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Finite element analysis of failed slope by shear strength reduction technique: a case study for Surabhi Resort Landslide, Mussoorie township, Garhwal Himalaya

Vikram Gupta[a], Rajinder K. Bhasin[b], Amir M. Kaynia[b], Vipin Kumar Chauhan[a], A.S. Saini[c], R.S. Tandon[d] and Thomas Pabst[b]

[a]Wadia Institute of Himalayan Geology, Dehra Dun, India; [b]Norwegian Geotechnical Institute, Oslo, Norway; [c]Birla Institute of Technology and Science, Pilani, India; [d]National Geotechnical Facility, Dehra Dun, India

ABSTRACT
Finite element analysis of failed slope of the Surabhi Resort landslide located in the Mussoorie township, Garhwal Himalaya has been carried out using shear strength reduction technique. Two slope models viz. debris and rock mass were taken into consideration in this study and have been analysed for possible failure of slope in future. Critical strength reduction factor (SRF) for the failed slope is observed to be 0.28 and 0.83 for the debris and rock mass model, respectively. A low SRF value of the slope revealed significant progressive displacement in the zone of detachment. This has also been evidenced in the form of cracks in the building of Surabhi Resort and presence of subsidence zones in the Mussoorie International School. These results are consistent with the study carried out by other workers using different approach.

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Finite element method; strength reduction factor; Surabhi Resort landslide; Mussoorie; Garhwal; Himalaya

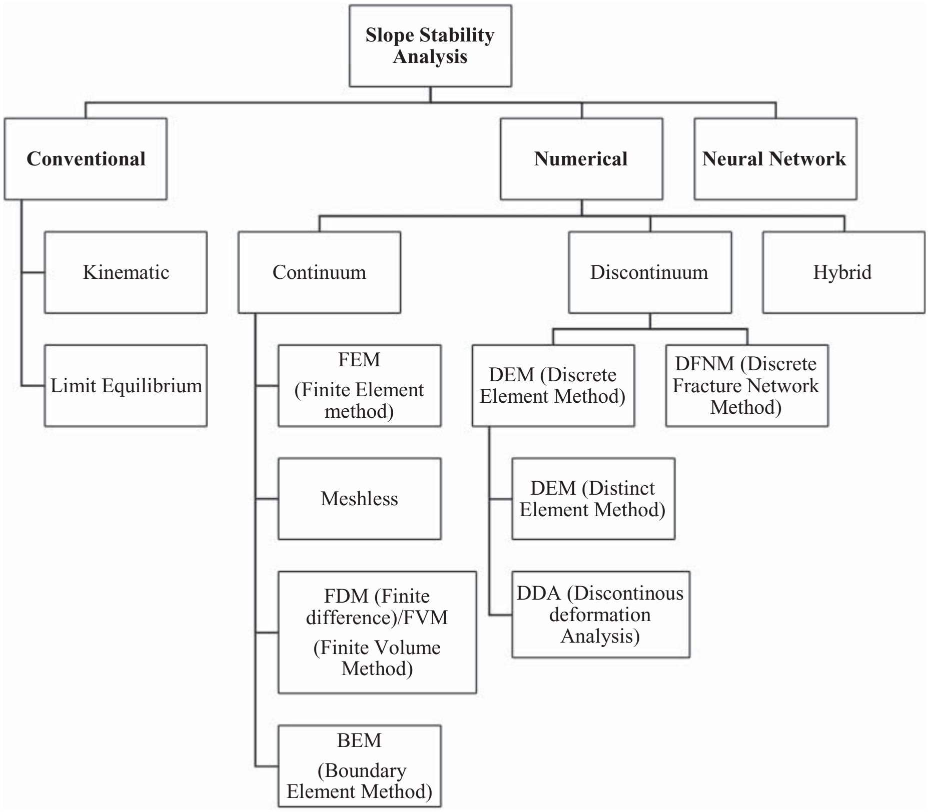
1. Introduction

Landslides and related mass movement activities are common in the Himalayan terrain, particularly during and immediately after a rainy season or during the earthquake (Paul et al. 2000; Gupta & Bist 2004; Gupta et al. 2013, Gupta et al. 2015). These landslides continue to pose a serious threat to the lives and properties in the region until the failed slopes attain a particular factor of safety. Recently, it has been noted that the frequency of the landslides in the Himalayan terrain has increased manyfold (Gupta & Sah 2008). Though there are many factors for the occurrences of landslides, mainly shift in the rainfall pattern, more extreme climatic events and increase in anthropogenic activities are some of the important factors for this increase in the frequency.

1998 Surabhi Resort landslide in the Mussoorie township (Gupta & Ahmad 2007), 1998 Okhimath landslide in the Mandakini valley (Sah & Bist 1999), 2001 Budha Kedar landslide in the Balganga valley (Sah et al. 2003) and 2003 Varunavat Parvat landslide in the Bhagirathi valley (Gupta & Bist 2004; Sarkar & Kanungo 2004) are some of the examples of disastrous rain induced landslides that struck the Garhwal Himalaya causing huge loss of lives and property in the region. This calls for detailed analysis of slope stability that can broadly be categorized into pre- and post-disaster analysis. Pre-disaster studies focus on identification of areas susceptible to landslide hazards and subsequent hazard zonation mapping whereas, post-disaster studies emphasize more on the

CONTACT Vikram Gupta vgupta@wihg.res.in

Table 1. Flow chart indicating available methods for slope stability analysis.



determination of causative factors for the occurrence of landslide, and the quantification of slope stability of the failed slopes in terms of factor of safety.

