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3D Numerical Modeling of a Couple of Power Intake Shafts and Head Race Tunnels at Vicinity of a Rock Slope in Siah Bishe Pumped Storage Dam, North of Iran

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Abstract: The Siah Bishe pumped storage project is now under construction. The investigation results, the stability analysis for the power intake shafts and head race tunnels at vicinity of a rock slope are discussed in this study. Power intake shafts and head race tunnels in Siah Bishe dam have a complex geometry. In site investigation was found 3 joint sets in the rock mass. Therefore first of all the structure analyzed for finding underground wedges by Unwedge software. After that overall and local stability of the power intake shafts, the headrace tunnels and slope were analyzed by two conditions (static and pseudo static). Complex geometry of the project due to stability analysis was used 3D modeling by ABAQUS software. Results show that when a rock slope is closed to a shaft, 2D modeling is wrong. Especially, there were some other underground spaces, like tunnels. Results of numerical 3D modeling show that maximum displacement occurs in the roof of the tunnels between transition zone-conjunction of shaft to tunnel- and tunnel portal.

Key words: Shaft, tunnel, rock slope, stability analysis, 3D modeling

INTRODUCTION

Iran Water and Power Resources Development Company was entrusted 1983 with the design of Siah Bisheh pumped storage scheme. The waterways of the plant are now under construction (Fig. 1) (Moshanir Consultant Engineer, 2002). It is located in the northern part of the Alborz Mountain, at a distance of 80 km from the Caspian Sea.

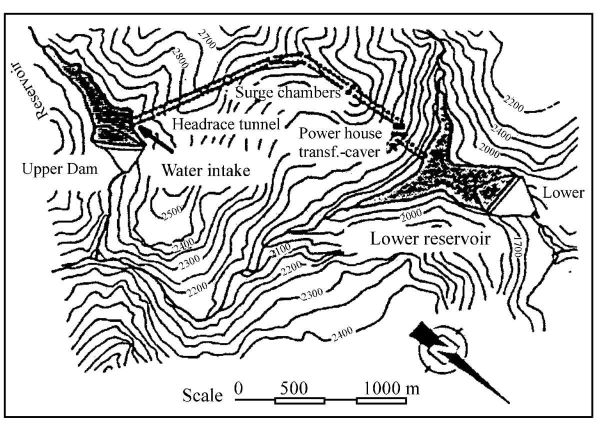

Fig. 1: Layout of Siah Bisheh pumped storage project

The creation of underground openings results in significant changes in stresses in the rock mass. In order to assess the stability of such openings and slops it is necessary that the stresses and deformations in the rock mass parameters, in the analysis, rather than using their approximate or average values (Hoek *et al.*, 1991). However, limitation in modeling is expected due to the difficulties associated with obtaining realistic input data. In the recent past, numerical simulation is being preferred over the modeling where in the rock mass properties are established either with the help of empirical methods, based on hundreds of case histories, or rational approaches, based on mainly laboratory or *in situ* testing (Giraud *et al.*, 1993). The evaluation of stress distribution, around the underground openings, is important for designing a proper support system. In addition to this, nonlinear constitutive behavior of the rock mass also is considered. In such a situation, finite element analysis is found to be quite efficient in handling such complexities (Bathe *et al.*, 1980). A critical review of the available literature, on stability analysis of underground openings, indicates that mostly analysis of underground openings are based on the rock mass parameters which are generally assumed or are representative of the rock mass. However, sufficient information on 3D finite element

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nonlinear analysis, of such shafts in conjunction with headrace tunnels in the rock slope is not available in the literature. As such, for an efficient design of underground openings realistic behavior of materials and an appropriate model for the analysis must be adopted. With this in view, relative suitability of 3D analysis has been investigated for power intake shafts and headrace tunnels in the rock slope of Sih Bisheh Pumped Storage dam by adopting *in situ* and laboratory rock mass properties.

The pumped storage plant is situated in layers of the Jurassic Shemshak formation. The Garmrudbar thrust fault separates the Jurassic Formation from the Triassic one (Alavi, 1996).

Height of power intake shafts is 50 m with 9.3 m diameters. Spacing of the shafts is 25 m (center to center). The shafts joint to two headrace tunnels with 7.5 m diameters (Farab and Tablieh Geotechnical Company, 2003). These underground spaces are located at vicinity of a rock slope in the Shemshak Formation. The formation consists of shaly, slightly sandy siltstone, sandstone and thin layered limestone.

The whole formation is folded and forms the southern flank of an anticline. The folding process caused a shearing of incompetent layers such as thin layers of beds but also between siltstones with different content of fines like clay or fine sand.

This study provides a brief explanation on geological conditions of the power intake portal area based on the latest observations and findings from the recent excavations for the access road and also presents the results of stability analysis of the inlet portal, intake gate shafts and the transition zone nearby the shafts in the headrace tunnels. This study also aims to determine rock support system required for stabilizing the excavations at inlet portal, intake gate shafts and the headrace tunnels nearby the shafts and slope.

