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# *Risk Analysis of High Hill Slopes - A Case History*

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**ABSTRACT**

The recent cloud burst and subsequent landslides in Kedarnath caused huge destruction to life and property. The incident demonstrates that the engineering practices are not utilized judiciously in such regions before the construction of any civil structures. Construction of several civil structures like dams, tunnels and transportation corridors requires excavation of several tons of earth material on a daily basis. This disturbs the natural equilibrium of the hill slopes and causes huge amount of stress relief leading to slope failure. The external triggering factors like rainfall and earthquake further accelerate the failure processes. Joints in a rock mass significantly reduce the stability of the entire structure. Planar, wedge and toppling are the most common mode of failure in jointed rock slopes. Therefore, slope stability analysis becomes very important in such types of construction activities, as there is an immediate threat to the travellers and local human settlement. In this study, cut slope sections along the right bank of river Sutlej, Luhri, and Himachal Pradesh have been taken to emphasize the influence of rock mass parameters on their stability. Field and laboratory study was incorporated with high end numerical simulation to analyze the stability of the area of interest. Later on, risk analysis was also done to account for the variability in rock mass parameters and to assess the probability of failure.

***Keywords:*** Cut slope; Probabilistic analysis; Risk analysis; Luhri; Jointed rocks

## 1. INTRODUCTION

The mighty Himalayas are the youngest highly active seismo-tectonic belt in India subcontinent. The geology and tectonics of Himalayas is complex as the rocks have suffered several stages of deformation resulting into number of big and small scale structures like folds, faults, foliations, joints etc. These structures impart several planes of weaknesses in rocks and reduce their strength appreciably. Due to the human inhabitation on the hills and increase in the number of travellers taking the roadways, stability of highly jointed rock masses on the road cut-slopes has become a major concern (Singh et al., 2013).

The stability of rock slopes is the manifestation of the type and frequency of discontinuities present in the rock mass (Einstein et al. 1983). A single joint seldom governs the stability of slopes rather than a set of discontinuities which as a whole constitutes the governing factor responsible for failure in rock slopes. Presence of these structural discontinuities in rocks makes them heterogeneous and a source for other agents to further degrade the material. Consequently, it becomes imperative to equally quantify the properties of intact rock material and discontinuities present in them (Jang et al., 2006).

In order to characterize a rock mass, ISRM (1978) suggested the quantitative description of all the discontinuities by assessing several parameters like orientation, persistence, spacing, aperture, roughness, infilling and joint wall strength. Often, in many design purpose, most of these parameters are neglected to simplify the model. Kim et al. (2007) working on non-persistent joints estimated block size of rock masses and suggested that if the joints are considered to be persistent, as in most designs, the size of the rock block tends to be underestimated. Further, the rock mass quality (*Q*) and Geological strength index (*GSI*) do not consider the effect of joint persistence and even though the rock mass rating (*RMR*) gives less weightage to persistence which is under estimated as pointed out by Kim (2002). In this regard, Kim et al. (2007) mentioned that the oversimplification may lead to underestimation of block size and overestimation of the size of the joints, ultimately affecting the number of removal blocks near the excavated face. This will increase the necessity of installation of support system even on a surface where it may not be a prerequisite.

Instability thus is a very common phenomenon in highly jointed rock mass. Complex jointing pattern gives rise to several modes of failure in rock mass like planar, wedge and toppling which bring further uncertainty in analysing the slopes (Kainthola et al., 2012a). Mapping of discontinuity orientation and spacing is not a challenging job but the parameters like persistence, block size and roughness remain hazy and hence, engineering geologists always come up with conservative results. Previously, several research articles have been articulated using different approaches around this region but no work suggests probabilistic analysis (Sarkar and Singh, 2008; Sarkar et al., 2008 & Singh et al., 2015). The present study, along the right bank slope of river Sutlej, Luhri, Himachal Pradesh, takes into account several probabilities of the joint distribution in a rock mass to render the cut slopes unstable.

## 2. STUDY AREA

### 2.1 Geology

The study area is a part of young geologically dynamic mountain chains of Lesser Himalaya located between Longitudes 310$^o$ 20' to 310$^o$ 22' and Latitude 770$^o$ 24' to 770$^o$ 30' (Fig.1). The main source of drainage is the river Sutlej which is responsible for shaping the beautiful landscape of the area. Sutlej flows through a narrow V-shaped valley characterized by rapid down-cutting. The river bed material can be traced to about 914m (above mean sea level) near Rampur and varies on moving further downstream (Srikantia and Bhargava, 1998). The area is characterized by highly deformed metamorphosed sedimentary and igneous rocks. Geologically, Kullu formation comprises the oldest rocks which are thrusted over the youngest Larji formation which gives rise to window in a window structure. The Kullu formation consists of central crystalline schist and gneisses, whereas Larji primarily constitutes dolomite, quartzite, along with slates and phyllites (Lakhera et al., 1980).

