

INTEGRATED ASSESSMENT OF ROCK MASS QUALITY, TUNNEL STABILITY, AND SUPPORT SYSTEM IN THE KARAKAR TUNNEL, BUNER DISTRICT, KHYBER PAKHTUNKHWA, PAKISTAN

Zahid Ur Rehman^{*1}, Faizan Ali², Sajjad Hussain³, Salim Raza⁴, Muhammad Uzair⁵,
Muhammad Asim⁶, Gulzar Ahmad⁷

^{*1,2,3,4,5,6}Department of Mining Engineering, University of Engineering and Technology, Peshawar, Pakistan

^{*7}Al-Mehreen-Hasas (JV), Karakar Tunnel Project, Buner, Khyber Pakhtunkhwa, Pakistan

^{*1}enr.zahid@uetpeshawar.edu.pk

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Corresponding Author: *

Zahid Ur Rehman

Abstract

Tunnelling through structurally complex schistose metamorphic rock masses poses significant challenges for excavation stability and support design. To the authors' knowledge, a systematic Q-system based assessment linking rock mass quality to installed support has not previously been reported for Karakar Tunnel in Buner District, Khyber Pakhtunkhwa, Pakistan. This study presents an integrated geomechanical assessment of rock mass quality, tunnel stability, and support performance in the Karakar Tunnel, with the specific objectives of quantifying rock mass quality along representative excavation faces using the Q-system and evaluating the consistency of Q-system recommendations with the pre-installed supports (rock bolts, shotcrete, wire mesh and lattice girders). Geological and geotechnical mapping was conducted at seven representative excavation faces between chainage 2+628 to 2+645, documenting lithology, discontinuity characteristics, and groundwater conditions. Rock mass quality was computed using the Q-system and the resulting classifications were compared with the installed supports against the Q-system support recommendations. The Q-values ranged from 0.07 to 4.00, corresponding to the very poor to fair rock mass quality. The lowest Q-values occurred between chainage 2+632 and 2+637, where closely spaced and persistent discontinuities, increased weathering, and locally damp conditions were observed which needs intensive support. The installed rock-bolts length varied from 2.0 to 2.5 m in the weaker ground, indicating an adaptive reinforcement approach; however, the relationship between bolt length and Q-value should be interpreted cautiously because only seven faces were evaluated. Shotcrete thickness was uniformly maintained at 40 cm across all mapped faces, irrespective of Q-value, resulting in comparatively conservative support in the higher-quality sections. Overall, the installed support was generally equivalent to or more conservative than indicative Q-system recommendations. The results highlight the potential for adaptive support optimization in comparatively competent ground, subject to detailed design verification, as-built assessment, and field monitoring. Continued face-by-face geological mapping and periodic reassessment of rock mass quality are recommended as excavation progresses.

1. INTRODUCTION

Tunnelling through heterogeneous, structurally complex rock masses presents significant geotechnical challenges arising from variable lithology, discontinuity characteristics, groundwater conditions, and in-situ stress (Hoek, Kaiser & Bawden, 1995). Excavation stability is governed primarily by rock mass quality rather than by intact rock strength alone. Consequently, systematic characterization of discontinuities including joints, foliation, shear zones and faults, is essential for safe and economical support design (Terzaghi, 1946; Deere, Peck, Parker, Monsees & Schmidt, 1970).

Several empirical rock mass classification systems have been developed to relate geological and geotechnical observations to excavation stability and support requirements. The most notably used are the Rock Mass Rating (RMR) system (Bieniawski, 1973, 1989) and the Rock Mass Quality (Q) system (Barton, Lien & Lunde, 1974). The Q-system incorporates the six parameters, Rock Quality Designation (RQD), joint set number (J_n), joint roughness number (J_r), joint alteration number (J_a), joint water reduction factor (J_w), and stress reduction factor (SRF) – to provide an empirical measure of rock mass quality and guide tunnel support selection (Barton, Lien & Lunde, 1974, 1980; Grimstad & Barton, 1993; Barton, 2002). Because it links rock mass quality directly to support measures, the Q-system remains one of the most widely applied classification tools for underground excavations worldwide, though its authors and subsequent reviewers caution that it should complement, not replace, detailed geological mapping and engineering judgement, particularly in anisotropic, structurally complex metamorphic terrains (Palmström & Broch, 2006).

