Qualitative stability assessment of cut slopes along the National Highway-05 around Jhakri area, Himachal Pradesh, India

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Several deformation phases in tectonically active Himalayas have rendered the rock masses very complex in terms of structure, lithology and degree of metamorphism. Again, anthropogenic activities such as roads, tunnels and other civil engineering constructions have led to a state of disequilibrium which in many cases, results in failure of rock masses. National Highway-05 around Jhakri area in India is a major connecting route to the China border in the hilly terrains of the state Himachal Pradesh. It cuts through the Himalayan rocks and has a hazardous history of landslides destroying human lives and interrupting communication very frequently. As a contribution towards the mitigation process, a study has been carried out along the highway to analyse kinematic stability and qualitative estimation of rock mass condition through rock mass classification systems. The kinematic analysis shows that the rock slopes are prone to planar and wedge failure. Rock mass rating for most of the locations lies between 7 and 34, representing a poor rock mass quality (Class IV), whereas slope mass rating is more disperse and ranges from 11 to 52 for most of the slopes (Class III, IV and V).

Keywords. Cut slope; kinematic analysis; continuous slope mass rating; geological strength index.

1. Introduction

Himachal Pradesh, primarily is a mountainous area with variable climate and uneven rugged topography. It covers parts of Shiwaliks, lesser Himalayas and higher Himalayas with an altitude ranging from 350 to 6975 m above mean sea level (Srikantia and Bhargava 1998). The mountains in the state are dissected by deep gorges and valleys. Tectonically and morphologically active young and dynamic Himalayas in this region are intersected by many dormant and active thrusts namely Himalayan Frontal Thrust (HFT), the

Main Boundary Thrust (MBT), Main Central Thrust (MCT), the Krol, the Giri, Jutogh and Nahan thrusts. These longitudinal tectonic fractures trend parallel to the Himalayas. Dislocations along these thrusts give rise to earthquake epicentres in the state. The state is assigned under zone IV and V of earthquake hazard zonation described in India hazard zonation map (Narula *et al.* 2000). According to National Institute of Disaster Management (NIDM), Govt. of India report, about 250 earthquakes with magnitude 4 and 62 earthquakes having magnitude of 5 and above have impacted the state so far. In addition to earthquakes,

monsoonal rain causes flood each year in the state which makes the region very prone to landslides. Again, frequent flash floods, cloud bursts and glacial retreat due to global warming weaken the rock mass, making it more vulnerable to failure. Landslide is the most common hazard in the state. Every year one or more major landslides affect the state causing loss of life, property and severe interruption in communication. In addition to rock mass strength and unfavourable orientation of joints and fractures, the intensity and effect of landslide in the area varies with various geophysical factors such as steepness and height of slopes, water ingress in rock joints and fractures, saturation of rock material due to heavy rains, melting snow and ice, seismic vibrations, frost action, change in vegetable cover and toe cutting by rivers and streams. Again, anthropogenic factors as a result of developmental activities such as poorly designed roads, deforestation, unplanned agricultural practice, and encroachment on steep slopes, further aggravate the process. The landslide vulnerability map of Himachal Pradesh is given in figure 1. Many hazardous landslides that have occurred in the past, along the National Highway (NH)-05 in the state have been recorded and the details are shown in table 1.

Apart from above history, another major landslide had occurred in 1993 along NH-05 due to flash flood which damaged 1 km of road in Jhakri region. The failed mass of the road cut had blocked the Sutlej River. Communication in the highway could be restored only after two months of the event (NIDM, Himachal Pradesh; <http://nidm.gov.in/pdf/dp/himachal.pdf>). Also, the road experiences major/minor landslides every year mostly after monsoonal rainfall. District disaster management authority of Shimla has reported the Rampur–Jeori sector, a stretch of 25 km in NH-05 as landslide vulnerable area.

