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ORIGINAL PAPER

ASSESSMENT OF ROCKFALL HAZARD OF HILL SLOPE ALONG MUMBAI-PUNE EXPRESSWAY, MAHARASHTRA. INDIA

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ABSTRACT

The rockfall hazard in high hills of deccan pleatue has been one of the major disasters in western part of India. The present study aimed to assess the stability of hill cut highway slope near Khandala tunnel along Mumbai-Pune expressway, Maharashtra, India. This study is significant as the Khandala tunnel is one of the five tunnels situated along the Mumbai-Pune expressway, which experiences frequent rockfall and slope instability, mostly in the rainy season. The assessment of slope failure in the present study performed by a combination of rockfall simulation and 2D slope stability analysis. The results of the slope analysis show that the slope is stable in dry condition having a factor of safety (FoS) 1.48. In contrast, it becomes very critical during the rainy season having FoS of 1.07. Rockfall analysis shows that the boulders came down on both carriageways with approximate maximum bounce height, total kinetic energy, and translational velocity of 9 m, 500 kJ, and 16 m/s, respectively. The preventive slope stabilisation measures suggested to minimise the failure problems and to capture the movement of falling blocks before reaching the road.

1. INTRODUCTION

Rockfall is the most critical hazard mainly observed in the high hilly region during rains or sometimes associated with natural disasters like earthquake and volcanic activities (Calder et al., 2002; Huang, 2011; Umrao et al., 2011, 2015, 2016; Ansari et al., 2014; Zheng et al., 2015; Singh et al., 2016). It is generally smaller in areal extent but may still be dangerous and cause damage to property and human lives. Blocking and shutting down of roads due to rockfall for days at a time is a constant problem in many mountainous regions. Therefore, rockfall hazard assessment for the construction of highways and planning the cities in mountain areas have crucial importance. Several studies performed for assessing the rockfall risk along roads, but minimal studies carried out to investigate the stability along Mumbai-Pune expressway, Maharashtra, India. The Mumbai-Pune expressway is India's first six-lane concrete, high-speed expressway and one of India's busiest roads. It spans a distance of 94.5 km connecting Mumbai and Pune. It has five illuminated, ventilated tunnels named as Bhatan, Madap, Adoshi, Khandala and Kamshet. Khandala tunnel is a curved pair of a tube. The northern portion is called Mumbai-Pune tube which is 320 m whereas the southern portion referred Pune-Mumbai tube of 360 m length. The heavy rainfall in June and July 2015 triggered disastrous rockfall near the Khandala tunnel which halted the traffic for almost two months.

In recent years, several qualitative and quantitative techniques used for the assessment of rockfall. The annual probability of loss of property value or life estimated by quantitative risk assessment (QRA) method (Corominas et al., 2005; Pantelidis, 2011; Ferlisi et al., 2012; Mignelli et al., 2012). In the rockfall hazard rating system (RHRS), a value given to the responsible factors such as geological character, slope geometry, average vehicles risk, the magnitude of failure, roadway width, rockfall history, the number of rockfall events, and climate to get a rating of the risk (Ansari et al., 2012; Pellicani et al., 2015). Numerical codes also used to simulate the trajectories of rock blocks falling down the slope and to assess the possibility of diverse rockfall paths (Guzzetti et al., 2002; De Almeida and Kullberg, 2011). Rockfall trajectory simulation is one of the complex tasks which plays a vital role in the understanding of the boulder fall paths. This type of model employs detailed information of the material, including the coefficient of friction, restitution, size and shape of falling blocks (Vijayakumar et al., 2011). The soundness of these models governed by the reliability of the input parameters and the rockfall processes, including free-

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falling, impact, rolling and sliding (Pellicani et al., 2015). The assessment of rockfall hazard has had much application, including for road cut slopes (Ansari et al., 2012, 2018; Ferlisi et al., 2012; Ahmad et al., 2013; Singh et al., 2013a), protection and preservation of historic sites and monuments (Wang et al., 2014; Topal et al., 2007, 2012) and effects on forests (Perret et al., 2004; Dorren et al., 2004). Though the rockfall studies carried out in many locations, the results obtained from one location may not apply to the other as there is high variability in input parameters, rockfall process, and energy distribution system. The rockfall hazard analysis using 2D and 3D rockfall programs carried out at Saptashrungi gad temple hill to assess the two parikrama paths by dividing the hill into eight different zones (Ansari et al., 2018). The rockfall hazard zone map prepared to demarcate potential sites of rockfall risks. The rockfall prone area in Crolles municipality (Massif de la Chartreuse, French Alps) was assessed through quantitative risk assessment procedure. The results expressed as the average surface area of the building destroyed each year, demonstrate (i) the protective effect of the forest cover for volumes from 1 cu. m to 7 cu. m, and (ii) the large share of total risk due to the intermediate volume classes (7-12 cu. m). The study also demonstrated the value of a quantitative QRA type approach for zoning and rocky risk management (Farvacque et al., 2019).