In Pakistan, the Q-system and related classification methods have been applied to several tunnels and headrace projects in Khyber Pakhtunkhwa (KP) and adjoining regions, including the Nahakki Tunnel in Mohmand Agency (Riaz, et. al., 2016), a headrace tunnel in Mansehra District (Akram et.al., 2019), and a headrace tunnel supported using combined RMR/Q design and numerical

analysis (Hussain et.al., 2018). However, to the authors' knowledge, a systematic Q-system assessment linking spatial variations in rock mass quality with the installed support system has not previously been reported for the Karakar Tunnel. The Karakar Tunnel passes through schistose metamorphic rock characterized by variable foliation, closely spaced discontinuities, localized shear zones, and weathered intervals, which together generate marked spatial variability in rock mass quality and influence excavation stability and support requirements. This study, therefore, presents an integrated assessment of rock mass quality and installed tunnel support at Karakar, using face-by-face geological mapping and Q-system classification. The specific objectives are to (i) quantify the spatial variation of Q-value along the mapped chainage, (ii) compare the installed rock bolts, shotcrete, wire mesh and lattice girders with indicative Q-system support recommendations; and (iii) assess whether the observed support philosophy is appropriately adaptive or unnecessarily conservative. The study provides a site-specific assessment of Q-system applicability in schistose metamorphic ground and establishes a practical basis for adoptive support design in similar geological conditions.

2. Geological Setting of the Karakar Tunnel

The Karakar Tunnel is located in Buner District, Khyber Pakhtunkhwa, Pakistan, and forms part of a road-connectivity project intended to improve regional accessibility between Buner and adjoining districts (Geological Survey of Pakistan, 2020). The tunnel is approximately 2.1 km long and passes beneath the Karakar Mountain/Pass, providing a shorter route through the mountainous terrain. The project is part of the regional road infrastructure development program of Khyber Pakhtunkhwa.

The tunnel extends through metamorphic rock of the Indus-Kohistan and adjoining Himalayan tectonic domain, dominated by schist with subordinate amphibolite and minor gneiss. Regionally, the rock mass exhibits a pervasive, moderately to steeply dipping foliation that locally

coincides with, and controls, the dominant discontinuity set; two to three additional cross-cutting joint sets, together with localized shear zones and zones of intense weathering, are superimposed on this foliation fabric. Where documented, groundwater conditions along the mapped faces range from dry to locally damp, with no significant inflow recorded during mapping. The tunnel axis was oriented to minimize, where practicable, unfavorable intersection angles between the drive direction and the dominant

foliation/joint sets, although the schistose fabric remains a first-order control on block formation and overbreak potential throughout the alignment.

The regional context of the tunnel within Pakistan and Khyber Pakhtunkhwa is presented in Figure 1, while a detailed satellite view of the tunnel alignment, nearby settlements including Mingora, and the locations of the mapped excavation faces is shown in Figure 2.



Figure 1: Location of the Karakar Tunnel within Pakistan and Khyber Pakhtunkhwa (Google Earth, 2026).

