Influence of Freeze-Thaw on the Stability of Road Cut Slopes near Atal Tunnel in Indian Himalayan Region

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Abstract

The rock slopes in the Indian Himalayan region are exposed to recurrent freeze-thaw. Freeze-thaw plays a dominant role in the instability of rock slopes in cold regions. Extensive field investigations have been carried on the two rock slopes of the Solang valley roadway in Himachal Pradesh. Freeze-thaw testing has been carried out in the laboratory on the rock samples collected from the studied slopes. The test has been repeated for 30 freeze-thaw cycles. The laboratory tests have been conducted to determine the mechanical properties of rock with and without freeze-thaw conditioning. The micro-photographs from scanning electron microscope have been obtained to identify the microscopic damages due to freeze-thaw activity. The macroscopic damages have been recognized in tensile, compressive and triaxial compressive strength of rock. Further, a three-dimensional numerical analysis has been performed to evaluate the stability and corroborate the results with the field observations. FT conditioning results reveal a reduction of about 27–39 % in rock strength and subsequently a decrease in the factor of safety by 10–39 % for rock slopes.

1. Introduction

The FT mechanism has been understood by two theories i.e. volumetric expansion (Tharp 1987) and ice segregation (Walder and Hallet 1985). Temperature and the duration of the FT cycle play a major role in the freezing depth. The rock slopes in the cold regions are generally prone to either diurnal FT or seasonal FT (Deprez et al. 2020). Research on microscopic and macroscopic damages on the rock under the influence of FT cycles has been conducted on fresh and weathered rock samples throughout the globe (Luo et al., 2014; Yavuz, 2011; Takarali et al., 2008; Turgul, 2004; Nicholson and Nicholson, 2000).

Ruedrich et al. (2011) investigated the physical weathering of rocks exposed to FT environment in building stones. Bayram (2012) predicted the loss in rock strength under the influence of cyclic FT conditions. Park et al. (2013) analysed the crack propagation and particle detachment using the Scanning Electron Microscope (SEM) photographs for rock samples conditioned up to 150 FT cycles. Ozbek (2014) concluded that the increase in porosity due to recurrent FT leads to weathering of rocks and that it has a deleterious impact on the strength of the rock materials. Momeni et al. (2016) proposed that the determination of ultrasonic wave velocity is the best parameter to understand the damage caused to rocks due to FT cycles. It detects the microscopic deterioration much before the macroscopic decay occurs. Ni et al. (2017) studied the decay in physico-mechanical properties when rock was subjected to cyclic FT by using chemical solutions in place of water. Mu et al. (2017) and Ma et al. (2018) concluded extensive experiments to understand the alteration in shear strength of joint planes and the dynamic compressive strength of rock under FT cycles. Li et al. (2018) investigated the influence of FT cycles on stability. Xu et al. (2019) used the analytical hierarchy process to evaluate the stability of the rock slope in the FT environment. Mousavi et al. (2020) degradation in strength of rock under FT cycles.

Instability in the rock slopes is frequently increased by freeze-thaw (FT) activity due to the climate change cycle occur in cold regions. Most of the Indian Himalayan region slopes are unstable and come under the

avalanche-prone area (Verma et al., 2021; 2019; Kumar et al., 2018). The slope failure can lead to fatalities, injuries and damages to infrastructures (Sardana et al. 2020; 2019a, b). Groundwater plays a significant role in slope failure (Jaswal et al., 2020). Water-rock interaction has an important impact on the safety and stability of rock slope engineering (Han et al., 2016). The Solang Valley roadway has been suffering from FT conditioning. About six months in a year, the slopes in this region are exposed to freezing, as the temperature drops to below sub-zero. Whereas for the remaining time, it is exposed to sunlight or rainfall. The rock slopes in this region are prone to rockfall (Verma et al., 2018). Allen (2016) observed the permafrost in the Kullu district. However, the effect of permafrost on rock behaviour and slope stability was missing. This study investigates how FT influences the mechanical rock properties and, subsequently, the stability of rock slopes. Since this area is unexplored by researchers so far in terms of the effect of freeze-thaw cycles, the lack of literature in the Indian Himalayan region concerning the influence of FT cycles makes this study necessary for this region.