### 2.2 Geotechnical Characterization

The rock cut slopes along this stretch are mainly banded gneisses with three sets of joints. The foliated parallel joint has large continuity in comparison to others and dips roughly in NNW to NNE direction. The other set of joint forms a wedge structure which is a major cause of failure along the road cuttings of right bank of the river Sutlej. The structural data for the joint parameters have been shown in Table 1.

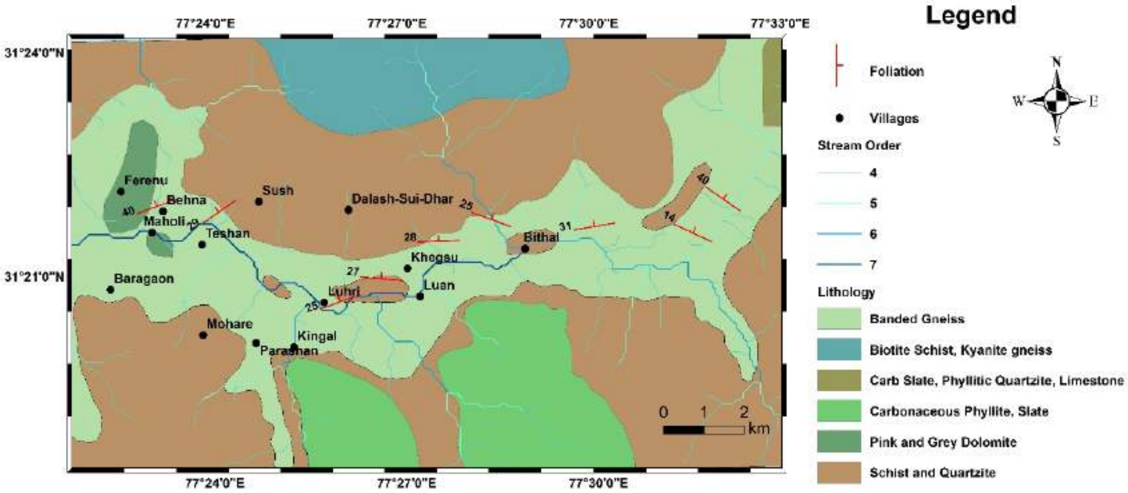



Fig. 1 - Geological map of the study area (after GSI, 1999)

Table 1 - Field observation of rock joint parameters

| Joint | Orientation | Persistence (m) | Spacing (m) | Aperture (mm) | Infilling | Roughness |
|---|---|---|---|---|---|---|
| Foliation (F) | $30^o/350^o$ | 3-10 | 0.7-1.2 | 3-4 | Clay | Rough undulating |
| Joint 1 (J1) | $65^o/175^o$ | 1-4 | 0.6-0.8 | 2-6 | Open | Smooth |
| Joint 2 (J2) | $80^o/245^o$ | 1-5 | 0.2-0.7 | 2-4 | Open | Slightly rough |

Field observation shows different sizes of wedge formation in gneisses mainly due to J1 and J2 joints (Figs. 2a & b). By using lower hemisphere equal area stereonet, structural data were plotted to infer the relationship between the slope face and geological discontinuities. Kinematic analysis, a geometrical means to identify the potential mode of failure in the rock masses, suggests that single plane sliding is possible in this region. The line of intersection of J1 and J2 joints, forming the wedges, lies within the failure envelope (Fig. 2c). Therefore, sliding will take place along J1 while J2 acts acts as release surface (Yoon et al., 2002). The red arrow shows failure direction which is in the southerly direction almost parallel to dip of the slope face. J1 also strikes nearly parallel to slope face, i.e., within $\pm 10^o$, therefore, chances of planar failure cannot be ignored.

The weathering condition of joint surface (J1) was assessed using Schmidt hammer and the weathering index or degree of weathering of discontinuous surface ($W_c$) was estimated using Eq. (1) as following:

$$W_C = \frac{\sigma_C}{JCS} \qquad \text{(Singh and Gahrooee, 1989)} \qquad (1)$$

Where $JCS$ is joint-wall compressive strength (MPa) and $\sigma_c$ is uniaxial compressive strength (MPa). $JCS$ was obtained using Eq. (2) (Barton and Choubey, 1977; Singh and Goel, 2002).

$$\log JCS = 8.8 * 10^{-4} \gamma R + 1.01 \qquad (2)$$

The parameters $\gamma$ is unit weight (26.5kN/m$^3$) and $R$ is Schmidt hammer rebound number.

