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Structural Controls on Rock Slope Instability in the Bhotekoshi Corridor, Sindhupalchowk, Nepal, Following the 2015 Gorkha Earthquake

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Abstract: This study critically examines the post-earthquake stability of a mountain slope in the Bhotekoshi Hydropower area, Sindhupalchowk District, one of the regions most affected by the 2015 Gorkha Earthquake of Mw 7.8. The weak geological formations of the area and steep terrain have repeatedly triggered slope failures, damaging highways, hydropower structures, and settlements along the Araniko Highway connecting Kathmandu to Tibet. Using Slope Mass Rating (SMR) and Stereographic Projection techniques, the study identifies key discontinuities, notably at 235°/67°, that control slope instability. The obtained RMR (27) and SMR (37.8) classify the slope as Class IV (Bad), denoting high instability and potential for large-scale planar and wedge failures. Stereographic analysis indicates that 57% and 28% of the area are critically prone to planar and wedge failures, respectively. These findings reveal significant structural weaknesses and emphasize the urgent need for systematic stabilization and continuous monitoring to safeguard hydropower infrastructure and communities in this seismically and geologically fragile region.

Keywords: ***Mountain slopes, Post-earthquake, Slope stability analysis, Slope stabilization, The 2015 Gorkha Earthquake.***

Introduction

Nepal Himalaya lies in a tectonically active region, characterized by fragile mountain and hill formations prone to slope instability. Landslides are common, causing loss of life, property, and infrastructure. This problem is particularly severe in Sindhupalchowk District, which was heavily affected by the 2015 Gorkha Earthquake, with most aftershocks concentrated in the district and nearby areas (Table 1). Roadside slopes along strategic routes, including the 115 km Araniko Highway connecting Kathmandu to Tibet, are highly vulnerable to traffic disruptions from slope failures triggered by steep terrain, weak geology, heavy rainfall, river floods, and seismic activity (Jha, 2014). The region's complex topography, combined with anthropogenic activities such as road haphazard construction and land use changes, further exacerbates slope instability and increases the risk of catastrophic failures.

Table 1, Major landslides in Nepal (DMG, 2018)

Year	Place	Loss
1967	Budhigandaki, Gorkha	9 died
1968	Budhigandaki, Gorkha	One bridge and 24 houses
1970	Tinau, Rupandehi	90 people lost
1971	Palakhu, Rasuwa	5 died
1976	Baglung	7 died and 3 bridges swept away
1982	Balefi, Sindhupalchowk	97 dead; many houses destroyed
1985	Trishuli, Rasuwa	-
1986	Tadi, Nuwakot	31 died; 24 houses and 3 bridge destroyed
1987	Sunkoshi, Sipa	98 people died; 229 houses, hydropower and highway destroyed
1988	Myagdi Khola, Myagdi	109 people died; 94 houses destroyed
1989	Aarukhola, Bajhang	16 people died; many houses destroyed
1996	Larcha, Sindhupalchowk	54 people lost; many houses destroyed
2010	Madikhola, Kaski	Fertile land and houses swept away
2014	Jure VDC, Sunkoshi	River blocked, claimed 156 lives, displaced hundreds, and caused extensive infrastructure and ecosystem damage.
2015	Mid Bhotekoshi and Upper Chaku	Damage Hydropower plants

The objective of this study is to assess the stability of mountain slopes affected by the Mw 7.8 2015 Gorkha Earthquake in Nepal, based on existing landslide inventories. The Bhotekoshi Hydropower area is selected as a case study due to its high landslide susceptibility and strategic importance for hydropower infrastructure. The study focuses on three specific objectives: (1) review of existing landslide inventory in the study area, (2) slope stability analysis using Slope Mass Rating (SMR), and (3) slope stability assessment through Stereographic Projection techniques.

The findings are expected to contribute to improved landslide hazard management and mitigation strategies in earthquake-affected mountainous regions. Figure 1 shows the Bhotekoshi Hydropower area, illustrating different types of landslide activity: (a) rockfall, (b) debris flow, and (c) debris flow occurring along the highway.

