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*Full Length Research Paper*

# Geotechnical and geological studies of NWCT tunnel in Iran focusing on the stabilization analysis and design of support: A case study

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**In this paper, a detailed geomechanical investigation of rock masses of North Water Convey Tunnel (NWCT) and its stability analysis has been carried out. The NWCT is located in the north of Iran and is to be constructed in-order to convey water for agriculture purposes. The main instability in the tunnel is joints and faults. The rocks mass encountered in the tunnel route are made of argillaceous, sandstone and shale. The tunnel has been divided into two parts, lot1 and lot2 having a length of 14 km and 26 km respectively. It is proposed to be constructed by telescopic shield method using a tunnel boring machine (TBM). In this study, the most suitable methods are utilized for the stability analysis and design of support of the tunnel. For the empirical investigation, the rock mass were classified based on RMR, Q, RSR, GSI and Rmi systems. The geomechanical properties of the rock mass were determined from the laboratory and field investigations. The results obtained from the analysis show that the tunnel is highly unstable due to the presence of a fault and hence strong supports are need in these regions. The support system used is concrete lining, as the tunnel in used for water conveyance. The tunnel alignment in lot1 is divided into 12 lithology types as; LI-SH1, LI-SH2, LI-SH3, LI-SH4, LI1, LI2, LI3, LI4, LI5, SI, CZ and FZ regions. Similarly, the tunnel alignment in lot2 is divided into 21 lithology types as; SH-ML1, SH-ML2, SH-ML3, MLI-SH1, ML-SH2, ML-SH3, ML-SH4, ML-SH5, SH-LS1, SH-LS2, SH-LS3, SH-LS4, LI2, LI3, LI4, LI5, LI6, LI-MA, LI-SH, CZ, FZ regions. A stability analysis is a necessity as during the tunneling instabilities, such as the presence of a shear zones, may cause an obstruction and delaying of TBM progressing rate.**

**Key words:** Tunnel boring, stability analysis, rock mass classification system, empirical and numerical methods, support pressure.

## INTRODUCTION

The NWCT tunnel, located in north of Iran, provides a part of water requirement to the north Iran tropical plains (Figure 1). The inlet and outlet portals of the tunnel are 635 m and 625 m higher respectively than the free water level and the maximum overburden point of this tunnel is 800 m. This paper presents the geological study of the complete tunnel path, including lot1 and lot2. The total length of the tunnel is 40 km; 14 km in lot1 and 26 km in lot2. The dip along the length of the tunnel is about 8/1000.

These experimental methods adopted in this study are based on the results of different thicknesses of the strata obtained by the boreholes used for the analyses of stability of the tunnel.

This study reflects the findings of the 40 km long tunnel area constructed by the TBM. The geometry of the tunnel shape is circular and the parameters of tunnel properties have been shown in Table 1. The geological study included the field and laboratories investigations and based on the results, the tunnel alignment of lot1 and lot

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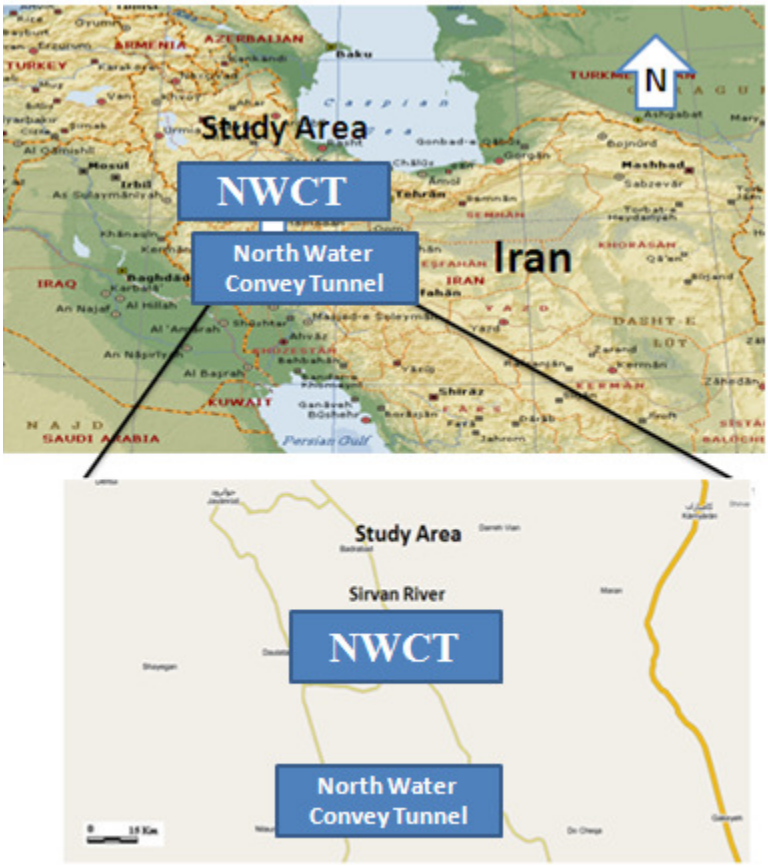



**Figure 1.** Location of studied area in Iran.

**Table 1.** General parameters of the tunnel path.

| Parameters | | Properties |
|---|---|---|
| Length of tunnel path | Lot 1 | 14 km |
| | Lot 2 | 26 km |
| Inlet tunnel free water surface | | 625 m |
| Outlet tunnel free water surface | | 635 m |
| Radius of tunnel boring operation | | 2.3 m |
| Maximum overburden | | 800 m |
| Dip | | 8/1000 |

2 was divided into 12 and 21 lithology types respectively.

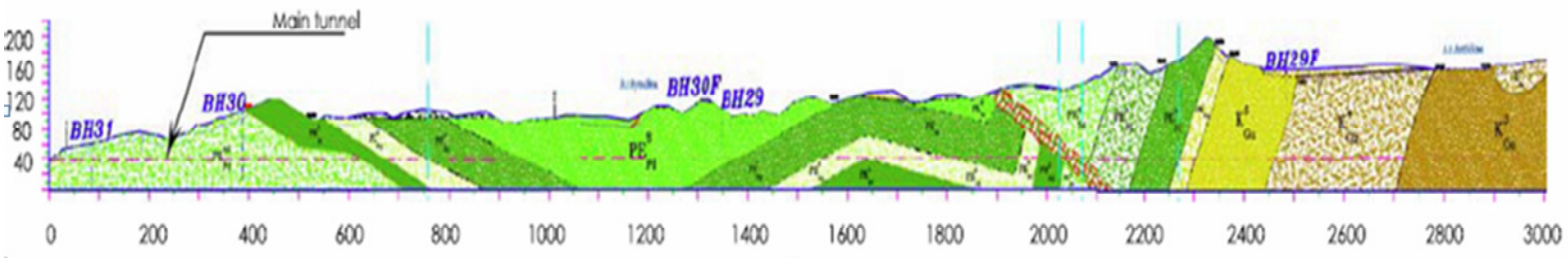
## Geological conditions

Based on the results of the samples and boreholes carried out, the transfer tunnel path passes through argillaceous sandstone and shale type of soil/rock as shown in (Figure 2).

Some stability problems were predicted at some locations along the alignment and hence a more detailed exploration was carried out. These locations are as follows:

1. Tunnel entrance.
2. Distance from some parts of the tunnel alignment to ground level may be low because at these places, the rocks are weak.
3. Depth of weathered rocks is high.
4. Shear regions and comminuted, and
5. Aquifer horizons.

Based on the results of studies made in the engineering geological zone, along the total route of the tunnel, regardless of surface sediments or hypothesis, the lithology of the rock/soil has been identified to include debris such as argillaceous and shale, sandstone and shale. The investigation of the types of lithology in tunnel path of lot 1 and lot2 was carried out. According to the field and visual investigation, including geological and geotechnical investigation (borehole, core logging and laboratory testing), the rock/soil mass has been identified to consist of 12 lithography types in lot 1 and 21 lithography types in lot2. The types of lithology identified are shown in Tables 2 and 3. The boundaries of types of lithography are according to the stratigraphy and in many cases for the geomechanical features, the lithography




| Geology | SH-ML1, SH-ML2 | ML-SH1 | ML-SH2 | ML-SH3 | ML-SH2 | SH-ML3 | ML-SH2 | ML-SH1 | ML-SH4 | ML-SH5 | SH-LS1 | SH-LS2 | SH-LS3 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Lithology | Argillaceous and shale | | | | | | | | | | Argillaceous and shale, sandstone and shale | | |
| Structure | Layered, jointed, folded | | Layered | Layered, jointed, folded | Layered | | | | Fractured | | Jointed, folded | | Layered, , folded |
| Average permeability (m/sec) | $15e^{-8}$ | | $2.5e^{-8}$ | | | | | | $15e^{-8}$ | | $1e^{-7}$ | | |

**Figure 2.** Longitudinal geological profiles of the excavated sections of NWCT (lot2).

was the main factor in separation and classification.

The category classification of massif regional characteristics in geomechanical features is illustrated in Table 4.

As is evident from Table 2, the rock mass, along the tunnel path in lot1, varies from very weak, thinly bedded, crushed and unstable to moderately strong, thick bedding and stable. Similarly, the rock mass along the tunnel path in lot2 (Table 3) varies from very weak, thin bedding, crushed and unstable to weak to moderately strong, crushed, medium bedding and unstable.

Also, based on the geomechanical features, the rock mass in lot1 consists of poor and crushed to very strong, massive with an average distance between discontinuities of significantly more than half a meter. Similarly, the geomechanical features in lot2 can be described as poor and crushed to semi-solid, medium to thick layers and the average distance between the discontinuities to be significantly less than half a meter.

A large number of tests were performed for geotechnical classification of the rock/soil mass. The characteristics included, rock strength, density, uniaxial tensile strength, shear strength parameters, Young's modulus and hardness to abrasion. The determination of the properties requires accurate sampling of all the samples of the rocks and perform laboratory tests on samples of the above mentioned and identify the basic lithology along the tunnel path. The physical and mechanical properties of the rock units along tunnel alignment in lot1 and lot 2 were evaluated and are presented in Tables 5 and 6 respectively.

