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Stress reduction factor characterization for highly stressed jointed rock based on tunneling data from Pakistan

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ABSTRACT

Stress Reduction Factor (SRF) which is the most difficult parameter to characterize in Q (Tunneling Quality Index) system is targeted in this study for highly stressed jointed rock in tunneling. As there are no criteria for this purpose in empirical tunnel design, an attempt has been made in this regard using 542 NATM tunnel sections mapping of four tunneling projects from Pakistan. These already supported sections are used for the back calculation of SRF. SRF values measured from the already available equations did not match with the calculated values from back analysis based on mining cases and without considering the magnitude of rock fracturing. Empirical equations proposed here are based on the data of these tunnel sections and match well with the calculated values. Two types of calculated SRF (SRF_Q and SRF_{QC}) from back analysis are dependent on the intact rock strength. In proposed equations, SRF_Q (SRF calculated from original Q-system equation) is dependent on the relative block size and the ratio of intact rock strength to major principal stress. The effect of intact rock strength on SRF is also determined from the available data by plotting SRF_{QC} (SRF calculated from normalized Q-system equation) against relative block size for different ranges of UCS and ratio of intact rock strength to major principal stress. The proposed equations are applied to calculate rock quality (Q or Q_c) for highly stressed cases of the jointed rock mass of head race and diversion tunnels of another four hydropower tunnels from Pakistan with various cross sections. The empirical support design of these tunnels based on Q or Q_c are numerically analyzed and verified in term of total displacement and plastic zones before and after support installation and the performance of liner was also validated based on capacity diagrams.

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1. INTRODUCTION

On priority basis, the first out of two foremost aspects which required focusing in the design stage of an underground projects like tunnel is a precise estimate of the probable ground condition and the possible behavior of the surrounding due to excavation. The second one is the cost and safety of the excavation which define the support system. The first feature can be addressed by rock mass characterization and the second one can be achieved by the proper classification of rock mass excavation environment which is used as a designing tool in tunneling. As per definition (Palmstrom 2001), rock mass characterization is the critical stage in the rock mass classification. In Rock engineering, Rock Mass Rating(RMR) system (Bieniawski 1989) (Celada 2014) and Rock Quality system(Q) (Barton 1974) (Barton 2002) have gained eclectic attention and are the most widely used empirical tool for tunnel design. Comparing the application of two systems for tunneling in high stress environment, Q-system have preference due to Stress Reduction Factor (SRF) for the purpose.

Q-system was developed based on the tunneling cases for hard & jointed rock (NGI 2015). The applications of Q-system are more in jointed rock (Palmstrom 2006) and work best for a tunnel with the Equivalent Dimension (D_e) from 2.5 to 30 and Q value from 0.1 to 40 (Palmstrom 2002). Q-system is comprised of 7 parameters as shown in Eq. (1) and Eq. (2) in which SRF is the most difficult parameter and hence should be characterized with great care.

The initial maximum value of SRF for the condition of competent rock with rock stress problem was 20 (Barton 1974). A first relationship for SRF characterization was based on principal field stress ratio (k), cover depth of tunnel (H) and uniaxial compressive strength of intact rock (σ_c) (Kirsten 1988). The key changes were made for SRF in hard massive rock in high stress environment and the increase were from 20 to 400 as shown in Table-1. These changes were based on the relation between RQD/J_n and SRF in hard rock under stress and no real stress problem was experienced for low RQD/J_n (Grimstad 1993). The ratio RQD/J_n is the relative block size which is suitable for distinguishing massive, rock-burst-prone rock and the rock-burst-prone rock has RQD/J_n ratio from 25-200 while typical jointed rock has an RQD/J_n of 10 (Barton 2002).

The last three sub-categories of Table-1 are suggesting SRF values for massive rock & do not recommend any value for jointed rock under same conditions. Stability problem in tunnel excavation is either from structural features or from high stress to strength condition and rock burst condition prevail when the rock surrounding the excavation is massive (Bhasin 1996). The underground excavation in highly stressed jointed rock is likely to be less harmful for rock bursting than massive (Palmstrom 1995) and hence a lower SRF value should be used. Destress blasting is successfully used as one of rock burst prevention method by fracturing the intact rock, decreasing the effective modulus of elasticity of the rock mass and its ability to carry high stress as a result stress concentration migrates and the risk of rock burst decrease (Konicek 2013) (Mazaira 2015) and the method is used in Scandinavia (Palmstrom 1995).

A similar relation like Kirsten was proposed for calculation of SRF in highly stressed jointed rock mass for the Australian underground mines where σ_1 is horizontal (Peck 2000). From the experience of head race tunnel of Nathpa Jharki project from India, SRF was calculated from H (overburden height), σ_c, J_r (rating for joint roughness) and J_n (rating for joint number) for moderately jointed rock falling in the category of competent rock having rock stress problem (Kumar 2004).

Table-1 SRF relation to stress-strength ratios for competent rock having rock stress problems (Grimstad 1993)

	Stress level	**σ_c/σ_1**	**σ_θ/σ_c**	**SRF* (old)**	**SRF** (new)**
1	Low stress, near surface, open joints	>200	<0.01	2.5	2.5
2	Medium stress, favorable stress condition	200-10	0.01-0.3	1	1
3	High stress, very tight structure. Usually favorable to stability, may be unfavorable to wall stability	10-5	0.3-0.4	0.5-2	0.5-2
4	Moderate slabbing after > 1 hour in massive rock	5-3	05-0.65	5-9	5-50
5	Slabbing and rock burst after a few minutes in massive rock	3-2	0.65-1.0	9-15	50-200
6	Heavy rock burst (strain-burst) and immediate dynamic deformation in massive rock	<2	>1.0	15-20	200-400

* Barton (1974) ** Grimstad (1993)

In this paper, two sub categories (4 & 5) of Table-1 were focused from 542 sections for jointed rock of 4 tunnel projects with variable geological and geotechnical data. Back analysis was used for the calculation of SRF from the geotechnical data along with installed support and span of the tunnel with the help of Q-system support chart. As the calculated SRF values from back analysis are different from the results of already published equations, the calculated SRF values were plotted against RQD/J_n and σ_c/σ_1. Equations for the SRF calculation were proposed from RQD/J_n and σ_c/σ_1. The same equations were used for tunnel support of other hydropower projects in the region and the recommended support were numerically evaluated.

2. PROJECTS DESCRIPTION

A brief description about the four excavated projects named Lawari Tunnel (LT), Kohat Tunnel (KT), Neelum Jhelum Hydropower (NJHP) project and Golen Gol Hydropower (GGHP) project are given. The 8509m long Lawari Rail Tunnel (LRT) project now Modified Road Tunnel (MRT) is an ultra-long road tunnel in Pakistan. As per the earlier plan, a rail tunnel was proposed with small cross sectional area called LRT, but later, after the completion of excavation work, using the same tunnel for trade with the Central Asia, enlargement of the tunnel was decided for two-way traffic called MRT. This MRT was completed in start of 2016. Five engineering geological units were summarized along the tunnel.

