Comparison of empirical methods with analytical methods of underground excavation design

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Abstract

Ground support requirements for a shallow 13 m diameter water tunnel were predicted using two empirical methods (RMR and Q-System). The predictions were compared with the results of a wedge analysis of the rock mass around the tunnel. In addition, a simple two-dimensional numerical model was run to examine the effect of internal water pressure on the rock mass around the tunnel. The study indicates that although empirical predictions may serve as a first pass design, they may not be adequate for stabilising all potentially unstable rock wedges and for preventing adverse effects of internal water pressures in low in situ stress environments.

1. INTRODUCTION

Empirical rock mass classification methods are useful tools for underground excavation design, particularly during early stages of a project. These methods semi-quantitatively describe the rock mass, and prescribe rock mass stabilisation measures based on experience gained elsewhere in similar ground conditions. Currently, the most widely used empirical methods are RMR (Bieniawski, 1973) and Q-System (Barton et al., 1974). The experience gained subsequent to the initial introduction of these methods enabled their creators to issue revised versions of RMR (Bieniawski, 1974, 1976, 1979 and 1989) and Q-System (Grimstad & Barton, 1993; and Barton & Grimstad, 1994).

In spite of the revisions, these methods have limitations, which have been discussed by several authors (Speers, 1992; Hudson & Harrison, 1997; Palmstrom et al., 2000; Peck, 2000; Mikula & Lee, 2003; Stille & Palmstrom, 2003; and Palmstrom & Broch, 2006). One approach to assess the limitations of empirical methods and to suggest improvements, if necessary, is to compare their predictions with those of analytical methods.

This paper applies RMR and Q-System to a shallow 13 m diameter hydropower tunnel, and compares their predictions with those made by an analytical method. The analytical method considered is block theory (Goodman & Shi, 1985) using UNWEDGE software code (Rocscience Inc., 2003). In addition, to assess the type of support required to prevent adverse effects of internal water pressure on the rock mass around the tunnel, a simple two dimensional numerical model was run using Universal Distinct Element Code (UDEC) (Itasca Consulting Group Inc., 2004). The results of the numerical modelling are also compared with the empirically derived supports.

2. TUNNEL SITE CONDITIONS

The 13 m diameter, 240 m long, horseshoe-shaped tunnel, constructed to feed three 80 MW power generating units, is part of the Chiew Larn Hydropower Project in Thailand. The tunnel is shallow and located in a hill slope. Tunnel alignment is 140^{o} E with a plunge of 10^{o}. The ground surface above the tunnel is uneven, but has an overall slope of about 10^{o} towards south (downstream) and about 20^{o} to the west. The tunnel overburden varies between 25 m and 50 m above crown level, with an average of approximately 30 m.

The tunnel was driven through greywacke sandstone. Along the tunnel the intact rock material is fresh, with an average UCS of 138 MPa and an average intact rock Young's Modulus of 51 GPa. Unit weight of intact rock is 26.5 kN/m^3 and Poisson's Ratio is 0.23. Three major discontinuity (joint) sets (Sets 1, 2 & 3) and two minor sets (Sets 4 & 5) are present in the rock mass (Ranasooriya, 1985). Mean orientations of joint sets, based on 328 measurements, are given in Table 1.

Of the three major joint sets only two are prominent within any selected length of the tunnel, with the third major set occurring at random. In the first 80 m of the tunnel, Sets 1 and 2 are prominent with Set 3 occurring at random. From 80 to 130 m, Sets 2 and 3 are prominent and Set 1 is random. From 130 to 240 m Sets 1 and 3 are prominent and Set 2 is random. Sets 4 and 5 are present randomly with no recognisable pattern. Joint surface roughness, waviness, aperture size and filling of all five sets vary as shown in Table 1. Although the joints are relatively open, the tunnel was mostly dry, with water dripping in some places. This is partly because the tunnel is below the regional groundwater level. A notable feature is that, when the joint surface conditions (roughness, aperture and filling) are at the worst observed state the joint spacing is at its best state (i.e. >2 m).

