the rating of parameters for the rock mass was determined which is presented in Table (5).
Obviously, these parameters are too conservative for the rock mass. The average stand-up time of the galleries is about one week and they need support them. According to this classification, the support system is as following alternatives:
(a) Shotcrete with 50 mm thickness in roof and walls.
(b) Rock bolts with 2 m length and 1.5 m spacing executed with steel mesh in crown.
It seems that the first alternative is more suitable due to existence of underground water and small block composing the rock mass.

Table 5. Rating for RMR parameters

Parameter	Description & Value	Rating
Uni-axial Compressive Strength of Intact Rock	30 GPa	4
RQD	75	15
Spacing of Discontinuities	(0.2 – 0.6) m	10
Condition of Discontinuities	Rough, slightly weathered, Joint wall separation < 2 mm	23
Ground Water Condition	Flowing	0
Adjustment for Joint Orientation	Fair	-5
Rock Mass Rating	Rock mass cohesion = 0.2 - 0.3 MPa	RMR = 47
	Internal friction angle of rock mass = 25~35 Deg.	Class III Fair Rock Mass

Q System

The ratings of parameters of Q system are listed in Table (6). According to the ratings, Rock Mass Quality, Q, is equal to:

$$Q = \frac{RQD}{J_n} \frac{J_r}{J_a} \frac{J_w}{SRF} = 0.55 \qquad (1)$$

Table 6. Rating for Q parameters

Parameter	Description & Value	Rating
RQD	75	75
Joint Set Number (J_n)	2 joint system and one bedding	9
Joint Roughness Number (J_r)	Planar, rough and irregular	1.5
Joint Alteration Number (J_a)	Filling of calcite, organic material, marl, slow weathered to unweathered	3.0
Joint Water Reduction Factor (J_w)	Water pressure 0.1 to 0.2 MPa	0.66
Stress Reduction Factor (SRF)	Competent rock, depth < 50 meter	5.0

The ESR coefficient is considered about 1.6 for the grouting galleries, so the equivalent dimension will be equal to 2.2. According to Q system, the galleries need to support. The Q System suggests the support system as tensioned anchor bolts executed in 1 m spacing and 5 cm thickness of shotcrete. The support pressure is estimated about 1.1 kg/cm^2 according to Q system.

ANALYTICAL METHOD

Ground Reaction Curve (GRC)

The elasto-plastic method suggested by Ladany (1974) is used for predicting the behavior of circular tunnels in special ground conditions with K (the ratio of initial horizontal to vertical stress is equal to 1) [Ladany, 1974]. Assume a 3.5 m diameter circular tunnel at a depth of 50 m. The GRC for this tunnel is demonstrated in figure (2). This figure. Shows that if the tunnel were remained unsupported, both walls and floor of tunnel reach to a stable condition after deformations smaller than 1 cm. But the behavior of roof does not tend to be stabilized and may results in problems. So, it is needed to reinforce or support.

Support Design and Analysis

Hoek and Brown (1980) have published equations that can be used to calculate the capacity of mechanically anchored rock bolts; shotcrete or concrete linings or steel sets for a circular tunnel. No useful purpose would be served by reproducing these equations here but they have been used to estimate the values plotted in figure (2). This plot gives maximum support pressures and maximum elastic displacements for 5 cm shotcrete installed in the circular tunnel mentioned above. Note that, in analytical method, the support is assumed to act over the entire surface of the tunnel walls. In other words, the shotcrete and concrete linings are closed rings [Ladany, 1974].