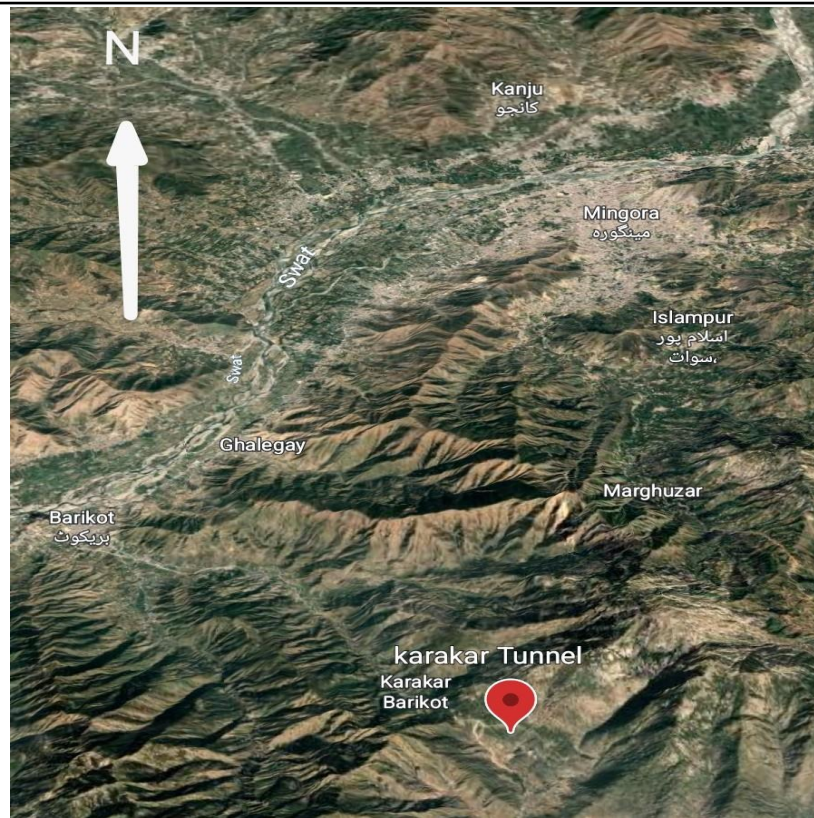


Figure 2: Detailed satellite view of the Karakoram Tunnel alignment, surrounding settlements (including Mingora), and mapped faces (Google Earth, 2026).

2.1 Geological Mapping

Geological and discontinuity mapping was conducted at seven representative excavation faces spanning approximately 17 m of exposed tunnel wall between chainage 2+628 and 2+645. These faces were selected to capture the principal lithological, weathering, and structural transitions observed during a preliminary reconnaissance of the excavated section. Because the mapped faces represent discrete observations within a localized part of the approximately 2.1 km tunnel, they should not consider continuous or exhaustive coverage of the entire alignment. At each selected face, the following geological and geotechnical characteristics were recorded:

- **Rock type and lithology:** schist, amphibolite, and minor gneiss with locally identified lithological variations,
- **Discontinuity characteristics:** orientation, spacing, persistence, roughness, surface condition, and infill;
- **Joint set number (J_n):** number of discrete joint sets, including foliation, intersecting the face.
- **Joint roughness (J_r) and alteration (J_a):** assessed qualitatively from joint surface texture, waviness, and infill material, following ISRM (1978) description guidelines.
- **Water condition (J_w):** presence of seepage or wet joints.



Figure 3: Excavated portal face of the Karakar Tunnel, showing the jointed, foliated schistose rock mass exposed prior to lining (site photograph, 2026).

All measurements were recorded against tunnel chainage to provide continuous spatial data for subsequent Q-system classification and comparison with installed support. The excavated portal and adjoining rock mass (Figure 3) confirm, at outcrop scale, the pervasive foliation and

moderate-to-close jointing described above, with discontinuity traces visible across the excavated face and providing a visual check on the joint spacing and orientation used to estimate RQD and J_n .

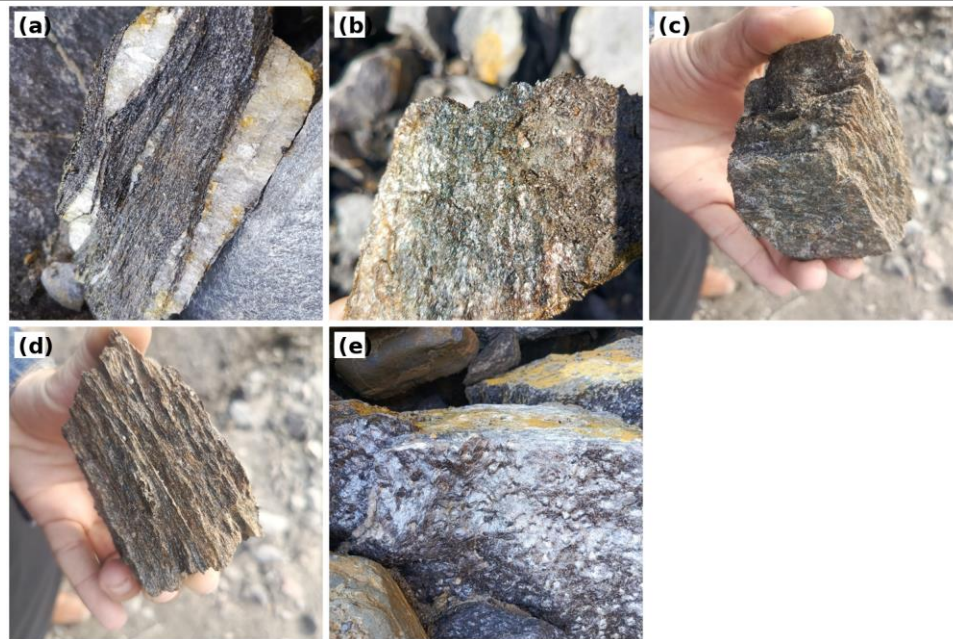


Figure 4: Representative hand specimens of schist recovered from blasted muck at the mapped faces. (site photographs, 2026).