GEOLOGY OF THE SITE

Geological condition of power intake area is shown in the geological map drawing in Fig. 2 (Kayson Company, 2005).

The power intake portal area is located in the Shemshak formation composed of mainly siltstone and sandstone with coal and or shale intercalations. Structurally, the power intake area is situated on south flank of a syncline (Fig. 2). The siltstone beds are slightly to moderately weathered, laminated, grey to dark grey, soft to moderately hard. Sandstone beds are slightly weathered, medium to coarse grained, grey and hard. The thickness of beds varies between 0.2-2.5 m. Generally there are three joint sets which are limited to the bedding plane in most cases. There is Fe-oxide on all joint surfaces.

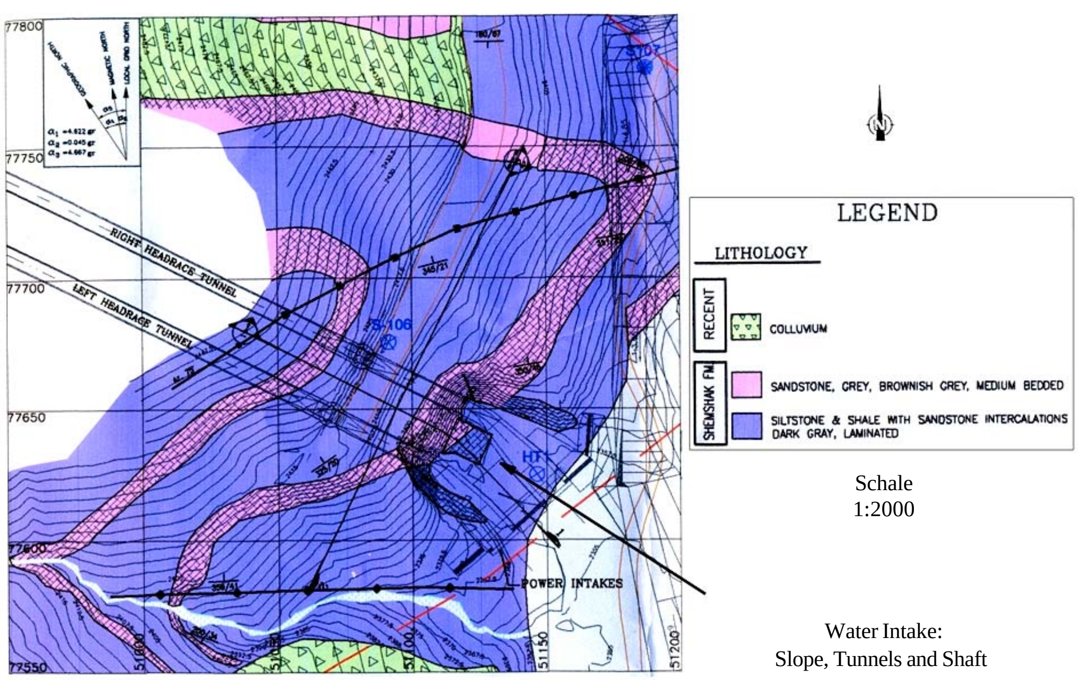

Fig. 2: Geological map of site of power intake shafts (Kayson Company, 2005)

Table 1: Orientations and Characteristics of major discontinuities in power intake area

Disc. No.	Dip. D/Dip (°)	Spacing (cm)	Persistence (m)	Aperture (mm)	Filling (material)	Roughness
B	345-355/25-35	20-140 (Ave.50)	> 30	Rock contact	-	Undulating-rating
J1	185-195/60-70	75-150	0.2-2.5	< 2	Fe-Oxide	Planar-rough
J2	085-095/75-80	200-300	0.2-2.5	< 5	Fe-Oxide	Planar-rough
J3	270-280/75-85	40-80	0.2-2.5	< 5	Fe-Oxide	Planar-rough

The bedding and major joints orientations and their characteristics have obtained from field surveying and are shown in Table 1.

GEOMECHANICAL PARAMETERS OF THE ROCK MASS

Orientations and characteristics of the discontinuities have been defined based on the field measurements. Shear strength parameters of the discontinuities are obtained based on the conditions of the joint surfaces, type of rock materials and using the earlier experience in this field (Hassani *et al*., 2008).

The shear strength parameters for the discontinuities have been selected based on earlier experience, the existing lithology (intercalations of coal, shale and sandstone) and the disturbances. A cohesion value of zero has been assumed due to the potential for filling material in the discontinuities being washed out by drainage water in the long term and friction angle of bedding planes and joint sets are measured to and respectively, as shown in Table 2.