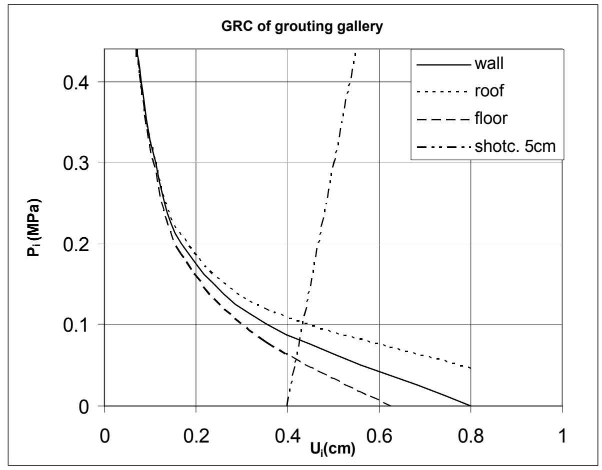

Fig.2. Ground Reaction Curve for the equivalent circular tunnel

NUMERICAL METHOD

Tunnel Stability Analysis and Support Requirements

For stability analysis of tunnel and designing the support system, two groups of models have been considered. One of these groups simulates dry condition and the other simulates groundwater with 18 meters depth. It will be shown that the existence of underground water has a great effect on stability and properties of support system of grouting galleries.

Dry Condition

The depth of galleries at the abutments is different (70 m in left abutment and 40 m in right abutment) as same as dip direction and dip of discontinuities. The galleries at each abutment have analyzed, separately, to have an understanding of the effect of the mentioned factors on induced stresses and stress concentrations and other parameters.

Left Abutment. The critical part of the gallery in left abutment has 70 m overburden. The DEM model and its discontinuities have shown in figure (3). First the ground reaches to a consolidated condition under its weight load. Then the gallery is excavated. The results shown that if the gallery remains unsupported, wedges and blocks will fall in roof (figure (4)) and will be dangerous. The deformation of walls is very small but the deformation of floor is about 1.8 mm and the roof is falling. Plastic zone is not created around the tunnel because of the good strength of rocks. However, it is necessary to support the gallery because of falling of the blocks of roof. For initial support, 5 cm shotcrete has been considered which is installed after 0.5 mm deformation of the roof. This support system is simulated by beam elements. After 1.8 mm deformation of roof, the support system and rock mass surrounding the gallery will reach to a stable condition. Figure (5) shows the axial loads in shotcrete elements, which are allowable.

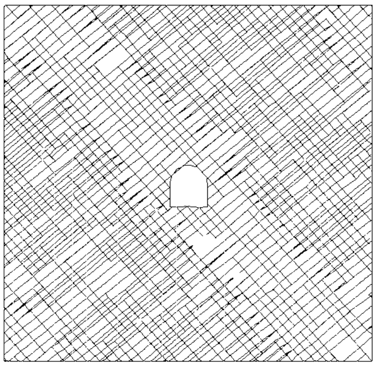

Fig. 3. DEM model of grouting gallery in left abutment

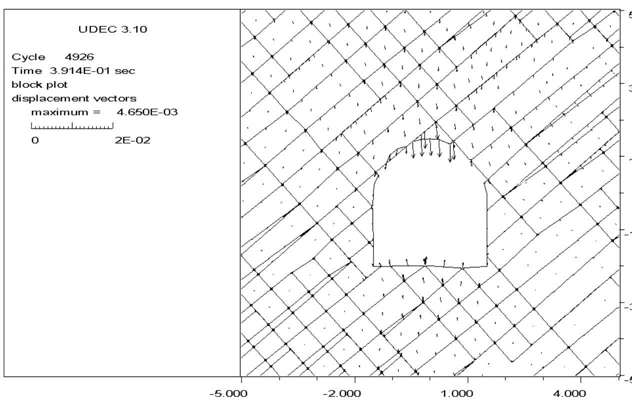

Fig. 4. Falling of blocks in the roof of gallery in left abutment

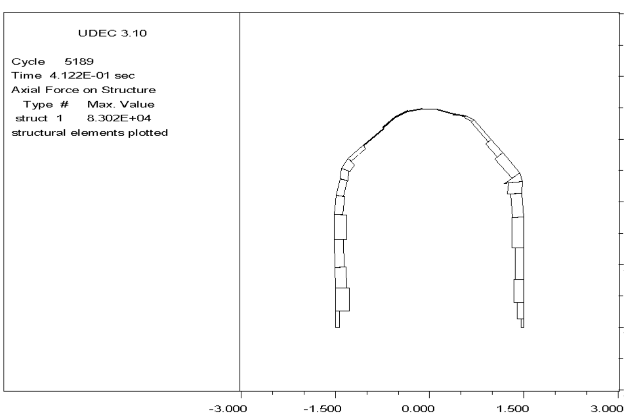

Fig. 5. Axial loads in shotcrete segments

The maximum vertical stress is about 4 MPa and is located in the walls of the gallery. The maximum horizontal stress is located at the junction of wall and roof and is about 0.8 MPa.