Table 1. Orientations and surface features of discontinuity (joint) sets

	Set 1	Set 2	Set 3	Set 4	Set 5
Dip Angle	76	79	37	62	44
Dip Direction	016	112	231	151	067
Persistence	>20m	>10m	>20m	3-20m	3-20m
Aperture	0.25 - 10mm	0.25 - 10mm	2.5 - 100mm	2.5 - 10mm	0.25 - 100mm
Filling	coated - clay	coated - clay	coated - sandy clay	coated	coated - sandy clay
Roughness	rough to slickensided				
Waviness	undulating to planar				
Spacing	minimum 0.6m – maximum >2m				

3. SUPPORT PREDICTION USING RMR AND Q-SYSTEM

The procedures for applying RMR and Q-System have been well known for more than three decades, and different versions of these methods are documented in the references cited earlier and in many texts on underground excavation design. For this study, RMR (Bieniawski, 1989) and Q-System (Barton & Grimstad, 1994) were used.

Two extreme rock mass scenarios were considered: the best and the worst combinations of ground conditions observed along the tunnel. RMR and Q ratings were assigned to the best and the worst ground conditions and the ratings are given in Table 2. Note that joint spacing is wider when the joint surface conditions are at their worst observed state, which is reflected in the RMR ratings.

Table 2. Ratings assigned for RMR and Q parameters

RMR			Q-System		
Parameter	Best	Worst	Parameter	Best	Worst
Strength	12	12	RQD	100	60
RQD	20	13	Jn	6	9
Spacing	15	20	Jr	3	1.5
Condition	20	0	Ja	2	6
Groundwater	15	10	Jw	1	1
Adjustment	-5	-10	SRF	2.5	5
RMR value	77	45	Q value	10	0.33

Table 3. Support recommendations by RMR and Q-System

Support type	RMR				Q-System			
	Best		Worst		Best		Worst	
	Roof	Walls	Roof	Walls	Roof	Walls	Roof	Walls
Bolts	L=4m S=2.5m locally	None	L=5m S=1.5-2m	L=5m S=1.5-2m	L=4m S=1.5-2m	None	L=5m S=1.5m	L=4m S=1.5m
MH/FR	MH-occasional	None	MH	None	None	None	FR	FR
Shotcrete	50mm (WR)	None	50-100mm	30mm	None	None	90-120mm	50-90mm

L - length; S - spacing; MH - wire mesh; FR - fibre reinforced; WR - where required.

On the basis of the RMR and Q values in Table 2 and the support recommendations provided by Bieniawski (1989) and Barton and Grimstad (1994), support types were selected for the tunnel roof and walls in both best and worst ground conditions, and are presented in Table 3. An Excavation Support Ratio (ESR) of 1.8 was selected for Q-System. Since RMR-derived supports are only relevant to a 10 m diameter tunnel, the bolt lengths recommended by RMR were increased by approximately 30% to take into account the 13 m diameter of the case tunnel.

4. WEDGE STABILITY ANALYSIS

A tetrahedral rock wedge stability analysis was performed to examine whether the supports predicted by the empirical methods are adequate for the failure mechanisms that can be identified by the wedge analysis. For this purpose the ubiquitous joint method was adopted using UNWEDGE software code, which can analyse tetrahedral wedges formed by three intersecting joints and the free surface of the excavation, and allows identification of all possible tetrahedral wedges. The use of the ubiquitous joint method was justifiable because the classification of the rock mass using the two empirical methods also assumed that joints were ubiquitous.

The analysis showed that 19 different combinations of joints had the potential to form tetrahedral rock wedges with apex height greater than 1 m. Table 4 shows the joint set combination, location, maximum apex height and maximum weight of the 19 rock wedges. Factor of safety (FOS) against failure of each wedge was computed using joint shear strength parameters representative of the best and the worst joint surface conditions considered for RMR and Q-System. The Joint shear strength parameters (best: c=10 kPa, $\Phi=30^{o}$) and (worst: c=0 kPa, $\Phi=20^{o}$) were estimated based on the suggestions given by Barton & Grimstad (1994) and considering the potential for water saturation of the joints. Since the tunnel overburden is between 25 and 50 m, it was assumed that the wedges are subjected to gravity loading only with no clamping effect. FOS of unsupported rock wedges in the best and the worst ground conditions, F-B and F-W respectively, are given in Table 4.