Keeping an eye on the problems mentioned above, a study was carried out along NH-05 covering a stretch of 12 km from Jhakri to Jeori in Shimla district, Himachal Pradesh, India. Studied locations are illustrated in Google Earth image (figure 2). The highway is located on the left bank of river Sutlej which makes the situation worse and threatens the stability of the road. Kinematic analysis and rock mass characterisation have been performed to evaluate the rock mass condition. Similar works have been performed recently by Umrao *et al.* (2015) and Singh *et al.* (2015). Sarkar *et al.* (2016) have also assessed the quality of some cut slopes along the same highway.

2. Geology and geotechnical details

The Himalayas being a classic example of continent–continent collision, comprises of two distinct tectogens separated by the Main Central Thrust (MCT). These are the lesser Himalayan tectogen and Tethys Himalaya tectogen who bear their own distinct geological history. The studied area from Jhakri to Jeori, a 15 km stretch in NH-05 is present in lesser Himalayan tectogen. The area lies in the Larji–Rampur–Wangtu window zone of Larji tectono-stratigraphic domain (Srikanthia and Bhargava 1998). Rocks in this area date back to Paleoproterozoic and Mesoproterozoic and indicate medium to high grade inverted metamorphism (Singh 1979). The rock types in the area are augen migmatite, biotite gneiss, quartz mica schist, garnet bearing quartz mica schist, muscovite biotite schist, and amphibolite, which are truncated by faults and other discontinuities (Singh 1979; Kundu *et al.* 2016a, b). Regional strike of lithological units is ENE–WSW which shows local variations in the trend (Singh 1979). The geological map of Sarahan Bushair (the investigated area) is given in figure 3.

A total of 12 locations were selected for the proposed study which covers four major rock types namely, foliated mica schist, strongly foliated mica schist, biotite gneiss, and crenulated mica schist. Rock masses in the area are foliated and blocky in nature, intruded by small quartz veins and commonly intersected by three sets of joints. Joint planes are filled by soft infillings in most rock masses and also remains unfilled in some places. Physical and geomechanical properties in both dry and saturated conditions for the rock materials were determined according to ISRM (1981) specifications. Details of these properties are given in table 2.

3. Stability evaluation (qualitative)

3.1 Kinematic analysis

It is an easy practice to use stereographic analysis for the determination of potential failure types and direction in jointed rock mass (Goodman 1976; Hoek and Bray 1981; Matherson 1988). The analysis always indicates geometrically admissible failures which takes slope and joint orientations into consideration. However, friction cone (joint friction angle) has been included in the analysis to

Figure 1. Landslide hazard zonation map of Himachal Pradesh, India (modified from landslide hazard map of SEEDS’ 2009 Himachal Pradesh vulnerability atlas).

Table 1. Major landslide history along NH-05 (source: NIDM, Himachal Pradesh; <http://nidm.gov.in/pdf/dp/himachal.pdf>).

Location/highway	Years	Damage
NH-05	1988–1995	During the flood of 1988, 1993 and 1995, 250, 350 and 475 m of road was washed away
NH-05, 292–293 km	1988–1995	During the flood of 1988, 1993 and 1995, 200, 500 and 300 m of road was washed away
NH-05, 307 km	1988–1995	During the flood of 1988, 1993 and 1995, 100, 150 and 600 m of road was washed away

satisfy both kinematic and frictional conditions for the failure. Kundu *et al.* (2016a, b) have modified the technique of kinematic analysis of wedge in a curved slope and analysed a curved slope, where they devised a way to represent varying degree of vulnerability of wedges along a curved slope. A stereonet analysis program DIPS 6.0 (2015) of Rocscience has been used for the analysis in order to find out modes of failure. Eleven slopes have been analysed for which the input parameters are presented in table 3. It has to be noted that the term ‘bedding plane’ has been used in this paper under the assumption that joint set having major

continuity would represent the same and should not be confused with sedimentary structure. As there are no systematic joints other than some irregular fractures in slope no. 10, it has not been considered for the analysis. Kinematic analysis for the locations shows that the slopes are highly vulnerable to wedge and planar failure except for slope no. 5 which is kinematically stable. Slope 1 shows potential for wedge failure towards 270° but with small block sizes (figure 4). Slope 2 (figure 5) consists of similar joints as of slope 1 and hence represents wedge towards

Figure 2. Studied locations plotted in Google Earth image. The yellow line denotes NH-05 and the light blue line represents Sutlej River.