## Engineering classification of rock mass

A number of methods are available to classify a rock mass based on a variety of parameters (Cai et al., 2007). Apuani et al. (2005) have also tried to find the physical and mechanical properties of rock masses at Stromboli. A specific upscaling theory of rock mass parameters exhibiting spatial

**Table 2.** Lithology of rock mass along tunnel (lot1).

| Geology | Stability state | |
|---|---|---|
| | Description | Type |
| LI-SH1 | Weak to moderately strong, crushed, moderately bedding, unstable | B |
| LI-SH2 | Moderately weak, thin bedding, crushed almost unstable | C |
| LI-SH3 | Moderately weak, thin bedding, crushed almost unstable | C |
| LI-SH4 | Weak to moderately strong, crushed, moderately bedding, unstable | B |
| LI1 | Moderately strong, thick bedding, little crushing, stable | A |
| LI2 | Moderately strong, thick bedding, little crushing, stable | A |
| LI3 | Moderately strong, thick bedding, little crushing, stable | A |
| LI4 | Moderately strong, thick bedding, little crushing, stable | A |
| LI5 | Weak to moderately strong, crushed, bedding medium, unstable | B |
| SI | Moderately weak, thin bedding, crushed almost unstable | C |
| CZ | Very weak, thin bedding, crushed and unstable | D |
| FZ | Moderately weak, thin bedding, crushed almost unstable | C |

**Table 3.** Property of rock mass in different units (lot 2).

| Geology | Stability State | |
|---|---|---|
| | Description | Type |
| SH-ML1 | Moderately weak, thin bedding, crushed almost unstable | C |
| SH-ML2 | Very weak, thin bedding, crushed and unstable | D |
| SH-ML3 | Very weak, thin bedding, crushed and unstable | D |
| MLI-SH1 | Weak to moderately strong, crushed, bedding medium, unstable | B |
| ML-SH2 | Moderately weak, thin bedding, crushed almost unstable | C |
| ML-SH3 | Moderately weak, thin bedding, crushed almost unstable | C |
| ML-SH4 | Moderately weak, thin bedding, crushed almost unstable | C |
| ML-SH5 | Weak to moderately strong, crushed, bedding medium, unstable | B |
| SH-LS1 | Very weak, thin bedding, crushed and unstable | D |
| SH-LS2 | Moderately weak, thin bedding, crushed almost unstable | C |
| SH-LS3 | Relatively weak, thin bedding, crushed almost unstable | C |
| SH-LS4 | Very weak, thin bedding, crushed and unstable | D |
| LI2 | Very weak, thin bedding, crushed and unstable | D |
| LI3 | Weak to moderately strong, crushed, bedding medium, unstable | B |
| LI4 | Moderately weak, thin bedding, crushed almost unstable | C |
| LI5 | Weak to moderately strong, crushed, bedding medium, unstable | B |
| LI6 | Weak to moderately strong, crushed, bedding medium, unstable | B |
| LI-MA | Moderately weak, thin bedding, crushed almost unstable | C |
| LI-SH | Moderately weak, thin bedding, crushed almost unstable | C |
| CZ | Very weak, thin bedding, crushed and unstable | D |
| FZ | Very weak, thin bedding, crushed and unstable | D |

**Table 4.** Category classification of massif regional characteristics and geomechanical features.

| Type | Geomechanical features |
|---|---|
| A | Very strong, massive, the average distance between discontinuities being significantly more than half a meter |
| B | Semi-solid to solid, medium to thick layers, the average distance between discontinuity being significantly less than half a meter |
| C | Semi-solid to weak, thin to medium layer, the average distance of discontinuity significantly less than 0.2 m |
| D | Poor, crushed |

**Table 5.** Physical and geotechnical properties of the rock along tunnel alignment (lot 1).

| Geology | Uniaxial compressive strength (MPa) | Tensile Strength (MPa) | Modulus of deformation (GPa) | Saturated density (g/cm$^3$) | Porosity (%) | Weathering in surfaces |
|---|---|---|---|---|---|---|
| LI-SH1 | 5-25(20)* 100-150(125)** | 1-2.5*,10-15** | 2-5*,15-25** | 2.5-2.7 | 5-15 | Moderately weathered |
| LI-SH2 | 50-100(75) | 5-10 | 5-25 | 2.35-2.85 | 5-10 | Highly weathered |
| LI-SH3 | 5-25(20)* 50-100(75)** | 2.5-10 | 2-6 | 2.3-2.7 | 1-5 | Moderately weathered |
| LI-SH4 | 5-25(20)* 50-100(75)** | 2.5-10 | 2-6*,20-50** | 2.4-2.65 | 2-6 | Slightly weathered |
| LI1 | 5-25(20)* 50-100(75)** | 1-2.5,5-10 | 15-25 | 2.5-2.7 | 5-15 | Slightly weathered |
| LI2 | 100-150(125) | 2.5-10 | 4-8,15-25 | 2.35-2.85 | 0.5-5.5 | Moderately weathered |
| LO3 | 100-150(125) | 5-15 | 5-15 | 2.65-2.85 | 0.5-2 | Moderately weathered |
| LI4 | 100-150(125) | 5-10 | 15-25 | 2.5-2.7 | 0.5-2 | Slightly weathered |
| LI5 | 50-100(75) | 5-15 | 5-10*,40-70** | 2.45-2.75 | 1-8 | Slightly weathered |
| SI1 | 25-50(35) | 0.5-1 | 2-5 | 2.6-2.7 | 1-2 | Moderately weathered |

*Shale; **lime

**Table 6.** Physical and geotechnical properties of the rock along tunnel alignment (lot 2).

| Geology | Uniaxial compressive strength (MPa) | Tensile strength (MPa) | Modulus of deformation (GPa) | Dry density (g/cm$^3$) | Porosity (%) | Weathering in surfaces |
|---|---|---|---|---|---|---|
| SH-ML1 | 25-50(35) | 1-3 | 4-6 | 2.4-2.5 | 10-15 | Moderately weathered |
| SH-ML2 | 25-50(35) | 1-3 | 4-6 | 2.3-2.5 | 10-15 | Highly weathered |
| SH-ML3 | 25-50(35) | 1-3 | 5-6.5 | 2.3-2.5 | 5-15 | Moderately weathered |
| ML-SH1 | 50-100(75) | 1-3 | 4-6.5 | 2.5-2.6 | 2-5 | Slightly weathered |
| ML-SH2 | 25-50(35) | 1-3 | 4-6.5 | 2.2-2.5 | 5-10 | Slightly weathered |
| ML-SH3 | 25-50(35) | 2-4 | 5-6 | 2.05-2.5 | 5-15 | Moderately weathered |
| ML-SH4 | 25-50(35) | 2-4 | 4-6 | 2.2-2.5 | 5-10 | Moderately weathered |
| ML-SH5 | 50-100(75) | 5-10 | 5.5-7 | 2.3-2.6 | 3-5 | Slightly weathered |
| SH-LS1 | 5-25(20) | 1-3 | 4-6 | 2.4-2.5 | 3-15 | Slightly weathered |
| SH-LS2 | 25-50(35) | 2-4 | 5-6 | 2.3-2.6 | 3-10 | Moderately weathered |
| SH-LS3 | 5-25(20) | 1-3 | 4-6 | 2.3-2.5 | 5-15 | Moderately weathered |
| SH-LS4 | 25-50(35) | 1-2,2-5 | 4-6,2-4 | 2.3-2.6 | 5-10 | Moderately weathered |
| LI2 | 100-150(125) | 2.5-6 | 15-30 | 2.5-2.6 | 2-5 | Slightly weathered |
| LI3 | 100-150(125) | 5-10 | 15-30 | 2.5-2.6 | 2-5 | Slightly weathered |
| LI4 | 50-100(75) | 2.5-6 | 5-10 | 2.5-2.6 | 2-5 | Slightly weathered |
| LI5 | 100-150(125) | 2.5-6 | 5-10 | 1.9-2.7 | 5-15 | Slightly weathered |
| LI6 | 100-150(125) | 5-10 | 15-30 | 2.5-2.5 | 2-5 | Slightly weathered |
| LI-MA | 25-50(35) | 1-3 | 2-5 | 2.2-2.6 | 2-5 | Moderately weathered |
| LI-SH | 5-50(35) | 1-3 | 2-5 | 2.2-2.6 | 2-5 | Moderately weathered |
| CZ | 5-150 | 5-10 | 5-7 | 2.5-2.6 | 2-5 | Highly weathered |
| FZ | 5-150 | 1-3 | 4-6 | 2.3-2.5 | 5-15 | Highly weathered |

variability was presented by Exadaktylos and Stavropoulou (2008). In this study, the rock mass classification were performed according to Rock Mass Rating (RMR), RMR modified, Geological Strength Index (GSI), Rock Structure Rating (RSR), Quality system (Q), and Rock Mass index (RMi) systems for the transfer tunnel and the properties of rock mass were determined by using these systems. In order to apply the rock mass

**Table 7.** Classification of rock mass along tunnel path based on suggested systems (lot1).

| Geology | RQD (%) | RMR | Modified RMR | Q | GSI | RMi | RSR |
|---|---|---|---|---|---|---|---|
| LI-SH1 | 75-90 | 36-56 | 26-46 | 1.9-5 | 25-30,50-56 | 1.05-4.3 | |
| LI-SH2 | 50-75 | 40-45 | 38-43 | 2-2.5 | 35-40 | 1.5-2.0 | |
| LI-SH3 | 50-90 | 37-45 | 35-42 | 1.5-2.5 | 30-50 | 2.5-3 | |
| LI-SH4 | 50-75 | 45-50 | 43-48 | 1-4.17 | 35-50 | 1.26-2.47 | |
| LI1 | 50-75 | 34-47 | 32-45 | 1.27-2.75 | 25-50 | 0.9-2.6 | |
| LI2 | 50-75 | 50-52 | 48-50 | 2-2.75 | 45-55 | 3-4 | |
| LI3 | 50-75 | 50-52 | 48-50 | 2.5-3 | 45-50 | 3-3.5 | |
| LI4 | 75-90 | 50-56 | 48-54 | 4-5 | 48-53 | 4-5 | |
| LI5 | 75-85 | 45-53 | 43-51 | 1.35-2.2 | 45-50 | 2-2.5 | |
| SI | 25-50 | 30-36 | 25-31 | 0.5-1 | 30-40 | 0.5-1 | |
| FZ | 25-50 | 36-38 | 34-36 | 0.5-1 | 25-35 | 0.5-1 | |
| CZ | 0-50 | 25-38 | 23-36 | 0.3-0.5 | 18-35 | 0.5 | |

classification systems, the tunnel alignment were divided into nine different zones; each of which has its own independent engineering properties. The RMR system was initially developed by Bieniawski (1974) on the basis of his experiences in shallow tunnels and was modified further (Bieniawsky, 1989). The studied parameters are rock quality designation (RQD), uniaxial compressive strength (UCS), discontinuity spacing, discontinuity condition, groundwater condition and discontinuity orientation. RQD has been used by Palmstrom (2005) for the measurements of and correlations between block size and rock quality designation.

