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Design of water diversion tunnels in the Moshampa dam project

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ABSTRACT
Moshampa dam is an embankment rockfill dam with clay core which has a height of 124 meters above bedrock with a catchment area of about 25000 km^2. The objective of Moshampa dam is to provide drinking water, agricultural use, and hydro-electric energy of 110 MW capacity from Qezel Ozan River. For diversion of the river during construction of the dam, two tunnels were designed in right abutment. Both tunnels have 8.5 meter internal diameters and about 440 meters in length with distance of 30 meters axis to axis; an access Tunnel will connect them together in the Gate Control house. After dam construction, left tunnel will be used as Bottom Outlet Tunnel. The Bottom Outlet Tunnel from intake shaft to Gate Control House will act as a pressure tunnel; the tunnel will be free flow after Gate Control House. The Right diversion tunnel will be plugged before Access Tunnel and will be used for access to Gate Control House from downstream. Tunnels have 185 meters maximum overburden and will be constructed perpendicular to rock mass layers. The rock mass includes limestone, tuff and mudstone. This article describes loading conditions, analysis and design phases which include finite element analysis of lining and rock mass interaction, internal and external water loads effects on lining and tunnel lining optimization and problems in design and construction of this project.

RÉSUMÉ
Le barrage de Moshampa est un barrage en remblai avec noyau d'argile de 124 mètres de hauteur à partir du substratum rocheux avec bassin d'apport d'environ 25000 km^2. L'objectif du barrage de Moshampa est de fournir l'eau potable, l'usage agricole, et l'énergie hydro-électrique de 110 MW de capacité de la rivière Qezel Ozan. Pour le dérivation de la rivière pendant la construction du barrage, deux tunnels ont été conçus dans le roc en rive droite. Les deux tunnels ont 8,5 mètres de diamètre interne et environ 440 mètres de longueur avec une distance de 30 mètres axe à axe. Un tunnel de connexion les relie ensemble dans la chambre des masses. Après la construction du barrage, tunnel de gauche sera utilisé comme tunnel de sortie en bas. La pression d'eau est dans le tunnel de sortie en bas d'admission du puits à la chambre de la porte, mais il y a le libre écoulement après la chambre de la porte. Le tunnel de dérivation droite sera bouché avant de tunnel branché et sera utilisé pour l'accès à la chambre de la porte de l'aval. Les tunnels ont 185 mètres au maximum de couverture; ils sont construits perpendiculairement à la masse rocheuse. La masse rocheuse comprend calcaire, tuf et mudstone. Cet article décrit les conditions de chargement, phases d'analyse et de conception qui comprennent l'analyse par méthode des éléments finis du revêtement et le son interaction avec la masse rocheuse, des effets des charges d'eau internes et externes sur le revêtement et l'optimisation de revêtement en béton du tunnel et des problèmes dans la conception et la construction de ce projet.

1 INTRODUCTION

The Diversion Tunnels Project is part of the Moshampa embankment dam project. The purpose of the Moshampa dam project is to provide irrigation and drinking water. Hydro power generation with a power capacity of 110 MW is another expected benefit. The project is located in Zanjan Province at Km. 116 E of Zanjan city and Km. 5 W of Moshampa village, across the Qezel Ozan River.

The Moshampa reservoir dam is a rock-fill embankment type with a water-impermeable central clay core. The dam is 100 m. high from streambed (124.5 m. from bedrock) and 400 m. long. Its crest is 10 m wide and at an elevation of 1273 m.a.s.l. The dam collects water from a catchment area that covers about 25,000 km^2. The general layout of the Moshampa embankment dam, structures and diversion tunnels alignment are shown in Figure 1 and water levels are listed in Table 1.

Table 1. Elevations of different situations.