The 1885m long 2 lanes KT is the first long road tunnel which was opened to traffic in June 2003 and has been constructed in a difficult geological setting. Limestone & shale are major rock types along the tunnel.

A total length of 28.5 km head race tunnel of NJHP project with a combination of single and twin tunnels with variable cross sections passing through Murre formation. Murre formation comprises alternative bed of sandstone (SS), siltstone, mudstone and shale. Twin tunnels excavated in sandstone with conventional method; the sandstone is subdivided into SS-1 and SS-2 based on their properties.

Out of the 3.8 km, the maximum part of the GGHP project head race tunnel (2775m approx.) is excavated in Granite rock and the remaining part of tunnel in metamorphosed rock types which are Quartz Mica Schist, Marble and Calcareous Quartzite.

3. Q-PARAMETERS FOR EXCAVATED PROJECTS

Tunneling quality index also known as Q-system was presented (Barton 1974) and various revisions for the system have been made (Grimstad 1993) (Barton 2002), which categorize the rock mass into 9 classes. The index of the system ranges from 0.001 to 1000 on the logarithmic scale and is calculated as:

$$Q=\left(\frac{RQD}{J_n}\right).\left(\frac{J_r}{J_a}\right).\left(\frac{J_w}{SRF}\right) \tag{1}$$

Where RQD is the Rock Quality Designation in a selected domain, J_n is the number of joint sets in the same selected domain, J_r is the rating for the roughness of the least favorable of these joint sets or filled discontinuities, J_a is the rating for the degree of alteration or clay filling of the least favorable joint set or filled discontinuity, J_w is the rating for the water inflow and pressure effects, which may cause outwash of discontinuity infillings, and SRF is the rating for faulting, for stress-strength ratios in hard rocks, for squeezing or for swelling (Barton 2002).

The role of σ_c is significant in rock mass properties and therefore, a normalization factor was applied to Eq. 1 and modified to Q_c (Barton 2002).

$$Q_c=\left(\frac{RQD}{J_n}\right).\left(\frac{J_r}{J_a}\right).\left(\frac{J_w}{SRF}\right).\left(\frac{\sigma_c}{100}\right) \tag{2}$$

New Austrian Tunneling Method (NATM) was used for the tunnel construction of these projects. Rock masses were classified and supports were applied according to the behavior of the rock. In NATM guidelines, the geological documentation is a part of the tunnel construction with conventional excavation (Schubert 2003) and hence the available data is sufficient for characterization of RQD, J_n, J_r, J_a, J_w and σ_c.

The Q-system parameters were characterized for those selected sections where relative block size (RQD/J_n) $\leq$ 13 and σ_c/σ_1 is in the range of 2-5. The summarized details of the data are shown in Fig. 1 and input parameters in Table-2.

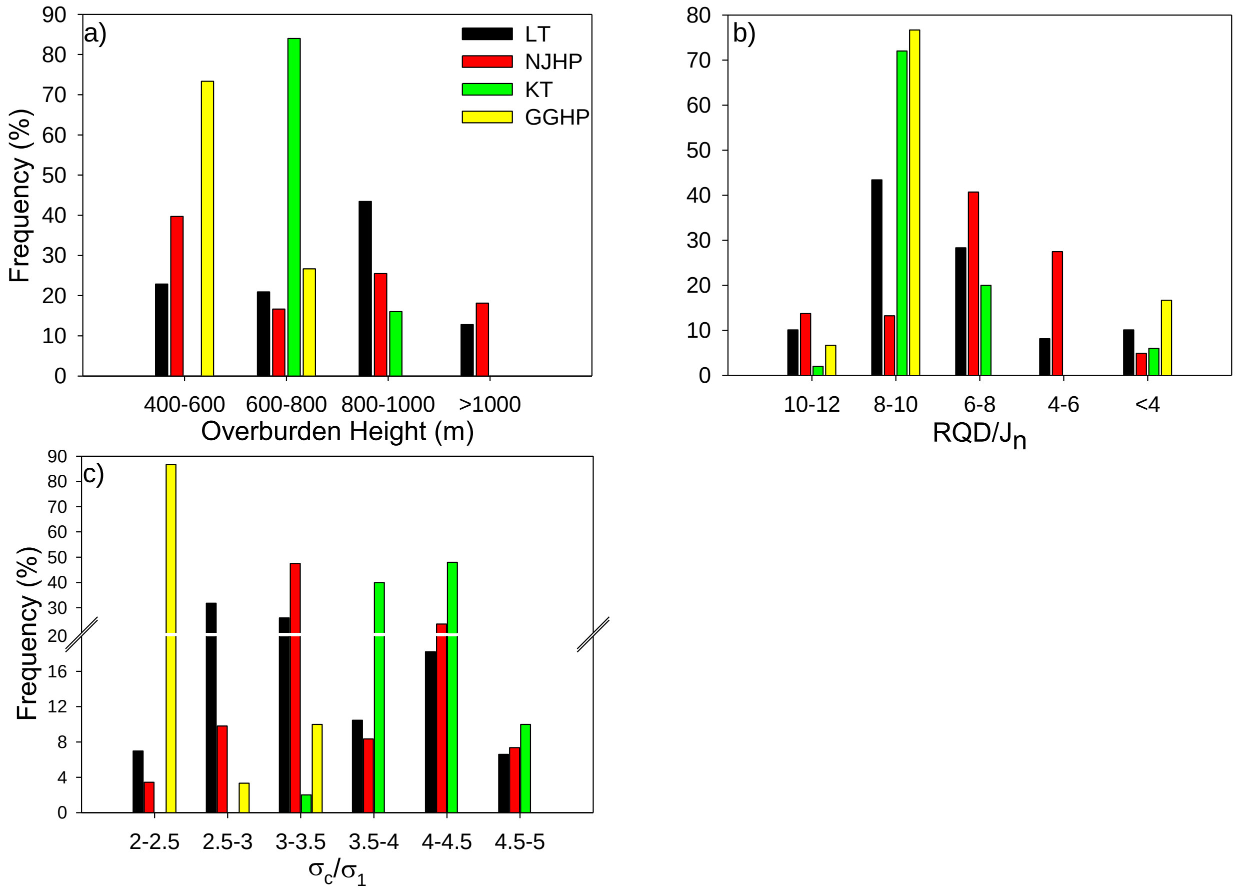


Fig. 1 Percentage frequency of a) Overburden Height b) RQD/J_n c) σ_c/σ_1 for four tunnel projects