2. Area Under Investigation

Solang valley is a side valley in the northwest of the Manali city of Kullu district of Himachal Pradesh, India. It is a popular tourist destination for ice skiing during winter and paragliding in the summer season. In the current scenario, the Manali-Leh highway is the only route to reach Leh from Manali that passes through Palchan, Rohtang Pass and Lahaul Spiti. Every year this highway is shut down during the winter season due to heavy snowfall. Palchan is a T-junction for Manali, Solang Valley and the Rohtang Pass (Fig. 1). Solang valley roadway plays a significant role in connecting the Palchan to the south portal of Rohtang tunnel (popularly known as Atal tunnel). This tunnel will provide all-weather connectivity from Manali to Lahul Spiti without passing through the Rohtang Pass. The rock slopes in the Solang Valley region are covered with snow (freezing effect) for about 5–6 months in a year and for the remaining period, these slopes are exposed to sunlight (thawing effect) and rainstorms. Thus, one natural freeze-thaw cycle takes about a year to complete.

Figure 1 Modified satellite image showing the location of rock slopes in Solang Valley road, Himachal Pradesh (*after* Google Earth)

2.1 Location

The slopes under investigation are located along the roadway from South Portal Rohtang Tunnel to Palchan in the Solang Valley region of Himachal Pradesh, India. Field investigations cover the three rock slopes along a stretch of about 15 *km* of Solang valley roadway (Fig. 3.24). The stretch is located between latitudes $32^{\circ}21'50.66''N$ to $32^{\circ}18'36.97''N$ and longitudes $77^{\circ}07'59.08''E$ to $77^{\circ}10'21.79''E$. The study area falls under toposheet number 143X3 of the survey of India. It comes under the northwest part of the Himalayas, with an elevation varying from 2364 to 3076 *m*. The coordinates and elevation of rock slopes of the area under investigation are provided in Table 1.

Table 1
Detail of road cut slopes along Solang Valley Road

Slopes under investigation	Latitude	Longitude	Elevation (<i>m</i>)	Distance from Rohtang tunnel (<i>km</i>)
South Portal of Rohtang Tunnel	32°21'50.66"N	77°07'59.08"E	3076	0
SL-1	32°20'02.1"N	77°08'42.61"E	2666	6.8
SL-2	32°19'29.0"N	77°08'58.1"E	2564	9.2

Table 1 Detail of road cut slopes along Solang Valley Road

2.2 Geology

The geological information reveals six major tectono-stratigraphic units around the Mandi-Rohtang area: Siwalik Group, Dharamsala Group, Larji-Rampur window Group, Chail Group, Jutogh Group and Vaikrita Group (Misra and Tewari, 1988), as shown in Fig. 2.

Figure 2 Geological map of a part of Kullu district, Himachal Pradesh (after Misra and Tewari, 1988)

The Beas/Vyas River originates from a glacier in the Higher Himalayas, cut across the topography and rocks are well exposed along the river section. The rocks in the study area are highly deformed and mylonitized. The rock types exposed around the study area are foliated micaceous quartzite, quartz-biotite schist, biotite porphyroblast schist, garnetiferous mica schist, kyanite-sillimanite schist and gneiss and fined grained banded gneisses.

2.3 Site Characterization

A detailed field investigation has been carried out on the two rock slopes along the Solang Valley roadway. The height and length of the slope SL-1 were ranging from 13 to 15 *m* and 80 to 85 *m* respectively. The roadway along this slope has a width of 6 to 8 *m*. Rock blocks that have the potential to fall were also analysed on the slope (Fig. 3a). The field photograph (Fig. 3b) shows that the slope comprises bedrock and weathered rock with a bulging portion on the bedrock. A concrete drainage passage was observed to be constructed along the toe of the slope to drain the water flow, especially during monsoon. The angle of the slope was varying from 65° to 90° in the direction of N030° ± 5°. Wedge failures were also observed on the slope surface.