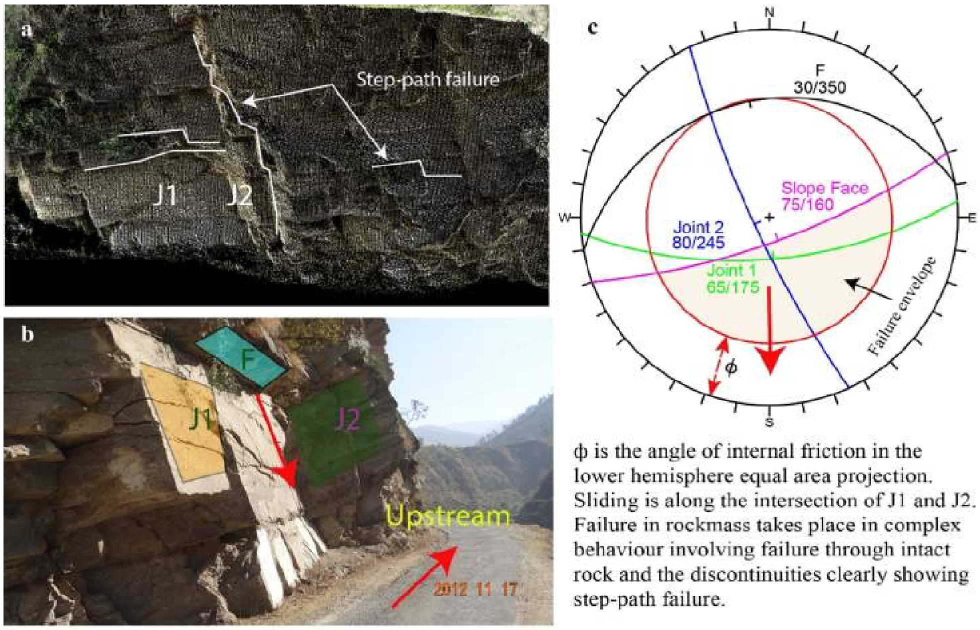



Fig. 2 - LiDAR point cloud data with field-photograph showing joint sets and failure direction (*Red-arrow indicates probable direction of failure*)

Depending upon the values of $W_c$, discontinuities were classified as given in Table 2. Using Eqs 1 & 2, a weathering index of 1.26 was obtained corresponding to a rebound value of 35, UCS of 85MPa, which indicates that the joint surface is moderately weathered (Table 2).

Table 2 - Variation of weathering index for discontinuities (Singh and Gahrooee, 1989)

| Weathering condition of discontinuities | $W_c$ |
|---|---|
| Fresh to slightly weathered | <1.2 |
| Moderately weathered | $1.2 < W_c < 2.0$ |
| Weathered | >2.0 |

The employment of different classification schemes for the deformed Himalayan rocks provides insight into their applicability, while at the same time acts as a precursor for suggesting slope strengthening methods. The gneissic rock in the study area is classified in terms of commonly available classification techniques like RMR and GSI to gauge into the in-situ rock mass conditions (Singh and Goel, 2011). Several parameters used in the classification systems are bound to change due to weathering during the engineering life of slopes. Therefore, these techniques will provide a descriptive idea of the present rock mass condition.

Initially, gneisses were classified using rock mass rating (RMR), the most commonly used classification system in engineering geology (Bieniawski, 1976). The five basic parameters used in RMR system have been carefully determined based on field and laboratory study to achieve $RMR_{Basic}$ (Table 3). In the present study, $RMR_{Basic}$ corresponding to J1 has been provided as it is the daylighting joint which creates avenues for both planar and wedge failure around this region. Groundwater condition in this area was assessed from the surface behavior of water in the vicinity of the studied section. At places, the rocks were dry but at the same time surface staining and damping was also observed in patches.

Table 3 - Ratings of different parameters used in the estimation of $RMR_{Basic}$

| UCS | RQD | Discontinuity spacing | Condition of discontinuity Surface | Groundwater condition | $RMR_{Basic}$ |
|---|---|---|---|---|---|
| 7 | 13 | 15 | 14 | 12 | 61 |

Surface excavation, where low stress levels are encountered, joint deformation predominates over the intact rock deformation (Bandis et al., 1983). This causes problem in assessing reliable estimates of the mechanical properties and response of a rock mass to any engineering design because the deformation has to take place through the intact rock bridge and the discontinuities (Bahaaddini et al., 2013). RMR fails to include such behaviour into the classification and may not be able to assess the importance of joint deformation behaviour or the role of joints in rock slope failure.

Geological strength index (GSI) was later introduced not as a replacement of RMR but as a more precise method for the classification of jointed and weaker rock mass (Marinos et al., 2005; Hoek et al., 1998). Sonmez and Ulusay (1999) further added two parameters for precise determination of GSI for cut slopes. The GSI for the studied slope is obtained from this method for the critical joint J1 (Table 4). The additional parameters in the modified GSI includes surface condition rating (SCR) estimated from roughness, infilling and weathering and surface rating (SR) obtained from volumetric joint count (Fig. 3). SCR values are taken from RMR whereas, SR has been obtained from volumetric joint count ($J_v$) following Sonmez and Ulusay (1999) as shown in Fig. 3.