Situation	Elevation (m.a.s.l.)
Normal Water Level (N.W.L.)	1267
Design Water Level (D.W.L.)	1270
Maximum Water Level (Max.W.L.)	1272.9
Minimum Water Level (Min.W.L.)	1253
Intake Water Level (Plug condition)	1226
Diversion Water Level	1215
Right Diversion Tunnel Intake	1176
Left Diversion Tunnel Intake	1184
Power Intake Level	1236
Cofferdam Level	1216

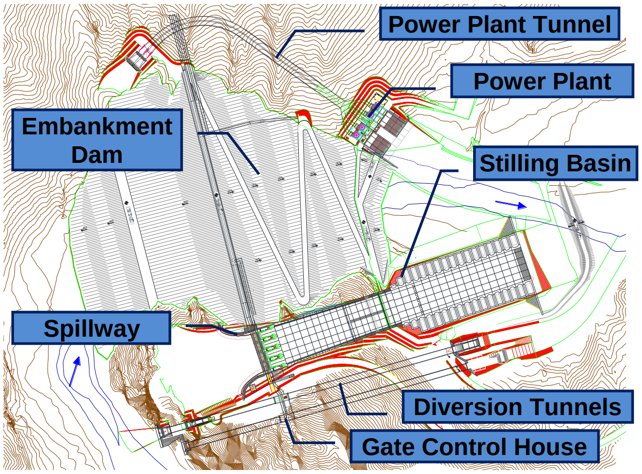

Figure 1. General layout of the Moshampa dam

2 Geology

Form the Iranian geological viewpoint, the area under investigation is located within the Sanandaj-Sirjan zone and central Iranian mountains. The Qezel Ozan River originates from the southern Sanandaj Mountains and flows northward. Different constituent formations of the basin include all types of pre-Cambrian-to-Quaternary period sedimentary, igneous, and metamorphic rocks. However, the area is mainly composed of erodible tertiary and quaternary formations. The Lugeon permeability test results indicate mainly medium to high degree of permeability (Lugeon. 1933)

3 Diversion tunnels

To divert the Qezel Ozan river water during the construction of the dam, two tunnels have been designed and topographically located on the right abutment of the dam site. The right and left tunnels will be 434.2 and 441.7 m long respectively, with an internal diameter of 8.5 m and with spacing of 30 meters axis to axis. An Access Tunnel will connect them together in Gate control house. Maximum overburden for the right and left tunnels are 160 and 185 m, respectively. Longitudinal profiles of the right and left diversion tunnels are shown in Figures 2 and 3 respectively.

After dam construction, the left tunnel will be used as Bottom Outlet Tunnel. The Bottom Outlet Tunnel from intake shaft to Gate Control House will act as pressure tunnel but will be free flow after Gate Control House. The Right diversion tunnel will be plugged before Access tunnel and will be used for access to Gate Control House from downstream

The bed rock of the tunnels consists of limestone, tuff, and mudstone of Oligocene to Miocene age from upstream to downstream, respectively. The upper red formation mudstones, constitutes the outlet trench and a very small portion of the tunnel outlets. The Qom formation layers have a 70 degree inclination toward the dip direction of 260 degrees. In other word, the tunnels will be excavated nearly perpendicular to the layers, providing the most suitable condition in terms of stability.

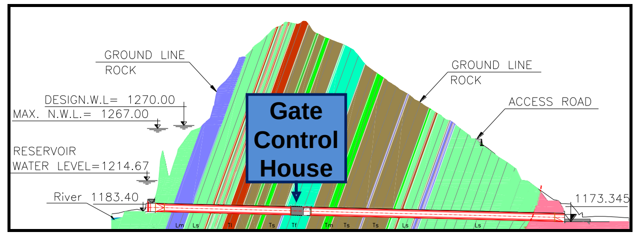

Figure 2. Longitudinal Profile of the right diversion tunnel

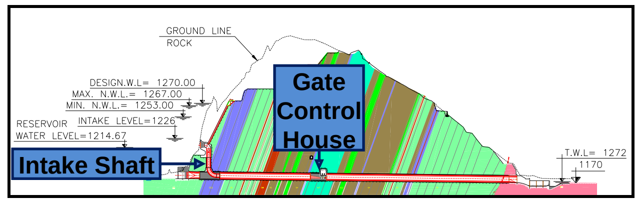

Figure 3. Longitudinal Profile of the left diversion tunnel

Considering the hydraulic gradient on the right abutment of the dam, the tunnel will be excavated under the groundwater level, and a maximum water column of 4.8-9.4 m will exist above the tunnel roof. Considering the relatively high permeability of bed rock mass, groundwater is expected to enter into the tunnel. For a natural drainage of water, the tunnel excavation is recommended to begin from the outlet side. Otherwise, certain preparations must be made for dewatering purposes.