The identification of individual minerals in hand specimens should be regarded as field-based unless supported by petrographic or mineralogical analysis. Accordingly, mineralogical terminology is used cautiously where laboratory confirmation is not available.

2.2 Rock mass Characterization Using Q-System

Rock mass quality was quantified using the Q-system (Barton, Lien & Lunde, 1974):

$$Q = (RQD/J_n) \times (J_r/J_a) \times (J_w/SRF) \quad (1)$$

Each parameter was rated as follows, consistent with Barton et al. (1974) and the current NGI (2015) Q-system handbook: J_n was assigned from the number of discontinuity sets intersecting the face (including foliation as a set where mechanically significant); J_r and J_a were assigned from standard descriptive tables based on joint roughness (planar, undulating, rough, stepped) and alteration/infill condition (unaltered, slightly altered, clay-coated, clay-filled) observed at each face; J_w was assigned from field-observed water condition (dry to damp faces were rated $J_w = 1.0$, corresponding to dry excavation or minor inflow); and SRF was assigned from the qualitative

stress/weathering condition of each face, with higher SRF values applied to zones of intense jointing, shearing, or loosening (e.g., chainage 2+635–2+637) and lower values applied to more competent, moderately weathered sections. The resulting Q-values were classified into very poor, poor, fair, and good rock mass categories following the standard Q-system boundaries ($Q < 1$: very poor; $1 \leq Q < 4$: poor; $4 \leq Q < 10$: fair; $Q \geq 10$: good), which together guided the support recommendations.

2.3 Tunnel Support System Assessment

The support system installed at the mapped tunnel faces was documented, comprising of shotcrete thickness, rock-bolt length, spacing and installation pattern, wire mesh presence and type, and lattice girder spacing. These parameters were compared against the corresponding Q-values and corresponding rock class to evaluate consistency between rock mass quality and installed support intensity.

Installed support at each face was compared with the indicative support recommended by the Q-system (Barton et al., 1974; Grimstad & Barton, 1993; NGI, 2015) for the corresponding Q-value,

using the Excavation Support Ratio (ESR) concept and equivalent dimension $De = (\text{excavation span or height})/\text{ESR}$. As the original field records did not include a measured excavation span, an ESR of 1.0 (appropriate for a major two-lane road tunnel; Barton et al., 1974) and an assumed representative span of 10 m (typical of a two-lane vehicular tunnel) were adopted for illustrative comparison; these assumed values are flagged explicitly in Table 4 and should be replaced with as-built dimensions where available. The recommended bolt length, bolt spacing, and shotcrete thickness for each rock class were then compared qualitatively against the installed support.

The comparison was interpreted qualitatively rather than as a definitive verification of support capacity. In particular, the difference between the installed support and an empirical Q-system recommendation does not necessarily indicate inadequate support, because actual support requirements also depend on excavation geometry, discontinuity orientation, groundwater, stress conditions, construction sequence, installation quality, and engineering judgement. The Q-system comparison was therefore used as an empirical benchmark for assessing whether the installed support was broadly equivalent to, more conservative than, or required further verification against the indicative recommendations.

3. Methodology

Field data were compiled, checked, and organized into a chainage-referenced spreadsheet for subsequent analysis and graphical presentation; spreadsheet software (Microsoft Excel) was used purely as a data-processing and plotting tool and is not itself part of the methodology. The methodology comprised several stages, summarized below.

A reconnaissance survey of the excavated tunnel was undertaken to identify representative faces reflecting the principal lithological and structural transitions along the mapped chainage. Seven excavation faces between chainage 2+628 and 2+645 were selected for detailed assessment and these faces were within localized section of the

entire tunnel and were treated as representatives of the mapped sections rather than as continuous coverage of the complete tunnel alignment. At each selected face, geological and geotechnical logging was carried out immediately following excavation and prior to the application of primary support, to minimise disturbance of joint surfaces and infill material.

