



Modified Q-index for prediction of rock mass quality around a tunnel excavated with a tunnel boring machine (TBM)

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Abstract

Rock mass quality is closely related to tunnel stability and supporting measures. The Q-system, based on the drilling and blasting method, is one of the most important methods for rock mass classification systems and provides reliable long-term protection for tunnel excavation and reinforcement. However, in comparison to the drilling and blasting method, tunnels excavated using the tunnel boring machine (TBM) method have smooth and integral walls. The number of structural planes in these tunnels, their extension lengths, opening widths, and other characteristics are significantly different from those excavated using the drilling and blasting method. These differences lead to prediction errors in rock mass quality when the Q-system is applied to tunnels excavated by a TBM, and the coincidence rate is less than 70%. In this study, a reduction factor R_{Kv} , based on the wave velocity test, is used to replace the RQD/Jn term in the Q-system to reflect the integrity of the rock mass. This replacement can overcome the shortcomings that result from the smooth walls in TBM tunnels by applying the wave velocity during tunnel construction. Based on multiple regression analysis of R_{Kv} , we established a Q_T method for rock classification of material surrounding TBM tunnels. This new method provides a prediction coincidence rate of more than 85%.

Keywords Rock mass classification · Rock mass integrity · Reduction factor · TBM tunnel · Q-system

Introduction

Rock mass classification is widely considered the most practical method for evaluating the quality of rock mass in underground engineering. The Q-system was originally developed for rock mass classification as a convenient tool for evaluating support systems in tunnels and rock caverns (Barton et al. 1974). Influencing parameters in Q-logging have been broadly discussed by Barton et al. (1974) and are generally expressed in the following equation:

$$Q = \left(\frac{RQD}{Jn} \right) \left(\frac{Jr}{Ja} \right) \left(\frac{Jw}{SRF} \right) \quad (1)$$

where RQD is the percentage of competent core sticks larger than 100 mm compared to the total length (Deere et al. 1967), Jn is the relevant rating for the number of joint sets, Jr is the rating for joint surface roughness, Ja is the rating for joint alteration, Jw represents the water condition of the joint, and SRF represents the rating for the rock mass in situ stress situation. Barton et al. (2002) improved the Q-value so that it is consistent for onsite characterization and tunnel design. However, conventional methods have primarily focused on the properties of rock masses relevant to construction to choose suitable support systems or excavation methods (Barton et al. 1974; Barton 2002, 2007; Grimstad and Barton 1993; Bieniawski 1989). In recent years, as computer science and numerical computation methods have developed, rock quality classification and stability evaluation methods have been mostly researched using simulation approaches, e.g., neural network method, distance discriminant method, and fuzzy mathematics method (Choi et al. 2009; Christe et al. 2011; Qiu et al. 2010; Ramamurthy 2004; Chen and Liu 2007; Liu and Chen 2007; Hoseinie et al. 2009; Aydin 2004; Ergül 2001; Jalalifar et al. 2011; González and Vallejo 2003; Hoseinie et al. 2008; Heuer 1995; Hoek & Diederichs 2006; Read et al. 1999). Many rock mass classification systems have been introduced in mining and civil engineering and are

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often used in many empirical design practices for rock engineering, despite potential differences from their original intended applications. A good example of a classification and an implementation mismatch is the application of Q-classification systems, which were primarily developed for the design of ground support systems but are currently used to estimate tunnel boring machine (TBM) performance for tunneling projects. Alber (1996, 2000) used rock mass strength to evaluate the specific penetration rate (mm/rev/MN/cutter), which is a modified rock mass rating (RMR) system for application in tunnels excavated using TBMs. Barton (1999, 2000) proposed a Q-method for TBMs based on an expanded version of the Q-system and considered all intact rock, rock mass, and machine parameters influencing the TBM penetration and advance rate. Bieniawski et al. (2006) introduced a new rating system to determine the rock mass excavatability (RME) indicator when choosing between the TBM and drill-blast methods for constructing a tunnel. The RME index was developed based on the case studies from over 400 tunnel sections and updated several times with more case histories (Bieniawski et al. 2007a, b; Bieniawski and Grandori 2007). As addressed in Barton and Bieniawski (2008), the RME was developed based on the RMR classification, similar to the way Q_{TBM} was developed based on the Q-system. Qi and Wu (2011) used a fuzzy mathematics method to establish a membership function for rock mass quality and uniaxial compressive strength, termed the rock mass integrity index, K_V . Liu et al. (2011) selected four indicators, including uniaxial compressive strength, rock strength stress ratio, rock mass integrity, and rock hardness, and divided the deep roadway surrounding the rock into four levels: very good, good, poor, and very poor, to predict the risk of the TBM becoming stuck. Sun et al. (2008) proposed a method to identify the surrounding RMR based on TBM tunneling parameters and slag material characteristics, and analyzed the characteristics of the TBM slag material to obtain the geological structure of the rock mass, state of the discontinuous surface, and groundwater information. Li et al. (2010) proposed a method for classifying the surrounding rock in the construction of the Qinling Mountain Tunnel; the method can also be used to classify the rock surrounding a TBM hydraulic tunnel according to the working efficiency during TBM construction, characteristics of the ballast, and water inrush conditions. Wu et al. (2006) indicated that there is a close relationship between the surrounding rock and rock drillability in TBM construction and divided the rock surrounding the tunnel into three levels: A (good), B (medium), and C (bad) according to the uniaxial compressive rock strength, rock abrasion resistance, and the integrality of rock mass.

