Journal of Himalayan Earth Sciences Volume 50, No. 1A, 2017 pp. 86-99

Back analysis of an earthquake triggered landslide in Mansehra district, Pakistan

Aisha Noor*, Sohail Akram, Luqman Ahmed and Waqas Sarwar
Institute of Geology, University of the Punjab, Lahore
**Corresponding Author's email: aisha.nooor@gmail.com*
Submitted: 24/03/2016, Accepted: 15/03/2017, Published online: 30/03/2017

Abstract

This paper investigates the slope failure along the Karakorum Highway near Sim Elahi Jung Village, District Mansehra Khyber Pakhtunkhwa, Pakistan by using numerical back analysis method. The slope face was ~80m high with a cut angle of 80 degree in quartz mica schist of Tanawal formation. The failure was triggered by 26th October 2015 earthquake of moment magnitude (Mw) of 7.5. The slope was investigated thoroughly for the geological conditions, slope geometry and the slid mass. During field studies, discontinuity surveys were conducted for onward assessment of the rock mass quality and orientation data for kinematic analyses. In view of rock mass quality determined from discontinuity surveys, back analyses were conducted considering slope material as Hoek-Brown material with the help of computer code Slide 6.0. The rock mass input parameters were adopted in the light of field and laboratory studies at the verge of failure. Back analysis conducted using estimated value of peak ground acceleration (PGA) have indicated that the geological strength index (GSI) value and the uniaxial compressive strength of the slope forming material were 24.0 and 29.0 MPa respectively at the time of failure. The analyzed slip surface and the observed ground slip surface were found consistent with each other, thus, confirming the validity of the model. The derived rock mass parameters are presented here for the evaluation of remaining slopes in the area having similar geological conditions and geometries.

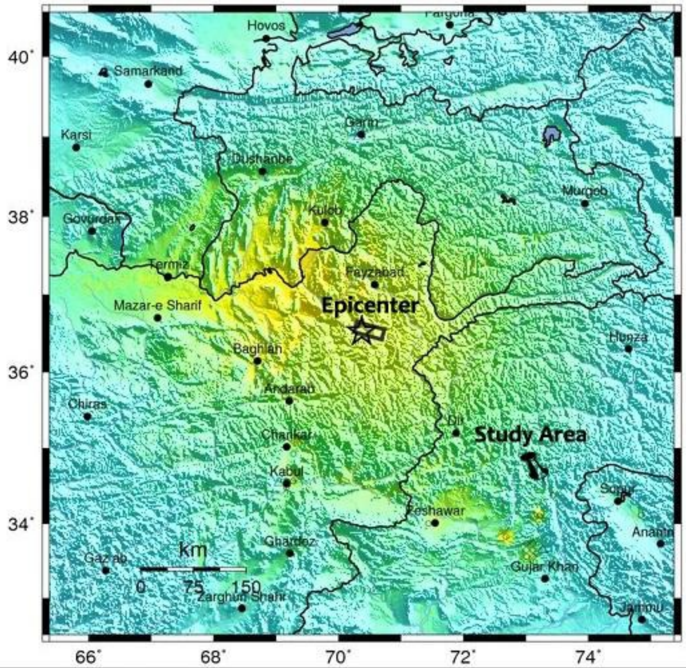
Keywords: Back analysis; Slope failure; Limit equilibrium analysis; Slope mass rating (SMR); Slope stability evaluation; Sensitivity analysis.

1. Introduction

At 14:09 (09:09 UTC) on 26th October 2015, an earthquake with a moment magnitude (Mw) of 7.5 of intermediate depth (231 km), located near the Hindu Kush region of Afghanistan (Southwest of Jurm; 36.524°N, 70.368°E), heavily trembled the main parts of Afghanistan and Pakistan along with some regions of India and China (USGS, 2015). As per United States Geological Survey (USGS) shake map (USGS, 2015), the area lies in V-VI category of instrumental intensity corresponding to moderate to strong shaking, resulting very light to light category of potential damage according to scale by Worden et al., (2012), as shown in figure 1. The earthquake resulted in 399 causalities and 2536 injured overall (Wikipedia, 26 October 2015). It also triggered numerous landslides throughout the region. This study focuses on estimation of rock mass parameters by back analysis of a 80m high cut slope that failed along the road parallel to Siran River near Sim Elahi Jung Village (Dist. Mansehra), 342 km southeast of the epicenter (Fig. 2).

The slid mass blocked the road for couple of days. The road has been constructed in the area by cutting the slopes at steep to very steep angles. The failed slope had an angle of about 80 degrees with the horizontal. This slope failure has highlighted the risk of the remaining road stretch that has more or less similar geometry. The estimation of the rock mass parameters at failures will lead to provide an important insight for the evaluation of cut slopes along the remaining road stretch with same geological conditions in response to future earthquakes.

Geologically, the failed slope lies in Tanawal formation comprising quartz mica schists in Hazara Nappe region. Hazara Nappe in the East of Besham crops out and is separated from Besham Nappe by the Thakot Shear Zone. The Hazara Nappe consists of metapsammites, metaquartzites and garnet-mica schists mainly of Tanawal formation with subordinate



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II–III	IV	V	VI	VII	VIII	IX	X+

Fig. 1. USGS Shake map for Oct 26, 2015 earthquake prepared according to scale by Worden et al., 2012 (after USGS, 2015).