Right Abutment. The other end of gallery is located in 40 meters depth in right abutment. The orientation of discontinuities is different with respect to left abutment as shown in figure (6). The roof is falling after excavation and support system is required. The wall of gallery has a small displacement and the bottom will be stable after 1.2 millimeters deflection. The problem of instability in these galleries is the falling of blocks in roof. However plastic zone is not created around the gallery.

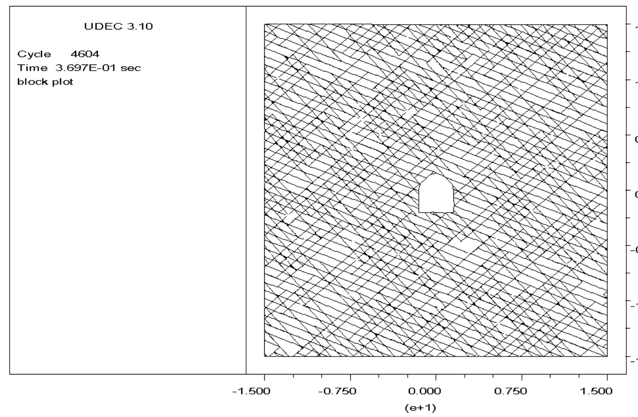

Fig. 6. DEM model of grouting gallery in right abutment

The maximum value of shear, horizontal and vertical stresses is about 600, 700 and 2.4 MPa, respectively. Shotcrete with 5 cm thickness is considered to prevent of wedges and blocks falling in the roof. The DEM model shows that the roof will reach to stability after 1.4 mm displacement by the installation of 5 cm shotcrete. The only reason for this result is the low stress in right abutment. The figure (7) is related to axial force in shotcrete and shows that the distribution of axial load in the support system is more uniform with respect to left abutment. It may be due to low stresses and more freedom of the blocks in right abutment. The stress calculation in shotcrete support indicates that 5 cm shotcrete is appropriate for supporting of the surfaces.

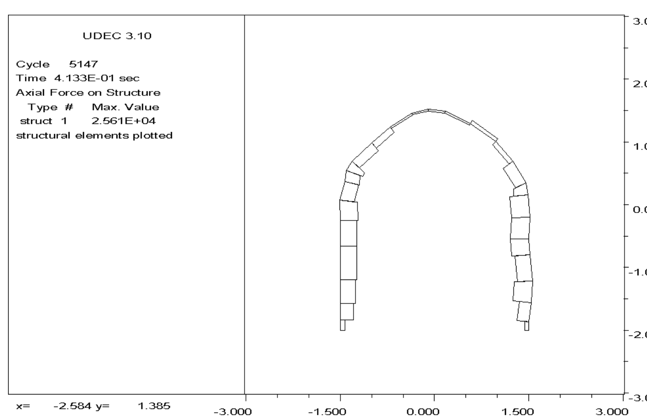

Fig. 7. Axial loads in shotcrete support of right abutment

Under the dam body. In this section, due to sensitivity of the structure, the gallery has analyzed with D.E. (Discrete Element) and F.D. (Finite Difference) methods. FLAC (Fast Lagrangian Analysis of Continua) program has been used for analysis which has been written on the basis of FDM. This part of gallery will be executed by cut and cover method and then the rockfill will place on it. So a great amount of stresses will apply on this section. In UDEC model, 4 joint set has considered to simulate the rock fragments on gallery body. The UDEC model has been shown in figure (8). So in both DEM and FDM model, first the gallery was excavated and lined with 70 cm concrete and then the pressure of dam body (70 m overburden of rockfill) was applied.