Table-2 Summarized detail of some input parameters for back analysis for four tunnel projects

<table>
<tr><th>Project</th><th>No. of Sections</th><th colspan="8">σ_c (MPa)</th><th>J_r</th><th>J_a</th><th>J_w</th><th colspan="2">Tunnel Span (m)</th></tr>
<tr><td>LT</td><td>258</td><td>75</td><td>65</td><td colspan="2">60</td><td colspan="2">56.25</td><td colspan="2">37.5</td><td>3</td><td>1</td><td>1</td><td>11.17</td><td>14.17</td></tr>
<tr><td rowspan="3">NJHP</td><td rowspan="3">204</td><td rowspan="3">100</td><td rowspan="3">90</td><td rowspan="3">80</td><td rowspan="3">75</td><td rowspan="3">60</td><td rowspan="3">50</td><td rowspan="3">45</td><td rowspan="3">40</td><td rowspan="3">1.5</td><td>3</td><td>1</td><td rowspan="3">9.68</td><td rowspan="3">12</td></tr>
<tr><td>2</td><td rowspan="2">0.66</td></tr>
<tr><td>1</td></tr>
<tr><td rowspan="2">KT</td><td rowspan="2">50</td><td colspan="4" rowspan="2">86</td><td colspan="4" rowspan="2">82</td><td rowspan="2">3</td><td rowspan="2">1</td><td>1</td><td rowspan="2">9.4</td><td rowspan="2">12.4</td></tr>
<tr><td>0.66</td></tr>
<tr><td>GGHP</td><td>30</td><td colspan="4">75</td><td colspan="4">54</td><td>3</td><td>1</td><td>1</td><td colspan="2">3.7</td></tr>
</table>

4. SRF CALCULATION

The latest support chart of Q-system (NGI 2015) was used for the calculation of SRF and according to it, the bolt length depends upon the tunnel span and Excavation Support Ratio (ESR) value which defines the safety requirement and to some degree on the quality of the rock mass. Different countries have different standards for safety so, the selection of ESR should be done according to the safety of working crew in that country (Palmstrom 2006). According to Q-support chart, the relation of the bolt length with equivalent diameter shows that ESR=1 can be used for both road and hydropower tunnels that's why the same ESR value is used for all the 4 projects in this study. The rock quality can be determined from rock bolt spacing and thickness of shotcrete either sprayed or not. As sprayed concrete was used in all projects, spacing of the bolts was related to the rock quality (Q) first followed by thickness of shotcrete.

Rock quality value obtained from different support categories and the supported span of the tunnel was defined Q or Q_c. Two different values of SRF (SRF_Q or SRF_{QC}) were calculated by taking the rock quality value either Q or Q_c by rearranging Eq. (1) and (2) to Eq. (3) and (4) respectively.

$$SRF_Q = \left(\frac{RQD}{J_n}\right).\left(\frac{J_r}{J_a}\right).\left(\frac{J_w}{Q}\right) \tag{3}$$

$$SRF_{QC} = \left(\frac{RQD}{J_n}\right).\left(\frac{J_r}{J_a}\right).\left(\frac{J_w}{Q_c}\right).\left(\frac{\sigma_c}{100}\right) \tag{4}$$

Out of 542 sections, only 44 section of the tunnels have σ_c equal to 100MPa, and the remaining is less than 100MPa. So, the SRF_{Qc} is less than SRF_Q depend upon the intact rock strength (σ_c).

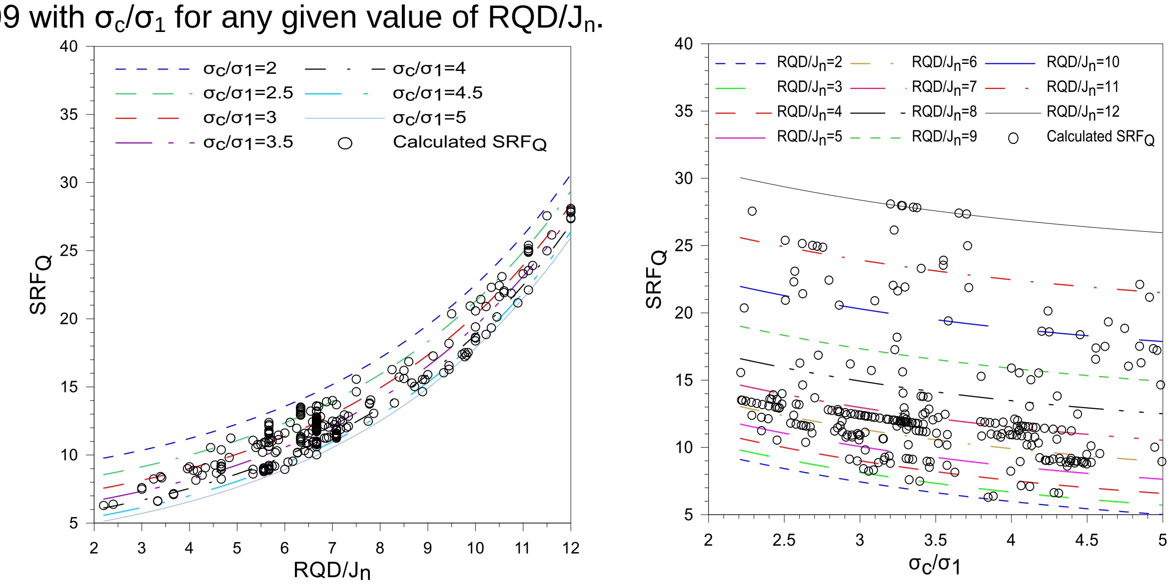
The empirical data obtained from Eq. (3) were plotted against RQD/J_n for different values of σ_c/σ_1 ($2<\sigma_c/\sigma_1<5$). Similarly, the same data obtained from Eq. (3) were plotted against σ_c/σ_1 for different values of RQD/J_n ($2.2\leq RQD/J_n\leq 12$). A new empirical equation based on the data was obtained as followed for calculation of SRF_Q value for jointed rock which matched the data in Fig. 2.

$$SRF_Q = 2.054\exp\left(0.205\frac{RQD}{J_n}\right) + 14.865\exp\left(-0.41\frac{\sigma_c}{\sigma_1}\right) \tag{5}$$

Taking the resultant value of the rock quality obtained from the Q-system support chart as Q_c, SRF_{QC} value obtained from Eq. (4) can be obtained from Eq. (5) for any value of intact rock strength by using the normalized factor.

