Geoscience Frontiers 6 (2015) 837–845

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Geoscience Frontiers

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Research paper

Stability investigation of road cut slope in basaltic rockmass, Mahabaleshwar, India

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ARTICLE INFO

Article history:
Received 7 August 2013
Received in revised form
25 February 2014
Accepted 2 March 2014
Available online 17 March 2014

Keywords:
Slope stability
UDEC
Basalt
Mahabaleshwar
Road cut

ABSTRACT

Slope failures along hill cut road slopes are the major nuisance for commuters and highway planners as they put the human lives at huge risk, coupled with immense monetary losses. Analysis of these vulnerable cut slopes entails the assessment and estimation of the suitable material strength input parameters to be used in the numerical models to accomplish a holistic stability examination. For the present study a 60 m high, basaltic and lateritic road cut hill slope in Mahabaleshwar, India, has been considered. A number of samples of both basalt and laterite, in their natural state were tested in the laboratory and the evaluated maximum, minimum and mean strength parameters were employed for the three cases in a distinct element numerical model. The Mohr-Coulomb failure criterion has been incorporated in the numerical model for the material as well as the joints. The numerical investigation offered the factor of safety and insights into the probable deformational mechanism for the three cases. Beside, several critical parameters have also been judged from the study viz., mode of failure, factor of safety, shear strain rate, displacement magnitudes etc. The result of this analysis shows that the studied section is prone to recurrent failures due to the capping of a substantially thick layer of weaker lateritic material above the high strength basaltic rock mass. External triggering mechanisms like heavy precipitation and earthquake may also accelerate the slope failure in this area. The study also suggests employing instant preventive measures to avert the further risk of damage.

1. Introduction

In most of the developing nations, highways excavated along the hill slopes are the only means of conveyance. Both, major and minor slope failure activities along these hills cut slopes can be of dire consequences. The vehicular toppling as a result of the slight rock falls is quite prevalent in many parts of the world. The slope collapses, even though of limited extent can be grave; they place the human lives in jeopardy and put the local economy to a standstill. Hence, due to the immense liability associated with highway slope collapses, slope stability is a major concern for highway planners from both economic as well as risk point of view (Hoek et al., 2000). These slope failure activities chiefly depend on the slope geometry, slope material strength, prevailing geo-hydrological conditions and the discontinuity characteristics, orientation and distribution among the rock mass (Souley and Homand, 1996; Bhasin and Kaynia, 2004; Kainthola et al., 2012a,b, 2013). The hill slopes composed of weaker material, capping a jointed rock mass are more prone to failure. To mitigate a calamitous event, a comprehensive analysis is required to understand their failure mechanism. Estimation of slope deformation and instability using numerical technique entails knowledge of the slope-mass's response to stress induced changes which in turn are a function of the stresses in the rock mass, deformation characteristics, the intact material strength, mechanical behaviour of the intact rock, and any discontinuities (Fournier, 2008). Over the year various analysis tools and techniques like empirical, analytical, statistical, limit equilibrium, finite element, finite difference, distinct element methods, neural networks, GIS and fuzzy logic have been applied to the problems pertaining to landslides and slope stability (Pradhan and Saied, 2010; Pradhan, 2011; Alavi and Gandomi, 2012; Kainthola et al., 2012a, b; 2013; Ramakrishnan et al., 2013; Singh et al., 2013a,b,c;

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Peer-review under responsibility of China University of Geosciences (Beijing)

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http://dx.doi.org/10.1016/j.gsf.2014.03.002

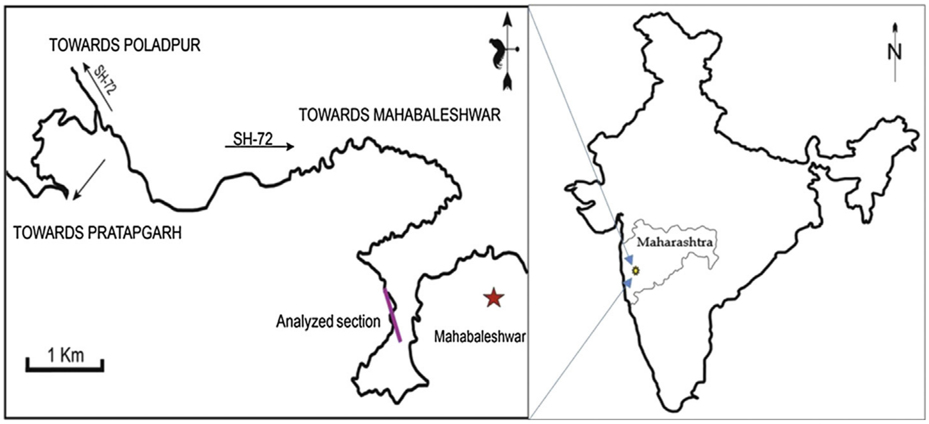

Figure 1. Location and the connectivity map of the study area.