Figure 3 Field photographs of slope SL-1 showing (a) vulnerable blocks on the upper section of the slope, (b) overall view of the slope

The height and length of the slope SL-2 were found to be 15–18 *m* and 40 *m* respectively. The dip of the slope was ranging from 77° to 82° with a dip-direction of N129° ± 5°. The upper portion of the slope was densely vegetated with trees (Fig. 4a). S-shaped folds were detected on the surface of the slopes.

Many wedge failures and vulnerable rock blocks associated with deep cracks were identified on the rock mass's failed portion (Fig. 4b). Based on the field survey, three sets of discontinuities were deciphered.

Figure 4 Field photographs of slope SL-2 showing **(a)** joint mapping on the slope and **(b)** overall view of the slope

A total of 40 observations of the discontinuities were plotted in the stereograph. Four sets of joints were observed on the slope SL-1 (Fig. 5a). The dip and dip direction of joints are $-57^{\circ}/151^{\circ}$, $80^{\circ}/094^{\circ}$, $77^{\circ}/353^{\circ}$ and $19^{\circ}/216^{\circ}$ for the joint sets $J1$, $J2$, $J3$ and $S0$, respectively. The joint spacing plotted in the histogram shows that joints $J1$, $J2$ and $J3$ are spaced close to wide, whereas the spacing of bedding joints ($S0$) was varying from extremely-close to moderate. The roughness and weathering were found to be slight to rough and slight to moderate, respectively. In comparison, three sets of discontinuities were observed on slope SL-2 from the 30 observations of poles (Fig. 5b). The orientation of these joint sets are represented as $J1:62^{\circ}/089^{\circ}$, $J2:79^{\circ}/159^{\circ}$ and bedding joint, $S0: 14^{\circ}/291^{\circ}$. The spacing of the joint sets $J1$ and $J2$ was varying from close to very-wide and, for bedding joint $S0$, it was extremely-close to moderate. The roughness and weathering were found to be slight to smooth and slight to high, respectively.

(a) Slope SL-1 (b) Slope SL-2

Figure 5 Stereograph plots showing discontinuities of rock slopes along Solang Valley roadway

3. Material And Methods

3.1 Laboratory Investigations

The rock samples of nearly-rectangular-shaped blocks were collected from the (i) freshly exposed surface of the slope and (ii) failed rock blocks that were dislodged from the slope to determine the influence of the FT conditioning on physico-mechanical properties of the rock and the stability of the rock slope.

3.1.1 Freeze-Thaw conditioning

FT conditioning on NX rock specimens was performed in the laboratory. The range of temperature and duration of FT for conditioning was decided based on the (i) recommendation given by the ASTM (2013) manual and (ii) temperature of the study area to replicate the effect of the natural FT cycle.

Figure 6 **(a)** Frozen rock specimens in a container, **(b)** Temperature curve for FT cycle

At the beginning of the FT test, NX size rock specimens were placed in the water-filled container such that specimens should be fully immersed. The freezing was carried out using a freezer at a temperature of -20°C (for approximately 12 h), followed by a thawing process, during which freeze specimens were placed in water at $+30^{\circ}\text{C}$ for the remaining 12 h. Figure 6a shows the frozen rock specimens after the freezing process. The test was repeated for 30 cycles of FT. The FT cycles follow Newton's law of cooling;

therefore, the almost same path would be followed by temperature change during each cycle (Mutluturk et al., 2004), as shown in Fig. 6b.

3.1.2 Scanning Electron Microscope analysis

SEM is the most frequent and multi-purposed microscope available to analyse the lithology of the rocks using electron beams (Zhou and Wang, 2007). In this section, the variation in the pores and cracks (generation or enlargement) in the rock specimens has been recognised by using SEM. The scattered pores and the fissility plane have appeared in the micro-photograph of rock specimen before FT conditioning (Fig. 7a). The enlargement and migration of the cracks have been observed in the micro-photograph of rock specimen conditioned with 30 FT cycles (Fig. 7b). Consequently, the microscopic results suggest that FT alters the microstructure of the rocks by initiating the new cracks and assisting in deepening or widening the existing cracks.