Table 4. Results of wedge analysis using UNWEDGE

Wedge #	Wedge characteristics				FOS-Best Conditions			FOS-Worst Conditions		
	Sets	Location	Apex(m)	Weight(kN)	F-B	F-BR	F-BQ	F-W	F-WR	F-WQ
1	1, 2, 3	Roof	3.9	983	0.00	0.77	1.39	0.00	1.49	2.17
2	1, 3, 4	Roof	2.2	261	0.00	1.02	1.96	0.00	2.14	2.53
3	1, 2, 4	Roof	5.2	585	0.39	0.84	1.64	0.07	1.41	1.38
4	2, 4, 5	Roof	4.2	628	0.41	1.18	1.49	0.07	1.65	2.31
5	1, 3, 5	Roof	1.2	254	0.27	2.52	4.57	0.09	6.16	7.51
6	2, 3, 5	R/wall	6.2	9140	1.66	2.30	2.60	0.82	1.96	2.34
7	2, 3, 5	L/wall	6.0	7829	1.64	2.23	2.53	0.79	1.94	2.36
8	3, 4, 5	R/wall	5.2	7328	0.87	1.55	1.85	0.37	1.51	1.85
9	3, 4, 5	L/wall	5.1	6339	1.18	2.05	2.64	0.53	2.25	2.84
10	1, 3, 5	R/wall	3.9	4227	0.88	1.51	2.16	0.38	1.76	2.25
11	1, 3, 5	L/wall	3.9	4166	1.08	2.45	3.05	0.48	2.86	3.66
12	1, 3, 4	L/wall	3.6	1025	1.13	2.21	3.20	0.48	2.71	3.43
13	1, 3, 4	R/wall	3.6	962	1.83	2.71	3.23	0.73	2.26	2.82
14	1, 2, 3	L/wall	1.9	293	1.26	2.65	4.11	0.48	2.62	3.31
15	2, 4, 5	R/wall	2.6	278	1.02	1.58	1.57	0.38	1.39	1.88
16	2, 4, 5	L/wall	2.4	260	1.94	3.21	5.01	0.51	3.44	3.78
17	1, 2, 3	R/wall	1.6	196	1.14	2.44	2.55	0.17	1.88	3.24
18	1, 2, 4	L/wall	1.2	47	3.05	5.59	5.82	0.51	2.56	3.68
19	1, 2, 4	L/wall	1.1	36	2.25	2.29	3.15	0.17	1.30	1.30

Note: F-B = FOS for best ground conditions with no artificial support; F-W = FOS for worst ground conditions with no artificial support; F-BR = FOS for best ground with 4m long bolts in a 2.5m x 2.5m pattern (as per RMR); F-BQ = FOS for best ground with 5m long bolts in a 2.0m x 2.0m pattern (as per Q); F-WR = FOS for worst ground with 5m long bolts in a 1.5m x 2.0m pattern (as per RMR); F-WQ = FOS for worst ground with 5m long bolts in a 1.5m x 1.5m pattern (as per Q).

The analysis was extended to examine whether the empirically predicted support could stabilise the 19 theoretically possible rock wedges. Since the wedges may be present in the best or the worst ground conditions, the analysis considered both the best and worst joint shear strength scenarios mentioned earlier. Table 4 also shows the FOS assuming that only the bolts recommended by the two empirical methods were installed. The bolts considered are cement grouted type with 100% bond efficiency and an ultimate tensile strength of 180 kN installed normal to the rock face.