Verma et al., 2013; Chang and Wan, 2015). The conventional continuum approaches like limit equilibrium methods, finite element methods and finite difference methods have certain disadvantages to model the behaviour of jointed rock masses, owing to the complex internal deformational mechanism (Kveldsvik et al., 2009; Lin et al., 2012). This has led to an increased usage of distinct element methods (DEM) and hybrid methods to simulate the behaviour of discontinuous rock masses (Stead et al., 2006). The Universal Distinct Element Code (UDEC), a DEM code was developed by Cundall to simulate the jointed blocky rock masses (Cundall, 1971). UDEC divides a rock mass into discrete blocks in which the block contacts represent a discontinuity. The relative displacements at block contacts are achieved by applying a force-displacement law at all contacts and Newton's force law at all the blocks (Cundall, 1980; Cundall and Hart, 1985; UDEC, 2011).

Previously, UDEC has been successfully used by various researchers to simulate a discontinuum media. DEM study has been carried out on the seismic response of a 120-m high rock slope of the Three Gorges Shiplock (Zhang et al., 1997). High natural hill slopes have also been simulated under static and dynamic conditions using UDEC, which provided valuable insights into deformational mechanism of hill slopes (Bhasin and Kaynia, 2004; Kveldsvik et al., 2009). Liu et al. (2004) used UDEC to investigate the response of blasting in jointed slope of an open cast mines. Previously, Kainthola et al. (2012b) carried out a finite element based slope stability assessment of the cut slopes of the same region. Singh et al. (2013a,b,c) estimated the velocity of block detachment in section of highly jointed basaltic cut slope, close to the present study area, for the rockfall analysis.

The present study deals with the distinct element simulation of a 51 m high blocky basalt cut slope topped with a 9 m thick lateritic layer. The cut slope is intersected with two major sets of discontinuities with a considerable variation in the dip angle. The section has seen sporadic events of failure activities. The investigation has been aimed relating the factor of safety of the slope (FOS) with the variable material strength. Twelve samples of both basalt and laterites were tested in the laboratory to assess the maximum, minimum and mean strength parameter, which have been utilized in the DEM numerical mode to gauge into the safety factor as well the deformational aspects of the slope.

2. Study area

The Mahabaleshwar area is an elevated Plateau surrounded on all sides by steep road climbs known as 'ghats'. Mesa and butte are the common geomorphic features in the study area. The present study focuses on a road cut section on state highway (SH-72), connecting Mahabaleshwar and Poladpur, Maharashtra, India (Fig. 1). The state highway is excavated on laterites and tholeiitic basalt which have been interlayered with red bole at some places.

Geologically, the formations belong to "Wai" subgroup, a part of the "Deccan Traps continental flood basalt" of Cretaceous–Tertiary age (Jay et al., 2009). The Wai subgroup consists of the lowermost Poladpur formation, followed by Ambenali formation and the upper most Mahabaleshwar formation (Beane et al., 1986). The basalt succession forming the Mahabaleshwar Plateau has been deformed into a gentle anticline–monocline plunging very gently towards the south (Mitchell and Widdowson, 1991). Typically, the basalts are composed of plagioclase phenocrysts together with clinopyroxene and olivine (Najafi et al., 1981). The region is capped with laterite layers (Fig. 2a). Bole beds are also observed in the study area as thin layers below the basaltic layers, representing the episodic flows of basaltic magma.

The laterites range from clayey to gravel size, composed mainly of montmorillonite, kaolinite and hematite. The rocks are fresh to moderately weathered, with high strength. The rock mass is intersected with two sets of discontinuities (J1 and J2) and another flow planes which gently dip inside the hill (Fig. 2b).

The prominent discontinuity sets, J1 dip towards the hill while J2 dips in the direction of the cut slope. The structural orientation of the discontinuities was assessed in the field (Table 1).

Structurally, the cut slope is undergoing a double plane sliding due to discontinuity sets J1 and J2 (Fig. 3a,b). The double plane sliding usually take place when the true dip of any of the discontinuity plane (T_{J1} and T_{J2}) lies in zone formed by the dip direction of the slope face (T_{sf}) and the intersection of the discontinuity planes (I_{J12}) (Yoon et al., 2002; Singh et al., 2013a,b,c).

The concerned cut slope section varies in height between 20 and 45 m. The analysed cut slope extends about 4 km in length. The erratic rock fall activity is quite visible along this patch particularly in the rainy season. The failed blocks range in size from 0.3 to 1 m^3 (Fig. 3a).

3. Model parameters

For proper and reliable estimation of input parameters for the numerical simulation, an extensive field work and laboratory experiments were carried out. The rockmass has been replicated as Mohr-Coulomb material in the numerical model, which follows a linear failure criterion. A number of samples of both basalt and laterite were tested in the laboratory to determine the bulk density, uniaxial compressive strength, Young's modulus, Poisson's