The GSI is a new rock mass classification system that was developed by Hoek (1994) and Hoek et al. (1995). This classification system is based on the structure of the rock mass and does not suggest a direct correlation between rock mass quality and GSI. Therefore, it can be estimated based on the correlation between GSI and RMR values.

The Q-system was developed as a rock tunneling quality index by Norwegian Geotechnical Institute (NGI) (Barton et al., 1974) and the last improved version was released in 2004. The Q-value can be calculated as follows:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (1)$$

This classification system includes six parameters of rock quality as following:

1. Rock quality designation (RQD)
2. The number of joint sets ( $J_n$ )
3. The joint surface roughness ( $J_r$ )
4. The degree of joint weathering and alteration ( $J_a$ )
5. Joint water reduction factor ( $J_w$ )
6. Stress reduction factor (SRF)

The three quotients in Equation (1) may be taken to represent the block size, the inter-block frictional shear strength and the active stress respectively.

Goel et al. (1995) suggested the parameter $Q_n$, for stress free form Q. In order to calculate $Q_n$, SRF is taken 1, which has been given in Equation (2):

$$Q_n = (\frac{RQD}{J_n} \times \frac{J_r}{J_a}) \times J_w \quad (2)$$

$$Q_c = Q\frac{\sigma_c}{100} \quad (3)$$

Where, $Q_c$ is uniaxial compressive strength (MPa) of the intact rock mass. These classification systems are used to estimate the rock mass parameters along the tunnel alignment and the results have been presented in Tables 7 and 8 for lot1 and lot2 respectively.

## Determination of rock mass strength parameters

### The generalized Hoek-Brown criterion

The generalized Hoek-Brown failure criterion for jointed rock masses is defined by:

$$\sigma_1^{'} = \sigma_3^{'} + \sigma_{ci}\left(m_b \frac{\sigma_3^{'}}{\sigma_{ci}} + s\right)^a \quad (4)$$

Where, $\sigma_1^{'}$ and $\sigma_3^{'}$ are the maximum and minimum effective principal stresses at failure, $m_b$ is the value of the Hoek-Brown constant m for the rock mass, $S$ and $a$ are constants which depend upon the rock mass

**Table 8.** Classification of rock mass along tunnel path based on suggested systems (lot2).

| Geology | RQD (%) | RMR | Modified RMR | Q | GSI | RMi | RSR |
|---|---|---|---|---|---|---|---|
| SH-ML1 | 50-75 | 45-50 | 35-40 | 3-3.5 | 35-45 | 0.5-1.5 | 37 |
| SH-ML2 | 50-75 | 40-45 | 30-35 | 1.5-2 | 25-35 | 0.5-1 | 28 |
| SH-ML3 | 50-75 | 40-50 | 30-40 | 2-2.5 | 40-45 | 1-2 | 38 |
| ML-SH1 | 75-90 | 60-65 | 50-55 | 6-6.5 | 40-55 | 8-12 | 54 |
| ML-SH2 | 50-75 | 45-50 | 35-40 | 2.5-3 | 45-50 | 2.5-3.5 | 46 |
| ML-SH3 | 50-75 | 45-50 | 35-40 | 2-2.5 | 35-45 | 2-2.5 | 40 |
| ML-SH4 | 50-75 | 50-55 | 40-45 | 3.5-4 | 40-45 | 4.5-5.5 | 45 |
| ML-SH5 | 75-90 | 60-65 | 50-55 | 5-6 | 40-55 | 8-12 | 52 |
| SH-LS1 | 50-75 | 40-45 | 30-35 | 2-3 | 35-40 | 0.5-1.5 | 37 |
| SH-LS2 | 50-75 | 45-50 | 35-45 | 4-4.5 | 40-45 | 2.5-3 | 39 |
| SH-LS3 | 50-75 | 45-50 | 35-40 | 2.5-3 | 35-40 | 1-2 | 37 |
| SH-LS4 | 75-90 | 40-50 | 35-45 | 1.5-2 | 40-45 | 1-2 | 40 |
| LI2 | 75-90 | 55-65 | 45-55 | 4-5 | 55-60 | 6-7 | 54 |
| LI3 | 75-90 | 55-65 | 45-55 | 5.5-6 | 55-60 | 7-8 | 54 |
| LI4 | 75-90 | 50-60 | 40-50 | 2-2.5 | 40-45 | 3-4 | 54 |
| LI5 | 75-90 | 50-60 | 45-55 | 5-5.5 | 50-55 | 7-7.5 | 61 |
| LI6 | 90-100 | 65-70 | 60-65 | 10-12 | 60-65 | 8-10 | 65 |
| LI-MA | 50-75 | 40-45 | 30-40 | 1.5-2 | 35-40 | 1-2 | 43 |
| LI-SH | 50-75 | 40-45 | 35-40 | 2-3 | 35-45 | 1.5-2.5 | 47 |
| FZ | 25-50 | 30-40 | 20-30 | 1-2 | 25-30 | 0.5-1 | 32 |
| CZ | 5-25 | 20-30 | 18-25 | 0.5-1 | 15-25 | 0025-0.75 | 26 |

**Table 9.** The result of engineering rock classification for different unites (lot 2).

| Unite | RQD (%) | RMR | Modified RMR | Q | GSI | RMi | RSR |
|---|---|---|---|---|---|---|---|
| SH-ML1 | 50-75 | 45-50 | 35-40 | 3-3.5 | 35-45 | 0.5-1.5 | 37 |
| SH-ML2 | 50-75 | 40-45 | 30-35 | 1.5-2 | 25-35 | 0.5-1 | 28 |
| SH-ML3 | 50-75 | 40-50 | 30-40 | 2-2.5 | 40-45 | 1-2 | 38 |
| ML-SH1 | 75-90 | 60-65 | 50-55 | 6-6.5 | 40-55 | 8-12 | 54 |
| ML-SH2 | 50-75 | 45-50 | 35-40 | 2.5-3 | 45-50 | 2.5-3.5 | 46 |
| ML-SH3 | 50-75 | 45-50 | 35-40 | 2-2.5 | 35-45 | 2-2.5 | 40 |
| ML-SH4 | 50-75 | 50-55 | 40-45 | 3.5-4 | 40-45 | 4.5-5.5 | 45 |
| ML-SH5 | 75-90 | 60-65 | 50-55 | 5-6 | 40-55 | 8-12 | 52 |
| SH-LS1 | 50-75 | 40-45 | 30-35 | 2-3 | 35-40 | 0.5-1.5 | 37 |
| SH-LS2 | 50-75 | 45-50 | 35-45 | 4-4.5 | 40-45 | 2.5-3 | 39 |
| SH-LS3 | 50-75 | 45-50 | 35-40 | 2.5-3 | 35-40 | 1-2 | 37 |
| SH-LS4 | 75-90 | 40-50 | 35-45 | 1.5-2 | 40-45 | 1-2 | 40 |
| LI2 | 75-90 | 55-65 | 45-55 | 4-5 | 55-60 | 6-7 | 54 |
| LI3 | 75-90 | 55-65 | 45-55 | 5.5-6 | 55-60 | 7-8 | 54 |
| LI4 | 75-90 | 50-60 | 40-50 | 2-2.5 | 40-45 | 3-4 | 54 |
| LI5 | 75-90 | 50-60 | 45-55 | 5-5.5 | 50-55 | 7-7.5 | 61 |
| LI6 | 90-100 | 65-70 | 60-65 | 10-12 | 60-65 | 8-10 | 65 |
| LI-MA | 50-75 | 40-45 | 30-40 | 1.5-2 | 35-40 | 1-2 | 43 |
| LI-SH | 50-75 | 40-45 | 35-40 | 2-3 | 35-45 | 1.5-2.5 | 47 |
| FZ | 25-50 | 30-40 | 20-30 | 1-2 | 25-30 | 0.5-1 | 32 |
| CZ | 5-25 | 20-30 | 18-25 | 0.5-1 | 15-25 | 0025-0.75 | 26 |

characteristics, and $\sigma_{ci}$ is the uniaxial compressive strength of the intact rock pieces.

In order to use the Hoek-Brown criterion for estimating the strength and deformability of jointed rock masses, three properties of the rock mass have to be estimated.

These are:

1. Uniaxial compressive strength $\sigma_{ci}$ of the intact rock mass,
2. Value of the Hoek-Brown constant $m_b$ for these intact rock mass, and
3. Value of the geological strength index (GSI) of the rock mass.