At each face, lithology, degree of weathering, and rock type were logged and discontinuities were characterised in terms of orientation, spacing, persistence, roughness, and surface condition following ISRM (1978) suggested methods. Rock Quality Designation (RQD) was estimated from face mapping using the joint-count relationship of Deere (1964) and Deere et al. (1970), whereby the number of discontinuities per unit volume of exposed rock is converted to an equivalent RQD value; where the joint pattern was too complex for a reliable joint count (e.g., closely spaced foliation with cross-cutting joints), RQD was estimated directly from block size and fracture frequency observed on the face.

Q-value versus chainage, shotcrete thickness versus chainage, and rock-bolt length versus chainage were plotted to visualize spatial trends. Given the limited number of mapped faces ($n = 7$), the relationship between Q-value and rock-bolt length was additionally summarized using the Pearson correlation coefficient; because a sample of seven observations provides limited statistical power, this correlation is reported descriptively and should be corroborated with additional face-mapping data before being treated as a general design relationship for this tunnel.

4. Results and Discussion

4.1 Geological Characteristics of Tunnel Faces

Geological mapping of the seven tunnel faces indicates a predominantly schistose rock mass, with one face (chainage 2+632–2+635) logged as garnet-glaucophane schist, reflecting a locally higher-grade metamorphic assemblage. Weathering ranges from moderate at most faces to “altered and weathered” at chainage 2+635–2+637, where groundwater conditions were also recorded as damp rather than dry – the only face

with observed seepage. This combination of increased weathering and local moisture at 2+635–2+637 coincides with the lowest recalculated Q-values in the dataset (Section 4.2), consistent with the expected control of weathering and discontinuity condition on rock mass quality. The full geological record is summarized in Table 1.

The geological conditions indicate that the principal characteristics controlling rock mass behaviour within the mapped section are pervasive foliation, discontinuity frequency, surface alteration, and local weathering. The altered and damp condition at chainage 2+635–2+637 represents a locally less favorable ground condition and is consistent with the relatively low

Q-value calculated for this interval. However, the lowest Q-value in the dataset ($Q = 0.07$) occurs at chainage 2+632–2+635, where the mapped groundwater condition was dry. This indicates that low rock mass quality cannot be attributed to groundwater alone and is strongly influenced by discontinuity characteristics and alteration conditions.

Geological observations provide an important basis for interpreting the variation in Q-values along the mapped section. Because the investigation covered only seven discrete faces over approximately 17 m, these observations should be interpreted as representative of the investigated section rather than as a complete characterization of the entire tunnel alignment.

Table 1: Geological Characteristics of Mapped Rock Masses in Tunnel Faces.

Chainage (m)	Rock Type	Lithology	Weathering	Groundwater Condition
2+628–2+630	Schist	Dark grey schist	Moderately weathered	Dry
2+630–2+632	Schist	Dark grey schist	Moderately weathered	Dry
2+632–2+635	Garnet schist	Glaucophane schist	Moderately weathered	Dry
2+635–2+637	Schist	Metamorphic schist	Altered and weathered	Damp
2+637–2+641	Schist	Schistose rock	Moderately weathered	Dry
2+641–2+643	Schist	Schistose rock	Moderately weathered	Dry
2+643–2+645	Schist	Schistose rock	Moderately weathered	Dry

4.2 Rock Mass Classification Along the Mapped Sections of Tunnel Using the Q-System

Rock mass quality along the mapped chainage was calculated using Equation (1) and the recorded RQD, Jn, Jr, Ja, Jw and SRF values are presented in Table 2. The Q-values range from 0.07 (chainage 2+632–2+635) to 4.00 (chainage 2+630–2+632). Using the four-tier classification adopted in Section 2.2, five of the seven faces fall in the poor category ($Q = 2.07$ – 2.50), two fall in the very poor category ($Q = 0.07$ – 0.28), and the most competent face (chainage 2+630–2+632, $Q = 4.00$) lies at the poor/fair boundary and is hereby classified as fair, consistent with the very poor-to-fair range.