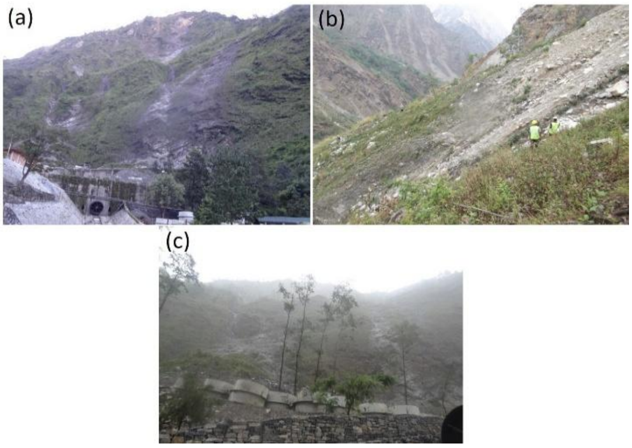

Figure 1, Bhotekoshi Hydropower area showing (a) rockfall, (b) debris slide, and (c) debris flow along the highway

Slope instability analysis

Slope stability was assessed using Rock Mass Rating (RMR) and Slope Mass Rating (SMR).

The RMR system, proposed by Bieniawski (1989) for tunneling and mining, has been extended to slope design. Bieniawski (1989) revised the ratings for various parameters. RMR classifies rock masses based on five field-determined parameters: (1) uniaxial compressive strength of intact rock, (2) Rock Quality Designation (RQD), (3) spacing of discontinuities, (4) condition of discontinuities, and (5) groundwater conditions. A sixth parameter, orientation of discontinuities, aids in assessing slope-specific stability issues.

The SMR modifies the RMR to account for slope-specific conditions, particularly discontinuity orientation relative to the slope face (Romana, 1991). It provides a more realistic assessment of slope stability in rock masses by incorporating geometric and kinematic factors. Key considerations include:

- Overall rock mass characterization (joint frequency, condition, water inflow)
- Difference between slope face strike and joint strike
- Joint dip versus slope dip, influencing potential planar or wedge failures
- Joint dip relative to normal joint friction on discontinuity surfaces

Stereographic projection

Stereographic projection is a graphical method for representing three-dimensional orientation data on a two-dimensional plane (Dahal, 2006). It is widely used in slope stability analysis and rock mass characterization to evaluate relationships between lines and planes. Projections are performed manually using a stereonet (Wulff net) or via computer, allowing visualization of discontinuity orientation and assessment of potential failure mechanisms.

Location and study area

The study area is situated in the Upper Bhotekoshi Valley within a run-of-the-river hydropower project on the Bhote Koshi River, a major tributary of the Sun Koshi River, in Sindhupalchowk District, central Nepal. It is located approximately 110 km northeast of Kathmandu, close to the Nepal–China border. The project includes a headworks with a side intake constructed about 500 m downstream of the confluence of the Bhote Koshi and Jung Khola near Tatopani. The powerhouse is located at Jhirpu Village, approximately 3.7 km downstream from the diversion dam. The project area is readily accessible via the Araniko Highway, which connects Kathmandu with the Nepal–China border at Kodari. The predominant rock types exposed in the study area include phyllite, schist, gneiss, quartzite, and minor metamorphic rocks.

Geological and geomorphological setting

Sherpa communities of Tibeto-Burmese origin. The valley experiences a monsoon-dominated climate, receiving approximately 2,500–3,000 mm of annual rainfall, of which 71–92% falls between June and September. Combined with elevations reaching 4,000 m asl and rugged topography with steep local relief, these conditions make the area highly susceptible to weathering, erosion, and slope instability. Geologically, the valley is underlain by steeply dipping phyllite, schist, gneiss, limestone, and quartzite, overlain by highly weathered colluvial and alluvial deposits. In the Himalaya, phyllite is considered the most landslide-prone lithology, followed by shale, schist, poorly cemented sandstone, limestone, gneiss, granite, and quartzite. Deeply weathered regolith is characterized by extensive gully erosion, while structurally controlled rockfalls commonly occur from steep phyllite cliffs. Large, slow-moving deep-seated translational landslides are widespread, rotational failures develop mainly where weathered schist is undercut, and channelized debris flows frequently occur along deeply incised tributaries. Based on the lithological classification (Table 2), the study area belongs to Group III, comprising slates, phyllites, and schists interbedded with quartzite and gneiss, indicating a moderate potential for lithology-controlled slope failure.