271° but with less vulnerability, as the intersection line is not inside the friction cone. Additionally, the slope face orientation, unlike slope 1 facilitates planar failure.

Slope 3 is exposed to wedge failure and trend of the intersection line is towards 301°. As its direction is adjacent to dip direction of the slope, the wedges are more open to failure. Furthermore, the block sizes in this zone measure up to 1.8 m³, making it more hazardous in terms of rock fall (figure 6).

Kinematic analysis has been performed for slope 4, which represents wedge towards 301° but failure is not probable due to low plunge angle of the intersection line (intersection point falls outside of failure zones). However, irregular fracture dominates the zone causing failure of rock mass (figure 7).

Slope 5, being more stable in the region, contains only one tight joint set and no possible failure is expected (figure 8). Besides, the location contains very less and insignificant irregular fractures which makes a fair rock mass.

Slope 6A (figure 9) and 6B (figure 10) are weak rock masses intersected by closely spaced joints. Kinematically, the rock masses are exceedingly prone to planar failure. Though analysis for Slope 6A shows a wedge failure with intersection line trend 343°, the wedges slip along one plane making single plane wedge failure. Figure 11 represents both planar and wedge failure for slope 7. Three joint sets form wedges with two failure directions along 293° and 357°. The wedge with intersection line 357°, fails along the joint plane dipping towards 312°.

Slope 8A comprises rock masses with kinematically unsafe wedges and planes. Intersection lines of probable wedge failures are towards 288° and 327°. The wedge with intersection line towards 327° slips along the joint plane dipping towards 216° (figure 12).

Slope 8B consists of rock mass cliffs with three joint sets, out of which one joint set is responsible for failure. Kinematic analysis shows planar failure towards 270° (figure 13).

Figure 3. Geological map of Sarahan Bushair area, district Shimla, Himachal Pradesh, India (after Singh 1979).

Table 2. Physical and geotechnical properties of rock materials.

Slope no.	Physical properties				Geomechanical properties			
	Dry unit weight (kN/m ³)	Saturated unit weight (kN/m ³)	Porosity (%)	Water absorption (%)	Uniaxial compressive strength (MPa)		Tensile strength (MPa)	
					Dry	Saturated	Dry	Saturated
1	27.13	27.38	2.52	0.91	48.2	34.3	5.1	3.6
2	27.09	27.31	2.47	0.91	38.3	29.7	4.6	3.4
3	27.04	27.26	2.44	0.9	36.5	27.7	4.2	3.1
4	27.01	27.23	2.42	0.89	36.3	24.3	4.0	2.6
5	26.96	27.19	2.39	0.87	35.6	21.0	4.15	2.3
6A	26.77	26.94	1.86	0.69	42.7	26.8	4.7	2.6
6B	26.77	26.93	1.83	0.68	43.4	25.3	5.2	2.7
7	26.52	26.71	1.59	0.61	45.9	32.5	4.9	2.6
8A	26.36	26.52	1.46	0.59	54.2	41.8	6.2	3.7
8B	26.36	26.5	1.43	0.56	57.1	43.0	6.3	3.9
9	26.32	26.46	1.39	0.52	71.5	54.5	8	5.2
10	26.61	26.79	1.86	0.68	54.6	35.9	6.2	3.6

Table 3. Joint and slope orientations for different cut slopes.