Based on Wu et al. (2006), it is clear that the TBM classification method emphasizes efficiency, which considers the rock mass parameters and the interaction between rock mass and TBM, is suitable for the rock quality evaluation in TBM tunnels. Furthermore, it can predict the mechanical behavior and geological problems in the front section of a tunnel.

However, Wu et al. (2006) found that large changes in lithology of the surrounding rock leads to significant errors in the results of the Q_{TBM} model, which cannot meet the requirements of the surrounding rock classification needed for TBM tunnels.

There are many factors influencing the grading of rock mass quality; there are some geological uncertainties, and the quality of the rock mass can be determined through empirical and qualitative methods. However, long-term engineering practice holds it necessary to establish a quantitative calculation method for the surrounding rock. This can avoid discriminant errors by field engineers for different experience levels, is beneficial to standardization, and is conducive to scientific development. Therefore, establishing a quantitative model to predict the rock mass of tunnels created with TBMs is our objective. In this study, the structural plane number, extension length, opening width, and other characteristics of TBM tunnels were analyzed and the causes of errors were investigated when the traditional Q-system was applied to the TBM tunnel. Then, based on the wave velocity data, a reduction factor R_{K_V} for rock mass integrity is introduced. The influence weight is calculated using back analysis and a new method is established for rock mass quality classification of TBM tunnels and verified using the data from the Jinping Tunnel.

Project description and structural plan characteristics of TBM tunnels

Project background

A large-scale hydropower project, Jinping II Hydropower Station, with a total installed capacity of 4800 MW, is located on the Yalong River in West China. The station diverts water to generate electricity. The average length of the four water diversion tunnels is 16.25 km and the largest burial depth is 2525 m. The largest geo-stress is 70 MPa and water stress is as high as 10.2 MPa. This is one of the most complex hydropower stations in the world.

The primary tunnel projects for this hydropower station were the construction of the drainage tunnel, two access tunnels, and four headrace tunnels (Fig. 1). Many remarkable characteristics, such as the large buried depth, long tunnel length, and large tunnel diameter have influenced the construction and operation of this large-scale underground hydropower project.

The #1 and #3 water diversion tunnels, 13 m in diameter, were excavated using a TBM, manufactured by Robinson and Herrenknecht AG. The #2 and #4 water diversion tunnels were excavated using the drill-blasting method. These four tunnels are parallel to each other, with a distance of only 50 m between them; these unique geological conditions