The stability assessment of the slope in terms of a factor of safety (FoS), displacement, and shear strain on both dry and saturated conditions become essential. It gives the idea about the reduction in FoS of the slope due to percolation of water, developed maximum displacement and its corresponding shear strain. Over the years, several numerical techniques have used to understand the slope failure mechanism and to estimate the factor of safety. Among all the numerical simulation approaches, the limit equilibrium methods (LEM) is simple and widely used among civil engineers and engineering geologist (Kanungo et al., 2013; Singh et al., 2016b; Umrao et al., 2016). However, LEM is a conventional technique and has limitations and a large number of assumptions (Sharma et al., 2016). To overcome these, various researchers have employed different numerical tools based on the continuum and dis-continuum approaches (Umrao et al., 2012, 2017; Singh et al., 2013 a, b; 2014; 2017; Kainthola et al., 2015). Finite element method (FEM) is one of the most potent tools for slope stability analyses. This analysis provides many advantages over conventional methods as it requires less prior assumptions, gives more accuracy and versatility (Singh et al., 2013 b; Sharma et al., 2016, 2017). In this method, the FoS determined by the combination of the finite element method (FEM) with the strength reduction technique (Matsui and Sam, 1992; Griffiths and Lane, 1999).

The present study aims to perform the stability analysis on a hill cut slope near Khandala tunnel along Mumbai-Pune expressway, located in Maharashtra, India. The stability investigation and rockfall study carried out using Phase^2 v.2.7 and RockFall v.4.0, respectively (Rocscience Inc, 2012). RocFall is an advantageous statistical analysis computer program based on the laws of motion and collision theory (Singh et al., 2016a). This program can determine the locations of the rock end point, energy, velocity, bounce height and falling rock blocks trajectory. Phase^2 is a computer program based on the finite element method in which the entire problem domain is discretised, then solves the solution of the entire problem. The maximum total displacement and developed maximum shear strain with strength reduction factor (SRF) are estimated using the Phase^2 numerical program. The results obtained by the numerical and statistical analyses validated by field observations and an actual rockfall event that occurred in 2015.

2. STUDY AREA

The rockfall site located at 18°46'4.07", 73°22'6.31" and 495 m of latitude, longitude and elevation, respectively along Mumbai-Pune expressway, near Khandala tunnel, Maharashtra, India. Khandala tunnel is one of the main tunnels along Mumbai-Pune expressway, which has undergone many slope failure and rockfall hazard problems mainly in the rainy season. The Deccan basalt of Cretaceous age covers study area with variable relief ranging from 20 m to 100 m. The study area is very near to Khandala hill station, which is the most promising area for hiking. The study area is in a tropical climate zone, with an average annual temperature of 26.5°C. The region receives heavy rainfall with an annual rainfall of around 900 mm and is one of the dampest places in Maharashtra. The monthly variations in precipitation and temperature are shown in Figure 1. The maximum rainfall and temperature are encountered during July to September and April to June, respectively. The total vertical height of the slope is about 25 m, out of which about 6 m is prone to failure (Fig. 2). The rockmass exposes few randomly oriented joints, which plays a significant character in the initiation of failure. The talus and green flora cover the top portion of the slope face, but the face of the cut slope is almost barren.

2.1. GEOLOGY OF THE AREA

The study area is covered by Deccan basalt of Upper Cretaceous to Lower Eocene age, which is occasionally intruded by several basic intrusive. The basalt is capped by laterite soil due to the high weathering rate of basalt and the favourable environmental condition for the alteration of the rockmass. The micro and macro investigation of the rock shows that the fine-grained crystals are dominantly present due to rapid cooling of the erupted magma. The chemical analyses revealed that the rock is basic in composition having lower silica content

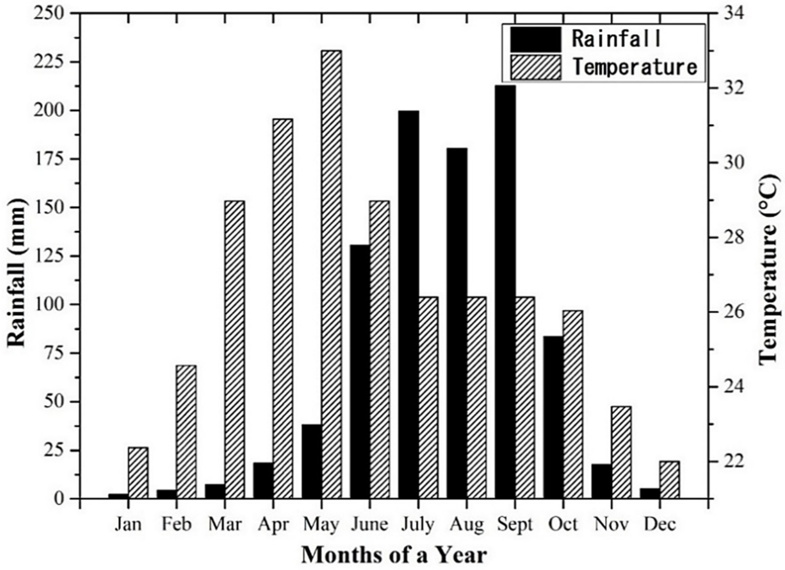

Fig. 1 Monthly variation of rainfall and temperature (Source: http://en.climate-data.org/location/967336/).

Fig. 2 A front view showing hill cut slope face with rockfall failure zone and fallen rock blocks.