To estimate the required initial support method, Q classification has been used (Barton et al. 1974, 1988). Considering the average Q obtained from the relation between NGI and VP for rock units (Barton 2006), values related to limestone (Inlet parts), tuff, limestone (Outlet part) and mudstone are 0.81, 0.17, 2.5 and 0.3, respectively. The initial tunnel support systems based on rock mass types are provided in Table 2.

Table 2. Proposed primary supports.

Rock Mass	Tunnel section	Primary support
Limestone and tuff	Inlet middle parts	Rock bolt (1.7*1.7 m pattern, 3 m length) 70 mm Shotcrete a layer of 3 kg/m^2 wire mesh
Limestone	Tunnel Outlet	Rock bolts (2*2 m pattern, 3 m length) 45 mm Shotcrete.
Mudstone	Tunnel Outlet	Steel frames with intervals of 2 m Fiber Shotcrete.

3.1 Design

3.1.1 Design loads

The design loads include weight of materials or Dead Load (D.L.), rock pressure, grouting pressure, and hydrostatic pressure. According to the hydrostatic pressure and in order to improve the rock surrounding the tunnel, contact and consolidation grouting for strengthening the diversion tunnels were performed at 2 and 10 bar, respectively. It is necessary to mention that, contact grouting is performed along the total route of diversion tunnels, but consolidation grouting is only done from inlets to Gate Control House. Due to the high pressure of consolidation grouting and studies done to avoid increasing tension and stress concentration in the lining; consolidation grouting is done on the total cross section of the tunnel, simultaneously.

For the design of the final lining, tunnels were analyzed under the internal and external water pressures and designed based on the ultimate strength method of ACI regulations and guidelines of US Army Corps of Engineers.

3.1.2 Loading Condition

Multiple load cases were used in the stability evaluation and design of concrete lining of tunnels. These load cases are described below.

3.1.2.1 Construction Condition

1. D.L. + Rock Pressure
2. D.L. + Contact grouting pressure
3. D.L. + Consolidation grouting pressure (between inlet and gate control house)

3.1.2.2 Plug Condition

4. D.L. + Rock Pressure + External water pressure

3.1.2.3 Operating Condition

5. D.L. + Internal water pressure resulting from diversion water level
6. D.L. + Internal water pressure resulting from reservoir water level in normal and extraordinary conditions (used just for left tunnel between shaft and gate control house)
7. D.L. + Rock pressure + External water pressure (Control gate closed from top)
8. D.L. + External water pressure (Control Gate closed from top)

3.1.3 Analysis under External Water pressure

Two-dimensional finite elements model under plane strain condition has been used for analysis purposes. The rock-structure interaction was modeled by using contact elements. (Figure 4)

Specified compressive strength of concrete lining of the diversion tunnels are listed in Table 3.

Table 3. Specified compressive strength of concrete for concrete lining of diversion tunnels.

Tunnel	Compressive strength of concrete (MPa)
Right diversion tunnel	25
Left diversion tunnel (From Intake to Gate control house)	30
Left diversion tunnel (From Gate control house to outlet)	35

External water pressures on lining were estimated based on ground water level and loading conditions. External water pressure behind the concrete lining is related to the rock mass permeability near the tunnel, consolidation grouting, and the number of cracks formed in concrete. In this project according to the uncertainty in uniformity of the joints and cracks in the rock mass near the tunnel, the tunnel is designed based on total external water pressure when the gate is closed. Also due to unsaturated rock mass near the tunnel in early days of dam operation and the low permeability of the rock mass, tunnels were designed under total external pressure.

Analysis results, including direct stresses, and shear stresses for individual loading cases defining critical paths in different sections of tunnel (Figure 16), have been surveyed with different angles and converted into axial, moment, and shear loads through integration. Some of results and Loadings for some of loading conditions are presented in Figures 5 to 15. The shear force, axial force and bending moment in the liner for different loading condition and sections of tunnels are listed in Table 4.