The relationship between the principal stresses at failure for a given rock is defined by two constants, the uniaxial compressive strength $\sigma_{ci}$ and a constant $m_i$. Wherever possible, the values of these constants should be determined by statistical analysis of the results of a set of triaxial tests on carefully prepared core samples.

One should note that the range of minor principal stress ($\sigma_3^{'}$) values over which these tests are carried out is critical in determining reliable values for the two constants. In deriving the original values of $\sigma_{ci}$ and $m_i$, Hoek and Brown (1980a) used a range of $0 < \sigma_3^{'} < 0.5\sigma_{ci}$ and, in order to be consistent, it is essential that the same range be used in any laboratory triaxial tests on intact rock specimens. At least five well spaced data points should be included in the analysis.

The influence of blast damage on the near surface rock mass properties has been taken into account in the 2000 version of the Hoek-Brown criterion (Hoek et al., 2000) as follows:

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \quad (5)$$

$$S = \exp\left(\frac{GSI-100}{9-3D}\right) \quad (6)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \quad (7)$$

Where, *D* is a factor which depends upon the degree of disturbance due to blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. For this project, according to the method of tunneling *D* is taken as 0.

## Rock mass parameters

The most important rock mass parameters for safe tunnel design are the deformation modulus ($E_{mass}$), uniaxial compressive strength of rock mass ($\sigma_{cmass}$) and Hoek-Brown constants. The physical and mechanical properties of the various rock units along tunnel alignment such as porosity, unit weight, uniaxial compressive strength, tensile strength and shear strength parameters(*c and φ*) were determined by performing laboratory tests. The results have been presented in Tables 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, and 22.

As evident from Tables 12, 14, 16 and 18, the friction angle of the joints along the tunnel path varies from a low of 7.6° to as high as 42°. Similarly, the cohesion (Tables 13, 15, 17 and 19) of joints along the tunnel length varied from 0.04 MPa to 2.04 MPa.

The cohesion, internal friction angle, tensile strength, comparative strength, total comparative strength and deformation modulus were determined, as said earlier, and shown in Tables 19, 20, 21 and 22. It is seen from these tables that the cohesion of the rock mass varies from a low of 0.39 MPa to as high as 3.5 MPa. Similarly, the angle of internal friction of the rock mass varies from 23°-53°.

## Strength of rock mass

It is important to understand the rock mass behavior and failure mechanism for safe design of underground spaces and tunnels (Stavropoulou et al., 2007). Although, the estimation of rock mass is carried out for the safe design and stabilization of excavated spaces, it will be difficult because of the presence of the planes of weakness cause anisotropy in rock properties (Basarir, 2008; Deb and Das, 2010; Tzamos and Sofianos, 2007). However, the main aim is to provide the estimate of the strength and deformation properties of the rock mass and to provide an initial estimate of the support requirements for safe design.

Bhasin and Grimstad (1996) have suggested to use eq. (8) for hard rocks, $\sigma_{ci}$ >100 (MPa) and Q>10, where $\gamma$ is the unit weight of rock mass in t/m$^3$.

$$\sigma_{mass} = \left(\frac{\sigma_{ci}}{100}\right)7\gamma Q^{\frac{1}{3}} \quad \text{(MPa)} \quad (8)$$

Singh et al. (1997) have modified the equation for Q<10:

$$\sigma_{mass} = 7\gamma Q^{\frac{1}{3}} \quad \text{(MPa)} \quad (9)$$

Trueman (1998) calculated the strength of rock mass by using RMR, as below:

$$\sigma_{mass} = 0.5e^{0.06RMR} \quad \text{(MPa)} \quad (10)$$

Barton (2000) expressed rock mass strength ($\sigma_{cmass}$)

**Table 10.** Strength parameters of fractures (As per Barton formula).

| | |
|---|---|
| Barton shear failure criterion | |
| Joint set 2 Gate 1 | |
| Input parameters | |
| Basic friction angle ($\varphi$) | 25 |
| Joint roughness coefficient (JRC) | 6 |
| Joint compressive strength (JCS) | 150 |
| Minimum normal stress | 2 |
| Output parameters | |

| Normal stress ($\sigma_n$) (MPa) | Shear strength ($\tau$) (MPa) | $d_\tau$ / $d_{\sigma n}$(DTDS) | Friction angle ($\varphi$) (degree) | Cohesive strength ($c$) (MPa) |
|---|---|---|---|---|
| 0 | 0 | ------ | ------- | ------- |
| 0.25 | 0.222 | 0.808 | 39.0 | 0.020 |
| 0.5 | 0.418 | 0.758 | 37.2 | 0.039 |
| 0.75 | 0.603 | 0.729 | 36.1 | 0.056 |
| 1 | 0.783 | 0.710 | 35.4 | 0.073 |
| 2 | 1.466 | 0.663 | 33.6 | 0.140 |
| 3 | 2.116 | 0.637 | 32.5 | 0.204 |
| 4 | 2.743 | 0.619 | 31.8 | 0.267 |
| 5 | 3.355 | 0.443 | 23.9 | 1.140 |
| 6 | 3.954 | 0.401 | 21.9 | 1.548 |
| 7 | 4.543 | 0.379 | 20.7 | 1.894 |
| 8 | 5.124 | 0.337 | 18.6 | 2.425 |
| 9 | 5.696 | 0.312 | 17.3 | 2.887 |
| 10 | 6.262 | 0.290 | 16.2 | 3.363 |
| 11 | 6.822 | 0.270 | 15.1 | 3.852 |
| 12 | 7.377 | 0.252 | 14.2 | 4.351 |
| 13 | 7.927 | 0.236 | 13.3 | 4.861 |
| 14 | 8.472 | 0.221 | 12.5 | 5.380 |
| 15 | 9.013 | 0.207 | 11.7 | 5.908 |
| 16 | 9.550 | 0.194 | 11.0 | 6.443 |
| 17 | 10.083 | 0.182 | 10.3 | 6.987 |

**Table 11.** Internal friction angle ($\varphi$) (degree) of rock joints along tunnel length (0-3 km) (lot 2).

| Geology | SH-ML1 | SH-ML2 | SH-ML3 | ML-SH1 | ML-SH2 | ML-SH3 | ML-SH4 | ML-SH5 | SH-LS1 | SH-LS2 | SH-LS3 | CZ1 | CZ2 | FZ1 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint 1 | 31 | 28.8 | 28.9 | 42 | 40.5 | 38 | 34.7 | 34.7 | 29.1 | 21.37 | 29.1 | 28 | 22 | 21 |
| Joint 2 | 23.5 | 27.9 | 25.8 | 33.5 | 26.9 | 32 | 27.06 | 29.1 | 28.2 | 31 | 29.9 | 22 | 22 | 22 |
| Joint 3 | 25.6 | 27.3 | 29.9 | 38 | 31 | 38 | 38.9 | 29.1 | 29.4 | 32 | 26.5 | 24 | 21 | 21 |
| Joint 4 | - | 33.1 | 29.9 | 38 | - | - | 36.9 | 35.9 | - | 29.7 | - | - | - | - |
| Bedding | 35.3 | 35.3 | 28.8 | 38.9 | 36.6 | 38.9 | 35.9 | 37.6 | 27.6 | 29.7 | 28.9 | 24 | 23 | 21 |

using the normalization of Q-values, as below, where $\gamma$ is the unit weight of rock mass (t/m$^3$).

$$\sigma_{mass} = 5\gamma\left(Q\frac{\sigma_c}{100}\right)^{\frac{1}{3}} \text{ (MPa)} \quad (11)$$

The calculated $\sigma_{cmass}$ values have been given in Table 23.

The $\sigma_{cmass}$ obtained by Bhasin and Grimstad (1996) method is 496 MPa. It could be determined for only one segment of the tunnel path. It was observed that the values of $\sigma_{cmass}$ vary from 20.24-52.74 MPa, based on the

**Table 12.** Cohesion *c* (MPa) of rock joints along tunnel length (0-3 km) (lot 2).

| Geology | SH-ML1 | SH-ML2 | SH-ML3 | ML-SH1 | ML-SH2 | ML-SH3 | ML-SH4 | ML-SH5 | SH-LS1 | SH-LS2 | SH-LS3 | CZ1 | CZ2 | FZ1 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint 1 | 0.077 | 0.026 | 0.148 | 0.158 | 0.151 | 0.094 | 0.255 | 0.255 | 0.243 | 0.053 | 0.243 | 0 | 0.1 | 0.1 |
| Joint 2 | 0.014 | 0.038 | 0.097 | 0.034 | 0.019 | 0.083 | 0.088 | 0.092 | 0.182 | 0.155 | 0.112 | 0 | 0 | 0 |
| Joint 3 | 0.029 | 0.05 | 0.112 | 0.094 | 0.081 | 0.094 | 0.189 | 0.092 | 0.13 | 0.118 | 0.074 | 0 | 0 | 0 |
| Joint 4 | - | 0.041 | 0.112 | 0.094 | - | - | 0.135 | 0.176 | - | 0.226 | - | - | - | - |
| Bedding | 0.057 | 0.057 | 0.148 | 0.072 | 0.089 | 0.072 | 0.176 | 0.245 | 0.216 | 0.226 | 0.148 | 0 | 0 | 0.1 |

**Table 13.** Internal friction angle ($\varphi$) (degree) of rock joints along tunnel length (3-10 km) (lot 2)

| Geology | SH-LS4 | LI2 | SH-LS3 | SH-LS2 | SH-LS1 | ML-SH5 | CZ | FZ |
|---|---|---|---|---|---|---|---|---|
| Joint 1 | 31 | 28.8 | 28.9 | 42 | 40.5 | 38 | 34.7 | 34.7 |
| Joint 2 | 23.5 | 27.9 | 25.8 | 33.5 | 26.9 | 32 | 27.06 | 29.1 |
| Joint 3 | 25.6 | 27.3 | 29.9 | 38 | 31 | 38 | 38.9 | 29.1 |
| Joint 4 | - | 33.1 | 29.9 | 38 | - | - | 36.9 | 35.9 |
| Bedding | 35.3 | 35.3 | 28.8 | 38.9 | 36.6 | 38.9 | 35.9 | 37.6 |