Table 1 Southwestern Deccan volcanics stratigraphy (Subbarao and Hooper, 1988).

Group	Subgroup	Formation
Deccan Basalt	Wai	Panhala
		Mahabaleshwar
		Ambenali
		Poladpur
	Lonavala	Bushe
		Khandala
	Kalsubai	Bhimashankar
		Thakurvadi
		Neral
		Igatpuri
		Jawhar

Table 2 Geotechnical properties of rock determined at dry and saturated conditions.

Geotechnical properties	Number of samples	Unsaturated condition				Saturated condition			
		Min.	Max.	Mean	St. Dev.	Min.	Max.	Mean	St. Dev.
Density (g/cm^3)	10	2.58	2.79	2.67	0.06	2.65	2.82	2.74	0.06
UCS (MPa)	10	43.56	71.52	56.23	9.20	31.25	69.89	50.50	11.96
BTS (MPa)	10	6.16	8.95	7.79	0.78	5.85	8.01	6.69	0.88
E (GPa)	10	37.41	53.25	44.73	4.64	34.12	52.10	42.76	5.16
Cohesion (MPa)	10	12.25	16.12	14.32	1.33	11.10	15.56	13.38	1.62
Angle of internal friction (°)	10	36.00	41.00	38.57	1.40	33.00	39.00	36.29	1.49

than average which indicates the great extent flow of the erupted lava before solidification. The area consists of highly dissected terrain with flat summits and entrenched valleys. The steeper portion of the area indicates harder rock, whereas the flat summits are the indication of the weak rock and prone to weathering and erosion. The lava flows depict the various type of geological structures such as joints, fractures, vesicles, veins, spheroidal weathering and amygdules. Mostly, three sets of joints observed in the study area, but some random minor fractures are also present, which play a significant role in the dislocation of the rockmass. Some joints and micro-fractures developed during the cooling of hot lava while some are of tectonic origin. Tectonic fractures and joints played a crucial role in the development of the drainage, escarpment and ridges. Total stratigraphic thickness about 3000 m, has been divided into three subgroups and eleven formations based on geochemical and field markers (Table 1). Stratigraphically, the study area belongs to "Lonavala" subgroup underlain by Bhimasankar Formation of Kalsubai subgroup and overlain by Bushe Formation of Lonavala subgroup.

2.2. *GEOTECHNICAL CHARACTERISATION*

The rockmass in the study area is highly jointed, with three significant sets of joint. Two joints are vertical to sub-vertical and third joints is nearly sub-horizontal. The joints distributed with very high irregular spacing. It varies from 47 to 65 cm between one set of joints. The continuity of joints of one set varies from 1 to 4 m separated by intact rock bridges. The joints are tight, undulating, and rough, and the surficial rocks show openings induced by uncontrolled blasting and excavation techniques. The higher intensity of joints is present in amygdaloidal basalt than the massive basalt. In addition to the field condition of the rockmass, the geotechnical properties of intact rocks evaluated by performing various laboratory experiments as per International Society for Rock Mechanics (ISRM), and American Society for Testing and Materials (ASTM) recommended standards. The NX size rock cores were drilled from the collected rock blocks using a diamond core drill bit. The intact core samples used for evaluation of strength parameters. All the geotechnical properties viz. density, unconfined compressive strength (UCS), Brazilian tensile strength (BTS), Young's modulus (E), cohesion and angle of internal friction determined for both dry and saturated conditions (Table 2).

The collected rock samples classified into three categories, namely massive, amygdaloidal and weathered basalts based on macroscopic observations.

3. METHODOLOGY

3.1. *ROCKFALL SIMULATION*

The rockfall process covers free fall, elastic bouncing, rolling, and sliding (Guzzetti et al., 2002; Dorren, 2003; Shuzhi et al., 2006; Labiouse and Heidenreich, 2009). It has been studied and modelled by an experimental method, which includes field tests as well as physical modelling (Giani et al., 2004) and

