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Rock reinforcement design for unstable tunnels originally excavated in very poor rock mass

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ABSTRACT: Number 7 Malatya railroad tunnel was excavated through the toe of a paleo-landslide material in 1930. Ever since that date, this horseshoe shape tunnel, having width and height of 5 m and 6 m, respectively, struggled with stability problems. A large amount of deformation (170 cm) developing through the years and leading to miss alignment of the tunnel was observed. In this paper, firstly a new approach developed for classifying very poor quality rock mass was presented. Secondly, evaluations were made on the stability of the tunnel. Thirdly, information is provided on rock reinforcement design. Finally, the results of finite element analysis (FEA) conducted for evaluating the stability of unreinforced tunnel and the tunnel reinforced by self drilling anchors (MAI-bolt) designed based on empirical method were given.

1 INTRODUCTION

Today, rock bolting is used in almost all types of underground constructions. Knowledge of the rock mass behavior and support effects of rock bolts are vital for the design and construction of underground structures in rock.

Malatya- Narli No: 7 railroad tunnel, situated at the central part of turkey, is a rail-road one that was initially excavated inside a valley of very poor quality rock mass. Due to the effect of the *in-situ* stress in the course of time, the deformations of the tunnel were exceeding the limit in which a huge collapse occurred and the tunnel was closed to the traffic.

In this study, the site investigation, the rock mass classification, and the experimental design of the rock bolt system were carried out in order to open the tunnel and to make it safe. At the same time, the numerical method was investigated to verify the empirical design approach.

During conducting this study, a new approach for classifying very poor quality rock mass was extracted.

2 ROCK MASS CLASSIFICATION

Rock mass classifications, which serve as powerful design aids in underground construction when used in conjunction with observational, analytical and numerical methods, provide a basis for characterizing the rock mass. For the qualification of the rock mass, RMR (Bieniawski, 1974, 1989), Q (Barton, 1974, 2002), and GSI (Hoek, 1994, 1997, 2000, 2002) systems are widely being used. These systems have been evolving due to the requirement of modifications for the different case studies; however, there are still doubts and confusions on the applications.

The rock mass surrounding the Malatya tunnel is composed of very poor rocks such as metasiltestone, clayey and silty sandstone, shale and phyllite.

Usually, the overall very poor quality rock mass is highly heterogeneous and anisotropoic owing to the combined effect of advanced weathering and severe tectonic stressing that gave rise to intense shearing which resulted in highly weathered rock masses. However, in view of the fact that the whole rock mass around the Malatya tunnel was completely decomposed and disintegrated as a result of the effect of highly historical horizontal stress (similar to great landslide), the rock mass was regarded as the homogenous material. Therefore, in order to investigate the tunnel section using the rock mass classification system, the tunnel length is considered as a unit along its axis.

The test results of core specimens taken from 67 boreholes were used as the input parameters for the assessment of rock mass classification systems. During rock mass classifying, it was difficult to obtain representative samples of the worst quality materials, especially when there are alternating weak materials. Then, Due to difficulty in retrieving the intact rock pieces, the uniaxial compressive strength σ_{ci}, and the material constant m_i, for example, are determined from tables published elsewhere.

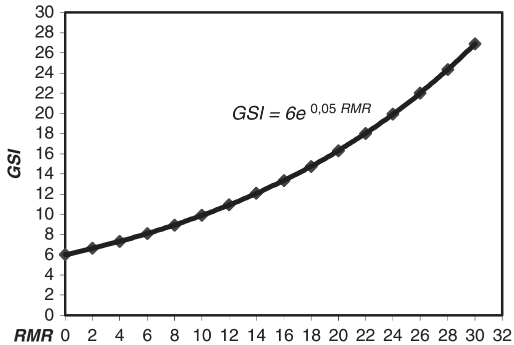

Figure 1. GSI vs RMR for very poor quality rock mass where RMR <30.

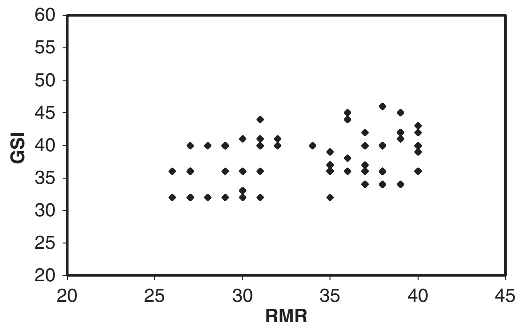

Figure 2. Distributive abundance of GSI vs RMR for weak rock mass in which RMR <40 (after Co şar, 2004).