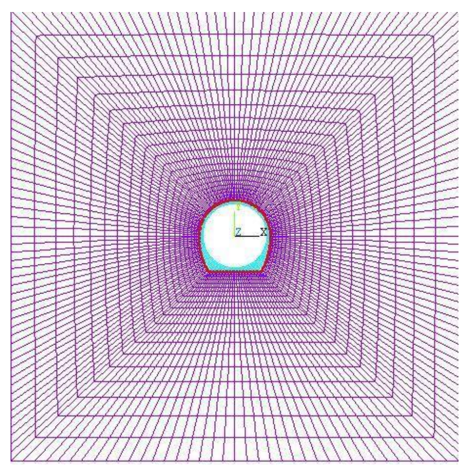

Figure 4. Finite element model of rock mass and tunnel

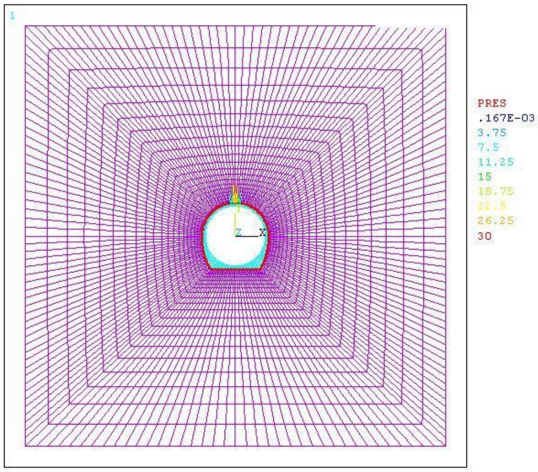

Figure 5. Loading of tunnel lining for loading condition 3

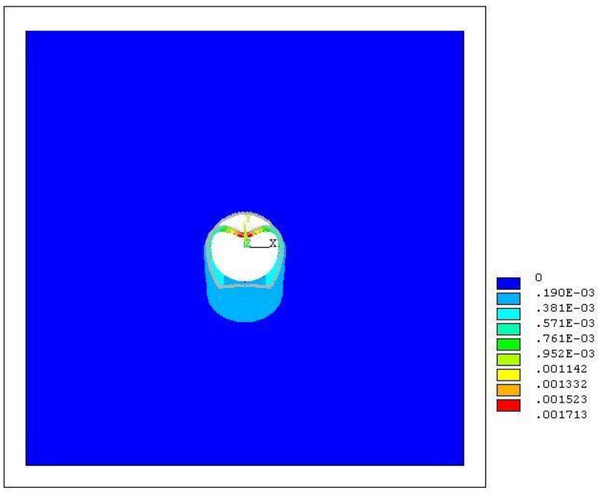

Figure 6. Deformation of tunnel lining for loading condition 3

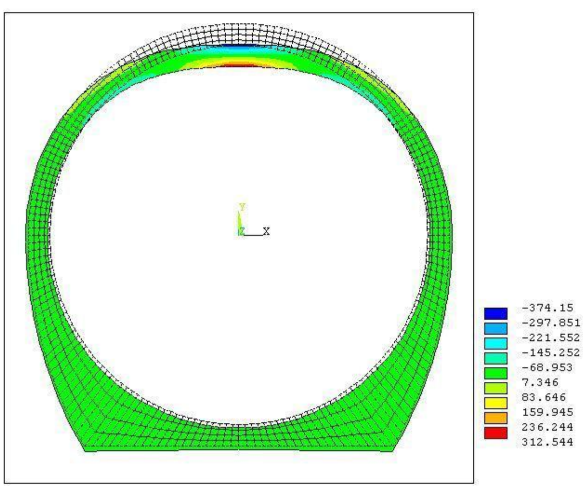

Figure 7. Maximum induced direct stress (X direction) in tunnel lining for loading condition 3

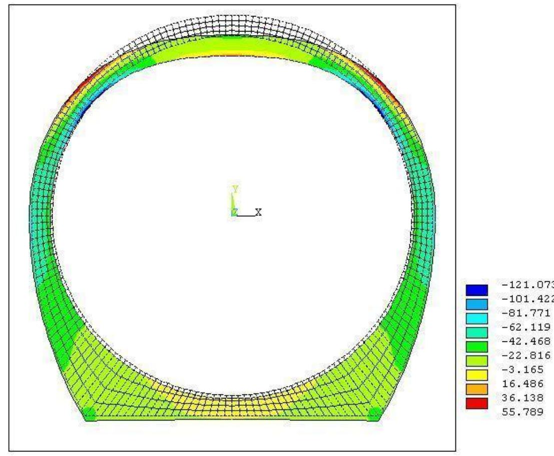

Figure 8. Maximum induced direct stress (Y direction) in tunnel lining for loading condition 3