Figure 7 SEM photographs of a rock specimen (a) before freeze-thaw condition showing cracks, (b) after 30th freeze-thaw cycle showing enlargement of cracks

3.1.3 Tensile, compressive and triaxial compressive strength

The influence of FT conditioning on rock strength has been estimated in the laboratory. The tensile strength, compressive strength and triaxial compressive strength have been estimated for the rock, with and without FT conditioning. Figure 8 shows the specimens used during laboratory investigations. The result shows that both the tensile and compressive strength of the rock decreases under the FT environment. Similarly, cohesion and friction angle for intact rock shows reductions in their magnitude (Table 2).

Table 2
Mechanical properties of rock specimens with and without FT conditioning

FT cycle	Tensile strength (MPa)	UCS(MPa)	Cohesion; C(MPa)	Friction Angle; $\varphi(^{\circ})$
No FT conditioning (0th cycle)	13.1	99.57	16.50	49.61
With FT conditioning (30th cycle)	7.89	71.92	14.11	40.98

Table 2 Mechanical properties of rock specimens with and without FT conditioning

Figure 8 Specimens used in the laboratory to estimate (a) tensile strength, (b) compressive strength and (c) triaxial compressive strength

3.2 Kinematic Analysis

Kinematic analysis is a very useful tool for identifying potential modes of failure in the field of rock engineering. It shows the possibility of detachment of rock blocks along with free space without disturbing the mechanical stability of adjacent blocks. It also examines the probability of different modes of failure governed by the unfavourable orientation of discontinuities. The method assumes friction is the sole responsible factor for sliding. The potential modes of failure in both the slopes were assessed by kinematic analysis using DIPS 6.0.

The stereo-graph of slope SL-1 suggests two wedge failures (W1 and W2) and a direct toppling. Wedge failures occur due to the interaction of joints J_2 - J_3 towards NE direction and J_1 - J_3 in between NE and E direction (Fig. 9a). The orientation of wedges W1 and W2 were determined as $72^\circ/036^\circ$ and $23^\circ/077^\circ$, respectively. The interaction of joints J_1 with the bedding joints was responsible for direct toppling occurs towards NE direction. Kinematic analysis of slope SL-2 shows wedge failure with an orientation of $62^\circ/089^\circ$ towards the E direction due to the intersection of joints J_1 and J_2 in the critical region (Fig. 9b).

Figure 9 Kinematic analysis showing wedge failures for slope SL-1 and SL-2

3.3 Rock mass Characterization Studies

The rock masses of the study areas were characterized using (i) Rock Mass Rating (RMR), (ii) Geological Strength Index (GSI), (iii) Slope Mass Rating (SMR).

The five parameters involved in calculating the basic RMR (Bieniawski, 1979) were evaluated. The rating was assigned to them based on their condition observed during the field investigations and laboratory investigations. The quantified values of GSI (Sonmez and Ulusay, 2002) were estimated based on 'Structure Rating' (SR) and 'Surface Condition Rating' (SCR). SR was determined using the value of J_v and SCR was determined based on field investigations. SMR (Romana, 1985) is a significant extension to the RMR for preliminary assessment of rock slopes; its values are ranging from 0 to 100. It is derived from basic RMR by incorporating adjustment factors derived from slope-joint relationships and factors derived from the method of excavation. The assigned ratings from all three rock mass classification systems are provided in Table 3.