Figures (9) and (10) show the axial load in concrete lining in DEM and FDM models, respectively. Both models show that maximum stresses are in walls. In the DEM model, the maximum stress value is about 2.287E6 N and in FDM model is 2.712E6 N. The FDM model shows more uniformity in stress distribution because of continuity of media. Maximum moments in concrete lining are in floor (figures (11) and (12)), 5.571E5 N.m. in DEM and 5.202E5 in FDM model) and the reason of this is that the floor is straight and act like a beam. The shear forces in the DEM and FDM models are about 1.198E6 N and 1.455E6 N (figures (13) and (14)), respectively. All of the stresses are in allowable level, so the lining is adequate.

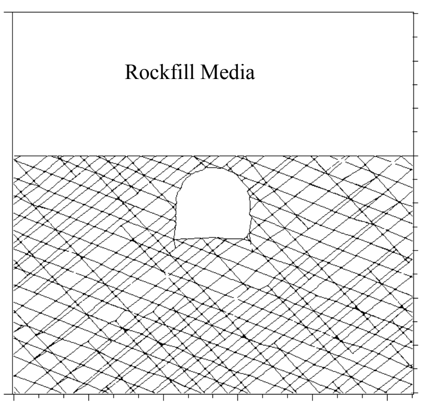

Fig. 8. DEM model of gallery under the dam body

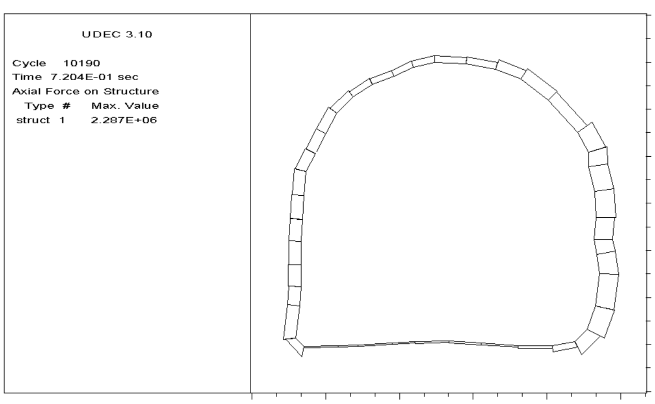

Fig. 9. Axial load in DEM model

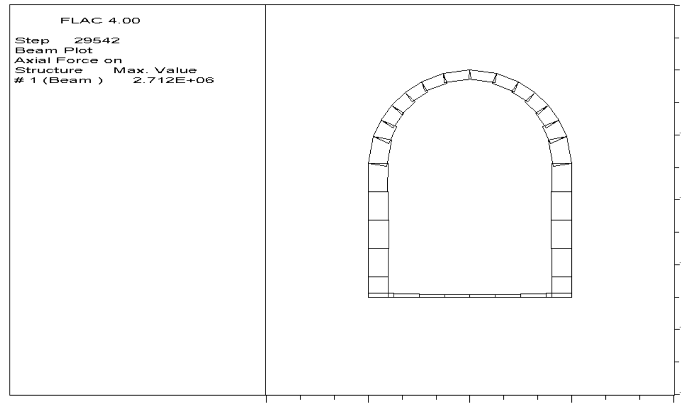

Fig. 10. Axial load in FDM model

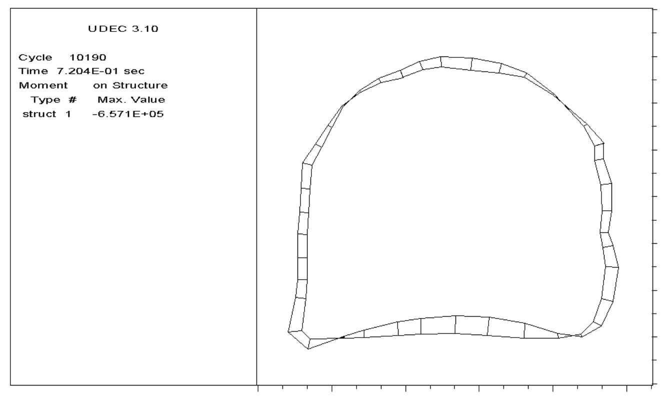

Fig. 11. Moment in DEM

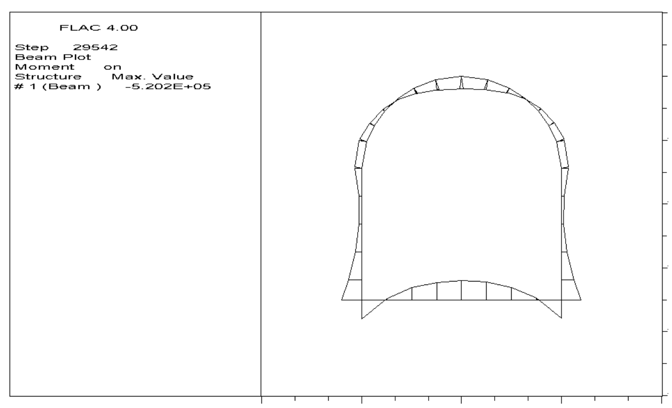