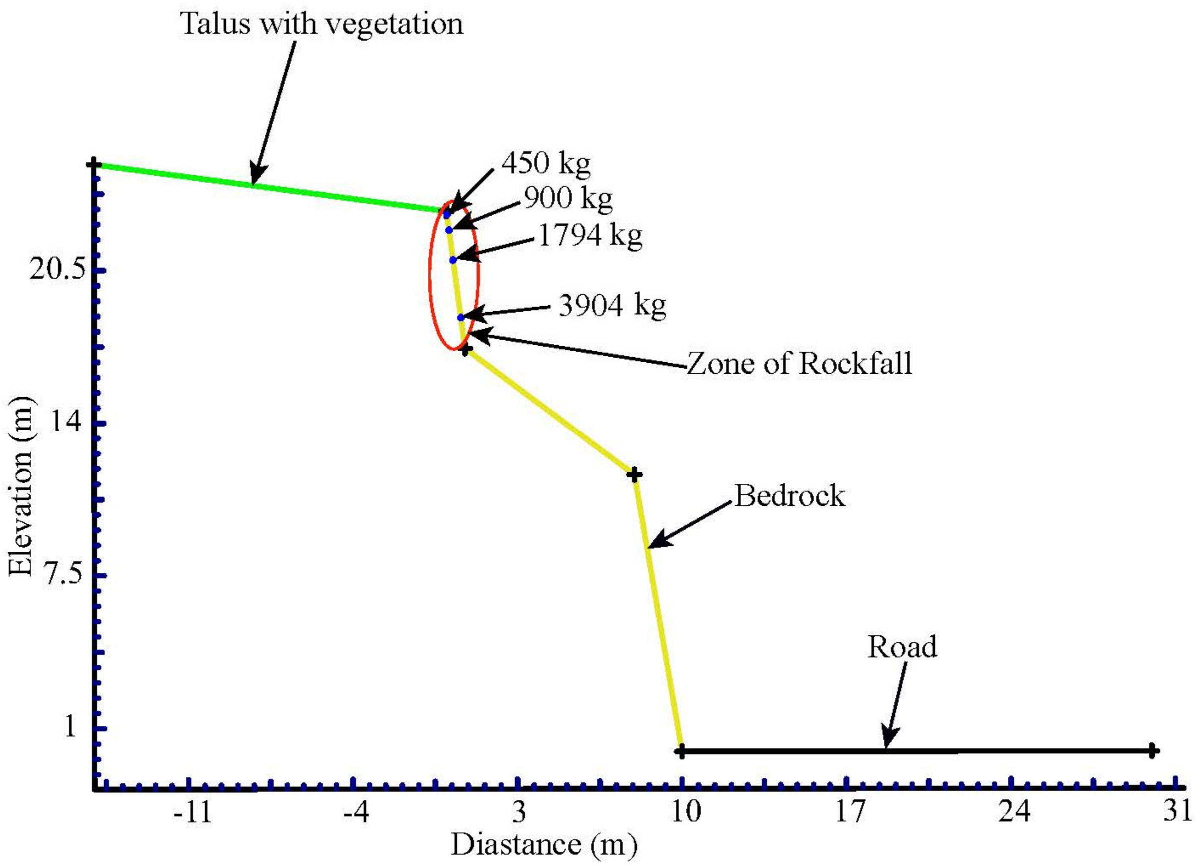

Fig. 3 Slope geometry for rockfall simulation showing vulnerable zone of failure.

the computational method by performing mathematical modelling (Azzoni et al., 1995). Mathematical models based on the lumped mass method and rigid body method. In the lumped mass method, the falling rock block is considered as a particle, and the reduction of the velocity after impact is modelled using the restitution coefficient. In the rigid block model, the rotation induced due to inertial moment is modelled using the shape and volume of the rock block (Chang et al., 2011). In the present study, the rockfall simulation performed using RocFall v 4.0 (Rocscience Inc, 2012) software, based on the lumped mass model. It is a statistical program to determine energy, velocity, and bounce height for the entire slope as well as for the location of rock endpoints. It also plays a vital role in the installation of remedial measures. A detailed field mapping carried out to find out the approximate slope geometry and the size of the potentially unstable rock blocks. The block volume (V_b) also estimated by using traditional formula assuming a persistent joint set (Kim et al., 2007; Palmstrom, 2001).

$$V_b = \frac{s_1 s_2 s_3}{sin\gamma_1\, sin\gamma_2\, sin\gamma_3} \qquad \text{(Eq. 1)}$$

Where s_1, s_2 and s_3 are the spacing in each joint set and γ_1, γ_2 and γ_3 are the angle between the joint sets. The high variation in joints persistence and spacing allows the formation of variable sizes of rock blocks. The weight of blocks is determined using the volume and the unit weight of the rock. In this study, 450 kg, 900 kg, 1794 kg, and 3904 kg mass of the rock blocks are determined and used for the simulation purpose. It was observed that the mass of the rock blocks increases towards the bottom of the slope as the spacing between the joints increased in the same manner. The slope geometry with the best possible approximation to represent the source of the rockfall is shown in Figure 3. The vertical height of the slope face above the road level is about 25 m. The upper, middle and lower slope faces are inclined at an angle of 82, 37, and 80, respectively. In addition to slope geometry, the coefficient of restitution (CoR) is another most important input parameter. It is defined as the ratio between the magnitudes of the rebounding and incoming velocities (Paronuzzi, 1989; Azzoni and Freitas, 1995). Its value can be measured by field tests (Evans and Hungr, 1993; Robotham et al., 1995; Giani et al., 2004), by back analysis (Descoeudres and Zimmermann, 1987; Paronuzzi, 1989; Pfeiffer and Bowen, 1989; Kobayashi et al., 1990; Evans and Hungr, 1993; Budetta and Santo, 1994) or by theoretical estimation (Bozzolo and Pamini, 1986; Kobayashi et al., 1990). The coefficient of restitution not only depends upon the slope material but also on

Table 3 Input parameters of rockfall on different surfaces.