**Table 14.** Cohesion *c* (MPa) of rock joints along tunnel length (3-10 km) (lot 2).

| Geology | SH-LS4 | LI2 | SH-LS3 | SH-LS2 | SH-LS1 | ML-SH5 | CZ | FZ |
|---|---|---|---|---|---|---|---|---|
| Joint 1 | 0.19 | 0.04 | 0.11 | 0.23 | 0.18 | 0.2 | 0.07 | 0.07 |
| Joint 2 | 0.08 | 0.04 | 0.11 | 0.2 | 0.06 | 0.05 | 0.03 | 0.08 |
| Joint 3 | 0.05 | 0.12 | 0.13 | 0.19 | 0.8 | 0.11 | 0.04 | 0.11 |
| Joint 4 | - | 0.23 | - | 0.26 | - | 0.17 | 0.02 | 0.09 |
| Bedding | 0.26 | 0.32 | 0.43 | 0.5 | 0.4 | 0.6 | 0.11 | 0.06 |

**Table 15.** Internal friction angle ($\varphi$) (degree) of rock joints along tunnel length (10-17 km) (lot 2).

| Geology | SH-LS1 | SH-LS2 | SH-LS3 | SH-LS4 | LI2 | ML-SH5 | CZ |
|---|---|---|---|---|---|---|---|
| Joint 1 | 33 | 28 | 26.7 | 29 | 33 | 31.0 | 28 |
| Joint 2 | 34 | 27.8 | 20.6 | 29 | 34 | 31 | 28.4 |
| Joint 3 | 26 | 22.6 | 25.8 | 18 | 37 | 22.5 | 28.3 |
| Joint 4 | 35 | 22.6 | 25.8 | 28.9 | 37 | - | 28.4 |
| Bedding | 35 | 28 | 28.9 | 25 | 25 | 25 | 7.6 |

**Table 16.** Cohesion *c* (MPa) of rock joints along tunnel length (10-17 km) (lot 2).

| Geology | SH-LS1 | SH-LS2 | SH-LS3 | SH-LS4 | LI2 | ML-SH5 | CZ |
|---|---|---|---|---|---|---|---|
| Joint 1 | 0.25 | 0.103 | 0.99 | 0.105 | 0.17 | 0.11 | 0.03 |
| Joint 2 | 0.15 | 0.152 | 0.65 | 0.105 | 0.11 | 0.11 | 0.02 |
| Joint 3 | 0.97 | 0.67 | 0.15 | 1.04 | 0.18 | 0.99 | 0.03 |
| Joint 4 | 0.17 | 0.67 | 0.15 | 0.155 | 0.18 | - | 0.02 |
| Bedding | 0.17 | 0.29 | 0.22 | 0.76 | 1.6 | 1.04 | 2.04 |

relationship suggested by Singh et al. (1977). It varies from 6.40-28.70 MPa as per Trueman (1998) relationship; and from 9.72-30.54 MPa as per Barton (2000) relationship. The $\sigma_{cmass}$ obtained using Trueman (1998) relationship is quite on the lower side.

**Deformation modulus of rock mass ($E_{mass}$)**

For determining the deformation modulus of rock masses along the tunnel alignment, different equations proposed by different researchers have been used as follow:

**Table 17.** Internal friction angle ($\varphi$) (degree) of rock joints along tunnel length (17-26 km) (lot 2).

| Geology | L1-SH | LI6 | LI5 | LI4 | LI3 | LI-MA | CZ | FZ |
|---|---|---|---|---|---|---|---|---|
| Joint 1 | 26.5 | 32 | 28 | 30 | 35 | 25.0 | 18 | 27 |
| Joint 2 | 28.3 | 24 | 36 | 27 | 19 | 23 | 25 | 27.4 |
| Joint 3 | 26 | 36 | 37 | 32 | 21.6 | 28 | 28 | 29 |
| Joint 4 | - | - | - | - | - | 31 | 28 | 28 |
| Bedding | 26 | 27 | 33 | 15 | 20 | 25 | 20 | 27 |

**Table 18.** Cohesion *c* (MPa) of rock joints along tunnel length (17-26 km) (lot 2).

| Geology | L1-SH | LI6 | LI5 | LI4 | LI3 | LI-MA | CZ | FZ |
|---|---|---|---|---|---|---|---|---|
| Joint 1 | 0.5 | 0.22 | 0.7 | 0.09 | 0.26 | 0.5 | 0.9 | 0.14 |
| Joint 2 | 0.2 | 1.5 | 0.2 | 0.4 | 2.4 | 0.9 | 0.08 | 0.06 |
| Joint 3 | 0.8 | 0.18 | 0.1 | 0.16 | 2.5 | 0.15 | 0.04 | 0.04 |
| Joint 4 | - | - | - | - | - | 0.16 | 0.07 | 0.08 |
| Bedding | 0.6 | 1.1 | 0.32 | 3.5 | 3 | 0.8 | 0.8 | 0.16 |

**Table 19.** Strength of rock mass in tunnel (0-3 km) (lot 2).

| Geology | Cohesion (MPa) | Internal friction angle (degree) | Tensile strength (MPa) | Comparative strength (MPa) | Total comparative strength (MPa) | Deformation modulus (MPa) |
|---|---|---|---|---|---|---|
| SH-ML1 | 0.156 | 36.43 | -0.017 | 0.34 | 1.69 | 1414 |
| SH-ML2 | 0.134 | 34.43 | -0.012 | 0.24 | 1.46 | 1060 |
| CZ1 | 0.219 | 35.04 | -0.007 | 0.24 | 1.91 | 1056 |
| ML-SH1 | 3.486 | 30.51 | -0.160 | 4.22 | 12.20 | 8367 |
| SH-ML3 | 0.166 | 38.31 | -0.007 | 0.24 | 1.91 | 1060 |
| ML-SH2 | 0.248 | 42.15 | -0.039 | 0.83 | 2.93 | 2812 |
| ML-SH3 | 0.243 | 40.77 | -0.045 | 0.83 | 2.72 | 2812 |
| CZ2 | 0.334 | 38.96 | -0.037 | 0.84 | 3.64 | 2495 |
| ML-SH4 | 0.350 | 35.11 | -0.025 | 0.60 | 3.19 | 7746 |
| ML-SH5 | 0.642 | 41.77 | -0.070 | 1.81 | 5.23 | 2495 |
| SH-LS1 | 0.242 | 29.97 | -0.015 | 0.30 | 1.80 | 1186 |
| SH-LS2 | 0.355 | 36.33 | -0.032 | 0.72 | 3.12 | 2310 |
| SH-LS3 | 0.278 | 31.86 | -0.021 | 0.43 | 2.11 | 1581 |
| FZ | 0.242 | 29.97 | -0.015 | 0.30 | 1.82 | 1186 |

Bieniawski (1978) has defined $E_{mass}$ as below:

$E_{mass}$ =2RMR−100 (GPa) RMR>5 (12)

Serafim and Pereira (1983) have proposed the Equation (14) for RMR<50:

$$E_{mass} = 10^{(\frac{RMR-10}{40})} \text{ (GPa)} \qquad \text{RMR<50} \qquad (13)$$

For Q>1 and generally for hard rocks, Grimstad and Barton (1993) have proposed the Equation (15):

$E_{mass}$ =25logQ (GPa) (14)

Mitri (1994) has defined the Equation (15) for determining $E_{mass}$ based on $E_i$, as below:

$$E_{mass} = E_i \left[ 0.5 \left( 1 - \left\{ \cos \pi \frac{RMR}{100} \right\} \right) \right] \text{ (GPa)} \qquad (15)$$

Singh et al. (1997) has proposed an equation for

**Table 20.** Strength of rock mass in tunnel (3-10 km) (lot 2).

| Geology | Cohesion (MPa) | Internal friction angle (degree) | Tensile strength (MPa) | Comparative strength (MPa) | Total comparative strength (MPa) | Deformation modulus (MPa) |
|---|---|---|---|---|---|---|
| SH-LS1 | 0.47 | 29 | -0.02 | 0.52 | 2.70 | 1732 |
| SH-LS2 | 0.98 | 40 | -0.08 | 2.02 | 7.80 | 5302 |
| SH-LS3 | 0.64 | 33 | -0.04 | 0.99 | 3.75 | 3080 |
| SH-LS4 | 0.54 | 38 | -0.06 | 1.34 | 4.20 | 4107 |
| ML-SH5 | 0.77 | 42 | -0.05 | 1.80 | 9.32 | 3652 |
| LI2 | 1.18 | 49 | -0.25 | 6.03 | 14.6 | 11548 |
| CZ3 | 0.44 | 38 | -0.02 | 0.60 | 6.10 | 1540 |
| CZ4 | 0.45 | 33 | -0.02 | 0.55 | 5.01 | 1372 |
| FZ2 | 0.40 | 28 | -0.01 | 0.35 | 2.37 | 1298 |
| CZ5 | 0.39 | 26 | -0.01 | 0.23 | 2.16 | 974 |

**Table 21.** Strength of rock mass in tunnel (10-17.5 km) (lot 2).

| Geology | Cohesion (MPa) | Internal friction angle (degree) | Tensile strength (MPa) | Comparative strength (MPa) | Total comparative strength (MPa) | Deformation modulus (MPa) |
|---|---|---|---|---|---|---|
| SH-LS1 | 0.44 | 23 | -0.015 | 0.299 | 1.824 | 1185 |
| SH-LS2 | 0.99 | 37 | -0.099 | 2.24 | 7.01 | 5302 |
| SH-LS3 | 0.41 | 24 | -0.015 | 0.299 | 1.824 | 1185 |
| SH-LS4 | 0.73 | 41 | -0.054 | 1.65 | 6.98 | 3976 |
| ML-SH5 | 1.3 | 45 | -0.158 | 4.48 | 15.6 | 1499 |
| LI2 | 2.14 | 51 | -0.50 | 12.05 | 29.21 | 13335 |
| CZ3 | 0.4-0.7 | 26-40 | -0.017- -0.036 | 0.25-1.2 | 2.5-12 | 1120-1778 |