Nevertheless, road excavation and slope cutting have further increased the susceptibility of the area to landslides.

Table 2, Slide potential of rocks of Nepalese mountain (Krahenbunl and Wagner, 1983)

Group	Rock type of Nepal	Lithological slide potential
I	Slate, phyllite and schist, closely interbedded respected with calc-slate, calc Schist, limestone, dolomite and dolomite quartz	Very High (LCPS 16)
II	Slates, phyllites and schists	High (LCPS 10)
III	Slates, phyllites and schists closely interbedded respect with quarzite and gneiss	Medium (LCPS 5-10)
IV	Gneiss	Medium to Low(LCPS 1-5)
V	Quartzite	Low (LCPS 1)
VI	Massive limestone and dolomite	Very Low (LCPS 0-1)

Rock types

Slates, phyllites, and schists interbedded with quartzite and gneiss are the dominant rock types in the study area. The bedrock is composed predominantly of weak, highly weathered phyllite, although comparatively stronger schist is exposed in some locations. Phyllite is a foliated metamorphic rock formed through the progressive metamorphism of slate, in which very fine-grained white mica develops a preferred orientation. It is typically gray to light greenish-gray in color and exhibits well-developed foliation. Most of the exposed rock mass is moderately to highly weathered, resulting in reduced strength and increased susceptibility to failure.

The combination of steep natural slope angles, road-cut excavations, adverse discontinuity orientations, and advanced weathering constitutes the principal factors controlling slope instability in the study area. These conditions significantly reduce the stability of the rock mass and increase the likelihood of structurally controlled failures, particularly along foliation and joint planes.

Materials and methods

Initially, a desk study was conducted to plan the workflow. Relevant literature, including reports, books, and journals, was reviewed, followed by collection of secondary data from government agencies, primarily under the Ministry of Environment. Field visits were then conducted to gather primary data. Collected data were analyzed using standard slope stability approaches to achieve the study objectives. Finally, results were documented, disseminated, and compiled into a finalized report with regular reviews and updates.

Relevant reports, articles, and publications were collected from a variety of sources, including the Central Library of Tribhuvan University, Kathmandu University Library, and the Himalayan Conservation Group. Additionally, information was gathered from the Ministry of Environment, Science and Technology website, as well as from other relevant publications to ensure comprehensive coverage of the subject.

Field visits were conducted for direct observation and measurement using a Schmidt hammer, GPS, and Brunton compass. The following information were gathered: 1) the number of measurement stations (four); 2) criteria for choosing stations (ease of access, representation of exposure along the highway corridor, various rock types); 3) the tools employed (Schmidt hammer, Brunton compass, GPS); 4) procedure followed.

Collected data were analyzed for Rock Mass Rating (RMR), Slope Mass Rating (SMR), Stereographic Projection, and landslide mapping to evaluate slope stability.

Slope Mass Rating

Field visits and measurements of discontinuities are the main methods for finding the SMR. For this Bieniawski geomechanics classification of Rock Mass Rating (RMR) system has been used to find M. R. Romana's Slope Mass Rating (SMR). The basic five components of RMR i.e. uniaxial compressive strength of the intact rock, rock quality designation (RQD), spacing of discontinuities, conditions of discontinuities and ground water conditions. The rating was obtained by summing the values assigned for the first five components. Then, overall rating was made by a final adjustment by consideration of sixth parameter (orientation of discontinuities) depending upon the intended project type i.e. slope. Thus, the RMR was determined using following relationship:

$$RMR = \sum R \qquad (1)$$

Definition of RMR = R, where R is the summation of all the rating values for the five classification criteria (R1-R5) according to Bieniawski (1989).

Uniaxial compressive strength (UCS) of the intact rock was calculated by using Schmidt hammer test. Schmidt hammer was used to find Schmidt hardness of the rock for the calculating UCS of the intact rock.

Rock Quality Designation (RQD) was estimated using the joint data. Palmstrom (1982) suggested that number of discontinuities per unit volume helps to estimate value of RQD. Following formula given was used for the calculation:

$$RQD=115-3.3J_v \qquad (2)$$

Where, J_v is the sum of the number of joints (total joints seen) per unit volume and known as volumetric joint count.