Input Parameters	Bedrock	Asphalt	Talus with vegetation
Normal restitution	0.35±0.04	0.4±0.04	0.3±0.04
Tangential restitution	0.85±0.04	0.9±0.04	0.8±0.04
Friction angle	35±2	33±2	30±2
Initial velocity (m/sec)	0.05		

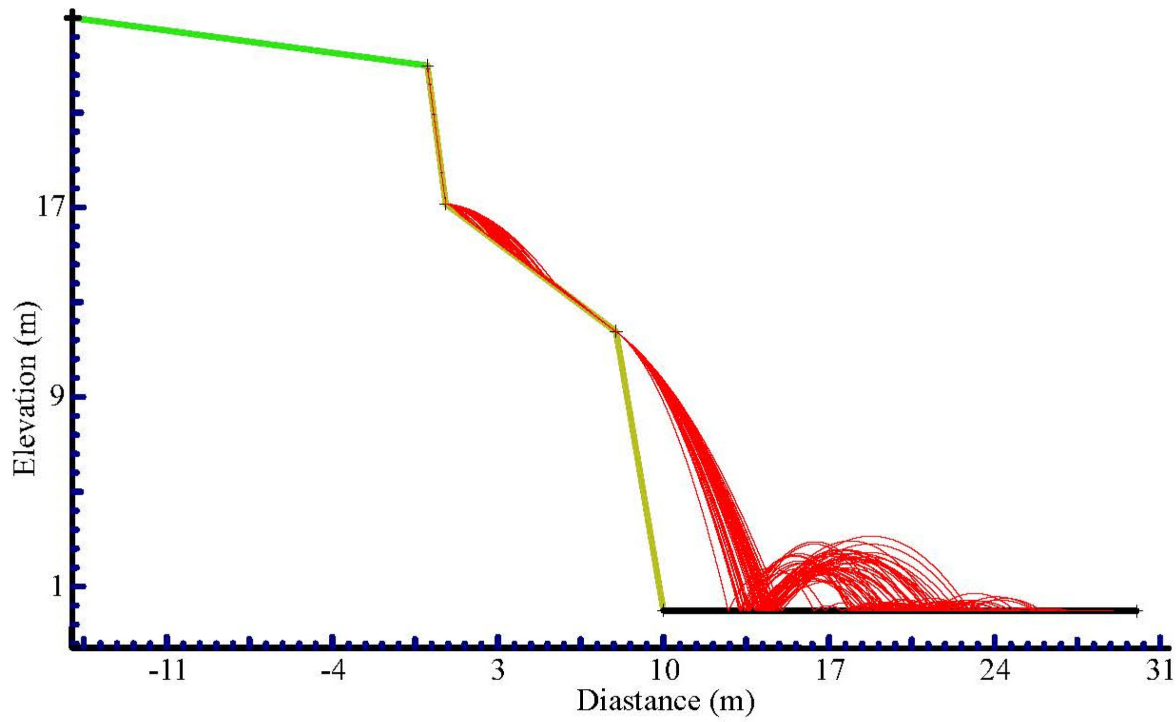

Fig. 4 Model of trajectories of the falling rock blocks.

various other parameters like slope angle, impact velocity, incident angle, the position of the impact, rotation, contact area, weight, size and shape of the falling blocks (Ansari et al., 2014; Vishal et al., 2016). In rockfall analysis, the CoR is evaluated only based on the outcropping material and vegetation. The analysis was performed by using the values of all input parameters from the published literature (Singh et al., 2013a; Ansari et al., 2015; Singh et al., 2015) (Table 3). Rockfall motion for bounce height, energy, and velocity and rock block terminal point is related to the geometry of the slope. The simulated trajectory of the rockfall is shown in Figure 4.

3.2. FINITE ELEMENT MODELLING

The finite element method (FEM) has emerged as a more robust alternative approach for the slope stability analysis over conventional methods. It needs very few prior assumptions for the stability analyses of the slope. The superiority of FEM over conventional limit equilibrium method can be found as no prior assumption to be made regarding shape or location of the failure surface, no concept of slice force and monitor progressive failure until universal failure reached (Kanungo et al., 2013; Mahanta et al., 2016; Singh et al., 2016b). In this method, the factor of safety (FoS) of the blocky slope is calculated in terms of a factor by which original shear strength parameter is divided to bring the slope to the limits of failure. This factor is termed as "strength reduction factor (SRF)", and method is called as "shear strength reduction (SSR)" technique (Matsui and San, 1992; Griffiths and Lane, 1999). The FoS along the sliding surface is defined as

$$F_s = \frac{\tau}{\tau_f} \quad \text{(Eq. 2)}$$

Where τ, the shear strength of the slope material, which is calculated through Mohr-Coulomb criteria

$$\tau = C + \sigma_n \tan\varphi \quad \text{(Eq. 3)}$$

And τ_f, the shear stress on the sliding surface which can be calculated as

$$\tau_f = C_f + \sigma_n \tan\varphi_f \quad \text{(Eq. 4)}$$

where C_f and φ_f are related to shear strength parameters of the slope by a factor called as strength reduction factor and defined as

$$C_f = \frac{C}{SRF} \quad \text{(Eq. 5)}$$

$$\varphi_f = tan^{-1}\left(\frac{tan\,\varphi}{SRF}\right) \quad \text{(Eq. 6)}$$