A good approach for classification of very poor rock mass as a case study for Athens schist formation was developed by Hoek, Marinos and Benissi (1998). As demonstrated, the new category named disintegrated rock, which defined as poorly interlocked, heavily broken rock mass with a mixture of angular and rounded rock pieces, was appropriate to Malatya tunnel ground.

For very poor quality rock mass where the value of RMR <30, a new exponential correlation between Rock Mass Rating (RMR) and Geological Strength Index (GSI) was developed by authors as shown in the Figure 1.

$$GSI = 6e^{0.05\,RMR}$$

This correlation is compatible with the relation obtained from case studies data of the Alpine region carried out by Morales *et al.* (2004). Moreover, the above relation satisfies the definition of the GSI introduced by Hoek (1994) in such a manner that when RMR = 0, the minimum value of GSI is 6.

More recently, Coşar (2004) showed a relation between GSI and RMR for weak rock mass (RMR <40) has been given by distributive graph as demonstrated in Figure 2.

In Malatya tunnel, on the basis of core logging, the value of RQD for almost all cores were estimated to be zero; further, the RMR or M-RMR has not been calculated soundly. For this reason, the two approaches could have been taken into consideration to be more sensible. The first one was to evaluate the cores based on observational judgment. In this respect, the rock mass was classified in five categories (R1, R2, R3, R4, R5) on the basis of its geological characteristics. In order to make this classification in terms of engineering parameters, a new supplementary classification was integrated as given in the Table 1. This classification system is capable of characterizing the very poor quality rock mass with respect to engineering parameters of rock mass like rock mass strength and rock mass deformation modulus using defined indices. For example, the rock mass defined by R_3E_2 indicates the rock that its strength and modulus falls into the range of 450–550 kPa and 424–584 MPa, respectively.

Table 1. New quantitative classification system for very poor quality rock mass and its correlation with GSI system.

GSI	Very poor quality rock mass strength (kPa)	Definition	Very poor quality rock mass deformation modulus (MPa)	Definition
6 to 7	250–290	R1	344–424	E1
7 to 8	290–370	R1	424–584	E2
8 to 10	370–450	R2	584–744	E3
10 to 13	450–550	R3	744–944	E4
13 to 16	550–650	R4	944–1144	E5
16 to 21	650–750	R5	1144–1344	E6
21 to 27	750–850	R6	1344–1544	E7

The next approach was to use the GSI system due to very poor quality rock mass. The Geological Strength Index (GSI), introduced by Hoek (1994), Hoek *et al.* (1995, 1997, 1998, 2000, and 2002) provides a system for estimating the reduction in rock mass strength for different geological conditions. For very poor quality rock masses the value of RMR is very difficult to estimate and the balance between the ratings no longer gives a reliable basis for estimating rock mass engineering parameters. Consequently, Bieniawski's RMR classification should not be used for estimating the GSI values for poor quality rock masses. Furthermore, for poor quality rock masses GSI <30, relatively few intact core pieces longer than 100 mm are recovered and it becomes difficult to determine a reliable value for RMR. In this circumstance, the physical appearance of material recovered in the core should be used as a basis for estimating GSI. Therefore, the GSI guideline chart introduced by, Hoek, Marinos, and

Table 2. The value of various rock mass classification systems of the Malatya tunnel.

Rock unit	RMR	GSI	New classification
Very poor quality rock mass	0	5–6	R1–E1

Benissi (1998), Marinos and Hoek (2000, 2001) was considered as a sole means of evaluating the GSI for Malatya ground. The interesting specification of this chart was to take into account two various types of poor rock masses in terms of their structure, namely; disintegrated and foliated/ laminated/ sheared rock masses.

A latest rock mass classification system based on GSI system developed by Morales *et al.* (2004) falls the Malatya poor quality rock mass into class 8. This class defines the poor rock mass as a result of intense joining or very low strength near to soils. In other words, the rock masses included in this class are those with a high fracture rating in which the joints are very close, without predominant weak surfaces. This group also includes rock masses where the material itself, or because of weathering, is low strength. This class also includes very weathered rock masses. Circular failure and rotating landslides are the most characteristic forms of instability. The GSI is under 27 and the σ_{ci} is no more than 15 MPa in this class. The value of RMR, GSI, and new classification systems for Malatya tunnel are presented in the Table 2.