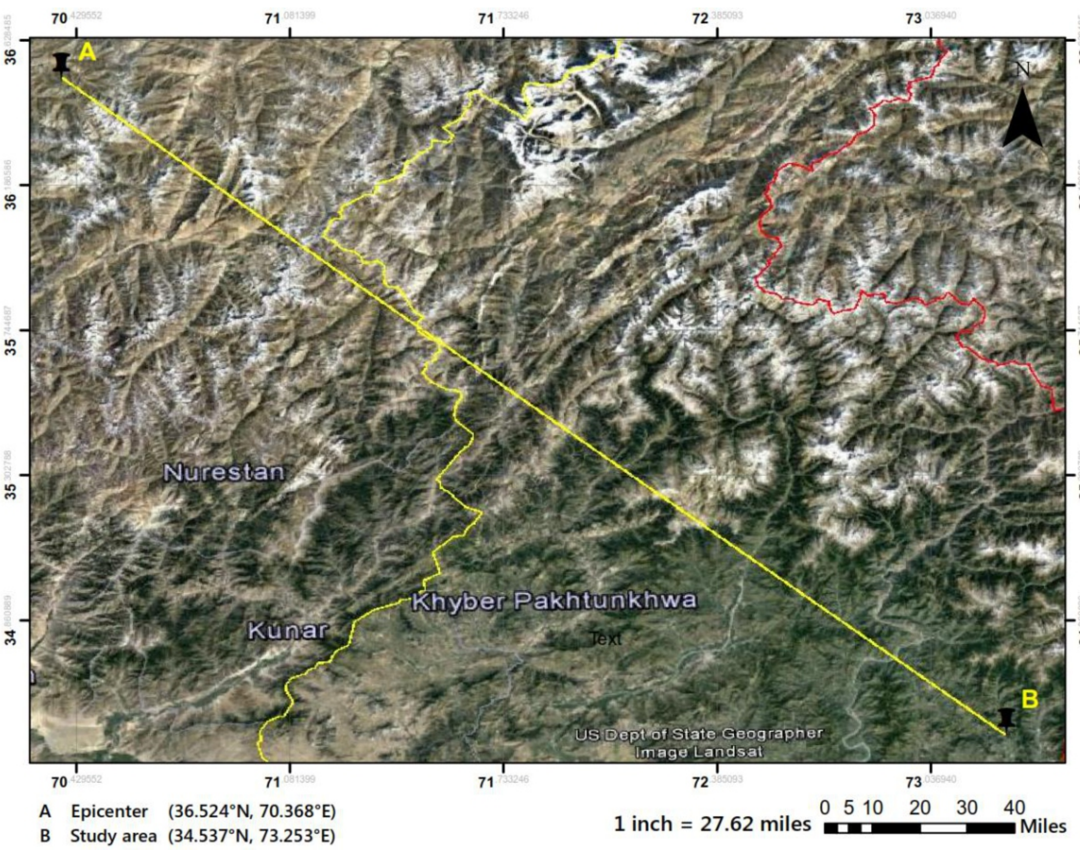


Fig. 2. Location of study area with respect to epicenter of earthquake 26th October, 2015 (Google Earth, 2015).

intrusion of porphyritic Mansehra Granite and stratification of dolomitic limestone and marl of Kingriali Formation (Calkins et al., 1969 and Treloar et al., 1989a). At valleys, alluvial and colluvial deposits are present while slopes have talus and scree/ slope washes are present at slope toes.

Tanawal Formation having Cambrian age, consists mainly of quartzite, quartzose schist, and schistose conglomerate, and is devoid of fossils (Ahmed et al., 1997). However, in the study area, quartz mica schist having trend NE-SW (subparallel to slope face) dipping in SE (in to the slope) is widely exposed. Generally, two well-developed joint sets are present in addition to the foliation. Figure 3 illustrates the geological map of the study and adjoining areas.

The back analysis of slope failure can be carried out if the failure mechanism is already established which generally aims at an improved geotechnical characterization of the rock mass under consideration. It is often performed in order to back calculate the in-situ parameters at failure by hit and trail method (Wang et al., 2013). Back analysis provides an effective understanding regarding the factors which control the slope stability and a realistic understanding of the rock mass conditions at the site. Two approaches that commonly used to perform back analysis are probabilistic and deterministic.

Deterministic approach defines a unique set of parameters by considering a unit factor of safety for the equilibrium conditions. However, probabilistic approach provides an extended opportunity to analyze multiple sets of parameters simultaneously and hence gives an indication to the extent of uncertainties in the back analysis (Ng et al., 2014). For the present failed slope, deterministic approach was followed, that is, to back calculate the in-situ rock mass parameters at the failure conditions from the observed information at the studied site and then matching the failure mechanism of the model with that of the observed one. The details are provided in the following section.

2. Methodology

The back analysis for slope stability involves three steps: modeling the geometry, establishing failure mechanism as that of ground conditions, estimation of rock mass parameters by back calculation at force and moment equilibrium conditions, and then varying the parameters in order to reach a rational set of parameters.

The inherent unreliability and difficulty in estimation of rock strength parameters by trial and error method can be controlled effectively by the observed characteristics and properties of the rock mass (Turner and Schuster, 1996).
In the light of general medthodology framework outlined above, a rigorous research methodology was devised for the back analyses of the failed slope.

The sequence and the flow of the main steps followed in the back analysis are illustrated in figure 4 and are summarized below:

- Field investigation of the failed slope; including assessment of slope geometry and failure mechanism, rock sampling for onward laboratory testing and recording of discontinuity parameters by conducting discontinuity surveys.

- Kinematic analysis using DIPS 6.0 (Rocscience Inc., 2012) on the orientation data collected during discontinuity surveys to assess the role of discontinuities towards slope instability.

- Empirical slope evaluation using Rock Mass Rating (RMR) (after Bieniawski, 1989) and Slope Mass Rating (SMR) (after Anbalagan et al., 1992) based on the intact rock strength and discontinuity characteristics. Also the derivation of Hoek-Brown rock mass parameters, (mb and s) based on the rock mass strength and structure that is, geological strength index (GSI).

- Global Numerical analysis with the actual slope geometry and adopted rock mass parameters for the determination of unit factor of safety (i.e. F.O.S=1) using Slide 6.0 (Rocscience Inc., 2010).

- Run the model with seismic loading , that is, applying the value of peak ground acceleration (PGA) estimated based on the available and applicable reltations and USGS shake map.

- Matching the failure mechanism of model with actual failure.

- Sensitivity analysis of geological structure i.e. GSI to match the failure mechanism.

The details of the study components are discussed in the following sections.

2.1. Field studies

The field studies were focused to investigate the failed slope for an assessment of the slope geometry and the contributing factors besides the earthquake. In this regards, slope failure features i.e. extent of slid mass (Fig. 5c), escarpment (Fig. 5b), tension cracks (if any), etc. were recorded on the base map comprising satellite image. Handheld GPS was used to record information on the base map. An estimation of the slid mass was made by drawing geological sections across the failed slope together with demarcation of observed thickness of slid mass. It was estimated that about 43750 m^3 material was displaced downslope during the earthquake.

Two discontinuity surveys on intact slope on either side of the failed slope were conducted to record the discontinuity characteristics for onward derivation of rock mass parameters. The discontinuity parameters i.e., orientation, spacing, persistence, aperture, joint wall strength and roughness of three identified joint sets (i.e., Foliation, F and joint set J1 and J2) were recorded following ISRM's suggested methods (Brown, 1981). Joint set J1 was found to dip in to the slope towards slope face having almost same trend as that of slope face, in SE direction. Foliations and joint set J1 were closely spaced while joint set J2 was moderately spaced with an average spacing of 8.5 cm, 7.4 cm and 23.2 cm respectively.

The slope geometry at the margin of failure was surveyed whereas an attempt was made to establish the groundwater conditions or phreatic surface during field studies. The height of the slope is 80m with and inclination of 80 degrees (Fig. 5d).