Slope no.	Bedding plane (S_0)		Joint plane (J_1)		Joint plane (J_2)		Joint plane (J_3)		Slope	
	Dip	Dip direction	Dip	Dip direction	Dip	Dip direction	Dip	Dip direction	Slope angle	Slope direction
1	49	326	63	200	—	—	—	—	85	290
2	45	345	54	194	—	—	—	—	88	330
3	51	357	78	220	—	—	—	—	80	280
4	38	4	71	219	—	—	—	—	90	330
5	43	5	—	—	—	—	—	—	90	300
6A	47	360	46	337	—	—	—	—	84	347
6B	43	2	—	—	—	—	—	—	90	15
7	33	356	82	17	43	312	—	—	85	295
8A	50	316	84	12	64	23	—	—	86	300
8B	17	302	64	274	44	85	—	—	82	270
9	—	—	62	293	61	76	70	21	75	60

J_1 = joint no. 1, J_2 = joint no. 2, J_3 = joint no. 3.

Slope 9 consists of rock mass intersected by three joint sets. The rock blocks are frequently falling due to unstable wedges. The rock mass forms wedges with two potential failure directions towards 4° and 70° . Wedge with intersection line trend 70° undergoes single plane wedge failure along the joint plane dipping towards 75° (figure 14).

3.2 Rock mass characterisation

For better communication in rock engineering community, many classification systems have been developed to quantify overall quality of rock mass for different purposes such as tunnelling, mining, foundation and rock slope. Some of the universally accepted and well-established classification systems are Slope Mass Rating (SMR) (Romana 1985; Tomas *et al.* 2007) for slope mass characterization, Rock Mass Rating (RMR) (Bieniawski 1973, 1976, 1979, 1989) for tunnelling, mining, foundation and slope applications, Rock Mass Quality (Q) (Barton *et al.* 1974) for tunnelling, Mining Rock Mass Rating (MRMR) (Laubscher 1990), Rock Mass Strength (RMS) (Selby 1980) and Geological Strength Index (GSI) (Hoek *et al.* 1998; Marinos and Hoek 2000) which is based on visual examination including weak and sheared rock masses. Though RMR was originally developed for tunnelling in moderate to hard rock mass, it was latter modified and adjusted over the years (Weaver 1975; Laubscher 1977, 1984; Romana 1985; Newman and Bieniawski 1986; Venkateswarlu 1986) for the application in mines, foundations, and slopes. Slopes in jointed rock masses are mainly controlled by joint orientations but there is no explicit consideration of the factor in RMR which render the rating system less applicable for slopes (Mahanta *et al.* 2016). Romana (1985) proposed quantitative definitions for correction factors based on joint orientations and their relationships which lead to SMR system.

This paper covers classification of rock masses along the road cut slopes using conventional Rock Mass Rating (RMR) system proposed by Bieniawski (1979, 1989) and modified slope mass rating using continuous functions for correction factors (Tomas *et al.* 2007) which was originally developed by Romana (1985). Kinematic analysis has been carried out prior to RMR and SMR for the investigated slopes to anticipate potential modes of failure. Then SMR correction factors have been calculated for the identified failure type. Geological strength index also has been calculated for

Figure 4. Kinematic analysis for slope no. 1.

Figure 5. Kinematic analysis for slope no. 2.

Figure 6. Kinematic analysis for slope no. 3.

Figure 7. Kinematic analysis for slope no. 4.

Figure 8. Kinematic analysis for slope no. 5.

Figure 9. Kinematic analysis for slope no. 6A.

Figure 10. Kinematic analysis for slope no. 6B.

Figure 11. Kinematic analysis for slope no. 7.

Figure 12. Kinematic analysis for slope no. 8A.

the slopes using quantified method (Sonmez and Ulusay 1999, 2002). Finally, a correlation between basic RMR and GSI is given for the studied area. Twelve slopes were studied out of which RMR, SMR and GSI have been determined directly for 11

slopes. For the slope no. 10, as it does not contain joints except some irregularly oriented fractures, only GSI could be determined from visual observation. Basic RMR has been estimated from the stated correlation (equation 9).