Spacing of the discontinuities, condition of discontinuities, ground water condition and orientation of discontinuity was observed on site and estimated using Table described by Bieniawski (1989).

Joint dip angle, joint dip amount, slope dip angle and slope dip amount were measured with the help of Brunton compass for SMR system. Finally, SMR was calculated by the relationship defined by Romana (1991) as follows:

$$SMR=RMR+(F_1*F_2*F_3)+F_4 \qquad (3)$$

Where, RMR is Rock Mass Rating as obtained before.

F1 depends on parallelism between joints and slope face strikes. Its range is from 1.00 to 0.15 (Table 2). Romana gave these values empirically but he also proposed following relationship.

$$F_1=(1-SinA)^2 \qquad (4)$$

Where A is the angle between the strike of the slope face and strike of discontinuities face.

F2 represents joint dip angle in the planar mode of failure. In a sense it is a measure of the probability of joint shear strength. Its value ranges from 1.00 to 0.15

F3 gives the relationship between the slope face and joint dip. In a planner mode of failure F3 refers to the probability that joints “daylight” in the slope face. Condition fair when slope face and joints are parallel. When the slope dips more than joints, very unfavorable conditions occur.

F4 is the adjustment factor for the method of excavation. Finally rated SMR values are classified as describes below:

Table 3, SMR classes defined (Romana, 1991)

Class	SMR	Description	Stability	Failures	Support
I	81-100	Very good	Completely stable	None	None
II	61-80	Good	Stable	Some Blocks	Occasional
III	41-60	Normal	Partially stable	Some joints or many wedges	Systematic
IV	21-40	Bad	Unstable	Planner or Big	Importance/ Corrective
V	0-20	Very bad	Completely Unstable	Big planner or soil like	Re-excavation

Stereographic projection

For slope stability analysis, the dip and dip direction of the natural slope surface and discontinuity planes were measured in the field using a Brunton compass. The collected structural data were analyzed using Stereonet software to evaluate the kinematic feasibility of slope failures and identify the potential failure mechanisms (Dahal, 2006).

Planar failure is considered possible when a discontinuity plane dips in the same general direction as the slope, with a strike difference of 20° or less, a dip angle lower than the slope angle, and a dip angle greater than the internal friction angle of the discontinuity.

Toppling failure is considered feasible when the discontinuity plane dips in the direction opposite to the slope face, the strike difference between the discontinuity and the slope is 20° or less, and the discontinuity dip exceeds the internal friction angle.

Wedge failure occurs when two intersecting discontinuity planes form a wedge whose line of intersection trends in the same general direction as the slope face.

Kinematic conditions for wedge failure are satisfied when the plunge of the line of intersection is less than the slope dip but greater than the internal friction angle, and the trend of the intersection line lies within approximately 20° of the slope dip direction.

Result and discussions

The data collected for RMR at different points above Bhotekoshi Hydropower project site are given below:

Station 1:

General description of rock = Hard schist

Number of joints = 20

Spacing between joints = 10 cm

Rock state = fresh/damp

Schmidt hammer value = 55

Station 2:

General description of rock = Weak phyllite

Number of joints = 25

Spacing between joints = 10-15 cm

Rock state = weathered/damp

Schmidt hammer value = 20

Station 3:

General description of rock = Weak phyllite rock

Number of joints = 22

Spacing between joints = 10-15 cm

Rock state = weathered/wet

Schmidt hammer value = 20

Station 4:

General description of rock = Weak phyllite rock

Number of joints = 8

Spacing between joints = 20 cm

Rock state = weathered/damp

Schmidt hammer value = 25

From above collected data from different location in the study area, RMR is calculated as per the standard chart, i.e. 27.