The dominant lithology comprises of quartz mica schist (Fig. 5a). The joint wall strength was estimated by Schmidt rebound hammer applied perpendicularly to the wall of discontinuities following ISRM's recommended procedure (Brown, 1981). Persistence, spacing and aperture of the discontinuities were measured with the help of measuring tape and scale. Surface roughness and undulation of the discontinuities were determined using profilo-meter. All the discontinuity data was recorded on the field pro-forma.

Ten representative rock specimens were collected for onward laboratory testing.

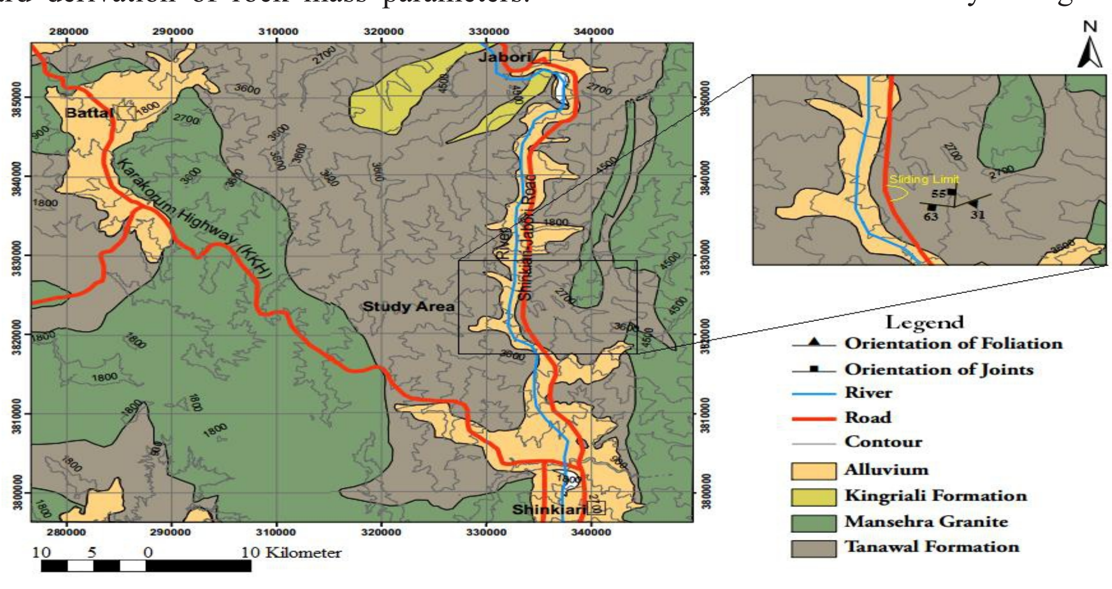


Fig.3. Geological map of Shinkiari and adjacent areas (Redrawn after GSP, 1975).

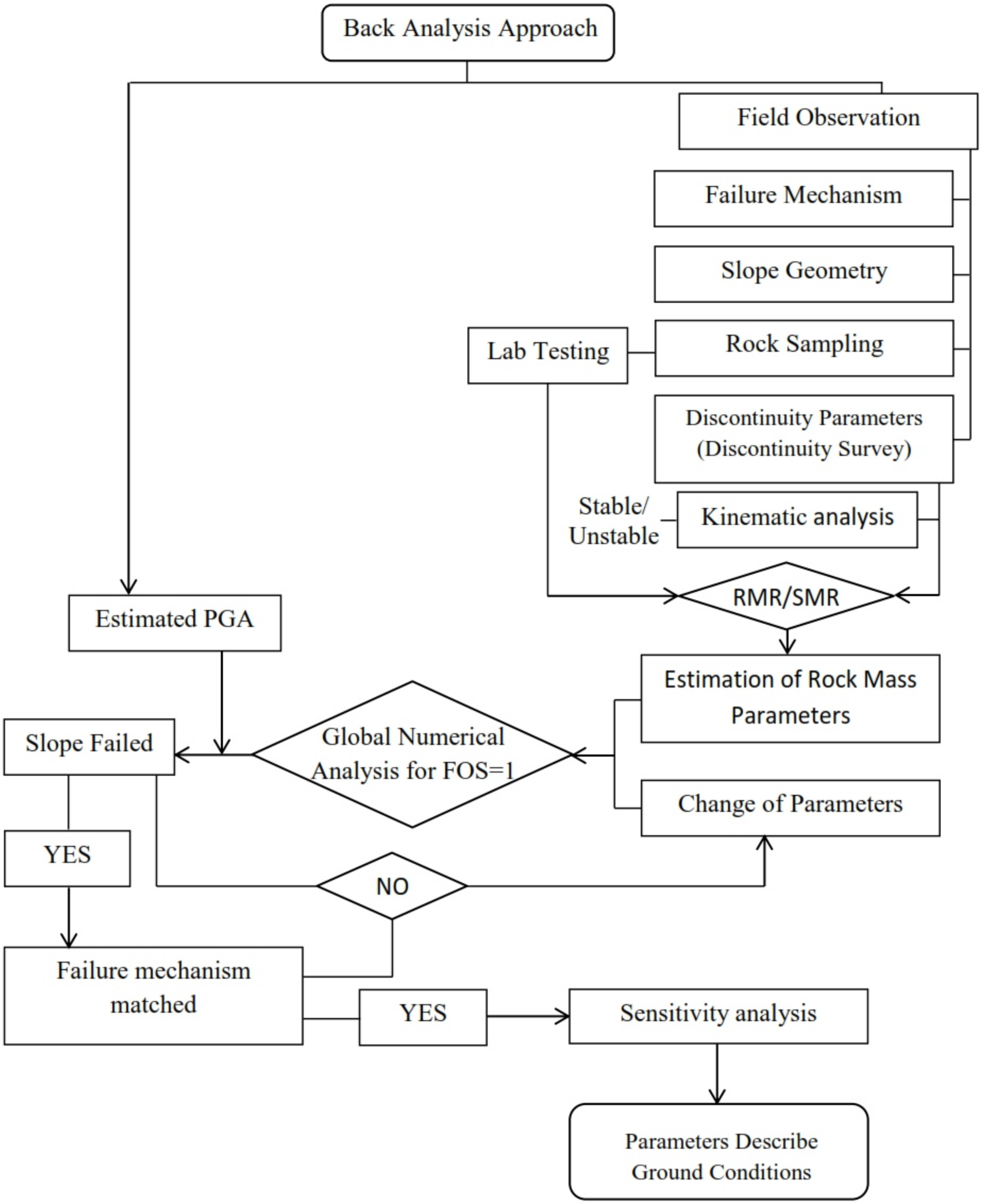


Fig. 4. The sequence and the flow of the main steps followed in the back analysis.