Table 4 - Values of parameters obtained for the estimation of *GSI* for jointed rock slopes

| Roughness rating | Weathering rating | Infilling rating | $J_v$ | Structure rating | *GSI* |
|---|---|---|---|---|---|
| 1 | 3 | 6 | 8 | 58 | 46 |

## 3. PROBABILISTIC WEDGE FAILURE ANALYSIS

Uncertainty in rock slope engineering may arise due to scattered values of joint orientation, persistence, slope geometries, shear strength values and many such parameters (Singh et al., 2013; Park et al., 2005; El-Ramly et al., 2002). Therefore, correct representation of these values in any design purpose becomes a great challenge. Probabilistic analysis accounts for the uncertainties in such values of input parameters like persistence, shear strength and joint orientation etc. using statistical variation. This allows the computation of a wide range in the values of safety factor thereby estimating the probability of failure (RocScience, 2012).

Slope failure in discontinuous rock mass mainly takes place through planar, wedge or toppling mode. Out of the three modes, the first two are the major causes of slope failure in most of the cases. Toppling on the other hand is common in sub-vertical joints. In the present study area, the joint ornamentation indicates that planar or more commonly wedge failure is dominant, therefore, wedge analysis was carried out using probabilistic analysis to account for variability in persistence, joint orientation and shear strength parameters using a RocScience software namely SWEDGE. The material properties of the intact rock were determined as per ASTM standard (ASTM D7012; ASTM E132 – 04) in the laboratory and tabulated in Table 5.

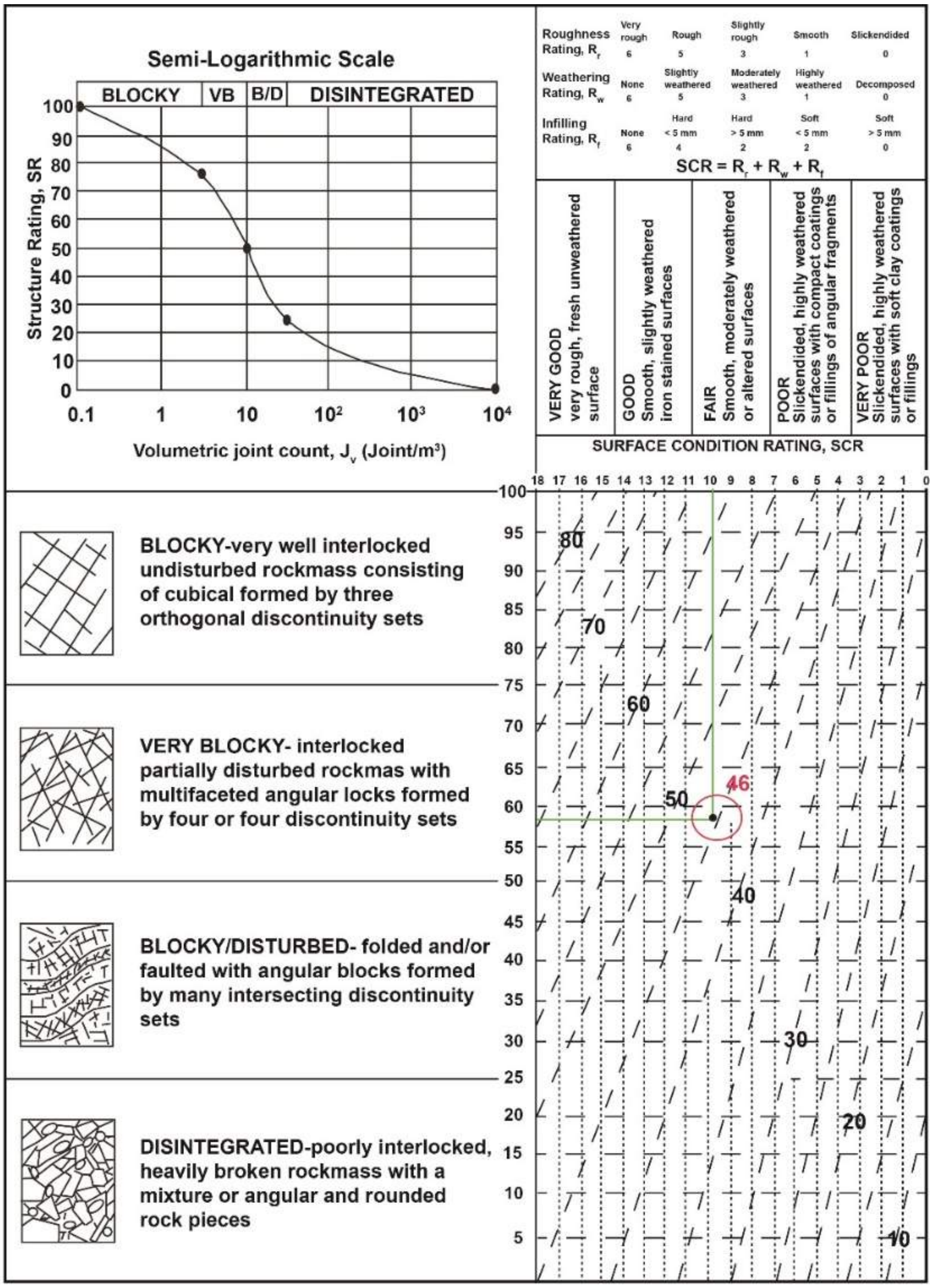



Fig. 3 - Modified GSI for jointed rock mass (Sonmez and Ulusay, 1999)