There are many methods for the stability analysis of slope. This include conventional methods and numerical methods. Conventional methods include kinematic and limit equilibrium analysis. Numerical methods are generally preferred over the conventional methods and these include continuum and discontinuum modelling. Analysis like finite element, finite difference and boundary element falls within the continuum modelling and discrete element and discrete fracture network falls within the discontinuum modelling. These methods of slope stability have been widely used world over (Aryan 2006; Ozbay & Cabalar 2015). All these methods are reviewed by Jing (2003) and the results are presented in table 1. This study introduces the slope stability of the failed slope of the Surabhi Resort landslide that occurred during 1998 in the Mussoorie township using the finite element analysis.

2. Background history of the Surabhi Resort landslide

The Surabhi Resort landslide has been studied and mapped in detail by Gupta and Ahmad (2007). The crown of the landslide (30°28′56.6″N and 78°03′02.5″E) is located at an elevation of about 1650 m, between km stone 6 and 7 on the Mussoorie–Kempty road in the Tehri district of Uttarakhand (figure 1). Surabhi Resort and Mussoorie International School (MIS) are located in the crown portion of the landslide. The landslide occurred because of the excessive rainfall during August 1998 and is a typical example of debris slide in its upper portion that transitions into debris flow towards lower slope. The scarp of the landslide is of arcuate shape with a width of about 100 m at the top.

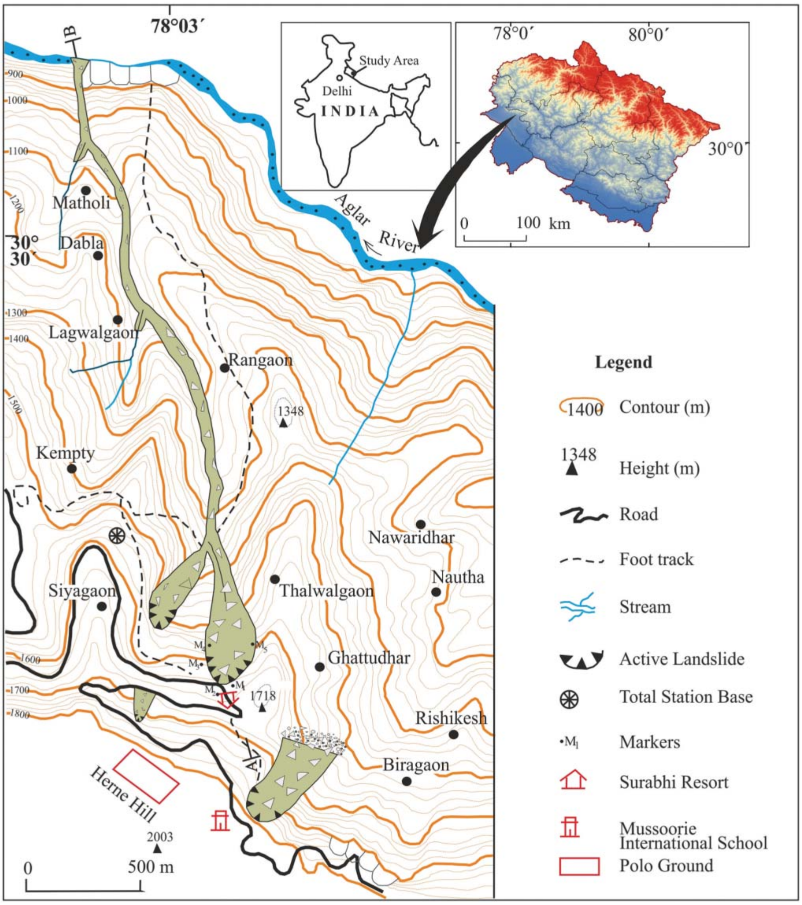


Figure 1. Location map of the study area (after Gupta & Ahmed 2007).

The width progressively increases to about 300 m towards central body of the landslide (figure 2). The dip of the scarp is about 60–70° due N10°E. The slide can be divided into three morphodynamic zones (figure 3). These are

(1) Zone of detachment (1650–1325 m above msl)
(2) Zone of transportation (1325 m – 1100 m above msl)
(3) Zone of accumulation (1100 m – 800 m above msl)

The landslide involved the displacement of quaternary material to a depth of about 30 m. No rocky outcrop in the main body of the landslide and along the slide track is visible, however, the scattered outcrops on the road section in the crown portion are highly crushed, weathered and pulverized. Geologically, the study area and its environs fall on the northern limb of the Mussoorie Syncline and are made up of meta-sediments belonging to Krol and Blaini Formations of the lesser Himalaya. The rocktype present is mainly dolomitic limestone together with subordinate variegated shales belonging to Krols. These rocks are highly folded, faulted, jointed and fractured. Four set of joints trending NE-SW, NNE-SSW, NNW-SSE and ESE-WSW render the breakage of these rocks into small pieces of about 3–4 cm length. The slide starting from the Surabhi Resort follows the first order drainage and passing through the agricultural fields of Thalwalgaon, Rangaon, Lagwalgaon, Dabla and Matholi villages joins the Aglar River (figure 1). In its total track length of 3.5 km, this landslide has affected 38 families and about 5 hectares of agricultural land along its track. It has also washed away eight watermills located along the channel (Uniyal & Rautela 2005; Gupta & Ahmad, 2007).