In the present study, the slope was analysed using Phase[2] software based on FEM. The input parameter for this approach is estimated either from the laboratory analysis or from the available literature published by many experienced geoscientists and engineers in the field of rock mechanics and mining engineering. The compressive strength (σ_c) of the intact rock was determined by laboratory analysis at both unsaturated and saturated conditions. Geological Strength Index (GSI) and material constant (mi) were estimated using the chart provided by Marinos and Hoek (2000). Mohr-Coulomb strength parameters were computed with the help of Roclab1.0 software (Roclab, 2007). The elastic modulus of the rock mass is another important parameter having a deep influence on the computed deformation before failure (Griffiths and Lane, 1999). It was determined based on GSI, compressive strength and disturbance factor (D) provided by Cai et al. (2004) as

$$E = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_c}{100}}\,10^{((GSI-10)/40)}$$

(In GPa; for σ_c<100 MPa) (Eq. 7)

The discretisation and mesh generation of the finite element method is a necessary task to solve the partial differential equation for practical problems. The two-dimensional slope boundaries are discretised using 3-node triangular plain strain, and then a uniform meshing option has been selected to generate finite element mesh. Several trial and error approach has been adopted to reach an acceptable mesh resolution. The load generated by both an initial vertical stress and a body force is computed. The initial vertical stress is determined from the weight of the material, and the unit weight of the material is used to calculate the body force. The stability of the slopes is governed by several possible failure criteria. Non-convergence of the solution is one of the suitable indicators of the failure (Griffiths and Lane, 1999), employed as a failure criterion in the present study.

4. RESULT AND DISCUSSION

Rocks of any type are prone to weathering and deterioration of strength with time. This leads to fracturing of rocks and further opening of joints, a process responsible for the promotion of rockfall/rockslides (Dorren, 2003). The purpose of the present study was to determine the source of rockfall, their damaging capacity, the trajectory of the falling body, bounce height etc. along with strain localisation and displacement observed due to joint deformation. For rockfall analysis, it is essential to identify the promoter as well as the size and volume of falling mass. Although, most rockfall results from the direct detachment of surficial rocks, rockslides can be sub-surficial. Past researchers have also observed rockfall as well as rockslides in basaltic rocks of similar geological formation and geotechnical conditions (Kainthola et al., 2012, 2015; Ahmad et al., 2013; Singh et al., 2015; Ansari et al., 2016). Keeping past failure activities and the mechanism associated with it in a note, a detailed analysis was carried out for both rockfall and rockslides.

For rockfall analysis, the source of potentially unstable rock blocks was identified during field investigation through joint mapping. The representable block size was estimated using equation 2 for persistent joint sets. For rockslide analysis by FEM, input parameters are used from Table 2. The results of the analysis are discussed in terms of parameters like bounce height, rock endpoints, the trajectory of blocks, translational velocity, kinetic energy along with displacement and shear strain due to joint deformation.

4.1. BOUNCE HEIGHT

The height to which a rock block will bounce depends upon the surface topography, surface composition, block composition, slope face angle, shape of the blocks and initial velocity given for the simulation (Topal et al., 2007; Lambert et al., 2012; Pellicani et al., 2015). The detached rock blocks started with slow initial movement against the frictional force along the slope face. The bounce height and the end points of the falling rock blocks are shown in Figure 5. The maximum bounce height achieved by the blocks is about 9 to 10 m above the slope. This value is slightly conservative as surface irregularities are not taken into consideration. The second and third major impacts took place on the roadside at about 13 m and 20 m from the base of the slope face, respectively. Bounce height continuously decreases after these two impacts. It was also observed that the rock blocks fell on both sides of the highway lane. For this reason, proper protection measures should be installed to minimise the risk of damage on both sides of the highway.

4.2. TOTAL KINETIC ENERGY AND TRANSLATIONAL VELOCITY

Total kinetic energy is directly proportional to the mass of the falling blocks and the square of the velocity. Therefore, the bigger size of blocks will be more disastrous as these blocks will tend to impart more energy. Translational velocity and total kinetic energy continue to rise after the impact on the road, reaching a maximum of 16 m/sec and 500 kJ, respectively. Perret et al. (2004) have classified the hazardous intensity zones into three categories based on total kinetic energy. The intensity zones have