Table 5 - Intact rock properties of the gneisses from the study area

| Parameters | Value | Parameters | Value |
|---|---|---|---|
| Cohesion (MPa) | 06 | Unit weight (kN/m$^3$) | 26.5 |
| Angle of internal friction ($^o$) | 38 | Elastic Modulus (GPa) | 25 |
| Uniaxial compressive strength (MPa) | 80 | Poisson’s ratio | 0.26 |

### 3.1 Estimation of Standard Deviation

Determination of reasonable values of the essential parameters from insufficient amount of data sets is an important job in geotechnical engineering. To obtain safety factor in terms of probability of failure, standard deviation of all the parameter needs to be quantified. Depending on the availability of the data, there are several methods for estimation of the standard deviation. Sufficient amount of data sets allows the standard deviation to be calculated through simple formula given by Eq. (3).

$$\sigma = \sqrt{\frac{\sum\left[\left(x_i - \bar{x}\right)\right]^2}{N-1}} \tag{3}$$

where $\sigma$ is the standard deviation, $x_i$ is $i^{th}$ value of the parameter $x$ ; $\bar{x}$ is the average value of the parameter $x$ and $N$ is the size of the sample.

In most of the cases of geotechnical engineering, problems arise due to insufficient usable data which do not allow the users to perform reliability analysis. One of the best methods for estimating standard deviation in such situations is the application of *Three sigma rule* (Dai and Wang, 1992; Duncan, 2000). The rule suggests that 99.73% of all the data of a normally distributed random variable fall within the three standard deviation of the average value (Fig. 4). Using this method, all the random parameters were assigned a mean and standard deviation.

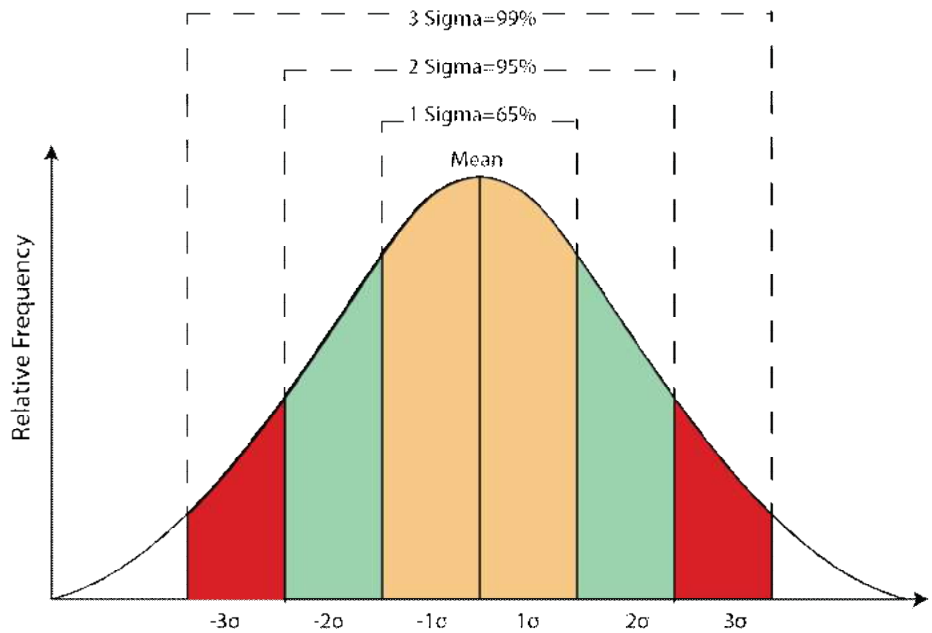



Fig. 4 - Three-sigma rule for the estimation of standard deviation of a random variable

Park et al. (2005) suggests conducting kinematic and kinetic analysis (includes forces acting on the rock mass) for both deterministic as well as probabilistic analyses. In a deterministic analysis, only a single value of all the parameters are taken to reach to a final value of safety factor whereas, probabilistic analysis considers a range of values. Only the shear strength parameters (cohesion and angle of internal friction) were allowed to behave randomly keeping the other parameters like slope height, unit weight constant. However, slope face was varied using uniform variation of $\pm 10^{o}$ to cater the variation observed in the field. In this study, probabilistic analysis was performed using most commonly used Monte Carlo simulation technique (Sazid et al., 2012). The statistical variation was done using normal probability density function (PDF), the best to represent variation in the parameters in geotechnical engineering.