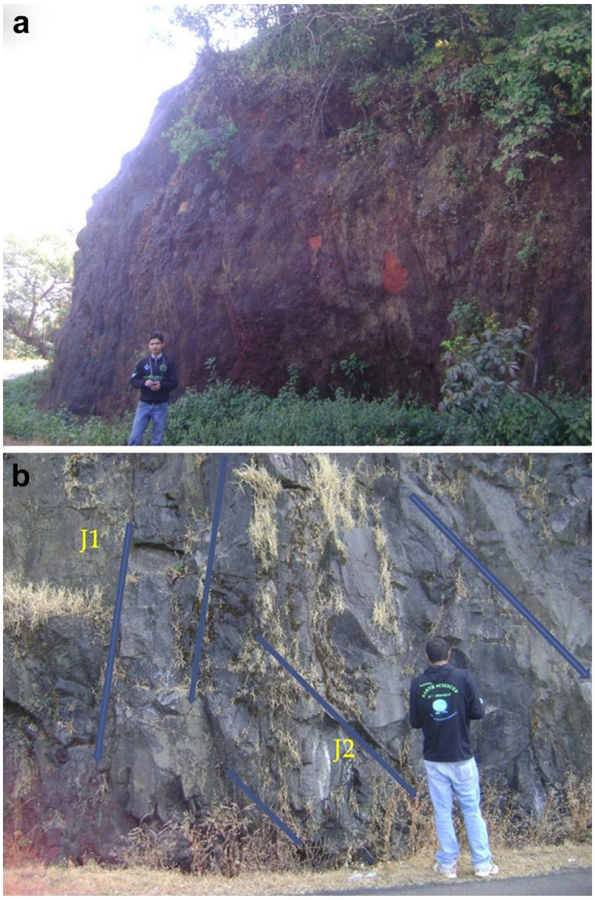

Figure 2. (a) Lateritic soil capping at the top of the Mahabaleshwar plateau, (b) discontinuity orientation in the area.

ratio, cohesion and friction angle (Table 2). Each sample was tested in compliance with the international testing standards (ISRM, 1981a, b).

Rock discontinuities are primarily responsible for weakness, deformability and movement in the rock masses (Barton, 1972). The amount of deformation is controlled by the joint stiffness characteristics. Hence, to gauge the mechanical response of stress redistribution on cut slopes, the assessment of normal joint stiffness (k_n) and shear joint stiffness (k_s) are vital. The discontinuity strength parameters, k_n and k_s, have been calculated using the formulation adopted by Liu et al. (2003).

4. Numerical simulation

Numerical models are computer programs that simulate the mechanical response of a rock mass subjected to a set of initial conditions such as in situ stresses, water levels and boundary conditions (Wyllie and Mah, 2004). For the present analysis, a universal distinct element code (UDEC) has been used to model the behaviour of the discontinuous road cut slope section. UDEC follows an explicit, time-marching procedure that models assemblies of distinct blocks which interact mechanically at their surfaces and corners (Lorig and Hobbs, 1990). The relative motion along the discontinuities is ruled by linear or nonlinear force-displacement relations for movement in both the normal and shear directions (UDEC 5.0, 2011); block interaction at contact points is controlled by strength parameters (Bhasin and Kaynia, 2004). UDEC makes it possible to calculate both stress and deformation in a discontinuous rock slopes and can further predict the slope behavior (Moares, 2011).

The present analysis has been carried out for a 60 m high steep jointed basalt slope (Fig. 4). The generated UDEC model has a 51 m high basalt rock mass, underlying the laterites of 9 m thickness. A ‘continuous’ joint system for “J1” and a ‘discontinuous’ joint system for “J2”, has been employed for the basaltic rock mass in the model. Joint set “J1” slants at 22°, while “J2” is inclined at 60°, in accordance with the relation between the direction of the analysed section and the apparent dip.

Table 1
Discontinuity orientation at the study area.

Discontinuity set	True dip amount (°)	Dip direction (°)	Spacing (cm)	Persistence (m)	Opening (mm)
J1	50 ± 5	330 ± 10	20–300	>2	~3
J2	80 ± 5	210 ± 10	20–400	>4	~4
Flows	12 ± 3	080 ± 5	5–100	>7	Tight

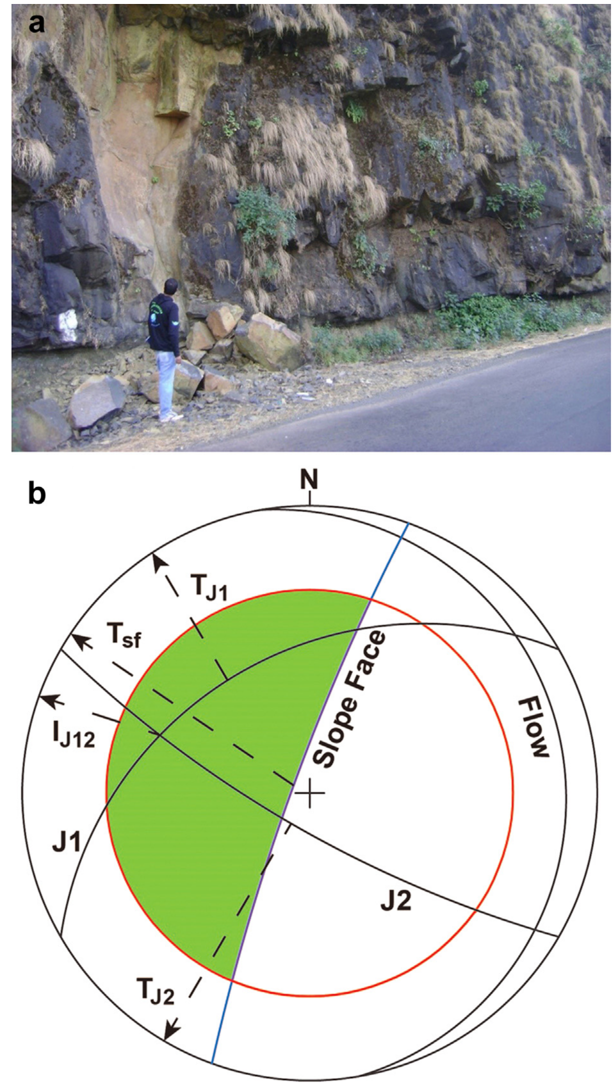

Figure 3. (a) Rock fall activity along the cut slope, (b) kinematic analysis of the jointed rock mass along the cut slope.