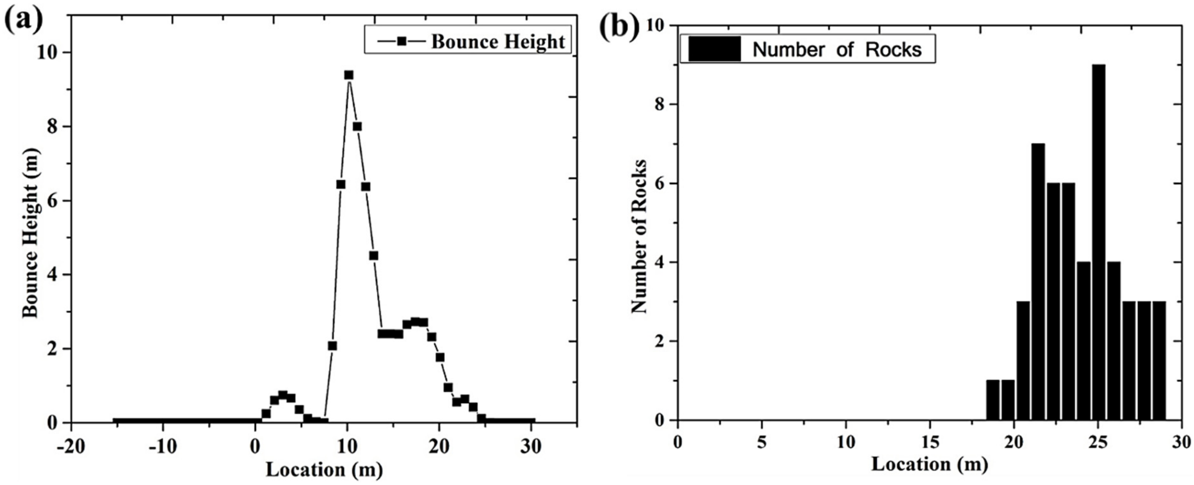

Fig. 5 (**a**) Bounce height variations with horizontal location and (**b**) horizontal location of rock endpoints.

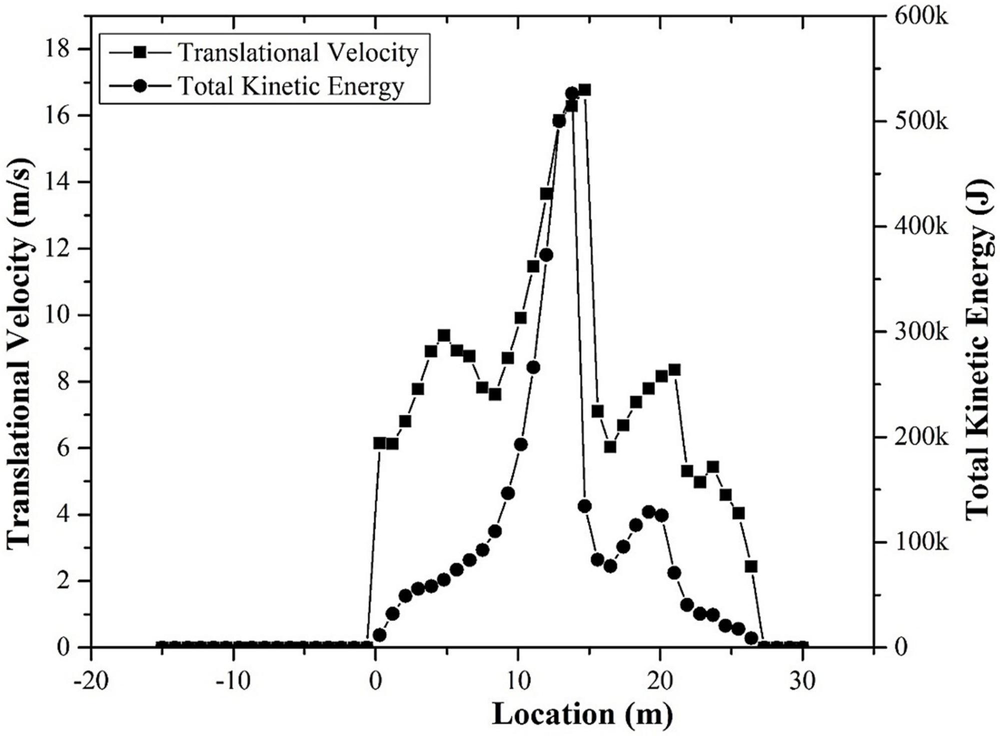

Fig. 6 Translational velocity and total kinetic energy variation with horizontal location.

a threshold total kinetic energy of 30 kJ, 300 kJ. The results from the present study fall into the highest intensity zone, which is about 500 kJ (Fig. 6). It was reported that the high-intensity zone could cause substantial damage, which was also observed in the present study by field observation.

4.3. DISPLACEMENT AND SHEAR STRAIN

The overall stability of the 25 m steep slope was evaluated using the FE method. The cross-jointed networks having spacings of 0.85 m and 1 m for bedding and cross joint sets, respectively. After analyses, the slope was found to be stable under the dry condition with a FOS of 1.48, while the FOS for the saturated condition was 1.07, which indicates slope is theoretically stable but practically unstable (Fig. 7). This shows that rainwater plays a very significant role in the reduction of FOS. Slope sections show maximum displacement near the toe converging toward the daylighted joints. At dry condition, maximum displacement was recorded about 1.17 m corresponding to a maximum shear strain of 0.21, while the 1.30 m of maximum displacement was observed corresponding to shear strain of 0.35. Shear strain contour covers the entire slope, which indicates the circular type of failure. Pain et al. (2014) identified