Table 3
Rating of various rock mass characterization studies along Solang valley roadway

RMR							
Location	UCS	RQD	Joint Spacing	Joint Condition	Groundwater Condition	RMR	
SL-1	7	8	8	9	15	47	
SL-2	7	3	8	6	15	39	
GSI							
Location	Structure Rating		Surface Condition Rating			GSI	
SL-1	27.4		8			34	
SL-2	21.5		6			27	
SMR							
Location	Failure	<i>RMR</i>	<i>F</i> ₁	<i>F</i> ₂	<i>F</i> ₃	<i>F</i> ₄	<i>SMR</i>
SL-1	W	47	0.15	0.4	-60	-8	35.4
	T		0.15	1	-25	-8	35.25
SL-2	W	39	0.15	1	-60	-8	22
where W, wedge failure; T, toppling failure							

Table 3 Rating of various rock mass characterization studies along Solang valley roadway

The comparison reveals that the rock mass description varied from poor to fair. SMR scores were low for both the slopes, even though RMR was considerably high (Fig. 10). This implies that even if the slopes have a fair or good rock mass quality, a slide can occur because of unfavourable orientations. This indicates the way joint orientation governs the stability in rock slopes. Therefore, the SMR method is recommended, as it incorporates the RMR and discontinuity orientation data to describe the rock mass and preliminary stability.

Figure 10 Comparison of popular rock mass classification systems based on rock mass description

3.4 Numerical Analysis

The numerical simulation uses mathematical models to analyse the mechanical behaviour of natural and excavated rock slopes. These models are computer-coded programs that characterize mechanical behaviour under a set of conditions (Wyllie and Mah, 2017). Three-dimensional Distinct Element Code (3DEC) is a three-dimensional tool based on the Distinct Element Method (DEM) that can be used for the discontinuum numerical modeling of the jointed rock slopes. It is suitable for studying the potential mode

of failure due to the presence of discontinuous features. The joint properties can be assigned independently to an individual or a set of discontinuity in a model. The DEM introduced by Cundall (1988) can include a large number of rock joints explicitly. In DEM, the rockmass is considered as an assembly of different intact rock blocks. The joints between intact blocks are treated as boundary conditions. The displacement and contact force at the boundaries of stressed intact blocks are evaluated from intact blocks assembly movements in previous steps.

4. Three-dimensional Dem Simulation

4.1 Three-dimensional slope geometry and input parameters

The field photographs of slope SL-1 show three major rock blocks extruded on the left side of the slope. Among those, one block was observed to be over-hanged on the upper-left portion (Fig. 3). The upper-right portion of the slope was observed to be extruded, whereas the lower-right portion was uniform. The extruded portion on the slope SL-2 was observed to be similar to the three-dimensional zigzag pattern, as shown in the field photograph (Fig. 4). The upper portion of the slope is an intact rock, whereas some loosened blocks were observed at the lower portion due to improper slope excavation.

A three-dimensional slope geometry was prepared that resembles the geometry of the actual slopes in the best approximation. The geometry of slope SL-1 incorporated the three major extruded blocks on the left side and one extruded block on the right side, as shown in Fig. 11a. The whole geometry of slope SL-2, both the upper and lower portions, was assumed as an intact rock. Three major joint sets were mapped in the three-dimensional model in the 3DEC program (Fig. 11b). Three major joint sets (two joint sets and one bedding joint) were mapped in the three-dimensional model in the 3DEC program.

(a) slope SL-1 (b) slope SL-2

Figure 11 Joint mapping in the 3D model of rock slope SL-1 and SL-2

The intact rock properties (with and without FT conditioning) are assigned to the blocks. The block properties, for instance, cohesion and friction angle has been determined using triaxial compressive strength, bulk modulus and the shear modulus have been determined by UCS test using strain gauges, tensile strength has been determined using BTS test and density of the rock have been determined in the laboratory in accordance with ISRM (1981) recommendations. The intact rock properties assigned to the blocks are provided in Table 4. Joint properties, for instance, dip, dip direction, cohesion and friction angle used in the numerical model were collected from field observation and previously published literature (Gischig et al., 2011; Barton, 1976). The behavior of the discontinuity sets is modeled using the Mohr-Coulomb slip model with residual strength. Intact rock bridges along large-scale discontinuities were modeled indirectly by assigning higher initial strength properties to discontinuities, as described by Jennings (1970) and recently applied by Fischer et al. (2010). The joint properties like joint stiffness were

evaluated from rockmass properties. The normal and shear stiffness of joints were estimated from rockmass deformation modulus, intact rock young's modulus and mean joint spacing with the help of Eq. 1–2. The joint properties (different properties for each joint) assigned to the three discontinuity sets for slope SL-1 and SL-2 are given in Table 5–6.

$$k_n = \frac{E_i E_m}{L(E_i - E_m)}$$

1

where E_m = rock mass modulus, E_i = intact rock modulus, k_n = joint normal stiffness, L = mean joint spacing.