3 STABILITY EVALUATION

Based on results gained from the closed-form solutions of the development of rock mass failure surrounding an unsupported circular tunnel subjected to equal stresses in all directions carried out by Duncan-Fama (1993) and Hoek, Kaiser and Bowden (1995), a rock mass is considered to be weak when its *in-situ* uniaxial compressive strength is less than about one third of the *in-situ* stress acting on the rock mass through which the tunnel is being excavated (Hoek, 1999). In this case, a sudden increase in convergence of tunnel occurs. As a first approximation, the *in-situ* stress can be assumed to equal the product of the depth below surface and the unit weight of the rock mass. Considering that the vertical *in-situ* stress due to gravity loading was 1.54 MPa with the assumption that the unit weight of very poor rock mass was 0.022 MN/m^3 as well as the rock mass strength was 0.156 MPa, the rock mass strength to *in-situ* stress ratio would be 0.101. Hence, even from this point of view, a huge amount of convergence would have been anticipated as the maximum wall deformation of tunnel had been recorded about 60 cm. Sakurai (1983) has suggested that the stability of tunnels can be assessed on the basis of the strain in the rock mass surrounding the tunnel. The strain is defined by the ratio of tunnel convergence to tunnel diameter. A critical strain of approximately 2% represents the boundary between stable tunnels that required minimal support and unstable tunnels that require special consideration in terms of support design. Monitoring of the Malatya tunnel implied that this critical value of strain was also exceeded.

4 ROCK REINFORCEMENT DESIGN

The support requirements for the Malatya tunnel were estimated from the data of engineering classification of rock mass (empirical design approach). Further, the rock bolt support appropriate to the Malatya tunnel was empirically designed and installed. The MAI-bolts, which is a self driving full column cement-grouted bolts, were chosen to be more suitable for very poor quality rock mass because drill holes are usually closed before installing the bolt and the injection operation associated with rock-bolting make the ground improved in terms of engineering parameters like strength, modulus of deformation, and Hoek-Brown constants. In other words, since the extent of yielding or broken zone is directly related to the material properties, any improvement of the strength and frictional parameters must reduce the extent of overstressed rock. The MAI-bolts develop load as the rock mass deforms. Relatively small displacements are normally sufficient to mobilize axial bolt tension by shear stress transmission from the rock to the bolt surface (Indraratna and Kaiser, 1990).

In order to increase the stability, 110 m long section of the tunnel was supported systematically by MAI-bolts having a diameter of 32 mm, length of 6 m or 9 m and spacing of 1 m (Divleli and Ünal, 2004). A total of 15 rock bolts were installed around the tunnel and the invert of the tunnel. Floor was supported by installing 5 grouted bolts with the length of 5 m. The rock reinforcement details of number 7 tunnel are shown in Figure 3.

For the rock reinforcement design, the section of the tunnel to be supported was divided into three parts namely; A1, A2, and B. A systematic rock bolting was performed at that section. The MAI-bolts having 32 mm diameter, 6 m and 9 m length were installed in the parts A and B, respectively. A total of 20 rock bolts were installed at a cross-section of tunnel whereby the roof of the tunnel could be reinforced by 5 MAI-bolts while the two sides of walls were supported by 10 MAI-bolts, and the invert of the tunnel floor could be formed by installing 5 grouted bolts with the length of 5 m.

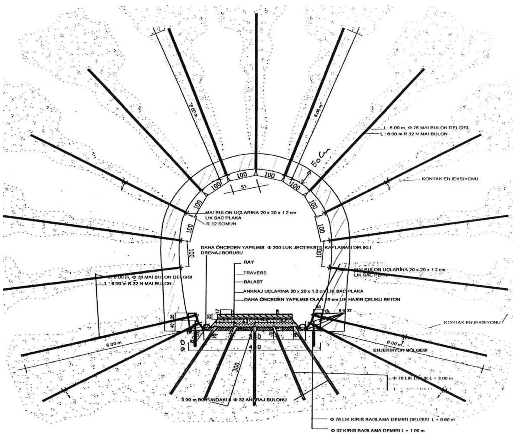

Figure 3. The sketch of rock reinforcement layout.