Figure 13. Kinematic analysis for slope no. 8B.

Figure 14. Kinematic analysis for slope no. 9.

3.2.1 Basic RMR

The RMR system, which is otherwise known as geomechanics classification, uses six parameters to classify a rock mass which are: (a) uniaxial compressive strength (UCS) of rock material, (b) rock quality designation (RQD), (c) spacing of discontinuities, (d) condition of discontinuities, (e) ground water conditions and (f) orientation of discontinuities. Out of these parameters, quantification for discontinuity orientation is not well defined in the system. Excluding the last one, first five parameters constitute basic RMR which is unadjusted for discontinuity orientations. The fourth parameter, i.e., condition of discontinuities again depends on five subparameters and those are (i) discontinuity persistence, (ii) aperture, (iii) surface roughness, (iv) infilling and (v) weathering.

Slopes were divided into zones based on uniformity in lithology, slope orientation, joint numbers

and spacing and each zone was considered separately for the classification. All the required parameters for basic RMR were obtained from field except UCS, which was determined in the laboratory. Individual ratings were given for each parameter and finally added together to find the final rating for the slope mass. The ratings for the parameters are assigned according to Bieniawski (1989). RQD was introduced on the basis of core recovery (Deere 1963), however, as cores are unavailable for the slopes, the relationship given by Palmstrom (1985) has been used to obtain RQD from volumetric joint count (J_v) (Zhang 2016). J_v is the total number of joints encountered in a cubic metre volume of rock mass. The relationship is given by

$$RQD = 115 - 3.3J_v. \quad (1)$$

Basic RMR (RMR_{basic}) for each slope has been calculated to determine RMR and SMR and is given

in table 4. Slope 5 has the highest value of 63 and slope 6A being the lowest, has got a value 40.5. The RMR for the cut slopes has been determined by adding the adjustment factors for joint orientation given by Bieniawski (1989). There are some slopes (slopes 6A and 6B) which show negative RMR. This negative value can be attributed to high adjustment factor for joint orientation and should be considered in the very poor quality category, representing the rock mass in Class V of Bieniawski (1989) interpretation. According to his interpretation, all other slopes belong to Class III and IV, representing poor to fair rock mass quality.

3.2.2 Continuous SMR

Romana (1985) proposed SMR system by adding four correction factors to the basic RMR. Out of four, three correction factors depend on the relationship between discontinuities and the fourth one depends upon slope excavation method. Initially SMR was proposed for planar and topple failure and the ratings for wedge failure was added to the system by Anbalagan *et al.* (1992). The expression for continuous SMR is same as SMR and is as follows (Singh and Goel 1999)

$$\text{SMR} = \text{RMR}_{\text{basic}} + F_1 \times F_2 \times F_3 + F_4 \quad (2)$$

where F_1 depends on difference between dip direction of joint (α_j) and slope (α_s). In case of wedge failure, it depends on difference between α_s and trend of line of intersection α_i .

F_2 depends on dip of joint (β_j) in case of planar failure and plunge of line of intersection (β_i) in case of wedge failure. For toppling failure, F_2 remains 1 everywhere.

F_3 depends on difference between β_j and dip of slope (β_s) for planar failure and difference between β_i and β_s for wedge failure. For topple failure, it depends on $\beta_j + \beta_s$.

F_4 is the adjustment for method of excavation.

The ratings suggested by Romana (1985) for F_1 , F_2 , F_3 , and F_4 are discrete and relies more on judgement of the investigator. So it needs an experienced person to allocate rating for the factors. Tomas *et al.* (2007) suggested continuous functions for F_1 , F_2 and F_3 which best fits the discrete values.

He has given the following equations for the continuous functions:

$$F_1 = \frac{16}{25} - \frac{3}{500} \tan^{-1} \left(\frac{1}{10} (|A| - 17) \right), \quad (3)$$

Table 4. Rock mass rating table of all the parameters for different slopes.