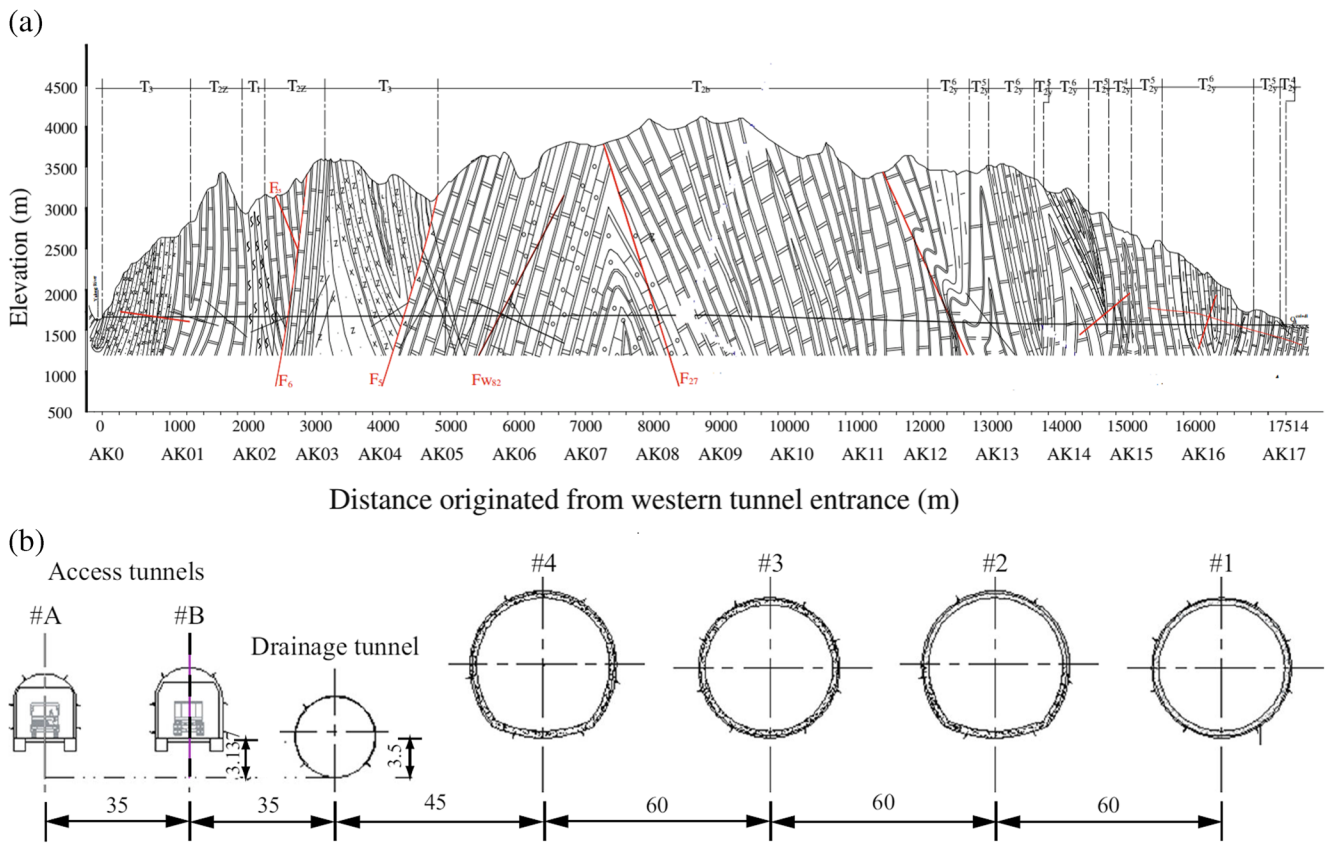


Fig. 1 (a) Cross section and (b) layout of tunnels in the Jinping Hydropower Station (reproduced from Li et al. 2012a, b; Zhang et al. 2012b; Xu et al. 2011)

provide a reliable basis to conduct a comparison of rock classification in TBM and drilled-blasted tunnels.

Headrace tunnels #1 and #3, 13 m in diameter with circular cross sections, were excavated using a TBM, while headrace tunnels #2 and #4, 13 m in diameter with horseshoe cross sections, were excavated using the drilling and blasting method. The axis spacing between the four tunnels is 60 m. Two access tunnels, #A and #B, were constructed parallel to the headrace tunnels.

Figure 2 is an image of the TBM manufactured by Robinson and Herrenknecht AG; the supporting window 5.5-m zone in the back of the cutting tools. The tunnel excavated using the drilling-blasting method is shown in Fig. 3, in which the exposed joints are visible. An image of the tunnel excavated using the TBM method is shown in Fig. 4; the tunnel is smooth and no joints are clearly exposed. Obtaining a large number of accurate structural data for this tunnel is challenging and classifying rock mass quality based on traditional geological data is difficult.

Database and data collection

There are two approaches to collecting field data for TBM performance prediction. One is to select the average values of the target parameters in a tunnel section with relatively long length, and the other is to choose the instantaneous

values of the target parameters in a tunnel section with relatively shorter length. This study uses the latter. Geological work is an important part of construction engineering and needs to be conducted simultaneously with tunneling using TBMs. The main geological work that needs to be undertaken throughout the entire construction process include advance geological forecasting and evaluation of the surrounding rock quality. We have been tracking the excavation of the tunnel for 3 years; the data were collected per 2 m of dig to predict the quality of the subsequent rock mass, and if the rock mass



Fig. 2 An image of the tunnel boring machine manufactured by Robinson and Herrenknecht AG



Fig. 3 An image of a clearly observable joint in the tunnel excavated using the drilling-blasting method

classification changed significantly, then reinforcement work began on the next geological engineering unit automatically.

During the tunneling process, we collected mainly the following data in the field and then determined the rock mass quality according to the following data.

(1) Description of lithology and structural plane

During tunneling, the conditions of the structural plane of the joint, including its length, width, filler, and shape, are described in detail. Geological mapping of the tunnel wall,



Fig. 4 An image showing that the wall is smooth in the tunnel excavated with the tunnel boring machine (TBM)

including logs of the conditions of the structural plane, was conducted during the daily maintenance of the machine.

In addition, the condition of structural plane includes mainly its dip and the angle between its strike and tunnel axis. The rating of the attitude of the structural plane on the roof and sidewall should be conducted separately for the underground projects with long spans and high sidewalls. Similarly, the attitude of the structural plane was recorded during geological mapping of the tunnel wall.

It is a well-known fact that the major structural plane, with its limited length, cannot fully represent the detailed characteristics of all existing structural planes in a geological engineering unit. Thus, the rock mass classification at different positions in the same geological engineering unit will be slightly different, and they should be subdivided to ensure the accuracy of the collected data. Therefore, the major structural plane was selected to determine the rock mass classification of the corresponding tunnel section.

(2) Scale of destruction of the surrounding rock

During the investigation, a 3D laser was used to measure the range and volume of the damage area; after this, their differences, based on their difference construction methods, were compared. There are four types of damage possible in the construction of a deep TBM tunnel: structural surface, stress, structural surface stress, and hang boot.

(3) Groundwater conditions

Four groundwater conditions were used as categories in this study: dryness, dripping or seepage, linear flow, and water inrush. The groundwater conditions were recorded simultaneously with the geological mapping of the tunnel wall.

(4) In situ stress

During the early stage of exploration, eight sets of stress tests were carried out. However, it is rarely possible to successively measure the in situ stress along a tunnel in the field. Therefore, the average overburden of each tunnel section was combined with the preliminary exploration result, and this value was adopted to characterize the in situ stress state of the tunnel.