The bolt density parameter β, which reflects the relative density of bolts with respect to the tunnel parameter and the convergence reduction of the tunnel walls, can be expressed as follows:

$$\beta = \frac{\pi d \lambda a}{S_L S_T}$$

where d is the bolt diameter, λ friction factor for bolt/ground interaction, a is tunnel radius, and S_T, S_L are circumferential and longitudinal bolt spacing, relatively. For the Malatya tunnel, with the event that $d = 32$ mm, $\lambda = 1$, $a = 2.7$, S_T, $S_L = 1 \times 1$ m, the value of β is calculated as 0.271. It means that a design with high bolt density was carried out at Malatya tunnel to control and reduce the convergence (closure) of the tunnel. For very high bolt densities ($\beta = 0.3$), convergence reductions of about 60% can be obtained (Indraratna and Kaiser 1990). The ratio of the bolt length to tunnel radius (*L/a*) is between 2.22 and 3.33 in Malatya tunnel. Since the reduction in total convergence attained is more pronounced for long bolts, it is expected that the convergence control of tunnel would be appropriately performed. For example, at $\beta = 0.3$ and *in-situ* stress of 14 MPa, a convergence reduction of approximate 60% can be achieved by long bolts.

5 NUMERICAL ANALYSIS OF TUNNEL STABILITY

The main concern of numerical analysis of an underground opening like tunnel is that of exactly modeling that depicts the real situation in which tunnel analysis is to be solved. The program PHASE^2 (A 2nd Plastic Hybrid Finite Element- Boundary Element Analysis for Calculating Stresses and Estimating Support Around Underground Excavations) (Rocscience, 1999) is used for modeling the ground and tunnel and calculating stresses and displacements to evaluate the extension of the broken zone surrounding tunnel in terms of strength factor.

In the constitutive model of very poor rock masses, it is assumed that the surrounding rock mass behaves as an elasto-perfectly plastic material in which failure involving slip along intersecting discontinuities is assumed to occur with zero plastic volume change i.e. zero dilatancy angle and non-associated flow rule of plasticity (Osgoui, 2004). In the post failure behavior of rock mass, a nonlinear Hoek-Brown yield criterion and a linear Mohr-Coulomb plastic potential with the constant dilatancy angle are used to model the stress-strain regime for very poor rock mass.

As mentioned previously, the Malatya tunnel was originally driven through a tectonized paleo-landslide material, which regards as a very poor rock mass. The primary support system of the tunnel is comprised of concrete lining combined with steel arch. Since its construction, the Malatya tunnel has been influenced with in-situ geomechanical problems like the effect of high horizontal stress and pore water pressure that ultimately led to serious collapse.

In order to correctly model the tunnel, four stages are considered as follows:

1. modeling the ground before excavation.
2. modeling the tunnel regarding previous situation (primary support system included).
3. modeling the tunnel concerning installation of the new reinforcement system.
4. modeling the tunnel pertinent to ground consolidation.

The input parameters of the rock mass for performing the numerical methods are given in Table 3.

5.1 *Results of numerical analysis*

Main focused issue of the results was the yield zone in the rock mass surrounding the tunnel and induced displacements around the excavation. The strength factor, which is the ratio of available rock mass strength to induced stress in accordance with the chosen failure criterion, reveals the yield zone. The maximum induced displacements around the tunnel show the progression of displacement on excavation boundary at various stages of tunneling. Maximum principal stresses, total induced displacements, and minimum strength factors for three stages are presented in the Table 4 while Figure 4 demonstrates the extent of the failure in terms of strength factor for stage 2 (primary support system i.e. before rock-bolting), stage 3 (after rock-bolting), and stage 4 (after ground consolidation).

As can be deduced from Table 4, the maximum induced displacement was recorded on left wall as 0.015 m at stage 2. It should be noted that this value is corresponding to induced displacement at the time of stable tunnel in which the strength factor is around unit i.e. the collapse was more likely to occur, on the other hand, this value of displacement was irrespective of large deformation at the time of land-sliding or active loading. After systematic rock bolting the displacement increment (tunnel convergence) would have been under control to follow a consistent value of 0.013 m. In the case of ground consolidation, the displacement increment followed again a consistent rate.