Fig. 12. Moment in FDM

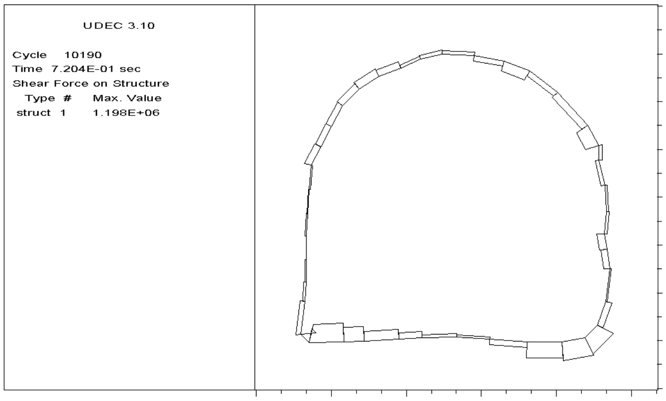

Fig. 13. Shear load in DEM

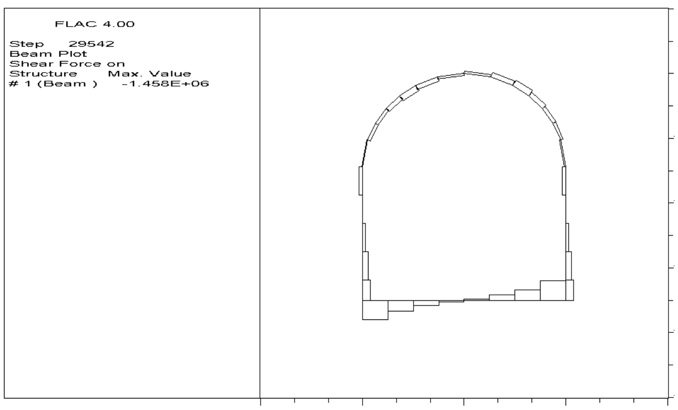

Fig. 14. Shear load in FDM

Underground water

Field investigation showed that groundwater exists and the galleries are at about 18 m under the groundwater surface. So this condition will affect the effective stresses and the properties of support system (thickness of shotcrete).

Left Abutment. In saturated condition, model showed that 5 cm shotcrete is not adequate, so 10 cm shotcrete is modeled. Obviously the existence of water reduces the effective stress and so more loads will apply to support system. The roof of the gallery deform to about 3.5 mm to reach to stability. Figure (15) shows the axial load on shotcrete segments. The amount of loads is more than of dry condition and the distribution of load have less uniformity with respect to dry condition (figure (15)). Because of special orientation of discontinuities, the axial loads in the right side of gallery are greater than left side.