$$SRF_{QC} = \left(SRF_Q\right).\left(\frac{\sigma_c}{100}\right) \tag{6}$$

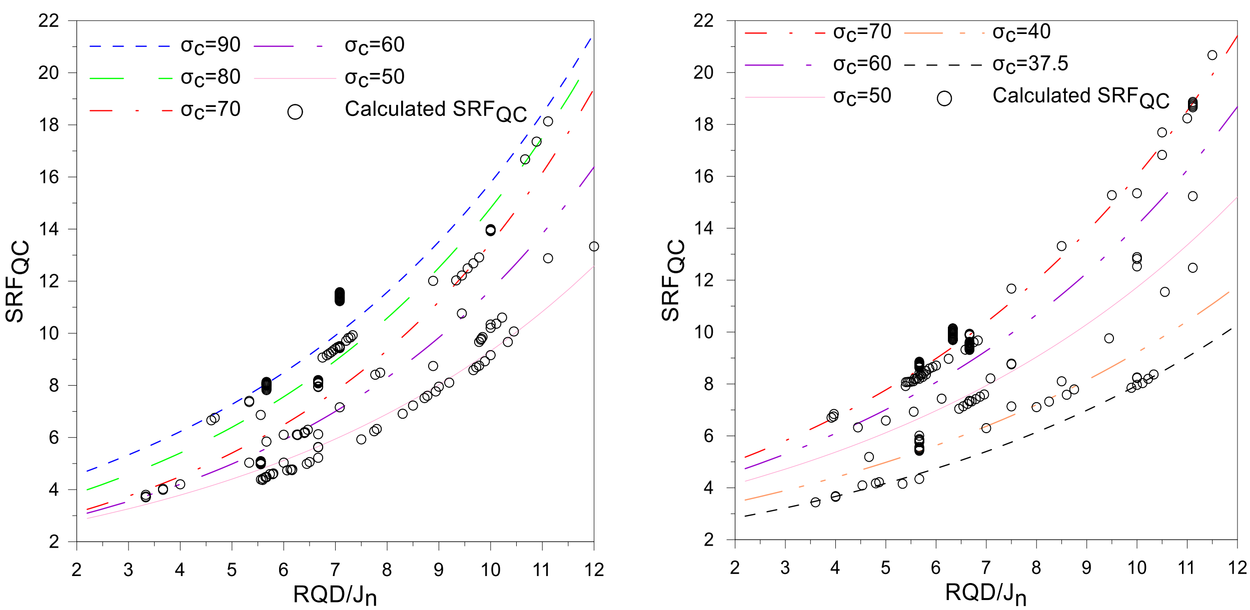
Fig. 2(a) shows SRF_Q increases significantly with RQD/J_n for its higher value. A maximum variation in RQD/J_n below 7 result a change of 5.4 in SRF_Q but for the same change in RQD/J_n above 7, SRF_Q changes by an amount of approximately 15.05 which is about 2.75 time. A total variation of SRF_Q is 20.82 can be observed with RQD/J_n for a given value of σ_c/σ_1. With increase in RDQ/J_n, the difference of SRF_Q for different values σ_c/σ_1 is comparatively low for a given value of RQD/J_n. A similar trend can be seen in the Fig. 2(b) but in a reverse order. For the same value of RQD/J_n, decreasing rate of SRF_Q with σ_c/σ_1 is high at its low value as compared to high value of σ_c/σ_1. Below 3.5 value of σ_c/σ_1, for a maximum variation in σ_c/σ_1 for a given value of RQD/J_n, change in SRF_Q value is 2.49. Variation in σ_c/σ_1 by an equal amount in its upper limit (above 3.5) can change SRF_Q by 1.35 approximately which is about 1.84 times low. Maximum changes in SRF_Q is 4.09 with σ_c/σ_1 for any given value of RQD/J_n.



a) SRF_Q Vs RQD/J_n for different σ_c/σ_1 b) SRF_Q Vs σ_c/σ_1 for different RQD/J_n

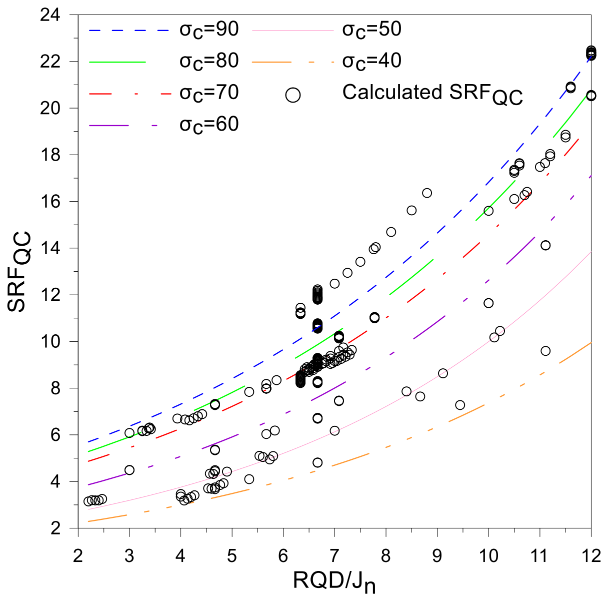
Fig. 2 Relation of SRF_Q with RQD/J_n and σ_c/σ_1

SRF_{QC} calculated from Eq. (4) or (6) were plotted against RQD/J_n for different values of σ_c and ranges of σ_c/σ_1 as shown in Fig. 3. SRF_{QC} value and changes of SRF_{QC} with RQD/J_n depend upon σ_c. For same range of σ_c/σ_1 ratio, high value of σ_1 requires for the rock with high σ_c. The high level of stress means more support is required and hence high value of SRF_{QC}. The maximum variation of SRF_{QC} with RQD/J_n is 16.81 for σ_c equal to 90 with σ_c/σ_1 range from 4-5 (Fig. 3(a)) and the minimum variation is 7.38 for σ_c equal to 37.5 (Fig. 3(c)). As during the design stage of the project, limited information is available and hence by knowing the σ_c which is the very basic parameter to be determined and the location of the tunnel, Fig. 3 can be used for the quick estimation of SRF_{QC} for different value of RQD/J_n. During the construction stage of the project, when maximum information is in hand, Eq. (5) and (6) can be used for the calculation of SRF_{QC}.



a) σ_c/σ_1=4-5

c) σ_c/σ_1=2-3



b) σ_c/σ_1=3-4

Fig. 3 Intact rock strength (σ_c) effect on SRF_{QC} for different ranges of σ_c/σ_1

5. COMPARISON OF SRF

Based on SRF relation to stress-strength ratio, 20 was the extreme value of SRF for competent rock having rock stress problem (Barton 1974). Eq. (7) was used for SRF calculation in the rock stress problem for south African mining field (Kirsten 1988).

$$SRF = 0.244K^{0.346}.\left(\frac{H}{\sigma_c}\right)^{1.322} + 0.176\left(\frac{\sigma_c}{H}\right)^{1.413} \tag{7}$$

As the unit weight of the rock for maximum sections in this study is about 0.027MN/m^3, Eq. (7) can be rewritten as Eq. (8).

$$SRF = 28.29\left(\frac{\sigma_1}{\sigma_3}\right)^{0.346}.\left(\frac{\sigma_c}{\sigma_1}\right)^{-1.322} + 0.00107\left(\frac{\sigma_c}{\sigma_1}\right)^{1.413} \tag{8}$$

A similar equation like Eq. (8) was proposed for Australian mining (Eq. (9)), where the high stresses are either due to mining depth or advancing mining front, acting on jointed rock (Peck 2000).

$$SRF = 31\left(\frac{\sigma_1}{\sigma_3}\right)^{0.3} . \left(\frac{\sigma_c}{\sigma_1}\right)^{-1.2} \tag{9}$$

The exponential derived Eq. (9) was agreed by Barton for SRF determination in jointed rock under stresses (Barton 2007).