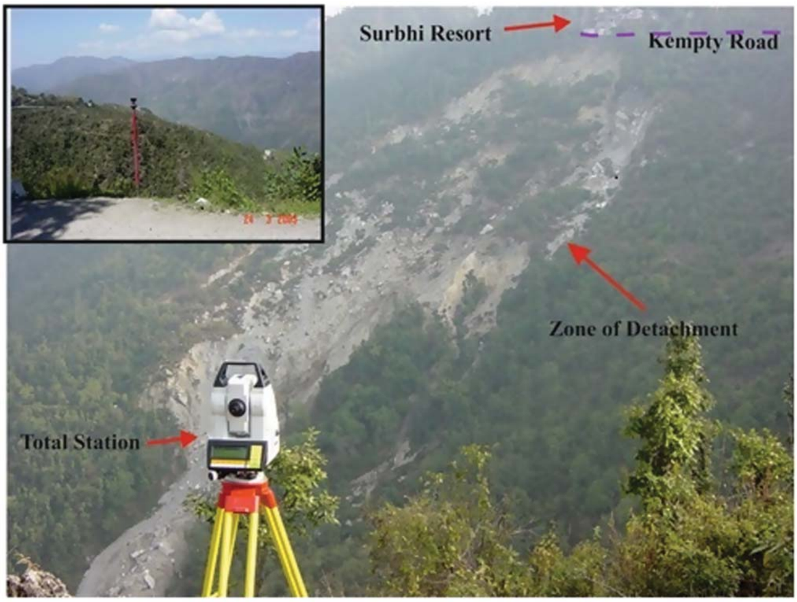


Figure 2. Panoramic view of the Surabhi Resort landslide.

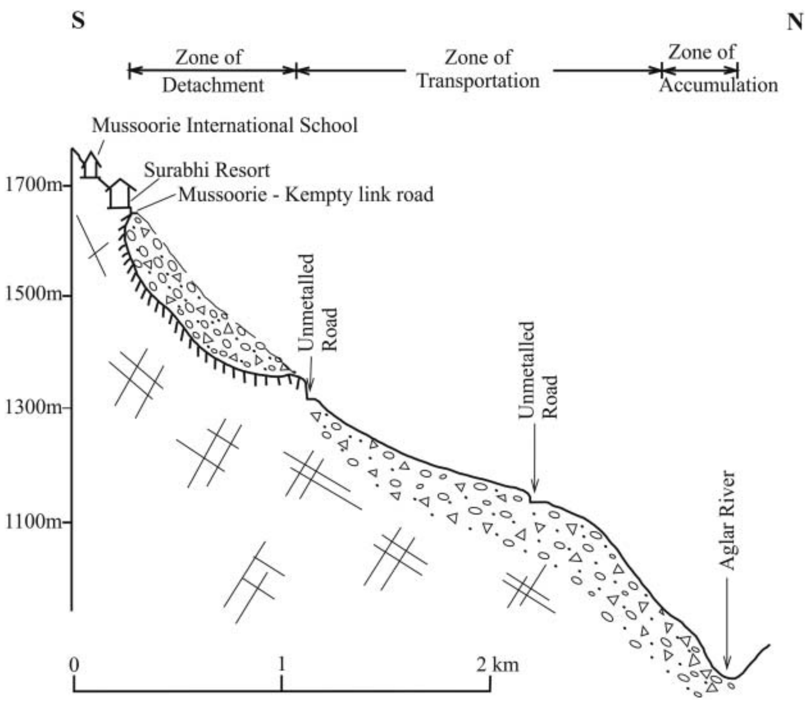


Figure 3. Cross-section of Surabhi Resort landslide depicting (1) Zone of Detachment (2) Zone of Transportation and (3) Zone of Accumulation (after Gupta & Ahmed 2007).

3. Methodology: material, model design and approach

3.1. Material properties

Surabhi Resort landslide is characterized by highly weathered and jointed dolomitic limestone, overlain by thick quaternary cover, therefore, geotechnical properties of both the material are taken into consideration. The material property of the debris as determined by Gupta and Ahmed (2007) has been used for the soil model, whereas for rock model, the strength and elastic properties of dolomitic limestone have been taken from Gupta and Ahmed (2007) and geological strength index (Hoek 1994). In our slope model, geological strength index (GSI) value has been taken as 20, considering the highly sheared, fractured and jointed rockmass (figure 8(*c*)). The coefficient of earth pressure at rest (*K*) has been taken as unity considering the complexity involved in determination of *in situ* stress as suggested by Talobre (1967). The material properties of soil and rocks used in this study are presented in Table 2.

3.2. Model design

Two model designs viz. debris model and rock model have been used for this study. Debris model involves the interface between debris cover and the underlain rock (figure 4(*a*)). In order to simulate shear and normal displacements of the debris-rock interface, an interface boundary is incorporated between overlying debris and underlying rock mass. This boundary provides relaxed connectivity to the overburden and underlying rock for relative displacement by facilitating separate joint elements across boundary (Pande et al. 1990). In this model, the behaviour of debris under load is considered as ideally elastic-plastic, while for rock mass, it is elastic. A separate rock model has been used to assess the response of hard rock under loading condition without the quaternary cover (figure 4(*b*)). A piezometric line has also been plotted ranging between 6 and 8 m below the slope surface.