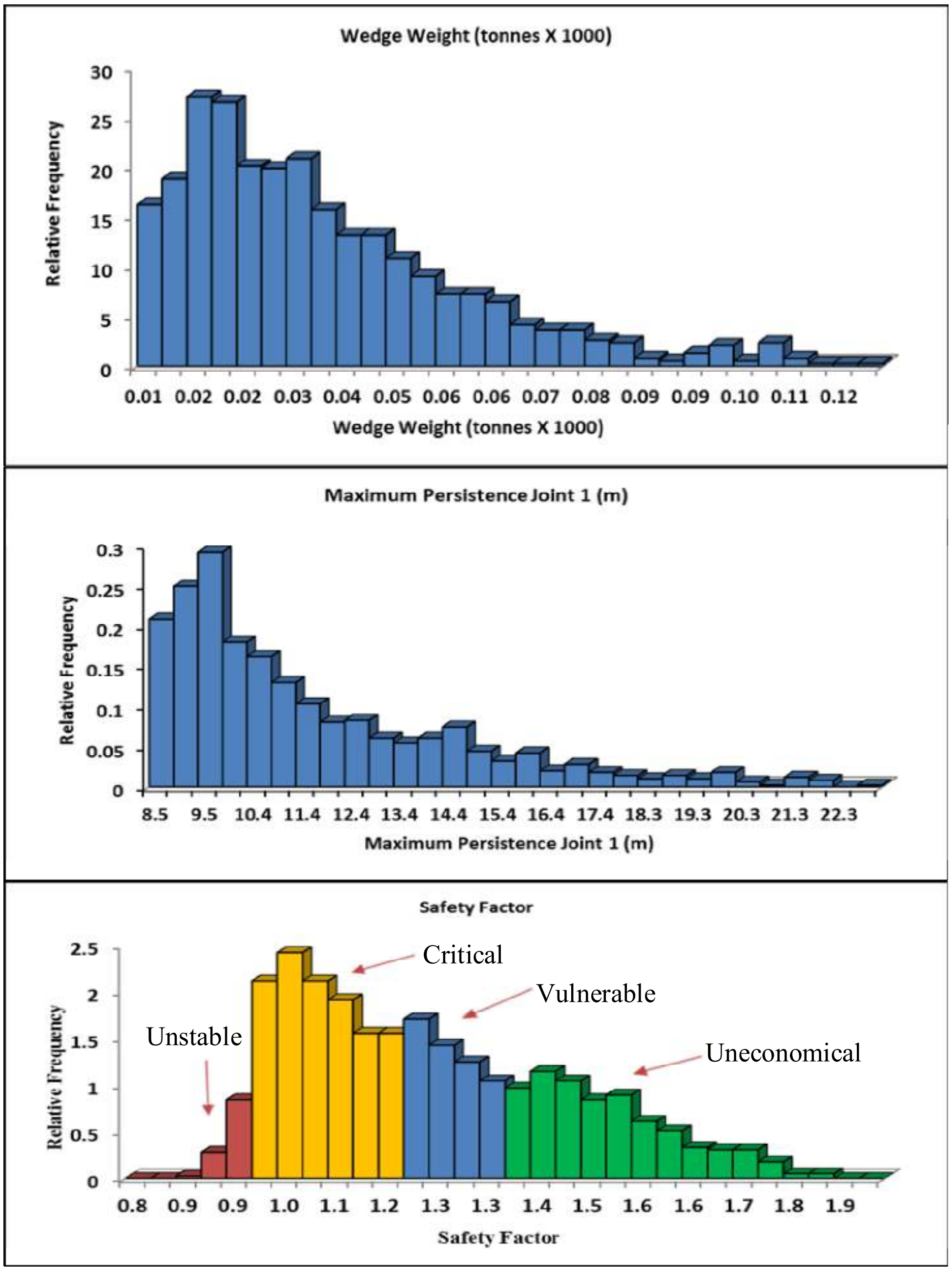



Fig. 5 - Probabilistic wedge failure analysis of the study area showing distribution of wedge weight, persistence of joint-1 and corresponding variation in safety factor

Swedge is a RocScience package capable of analytically computing both deterministic and probabilistic analysis. Such analysis assumes a single planar failure surface for the estimation of safety factor. Therefore, an increase in height would promote the chances of failure making it a very sensitive parameter. Although, the total height of the slope is around 25m and the wedge height is taken as 6m based on field observation**.** The deterministic value of safety factor was calculated to be around 1.3, suggesting stable slope conditions. The results of probabilistic wedge failure analysis are shown in Fig. 5. The PDF of wedge-weight and persistence shows the probability of relative frequency of their occurrences. Different peaks in the graph correspond to randomness encountered in the shear strength values ($c$ and $\phi$) along the daylighting joints (J1) during stability analysis (Fig. 5). Statistical variation of joint parameters and shear strength values suggests altogether a different scenario about the slope performance. The study area shows about 13% chances of failure indicated as red bars (Fig. 5). These figures only shows the chances of failure corresponding to a critical safety factor of one but the slope at factor of safety equal to 1 is at the verge of stability or failure. Practically, assuming 1.3 as reasonably safe value of safety factor, the chances of failure will further increase (Fig. 6). Therefore, statistical variation highlights all those cases which may not have been possible through the conventional or routine approaches. An unstable slope shows SF<1, critical slope with a susceptibility to failure due to slight modification <1.3, a stable slope indicating long-term stability is expected to be around 1.3, whereas, maintaining SF value beyond 1.3 may prove to be highly uneconomical (Fig. 5). So, for safer and long-term design of structures in jointed or weak rock mass, it is suggested to perform analysis that can consider a wide range of data for all random parameters which are critical in promoting slope failure.