For different values of k ($\frac{\sigma_h}{\sigma_v} = 0.3, 0.5, 0.75, 1$) , SRF was determined by using Eq. (8) and (9) and was plotted against σ_c/σ_1 as shown in Fig. 4. SRF calculated from Eq. (3) and (4) is very scattered and hence Eq. (7) and (9) cannot be used for SRF calculation in tunneling. Eq. (7) and (9) are focusing on the ratio of σ_c/σ_1 but not considering how much the rock is jointed.

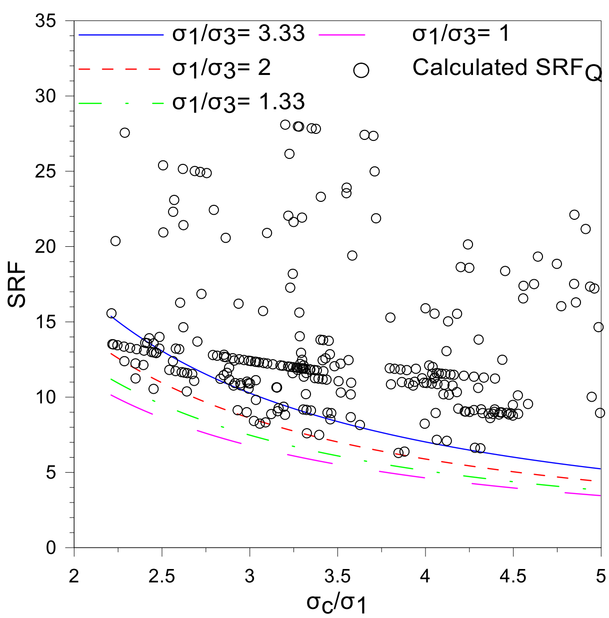
(Kumar 2004) developed Eq. (10) for SRF calculation from the Nathpa Jharki project data in India.

$$SRF = 5.84\left(\frac{\sigma_c}{H}\right)^{0.001} . \left(\frac{J_r^3}{J_n}\right) + 2.58 \tag{10}$$

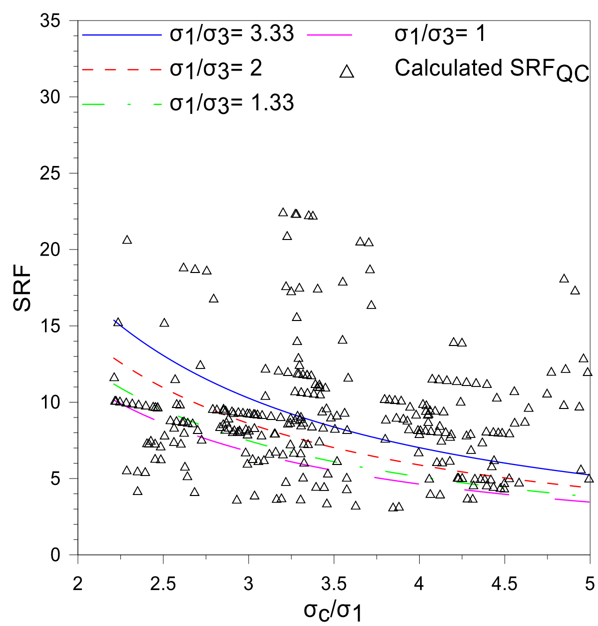
Putting unit weight of the rock equal to 0.027MN/m^3, Eq. (10) can be rewritten as Eq. (11).

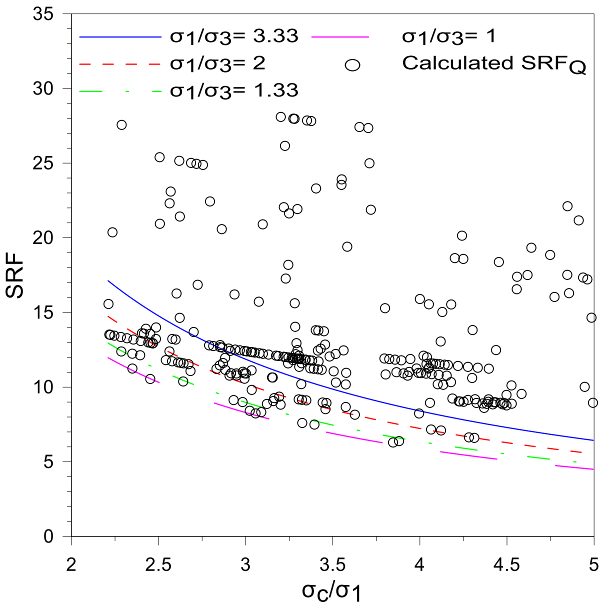
$$SRF = 5.82\left(\frac{\sigma_c}{\sigma_1}\right)^{0.001} . \left(\frac{J_r^3}{J_n}\right) + 2.58 \tag{11}$$

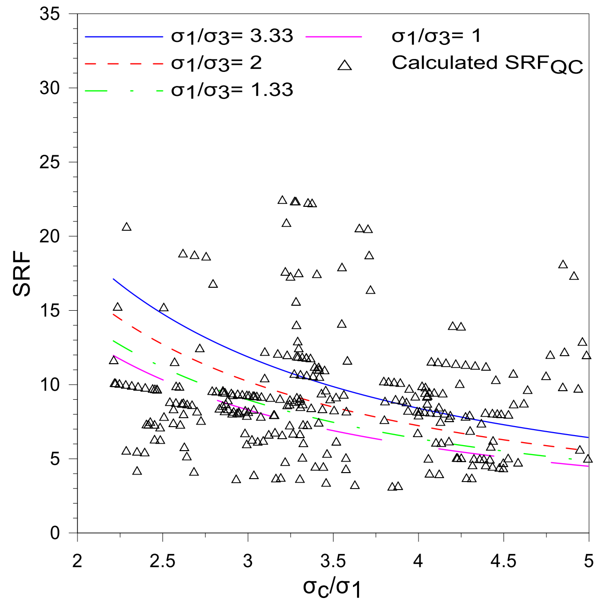
As can be seen from Eq. (11) that SRF has no significant variation with $\frac{\sigma_c}{\sigma_1}$ and the results from Eq. (3) and (4) are too different from Eq. (11).