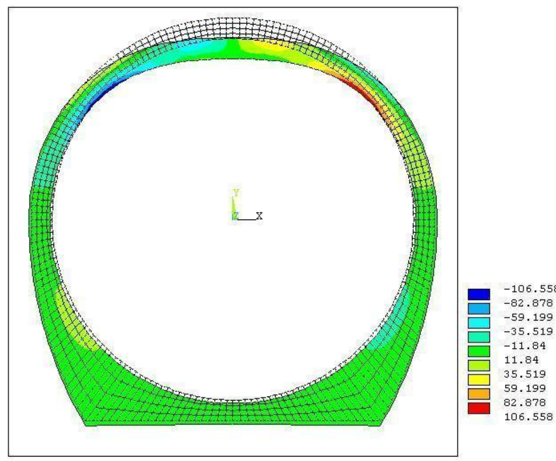

Figure 9. Maximum induced shear stress in tunnel lining for loading condition 3

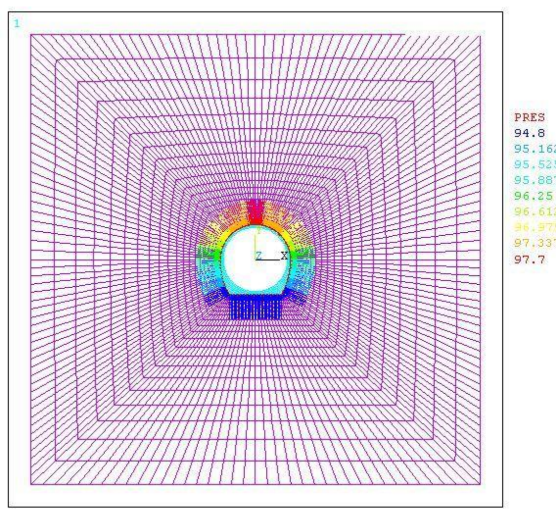

Figure 10. Loading of tunnel lining for loading condition 7

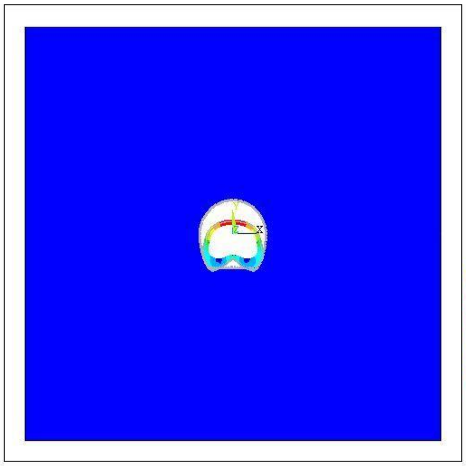

Figure 11. Deformation of tunnel lining for loading condition 7

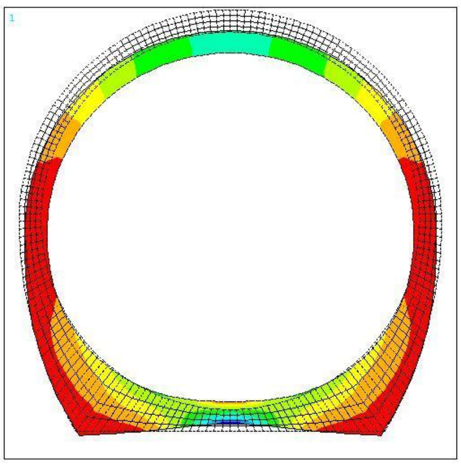

Figure 12. Maximum induced direct stress (X direction) in tunnel lining for loading condition 7