Similarly, with the same analogy, the joint shear stiffness can be evaluated from Eq. 5.

$$k_s = \frac{G_i G_m}{L(G_i - G_m)}$$

2

where G_m = rock mass shear modulus, G_i = intact rock shear modulus, k_s = joint shear stiffness, L = mean joint spacing.

Table 4
Block properties used in the DEM analysis

Property	Without freeze-thaw	After 30th freeze-thaw cycle	Unit
Density	2689	2701	kg/m^3
Bulk modulus	12.51	3.38	GPa
Shear modulus	9.0	2.97	GPa
Cohesion	16.50	14.11	MPa
Friction angle	49.61	40.98	degree
Tensile Strength	13.68	7.37	MPa

Table 5
Joint properties used in the DEM analysis of slope SL-1

Property	Joint J_1	Joint J_2	Joint J_3	Unit
Dip	83	77	9	degree
Dip Direction	121	323	186	degree
Spacing	4	4	3	<i>m</i>
Joint normal stiffness	4.85	1.62	1.21	<i>GPa / m</i>
Joint shear stiffness	1.77	0.59	0.44	<i>GPa / m</i>
Friction angle	29-33			degree
Cohesion	0.6-1			<i>MPa</i>

Table 6
Joint properties used in the DEM analysis of slope SL-2

Property	Joint J_1	Joint J_2	Joint J_3	Unit
Dip	62	79	14	degree
Dip Direction	14	84	216	degree
Spacing	3	4	2	<i>m</i>
Joint normal stiffness	2.77	3.36	1.51	<i>GPa / m</i>
Joint shear stiffness	1.08	1.22	0.55	<i>GPa / m</i>
Friction angle	27-33			degree
Cohesion	0.2-0.8			<i>MPa</i>

4.2 Effect of the Freeze-Thaw Cycles on Displacement Contours and FoS

The numerical stability analysis for rock slope SL-1 was carried out under two conditions i.e. without (after 0th cycle) and with (after 30th cycle) FT conditioning. The DEM results without any FT conditioning show a maximum total displacement of 10.5 *mm* on the upper over-hanged block on the left portion of the slope. In comparison, the adjacent blocks on the same portion show a displacement of 8–10 *mm* (Fig. 12a). The displacement found on the block of the upper-right portion of the slope was 4–8 *mm*. The over-hanged block size in the air is larger than the other extruded blocks on the slope, making it more unstable than the others.

The maximum total displacement of the blocks after the 30th cycle of FT conditioning was increased to 17.9 *mm* and the factor of safety dropped to 1.07 from 1.77 (Fig. 12b). The FT conditioning degrades the physico-mechanical rock that results in higher displacement and less stability. The blocks that have high displacement are unstable and prone to failure.

Figure 12 Total displacement contours and FoS for the slope SL-1 **(a)** without freeze-thaw conditioning, and **(b)** after 30th cycle of freeze-thaw conditioning

Figure 13 Total displacement contours and FoS for the slope SL-2 **(a)** without freeze-thaw conditioning, and **(b)** after 30th cycle of freeze-thaw conditioning

In the stability analysis of slope SL-2, the block displacement shows a maximum value of 8.3 *mm*, whereas the displacement at the upper portion of the slope ranging from 3.5 to 6 *mm* (Fig. 13a). The value of the safety factor was found to be 2.05 after assigning the block properties estimated for the 0th cycle of FT conditioning. Unlike field conditions, no failure in the lower portion was observed in the DEM, as the lower portion of the slope was considered an intact rock in the DEM simulation. The results of DEM after assigning the block properties (estimated for 30th FT cycle) show a considerable increase in the displacement i.e. 29.1 *mm* and a drop in the safety factor to 1.84 (Fig. 13b) due to the deterioration of physico-mechanical properties of rock blocks after the FT conditioning. The zone which shows the maximum displacement is prone to failure. Thus, for rockfall assessment, the source point should be considered at the slope's upper portion.