Table 2. Material properties used in the model.

Material		Material model	Material value
Dolomitic limestone		Generalized Hoek & Brown (Hoek et al. 2002) $\sigma_1 = \sigma_3 + \sigma_{ci}\left[m_b\left(\frac{\sigma_3}{\sigma_{ci}}\right) + s\right]^a$ $m_b = m_i e^{[(\mathrm{GSI}-100)/(28-14D]}$ $s = e^{[(\mathrm{GSI}-100)/(9-3D]}$ $a = \frac{1}{2} + \frac{1}{6}\left[e^{\left[-\left(\frac{\mathrm{GSI}}{15}\right)\right]} - e^{\left[-\left(\frac{20}{3}\right)\right]}\right]$ D = Disturbance factor m_i = Intact rock property	• Young's modulus (MPa) = 20,000 • Poisson ratio = 0.25 • Intract rock UCS (σ_{ci}) =25MPa • Geological strength index (GSI) = 20 • Material constant (m_i) = 8 • m_b parameter = 0.459 • s parameter = 0.0001 • a parameter = 0.544 • D = 0 (for Natural slopes, Hoek et al. 2002)
Interface (Joint boundary)		Mohr Coulomb slip criteria	• Cohesion (c) = 0 kN/m^2 • Friction angle = 33° • Normal stiffness (GPa) = 133 • Shear stiffness(GPa) = 13.3 (Pal et al. 2012)
Overburden debris	Crown	Mohr-Coulomb $\tau = c + \sigma \tan\emptyset$	• Cohesion (c) = 63.2 kN/m^2 • Ø (°) = 21.9 • Young's modulus (Mpa) = 170 • Poisson ratio = 0.28
	Toe	Mohr-Coulomb $\tau = c + \sigma \tan\emptyset$	• Cohesion (c) = 65.4 kN/m^2 • Ø (°) = 38.4 • Young's Modulus (Mpa) = 40 • Poisson *ratio* = 0.28

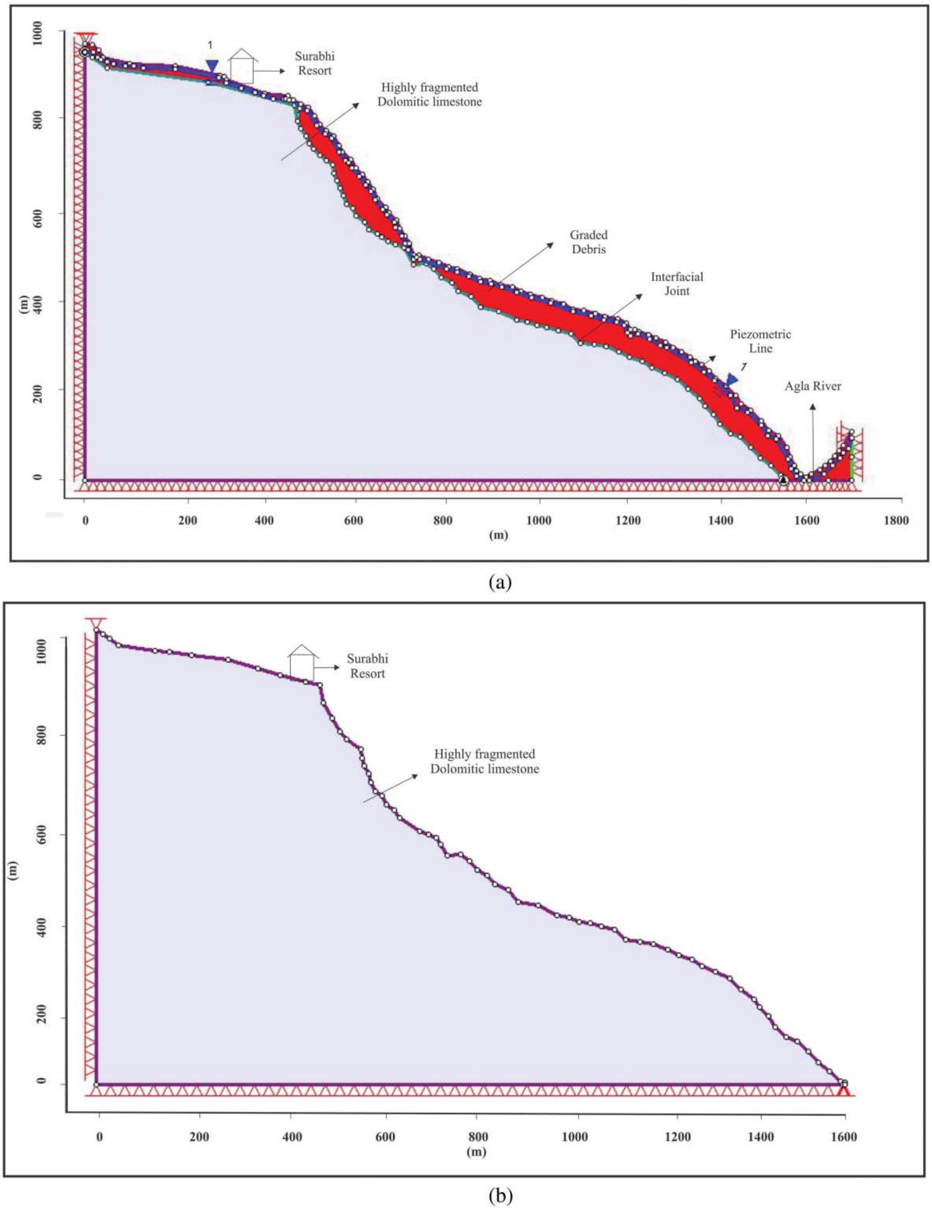


Figure 4. (a) Numerical model for overburden and (b) for the underlying dolomitic limestone rock mass.