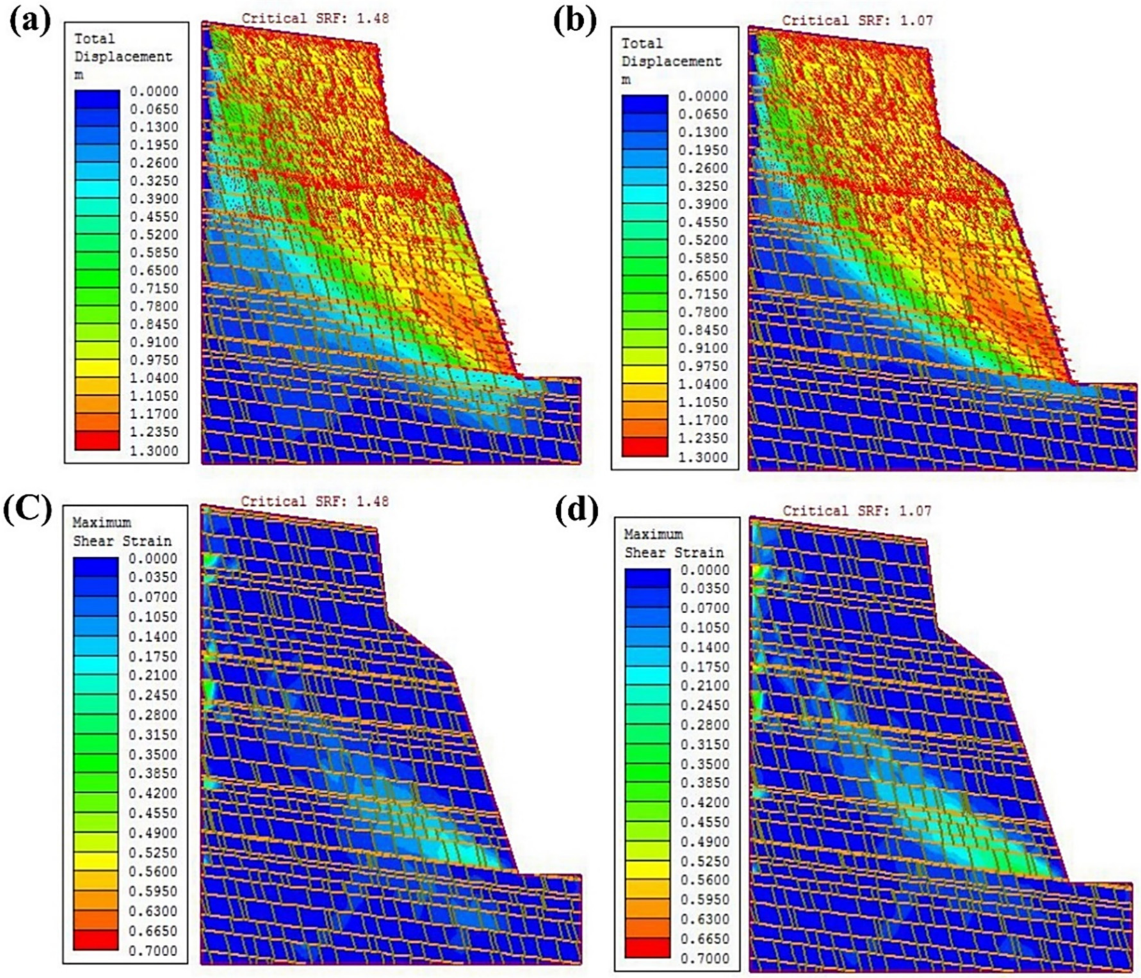

Fig. 7 FE models showing **(a)** total displacement at dry condition **(b)** total displacement at saturated condition **(c)** maximum shear strain at unsaturated condition **(d)** maximum shear strain at saturated condition.

the mode of failure based on variation in shear strain contour pattern in the jointed cut slope. Similarly, Mahanta et al. (2016) have investigated the potential failure zones along NH-305, India, by using strain contour pattern.

5. SUGGESTED PREVENTIVE MEASURES

It was found by detailed field investigation that the presence of three sets of joints with some irregular cracks and high-intensity rainfall are the main cause of the rockfall. The rainwater percolates through joints, widen fractures and reduces shear strength of joints. Thus, rock bolting along with fiber-reinforced shotcrete are suggested to avoid dislodge of rock blocks from the slope. Further, inadequately installed barriers are unable to prevent rockfall onto the expressway. Therefore, proper installation of the rockfall barriers are needed to minimise the risk of damage and reduces the kinetic energy of the falling blocks. For the suitable installation of the barrier, proper investigation of the rockfall source area, scaling, anchoring, and, if possible, cable mesh is mandatory. It is recommended to take precautionary measures against rockfall hazards at the site to safeguard the tunnel and commuters along the expressway.

6. CONCLUSION

The study investigates the stability and rockfall at hill cut highway slope near Khandala tunnel along the Mumbai-Pune expressway. The presence of three sets of joints and torrential rainfall are the major factors promoting the rockfall and instability of the slopes. This study revealed that the rockfall affects both sides of the expressway, and at least two bounces took place on the road. The maximum bounce height occurred about 9 to 10 m above the slope. As per hazardous classification, this study falls into the highest intensity zone as the total kinetic energy attained about 500 kJ, and the maximum translational velocity reached about 16 m/s. This amount of kinetic energy and velocity can cause substantial damage, which is verified by the field observations. The finite element analysis of the slope demonstrates that the slope is stable with FoS of 1.48 under the dry condition. While it shows the slope is theoretically stable with FoS of 1.07 but depending upon the complex climatic conditions and vibrations produced due to daily high-density traffic, the slope can undergo recurrent failures, as exposed by the field study. Appropriate preventive measures to minimise the frequency of slope failure and rockfall have been suggested.

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