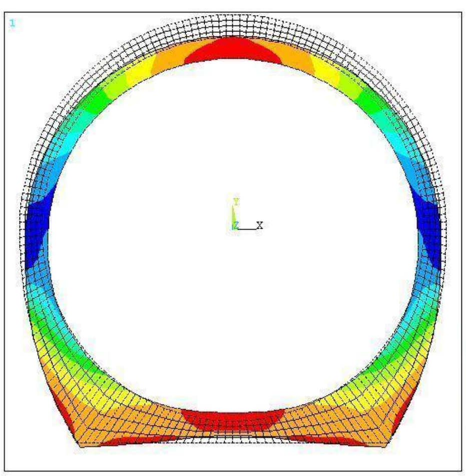

Figure 13. Maximum induced direct stress (Y direction) in tunnel lining for loading condition 7

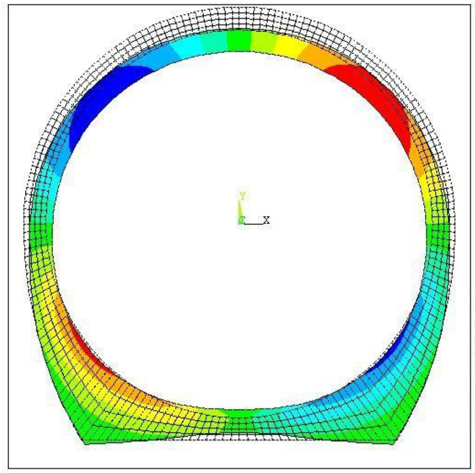

Figure 14. Maximum induced shear stress in tunnel lining for loading condition 7

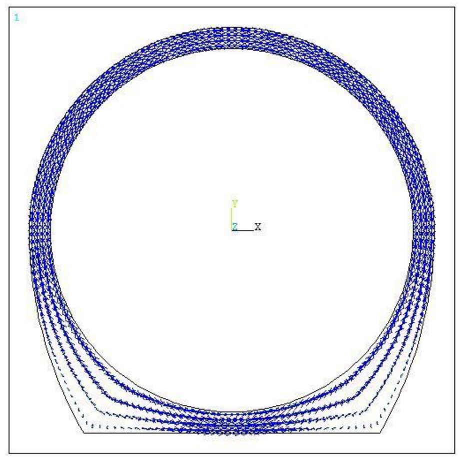

Figure 15. Distribution of Tensile and compressive stresses vectors in tunnel lining for loading condition 7

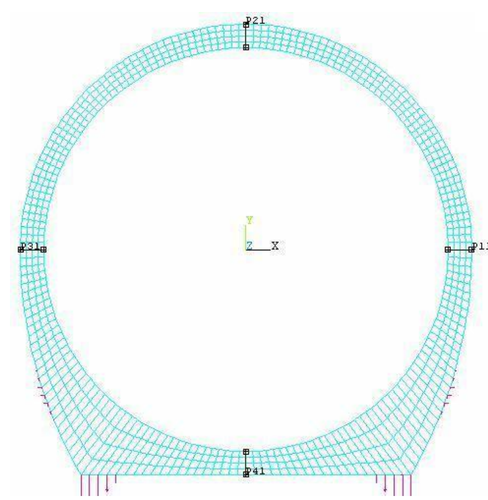

Figure 16. Arrangement of defined paths for Direct and Shear Stress Reading

Table 4. Induced Shear and Axial force and bending moment in the liner for different load conditions.

Loading Condition	Path	Vu[1] (ton)	Pu[2] (ton)	Mu[3] (ton.m)
1	P11	2.73	84.85	0.9
	P21	0.00	79.7	6.7
	P31	-2.73	84.85	0.9
	P41	0.00	62.62	2.08
2	P11	0.48	27.52	0.84
	P21	0.00	20.79	13.82
	P31	-0.48	27.52	0.84
	P41	0.00	20.58	0.82
3	P11	0.56	36.22	1.02
	P21	0.00	29.37	19.6
	P31	-0.56	36.22	1.02
	P41	0.00	24.96	1.00
4	P11	5.49	383.74	0.17
	P21	0.00	375.62	3.01
	P31	-5.49	383.74	0.17
	P41	0.00	364	22.17
7	P11	6.42	665.69	3.65
	P21	0.00	659.32	0.39
	P31	-6.42	665.69	3.65
	P41	0.00	638.57	40.81
8	P11	5.36	612.36	3.15
	P21	0.00	598.95	0.07
	P31	-5.36	612.36	3.15
	P41	0.00	614.04	46.11

[1]Shear Force
[2]Axial Force
[3]Bending Moment

3.1.4 Analysis under Internal Water pressure

When the rock near the tunnel is still not completely saturated, and the tunnel is in operation, there is only the internal pressure and the tunnel acts as a pressure tunnel. In this condition, tunnel control is based on governing design criteria: limiting the width of cracks, stresses in the reinforcement, and water losses.