(5) P-wave velocity

P-wave velocity is an important indicator of rock integrity. Before the tunnel excavation, the longitudinal wave velocity data can be collected for advance prediction, with the intactness index of the rock mass being used as a main quantitative index to characterize the degree of intactness of the fractured rock mass. The p-wave velocity in the rock mass can generally be measured using different acoustics tests including the single-hole, cross-hole, and hammering methods. Since drilling direction can significantly affect the measured p-wave velocity in the fractured rock mass, the cross-hole method was

Table 1 Number of structural planes in excavated tunnels documented from position AK1 + 300 to AK4 + 600

	Tunnel #1	Tunnel #2	Tunnel #3	Tunnel #4
Construction method	TBM	Drilling and blasting	TBM	Drilling and blasting
Number of documented structural planes	3212	5224	3230	5031

preferred in this study. To obtain the p-wave velocity of the representative area in each engineering geological unit along the tunnel axis, which would have the advantages of high resolution, no damage, and fast data processing time, geological defects need to be predicted within 30 m and the p-wave time profile, energy size of each reflector, and spatial distribution of the reflector in the detection range must be obtained.

The p-wave velocity in the fractured rock mass is, to some extent, slower when structural planes and filling materials exist. The initial and decreased p-wave velocity can reflect the physical and mechanical properties of either the intact rock block or the fractured rock mass, respectively. Thus, the rock mass intactness

index comprehensively indicates both the degree of development and the existing conditions of the structural planes.

Characteristic of the structural plan of the TBM tunnel

The number of structural planes is the key factor for classifying rock mass quality. In Table 1, the number of exposed structural planes in four diversion tunnels are provided for locations AK1 + 300–AK4 + 600. The numbers of structural planes exposed and documented in the #1 and #3 TBM tunnels are 65% of those documented in the #2 and #4 drilling and blasting tunnels because of the smoothing of the tunnel walls during the TBM excavation process. The TBM process makes only some of the structures observable at the surface and the data underestimate the true number of structures.

In addition, the combination of structural planes and mechanical properties has an impact on the stability of the surrounding rock, which is especially evident in the tunnel excavated using the drilling and blasting method. A long-term follow-up survey revealed that the TBM tunnels have significantly different structural surfaces compared to the drilling and blasting tunnels, due to the unique TBM drilling style (Table 2).

Table 2 Comparison of structural surface characteristics in tunnels excavated using different methods

Index	Tunnels excavated using the TBM method	Tunnels excavated using the drilling and blasting method
Structural surface shape	Slight disturbance of surrounding rock, and the original state of structure surface has been maintained. The structure surfaces in tunnels #1 and #3 are flat and smooth, a small quantity is undulated, and occasionally the shape is irregular	The disturbance of surrounding rock is strong, and the original state of the structure surface is not maintained. The morphology is frequently undulate, zigzag, stepped, and irregular in tunnels #2 and #4
Opening width	Shear joints are common, and most of the joints are closed and straight. Because of the special construction technology of TBMs, the opening width of the joint is small and only a small number of shear joints have openings of 0.1–1.0 cm, filled with rock fragments or mud. In addition, the tensile fracture belts have openings generally in the 10–50 cm range. The openings of the crushing fracture belts are relatively small, generally 10–30 cm. Construction with the TBM does not cause the deterioration of these adverse structural surfaces	The types of joints dominantly closed shear joints and rock layers. There are more tensile joints compared to the TBM tunnels. Shear joints are 0.1–0.5 cm wide, tensile joints are 0.5–2.0 cm wide, tensile fracture belts are 10–50 cm wide, and the opening widths of the crushing belts are relatively small, generally 10–30 cm. Due to the influence of blasting, the widths of the openings provide disadvantageous structure planes, which affects tunnel stability and causes damage
Extended length	The extended length is generally shorter, with fewer structural surfaces exposed due to the slight disturbance of surrounding rock. The structure plane is observably affected by rock powder, which was produced by the friction between the cutting head and surrounding rock	The extended length is generally longer and the failure of rock mass along the structure plane is strong. The structure planes are fully revealed and there is a certain extension on based from the original structural plane
Newly formed joint	The disturbance of the surrounding rock is slight, with few new fissures, observed around the existing joints	Under the influence of blasting, there are many new cracks; some were caused by stress relaxation and unloading when the tunnel was excavated
Combination of structural planes	There are fewer combination failures composed of structural planes; there are occasional wedge failures composed of shear joints and rocks	Combination failures composed of structural plane are common. Because the structural planes are fully exposed, combined structural plane damage may be due to adverse combinations of structural planes, potentially from damage to the rock mass during blasting

Table 3 Rock mass quality classification standard in the field

Rock mass grade	Stability evaluation of surrounding rock in the tunnel
I	The stability is good overall, the surrounding rock is stable for a long time, and there are no unstable rock blocks
II	The stability is generally good, the surrounding rock is basically stable, no plastic deformation occurs, but local composite block instability may occur
III	The stability of the local rock block is poor, the strength of surrounding rock is insufficient; plastic deformation may occur locally and may cause failure if there is no support. It is stable for a short time for intact soft rock
IV	Instability, the self-stabilization time of the surrounding rock is very short, and all types of deformation and failure can occur on a large scale
V	Extremely unstable, the surrounding rock cannot be self-stabilized, and deformation and failure are serious

In summary, the exposed features and microscopic features of the structural surfaces are quite different for two different construction technologies. Using documented structural plane data as factors to evaluate the stability of the surrounding rock in the TBM tunnel will generate large errors in the traditional Q-system.