a) Comparison with Eq.7







b) Comparison with Eq. 9

Fig. 4 Comparison of calculated SRF (SRF_Q & SRF_{QC}) with already available equations

6. NUMERICAL MODELLING

Four hydropower projects named Diamer Basha Dam (DBD), Kohala Hydropower (KHP), Bunji Hydropower (BHP) and Dasu Hydropower (DHP) are in the design stage under Water and Power Development Authority (WAPDA) Pakistan. Results from the 7 boreholes (BDR-08, 10, 21, 22, 24, 25 and 26) along the 15.4 m wide diversion tunnels of DBD project shows that two major rocks are available named Gabbronorite (GN) and Ultramafic Association (UMA) and 67 % of the RQD/J_n is below 12 and only 7.6 % is above 25. The frequency (%) of RQD and joint set are shown the Fig. 5.

RQD from 3 boreholes changes from 25 to 75 % and J_n rating value is from 4 to 12 for the 17mx20m proposed diversion tunnel of DHP project which will pass through amphibolite rock.

The exploration report along the 10 m span head race tunnels of BHP project shows that the cumulative frequency (%) is 90 for RQD is less than 80 and 80 for RQD is less than 60. According to exploration report, the borehole logs indicate that there are typical three discontinuity sets. Major part of the 8 km long tunnels will pass through Iskere Gneiss (IG) and Amphibolite.

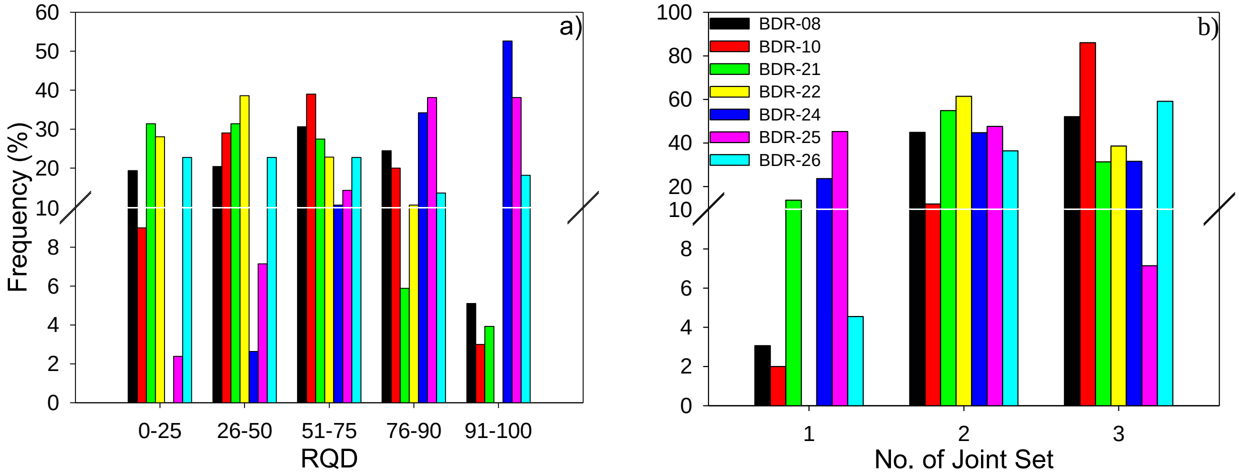


Fig. 5 Percentage frequency of a) RQD and b) number of joint sets for 7 boreholes along the diversion tunnel of DBD project

The 17.5km long and 8.5m span head race tunnel is a major component of KHP project which will be excavated mostly in SS-1 and SS-2 and assessment of 13 bore holes (BH-01, 02, 03, 04, 05, 08, 09, 10, 11, 12, 13, 15, 26) indicate that in most of the boreholes, degree of jointing is high to very high resulting in RQD < 50%. Only BH-09 and BH-26 have predominantly a moderate degree of jointing which correspond to a range of RQD from 50% to 75% (Munir 2013). For simplicity and reducing the number of cases to be numerically evaluated, J_r=3, J_a=1 and J_w=1 were considered for empirical support design.

The average properties of each intact rock available for different projects and GSI are given in the Table-3.

Table-3 Summary of intact rock properties and GSI for different rock types of the four projects

Project	Rock Type	Unit weight (KN/m^3)	UCS (MPa)	E (GPa)	Poisson Ratio	mi	GSI
DBD	GN	28.6	100	42	0.2	23	71
	UMA	31.7	80	24	0.152	25	58
KHP	SS-1	27	80	40	0.2	17	60
	SS-2	27	50	30	0.15	17	50
BHP	IG	26.86	50.7	20.81	0.173	28	61
	Amphibolite	27.67	61	27.4	0.179	26	67
DHP	Amphibolite	29.7	111	37.8	0.11	26	70

As the main aim of the current study is SRF determination in rock stress problem for jointed rock, so different overburdens were used for creating different cases of σ_c/σ_1 ratio. For different values of RQD/J_n and σ_c/σ_1, SRF (SRF_Q and SRF_{QC}) were calculated from Eq. (5) and (6) and corresponding Q or Qc value were found using Eq. (1) and (2). Based on Q or Q_c value and tunnel size, the support system was determined empirically from Q-system chart as shown in Table-4.

Table-4 Empirical Support recommendation for different tunnels by Q-system

Project	Rock Type	Tunnel size	RQD/J_n	σ_c/σ_1	Q or Q_c	Rock Bolt Length (m)	Rock Bolt Spacing (m)	Shotcrete thickness (cm)
DBD	GN	15.4x15.4	10	3.5	1.54	4	1.7-1.8	9-12
	UMA	15.4x15.4	9	3	1.56	4	1.7-1.8	9-12
KHP	SS-1	8.5x8.5	8	2.5	1.51	2.9	1.7-1.8	6-9
	SS-2	8.5x8.5	7	4	1.83	2.9	1.7-1.8	6-9
BHP	IG	10x10	6	4.5	1.92	3	1.7-1.8	6-9
	Amphibolite	10x10	3.5	2	0.98	3	1.5-1.7	9-12
DHP	Amphibolite	17x20	5	5	1.97	5	1.7-1.8	9-12

The intact rock properties were extrapolated to the rock mass as shown in Table-5 with the help of RocLab software which is based on the generalized Hoek-Brown criteria (Hoek 2002).