3.3. Approach

Finite element method (FEM) is one of the numerical methods which has been widely used for slope stability analysis (Pouya & Ghoreychi 2001; Sitharam 2009). It lacks the need of presumption about slip surface and its output also depends upon global response (integrated response of constitutive elements). In continuum modelling (like FEM), a slope is considered as a single entity unlike pile of separate blocks in discontinuum modelling. To incorporate discontinuity in the model, joint element having relaxed connectivity is used. FEM has been used widely for such kind of problems

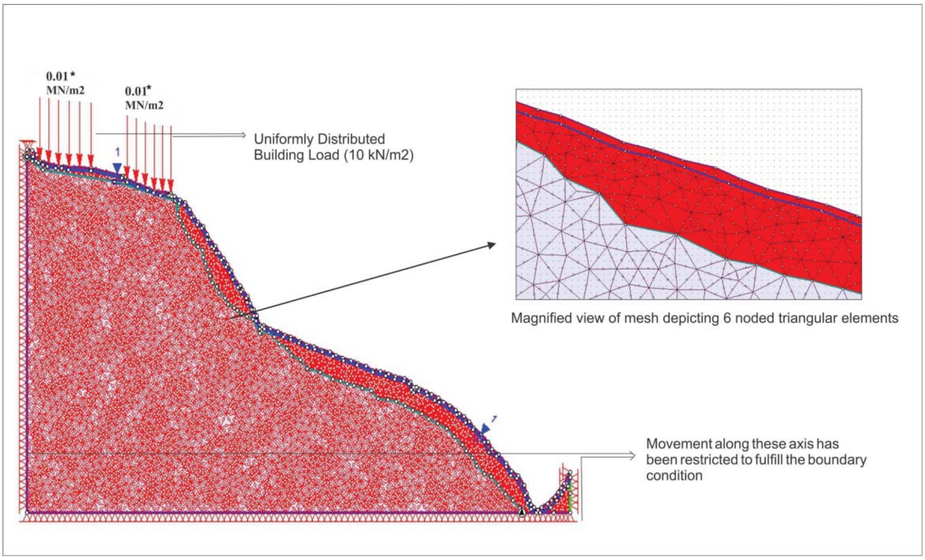


Figure 5. Finite element model of the slope after discretization and mesh generation.

(Crosta et al. 2003; Kanungo et al. 2013; Troncone et al. 2014). It involves the interrelationship of forces, stress, strain and displacements. In general, stress method and displacement method have widely been used to solve simulated mathematical equations. In this study, displacement method which establishes the equation of equilibrium has been used to determine total displacement on the slope.

Since the FEM works on the principle of discretization of fixed number of elements, triangular plain strain elements having six nodes were used. A total of 5345 elements with 11,010 nodes were created to mesh the entire model so that the simulated distance between consecutive nodes is about 3 m. The contiguity of mesh was checked to ensure that the mesh consists of a single continuous region. Figure 5 shows the used numerical model after discretization. Only 1.6% (88 out of 5345) elements in debris model and 0.64% (9 out of 1405) elements in rock model pretended problem in quality. After several trial & error, following values were found reasonable to define the quality of elements in our mesh;

Poor quality elements are those with:

(1) (Maximum side length) / (minimum side length) >10.00
(2) Minimum interior angle <20.0 degrees
(3) Maximum interior angle >120.0 degrees.

Fixed boundary conditions (zero displacement) has been used along the lateral sides and at the base of the model, however, the slope face and the rock-debris interface were kept free for displaying strain and displacement. Both internal and external loads have been applied on the model. Internal loads include field stress and body forces, whereas load due to Surabhi Resort and MIS building located in the crown portion of landslide is considered as external load. The ratio of horizontal and vertical stress has been taken as unity. External load of the buildings (MIS and Surabhi Resort) has been kept constant with a value of 10 kN/m^2 as per the British standards (BS 1996).

After selecting appropriate model and the material properties, shear strength reduction (SSR) technique, as proposed by Matsui & San (1988) has been used to determine strength reduction factor (SRF). In this technique, c and φ values of the slope materials are reduced till the failure occurs and it is assumed that the failure mechanism of slope is directly related to the development of shear strain (Roscoe 1970). The iterative non-convergence failure criteria is used for SSR approach (Nian et al. 2011). Finally, the modelling has been carried out using Phase^2 (version 6.024) software (Rockscience Inc 2008).

4. Assumptions

In this study, elastic-perfectly plastic material model also known as the 'ideally' elastic-plastic material model is used for the slope stability analysis (Griffiths & Lane 1999; Nian et al. 2011). In this model, elastic modulus and poisson's ratio govern material behaviour in the elastic phase and the applied strength criterion governs material behaviour in the plastic phase. For our analysis, we have used Mohr−Coulomb strength criterion for the overburden soil and the generalized Hoek−Brown criterion for the rock mass.