Geomechanical classification of various types of rock masses in Shemshak formation have been performed using GSI classification system (Hoek, 2001). The required data obtained from field observations and joint survey data collected from direct measurements on rock exposures and laboratory tests. Estimate of rock mass parameters has been done using RocData. This information is shown in Table 3. As shown in this table, only one rock mass type has been identified in the power intake and gate shafts area, from the lithology and degree of weathering point of view, which is slightly to moderately weathered, moderately disturbed siltstone, with intercalation of coal, shale and sandstone (Flysch Type C, GSI = 38).

Dilation angle for rock mass is calculated by below formula (Arshadnejad *et al*., 2006):

$$\begin{cases} \delta = -0.28\phi\left[\dfrac{GSI}{100}\right]^2 + 0.85\phi\left[\dfrac{GSI}{100}\right] - 0.23\phi \\ 30 \langle GSI \langle 75 \\ \delta = 0^\circ \qquad GSI \le 30 \end{cases} \tag{1}$$

Table 2: Orientations and shear strength parameters of discontinuities

Formation	Type of disc	Orientation Dip Dir./Dip	Cohesion (kPa)	Friction angle (Degree)
Shemshak F.	B	350/30	0	23
	J1	190/65		
	J2	90/77	0	27
	J3	275/80		

Table 3: Rock mass characteristics and parameters at power intake portal and gate shafts (Kayson company, 2005)

Characteristics and parameters	Observation
Formation	Shemshak formation (flysch type C)
Lithology	Inter layers of Sandstone and Siltstone in similar amount (50% - 50%)
Degree of weathering	Slightly to moderately
Hoek brown failure criteria for intact rock m_i	10
Uniaxial compressive strength of intact rock σ_c	50
Geological strength index (GSI)	38
Disturbance factor	0.8
Slope height (m)	50 and 35
Unit weight (kN m^{-3})	26
Deformation modulus (MPa)	2100
Poisson ratio	0.27
Dilation angle (Degree)	2.155
Shear strength parameters of rock mass	
C (MPa)	0.28
Φ(Degree)	41.00
Shear strength parameters of rock-concrete contact	
Peak	
C_p (MPa)	0.28
Φ_p (Degree)	41.00
Post failure	
C_{pf} (MPa)	0.21
Φ_{pf} (Degree)	26.00

WEDGE ANALYSIS

Discontinuity analysis in underground spaces is very important. Because of it occurs individually displacement around the underground space (Goodman, 1989). Therefore it must be separately analysis, before numerical modeling (Brady and Brown, 1993).

For this analysis was used Unwedge 3.0 software. All of the wedges are stable. Bolt spacing in the shafts is 22 m with 5 m length and it is 1.5 1.5 m with 4 m length in the tunnels. Shotcrete thickness in the shafts and the tunnels are 10 and 15 cm, respectively. Table 4 and 5 show that the results of underground wedge on the shafts and

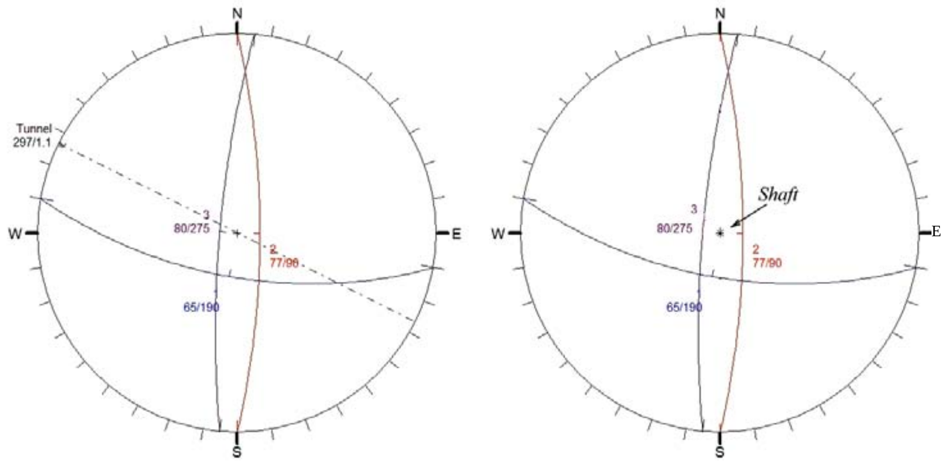

Fig. 3: Stereonets of joint sets (Dip and Dip Direction) in the shafts and tunnels