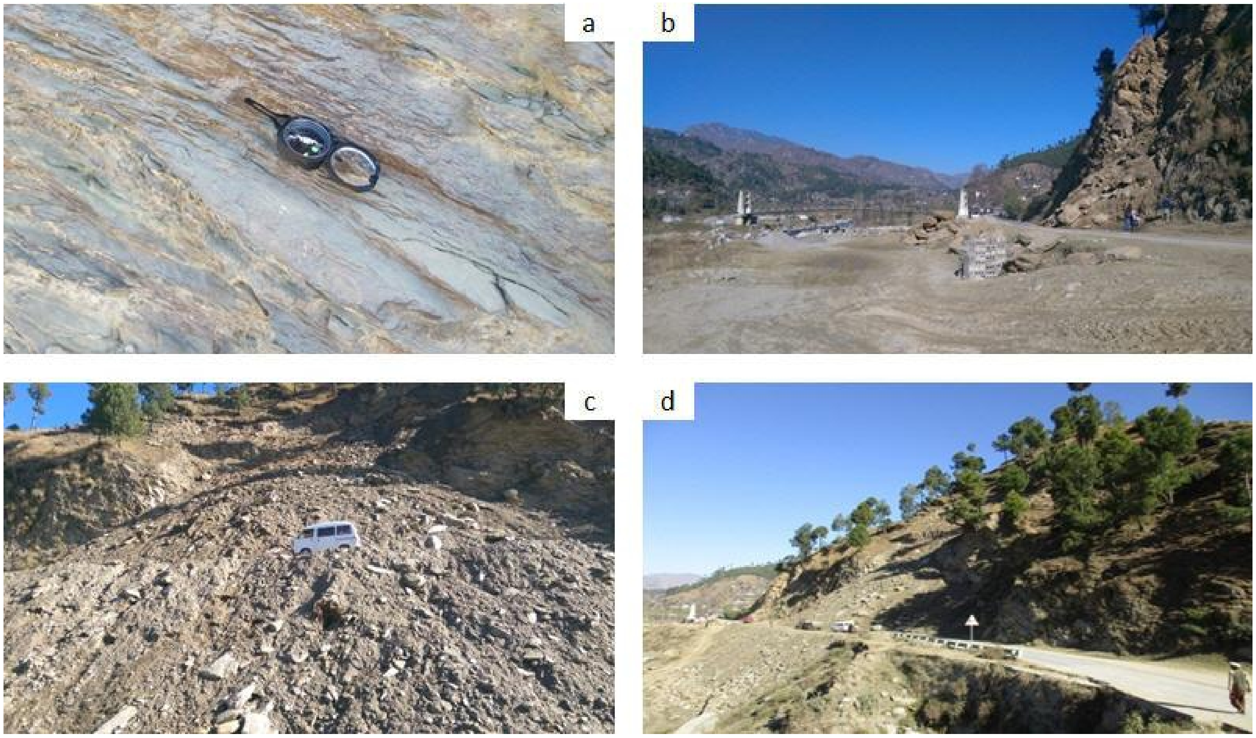


Fig. 5. Photographs showing: a) Close view of the schist at study area, b) Upstream view of the slide and Siran River, c) Sliding area limits, and d) panoramic view of the slide.

2.2. Laboratory testing and adoption of geotechnical parameters

The rock specimens collected during field studies were tested in the laboratory for the determination of rock unit weight (y) following ISRM's suggested method (after Brown, 1981). The determined bulk density was found to vary from 23.6 to 26.40kN/m^3. An average value of 25.48kN/m^3 was adopted.

Likewise, the average value of Schmidt Rebound number (Rn) was adopted as 20.

Using adopted values of both unit weight and rebound hammer number, uniaxial compressive strength (σci) was estimated with the help of correlation chart of Miller (Miller, 1965) as per ISRM guidelines (Brown, 1981). The estimated value of σci of 29.0MPa was adopted for onward empirical estimation and rock mass parameters.

2.3. Kinematic analysis

The orientations of discontinuities recorded during the field work were analyzed by computer code DIPS 6.0 (Rocscience Inc., 2012) to evaluate the modes of failure in the rock cut slope. The basic input parameters for kinematic analysis are dip and dip direction of discontinuity planes (measured by discontinuity surveys). The orientation of discontinuities with respect to slope face orientation indicate the likelihood of various modes of rock slope failure, that is, plane failure, wedge failure, toppling and circular failure (Hoek and Bray, 1981). The results of kinematic analysis are presented in figure 6. The daylighting of joint (J2) on slope face indicates the likelihood of planar failure in the slope (Fig. 6). Wedge failure is also likely as joint sets J1 and J2 forming the wedge failure whose line of intersection daylights on the slope face.

Therefore, the slope failure mechanism is anticipated to be governed by the discontinuity orientations at site. It leads to suggest that the slope failure occurred along the road during earthquake do have contribution of the unfavorably oriented discontinuities. The outcome of kinematic analysis is summarized in Table 1.

2.4. Slope mass rating (SMR)

Slope Mass Rating (SMR) was developed by Anbalagan and others (Anbalagan et al., 1992) by intoducing four orientation adjustment factors (i.e. F1, F2, F3 and F4) to Bieniwski's Rock Mass Rating - RMR (Bieniawski, 1989). The purpose was to characterize the rock mass along for slopes and evaluate the slope instability, induced by orientation and distribution of discontinuities, in terms of the probability of failure.

In the present study, SMR was also used to assess the slope stability using the empirical approach in view of rock mass behavior characterized by intact rock strength parameters and discontinuity characteristics.

The discontinuity survey data was used to determine the SMR values for the failed slope. In addition to the basic first five parameters of RMR, orientation adjustment factors (F1, F2, F3 and F4) were used to calculate SMR as given below:

$$\text{SMR} = \boldsymbol{RMR_{basic}} + (\boldsymbol{F_1}.\boldsymbol{F_2}.\boldsymbol{F_3}) + \boldsymbol{F_4} \quad (1)$$

The ratings for uniaxial compressive strength (UCS) of intact rock, rock quality designation (RQD), average joint spacing, condition of joints (including persistence, aperture, roughness, infilling and weathering) and ground water condition were used according to Bieniawski classification (Bieniawski, 1989), while adjustment factors F1, F2 and F3, (related to joint orientation with respect to slope orientation) and F4 (correction factor for excavation method), were used to determine SMR. The summary and calculation of RMR and SMR are provided in Tables 2 and 3 respectively.

The determined SMR value is 12 that designate the exposed rock as “very bad” class.

2.5. Rock mass and seismic parameters

Limit-equilibrium slope stability analysis treats rock as rock mass similar to soil for the identification and calculation of resisting and mobilizing forces along circular slip surface. For the evaluation of underground structures in rock mass and along the rock slopes, rocks are generally treated as Hoek-Brown material

Table. 1. The kinematic analysis results of the discontinuities.