Table 2
Geo-mechanical properties of the slope material.

		Bulk density (g/cm^3)	Cohesion (MPa)	Uniaxial compressive strength (MPa)	Angle of internal friction (°)	Young's modulus (GPa)	Poisson's ratio	Normal joint stiffness (GPa/m)	Shear joint stiffness (GPa/m)
Maximum	Basalt	2.69	7.8	61	38	37.5	0.22	33.82	15.36
	Laterite	2.15	0.150	4.5	23.7	5	0.30	–	–
Mean	Basalt	2.65	4.5	47	31.4	28.6	0.26	29.7	12.27
	Laterite	2.03	0.12	3.2	21.6	4.35	0.315	–	–
Minimum	Basalt	2.61	1.2	33	24.8	23.7	0.29	25.6	9.18
	Laterite	1.9	0.091	1.9	19.6	3.7	0.33	–	–

The slope has been simulated with exaggerated joint spacing to speed up the computation process. A total of thirteen monitoring points were established in the model: "A" to "M". Monitoring locations "A" to "I" were created along the block contacts on the basaltic rock mass to assess maximum shear displacement and maximum shear stress generated along these points, while monitoring points "J" to "M", where created to calculate xy-stress on the overlying lateritic soil. Three separate conditions, using the calculated material properties i.e., maximum, minimum and mean, have been simulated using the UDEC (Table 2). The left and bottom boundary of the model has been kept fixed while the top and right boundary are free. The factor of safety (FOS), maximum shear displacement, maximum displacement and velocity vectors have been evaluated to gauge into the insights of the failure mechanism.

4.1. Case I

In case I, maximum strength attained from the laboratory experiments were used as the input strength parameters in the numerical model (Fig. 5a,b).

The model ran for 17,120 cycles to attain the force displacement equilibrium. An FOS of 1.31 was calculated for the hill slope using the highest material strength (Table 2). Maximum strain generation was observed on the upper cut slope near the monitoring location "A". The maximum shear displacement along the joints was calculated to be 612E-04 m, at the toe of the lower cut slope near monitoring point "G". The strain generation is lesser in magnitude due to the relative higher strength of the material in this case. The magnitude of shear displacement and strain rate is very low, also indicated by stable FOS, which shows that the rock mass with this strength will remain stable for a considerable period of time unless acted upon by other triggering forces like heavy precipitation and earthquakes.

In case II, the concerned hill slope was analysed using the minimum material strength attributes accomplished from geo-mechanical laboratory tests (Fig. 6a and b).

The UDEC model gave an FOS of 0.97 for the hill slope, much below the critical safety value, when the least strength parameters were used in the simulation. A maximum displacement vector of 1.772 cm is developed near the monitoring location "B", which is about 12 m above the upper road cut. The failure mechanism based

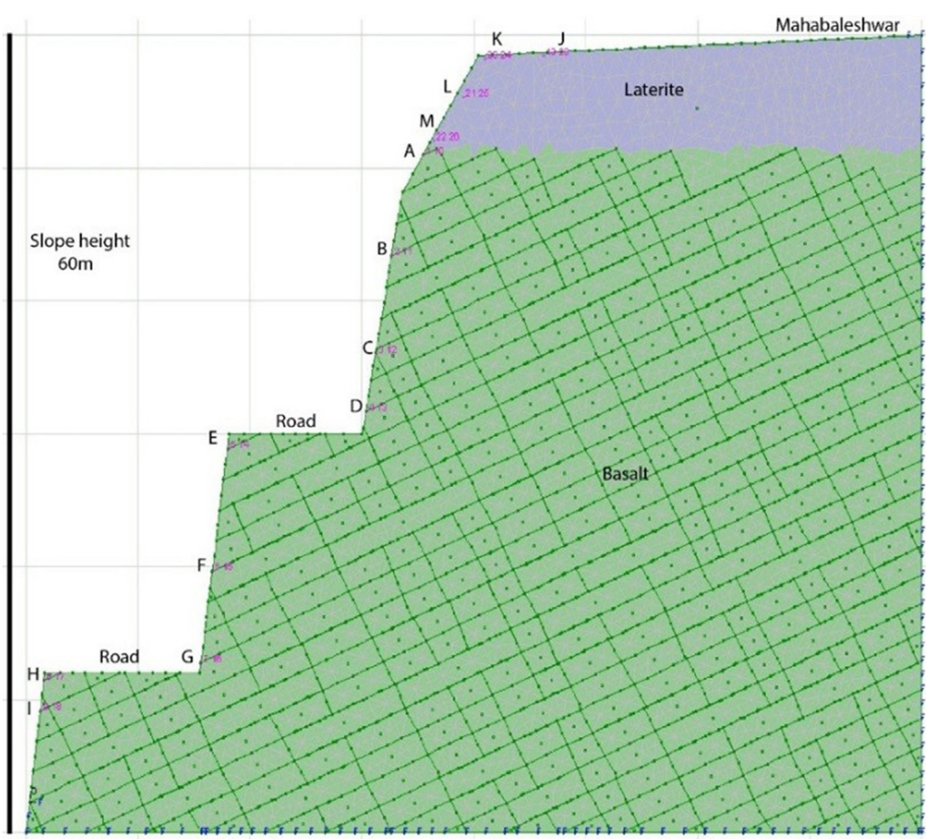

Figure 4. Modelled geometry of the hill cut slope with monitoring points, "A" to "M".