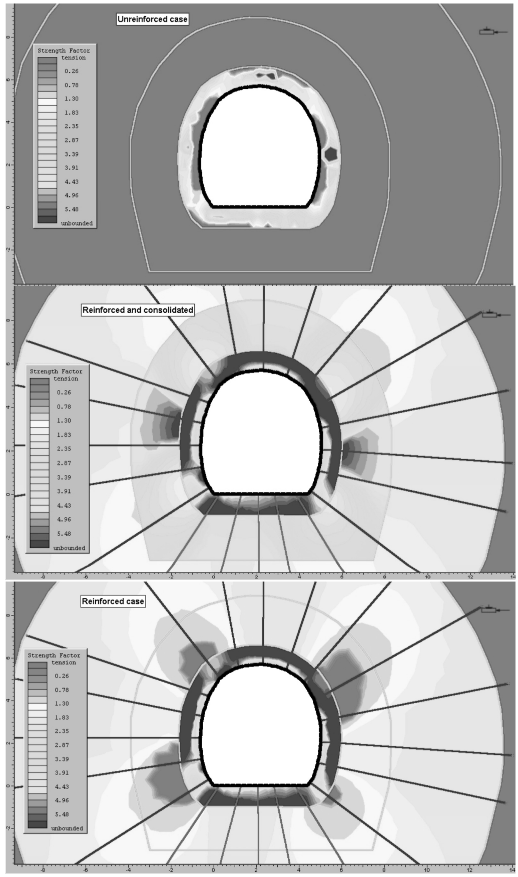

Figure 4. Extension of the failure zone around the tunnel and its decrease using rock bolting and consolidation.

Table 3. Input parameters for performing numerical analysis.

Property	Value
Failure criterion	Hoek & Brown (2002 version)
Geological strength Index (GSI)	5 or 6
Material type	Isotropic
Young's modulus (MPa)	167.68
Poisson's ratio	0.2
Compressive strength (MPa)	0.156
*m parameter (peak)	0.235
*s parameter (peak)	2.6e-5
Material type	Elastic analysis performed Plastic analysis performed
Dilation angle	Zero (non-associated)
*m parameter (residual)	0.235 (Elasto-perfectly plastic analysis)
*s parameter (residual)	2.6e-5 (Elasto-perfectly plastic analysis)

Table 4. Results from finite element analysis for 3 stages of tunneling.

Location	Parameter	Stage 2 (primary support system)	Stage 3 (reinforced tunnel by rock-bolts)	Stage 4 (reinforced tunnel + ground consolidation)
Roof	Total displacement (u_t) m	1.5E-2	1.2E-2	1.2E-2
Right wall		1.3E-2	1.3E-2	1.3E-2
Left wall		1.6E-2	1.3E-2	1.3E-2
Floor		1.4E-2	1.3E-2	1.3E-2
Roof	Maximum principal stress (σ_1) MPa	11.33	4.19	3.68
Right wall		2.95	1.51	1.22
Left wall		5.46	2.38	1.95
Floor		10.75	4.26	3.68
Roof	Minimum strength factor	1.22	3.29	3.68
Right wall		Tension*	1.58	2.01
Left wall		Tension	1.45	1.85
Floor		1.57	3.30	3.70

* The strength factor is less than one and the collapse is certain to occur.

As far as the strength factor is concerned, the tunnel was bound to collapse at stage 2 due to tension extension around tunnel as substantiated by numerical analysis. However, a great improvement in strength factor of tunnel boundary and rock mass around tunnel would have been made in the stage 2, where a systematic rock-bolting operation was carried out. The tension forms of failure mode could have been exchanged into strength factor of 1.48 and 1.58. Having been consolidated, the quality of rock mass surrounding the tunnel was improved in terms of engineering parameters; hence, the strength factor of tunnel boundary was remarkably increased. The main objective of rock mass consolidation was to increase the span of the tunnel for the railroad traffic requirements.

6 CONCLUSIONS

This study was carried out with a view to firstly classify very poor quality rock mass based on engineering parameters such as strength and modulus and to identify the effect of rock bolting on tunnel reinforced by MAI-bolts.

A new relationship, furthermore, correlating the GSI-system and RMR for very poor quality rock mass (RQD and RMR are equal to zero) was extracted.

A very poor rock mass entails a high-density rock bolting having the length more than span of the tunnel. For this purpose, a systematically rock bolt system with relatively high density ($\beta = 0.271$) and the ratio of bolt length to tunnel radius ($L/a = 2.22$ and 3.33) was designed.

Numerical analysis of tunnel proved that with the help of the rock bolting, a considerable increase in strength factor (decrease in extension of tension zone) around tunnel could be obtained. On the other side, displacements could be controlled.

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