As the strength of rock mass is scale dependent and the ratio of laboratory and field strength may vary by up to a factor of ten (Goodman 1980), the GSI values were obtained directly from the field. The GSI classification takes rock mass, having spatially distributed discontinuities, into consideration. Also, as the inter-relations among the casual factors of landslides are generally non-linear in nature, therefore non-linear generalized Hoek-Brown criteria has been used.

5. Results and analysis

The results of the SSR analysis performed using Phase^2 on the debris and rock models are described below:

5.1. Debris model

The computation of debris slope model reveals significant value of displacement in the zone of detachment. The analysed model is presented in absolute horizontal (figure 6(*a*)) and absolute vertical (figure 6(*b*)) formats which are defined below.

5.1.1. *Absolute horizontal displacement (AHD)*

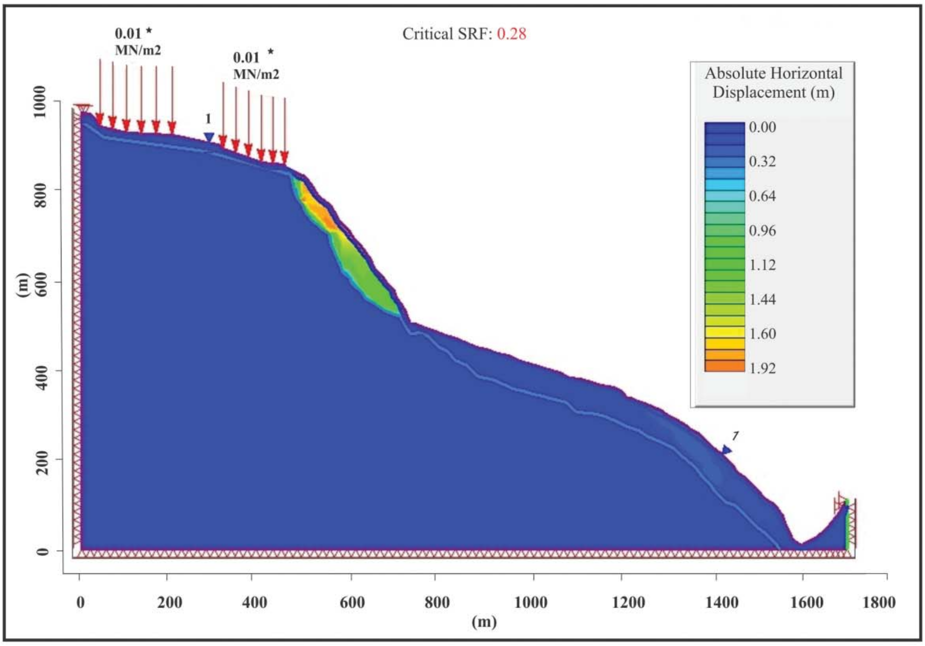
The term absolute here refers to positive displacement. This output (figure 6(*a*)) shows a progressive trend of displacement towards crown portion in the zone of detachment. Displacement of the order of 1.4−1.6 m in the upper part (near the crown) and 0.8−1.0 m in the lower part of zone of detachment is observed in our analysis. The results indicate that the crown portion of the landslide is vulnerable to failure in future.

5.1.2. *Absolute vertical displacement (AVD)*

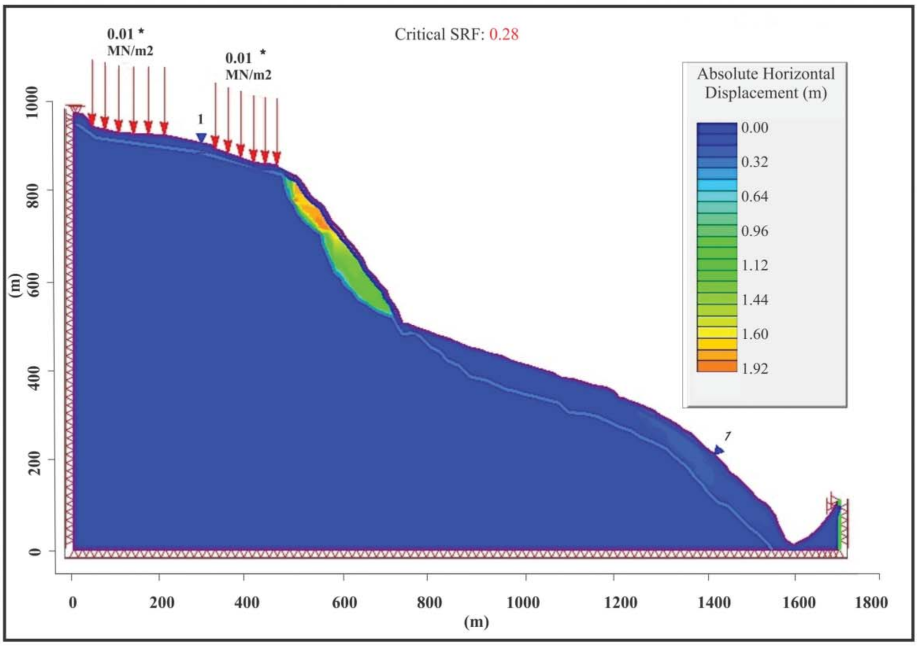
In comparison to AHD in the debris model, the value of AVD is observed to be less. However, the trend of progressive displacement within the zone of detachment (figure 6(*b*)) is also found here. Upper part reveals a displacement order of 0.6−0.9 m while in lower part of detachment zone, 0.3−0.5 m displacement is detected.