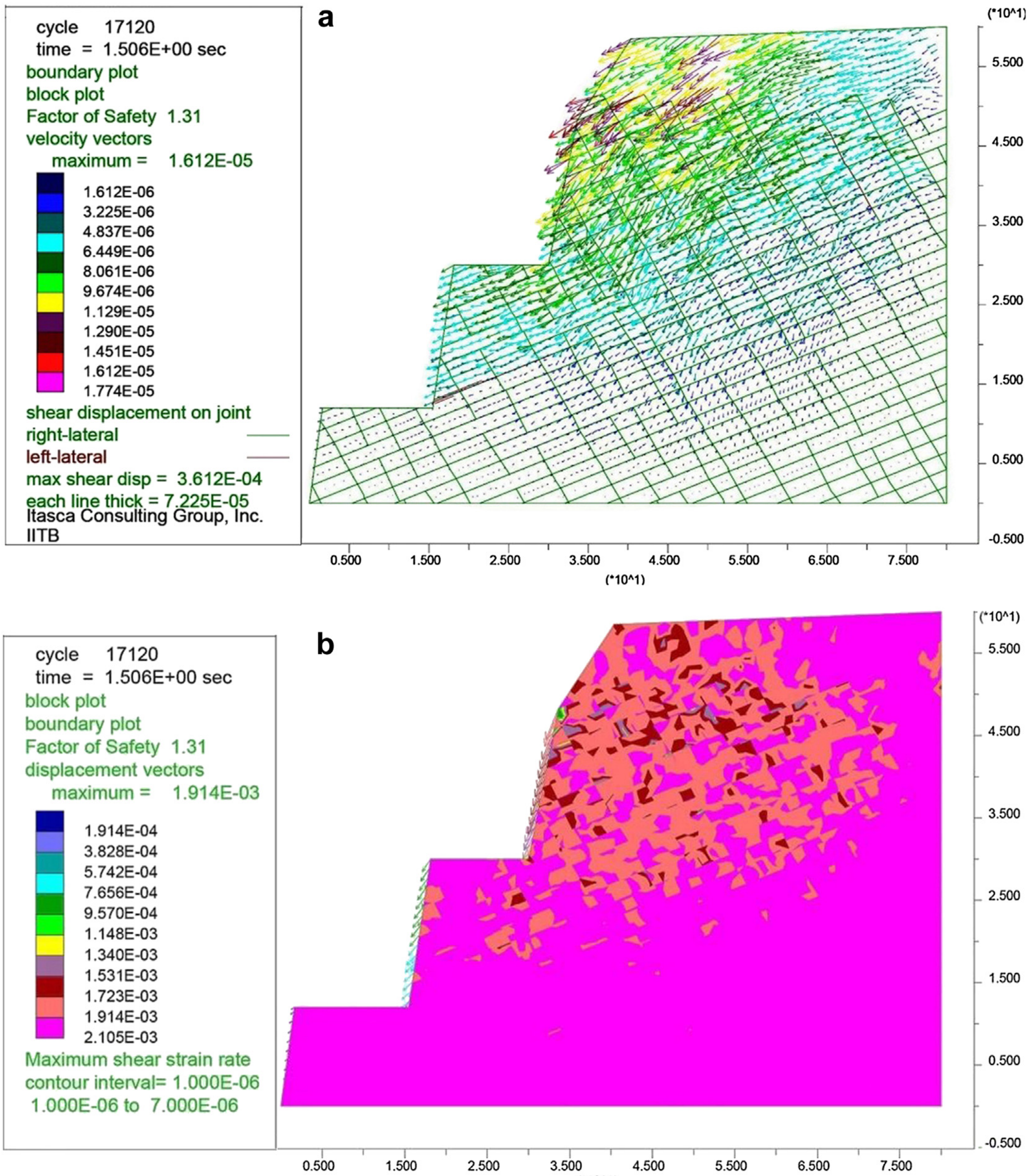

Figure 5. UDEC results for the case I, showing (a) velocity vectors and (b) displacement vectors along with shear strain rate contours.

on the strain concentration presented in the numerical model is in conformity with the field observations (Figs. 2 and 6b).

There has been a considerable decrease in shear strain rate and shear displacement when compared to the case I. In case II, the FOS lessens to 0.97, which indicates that the, highly degraded capping above the higher strength rock mass is prone to failure and may require instant strengthening.

4.2. Case III

The case III, utilized the mean strength parameters from the laboratory tests (Fig. 7a,b). The FOS achieved was 1.14, implying a theoretical stability for the hill slope but a highly vulnerable state.

A maximum displacement vector of 0.025 cm was yielded at the monitoring location "M". In the basaltic mass, a maximum shear displacement of 6.65E-04 m was obtained at the upper reaches along the joint planes. Due to the high strain concentration, there is a high probability of planar failure along the discontinuity planes near the monitoring location "M", which will invariably pave way for the collapse of the lateritic mass lying above it (Fig. 7b).