RMR for Location 1:

Rating for uniaxial compressive strength of axial load (R1) =12 (Using Schmidt hammer hardness and density of rock to find out Uniaxial Compressive Strength)

Rating for Rock quality designation (R2) = 8

Rating for Spacing between discontinuities (R3) = 8

Rating for Condition of discontinuities (R4) = 10

Rating for Groundwater Conditions (R5) = 20

Total RMR without orientation,

RMR=12+8+8+10+20 = 58

RMR with orientation for Slope (taking fair condition),

RMR= 58-25 = 33

RMR for Location 2:

Rating for uniaxial compressive strength of axial load (R1) = 4 (Using Schmidt hammer hardness and density of rock to find out Uniaxial Compressive Strength)

Rating for Rock quality designation (R2) = 8

Rating for Spacing between discontinuities (R3) = 8

Rating for Condition of discontinuities (R4) = 10

Rating for Groundwater Conditions (R5) = 20

Total RMR without orientation,

RMR=4+8+8+20+10 = 50

RMR with orientation for Slope (taking fair condition),

RMR= 50-25 = 25

RMR for Location 3:

Rating for uniaxial compressive strength of axial load (R1) =4 (Using Schmidt hammer hardness and density of rock to find out Uniaxial Compressive Strength)

Rating for Rock quality designation (R2) = 8

Rating for Spacing between discontinuities (R3) = 8

Rating for Condition of discontinuities (R4) = 7

Rating for Groundwater Conditions (R5) = 20

Total RMR without orientation,

RMR=4+8+8+20+7 = 47

RMR with orientation for Slope (taking fair condition),

RMR= 47-25 = 22

RMR for Location 4:

Rating for uniaxial compressive strength of axial load (R1) =4 (Using Schmidt hammer hardness and density of rock to find out Uniaxial Compressive Strength)

Rating for Rock quality designation (R2) = 17

Rating for Spacing between discontinuities (R3) = 10

Rating for Condition of discontinuities (R4) = 10

Rating for Groundwater Conditions (R5) = 10

Total RMR without orientation,

RMR=4+17+8+10+10 = 51

RMR with orientation for Slope (taking fair condition),

RMR= 51-25 = 26

Then, RMR = 27

The major discontinuities of slope in four different locations of the study area are given below:

Table 4, Major discontinuities of slope

No.	Dip Direction (Azimuth)	Dip Amount
1	304°	71°
2	235°	67°
3	231°	80°
4	230°	76°

The main discontinuity among them is 235°/67° and these joint sets could play main role in plane as well as wedge failures around the area.

Calculation of SMR

RMR from the calculation as per standard chart = 27

F1 = 0.7 (20°-10°)

F2 = 1 (dipping more than 45°)

F3 = -6 (difference is 0°-10°)

F4 = 15 (Natural Slope)

Then, SMR = 27 + (0.7 x 1 x (-6)) + 15 = 37.8

The calculated value lies on IV class (Bad), unstable having planner or big wedges failure of SMR and needs to be corrective measures in the slope.

Stereographic projection

Using 'Stereonet 8' graphical interpretation of slope stability is done. As shown in Figure 2 and 3, there is only presence of some plane failure and wedge failure according to the conditions for those failures.

The diagrams are based on stereographic projections created using Stereonet 8 in order to study the kinematic stability of slopes in the highway area near Bhotekoshi Hydropower Project site. Based on the plot of the orientation of the slope plane and the orientation of the major joint sets found at the site, the stereonets help us understand the potential failure mechanisms. The geometric relation between the great circles of the slope and the joints demonstrates the structural control of the stability, and shows that there are kinematically favorable conditions for plane and wedge failures.

Discussion

The rock slope investigated along the Bhotekoshi Hydropower Project corridor exhibits unfavorable stability conditions based on both the Rock Mass Rating (RMR) and Slope Mass Rating (SMR) classifications. The calculated SMR value of 37.8 places the slope within the Bad (Class IV) category, indicating an unstable rock mass that is susceptible to structurally controlled failures.

According to the SMR classification proposed by Romana (1991), slopes in this category require appropriate stabilization and corrective measures before they can safely support engineering structures. The relatively low rock mass quality, combined with adverse discontinuity orientations, significantly increases the likelihood of instability.

The findings are consistent with those reported by Kafle (2010) for the Opi Landslide, which recorded an SMR value of 34, also falling within the Bad (Class IV) category. That study documented the occurrence of planar and large wedge failures, demonstrating that slopes with similar engineering geological characteristics are highly vulnerable to structurally controlled failure.

The Bhotekoshi Hydropower site shares many of the same geological and geomorphological attributes as the Opi Landslide, including steeply inclined slopes, weathered phyllite interbedded with gneiss and other metasedimentary rocks, a weak rock mass derived from clay-rich parent materials, and predominantly concave slope morphology. These similarities provide a valuable regional benchmark supporting the present interpretation that the investigated slope is inherently unstable.