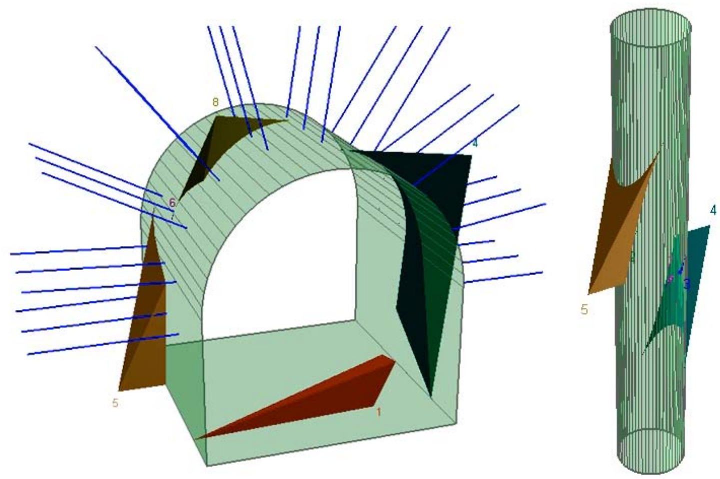

Fig. 4: Wedges in the shafts and tunnels

Table 4: Results of underground wedge analysis in the water intake shafts with Unwedge 3.0 software

	Wedge No.	
Shafts wedges	4	5
Weight (tonnes)	268.75	263.48
Safety factor	5.61	57.33

Table 5: Results of underground wedge analysis in the water intake tunnels with Unwedge 3.0 software

	Wedge No.			
Tunnels wedges	1	4	5	8
Weight (tonnes)	3.06	16.72	4.79	0.92
Safety factor	Stable	31.76	356.24	197.91

tunnels after installation of the supports. Figure 3 shows the Stereonets of joint sets in the shafts and tunnels and Fig 4 shows the wedges in the shafts and tunnels.

NUMERICAL MODELING OF THE POWER INTAKE SHAFTS AND HEAD RACE TUNNELS

For the power intake portal, two sets of analysis were carried out. Firstly, the overall stability of the rock slope with out the tunnels and the shafts was analyzed in 2 dimensions. When there is a transition zone in the rock mass 3D modeling is the best technique for numerical modeling (Roth, 2001; Brady and Brown, 1993; Goodman, 1989). After modeling, results show that this method (2D modeling) is wrong, when some underground spaces are nearby to the slope. Because of some parts of model show infinite displacement that is not true. Modeling of the shafts and tunnels in separately, obtained the same results. For these modelings were used Phase 2 and slide softwares.

After that was used ABAQUS software for 3D modeling of all structures with together. In the 3D model were used 21,078 triangle elements. This type of element is suitable for complex geometry (Goodman, 1976), like topography in mountain. Type loading selected gravitational model and whole side of the model fixed by 3 degree of displacements and 3 degree of rotations. Scale of the model is 1:1.

Discontinuities in the rock masses normally reduce the strength properties. These characteristics can be model by a continuous media, based on reduced strength of geomechanical parameters (Hoek, 2000). Therefore the method was used for 3D numerical modeling by ABAQUS software.

Time of the running was 1843 min and computer's CPU was Pentium 4, 3.0 GHz with 384 MB. Figure 5 shows the numerical model with all of structures that was made by *ABAQUS* soft ware.

DISCUSSION

For the stability analysis were made 6 models with different loadings. In the modeling was used reaction of some supports system (Hoek, 1999), like shotcrete and rock bolt. Anchor bolts installed by pre-tension force with 70% of bearing capacity of the bolts (Hoek, 2000).

Overall stability analysis of both the power intake slope and the gate shafts were performed in two conditions, static and pseudo static conditions. In this case a seismic coefficient 0.35 g was considered for horizontal PGA. In model number 5 and 6, was considered dynamic loading (pseudo static). No water pressure was considered in none of the stability analysis of the gate shafts, because they are above the underground water table. Figure 6 and 7 show that the structures after modeling. Power intake slope is located between top of the tunnels and the shafts. For stability analysis of this location was used analytical methods (Ortigao and Sayao, 2004; Wyllie Mah, 2004) and numerical modeling, together. Results of modeling shows a minimum safety factor for unsupported slope and supported slope with static and pseudo static conditions was taken as 0.94, 1.57 and 1.34, respectively.

Results of overall stability analysis (minimum safety factor) of power intake shafts, head race tunnels and rock slope are shown in Table 6. As shown in Table 6, the results are satisfactory and in general the power intake portal is stable in both static and pseudo static conditions. Figure 8 shows that displacements around the tunnels and the shafts. Maximum displacement occurs between the two shafts and the value is 8.93 mm.

Figure 9 shows distribution of displacements on the structures into the rock mass. Results of modeling and stability analysis of all the structures for design of supports are summarized in Table 7. Maximum displacement occurs between transition zone and tunnel portals. It is located on the roof of right tunnel and its value is 7.52 mm. The displacement is small and acceptable (Sakurai, 1997). Figure 10 shows this location and its displacement.