5. Results and discussion

The three cases were analysed using a DEM approach based on the variance of the material strength parameters for the same slope (Table 3). The maximum FOS computed for the slope was 1.31, rendering it stable under the normal conditions for the materials maximum strength. The maximum right lateral shear stress of 0.3 and 0.4 MPa was generated along the jointed contact near the monitoring point "D" and "G", respectively (Fig. 8a). The maximum right lateral shear stress of 0.2 MPa was generated along the monitoring location "C".

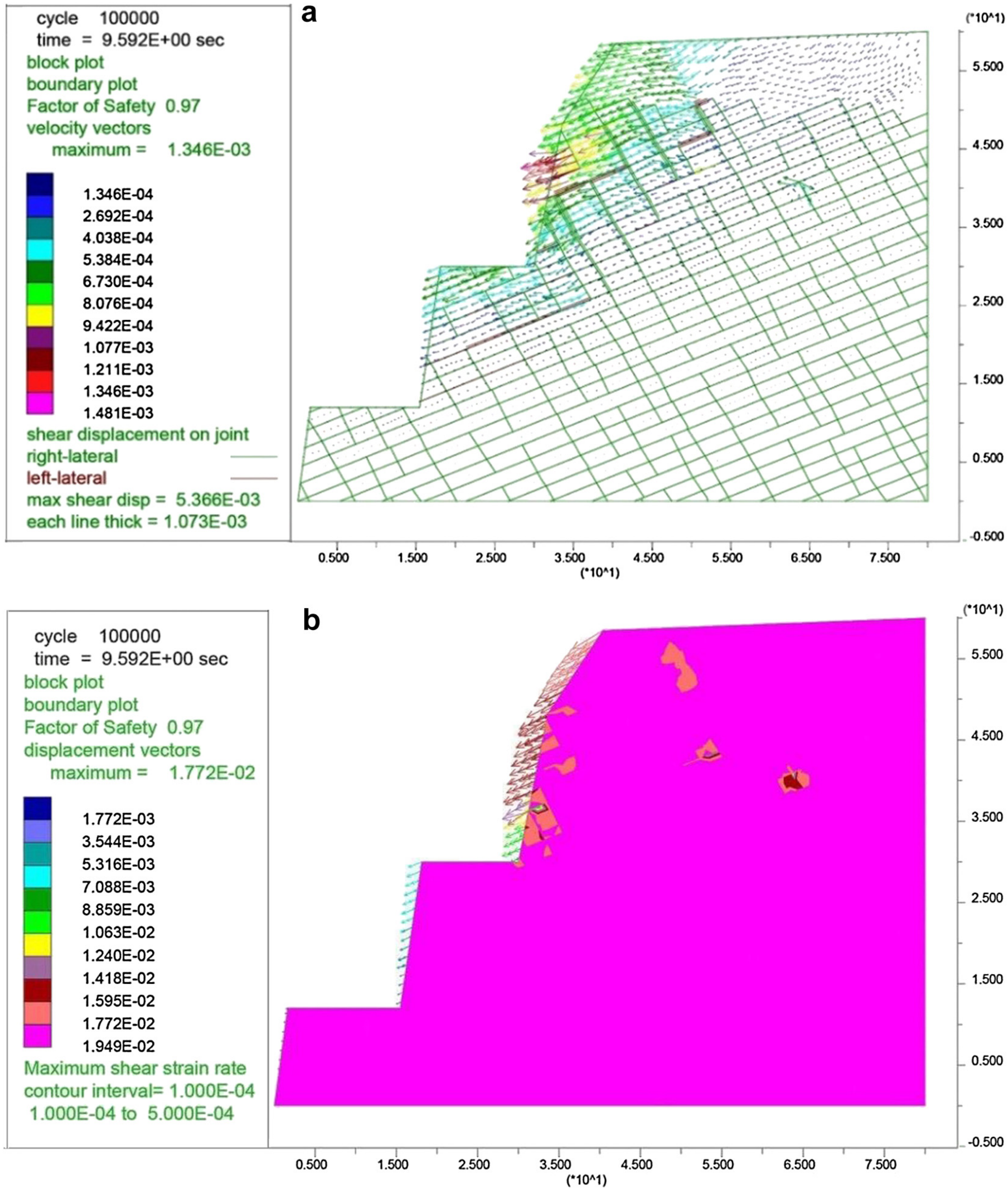

Figure 6. UDEC results for the case II, depicting (a) velocity vectors and (b) displacement vectors along with shear strain rate contours.

The maximum shear stress peaked at the location "M" for the laterites, which gradually decreases to 0.01 MPa, maximum displacement being 1.914E-03 m. The numerical model gave an FOS of 0.97 when the minimal strength properties for the concerned cut slope were used. The maximum shear stress of 0.2 MPa and displacement of 1.772E-02 m was generated along the jointed contact at location "G" (Fig. 8b). The shear strain concentration displays a failure along the upper basalt cut slope along "C" and "D". The numerical results are in accord with the kinematic analysis and the field observations (Fig. 3a,b). The model also depicts a high shear stress concentration along the location "M", at the laterites.