**Table 22.** Strength of rock mass in tunnel (17.5-26 km) (lot 2).

| Geology | Cohesion (MPa) | Internal friction angle (degree) | Tensile strength (MPa) | Comparative strength (MPa) | Total comparative strength (MPa) | Deformation modulus (MPa) |
|---|---|---|---|---|---|---|
| LI3 | 3.1-3.5 | 42-44 | -0.42 - -0.61 | 10-13 | 24-27 | 1330-1770 |
| LI4 | 1.6-1.7 | 36-37 | -0.08 - -0.12 | 2.5-3.4 | 10.5-11.5 | 480-650 |
| LI5 | 1.7-2.06 | 47-49 | -0.29 - -0.42 | 7.5-10 | 22-24 | 100-130 |
| LI6 | 1.7-2.1 | 52-53 | -0.42 - -0.61 | 10-13 | 24-27 | 1330-1770 |
| LI-MA | 0.52-0.58 | 36-38 | -0.05 - -0.07 | 0.95-1.3 | 4.5-5.0 | 250-350 |
| LI-SH | 1.05-1.15 | 33-35 | -0.04 - -0.06 | 1.3-1.8 | 5.5-6.25 | 350-470 |
| CZ | 0.91-0.97 | 28-29 | -0.015 - -0.018 | 0.5-0.59 | 5.6-6.1 | 130-150 |
| FZ | 0.64-0.75 | 39-42 | -0.03 - -0.04 | 0.99-1.5 | 10.12 | 170-230 |

determining $E_{mass}$ by using $E_i$ and Hoek-Brown constants:

$$E_{mass} = E_i (S_m)^{1/1.4} \text{ (GPa)} \quad (16)$$

$$S_m = \frac{7\gamma Q^{\frac{1}{3}}}{\sigma_{ci}} \quad (17)$$

For weak rocks, $\sigma_{ci}$<100 MPa, Hoek and Brown (1998) have found a correlation between $E_{mass}$ and GSI:

$$E_{mass} = \sqrt{\frac{\sigma_{ci}}{100}} \times 10^{\left(\frac{GSI-10}{40}\right)} \text{ (GPa)} \quad (18)$$

Read et al. (1999) have proposed the Equation (19) for calculating $E_{mass}$ based on RMR value of rock mass.

$$E_{mass} = 0.1\left(\frac{RMR}{10}\right)^3 \text{ (GPa)} \quad (19)$$

**Table 23.** Strength values ($\sigma_{cmass}$) obtained from different equations (lot 2).

| Equation No. | 8 | 9 | 10 | 11 |
|---|---|---|---|---|
| SH-ML1 | ------- | 25.40 | 8.64 | 12.80 |
| SH-ML2 | ------- | 20.24 | 6.40 | 10.20 |
| SH-ML3 | ------- | 52.74 | 8.64 | 11.08 |
| ML-SH1 | ------- | 71.56 | 21.26 | 21.33 |
| ML-SH2 | ------- | 22.56 | 6.40 | 11.61 |
| ML-SH3 | ------- | 20.41 | 6.4 | 10.27 |
| ML-SH4 | ------- | 25.56 | 11.67 | 12.86 |
| ML-SH5 | ------- | 30.27 | 21.26 | 19.25 |
| SH-LS1 | ------- | 23.27 | 6.40 | 9.72 |
| SH-LS2 | ------- | 27.80 | 8.64 | 13.98 |
| SH-LS3 | ------- | 23.54 | 8.64 | 9.83 |
| SH-LS4 | ------- | 20.69 | 7.4 | 10.40 |
| LI2 | ------- | 29.47 | 18.30 | 22.67 |
| LI3 | ------- | 32.00 | 18.30 | 24.61 |
| LI4 | ------- | 23.39 | 13.56 | 15.20 |
| LI5 | ------- | 28.00 | 13.56 | 21.53 |
| LI6 | 49.6 | --------- | 28.70 | 30.54 |
| LI-MA | ------- | 20.25 | 6.40 | 9.98 |
| LI-SH | ------ | 22.80 | 6.40 | 11.24 |

Barton (2002) expressed $E_{mass}$ based on $Q_c$, as follows:

$$E_{mass} = 10Q_c^{\frac{1}{3}} \text{ (GPa)} \quad (20)$$

The calculated $E_{mass}$ values have been given in Table 24.

The $E_{mass}$ was calculated using various relationships as mentioned above and it was observed that it varies from a low of 0.24 GPa to a high of 35.0 GPa.

## In-situ stresses

The main origins of in-situ stresses are geological conditions and geological history of the area. The magnitude of in-situ stresses in rock mass can changes from zero to high amounts, as much as rock strength. Based on the information that is obtained from measuring in-situ stresses in different places, the magnitude of horizontal stress is usually more than the vertical stress at shallow depths (less than 500 m); whereas, they trend to a hydrostatic state at a depth of about 1000 m below the surface. In this study, for a better understanding, $\sigma_h$ is assumed to be equal to $\sigma_v$ as suggested by Hoek (2003). The stresses at NWCT site have mainly arisen from tectonic forces and overburden pressures. For the determination of the stresses, no field or laboratory tests have been carried out. However, they were calculated as:

$$\sigma_V = \gamma Z \quad (21)$$

Where: $\sigma_V$ = vertical stress (t/m$^2$), $\gamma$ = unit weight of rock mass average (2.62 t/m$^3$), and $Z$ = tunnel depth below surface in m.

## Evaluation of squeezing conditions

The squeezing of rock is a process of large deformation which occurs around the tunnel due to stress concentration and material properties and it is a major factor for predicting rock behavior in underground excavations. This phenomenon usually occurs in weak rocks under the influence of high pressures. As a consequence of squeezing, large ground deformation occurs around the tunnels and underground openings in poor rocks. Large deformations, low shear strength and high in-situ stresses are the most effective factors on the stability and convergence of tunnel walls and face. The magnitude of tunnel convergence, rate of deformation and extent of the yielding zone around the tunnel depend on rock mass properties and critical zone around the tunnel. Generally, rock type, rock mass strength, ground water flow, pore pressure, state of stress and support techniques are the most important effective factors on the squeezing behavior (Juneja et al., 2010; Kim et al., 2006; Farrokh et al., 2006; Panthi and Nilsen, 2006; Barla et al., 2010). Singh et al. (1992), based on 39 case histories, by collecting data on rock mass quality ($Q$), and overburden ($H$), defined an equation for predicting squeezing conditions:

$$H = 350Q^{\frac{1}{3}} \quad (22)$$

Where: H = overburden (m), and $Q$ = rock mass quality.

For squeezing condition:

$H >> 350Q^{1/3}$

Goel et al. (1995) have proposed a simple empirical approach which is based on the rock mass number ($Q_n$), as follows:

$$H = (27 \times Q_n^{0.33})B^{-0.1} \quad (23)$$

Where:

$H$ = overburden (m), $B$ = tunnel span or diameter (m), and $Q_n$ = rock mass number.

If the right side of the equation is equal or bigger than the left side, squeezing conditions will occur. Jethwa et al. (1984) defined Equation (24) for determining the degree of squeezing on the basis of the rock mass uniaxial compressive strength and the tunnel depth below the

**Table 24.** Calculated $E_{mass}$ values from empirical methods for different rock mass (lot 2).

| Equation | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |
|---|---|---|---|---|---|---|---|---|---|
| SH-ML1 | --- | 8.60 | 12.80 | | | 0.73 | 1.4 | 10.72 | 14.82 |
| SH-ML2 | --- | 6.50 | 6.08 | | | 0.58 | 1.05 | 7.68 | 12.06 |
| SH-ML3 | --- | 7.50 | 8.81 | | | 0.63 | 1.50 | 9.12 | 13.11 |
| ML-SH1 | 25 | --- | 19.90 | | | 0.44 | 2.55 | 24.41 | 18.42 |
| ML-SH2 | --- | 8.66 | 11.00 | | | 0.66 | 1.74 | 10.72 | 14.01 |
| ML-SH3 | --- | 8.66 | 8.81 | | | 0.58 | 1.40 | 10.72 | 13.11 |
| ML-SH4 | 5 | --- | 14.35 | | | 0.73 | 1.50 | 14.47 | 15.53 |
| ML-SH5 | 25 | --- | 18.51 | | | 0.40 | 2.55 | 24.41 | 17.65 |
| SH-LS1 | --- | 6.50 | 9.95 | | | 1.64 | 0.99 | 7.68 | 13.57 |
| SH-LS2 | --- | 8.60 | 15.71 | | | 0.79 | 1.51 | 10.72 | 16.20 |
| SH-LS3 | --- | 8.60 | 11.00 | | | 1.18 | 0.99 | 10.72 | 14.01 |
| SH-LS4 | --- | 7.50 | 6.1 | | | 0.6 | 1.51 | 9.12 | 12.06 |
| LI2 | 20 | --- | 16.33 | | | 0.24 | --- | 21.60 | 16.51 |
| LI3 | 20 | --- | 19.00 | | | 0.26 | --- | 21.60 | 17.92 |
| LI4 | 10 | --- | 8.81 | | | 0.32 | 2.21 | 16.64 | 13.11 |
| LI5 | 10 | --- | 18.00 | | | 0.26 | --- | 16.64 | 17.38 |
| LI6 | 35 | --- | 26.01 | | | 0.32 | --- | 310.75 | 22.24 |
| LI-MA | --- | 6.50 | 6.10 | | | 0.53 | 1.31 | 7.68 | 12.06 |
| LI-SH | --- | 6.50 | 9.95 | | | 0.65 | 1.41 | 7.68 | 13.57 |

**Table 25.** Classification of squeezing behavior according to different approaches (lot2).

| Equation | 22 |
|---|---|
| SH-ML1 | 518.07 |
| SH-ML2 | 422.1 |
| SH-ML3 | 458.85 |
| ML-SH1 | 644.7 |
| ML-SH2 | 490.35 |
| ML-SH3 | 458.85 |
| ML-SH4 | 543.55 |
| ML-SH5 | 617.75 |
| SH-LS1 | 474.95 |
| SH-LS2 | 567 |
| SH-LS3 | 490.35 |
| SH-LS4 | 422.1 |
| LI2 | 577.85 |
| LI3 | 627.2 |
| LI4 | 458.85 |
| LI5 | 608.3 |
| LI6 | 778.4 |
| LI-MA | 422.1 |
| LI-SH | 474.95 |

surface as:

$$N_c = \frac{\sigma_{cm}}{\gamma \times H} \quad (24)$$

The classification of squeezing behaviors, according to different approaches, has been given in Table 25.