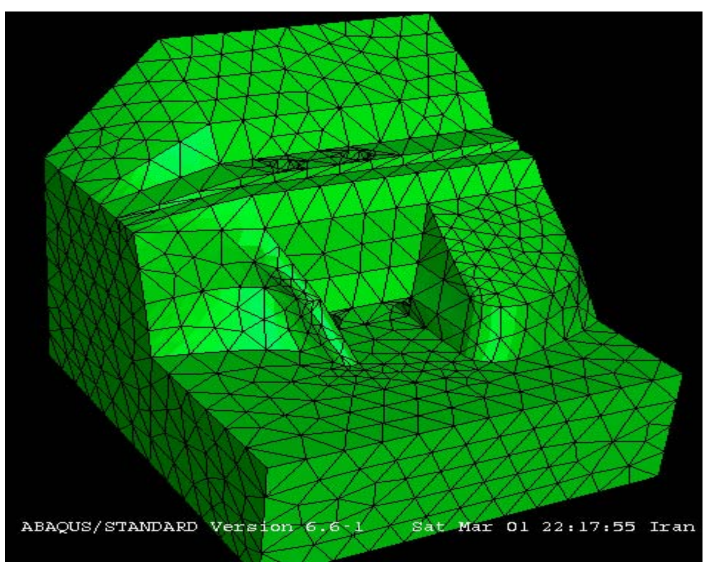

Fig. 5: 3D numerical model of power intake shafts and head race tunnels at vicinity of a rock slope (ABAQUS)

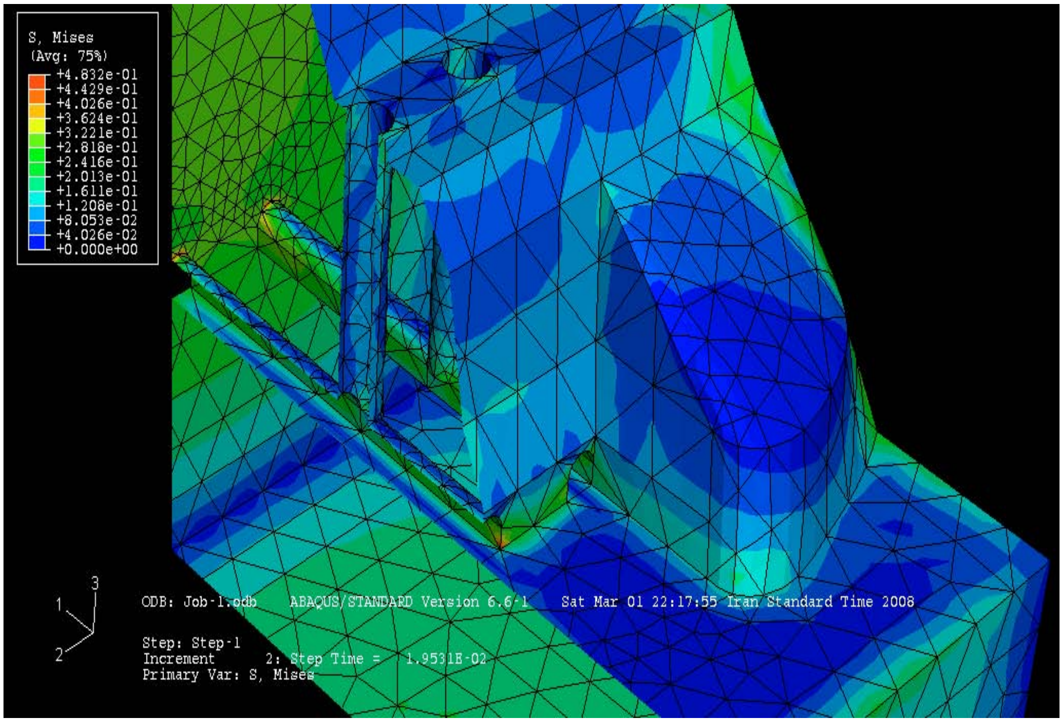

Fig. 6: Stress distribution in vertical section of the shafts and tunnels at vicinity of rock slope