New TBM classification prediction model based on the Q-system

Based on the documented differences between the tunnel surfaces, discontinuities in the structural planes would result in unreliable rock mass quality classifications for tunnels excavated using the TBM method. Therefore, it is necessary to modify the classification method using feasibly obtained

survey data. Abundant wave velocity data were collected during construction and can be applied to the classification of the rock mass integrity. Based on our long-term investigations, the introduction of the rock mass intactness index can improve the existing Q-evaluation system. Therefore, we integrate the rock mass integrity factor into the Q-rock quality classification system, improving the accuracy of surrounding rock evaluation and providing a reference for other predictions.

Rock mass classification in the field

The accuracy of the surrounding rock classification is closely related to the stability and support of the surrounding rock. In the field, we comprehensively determined the surrounding rock classification results based on the failure characteristics and the stability of the surrounding rock (Table 3); the accuracy of this method depends on the experience of the engineering geologist. The rock mass grades for tunnels #1 and #3, from AK14 + 200 to AK14 + 700 are shown in Tables 4 and 5, respectively. We have been tracking the tunnel excavation for 3 years; the excavation length was 2 m per dig, the data were collected to predict the quality of the subsequent rock mass, and the support measures were provided on time. In other words, there were 500 sets of data per 1 km and the length of the sample is 5 km; thus, there were 2500 sets of data in total. The area of the same rock mass can be summarized by the pile number.

Regression analysis of weight index of rock mass integrity

According to the statistical results of the structural planes in a typical tunnel (Table 1), the number and spacing of joints were not fully revealed in the TBM tunnels. In the Q-system, $RQD/$

Table 4 Rock mass quality grade for tunnel #1 excavated using TBM

No.	Position	Failure styles	Stability	Rock mass grade according to field criterion
1	14,700–14,686.5	Rock falls locally	Generally stable	II
2	14,686.5–14,635	Failure locally due to joint combination	Poor stability	IV
3	14,635–14,614	Slight rock burst	Generally stable	II
4	14,614–14,600	Failure locally due to joint combination	Poor stability	IV
5	14,600–14,560	Many blocks fall	Poor stability	IV
6	14,560–14,500	Failure locally due to joint combination	The stability of the local rock block is poor	III
7	14,500–14,480	Rock falls locally	The stability of the local rock block is poor	III
8	14,480–14,440	Rock falls locally	The stability of the local rock block is poor	III
9	14,440–14,415	Failure locally due to joint combination	Poor stability	IV
10	14,415–14,400	Rock falls locally	The stability of the local rock block is poor	III
11	14,400–14,360	Failure locally due to joint combination	The stability of the local rock block is poor	III
12	14,360–14,350	No apparent damage	Generally stable	II
13	14,350–14,315	Failure locally due to joint combination	Poor stability	IV
14	14,315–14,300	No apparent damage	Generally stable	II
15	14,300–14,233	Slight rock burst	Generally stable	III
16	14,233–14,200	Failure locally due to joint combination	The stability of the local rock block is poor	III

Table 5 Rock mass quality grade for tunnel #3 excavated using a TBM

No.	Position	Failure styles	Stability	Rock mass quality according to field criterion
1	14,700–14,600	Slight rock burst	The stability of the local rock block is poor	III
2	14,600–14,532	Failure locally due to joint combination	The stability of the local rock block is poor	III
3	14,532–14,517	Slight rock burst	Generally stable	II
4	14,517–14,500	Failure locally due to joint combination	The stability of the local rock block is poor	III
5	14,500–14,480	Failure locally due to joint combination	Poor stability	IV
6	14,480–14,400	Failure locally due to joint combination	The stability of the local rock block is poor	III
7	14,400–14,335	Failure locally due to joint combination	The stability of the local rock block is poor	III
8	14,335–14,300	Failure locally due to joint combination	The stability of the local rock block is poor	III
9	14,300–14,245	Failure locally due to joint combination	The stability of the local rock block is poor	III
10	14,245–14,200	Failure locally due to joint combination	The stability of the local rock block is poor	III

Jn represents rock integrity. However, collecting data and making the subsequent calculations are challenging in the TBM tunnels and that leads to prediction errors in rock mass quality classification using the Q-system. A potential solution to this problem is to use R_{Kv} , based on the wave velocity test, for replacing the RQD/Jn term in the Q-system, and accurately determine the integrity of the rock. In this replacement, the wave velocity data from the construction survey is applied to document the joint features that were not observable in the TBM tunnel.

The intactness index of the rock mass Kv is a parameter similar to RQD/Jn , which can also reflect the degree of rock integrity. Its value can be obtained from the wave velocity test and is easy to assess qualitatively. The Kv of the rock mass is divided into five grades according to the integrity of the rock mass (GBT 50218-2014). The key to applying the proposed Q-system is to properly evaluate the Kv grade.