5.2. Stress development along interface

The interface in our debris model which provides relaxed connectivity to underlying rockmass and overlying debris presents progressive non-linear development of stress, down the slope (figure 7). Interestingly, normal and shear component of stress are found to be symmetric with each other in this development.



(a)



(b)

Figure 6. (a) Absolute horizontal displacement in debris slope. The location and elevation coordinates are in meters. (b) Absolute vertical displacement in debris slope. The location and elevation coordinates are in meters.

5.3. Rock model

The rock slope model which is also analysed under gravity, *in situ* stress & building load presents interesting pattern of displacement. Absolute horizontal (figure 8(*a*)) and absolute vertical (figure 8(*b*)) displacement outputs of rock slope model are presented below.

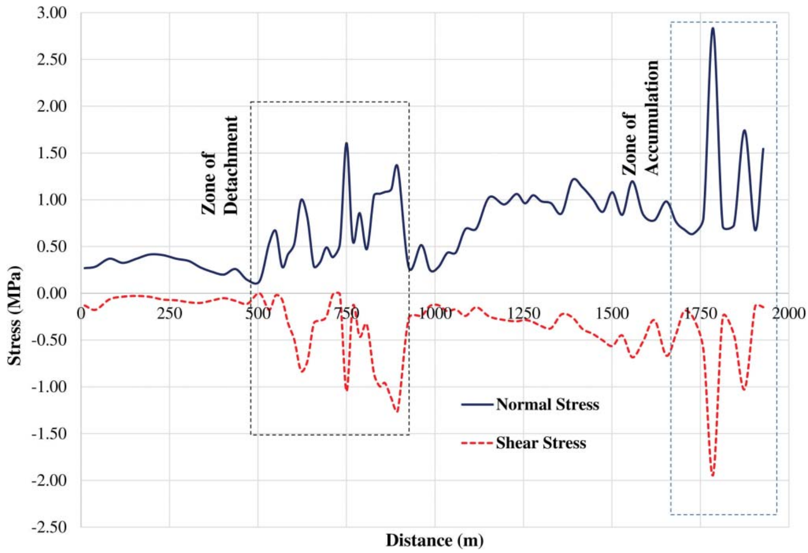


Figure 7. Normal and shear stress accumulation at the debris-rock interface.

5.3.1. *Absolute horizontal displacement (AHD)*

This value in the rock slope model also put forward a progressive regime of displacement in detachment as well as accumulation zone with an order of 0.05−0.07 m and 0.04−0.05 m, respectively (figure 8(*a*)). Another noteworthy outcome is a slip surface kind pattern of displacement which strengthen the applicability of FEM analysis over limit equilibrium methods which require presumption about slip surface.

5.3.2. *Absolute vertical displacement (AVD)*

AVD value in the model exposes a pressure bulb pattern at the crown portion in which progressive displacement can be noticed (figure 8(*b*)). High normal stress in this region may be responsible for this pattern. In this pressure bulb pattern, a progressive regime of displacement with an order of 0.04−0.07 m is observed.

6. Discussion

There are many methods for the stability analysis of slope. These are categorized into conventional and numerical methods. We have numerically modelled Surabhi Resort, rain triggered landslide using FEM in the static condition considering it as continuum by SSR approach. By determining the factor of safety of failed slope and field measurement of displacement, post disaster analysis is carried out. While by using FEM methodology, stress & displacement development in the slope is determined to focus on probable failure (pre-disaster analysis). The analysis was performed using *PHASE*^{2} software. FEM, widely accepted method for numerical modelling of slopes works on the principle of discretization of whole design into fixed number of elements through which continuous variation in material properties take place. A 2D, six node triangular plane strain elements have been used to discretize the slope design for debris and rock (dolomitic limestone) model. It has been observed that six-node elements improves the accuracy particularly when used with non-convergence failure criteria in SSR approach (Nian et al. 2011). Therefore, SSR approach with non-failure criteria has been adopted. Since the maximum shear strain of failure zone coincides with the rupture surface, it is thus assumed that failure mechanism of slope is directly related to the development of shear strain.