Label		Dip	Dip direction
Slope Face		80	265
Mean Set Planes	Foliation (F)	31	163
	Joint set 1 (J_1)	63	188
	Joint set 1 (J_2)	55	275
		Plunge	**Trend**
Line of Intersection between J_1 and J_2		49	241

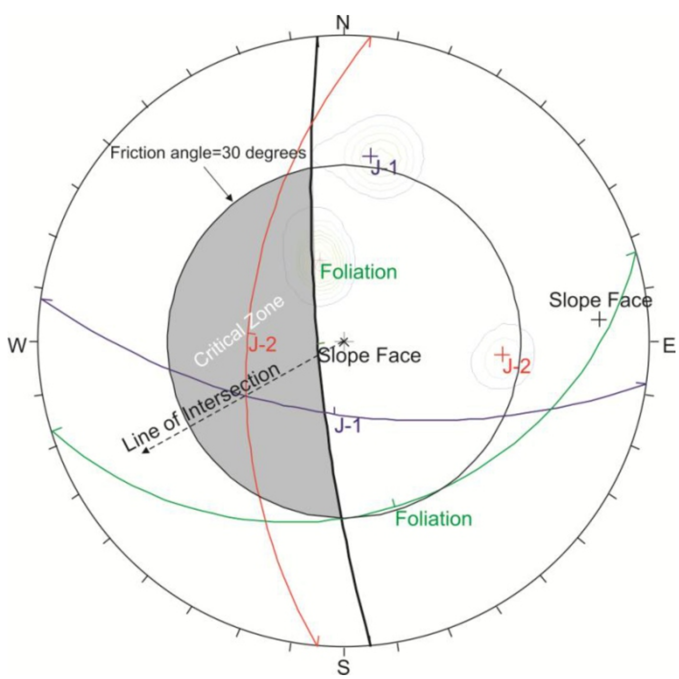


Fig.6. Mean great circles of the three major joint sets along with the slope face.

following Hoek-Brown strength and failure criteria (after Hoek and Brown, 1998). Hoek-Brown model considers rock mass as homogeneous and isotropic material with the inclusion of rock structure parameter, Geological Strength Index (GSI). GSI basically reduces the rock mass strength from that of intact rock strength corresponding to rock structure characteristics. It ranges from 1 (very weak rock structure) to 100 (Intact rock with very few discontinuities). For the estimation of the GSI for the failed slope, following relation was used after Marinos et al., (2005).

$$GSI = RMR - 5 \text{ for } RMR \geq 23 \qquad (2)$$

RMR value of 36 was calculated based on the rock mass data (Table 2). So the GSI value of 31 was obtained. The other parameter in Hoek-Borwn material characterization is material's constant (mi) that has been adopted from RocData 5.0 (Rocscience Inc., 2005) due to unavailability of extensive laboratory testing. RocData 5.0 provides 12 as value of mi for the schists which was adopted for quartz mica schist. Uniaxial compressive strength (σ_{ci}) determined from Schmidt rebound hammer was 29.0 MPa.

Table. 2. Details of Rock Mass Rating (RMR) calculations.

Parameters	Range measured by discontinuity surveys	Average Value	Rating
1 .Uniaxial Compressive Strength of Intact Rock (MPa)	28.00-30.00	29.00	4
2. Rock Quality Designation* (%)	17.65	17.65	3
3. Spacing (cm)	9.13-11.39	10.26	8
4. Persistence (m)	4.2-7.4	5.8	2
5. Aperture (mm)	1.3-2.1	1.7	1
6. Roughness	Slightly Rough	Slightly Rough	3
7. Infilling (mm)	5.5-6.5	>5	0
8. Weathering Conditions	Slightly Weathered	Slightly Weathered	5
9. Ground Water Condition	Damp	Damp	10
Overall RMR Rating (RMR_{basic})	(RMR_{basic} = 1+2+3+4+5+6+7+8+9)		36
*RQD = 115 - (3.3 × J_v) ; J_v= 29.5/m^3 (Palmstrom, 1995)			

Table. 3. Details of Slope Mass Rating (SMR) calculations.

Calculation of Adjustment Factors F_1, F_2, F_3 and F_4			
Details of Orientation			
	Dip	**Dip Direction**	
Joint J_1	63°	188°	
Joint J_2	55°	275°	
Slope Face	80°	265°	
Details of line of intersection of J_1 and J_2			
Trend = 241°			
Plunge = 49°			
Condition 1: Considering the trend of line of intersection and plunge of J_1 and J_2 and the slope			
Adjustment Factor	**Estimated Value**	**Rating**	**Description**
$F_1 = \lvert \alpha_i - \alpha_s \rvert$	24	0.40	Favorable
$F_2 = \lvert \beta_i \rvert$	49	1	Very Unfavorable
$F_3 = \lvert \beta_i - \beta_s \rvert$	-31	-60	Very Unfavorable
F_4 (Method of Excavation)	-	0	By Mechanical Excavation
$[(F_1 . F_2 . F_3) + F_4]$	-	-24	-
Overall SMR Rating	SMR = $RMR_{basic} + (F_1 . F_2 . F_3) + F_4$	12	-
Where α_i = Trend of line of intersection of J_1 and J_2 α_s = Dip direction of slope face β_i = Plunge of line of intersection β_s = Dip of slope face			

These three adopted parameters (i.e. GSI, mi & σ_{ci}) were in put in RocData 5.0 for the derivation of Hoek-Brown rock mass parameters mb (material constant for rock mass) and s (constant for best-fit curve).

For the estimation of external earthquake loading, two ways were adopted. Firstly, the shake map (Fig. 1) of the Hindukush earthquake (USGS, 2015) prepared based on the scale proposed by Worden et al., 2012 was used to locate the study area and accordingly a value of 0.062g as peak ground acceleration (PGA) was picked corresponding to shake intensity. Secondly, the value of PGA was determined using relation proposed by Youngs and coworkers (Youngs et al., 1997).

$$\ln(y) = 0.2418 + 1.414M + C_1 + C_2(10 - M)^3 + C_3 \ln\left(r_{rup} + 1.7818e^{0.554M}\right) + 0.00607H + 0.3846Z_T \quad (3)$$

Where,
y = spectral acceleration in g
M = moment magnitude
H = depth in km

Z_T = source type (0 for interface and 1 for intraslab)
R or r_{rup} = closest distance to rupture in km
C_1, C_2 and C_3 are estimated for least square estimation and C_1 = 0.2418, C_2 = 1.414, C_3 = -2.552

This relation was specifically used for reasons that it is applicable to Hindukush Subduction Zone (designated as intermediate earthquake region) which is shallower than the Japanese subduction zones where various such relations have been developed and updated recently. Also this relation has been used by USGS recently for the seismic hazard mapping in Afghanistan and adjoining areas (Byod et al., 2007). In addition, the Youngs' Intra-slab ground motion equation gives more realistic estimation of seismic loading in the Hindukush and surrounding region.