For the third case, the mean material strength of the slope was used in the DEM simulation which yielded an FOS of 1.41. The slope, though stable at this FOS, can become quite susceptible to failure against heavy downpour and other activities which tend to weaken the slope mass. A maximum shear stress of 0.25–0.3 MPa was generated at monitoring points "D" and "G", while a maximum right lateral shear stress of 0.17 MPa at "C" (Fig. 8c).

The maximum stress along the upper lateritized top has been 0.024 MPa, relating to the deformation along the lower reaches of the laterites. In all the three cases, the maximum shear stress has been generated along the toe of the basalt road cuts and near the boundary between basalt and laterite. Based on the numerical study and the strain generation the most probable failure mechanism can be planar rock falls along the jointed basalt and the circular failure of the lateritic soils on the top.

6. Conclusion

The slope material in the studied section comprises of lateritic capping lying above the jointed basaltic rock mass of appreciable

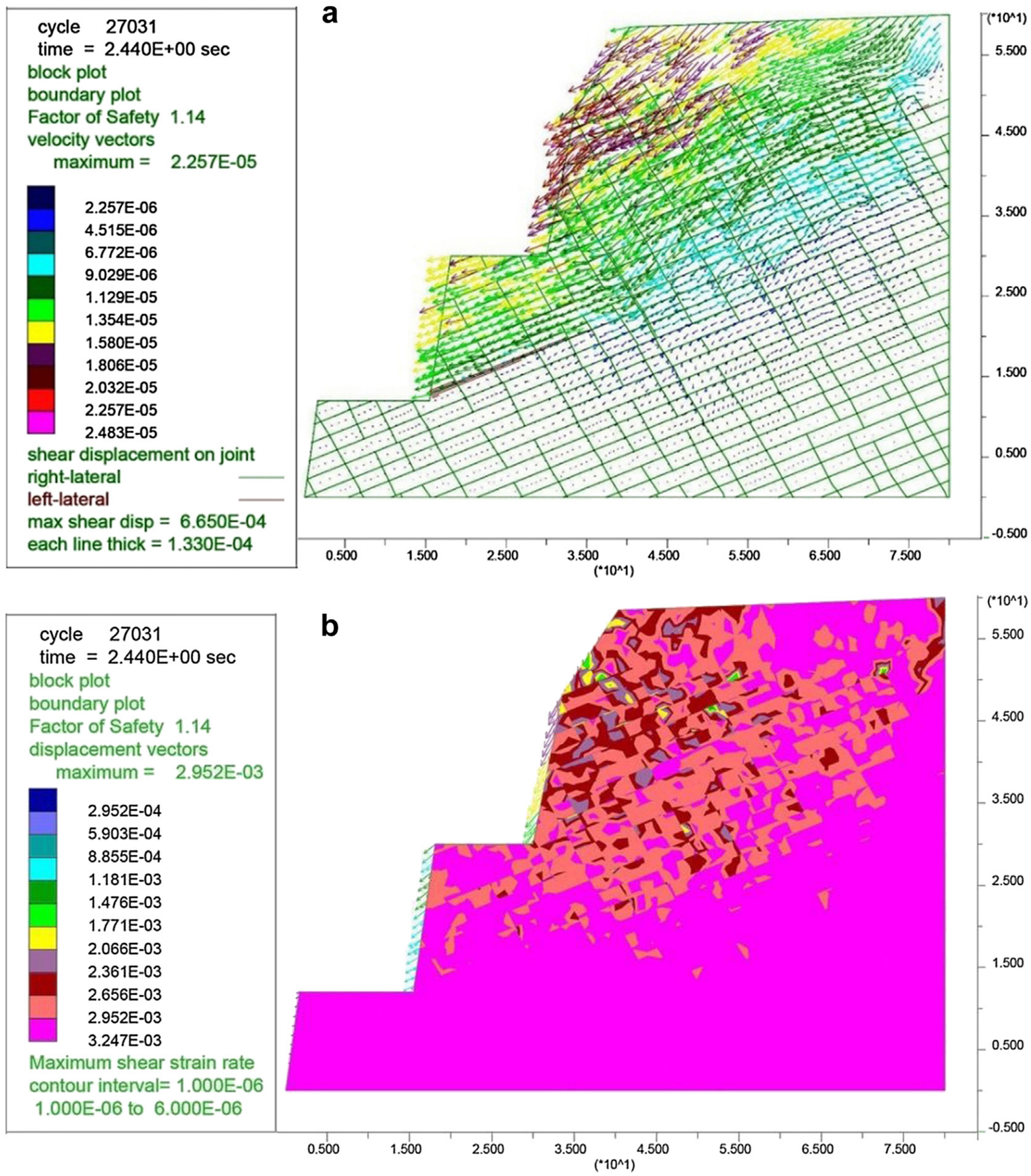

Figure 7. UDEC results for the case III, portraying (a) velocity vectors and (b) displacement vectors together with shear strain contours.