Stereographic projection further confirmed the dominant failure mechanisms within the study area. Planar failure was identified in Figures 2(a), 2(c), 2(d), and 3(b), while wedge failure was recognized in Figures 2(b) and 3(c). Figure 3(a) showed no kinematically feasible failure. Overall, four of the seven stereographic analyses (57%) indicate planar failure, whereas two analyses (28%) indicate wedge failure. These results demonstrate that planar failure is the predominant instability mechanism, although wedge failure also represents a significant hazard in specific slope sections.

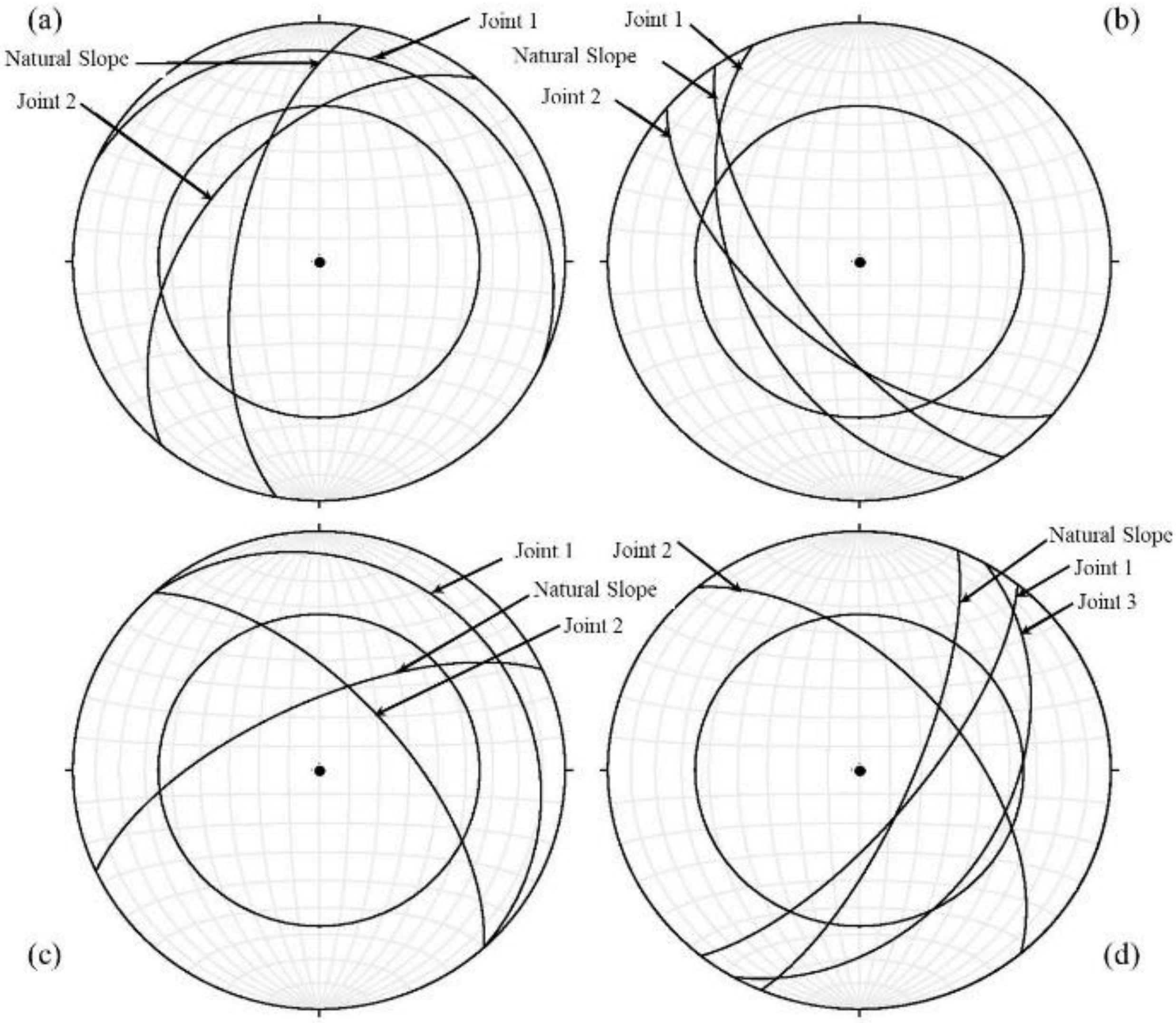

Figure 2, Stereographic projections of slope at Highway near Bhotekoshi Hydropower site: (a), (b), (c) and (d)