Slope no.	UCS (A)	RQD (B)	Joint id	Joint spacing (C)	Joint condition (D)	Ground water rating (E)	RMR _{basic} (A + B + C + D + E)	Adjustment factor for joint orientation (F)	RMR (RMR _{basic} + F)
1	6	13	S ₀	8	9	15	51	-25	26
2	5	13	S ₀	10	10	15	53	-25	28
3	5	17	J ₁	9	12	15	58	-25	33
4	5	13	S ₀	10	8	15	51	-25	26
5	5	17	S ₀	10	16	15	63	-5	58
6A	4.5	13	J ₁	10	7	6	40.5	-50	-9.5
6B	4.5	13	S ₀	9	9	6	41.5	-50	-8.5
7	6	8	J ₁	5	10	15	44	-25	19
8A	7	13	S ₀	8	13	15	56	-25	31
8B	8	8	S ₀	10	11	15	52	-25	27
9	9	17	S ₀	8	10	15	59	-25	34
10*							42	-10	32

S₀ = bedding plane, J₁ = joint no. 1.

*RMR for the slope 10 is determined from the derived empirical relation between basic RMR and GSI (equation 9) for the studied area.

Table 5. Slope mass rating for different slopes using continuous functions.

Slope no.	Planar			Wedge			Continuous SMR = (RMR _{basic} + (F ₁ *F ₂ *F ₃)+F ₄)		
	F ₁	F ₂	F ₃	F ₁	F ₂	F ₃	F ₄	RMR _{basic}	W
1	—	—	—	0.51	0.72	-59.63	0	51	29
2	0.71	0.91	-59.57	0.18	0.22	-59.73	0	53	51
3	—	—	—	0.51	0.76	-59.58	0	58	35
4	—	—	—	0.34	0.25	-59.73	0	51	46
5	0.17	0.90	-59.59	—	—	—	0	63	—
6A	0.85	0.92	-59.50	0.95	0.92	-59.51	0	40.5	-12
6B	0.77	0.90	-59.59	—	—	—	0	41.5	—
7-W ₁	0.64	0.90	-59.55	0.35	0.68	-59.64	0	44	30
-W ₂	—	—	—	0.99	0.89	-59.57	0	44	-8
8A-W ₁	0.67	0.94	-59.47	0.37	0.94	-59.48	0	56	35
W ₂	—	—	—	0.80	0.92	-59.52	0	56	12
8B	0.95	0.97	-58.94	—	—	—	0	52	—
9-W ₁	0.67	0.97	-58.64	0.19	0.59	-59.58	0	59	52
-W ₂	—	—	—	0.85	0.97	-58.73	0	59	11

W₁ = wedge no. 1, W₂ = wedge no. 2.

$$F_2 = \frac{9}{16} + \frac{1}{195} \tan^{-1} \left(\frac{17}{100} B - 5 \right). \quad (4)$$

For planar and wedge failure,

$$F_3 = -30 + \frac{1}{3} \tan^{-1} C. \quad (5)$$

And for topple failure,

$$F_3 = -13 - \frac{1}{7} \tan^{-1} (C - 120) \quad (6)$$

where A = parallelism between slope strike and joint strike for planar and toppling failures and difference between slope strike and plunge direction of angle of intersection for wedge failure. B = dip of joint in planar failure and plunge of angle of intersection in wedge failure.

C = difference between angle of slope and dip of joint for planar failure, difference between angle of slope and plunge of line of intersection for wedge failure and addition of slope angle and dip of joint, in case of toppling failure.