For controlling tunnel under the internal pressure, the analytical method of Schleiss (Schleiss 1988) was utilized. The geological profile of the tunnel shows low permeability of the rock mass. However, calculations were done based on permeability of 10 Lugeon. The influence of internal pressure on width of cracks and stresses in the reinforcement is illustrated in Fig.17. The influence of internal pressure on water losses per unit length of tunnel (q), and external radius of the rock zone affected by the seepage (R) is illustrated in Fig.18.

The sudden decrease in crack width and stresses in reinforcement is due to the formation of new series of cracks. According to Schleiss (1997), width and spacing of cracks are related to tensile stresses in the reinforcement and concrete, and the cohesion between them. Also, the strain and stress in the reinforcement are not constant, have a parabolic distribution, and related to the history of cracking. In this method, by increasing the internal pressure at each stage, by use of equilibrium and compatibility of deformations equations, radial deformation of concrete and rock mass, stresses in the reinforcement, number and width of cracks, water losses due to concrete and rock permeability are calculated. At each stage, by increasing the internal water pressure, resulting tensions in the lining are controlled. As soon as the tensile strength of the concrete is exceeded, the next series of cracks will be formed. In the next series of cracks, the spacing of the cracks is reduced to half and the number of cracks doubled. The sudden reduction in crack width and stresses in reinforcement with increasing internal pressure is related to the formation of new series of cracks.

According to Schleiss (1997), the design of the reinforcement is governed by the following criteria:

- Limit stresses in the reinforcement to half the ultimate tensile stress of the steel
- Crack width limitation of 0.3 mm.
- The leakage out of the tunnel (the saturated rock zone), should not extend to the natural ground surface.

Table 5 gives results obtained from the effects of internal pressures on lining for normal condition and when rock is not saturated.

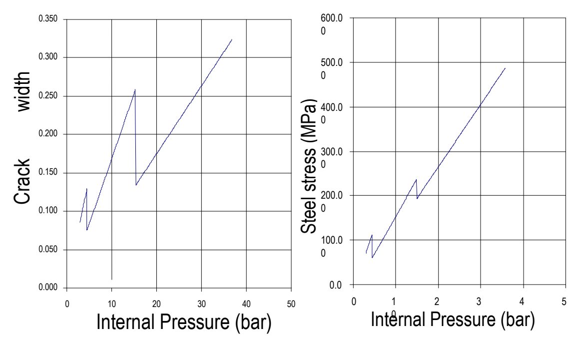

Figure 17. Width of cracks and steel stress as a function of internal pressure

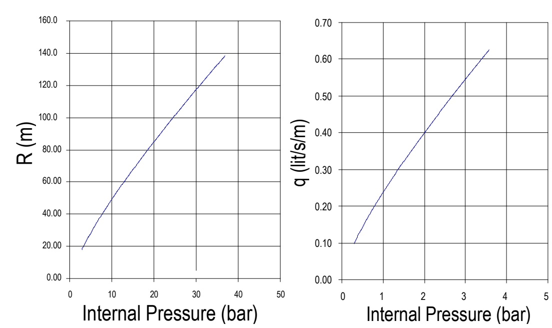

Figure 18. External radius of the rock zone affected by the Seepage (R), and water losses per unit length of tunnel (q) as a function of internal pressure

Table 5. Results obtained from the effects of internal pressures on lining in normal condition.