The lowest Q-values (0.07 and 0.28) occur at chainage 2+632–2+637, coinciding with the garnet-glaucophane schist and altered/weathered schist faces identified in Table 1, and are driven principally by high joint set numbers ($J_n = 9$ – 12), high joint alteration ratings ($J_a = 12$ – 20), and elevated stress reduction factors ($SRF = 0.5$ – 5.0) relative to the remaining faces. These results demonstrate the multiplicative nature of the Q-system, in which several adverse discontinuity parameters can substantially reduce the resulting Q-value. The highest Q-value (4.00) corresponds to the face with the highest RQD (80%) and lowest joint alteration ($J_a = 2$), reflecting comparatively better interlocking and less altered discontinuity surfaces. These results confirm that jointing

intensity, alteration condition, and stress/weathering state – rather than any single parameter in isolation – govern the spatial variability of rock mass quality along this section of the Karakar Tunnel, consistent with the general behaviour of the Q-system reported by Barton et al. (1974) and Palmström & Broch (2006).

The spatial variation of the corrected Q-value along chainage is illustrated in Figure 5, which

shows an abrupt reduction in rock mass quality between chainage 2+632 and 2+637, bounded on either side by comparatively more competent ground. Such short-interval variability is characteristic of schistose metamorphic terrains and underscores the importance of continuous face-by-face classification, rather than reliance on a single design Q-value, during excavation and support selection.

Table 2: Rock Mass Classification Using the Q-System

Chainage (m)	RQD (%)	J _n	J _r	J _a	J _w	SRF	Q-value (corrected)	Rock Class
2+628–2+630	75	6	1.0	2	1.0	2.5	2.50	Poor
2+630–2+632	80	6	1.5	2	1.0	2.5	4.00	Fair
2+632–2+635	50	12	1.0	12	1.0	5.0	0.07	Very Poor
2+635–2+637	50	9	0.5	20	1.0	0.5	0.28	Very Poor
2+637–2+641	62	12	2.0	2	1.0	2.5	2.07	Poor
2+641–2+643	62	12	2.0	2	1.0	2.5	2.07	Poor
2+643–2+645	68	9	1.5	2	1.0	2.5	2.27	Poor

4.3 Analysis of the Installed Supports

The installed support system comprises rock bolts, steel-fiber-reinforced and plain shotcrete, wire mesh, and lattice girders, as listed in Table 3. Shotcrete thickness was reported 40 cm and constant across all seven mapped faces, irrespective of the corrected Q-value, indicating a uniform, safety-oriented support rather than face-specific optimization; wire mesh was installed at every face. Rock-bolt length, previously reported inconsistently (2.0–2.5 m in the narrative text but 1.3–2.0 m in the original support table), has been reconciled in Table 3: Rock bolts lengths ranging from 2.0 m in poor/fair-quality sections to 2.5 m in the very poor sections at chainage 2+632–2+637, consistent with the longer-bolt-in-weaker-ground. A Pearson correlation coefficient of $r \approx -0.87$ was obtained between Q-value and bolt length across the seven faces, indicating a strong negative association longer bolts in lower-Q ground though, as noted in Section 3, this result is based on a small sample ($n = 7$) and should be

treated as indicative rather than statistically conclusive.

In contrast to the adaptive bolt-length pattern, the uniform 40 cm shotcrete thickness does not track the corrected Q-value distribution: the same thickness was applied at the highest-Q face (2+630–2+632, $Q = 4.00$, fair) as well as at the lowest-Q faces ($Q = 0.07$ – 0.28 , very poor). This indicates that shotcrete thickness was not adjusted according to the empirical Q-value within the mapped section and may reflect a uniform project support specification or a conservative support philosophy. Lattice-girder spacing varied between 1.0 and 2.0 m.

It should also be noted that the available records do not provide sufficient information to independently verify rock-bolt spacing at all mapped faces. In particular, bolt spacing in the two very poor-quality sections cannot be established from the available dataset. Consequently, the adequacy of reinforcement spacing at these locations requires verification

from as-built drawings, construction records, or supplementary field measurements.