The effect of empirically recommended shotcrete layers was also analysed using UNWEDGE, which can compute punching shear capacity of shotcrete along the edge of a rock wedge. The results of the analysis (not provided here) showed that shotcrete increased the FOS of large rock wedges beyond the desired level. (A FOS of 1.5 and 2 for the walls and roof, respectively, were selected for long term stability.) The shotcrete layers would also provide the necessary support for the smaller rock blocks in between bolts and for fractured ground and would meet the selected FOS. The results of the analysis show that the supports predicted by the two empirical methods for the worst ground conditions are adequate to stabilise the theoretically possible tetrahedral rock wedges.

For the tunnel walls in the best rock mass, the two empirical methods recommend no support (Table 3). However, the analysis showed that there is potential for 14 different tetrahedral rock wedges in the tunnel walls (Table 4). Four of these wedges (# 8, 10, 11 & 15) have a FOS of less than or equal to one indicating potential instability in the best rock mass. Three more wedges (# 9, 12 & 17) have a FOS of less than 1.2, which is considered to be below the acceptable level.

For the tunnel roof in the best rock mass conditions, RMR recommends rock bolts plus mesh and 50 mm of shotcrete where required (mesh and shotcrete are probably for fractured ground, if present, and not for the entire roof). As can be seen from Table 4 there is potential for five rock wedges with zero or near zero FOS in the roof. First three of these wedges, with maximum possible weights of 983, 261 and 585 kN, will have a FOS of less than or equal to one, if RMR recommended rock bolting pattern for the best rock mass conditions is used (Table 4). With the same bolting pattern the fourth wedge with a maximum possible weight of 628 kN will have a FOS of only 1.18. The analysis showed that shotcrete would increase the FOS of these possible rock wedges to an acceptable level. However, RMR does not recommend shotcrete for large rock wedges in the roof.

With the Q-System recommendation for the tunnel roof in the best rock mass conditions the FOS for wedge nos. 1, 3 and 4 are below the acceptable level for long term stability of large rock blocks in the tunnel roof. Q-System does not recommend shotcrete for the roof in the best rock mass.

During excavation of the tunnel, several potentially unstable large rock blocks were identified within both the best and worst ground conditions described earlier. These blocks were temporarily stabilised using 6, 4 and 3 m long mechanically anchored and resin grouted rock bolts.

5. ASSESSMENT OF INTERNAL WATER PRESSURE EFFECT

Two important design considerations for water tunnels are hydraulic jacking and water leakage. Hydraulic jacking or uplift of the surrounding ground can occur if water pressures imposed within a rock mass are greater than the in situ compressive stresses in the rock mass (Benson, 1989).

In the case tunnel the internal water pressure along the centreline range from 0.45 to 0.82 MPa, and the gravity induced vertical stress at crown level (assuming rock mass density of 25 kN/m^3) range from 0.61 to 1.07 MPa depending on the overburden thickness. At any given point along the tunnel the internal water pressure is less than the confinement stresses due to overburden. However, since the natural groundwater pressure along the tunnel alignment is less than the water pressure inside the tunnel, water loss by seepage is possible through open interconnected joints. Further, the seepage may cause instability at the ground surface, particularly on the hill slope. This may be demonstrated by a simple two dimensional numerical model using UDEC software code.

For this purpose, and also to examine the type of support required to prevent the adverse effects of internal water pressure on the rock mass around the tunnel, a two dimensional numerical model was run using UDEC. The model was constructed assuming jointed rock from the natural ground surface, and ignoring the near surface soil profile. Two major joint sets (Sets 2 and 3), which are sub-parallel to the tunnel axis, were included in the model. The model assumed that joint spacing is 2 m and that joints are fully persistent with the best joint shear strength parameters considered earlier. Joint aperture size was varied to reflect the observed site conditions given in Table 1.

In situ stresses were assumed to be due to gravity only. The intact rock blocks were assumed to be elastically deformable, with intact rock bulk modulus of 30 GPa and shear modulus of 20 GPa based on the laboratory determined intact rock Young's modulus and Poisson's ratio mentioned earlier.

Estimated joint normal and shear stiffness values of 800 MPa/m and 100 MPa/m, respectively, were used, but were varied to investigate the sensitivity of the model and found that the higher the joint stiffness, the lower the maximum displacement of rock blocks.