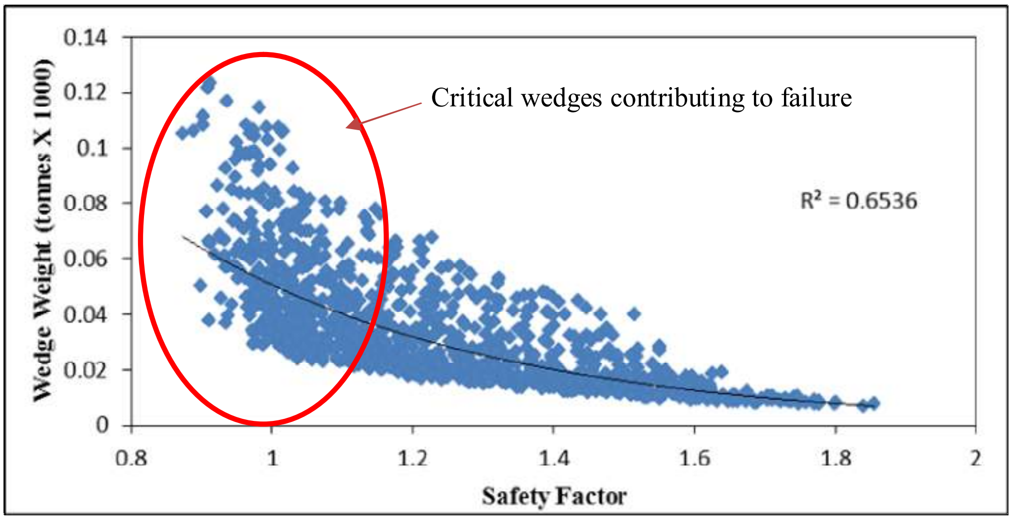


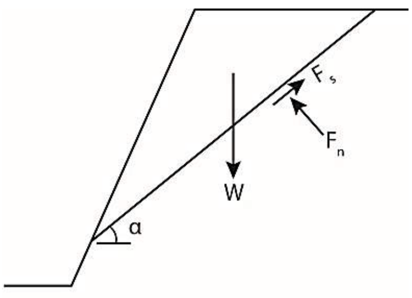

Fig. 6 - Correlation between wedge weight and safety factor

## 4. RISK ANALYSIS

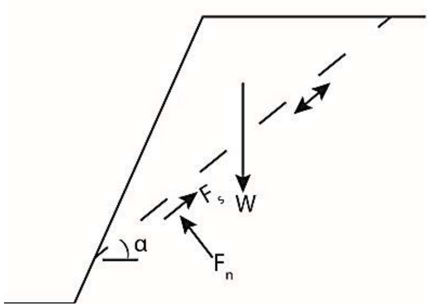
Joints in a rock mass can be continuous for a great length or may be segmented due to the presence of other discontinuities. In either case, it presents a great challenge while designing any structure or analysing stability of slopes. They are the weakest zone in the rock mass and therefore should be given considerable importance while constructing any engineering design (Kainthola et al., 2012b). Joints have the ability to significantly reduce the available resistance of the rocks by creating open spaces which are often filled with secondary infillings. In several cases, the type of infilling material can also govern the shear resistance to failure. Clayey material residing in fractures can certainly aggravate the situation and reduce the resistance available along the sliding surface (Wyllie and Mah, 2004). There are certain important parameters like orientation of discontinuities, persistence which are very crucial and should be dealt with great knowledge and understanding. Together, these parameters play an important role in deciding the fate of any rock slopes.

Degree of persistence ($k$) is the continuation of the trace length of any discontinuity in a given direction and statistically it can be defined as the fraction of area that is actually discontinuous (Einstein et al., 1983). This joint parameter varies from 0 to 1, in which, 0 suggests that a particular discontinuity ends while intersecting others, whereas, 1 corresponds to large number of discontinuities that goes through the other ones (Jennings, 1970).

Jennings (1970) defined joint persistence and mobilized cohesion and persistence for both intact rock bridge and discontinuities (Figs.7a & b). This allowed calculation of safety factor for different values of cohesion and persistence possible. Using this formula, risk analysis was done using Monte Carlo simulation using @Risk code (Palisade Corporation, 2011). The results show that persistence is a sensitive parameter for rock slope related problems. Large and continuous joints will show low values of safety factor whereby, there is a drastic change in its value as persistence decreases (Fig. 8).