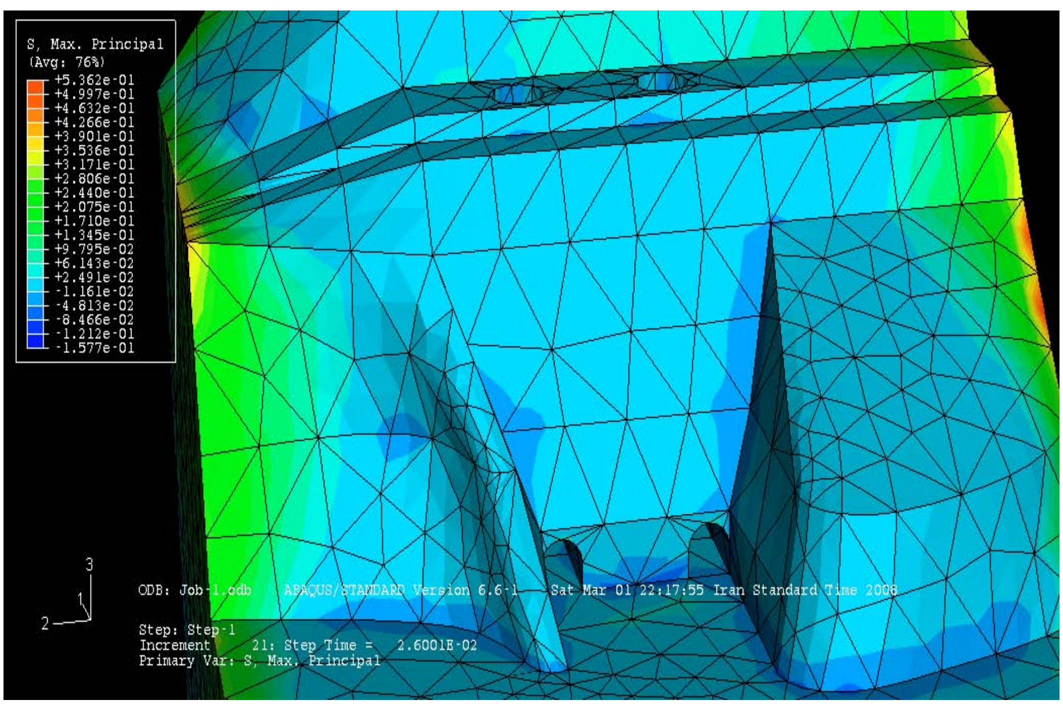

Fig. 7: Stress distribution (sigma 1) in the rock mass with shafts and tunnels at vicinity of rock slope