Four sections across the tunnel representing different ground profiles and internal water heads were considered. Two cases were modelled for each section. Case 1: bolts installed as per empirical recommendations for the best ground conditions, and steady-state seepage was considered. Case 2: a fully impermeable liner installed covering the entire tunnel periphery.

The results of Case 1 modelling showed that seepage would occur through the rock mass in the four sections considered, even if the joint apertures were at their observed lowest range (0.25-0.5 mm). The Case 1 modelling also showed that, instability may occur at the ground surface (on the hill slope) when the overburden thickness is about 30 m (site average). The possible seepage paths, displacement and velocity vectors are shown in Figure 1 for a section where overburden thickness above crown is 32 m (vertical stress is 0.78 MPa) and internal water pressure is 0.6 MPa.

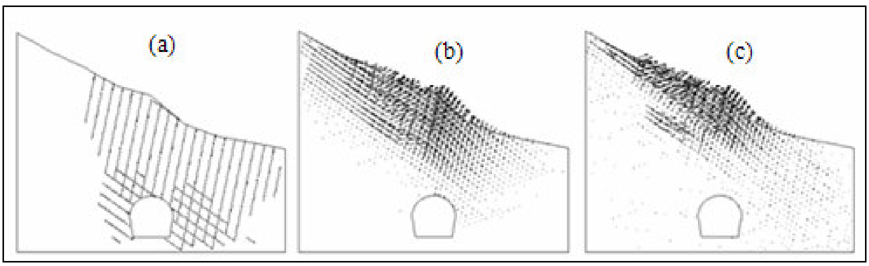

Figure 1. (a) Possible flow paths, (b) Displacement vectors, (c) Velocity vectors

Case 3, which assumed no flow through the tunnel periphery, showed insignificant movement at the ground surface. This demonstrates that an impermeable liner is required to prevent seepage from the tunnel and to minimise the risk of instability on the hill slope above the tunnel.

The empirical methods considered in this study claim that the predicted support measures represent permanent support and that the predictions take into account the purpose of the excavation. For instance, Q-System (Barton & Grimstad, 1994) recommends an ESR value of between 1.6 and 2.0 for predicting support requirements for hydropower tunnels. And, Bieniawski (1989) presents an example application of RMR to a shallow water tunnel with an overburden of between 15.3 and 61 m above crown level. This overburden thickness range is comparable to that of the case tunnel.

As mentioned earlier the empirical methods recommended rock bolts and shotcrete for the worst ground conditions intersected in the tunnel. For the best rock mass intersected in the tunnel no support is recommended for the walls and only bolts are recommended for the roof. Clearly, the empirically derived supports are unlikely to eliminate seepage, which could eventually lead to instability at the ground surface. Application of shotcrete along the entire tunnel periphery including the invert, additional to the empirical recommendations for the worst ground conditions, may be an option to control seepage. However, since shotcrete is known to have numerous incipient cracks formed by shrinkage, expansion and shear movement etc, it may not completely eliminate seepage and the risk of instability at the ground surface. It would perhaps delay the problem, as the pressures tend to build up slowly due to decreased flow through the shotcrete.

Considering the potential for leakage losses and hydraulic jacking, the tunnel was fully steel lined with the annulus between the tunnel and the lining filled with concrete. Rock bolting was used only as a temporary rock mass stabilisation measure. No shotcrete was used.

6. CONCLUSIONS

The wedge stability analysis showed that the rock support measure predicted by RMR and Q-System provide adequate safety factors against tetrahedral wedge failure mechanisms identified within the worst case rock mass conditions in the tunnel.

For walls in the best rock mass, the two empirical methods do not recommend any artificial support. In contrast, the wedge analysis showed that rock bolts could be warranted for walls in the best rock mass conditions.

Since the tunnel is shallow and located on a hill slope, seepage could occur through interconnected joints in the rock mass. Numerical modelling showed that even with the lowest joint apertures of the rock mass, seepage may occur and cause instability at the ground surface. These aspects are not considered in the two empirical methods.

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