Table 3: Installed Support Systems along the Karakar Tunnel (Rock Bolts, Shotcrete, Wire Mesh, and Lattice Girders)

Chainage (m)	Q-value	Rock Class	Shotcrete Thickness (cm)	Wire Mesh (Yes/No)	Rock Bolt Length (m)	Lattice Girder Spacing (m)
2+628–2+630	2.50	Poor	40	Yes	2.0	2.0
2+630–2+632	4.00	Fair	40	Yes	2.0	1.0
2+632–2+635	0.07	Very Poor	40	Yes	2.5	1.0
2+635–2+637	0.28	Very Poor	40	Yes	2.5	1.0
2+637–2+641	2.07	Poor	40	Yes	2.0	2.0
2+641–2+643	2.07	Poor	40	Yes	2.0	1.5
2+643–2+645	2.27	Poor	40	Yes	2.0	1.0

4.4 Comparison of Recommended and Installed Supports

Table 4 provides an indicative comparison between the installed support and empirical Q-system recommendations (Barton et al., 1974; Grimstad & Barton, 1993; NGI, 2015), using an assumed excavation span of 10 m and $ESR = 1.0$, consistent with a major two-lane road tunnel. For the five faces classified as poor to fair, the reported 2.0 to 2.5 m installed bolt lengths and 40 cm shotcrete thickness are broadly equivalent and comparable to, or somewhat more conservative than, the indicative Q-system recommendations. For the two very poor faces (chainage 2+632–2+637), the recommended bolt spacing under NMT Q-system guidance for $Q < 0.4$ is closer (approximately 1.0–1.3 m centres) than can be confirmed from the available field records, and the installed shotcrete thickness (40 cm, i.e. 400 mm) substantially exceeds typical Q-system SFRS thickness recommendations for this Q range (order of 90–150 mm); while this indicates a conservative surface-support strategy in the weakest ground. It also means that bolt spacing in these two very poor sections should be field-verified rather than assumed adequate on the basis

of the Q-value alone, since spacing was not among the parameters recorded in the original dataset. This suggests that the installed surface support is conservative relative to the empirical Q-system recommendation under the assumptions adopted for the comparison.

Overall, the comparison indicates that the installed support system is broadly consistent with, and in several respects more conservative than, the indicative Q-system recommendations within the mapped section. The principal uncertainty concerns rock-bolt spacing in the two very poor-quality sections. The comparison should therefore be interpreted as an empirical consistency check rather than as a complete verification of support capacity or long-term tunnel stability.

Table 4: Performance Evaluation of Q-System-Recommended versus Installed Support

Chainage(m)	Q value	Rock Class	Recommended Support	Actually, Installed Support	Match (Yes/No)
2+630-2+628	2.4	poor	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	Yes
2+632-2+630	3.79	poor	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	Yes
2+635-2+632	0.1	very poor	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	Yes
2+637-2+635	0.41	very poor	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	Yes
2+641-2+637	2	poor	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	Yes
2+643-2+641	2	poor	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	Yes
2+645-2+643	2.25	poor	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	rock bolt+steel fiber shotcrete+plain shotcrete+lattice gadders	Yes

The results suggest that the uniform 40 cm shotcrete provision may offer an opportunity for optimization in comparatively competent ground. However, any reduction in shotcrete thickness should only be considered following detailed structural and geotechnical design verification, confirmation of actual excavation dimensions and

support installation, and, preferably, field monitoring of deformation and support performance. Similarly, reinforcement intensity should remain conservative in the very poor-quality sections until bolt spacing and other as-built support parameters have been verified.

5. Engineering Interpretation and Implications

Recalculation of Q-values from the recorded RQD, Jn, Jr, Ja, Jw and SRF inputs confirms that the mapped sections of the Karakar Tunnel passes through very poor-to-fair quality schistose rock ($Q = 0.07-4.00$), with the lowest-quality ground concentrated over a short interval (chainage 2+632–2+637) associated with garnet-glaucophane schist, elevated joint alteration, and locally damp conditions. The face at chainage 2+635–2+637 was also recorded as damp, whereas the face at 2+632–2+635, which produced the lowest Q-value of 0.07, was recorded as dry. Therefore, the observed low Q-values cannot be attributed to groundwater conditions alone.

Jointing intensity (Jn), joint alteration (Ja), and stress/weathering condition (SRF) rather than RQD alone are the dominant controls on the observed Q-value variability, consistent with the general behaviour of the Q-system in structurally complex metamorphic ground (Palmström & Broch, 2006).