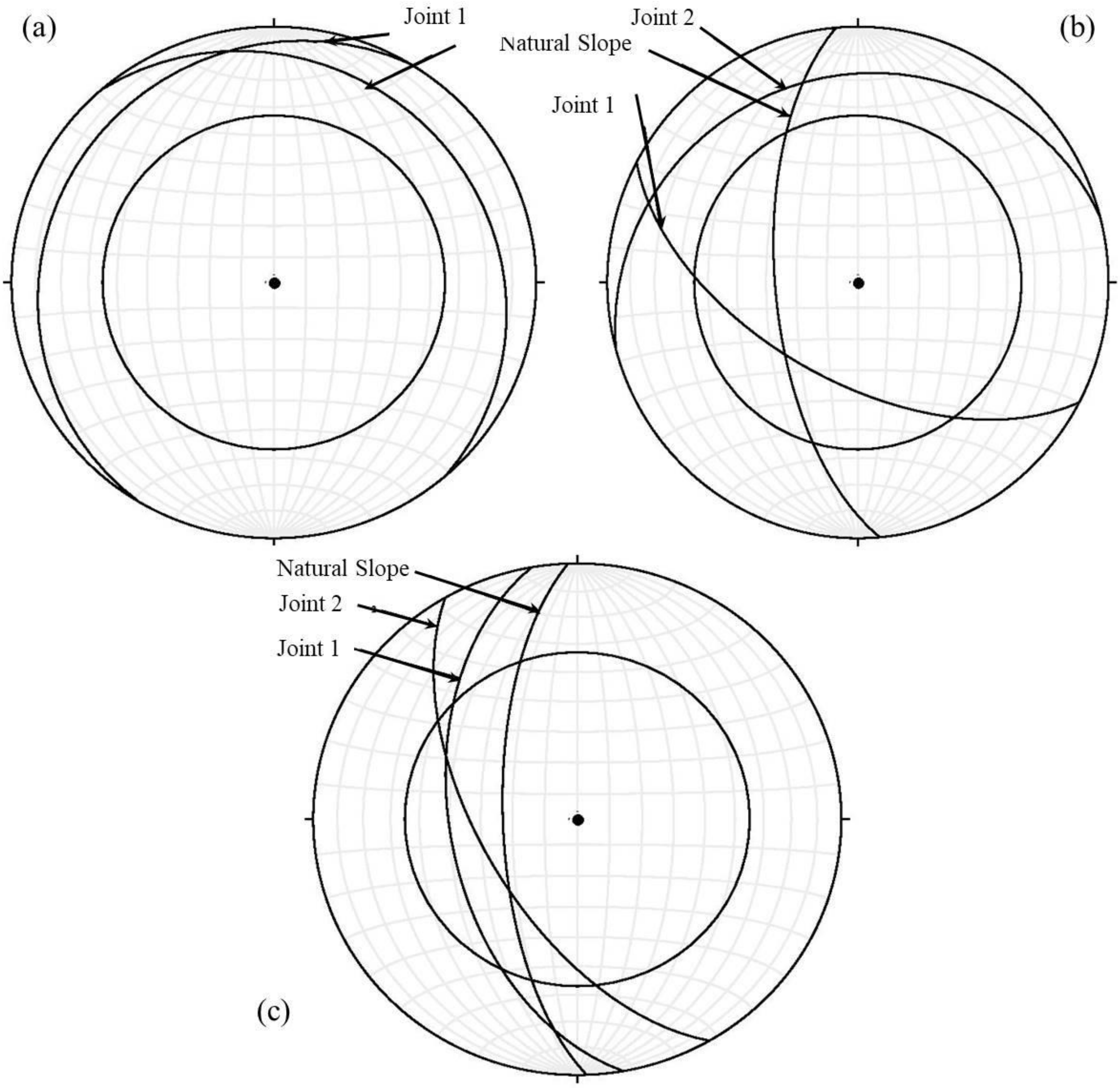

Figure 3, Stereographic projections of slope at Highway near Bhotekoshi Hydropower site: (a), (b) and (c)

The engineering implications of these findings are substantial. The calculated SMR value of 37.8 clearly indicates the need for stabilization measures, as recommended by Romana (1991).