Based on the Standard for Engineering Classification of Rock Mass (GBT 50218-2014), which describes *Jn* in the Q-system, we can obtain the parameters and grade values of rock mass integrity, RQD , and structural surface features *Jn*. Then, the value of RQD/Jn can be calculated (Table 6).

Based on the rock mass integrity, the RQD/Jn value corresponding to the rock integrity index Kv can be obtained. The

relationship between R_{Kv} and rock mass intactness index is shown in Table 7 and Fig. 5.

Using Statistical Product and Service Solutions (SPSS) software, the formula of R_{Kv} and the intactness index of the rock mass can be expressed as in Eqs. 2–3. In addition, we set up the equation separately so that the R_{Kv} value can be calculated accurately.

$$R_{Kv} = 0.5477e^{1.0086Kv} \quad R^2 = 0.9989 \quad (0 < kv \leq 0.75) \quad (2)$$

$$R_{Kv} = 170 kv - 140 \quad R^2 = 1.0 \quad (0.75 < kv \leq 1.0) \quad (3)$$

The integrity index of the rock mass Kv is the square of the ratio of the compressional wave velocity between the rock mass and the rock, and can be determined using the following dynamic method:

$$kv = \left(\frac{V_{pm}}{V_{pr}} \right)^2 \quad (4)$$

where V_{pm} is the wave velocity of the rock mass and V_{pr} is the wave velocity of the intact rock.

Table 6 Relationship between rock mass integrity, RQD , and *Jn*

Integrity of rock mass	Rock mass structure	RQD		Number of structural planes (<i>Jn</i>)		RQD/Jn	
		RQD (%)	Score	Number of structural planes *	Score *		
Integral	Massive or giant-thick layer	90–100	90–100	Without joints or a set of joints	0.5–3	200–30	
Relatively integral	Block or thick layer	75–90	75–90	Two sets of joints	3–6	30–12	
Poorly integral	Mosaic or thin layer or blocky-fractured	50–75	50–75	Three sets of joints	6–12	12–4	
Relatively crushed	Blocky-fractured or cataclastic structure	25–50	25–50	Four sets of joints	12–15	4–1.5	
Crushed	Lumpy or fragmental	0–25	0–25	More than four groups of joints	15–20	1.5–0.5	

Note: * based on the Q-system

Table 7 Relationship between rock mass integrity and RQD/J_n and K_v

Rock mass integrity	Integrity index of rock mass K_v	RQD/J_n
Integral	0.75–1	200–30
Relatively integral	0.55–0.75	30–12
Poorly integral	0.35–0.55	12–4
Relatively crushed	0.15–0.35	4–1.5
Crushed	0–0.15	1.5–0.5

Modified Q-system model for the TBM tunnel

The modified Q_T model for TBM tunnels is described as follows:

$$Q_T = R_{KV} \times \left(\frac{J_r}{J_a}\right) \times \left(\frac{J_w}{SRF}\right) \tag{5}$$

where R_{KV} is the reduction coefficient of the rock mass, which represents the rock mass integrity; J_r is the rating for joint surface roughness; J_a is the rating for joint alteration; J_w is the water condition of the joint; and SRF is the rating for the rock mass in situ stress. Therefore, a site engineer can calculate the Q_T value using the corresponding parameters.

Results

Geological conditions of the verified tunnel

In this study, the rock mass quality classifications were calculated for sections #1 (K15 + 790–K9 + 931) and #3 (AK15 + 900–AK9 + 700) of the diversion tunnel using the new Q_T model.

There is a series of folds developed in the tunnel area. The lithology is alternately repeated with T_{2Y^4} , T_{2Y^5} , and T_{2Y^6} on the east side of the tunnel; the lithology is marble and

Table 8 Mechanical parameters of rock tunnel

Lithology	Compressive strength (MPa)	
	Natural strength	Saturable strength
Medium fine-grained sandstone of medium thickness (T_3)	104–152	71–114
Zagunao group marble (T_{2Z})	70–90	55–78
Medium-thick marble (T_{2b})	90–100	75–85
Striped mica marble (T_{2Y^4})	85–90	55–62
Medium-thick marble (T_{2Y^5})	70–95	65–85
Argillaceous limestone (T_{2Y^6})	70–75	60–70

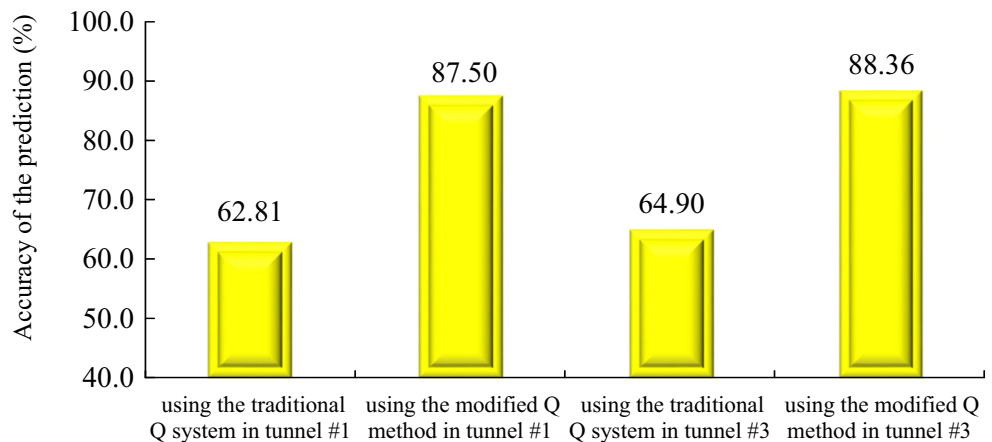
crystalline limestone (T_{2b}) in the middle of the tunnel; and T_{2Z} and T_3 on the west side of the tunnel. The tunnel zone traverses the Triassic middle and upper marble, limestone, crystalline limestone and sandstone, and slate. Based on the experimental results, the rock mechanics parameters of the tunnel area are presented in Table 8.