As is evident from Table 25, the squeezing potential of rock mass in lot2 is very high and it varies from a low of 422.1 to a high of 778.4. These values are quite significant and it must be provided with suitable support lining.

## Support pressures

Rock mass classification systems form the backbone of the empirical methods and were applied for the estimation of the support pressure and the design of tunnel support. In this study, for the estimation of support pressure ($P_i$), Q and RMR classification systems have been used.

Unal (1983) has presented Eq. (25) which can be used to calculate the tunnel support pressure based on RMR:

$$P_i = (100 - RMR)\gamma \times \frac{B}{100} \quad (25)$$

Where: $P_i$ = Support pressure (kg/m$^2$), $B$ = Tunnel width (m), and $\gamma$ = Unit weight (t/m$^3$)

Goel and Jethwa (1995), based on rock mass rating (RMR), have defined eq. (26) for predicting squeezing conditions in tunnels:

$$P = \frac{7.5B^{0.1} \times H^{0.5} - RMR}{20RMR} \quad (26)$$

**Table 26.** Support pressure using empirical equations for different zones (lot 2).

| Equation | 25 |
|---|---|
| SH-ML1 | 5.92 |
| SH-ML2 | 6.35 |
| SH-ML3 | 6.07 |
| ML-SH1 | 4.4 |
| ML-SH2 | 3.51 |
| ML-SH3 | 5.38 |
| ML-SH4 | 5.13 |
| ML-SH5 | 4.23 |
| SH-LS1 | 6.48 |
| SH-LS2 | 5.92 |
| SH-LS3 | 5.92 |
| SH-LS4 | 6.2 |
| LI2 | 4.7 |
| LI3 | 4.7 |
| LI4 | 5.3 |
| LI5 | 4.76 |
| LI6 | 3.81 |
| LI-MA | 6.35 |
| LI-SH | 6.35 |

Where, $P$ is support pressure (MPa), $B$ is tunnel width (m) and $H$ is tunnel depth below the surface (m).

The amount of support pressure has been calculated for all zones of the tunnel path. Also, the summary of offered relationships by some researchers for calculating support pressure has been presented in Table 26.

As seen from the Table 26, the support pressure varies from a low of 3.81 MPa to a high of 6.48 MPa. These pressures are quite significant and warrant suitable lining in the tunnel. There are a large number of various relationships for measuring the support pressure in tunnels and a few widely used relationships are shown in Table 27.

## Tunnel support assessment

The design of support for tunnels in weak rocks is an interactive method. There are different approaches for evaluating tunnel support system and the interaction between the surrounding rock mass and supporting system. In this study for the safe tunnel design, a combination of empirical methods was adopted.

### METHODS

In this study, the support systems were assessed based on RMR and Q classification systems. The suggested support systems, based on RMR system with stand up times for each geotechnical zone, have been presented in Table 28 and Figure 3, without installation of the support systems, the maximum and minimum stand up times for zones.

Barton et al. (1974), relating the Q-index with the stability and support requirements of underground excavations, have defined an additional parameter which is called the equivalent dimension $D_e$ of excavation. This dimension is obtained by dividing the span, diameter or wall height of excavation by a quantity called the Excavation Support Ratio, ESR. Hence:

$$D_e = \frac{Excavation\ span, diameter\ or\ height\ (m)}{Excavation\ Support\ Ratio\ (ESR)} \quad (27)$$

The value of ESR is the so-called excavation support ratio. It ranges between 0.5 and 5 and has been presented in a tabular form in a number of research papers for various types of excavations. For the diversion tunnel, the excavation support ratio, ESR is defined as 1.6. Hence, for an excavation span of 5 m, the equivalent dimension, $D_e$, is 3.13. Barton et al. (1974)'s equivalent dimension, $D_e$, plotted against the value of $Q$, is used to define a number of support categories in a chart published in the original paper. This chart has recently been updated by Grimstad and Barton (1993) to reflect the increasing use of steel fibres, reinforced shotcrete in underground excavation support. The estimated support categories for each geotechnical zone along the diversion tunnel have been shown in Figure 4. Also, the recommended types of support have been presented in Table 9.

## RESULTS

This study is a contribution to the investigations and support design of a tunnel project in Iran in difficult geological condition using a combination of empirical methods. NWCT is a project with complicated geological conditions and this paper discusses it as a case study.

A geological study, included the field and laboratories investigations, was carried out and based on the results, the tunnel alignment of lot1 and lot 2 was divided into 12 and 21 lithology types respectively. Some stability problems were predicted at some locations along the alignment and hence a more detailed exploration was carried out.

Based on the results of studies made in the engineering geological zone, along the total route of the tunnel, regardless of surface sediments or hypothesis, the lithology of the rock/soil was identified to include argillaceous and shale, sandstone and shale. According to the field and visual investigation, including geological and geotechnical investigation (borehole, core logging and laboratory testing), the rock/soil mass has been identified to consist of 12 lithography types in lot 1 and 21 lithography types in lot2.

The boundary of types of lithography are according to the stratigraphy and in many cases for the geomechanical features, the lithography was the main factor in separation and classification.

The rock mass, along the tunnel path in lot1, varies from very weak, thinly bedded, crushed and unstable to moderately strong, thick bedding and stable. Similarly, the rock mass along the tunnel path in lot2 (Table 3) varies from very weak, thin bedding, crushed and

**Table 27.** Different relationships suggested by some researchers for measuring support pressure.

| | References | Equation |
|---|---|---|
| Rock mass parameters ($\sigma_{cmass}$) | Bhasin and Grimstad (1996) | $\sigma_{mass} = (\frac{\sigma_{ci}}{100})7\gamma Q^{\frac{1}{3}}$ |
| | Singh et al. (1997) | $\sigma_{mass} = 7\gamma Q^{\frac{1}{3}}$ |
| | Trueman (1998) | $\sigma_{mass} = 0.5e^{0.06RMR}$ |
| | Barton (2000) | $\sigma_{mass} = 5\gamma\left(Q\frac{\sigma_c}{100}\right)^{\frac{1}{3}}$ |
| Deformation modulus of rock mass ($E_{mass}$) | Bieniawski (1978) | $E_{mass} = 2RMR - 100$ |
| | Serafim and Pereira (1983) | $E_{mass} = 10^{(\frac{RMR-10}{40})}$ |
| | Grimstad and Barton (1993) | $E_{mass} = 25\log Q$ |
| | Mitri (1994) | $E_{mass} = E_i\left[0.5\left(1-\left\{\cos\pi\frac{RMR}{100}\right\}\right)\right]$ |
| | Singh et al. (1997) | $E_{mass} = E_i (S_m)^{1/1.4}$ |
| | Hoek and Brown (1998) | $E_{mass} = \sqrt{\frac{\sigma_{ci}}{100}\times 10^{(\frac{GSI-10}{40})}}$ |
| | Read et al. (1999) | $E_{mass} = 0.1\left(\frac{RMR}{10}\right)^3$ |
| | Barton (2002) | $E_{mass} = 10Q_c^{\frac{1}{3}}$ |
| Constant of rock mass (m-s) | Hoek et al.(1995) | $m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right)$ <br> $S = \exp\left(\frac{GSI-100}{9-3D}\right)$ |

**Table 28.** Suggested support systems, assessed based on rock mass rating system (RMR) with stand up times for each geotechnical zone.

| RMR | Anchoring Φ 20 mm | Shotcrete | Ribs | Zone |
|---|---|---|---|---|
| 81-100 | - | - | - | - |
| 61-80 | Locally bolts in crown,3 m long, spaced 2.5 m, with occasional wire mesh | 50 mm in crown where required | | ML-SH1, ML-SH5, LI6 |
| 41-60 | Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown | 50-100 mm in crown,30 mm in sides | | SH-ML1, SH-LS1, SH-ML2, SH-LS2, SH-ML3, SH-LS3, ML-SH2, SH-LS4, ML-SH3, LI2, LI3, ML-SH4, LI4, LI5, LI-MA,LI-SH |
| 21-40 | Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh | 100-150 mm in crown, 100 mm in sides | Light ribs spaced 1.5 m where required | |
| <20 | Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert | 150-200 mm in crown, 150 mm in sides and 50 mm in face | Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert | |

unstable to weak to moderately strong, crushed, medium bedding and unstable.

Based on the geomechanical features, the rock mass in lot1 were observed to consist of poor and crushed to very strong, massive with an average distance between discontinuities of significantly more than half a meter. Similarly, the geomechanical features in lot2 were described as poor and crushed to semi-solid, medium to thick layers and the average distance between the discontinuities to be significantly less than half a meter. A large number of tests were performed for geotechnical classification of the rock/soil mass. The characteristics included, rock strength, density, uniaxial tensile strength, shear strength parameters, Young's modulus and hardness to abrasion. The physical and mechanical properties of the rock units along tunnel alignment in lot1 and lot 2 were evaluated.