5. Discussion

The stability of the slope depends upon the structural discontinuity, geo-mechanical properties of rock and the environmental conditions. The slope SL-1 shows *fair* rockmass and slope SL-2 shows *poor* rockmass based on RMR and GSI values. Compared to the SMR results, both the slopes fall under the unstable category i.e. SMR value of 21–40. The kinematic analysis shows that the wedge failure is prominent in both slopes. Further, laboratory investigations carried out in this study reveal that the mechanical properties of rock degrade under FT conditioning. The climatic information discloses that

over a half-year, the rock slopes maintained to a temperature below sub-zero. Figure 14 shows that FT conditioning of deteriorated rock has a substantial effect on slope stability. The higher displacement of the block in the analysis due to deteriorated rock properties leads to decreased FT. The FoS observed very-close to 1.0 for slope SL-1 and 1.8 for slope SL-2 for rock specimens conditioned with FT. The initially damaged rocks are further progressed deeper because of deterioration caused by FT conditioning.

Figure 14 Influence of FT on the stability of the slopes

6. Conclusion

The DEM analysis aims to forecast the movement, identify the vulnerable section, and determine the slope's safety factor. The blocks that show higher displacement are prone to failure and can be considered an unstable zone. These unstable zones can be further treated as source points for the quantified risk assessment of falling rock blocks. The results of the DEM validate the finding of kinematic analysis and field observations. The wedge formations due to discontinuity sets and the unstable zone observed in the DEM of the rock slopes were validated the wedge failures observed in the kinematic analysis and other modes of failure observed in the field survey.

The FoS in DEM was found close to 1. Therefore, it is concluded that the slope is stable to critically stable. Even a small local or global disturbance tends to reduce FoS and lead to slope failure. Therefore, it is concluded that the FT cycle has a significant influence on rock slope stability. As the FT cycle increases, the slope movement increases and the safety factor of the slope decreases.

Declarations

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Conflicts of Interest

The authors wish to confirm that there are no conflicts of interest with this research article.

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Figures

Figure 1

Modified satellite image showing the location of rock slopes in Solang Valley road, Himachal Pradesh (*after* Google Earth)

Figure 2

Geological map of a part of Kullu district, Himachal Pradesh (*after* Misra and Tewari, 1988)

Figure 3

Field photographs of slope SL-1 showing (a) vulnerable blocks on the upper section of the slope, (b) overall view of the slope

Figure 4

Field photographs of slope SL-2 showing (a) joint mapping on the slope and (b) overall view of the slope

(a) Slope SL-1

(b) Slope SL-2

Figure 5

Stereograph plots showing discontinuities of rock slopes along Solang Valley roadway

Figure 6

(a) Frozen rock specimens in a container, (b) Temperature curve for FT cycle

Figure 7

SEM photographs of a rock specimen **(a)** before freeze-thaw condition showing cracks, **(b)** after 30th freeze-thaw cycle showing enlargement of cracks.

Figure 8

Specimens used in the laboratory to estimate (a) tensile strength, **(b)** compressive strength and **(c)** triaxial compressive strength

Figure 9

Kinematic analysis showing wedge failures for slope SL-1 and SL-2

Figure 10

Comparison of popular rock mass classification systems based on rock mass description

Figure 11

Joint mapping in the 3D model of rock slope SL-1 and SL-2

(a)

(b)

Figure 12

Total displacement contours and FoS for the slope SL-1 (a) without freeze-thaw conditioning, and (b) after 30th cycle of freeze-thaw conditioning

(a)

(b)

Figure 13

Total displacement contours and FoS for the slope SL-2 (a) without freeze-thaw conditioning, and (b) after 30th cycle of freeze-thaw conditioning

Figure 14

Influence of FT on the stability of the slopes