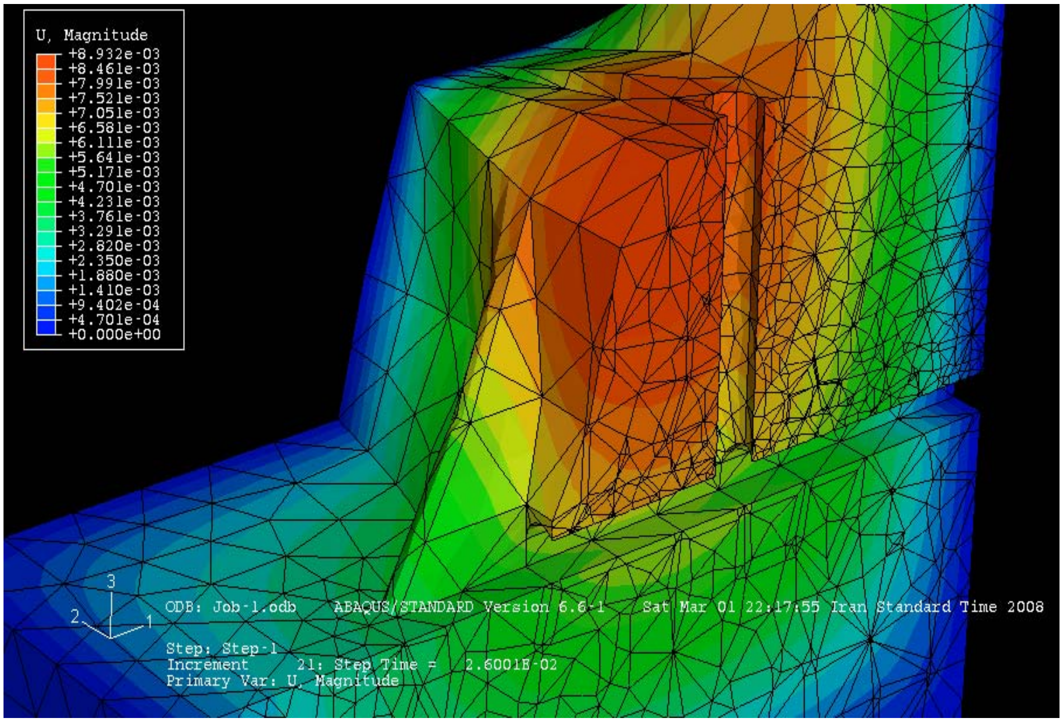

Fig. 8: Displacements around the tunnels and the shafts

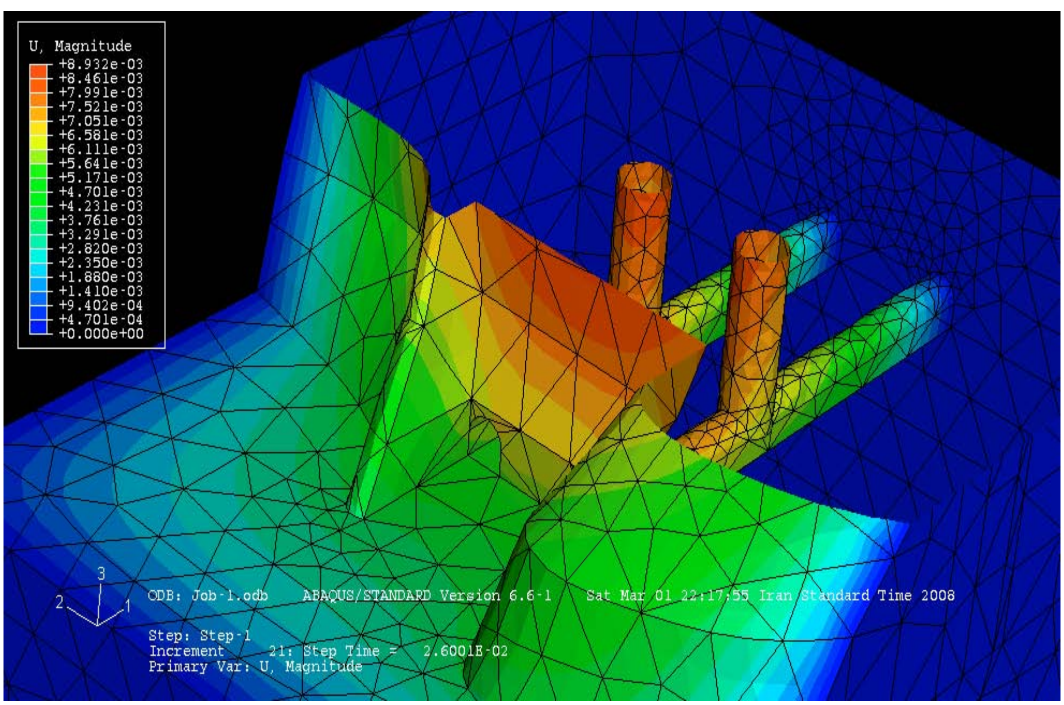

Fig. 9: Displacements in the walls of the shafts, tunnels and rock slope in meter

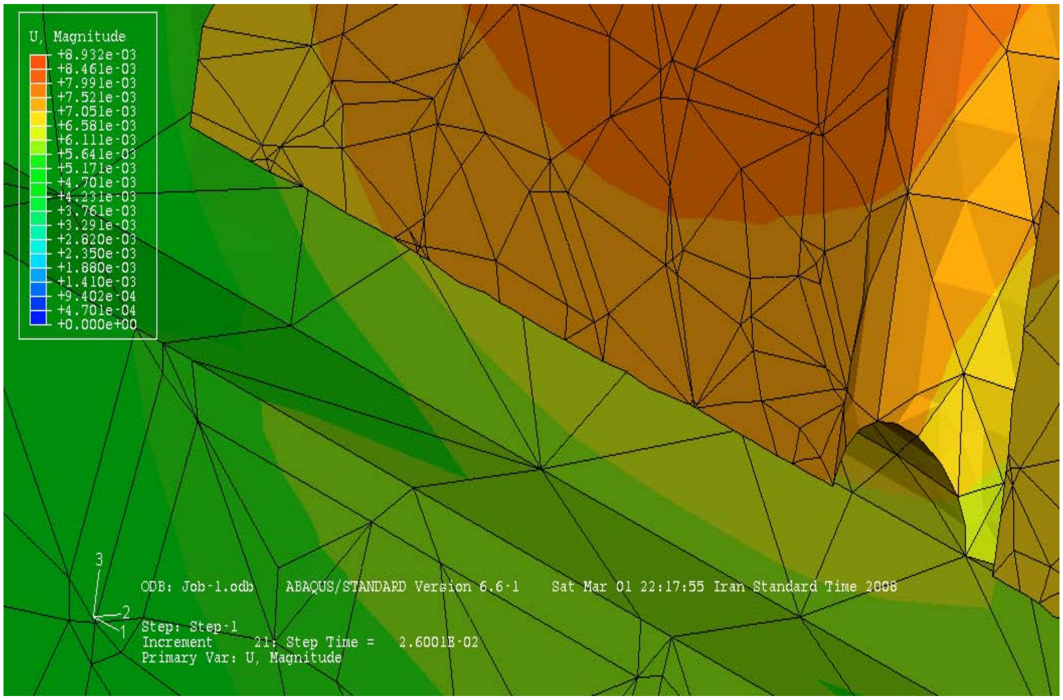

Fig. 10: Displacements on the top of the roof of right tunnel in meter

Table 6: Minimum safety factors in whole of 3D model in static and pseudo static conditions

	Safety factor		
Location	Unsupported	Static condition	Pseudo static (0.35 g)
Slope	0.94	1.57	1.34
Left shaft	1.03	2.81	2.16
Right shaft	1.09	2.88	2.21
Left tunnel	0.71	2.23	1.64
Right tunnel	0.76	2.27	1.68