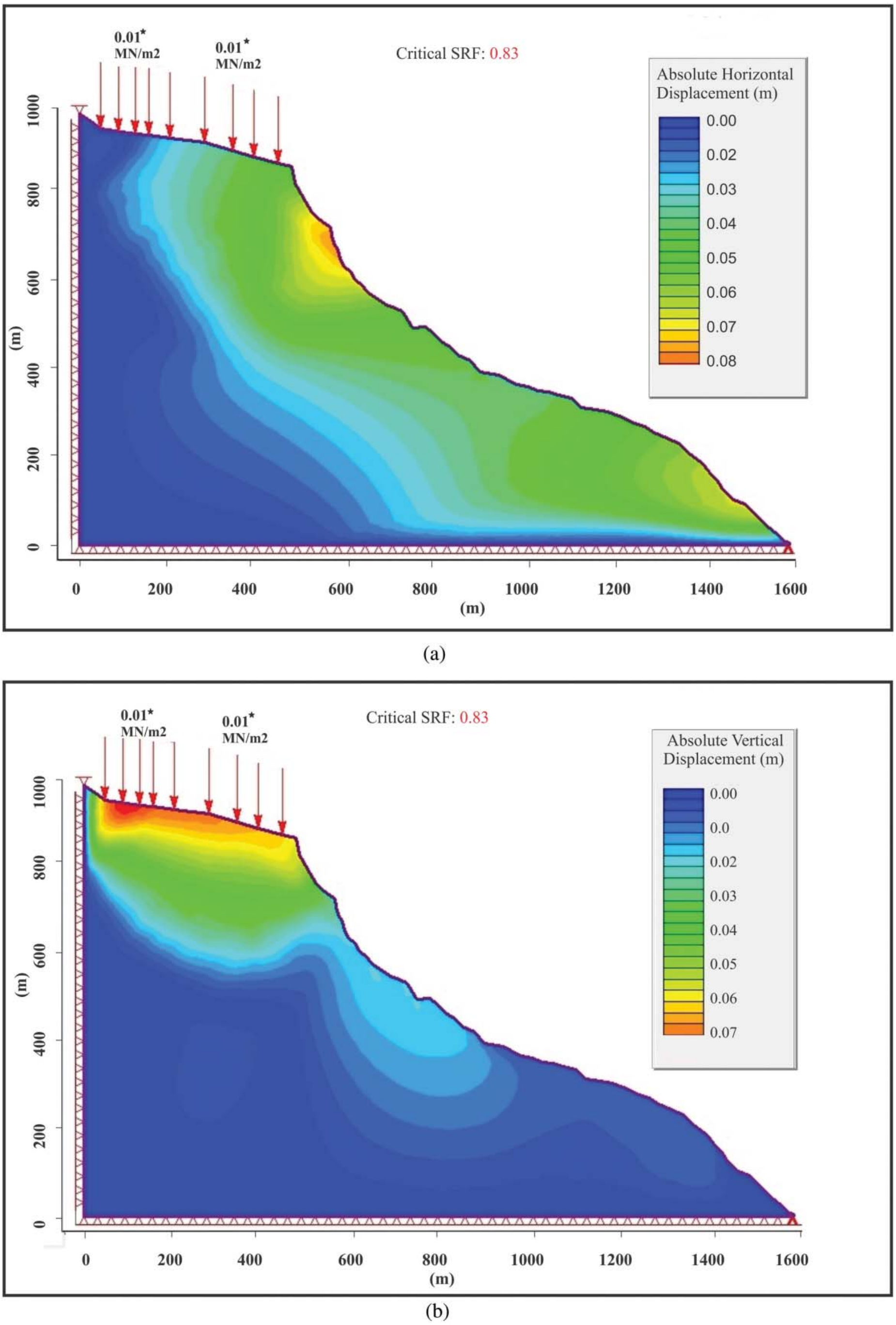


Figure 8. (a) Absolute horizontal displacement in rockmass slope. The location and elevation coordinates are in meters. (b) Absolute vertical displacement in rockmass slope. The location and elevation coordinates are in meters. (c) Field photograph of the crown portion showing intensely fractured rockmass.

The analysis of the model envisages the horizontal displacement of the order of 0.8−1.6 m in the debris model and about 0.05−0.07 m in the rock model in the crown portion of the landslide. The crown portion of the landslide has also been monitored with precise instrument total station during Jan.−Nov. 2005, after the occurrence of the landslide (Gupta & Ahmed 2007) and the results

(c)

Figure 8. (Continued)

confirmed that the crown portion is moving. The higher concentration of maximum shear strain and total displacement in the zone of detachment and zone of accumulation may be attributed to steep slopes (45°–65°) in these regions. The loose overburden on these slopes and highly jointed rock mass are more susceptible to displacement in these regions. Vertical displacement value in the rock model exposes a pressure bulb pattern at the crown portion (figure 8(*b*)). High normal stress in this region may be responsible for this pattern. In this pressure bulb pattern, a progressive regime of displacement with an order of 0.04–0.07 m is also observed.

The same slope has also been studied statically and dynamically by Pal et al. (2012) using discontinuum approach. They observed a maximum shear displacement of 5.8–9.0 cm and maximum net horizontal displacement of 67–69 cm in static analysis and concluded the detachment zone as vulnerable. The displacement results obtained from both continuum and discontinuum approach for the rock model are more or less similar. The displacement in the area is evidenced by the presence of subsidence zones in the MIS and cracks in the buildings of Surabhi Resort and MIS. This demands an immediate step to be taken so as to avoid any further destruction to the buildings and to check the movement of the slope.

7. Conclusions

Two separate design models for overlying debris and underlying rock mass have been prepared and run using Phase^2 software. The FEM analysis results of our slope model (figure 6(*a*) and 6(*b*)) are found to be in correlation with the high resolution displacement data by Gupta and Ahmed (2007) which presented a displacement order of 4–14 mm/year in the zone of detachment. The pulverized and intensely fractured exposure of rockmass (figure 8(*c*)) in zone of detachment supports our analysis finding having high displacement in this region (figure 8(*a*) and 8(*b*)).The study draws the following conclusions:

- Debris model and rock model yielded a factor of safety of 0.28 and 0.83, respectively indicating that the failed slope is highly unstable and thus, need a continuous monitoring. The crown portion of the landslides is highly unstable and this has also been evidenced in the field.

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Disclosure statement

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