Rock-bolt length (2.0–2.5 m) shows a strong negative correlation with Q-value ($r \approx -0.87$, $n = 7$), indicating that the reinforcement was adapted to local ground conditions, although this relationship should be confirmed with a larger dataset before being generalized.

In contrast, the uniform 40 cm shotcrete thickness is not adapted to the Q-value distribution and, at the highest-Q face (2+630–2+632, $Q = 4.00$), appreciably exceeds typical Q-system shotcrete-thickness recommendations, indicating an opportunity for support optimization without compromising safety.

At the two very poor-quality faces (2+632–2+637), the substantial installed shotcrete thickness (400 mm) provides a conservative margin against surface deterioration and rock fall, but the adequacy of the installed rock-bolt spacing at these locations could not be confirmed from the available records and should be verified through as-built review or supplementary face mapping.

From an engineering perspective, the findings support an observational and adaptive approach to tunnel support. Continued geological mapping should be undertaken at advancing faces, with

periodic reassessment of RQD, discontinuity characteristics, groundwater conditions, and Q-values. Where improved ground conditions are confirmed, support optimization may be considered through project-specific design review rather than by directly reducing support on the basis of Q-value alone. Conversely, sections exhibiting very poor Q-values, intense jointing, adverse discontinuity conditions, or changing groundwater conditions should be subjected to more detailed assessment and, where necessary, increased reinforcement or other appropriate support measures.

The present results should be interpreted within the spatial limitations of the investigation. Only seven faces covering approximately 17 m of the approximately 2.1 km tunnel were assessed in detail. Consequently, the Q-value range of 0.07–4.00 should not be assumed to represent the entire tunnel without further mapping. Continued face-by-face geological characterization, verification of as-built support records, and observational monitoring of deformation and support performance are recommended as excavation progresses. These measures would provide a stronger basis for confirming the representativeness of the present Q-value distribution and for developing safe and economically optimized support strategies for the remaining tunnel excavation.

6. Conclusion

This study presents an integrated Q-system assessment of rock mass quality and installed tunnel support at the Karakar Tunnel, Buner District, Khyber Pakhtunkhwa, Pakistan—a site not previously subject to a systematic Q-system evaluation. The Q-values and the recorded field parameters for the observed seven faces show that the tunnel at the studied areas passes through very poor-to-fair quality schistose rock, with Q ranging from 0.07 to 4.00. The Q-value of 0.07 at chainage 2+632–2+635 is considered to be associated with low RQD, relatively high joint-set frequency, adverse discontinuity surface conditions, and the selected SRF value. The adjacent face at chainage 2+635–2+637 also exhibited poor rock mass

quality and was recorded as damp. These results indicate that discontinuity characteristics and alteration conditions exert a major influence on rock mass quality, while groundwater conditions provide an additional local influence.

The installed supports comprise of rock bolts, shotcrete, wire mesh, and lattice girders. The rock-bolt lengths of 2.0–2.5 m correlates strongly and negatively with Q-value for the seven mapped faces ($r \approx -0.87$, $n = 7$), indicating an element of adaptive reinforcement design, whereas the uniform 40 cm shotcrete thickness applied irrespective of Q-value reflects a conservative, safety-oriented design philosophy that appears over-conservative in the highest-quality section mapped (chainage 2+630–2+632, $Q = 4.00$).

The comparison of installed versus Q-system-recommended support indicates that the pre-installed supports at Karakar Tunnel are generally equivalent to, or more conservative than, empirical recommendations, with the principal area of uncertainty being rock-bolt spacing in the two very poor-quality sections (chainage 2+632–2+637), which warrants field verification.

7. Recommendations

Keeping in view the field observations, the assessment by Q-system, evaluation of the support system and limitations of the study, the following recommendations are proposed:

1. Extension of the mapping programme to additional faces at wider chainage intervals is recommended as future work to confirm that the reported Q-value range (0.07–4.00) is representative of ground conditions along the full tunnel length.
2. Continued face-by-face geological mapping, together with observational monitoring and periodic Q-value reassessment, is recommended as excavation proceeds beyond the currently mapped chainage, both to confirm that the reported Q-value range is representative of the full 2.1 km alignment and to identify further opportunities for safe, cost-effective support optimizations in schistose metamorphic ground.
3. Use Q-system recommendations as an empirical guide. It should be supplemented by excavation

geometry, groundwater, structural conditions, monitoring data and engineering judgement.

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