Table 7: Characteristics of the supports in shafts, tunnels and rock slope

	Rock bolt	Shotcrete
Shafts	Fully grouted rock bolt	Reinforced shotcrete
	Length: 5000 mm, Diameter:	Thickness: 100 (mm)
	25 (mm) Spacing: 2000×2000 (mm)	6@150×150 (mm)
Tunnels	Tension rock anchor	Reinforced shotcrete
	Length: 4000 mm, Diameter:	Thickness: 150 (mm)
	32 (mm) Spacing: 1500×1500 (mm)	6@150×150 (mm)
Slope	Fully grouted rock bolt	Reinforced shotcrete
	Length: 6000 mm, Diameter:	Thickness: 200 (mm)
	32 (mm) Spacing: 2000×2000 (mm)	6@150×150 (mm)

CONCLUSION

Power intake shafts and head race tunnels in Siah Bishe dam have a complex geometry. In site investigation was found 3 joint sets in the rock mass. Consequently, first of all the structure analyzed for finding underground wedges. After that overall and local stability of the power intake shafts, the headrace tunnels and slope were analyzed by two conditions (static and pseudo static). The results show that in the complex geometry, 2D modeling is wrong. Especially, when there is a transverse shaft to a tunnel. In this situation 3D modeling is necessary. The 3D modeling show that maximum displacement occurs on the roof of tunnels between transition zone and tunnel portal (about 7 mm). Consequently, pattern of bolting in the location and around of transition zone, will be closer than far away points in the tunnels and the shafts. This reduction of bolt spacing is about 30% (11 m). Moreover thickness of shotcrete increases about 30% (200 mm), in the tunnels. Pre-tensioning force for rock anchor in the tunnels is about 70% of bearing capacity (13 tons). These Patterns of rock bolts and shotcrete supply the stability of wedges in the shafts, tunnels and slope in static and pseudo static conditions.

REFERENCES

Alavi, M., 1996. Tectono stratigraphic synthesis and structural style of the alborz mountain system in Iran. J. Geodynamic, 21: 1-33.

Arshadnejad, Sh., H. Poshtvan and H. Parsaee, 2006. Determination of *Optimum pillar* size by empirical and numerical methods based on ground reaction curve-case study, Soltan abad's underground salt mine. 7th Tunneling Conference in Iran, pp: 849-865.

Bathe, K.J., M.D. Snyder, A.P. Cimento and W.D. Rolph, 1980. On some current procedures and difficulties in finite element analysis of elastic plastic response. Comput. Struct., 12: 607-624.

Brady, B.H.G. and E.T. Brown, 1993. Rock Mechanics for Underground Mining. 1st Edn., Chapman and Hall, London, pp: 571.

Farab and Tablieh Geotechnical Company, 2003. The investigation and Geotechnical characteristics of Headrace Tunnel at Siah Bishe Dam. North Iran, Project Report No. 42.

Giraud, A., J.M. Pricard and G. Rousset, 1993. Time dependent behaviour of tunnels excavated in porous rock mass. Int. J. Rock Mech. Min. Geomech. Abstr., 30: 1453-1459.

Goodman, R., 1976. Methods of Geological Engineering in Discontinuous Rocks. 1st Edn., Methods of Geological Engineering in Discontinuous Rocks. St. Paul, Minnesota, pp: 472.

Goodman, R., 1989. Introduction to Rock Mechanics. 1st Edn., John Wiley and Sons, New York, pp: 644.

Hassani, H., Sh. Arshadnejad and H.R. Hajhassani, 2008. Optimum static and dynamic design of displacement chamber of headrace tunnel with bedding parallel shear zones in Siah Bishe Dam, Iran. J. Tunnel. Underground Space Technol., 23: 531-538.

Hoek, E., M.W. Grabinsky and M.S. Diederichs, 1991. Numerical modelling for underground excavation design. Trans Inst. Min. Metall. Sect., A100: 22-30.

Hoek, E., 1999. Support for very weak rock associated with faults and shear zones. Proceeding of the International Symposium on Rock Support and Reinforcement Practice in Mining, Kalgoorlie, Australia, March 14-19, 1999.

Hoek, E., 2000. Practical Rock Engineering. 1st Edn., Dehkhoda Publisher, Iran, pp: 512.

Hoek, E., 2001. Big tunnels in bad rock. ASCE. J. Geotech. Geoenviron. Eng., 127: 726-740.

Kayson Company, 2005. Siah Bishe pumped-storage project, upper dam power intake and gate shafts. Project Report No. 300.

Moshanir Consultant Engineer, 2002. The siah bishe pumped storage project in Iran. Project Report No. 39.

Ortigao, J. and A. Sayao, 2004. Handbook of Slope Stabilization. 1st Edn., Springer, New York, pp: 478.

Roth, P., 2001. Comparison between analytical and numerical stability analyses using FLAC for shallow tunnels in soft rock. FLAC and Numerical Modeling in Geomechanics, A.A. Balkema, pp: 399-402.

Sakurai, S., 1997. Lessons Learned from field measurements in tunneling. Tunnel. Underground Space Technol., 12: 453-460.

Wyllie, D. and C. Mah, 2004. Rock Slope Engineering. 1st Edn., Spon Press, New York, London, pp: 431.