The computer software FLAC (Fast Lagrangian Analysis of Continua), an explicit 2D finite difference program that is suited for sequential excavation modelling was used in the analysis. Horse shoe shape tunnel was selected as it is the better shape for drill and blast method during such circumstances and is the commonly used tunnel shape in Pakistan.

Table-5 Summary of the rock mass properties for different rock types of the four projects

Project	Rock Type	m_b	S	a	c (MPa)	ϕ^o	E (GPa)	σ_t
DBD	GN	8.16	0.04	0.501	5.939	48.78	31.544	0.488
	UMA	5.58	0.009	0.503	4.256	44.79	11.392	0.135
KHP	SS-1	4.074	0.0117	0.503	4.43	40.67	20.8	0.231
	SS-2	2.85	0.004	0.506	1.762	41.49	9.215	0.068
BHP	IG	6.95	0.013	0.503	2.264	49.6	11.294	0.096
	Amphibolite	8	0.026	0.502	4.848	44.48	18.47	0.195
DHP	Amphibolite	8.91	0.036	0.501	5.4	52.09	27.7	0.445

The support system that was empirically determined was used in numerical analysis and the results before and after support were analyzed in term of plastic zone in the vicinity of tunnel and displacement in rock mass.

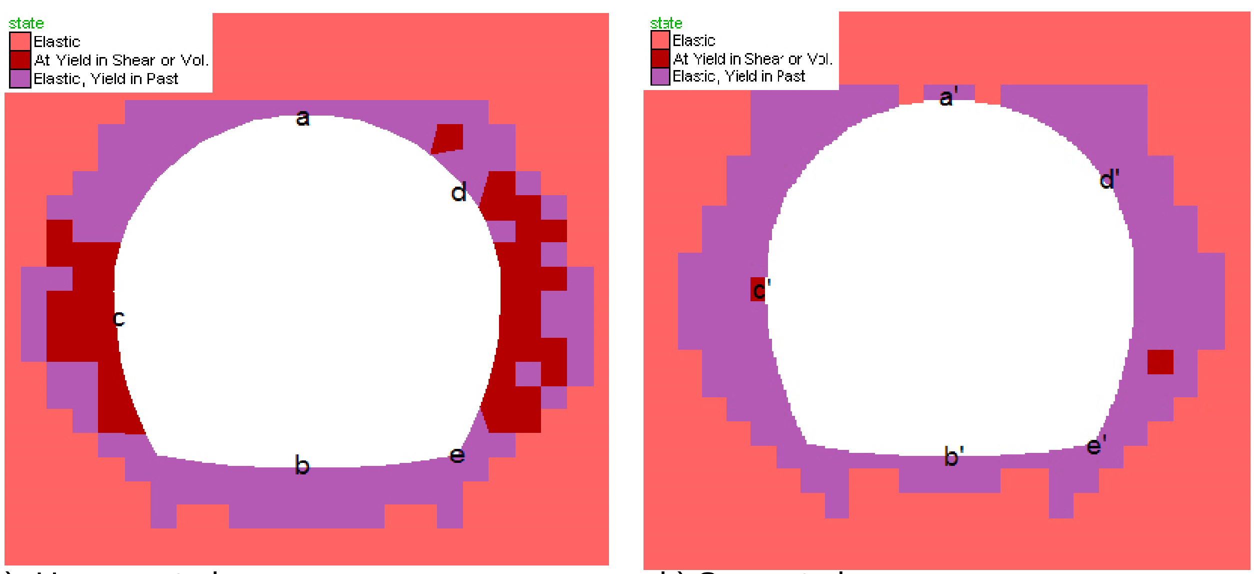
As can be seen from Table-6 and Fig. 6, after the installation of recommended support shown in Table-4, total displacement decrease. The maximum decrease in total displacement with the application of empirical support as per Q-system chart at crown (point a) is for SS-2 rock of KHP project. For point b (invert) & c (spring line), the leading case in term of total displacement drop is SS-1 of KHP project. For similar analysis, cases which show maximum reduction in total displacement at point d (shoulder) & e (intersection point of tunnel wall and invert) is the UMA rock of DBD project. As it is a plain strain condition, displacement at same point but in opposite direction of the vertical centerline is same.

Table-6 Total displacement (cm) at different points along the tunnel perimeter with and without support

Project	Rock Type	a	a′	b	b′	c	c′	d	d′	e	e′
DBD	GN	1.58	1.53	1.09	1.03	.44	0.42	1.22	1.13	.50	0.48
	UMA	4.22	4.18	2.91	2.83	1.17	1.11	3.5	2.8	1.49	1.42
KHP	SS-1	1.49	1.45	1.33	1.15	0.59	0.44	1.23	1.18	0.73	0.71
	SS-2	1.32	1.20	1.09	1.04	.38	0.32	1.01	0.95	0.66	0.63
BHP	IG	.97	.94	.83	.81	0.25	0.24	0.69	0.68	0.425	0.40
	Amphibolite	1.75	1.68	1.40	1.39	.53	0.48	1.36	1.31	0.78	0.78
DHP	Amphibolite	1.51	1.49	1.06	.97	0.52	0.49	1.18	1.17	0.36	0.35

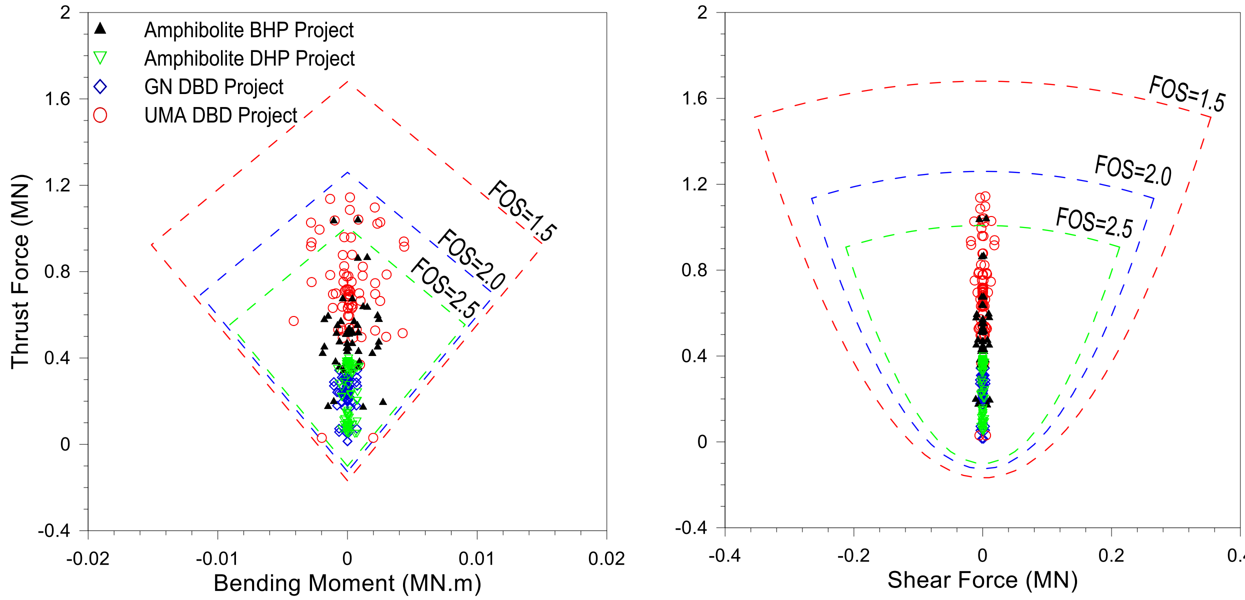
Plastic zone before and after support was analyzed for all seven cases of four projects. According to plasticity theory, a plastic zone occurs around a tunnel after excavation when induced stresses exceeds rock mass strength. The application of empirically recommended support not only reduce the number of yield elements but also decreases the extent of plastic zone substantially for all the cases. With support, yield in tension was not observed while yield in shear is either zero or substantially reduced. For the case of KHP project SS-1 rock, where the yield in shear is still the maximum comparatively for the other cases is shown in the Fig. 6 but with application of support this has been decrease enormously.