There are four groups of fissures developed in the area: (1) $N5-30^\circ W, SW$, the joints are dense, the structure surface is smooth and parallel to the structure; (2) $N60-80^\circ W, SW \angle 10-25^\circ$, the dip angle is larger, the structural plane is wavy, and the elongation length is larger; (3) $N0-30^\circ E, SE$, the joints are parallel to the rock layer and mostly closed; and (4) $N30-60^\circ E, SE \angle 10-35^\circ$, the dip angle is gentle, mostly open, the structural plane is undulating, and the extension is longer.

According to the stress test, the maximum principal stress at 1600 m in the tunnel line is 70.1 MPa and the minimum principal stress is 30.1 MPa. After the excavation of the whole section, the horizontal stress is larger at the arch of the tunnel at 54 MPa and the vertical stress component is larger on both sides of the arch at 120 MPa.

Most of the rock of the Jinping Tunnel has a thick layer and block structure, and the joint distance is greater than 0.4 m. In

Fig. 5 Comparisons of the rock mass quality prediction accuracy in TBM tunnels



Results of classifications using different methods in TBM tunnels (#1 and #3)

Table 9 Rock mass quality classification results for the #1 TBM tunnel

Grade of rock mass		II	III	IV
Results of site classification	Length (m)	1211	4437	190
	Proportion (%)	20.3	74.5	3.2
Results of Q_T classification	Length (m)	1195	4506	259
	Proportion (%)	20	75.7	4.3
Coincidence rate	(%)	87.50		

In addition, the tunnel includes whole and mosaic structures, thin layers, and fractured rock masses. According to the field investigations, the rock mass structure of the tunnel has a great influence on the deformation and classification of the surrounding rock. The destruction of the structural surface type occurs mainly in fractured and inlaid structures, and stress-type fractures develop in the whole structure and block structures.

Results of the modified Q-classification method

The prediction results of the rock mass classification for tunnels #1 and #3 are presented in Tables 9 and 10, respectively. The proposed Q_T system is more suitable for predicting the rock mass quality in the TBM tunnels. The prediction accuracies in the validation sections for tunnels #1 and #3 are 87.50 and 88.36%, respectively. These results meet the rock mass quality classification requirements in the TBM tunnels, indicating that the Q_T system can provide reliable prediction results rapidly and ensure efficient and safe tunnel construction.

Discussion

Reliability validation must be performed for every new model as it is very important for the accuracy of predictions. Therefore, part of the TBM tunnel (#1 and #3) is selected for a validation analysis of the modified Q-system. In the study, K1 + 100 to K3 + 600 are typical of the sections in these TBM tunnels. The rock mass quality of these sections was

Table 10 Rock mass quality classification results for the #3 TBM tunnel

Grade of rock mass		II	III	IV
Results of site classification	Length (m)	1772	4211	200
	Proportion (%)	28.66	68.1	3.24
Results of Q_T classification	Length (m)	1631	4416	136
	Proportion (%)	26.37	71.43	2.2
Coincidence rate	(%)	88.36		

calculated using the traditional and modified Q-systems for tunnels, the results were compared with the field classification results, and the differences are shown in Fig. 5.

Figure 5 clearly shows that the prediction accuracy of the traditional Q-system is 62.87% and 64.90% in tunnels #1 and #3, respectively; these values are low (< 70%). After modification, the prediction accuracies become 87.50% and 88.36%, respectively, which are >85%. The traditional Q-system is not

**Fig. 6** Type of damage to the surrounding rock in a TBM tunnel

as accurate in a TBM tunnel because accurate structural plane numbers are not observed in these tunnels, and the degree of influence of the combined structural surface is weakened. The large errors in the traditional Q-system in TBM tunnels are generated from inaccurate structural plane numbers, which have a higher score than the actual classification. The worst result of the higher scoring is that the builder is led to assume that rock mass quality is good and therefore low reinforcement measures can be adopted; this can be very dangerous for engineering and construction personnel. The revised model is clearly more consistent with geological conditions, thus greatly reducing the project risk.

In terms of the applicability of the new model, it will be applied mainly in rigid rock tunnels excavated by TBMs. Based on the p-wave and geological data, the rock mass quality in front of the excavating face can be predicted. Applying the modified Q-model could help improve the reliability and flexibility of supporting measures, because there are 5.5-m-long open sections between the excavating face and the shield in the TBM's tunneling process. If the quality of the surrounding rock cannot be evaluated, support measures cannot be put up in time, predictions are inaccurate, and support strength is insufficient, such that serious problems could occur.