A value of 0.043g for PGA was determined using equation 3 (after Youngs et al., 1997). By the comparison of both values of PGA, 0.043g was adopted to perform all analyses for pseudo static conditions.

2.6. Limit equilibrium analysis

In Slide 6.0, the geometry of the failed slope was modeled and the rock mass parameters were input to get the factor of safety (FOS). In Slide 6.0, FOS can be calculated following various methods, however, for the present study, Bishop Simplified, Janbu Simplified, Spenser and Corps of Engineers 1 were implemented.

The modeling was targeted to get unit factor of safety (i.e. FOS=1). At this point, the resisting and mobilizing forces are in equilibrium and minor external loading added to mobilizing forces (earthquake induced) may trigger failure. In initial trials of limit-equilibrium analyses, FOS was found greater than 1. However, with the systematic variation of GSI from 31 to 20, the changed set of mb and s were used (Fig. 7).

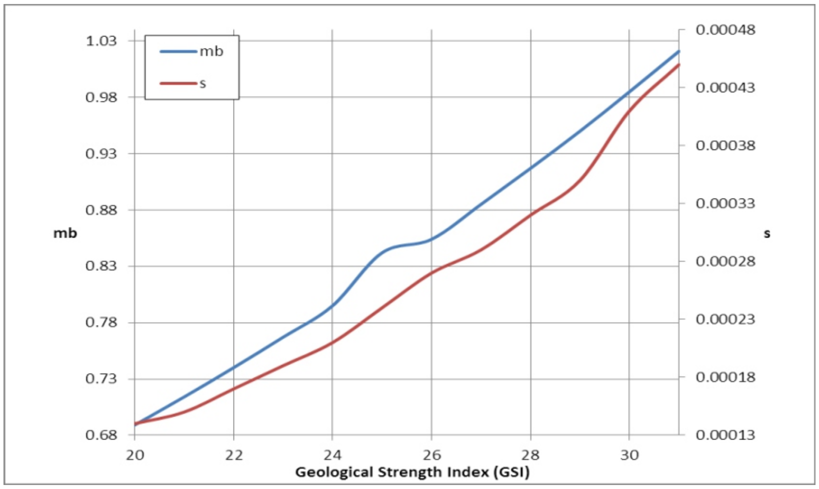


Fig.7. The variation of Hoek-Brown rock mass constants mb and s with GSI estimated by RocData 5.0

Finally after many modeling trials, FOS=1 was obtained corresponding to GSI=24, with the same values of mi & σ_{ci} (Fig. 8 and 9a).

With the same set of rock mass parameters, earthquake loading (0.043) was applied in horizontal direction towards the slope face.

The model was rerun and FOS less than unity (that is <1) was obtained indicating higher shear stresses than shear strength of the material along the slip surface (Table 6). This was exactly the case of slope failure.

Next step was to examine the failure slip surface (Fig. 9b) with that of actual one (Fig. 5d). A comparison of the slid mass was also made between the model and that of actual failed slope.

At this stage the rock mass parameters, that is, values of GSI, mi & σ_{ci} were considered the most probable parameters at the time of slope failure during earthquake that induced earthquake loading corresponding to 0.043g.

Further parametric sensitivity analyses were also undertaken by running various models with the change of parameters to investigate the sensitivity of FOS towards the various parameters

3. Results and discussions

The outcome of kinematic analysis is provided in figure 6. The figure shows that plane failure is likely along joint set J2 which is dipping at moderate angle towards slope face and daylighting on the slope face. The joint dip is less than slope face and greater than angle of friction (assuming 30 degrees). From the figure 9b, it can be interpreted that the lower part of the slip surface is along the J2 near the slope toe, hence, contributing to global failure of the slope.

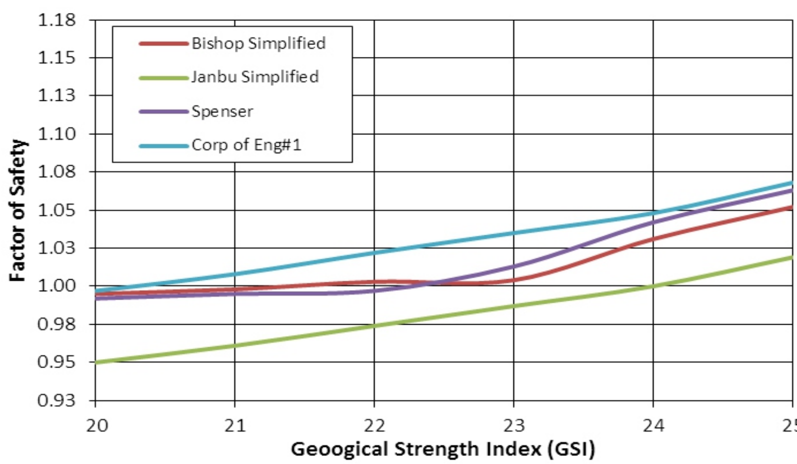


Fig. 8. Sensitivity plot of the factor of safety against the GSI values.

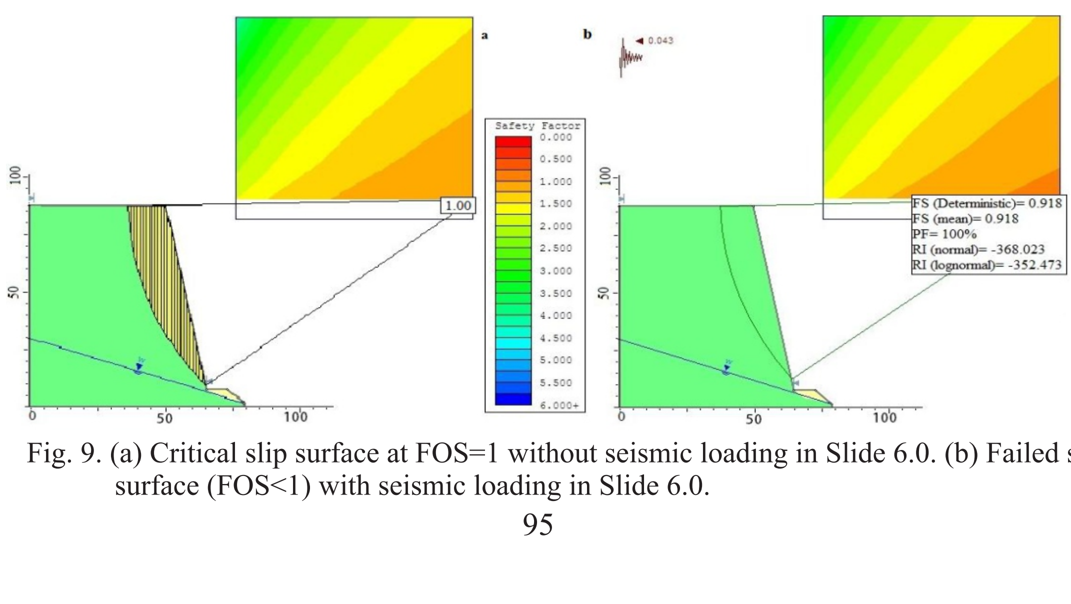


Fig. 9. (a) Critical slip surface at FOS=1 without seismic loading in Slide 6.0. (b) Failed slip surface (FOS<1) with seismic loading in Slide 6.0.