Capacity diagrams which represent graphically the relationships between thrust-bending moment and thrust-shear force are used to evaluate the stability of liner (Carranza-Torres 2009). Based on the proposed thickness of shotcrete from Table-4, capacity diagrams were plotted for shotcrete thickness of 12 cm (four cases) and 09 cm (three cases) for factor of safety (FOS) equal to 1.5, 2 and 2.5. As can be seen from the Fig. 7, that none of the case crossing the limit of FOS equal to 2.

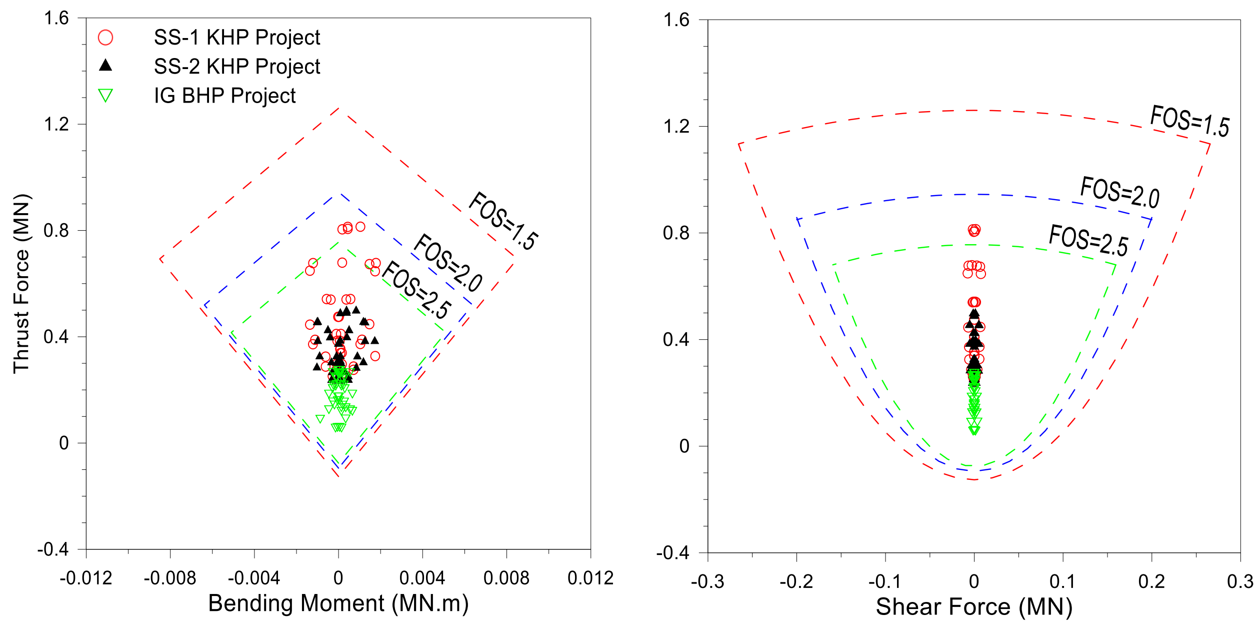


a) Unsupported b) Supported

Fig. 6 Extent and number of plastic zone around the tunnel before and after support



a) 12-Centimeter(cm)-thick shotcrete



09-Centimeter(cm)-thick shotcrete

Fig. 7 Shotcrete stability analysis through capacity diagrams

7. CONCLUSION

1. In high stress environment, using Q-system as a tunnel designing tool has preference over RMR system due to SRF for competent rock having rock stress problem. Rock slabbing and rock bursting are the usual phenomena related to massive rock in high stress environment and Q-system have a range of SRF values based on σ_c/σ_1. These massive rock failure phenomena are not as much high in jointed rock, proved the positive impact of jointing which sometime artificially produces by destress blasting and due to the said reason, SRF value of massive rock cannot be used for jointed rock under the same condition. Q-system is not providing SRF values for highly stressed jointed rock.
2. Data from eight tunneling projects of Pakistan shows that the rock is usually jointed with relatively low RQD/J_n values. SRF characterization for highly stressed jointed rock environment extent the application of Q-system for this situation.
3. Determining the rock quality value from installed support and tunnel dimension through back calculation, the original and modified Q-system (Q and Q_c) equation were used for the SRF calculation. Empirical equations are suggested for SRF characterization in competent rock having rock stress problem in case of jointed rock from 542 tunnel section of four tunneling projects data which are already supported. In proposed equations, SRF depend upon the intact rock strength, ratio of intact rock strength to major principal stress and relative block size. The proposed empirical equations are based on the data for $RQD/J_n \leq 12$, $100 \geq \sigma_c \geq 37.5$ and $2 < \sigma_c/\sigma_1 < 5$.
4. The already available equations are used for SRF calculations in jointed rock. The results show that the back calculated SRF are not matching with the measured values from already available equations. Eq. (7) and (9) are from mining experience of South Africa and Australia respectively and not considering how much the rock is fractured. Eq. (10) is based on tunnel project data and the variation of σ_c/σ_1 has a negligible effect.
5. The proposed equations are used for SRF characterization of four tunnels which are in the design stage and empirical support suggested by Q-system support chart are numerically evaluated. With application of support, total displacement and extent of plastic zone decreases. Evaluation of shotcrete through capacity diagram also confirms its stability.

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