In addition, it is generally believed that the tunnel wall excavated by the TBM is intact and generally stable; however, there was damage controlled by the structural surface and this has significant impacts on the stability of the surrounding rock. There are four types of damage possible in the construction of a deeply buried tunnel built using a TBM tunnel (Fig. 6): structural surface type, stress type, structural surface stress type, and hang boot type. Through the field tracking survey, we founded that hang boot damage is due to the repeated loading of the boot on the surrounding rock. During tunneling, the hang boot is in direct contact with the surrounding rock, and as the tunnel wall does not have any anchors or other support measures, this aggravates the damage to the surrounding rock.

Figure 6a shows stress-type damage in a tunnel excavated using a TBM. Its destructive characteristics are that the edge of the fallen block came from the intact rock, and the rupture is due to high stress. These deformations are very common under high-stress conditions, occur not far from the tunnel face, and can easily cause a second collapsed if the rupture rang intensified; the deformation time before the failure is shorter. This type of failure occurs mainly in the tunnel section where rock structure is intact.

Structural surface/stress-type damage (Fig. 6b). Its destructive characteristics are that the edge of a fallen block forms part of the structural plane and the intact rock. The destruction of rock mass in the high-stress zone has great destructive power due to the energy released. The size of the rupture that induces this type of damage is generally small and basically rigid. Common faults include shear bands of a single fracture or a combination of several joints. Large-scale faults, especially those with soft fillings, do not induce damage of the structural surface stress type.

Structural surface-type damage, such as that in Fig. 6c, occurs when the edge of the fallen block is controlled by two or more structural planes that collapsed under the action of gravity. It occurs mainly in tunnel sections where the structural surface is relatively well-developed.

Hang boot damage (Fig. 6d) is a unique form of destruction in tunnels excavated using TBMs. The damage is mainly on the wall of the supporting boots. The main reason for the rock failure is the relaxation of the surrounding rock after the boot has been unloaded.

In the process of excavating the diversion tunnel, the study team cooperated closely with the construction schedule and adopted laser measurements to measure secondary damage after excavation. The investigation showed that the scale of the damage is different when different construction methods are used in a section with the same geological engineering conditions. Figure 7 compares the length of the damage between the two different methods.

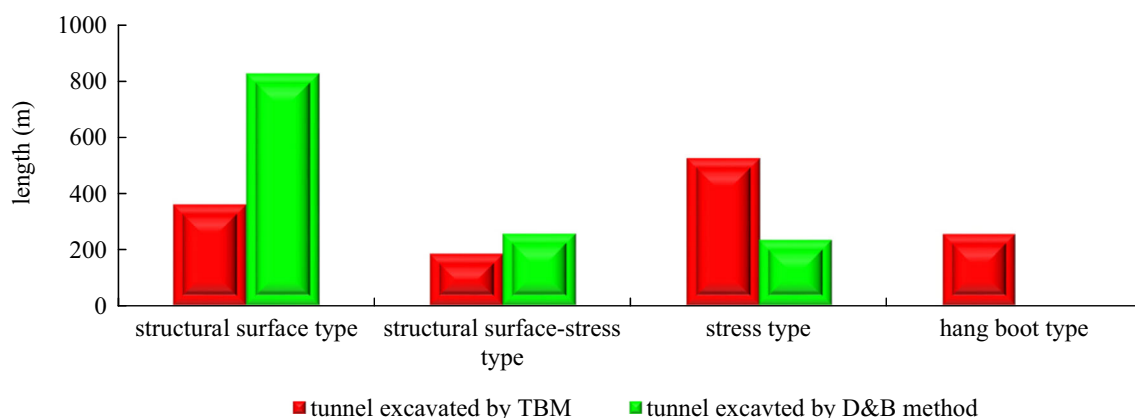


Fig. 7 Comparison of the damage in the rock surrounding tunnels constructed using different methods

Figure 7 shows that the length of the structural surface and structural/stress-type damage in TBM tunnels is shorter than those from the drilling and blasting methods. However, the length of the stress type and hang boot type damage is larger in the TBM tunnel than in the drilling and blasting tunnels.

Conclusions

In tunnels excavated using the TBM method, observations of the structural planes are significantly different than those in tunnels excavated using the drilling and blasting method. Based on the wave velocity test, R_{Kv} replaces the RQD/Jn term in the Q-system and accurately predicts the integrity of the rock mass. Based on multiple regression analysis of the R_{Kv} value, a new application for the Q-system was developed to evaluate the rock mass quality in TBM tunnels. The results from the case study for the Jinping II Hydropower Station can be summarized as follows:

In the tunnel excavated using the TBM method, the tunnel walls were smooth and the surrounding integrity was high. The number of the exposed structural planes was 65% of that in the tunnel excavated using the drilling and blasting method. Moreover, the extension length and the opening width of the structure planes were significantly different from those of the tunnels excavated using the drilling and blasting method.

The RQD/Ja index that represents the rock mass integrity in the traditional Q-system is not easily measured in TBM tunnels and can be replaced by an integrity index Kv obtained using the wave velocity measurements. The modified classification method achieves a prediction accuracy of more than 85%, compared to the accuracy of 67% achieved by the traditional system.

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