The dominant discontinuity set (235°/67°) exhibits the greatest kinematic potential for sliding, making it the principal structural control on slope instability. Furthermore, the weathered phyllitic rock mass possesses reduced shear strength and durability, thereby increasing its susceptibility to failure under rainfall, seismic loading, and excavation activities.

Consequently, stabilization measures such as rock bolting, drainage improvement, retaining structures, rockfall protection systems, and systematic slope monitoring should be incorporated into the engineering design.

Quantitative stability analyses, including limit equilibrium and numerical modeling, are also recommended to complement the empirical classifications and provide a more comprehensive assessment of long-term slope performance.

Conclusion and recommendation

The exclusive conclusions drawn from this study, together with the major recommendations based on the engineering geological assessment and slope stability analysis, are presented in the following sections. These conclusions summarize the key findings of the investigation, while the recommendations provide practical guidance for future slope management, hazard mitigation, and infrastructure planning within the study area.

Conclusion

The rock slope stability assessment conducted in the Bhotekoshi Hydropower Project area using the Rock Mass Rating (RMR), Slope Mass Rating (SMR), and stereographic projection methods indicates that the investigated slopes are generally unstable and susceptible to structurally controlled failures. The RMR classification reveals that the rock mass quality ranges from fair to poor, reflecting the influence of highly fractured rock masses, persistent discontinuities, and unfavorable geological conditions.

The calculated SMR values classify the majority of the investigated slopes as Class IV according to Romana (1991), representing an unstable slope condition with a high likelihood of failure unless appropriate stabilization measures are implemented. Kinematic analysis based on stereographic projection further demonstrates that planar failure is the dominant failure mechanism, occurring in approximately 57% of the analyzed slope sections where discontinuity orientations are favorably aligned with the slope face.

Wedge failure is identified as the second most significant failure mode, accounting for about 28% of the investigated locations due to the intersection of two or more discontinuity sets. The remaining slope sections are considered relatively stable with respect to the analyzed failure mechanisms.

Overall, the integrated application of RMR, SMR, and stereographic analysis provides a comprehensive evaluation of rock slope stability and highlights the critical role of geological structures in controlling instability within the Bhotekoshi Hydropower Project area.

Recommendation

Slope sections classified as SMR Class IV should be given high priority for continuous monitoring and detailed engineering geological investigation because they represent unstable rock masses with a significant potential for failure. Regular field inspections and periodic reassessment are recommended, particularly after intense rainfall or seismic events.

Rock slopes where planar and wedge failure mechanisms have been identified through stereographic projection should be evaluated individually using detailed structural mapping and site-specific stability analyses to better understand their failure potential and support appropriate mitigation planning.

Where kinematic analyses indicate the possibility of instability, suitable stabilization and protection measures, including rockfall barriers, retaining walls, rock bolting, shotcrete, drainage systems, and other engineered slope protection techniques, should be implemented based on detailed geotechnical investigations and sound engineering design principles.

Comprehensive geological, geotechnical, hydrogeological, and structural investigations should be carried out before the planning and construction of roads, hydropower projects, tunnels, and other infrastructure to ensure safe and sustainable development in the study area.

Future studies should incorporate quantitative slope stability analyses, including limit equilibrium and numerical modeling approaches, to complement the empirical stability classifications presented in this study and provide more reliable estimates of slope performance under both static and seismic loading conditions.

Article Note

This article is a full-length paper developed from **Extended Abstract #212**, entitled "**Stability of Mountain Slopes Affected by the 2015 Gorkha Earthquake: A Case Study of Bhotekoshi Hydropower, Sindhupalchowk, Nepal**", which was presented at the 15th Asian Regional Conference of the International Association for Engineering Geology and the Environment (ARC-15 of IAEG), held in Kathmandu, Nepal, from 27 to 29 November 2025.

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