strength. Post monsoon, the semi-consolidated lateritic mass in this region is highly vulnerable to recurrent slope failures. Hence, due to extreme climatic condition, a single material strength value may not represent the variance in the strength attributes. Therefore, for a complete stability investigation a 60 m high composite slope was analysed using its maximum, minimum and mean strength properties. The FOS (1.31) calculated for the hill cut slope using the maximum slope mass strength render it to be stable in the current natural state, while for the minimum strength, it fails at an FOS of 0.97. The displacement vector of 1.914E-03 m at the base of the lateritic layers for the case I (FOS = 1.31) doesn't rule out the occurrence of failure and poses an imminent probability of a failure, in case of a heavy rain. For the mean values used, the slope is in a critical state (FOS = 1.14). This represents a highly susceptible state of stability for the hill slope. Though, the FOS portrays the hill slope to be relatively stable and the maximum displacement vectors (2.592E-03 m) are comparatively less; but the possibility of failure cannot be ruled out due to the fragile variation in material strength as a result of the action of external factors, mainly water, which leads to considerable reductions in shear strength. Hence, as the area is anticipated to receive heavy rainfall every year during the monsoons, it is recommended to apply appropriate remedial measures, viz., and erection of concrete retaining walls along with drainage holes, to prevent further instances of any slope failure.

Table 3
DEM analysis results for the hill slope.

	Factor of safety	Maximum displacement vectors (m)	Maximum velocity vector (m/s)
Case I	1.31	1.914E-03	1.612E-05
Case II	0.97	1.772E-02	1.346E-03
Case III	1.14	2.592E-03	2.257E-05

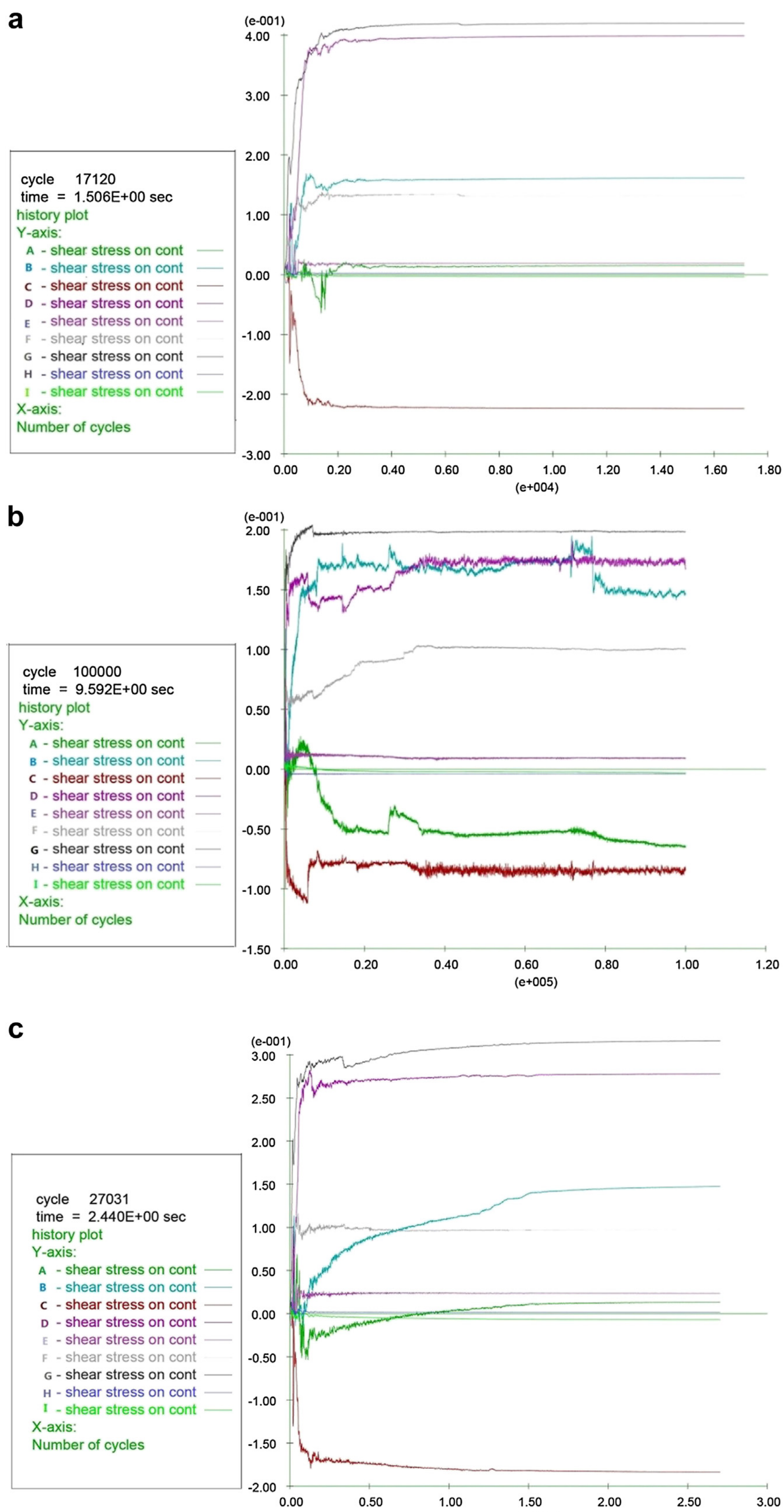

Figure 8. Shear stress plots for the monitoring locations along the slope face for (a) Case I, (b) Case II and (c) Case III.

Acknowledgement

The authors would like to thank, CSIR[§], DST[ß] and UGC[Ƨ], India for the grant to carry out the research work.

[§]Ashutosh Kainthola; [ß]T N Singh; [Ƨ]P K Singh.

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