Besides, wedge failure (Fig. 6) has been identified along the intersection line of joints J1 and J2. This wedge failure has also been considered to contribute to the overall failure of the slope.

The description of the rock mass according to SMR rating is given the Table 4. This empirical rating of slope also indicates the likelihood of slope failure as it gives 90% probability of slope failure.

Wardlaw, B.R., Mei, S., 1999. Refined conodont biostratigraphy of the Permian and lowest Triassic of the Salt and Khisor ranges, Pakistan. Proceedings International Conference Pangea and Paleozoic–Mesozoic Transition, Wuhan, 154-156.

From RMR the Hoek-Brown Parameter GSI was determined using relation given in Equation 2. This value of GSI was adopted as guideline for onward picking up the rest of the parameters (mb & s) of Hoek-Brown criteria. However, the parameters corresponding to adopt GSI, did not produce FOS close to unity but bit higher. This scenario indicated bit over-estimation of the GSI through field conditions. Consequently, at systematic variation of GSI from 31 to 20, FOS of 1.0 was determined in Janbu simplified method corresponding to GSI=24 (i.e. mb = 0.795 and s = 0.00021). The details of results are mentioned in Table 5 below.

At this point horizontal earthquake loading corresponding to PGA=0.043g was applied in Slide 6.0 to fail the critical slip surface in simulated earthquake loading conditions and to estimate the slip mass volume. The model has shown the slope failure along a semi-circular slip surface. The depth of the slip surface was measured and the failure mechanism was examined in comparison to actual slope failure. Both the failures and the slid mass quantities were found in agreement. This model has almost reproduced the field slope failure mechanism.

By taking the adopted parameters as guidelines, further modelling was undertaken for the sensitivity of FOS against uniaxial compressive strength (σci). A graph of FOS against the variation of UCS has been shown in figure 10. This figure shows that FOS of the slope increases with the increase of σci of quartz mica schist.

In summary, it was understood from this analysis that estimation of rock mass parameters can be measured reasonably realistic by back analysis. However, the field observations are a very good mean to approximate the first guess of these parameters to be adopted for the back analysis.

Volume of slid mass estimated from Bishop simplified, Janbu simplified and Spencer methods are also in agreement with observed values given in Table 7.

Table 4. Classification of Rock mass on the basis of SMR

Rock mass class	V (Very bad)
Stability	Completely unstable
Failures	Big planar or soil like or circular
Probability of Failure	0.9

Table 5. Details of the back analysis without any seismic loading computed with Slide for Hoek-Brown Criteria

Analysis & Surface				
Number of Slices	**Tolerance**	**Maximum Number of iterations**		**Surface Type**
25	0.005	50		Circular
Global Minimums (Without Seismic Load in Sensitivity Analysis)				
Method	**Factor of Safety**	**Horizontal Moment (kN.m)**		**Total Slice**
		Resisting	**Driving**	**Area (m^2)**
Bishop Simplified	1.032	2.47888e+006	2.4023e+006	821.569
		Horizontal Forces (kN)		
Janbu Simplified	1.000	9254.61	9253.12	944.763
Spencer	1.042	9847.16	947.65	944.763
Corp of Engineer 1	1.048	9487.5	9487.87	944.763

Table 6. Details of the back analysis after applying seismic loading computed with Slide for Hoek-Brown Criteria.

Global Minimums (With Seismic Load in Both Probability & Sensitivity Analysis)				
Seismic Load Coefficient (Horizontal) = 0.043				
Method	**Factor of Safety**	**Horizontal moment (kN.m)**		**Total Slice Area (m^2)**
		Resisting	**Driving**	
Bishop Simplified	0.991	2.4108e+006	2.43263e+006	821.569
		Horizontal forces (kN)		
Janbu Simplified	0.918	7352.88	8011.18	821.569m^2
Spencer	0.988	8225.54	8326.9	821.569 m^2
Corp of Eng#1	0.970	9463.05	9755.4	944.763 m^2

Table 7. A comparison of the mass eroded by the observed values and computed slip surface areas.

Observed value	25 m × 35 m × 50 m	Volume eroded = 43750 m^3
Method	**slice area (m^2)**	**Volume of the slip surfaces= slice area × slide width (50 m)**
Bishop simplified method	821.569	41078.45 m^3
Janbu Simplified	821.569	41078.45 m^3
Spencer	821.569	41078.45 m^3
Corp of Engineer1	944.763	47238.15 m^3

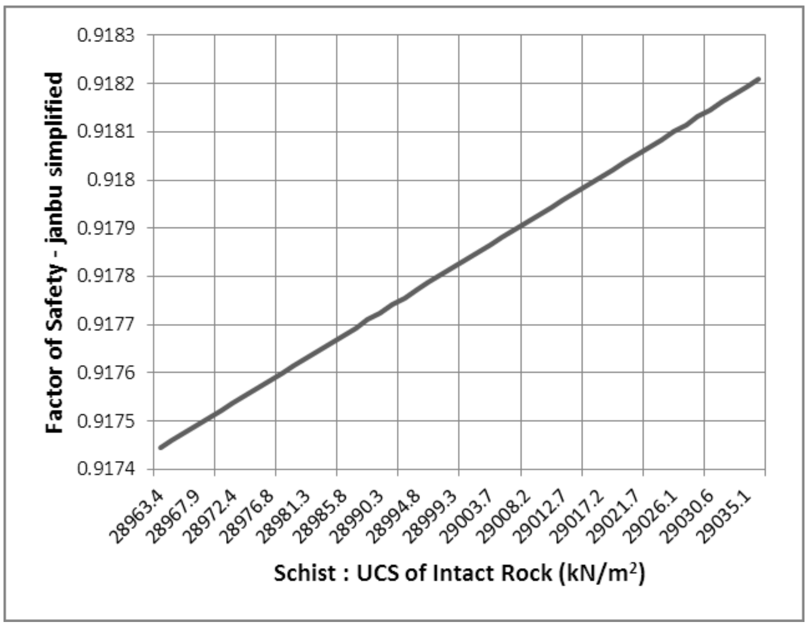


Fig. 9. Factor of Safety v/s UCS of intact rock after failure by applying seismic loading.