The adjustment factors for SMR have been determined from continuous functions from equations (3, 4 and 5). All the slopes excluding slope 10 (due to unavailability of joints or systematic fractures) have been classified according to SMR system. The ratings for planar and wedge failures are given in table 5. The rock masses are very bad to bad in quality for most of the slopes and normal for slopes 4 and 5. Negative values are obtained for slope 6A and 8B for planar failure and 6A and 7 for wedge failure. These slopes with negative values can be considered in very bad rock mass and represent Class V of Romana (1985) interpretation. Slopes 4 and 5 represent the rock mass in Class III. Slopes 1 and 3 come under the Class IV, while slopes 2, 6B, 8A and 9 are in Class V. Romana (1985) has given a stability interpretation for slopes on the basis of SMR. Accordingly, slopes 2, 6A, 6B, 7, 8A, 8B and 9 are in very critical condition (completely unstable), slopes 1 and 3 are unstable, while slopes 4 and 5 are partially stable. From the field observations, it is envisaged that for slope nos 2, 4, 8B and 9, the stability is underestimated by SMR. This is due to exclusion of shear strength of joints in the rating system. However, if we count for the shear strength of the joints, it may provide a better approximation to the natural condition. For this type of conflict in the rating systems, solutions through numerical simulation is suggested by many researchers.

SN = Slope number

Figure 15. Plotted values in SR vs. SCR graph from Sonmez and Ulusay (2002) to obtain GSI.

3.2.3 Geological strength index

Geological strength index (GSI) is a convenient rock mass classification system which can be estimated in the field from visual observation of blockiness and discontinuity surface conditions of the rock mass (Hoek *et al.* 1998). Though it does

not consider discontinuity orientation, it is very helpful in determining overall rock mass health. The GSI for 11 slopes have been calculated according to modified quantitative GSI system proposed by Sonmez and Ulusay (1999, 2002), while slope no. 10 is estimated from visual observations (Hoek *et al.* 1998; Marinos and Hoek 2000). There are two

Table 6. Quantified GSI for studied slopes.

Slope no.	J_v	SR	R_r	R_f	R_w	SCR	GSI
1	17	30.2	3	2	3	8	35
2	13	34.9	3	0	5	8	36
3	10	39.5	3	2	3	8	38
4	14	33.6	3	0	5	8	36
5	11	37.8	5	2	5	12	47
6A	18	29.2	2	0	3	5	27
6B	18	29.2	3	0	5	8	34
7	21	26.5	3	0	5	8	33
8A	19	28.3	5	2	5	12	43
8B	21	26.5	5	0	5	10	37
9	8	43.4	3	0	5	8	39
10	—	—	—	—	—	—	29

Figure 16. Linear correlation of RMR_{basic} and GSI for 11 slopes of the studied area.

factors considered for the index, one is structure rating (SR) and another is surface condition rating (SCR). These two factors are represented by the following equations (Sonmez and Ulusay 2002).

$$SR = 17 \ln J_v + 79.8 \quad (7)$$

$$SCR = R_r + R_w + R_f \quad (8)$$

where J_v = volumetric joint count, R_r = roughness rating, R_w = weathering rating, and R_f = Infilling rating.

The value of SR and SCR are plotted in the graph (figure 15) to obtain GSI. The GSI value for slope no. 10 is estimated from visual observation. Obtained quantified GSI are given in table 6.

A correlation has been derived from linear fitting of RMR_{basic} vs. GSI for 11 slopes for the

studied area (figure 16). The equation derived from empirical relation is

$$\text{Basic RMR} = 6.46 + \text{GSI} \times 1.23. \quad (9)$$

This empirical equation may be well applicable to rock masses of similar kind of lithology and geological structures which are highly disturbed.

4. Conclusions

The kinematic analysis and geomechanical classifications of the rock mass are very important tools for the assessment of vulnerable slopes. Twelve cut slopes have been taken along NH-05 in Jhakri area and the SMR values indicate that the rock masses range from very bad to bad in quality for most of the slopes and normal for slope 5. The RMR represents poor to very poor rock mass quality except for location 5 which is fair in quality. Moreover, the kinematic results match with the field observations. The results obtained through continuous SMR can be useful in designing support systems and to strengthen the existing vulnerable slopes for safe transportation in the study area. It is further recommended to conduct numerical analysis for those slopes where SMR seems to underestimate the stability due to high joint shear strength.

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