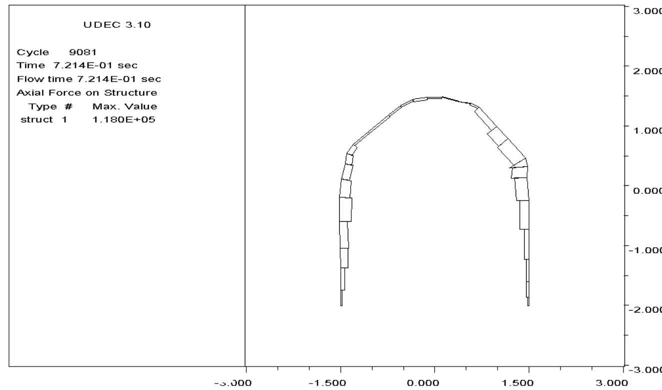

Fig. 15. Axial loads in 10 cm shotcrete support with groundwater

The distribution of flow rate shown in figure (16) indicates that the max flow rate in joints connected to gallery is about 0.000131 m^3/s (470 lit/h). The total flow rate come into the gallery will be about 0.00275 m^3/s (9900 lit/h). This huge amount of inflow reduces the effective stresses and so, induces the axial load applied to lining.

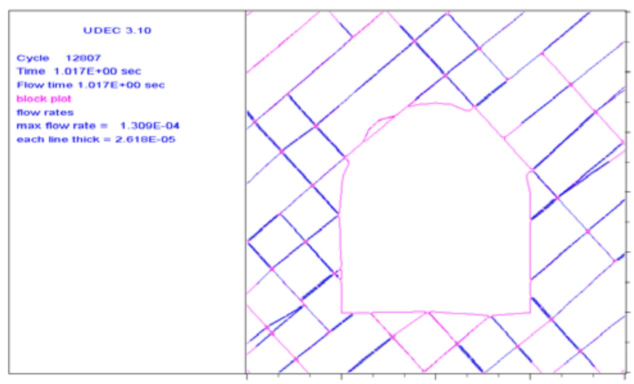

Fig. 16. Flow rates in joints around tunnel (Thickness of lines is proportional to the flow rates)

Right Abutment. In right abutment, 5 cm thick shotcrete is not adequate like the right abutment and 10 cm thick shotcrete applied in model. In this model, the displacement of roof after execution of shotcrete lining is about 1.7 mm. The maximum flow rate in the adjacent joints is about 0.000285 m^3/s (1062 lit/h). So the total inflow in 21 joints ending in tunnel is about 0.0060 m^3/s (21600 lit/h). The difference of inflow in galleries of left and right abutments is due to smaller overburden of right abutment, which causes lower closer and greater permeability of joints (note that the depth of underground water in the abutments is equal). Distribution of axial loads in shotcrete is shown in figure (17), which is more than the same in dry condition.

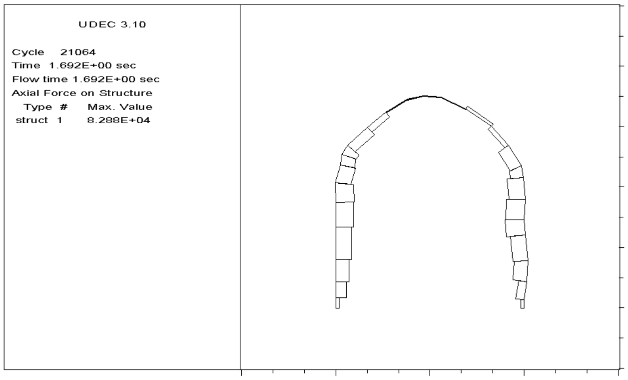

Fig. 17. Axial loads in 10 cm shotcrete segments with ground water

CONCLUSION

Based on the above analyses and studies, it can be concluded the following results:

1. According to RSR and RMR classifications, the appropriate support systems are as 6 and 5 cm of shotcrete, respectively.
2. Q classification shows that the appropriate support system is tensioned anchor bolts executed in 1 m spacing and 5 cm thickness of shotcrete
3. Numerical mechanical models (DEM) showed 5 cm shotcrete is adequate for galleries in abutment in dry condition.
4. Hydro-mechanical models (DEM and FDM) showed that in the existence of underground water, 10 cm shotcrete is efficient for support of galleries in abutments.
5. Existence of underground water reduces the effective stresses and so, more loads will be applied to support system. Therefore in this condition, the shotcrete thickness must be greater than dry condition.
6. The numerical DEM and FDM analysis of galleries under the dam body showed high stresses in the support system (70 cm reinforced concrete). It seems that in cut and cover tunneling method, higher loads may apply to support systems with respect to normal excavation of tunnels.

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