In this study, the rock mass classification were

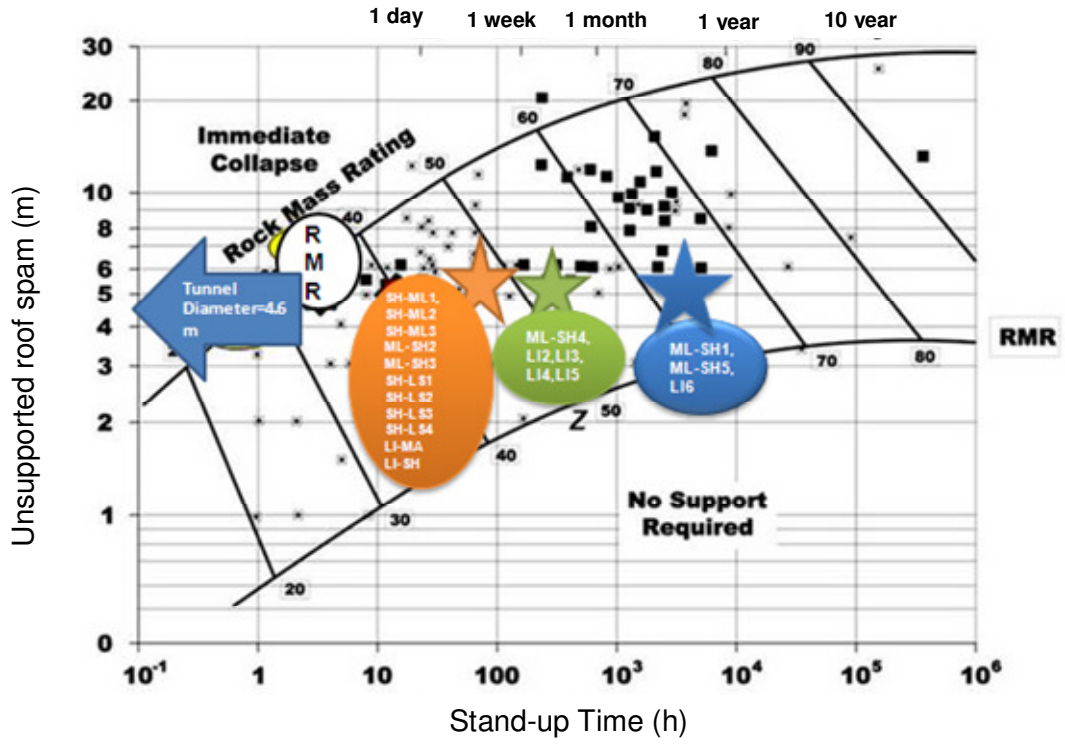



**Figure 3.** Relationship between Stand-up time, span and RMR classification (after Bieniawski, 1989).

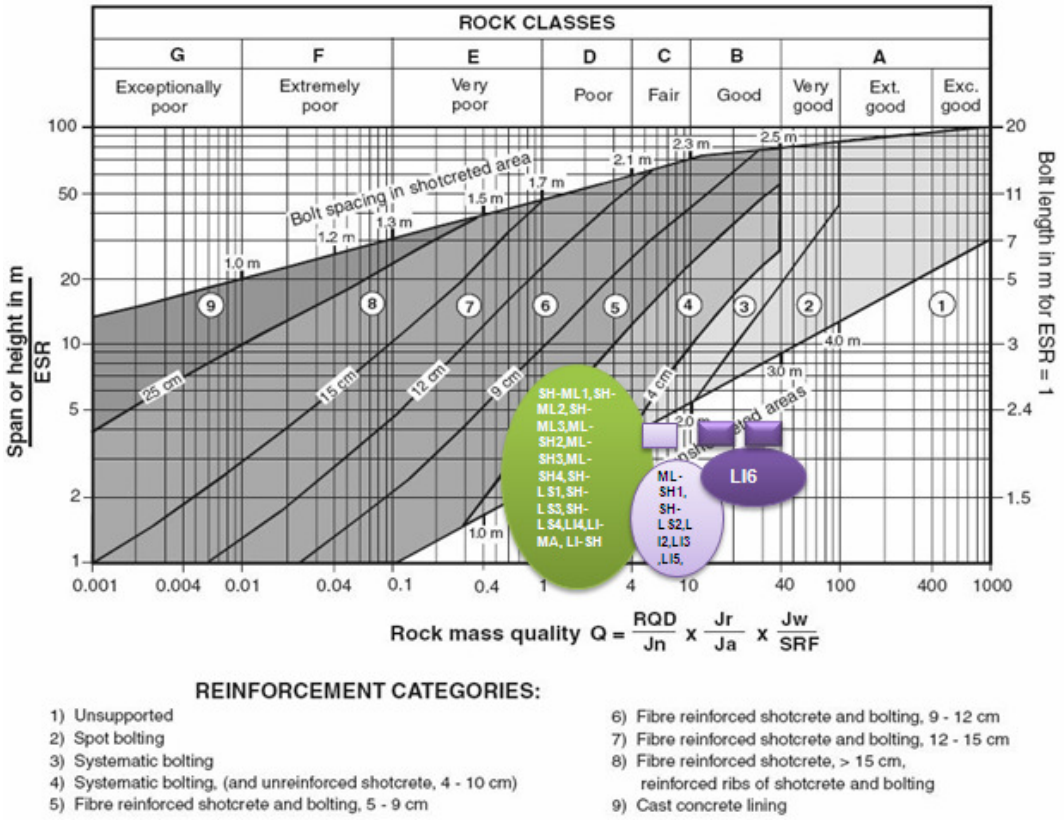



**Figure 4.** Estimated support categories based on the tunneling quality index Q (after, Palmstrom and Broch, 2006).

performed according to Rock Mass Rating (RMR), RMR modified, Geological Strength Index (GSI), Rock Structure Rating (RSR), Quality system (Q), and Rock Mass index (RMi) systems for the transfer tunnel and the properties of rock mass were determined by using these systems.

The most important rock mass parameters for safe tunnel design are the deformation modulus ($E_{mass}$), uniaxial compressive strength of rock mass ($\sigma_{cmass}$) and Hoek-Brown constants. The physical and mechanical properties of the various rock units along tunnel alignment such as porosity, unit weight, uniaxial compressive strength, tensile strength and shear strength parameters(*c and φ*) were determined by performing laboratory tests. The friction angle ($\varphi$) of the joints along the tunnel path varies from a low of 7.6° to as high as 42°. Similarly, the cohesion ($c$) of joints along the tunnel length varied from 0.04 MPa to 2.04 MPa.

The cohesion, internal friction angle, tensile strength, comparative strength, total comparative strength and deformation modulus were determined. It was observed that the cohesion of the rock mass varies from a low of 0.39 MPa to as high as 3.5 MPa. Similarly, the angle of internal friction of the rock mass varies from 23°-53°.

It was observed that the values of $\sigma_{cmass}$ vary from 20.24-52.74 MPa, based on the relationship suggested by Singh et al. (1977). It varies from 6.40-28.70 MPa as per Trueman (1998) relationship; and from 9.72-30.54 MPa as per Barton (2000) relationship. The $\sigma_{cmass}$ obtained using Trueman (1998) relationship was quite on the lower side. Similarly, the deformation modulus ($E_{mass}$) was calculated using various relationships as mentioned above and it was observed that it varies from a low of 0.24 GPa to a high of 35.0 GPa.

The squeezing of rock is a process of large deformation which occurs around the tunnel due to stress concentration and material properties and it is a major factor for predicting rock behavior in underground excavations. This phenomenon usually occurs in weak rocks under the influence of high pressures. The squeezing potential of rock mass in lot2 was observed to be very high and it varied from a low of 422.1 to a high of 778.4. These values were quite significant and it must be provided with suitable support lining.

Rock mass classification systems form the backbone of the empirical methods and were applied for the estimation of the support pressure and the design of tunnel support. The support pressure varied from a low of 3.81 MPa to a high of 6.48 MPa. These pressures were quite significant and warrant suitable lining in the tunnel.

There are different approaches for evaluating tunnel support system and the interaction between the surrounding rock mass and supporting system. The value of ESR is the so-called excavation support ratio. It ranged between 0.5 and 5 and has been presented in a tabular form in a number of research papers for various types of excavations. For the diversion tunnel, the excavation support ratio, ESR is defined as 1.6. Hence, for an excavation span of 5 m, the equivalent dimension, $D_e$, is 3.13. Barton et al. (1974)'s equivalent dimension, $D_e$, plotted against the value of $Q$, was used to define a number of support categories in a chart published in the original paper. The estimated support categories for each geotechnical zone along the diversion tunnel have been shown, and also, the recommended types of support have been presented.

## Conclusions

A comprehensive engineering geological assessment of rock masses has been carried out at the site of NWCT in north of Iran. The geotechnical properties of these rocks have been carefully assessed based on the laboratory and field investigations for diversion tunnel support design. The classification results of the rock masses of tunnel alignment based on RMR, Q, GSI, RQD, RMi, modified RMR and RSR classification systems, shows that the rock mass available in lot1 is of 12 lithology types namely, LI-SH1, LI-SH2, LI-SH3, LI-SH4, LI1, LI2, LI3, LI4, LI5, SI, CZ and FZ. Similarly, there are 21 lithology types namely, SH-ML1, SH-ML2, SH-ML3, MLI-SH1, ML-SH2, ML-SH3, ML-SH4, ML-SH5, SH-LS1, SH-LS2, SH-LS3, SH-LS4, LI2, LI3, LI4, LI5, LI6, LI-MA, LI-SH, CZ, FZ along the tunnel alignment in lot2. These regions are classified as very poor and fair rocks argillaceous and shale, sandstone and shale. Based on the results, the tunnel alignment was divided into 12 and 21(lot1 and lot 2) geotechnical zones and the tunnel support designs were accomplished by empirical method. The results obtained from different methods are in agreement with each other and for the safe tunnel support design, and using a combination of them is recommended.

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