4. Conclusions and recommendations

Back analysis of 80m high slope that failed in 26th October earthquake was undertaken. The field studies as well as numerical modelling were conducted to arrive at suitable rock mass parameters that rock slope possessed at the time of failure. The estimated parameters are σ_{ci} = 29.0 MPa, GSI= 24, γ = 25.48 kN/m^3 and m_i =12.

These parameters were found within the estimation of the field observations and measurements.

Earthquake loading based on the USGS (2015) and Youngs et al., (1997) provided the likely value of PGA in the project area at the time of earthquake and slope failure. The analyses with these values of PGA reproduced the slope failure of October 26th earthquake by providing similar failure mechanism, depth of slip surface and quantity of slid mass. Based on this study, following conclusion and recommendation are made;

- The back analysis is very good tool to estimate the rock mass parameters at the failure.
- Together with the rock mass parameters, importance of interaction of the orientations

of the discontinuities and slope face (kinematic analysis) cannot be ruled out.

- Slope stability evaluation using SMR can be helpful for the initial stage of the analysis. The probability of failure calculated by SMR is 90% whereas the FOS calculated by the numerical model is also in the range of 0.9, which indicate a general agreement between the limit equilibrium and the empirical classification methods at least for this project site.

- In view of the output of this back analysis study, all the slopes having similar geometries and geological conditions should be examined for future earthquakes and suitable remedial measures should be adopted to make the slopes stable for the future earthquakes.

- At the unstable and potentially unstable slopes, warning signs should be provided for the safety of the traffic.

- Regular monitoring of the potentially unstable slope and record of the slope failures will help design suitable rock cuts in similar rock conditions and viable preventive measures for stabilization.

Authors' contribution

Aisha Noor involved in field work, data analyses and drafted the manuscript. Sohail Akram supervised the filed work, provided guidance through out the research work and also critical reviewed the manuscript. Luqman worked in field, data analyses, write up and literature review. Waqas Sarwar was involved in field work and preparation of figures.

References

Ahmed, H., Cheema, M.R., Fatmi, A.N., Iqbal, M.W.A., Raza, H.A., Raza, S.M., Shah, S.M.I., 1997. Stratigraphy of Pakistan, Quetta.

Anbalagan, R., Shamra, S., Raghuvanshi, T.K., 1992. Rock Mass Stability Evaluation using modified SMR approach, 6th National Symposium on Rock Mechanics, Proceedings.

Bieniawski, Z.T., 1989. Engineering rock mass classifications, Wiley, New York.

Brown, E.T., 1981. Rock Characterization Testing & Monitoring ISRM Suggested Methods, Pergamon Press, Oxford, NewYork, Toronto, Sydney, Paris, Frankfurt.

Byod, O.S., Charles, S.M., Kenneth, S.R., 2007. Preliminary earthquake Hazard Mapping of Afghanistan, open file Report, 2007-1137, USGS.

Calkins, J.A., Offield, T.W., Abdullah, S.K.N., Ali, S.T., 1969. Geology and Mineral Resources of Southern Hazara District, West Pakistan and part of Western Azad Kashmir, U. S Geological Survey Project Report (IR) PK-43.

Google Earth 2015, Google Earth, viewed 15 February 2016, < HYPERLINK "https://www.google.com/earth/" https://www.google.com/earth/ >.

Hoek, E., Bray, J., 1981. Rock Slope Engineering, 3rd ed, Institute of Mining and Metallurgy, London, UK.

Hoek, E., Brown, E.T., 1998. Practical estimates of rock mass strength, Rock Mechanics and Mining Science, 34, 1165-1186.

Marinos, V., Marinos, P., Hoek, E., 2005. The geological strength index: applications and limitations, Bulletin of Engineering Geology and Enviornment, 64, 55-65.

Miller, R.P., 1965. Engineering Classification and Index properties for Intact Rock, Ph.D. thesis, University of Illinois.

Palmstrom, A., 1995. RMi-A rock mass characterization system for rock engineering purposes, Ph.D. thesis, University of Oslo, Norway.

Rocscience Inc. 2005. RocData Version 5.0 - Rock, Soil and Discontinuity Strength Analysis, www.rocscience.com, Toronto, Ontario, Canada.

Rocscience Inc. 2010. Slide Version 6.0 - 2D Limit Equilibrium Slope Stability Analysis, www.rocscience.com, Toronto, Ontario, Canada.

Rocscience Inc. 2012. Dips Version 6.0 - Graphical and Statistical Analysis of Orientation Data', www.rocscience.com, Toronto, Ontario, Canada.

Treloar, P.J., Broughton, R.D., Williams, M.P., Coward, M.P., Windley, B.F., 1989a. Deformation, metamorphism and imbrication of the Indian Plate south of the Main Mantle Thrust, North Pakistan, Journal of Metamorphic Geology, 7, 111-125.

Turner, A.K., Schuster, R.L., 1996. Landslides: Investigation and Mitigation, Transportation Research Board Special Report 247, United States.

USGS 2015. Earthquake Hazards Program, viewed 15 February 2016, < HYPERLINK "http://earthquake.usgs.gov/earthquakes/eqarchives/poster/2015/20151026.php%20%5bAccessed%2015%20Feburary%202016%5d."http://earthquake.usgs.gov/earthquakes/ eqarchives/poster/2015/20151026.php [Accessed 15 February 2016]. >.

USGS 2015. Earthquake Hazards Program, viewed 15 February 2016, < HYPERLINK "http://earthquake.usgs.gov/earthquakes/eventpage/us10003re5" \l "shakemap" http://earthquake.usgs.gov/earthquakes/eventpage/us10003re5#shakemap >.

Wang, L., Hwang, J.H., Luo, Z., Juang, C.H., Xiao, J., 2013. Probabilistic back analysis of Slope failure - A case study in Taiwan, Computers and Geotechnics, 51, 12-23.

Wikipedia 26 October 2015. October 2015 Hindu Kush earthquake, viewed 15 February 2016, < HYPERLINK "https://en.wikipedia.org/wiki/October_2015_Hindu_Kush_earthquake" https://en.wikipedia.org/wiki/October_2015_Hindu_Kush_earthquake >.

Worden, C.B., Wald, D. J., Lin. K., Garcia D., 2012. A Revised ground motion and intensity interpolation scheme for Shake Map, Bulletin of Seismic Society America, 100, 3083-3096.

Youngs, R.R., Choiu, S.J., Silva, W.J., Humphrey, J.R., 1997. Strong Gorund Motion attenuation Relationships for subduction zone earthquakes, Siesmological Research Letters, 68 (1), 58-73.