Width of the cracks (mm)	0.15
Steel Stress (MPa)	142.4
R, Seepage radius (m)	46.2
q, Water losses per unit length of tunnel (litr/s/m)	0.21

3.2 Construction

Excavation of the tunnels was done by drill and blast method from inlet and outlet, simultaneously. Excavation was done in two stages. First of all, half upper of tunnel was excavated and after that lower half of tunnel was excavated. (Figures 19 to 22)

With the start of excavation and knowledge of geological conditions, an engineering classification of RMR and Q was determined that considered rock support could be reduced to spot bolting. In cases where the rock mass was in good condition, only shotcrete was used.

Comparison of RMR and Q values obtained after excavations with RMR and Q values which are calculated based on data obtained from right bank galleries and outcrops, indicate that these values are similar to those obtained during excavation and the values calculated from the P wave velocity (Vp) are more conservative. (Table 6)

Table 6. Rock mass classification.

Location	RMR	Q	Rock type	Stages
Outcrops	49-60	1.79-5.75	Limestone Tuff	Feasibility Final design
Galleries	38-51	2.39-6.89	Limestone Tuff	Feasibility Final design
Tunnel outlet[1]		0.3		Feasibility Final design
Tunnels[1]		0.17-2.5	Limestone Tuff	Feasibility Final design
Tunnels	17-71	1.48 - 28	Limestone Tuff	Costruction
Tunnel outlet	7-30	0.02 - 0.2	Mudstone	Costruction

[1]Q calculated based on Vp Values

Data obtained during the tunnel excavation caused revision of the parameters of rock mass and also concrete lining specifications compared to first phase of design.

In initial studies, ground water infiltration into the tunnel cavity was predicted but during the excavation it was revealed that ground water level is below the spring line of the low level tunnel (left diversion tunnel).

Figure 19. View of right diversion tunnel from Intake.

Figure 20. External views of diversion tunnels from outlet.

Figure 21. View of diversion tunnels construction.

Figure 22. Internal view of concrete lining of tunnel

Concrete lining of diversion tunnels was constructed in two stages of floor and roof. Two hydraulic jacks metal forms were used.

One of the problems in the project was an alkali Aggregate Reaction (AAR). According to the results of Petrographic examination of aggregates (ASTM C295-90) and ASTM C 1260, ASTM C227, high alkali aggregate reaction was determined. One of these results is illustrated in Figure 23. Some studies have been done for reducing the effects of Alkali reaction. Finally, Pozzolanic cement with 25% amount of Pozzolan was used instead of Portand cement type II in concrete.

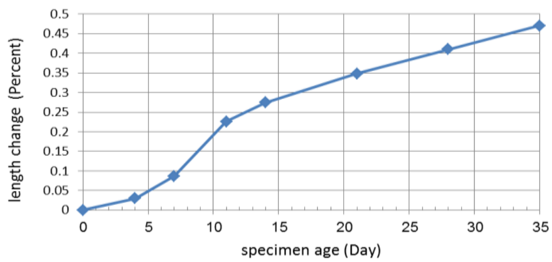

Figure 23. Results of standard test method for potential alkali reactivity of aggregates (ASTM C1260).

3.3 Conclusion

One of the important items in design and construction of tunnels is the review and revision of the initial design of the tunnels during the construction process. Geological studies and preliminary estimates of the rock parameters are also important in design and construction of tunnels. The results of the rock parameters obtained during excavation, caused revision of primary support and final lining of tunnels and the project will save a considerable amount of money.

According to the studies and calculations, considering the thickness of the lining and the required reinforcement, a concrete lining thickness of 55 cm. was retained. Figure 24 shows the cross-section of the diversion tunnels.

Another problem in this project is alkali aggregate reaction (AAR) in concrete. Based on the studies carried out in this project and also recommendations in standards and articles; special Pozzolanic cement with 25% pozzolan was used for concrete production.

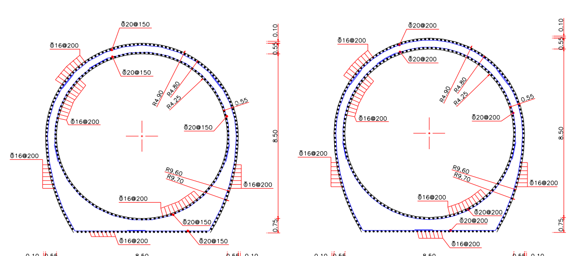

Figure 24. Cross-section of concrete lining for left and right diversion tunnels, from left to right.

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