(a) Continuous joint




(b) Discontinuous joint

Fig. 7 - Simple planar failure affected by joint persistence (Eberhardt et al., 2004)

Equations 4 and 5 are used for the calculations of factor of safety for Figs. 7a & 7b respectively.

$$FS = \frac{cA + (W\cos\alpha)\tan\phi}{W\sin\alpha} \quad (4)$$

$$FS = \frac{\left[(1-k)c_i + kc_j\right]A + W\cos\alpha\left[(1-k)\tan\phi_i + k\tan\phi_j\right]}{W\sin\alpha} \quad (5)$$

Where,

| | | |
|---|---|---|
| $i,j$ | = | Intact and joint property, |
| $\alpha$ | = | Angle of sliding plane, |
| $\varphi$ | = | Angle of internal friction,, |
| $c$ | = | Cohesion, |
| $A$ | = | Area, |
| $k$ | = | Persistence = [ΣJoint/Σ(Joint+Gap)], |
| $W$ | = | Weight of wedge, and |
| $F_s$, $F_n$ | = | Shear and normal force. |

Modelling a slope with persistent joint sets, as in most cases, will tend to under-predict the block size resulting into removal of large number of blocks in comparison to in-situ conditions. On the other hand, inability to determine reasonable shear strength values of rock-bridge and discontinuities may lead to estimation of lower rock mass strength and excessive cost of support. The results obtained from this paper clearly demonstrate the influence of persistence on safety factor and will allow the designers to apply just sufficient support system reducing the total cost. A wide range of data is available through such analysis which will account for unforeseeable circumstances.

RMR value shows that the rock mass condition is in good category which clearly under predicts the behaviour of rock. On the other hand, GSI value is quite low which is relatively more accurate in prediction of jointed rock mass condition. Probabilistic analysis also supports GSI values and estimates >13% chances of slope failure along the analyzed cut slope.

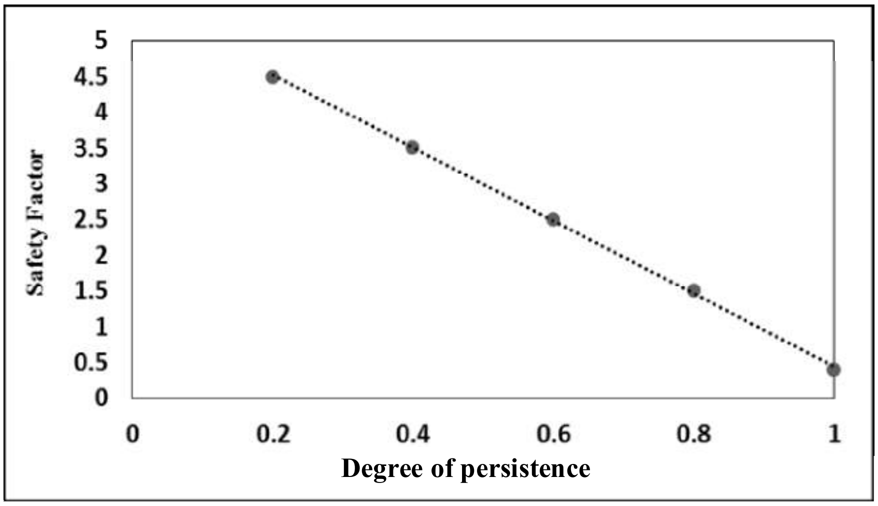



Fig. 1 - Variation of safety factor with degree of persistence

## 5. CONCLUSIONS

The study focusses on the role of discontinuities in slope performance by utilizing different techniques including detailed field observation. Geological and geotechnical characterization of the cut slope section was done from visual inspection and several widely used classification techniques like *RMR* and *GSI*. Modified *GSI* was given more attention because of their applicability in slopes and jointed rock mass and also because of the parameters used in its classification. Later, weathering condition of the joint surface was estimated based on $\sigma_c$, *JCS* and Schmidt hammer values. Kinematic analysis suggested both planar and wedge type of sliding because of the critical orientation of joint planes and slope face. The value of safety factor based on deterministic approach was around 1.3, which puts the slope section in stable domain. But to further verify this, probabilistic analysis was done because of the range of scattered values for several parameters. Joint parameters were given more importance in this regard and proper PDF was defined for each case. The results show that there is about 13% chance of failure suggesting the value of safety factor can go beyond 1 for several cases. For long term stability of slopes, a stable safety factor is generally taken as 1.3. Analytical formulation was also used to derive the values of safety factor where intact rock and joint shear strength values along with persistence were given more emphasis. Plot of safety factor and persistence shows that there is a drastic reduction in slope performance with increase in the persistence of joints. Therefore, while designing any structure for long term stability, joint data should be carefully studied and probabilistic approach should be preferred.

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