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A hybrid approach to road-cut slope stability assessment: integrating empirical and numerical modelling

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Abstract

Landslides along Himalayan Road corridors pose persistent threats to infrastructure due to steep terrain, intensely jointed rock masses, and continuous road-widening activities. This study develops an integrated assessment framework to evaluate the stability of three representative road-cut slopes in Mussoorie, India, through combined field investigations, empirical rock mass classifications, and numerical analysis. Kinematic assessment and RMR–SMR–GSI classifications were performed to characterise failure mechanisms and rock mass conditions. Subsequently, stability was quantified using Limit Equilibrium Analysis (LEA) and Finite Element Analysis (FEA) with the Generalised Hoek–Brown and Barton–Bandis models. LEA results indicate marginal stability with $FOS \approx 1.0$, whereas joint-controlled FEA shows a 20–40% decrease in strength reduction factor ($SRF \approx 0.60–0.80$), consistent with observed field deformation patterns. The study demonstrates that integrating discontinuity stiffness significantly improves reliability over single-method approaches, providing a realistic basis for mechanism-specific stabilisation of Himalayan road-cut slopes.

Keywords Road-cut slope, Landslide, Finite element analysis, Generalised Hoek Brown, Rock mass rating (RMR), Slope mass rating (SMR), Geological strength index (GSI), Limit equilibrium analysis

1 Introduction

Landslides are characterised by the downslope movement of surface materials in forms such as topple, slide or fall [11, 12]. In mountainous terrain like the Himalayas, alongside natural triggers such as intense rainfall and seismic loading, anthropogenic activities such as slope cutting for road widening have emerged as major destabilising factors [45]. With rising population pressures and the concomitant need for new transportation corridors or expansion of existing infrastructure, the indiscriminate excavation of slopes without thorough geotechnical investigation exacerbates instability [45]. Such unplanned road-cut slope failures can lead to significant infrastructure loss [13]. It is therefore imperative to conduct detailed evaluations of geotechnical and geological



conditions before expansion activities and to implement appropriate mitigation measures to minimise landslide risk [34].

A wide range of approaches has been adopted for slope stability evaluation in the Himalayas, broadly encompassing empirical classifications, kinematic analysis, and numerical simulations. Rock mass classification systems such as the Rock Mass Rating (RMR), Slope Mass Rating (SMR), and Geological Strength Index (GSI) remain widely used due to their simplicity and ability to provide rapid field-based stability estimates [7, 23, 40]. However, their semi-quantitative nature and dependence on subjective field observations often limit their precision, particularly in highly jointed and heterogeneous rock masses characteristic of Himalayan slopes [38, 48]. Kinematic analysis, based on stereographic projections of joint orientations, has been employed extensively to identify potential failure mechanisms such as planar, wedge, and toppling failures [22, 44]. Although effective for detecting structurally controlled instabilities, it does not provide factor of safety (FOS) values or capture stress–strain behaviour, necessitating complementary analyses [30].

To address these limitations, numerical modelling techniques such as Limit Equilibrium and Finite Element Methods have gained prominence for their capability to incorporate stress redistribution, non-linear material behaviour, and progressive failure through the shear strength reduction (SSR) technique [14, 17]. In Himalayan rock masses, investigations demonstrate that although the Mohr–Coulomb model is still widely adopted, the Generalised Hoek–Brown criterion offers a more realistic representation of fractured, jointed materials [1, 18, 38]. More recent studies reinforce this shift toward advanced constitutive modelling: recent work has shown that finite-element and limit-equilibrium techniques capture the complexity of structurally controlled slopes in the Himalayas more effectively [27]; investigations into jointed Himalayan rock masses further highlight that deformation behaviour is better represented by discontinuity-sensitive formulations based on the Hoek–Brown failure criterion [33]; subsequent research demonstrates enhanced predictive performance when integrating the GHB model with SSR-based numerical modelling for rocky slopes [10, 46]; and field-numerical studies of road-cut slopes in Himachal Pradesh reaffirm that non-linear models gain traction under the region's complex geological settings [31].

Despite extensive research in the region, only a limited number of studies integrate empirical classifications, kinematic assessment, and fully non-linear numerical modelling within a single unified framework. A notable gap also persists in quantifying the influence of discontinuity-related parameters, particularly normal and shear stiffness, on stability, even though these parameters govern deformation behaviour in jointed rock masses. This limitation is especially pronounced in the Lesser Himalayas, where complex geology, intense rainfall, and rapid anthropogenic modifications require rigorous, multi-method approaches to realistically capture slope response under varying environmental and structural conditions.

The primary objective of this study is to develop an integrated, multi-method stability assessment for three representative road-cut slopes in Mussoorie by combining detailed field measurements with empirical rock mass classifications (RMR, SMR, and GSI), kinematic analysis, and advanced LEA–FEA modelling. Incorporating the Generalised Hoek–Brown criterion allows the analysis to capture both global factor of safety trends and stress–strain-driven failure mechanisms. The novelty of the work lies in (i)

evaluating slope behaviour with and without discontinuities using a combined Generalised Hoek–Brown and Barton–Bandis representation of rock mass strength, (ii) quantifying the influence of joint normal and shear stiffness on stability, and (iii) cross-validating empirical, analytical, and numerical results under identical geological conditions. Applied to the Mussoorie corridor in Uttarakhand, this framework provides a robust, multi-criteria evaluation of slope stability that overcomes the limitations of single-method approaches. It also offers a replicable methodology for assessing geotechnical risks along infrastructure corridors in similarly complex and geo-dynamically sensitive Himalayan terrains.

2 Study area details

Mussoorie, located in the Lesser Himalayan region of Uttarakhand, India, is highly susceptible to landslides due to its steep terrain, intense monsoonal rainfall, and tectonically active setting near the Main Boundary Thrust. The town falls under Seismic Zone IV [9], indicating high seismic vulnerability, particularly for its residents and the substantial seasonal tourist population. The area is geologically characterised by the Krol, Blaini, Jaunsar, and Tal Formations, with the study corridor predominantly consisting of thickly bedded, highly jointed dolomitic limestone of the Krol Formation [5, 51]. These jointed lithologies, intersected by multiple fault systems, significantly reduce rock mass integrity and contribute to recurrent instabilities.

The study area lies within toposheet 53 J/3 of the Survey of India, at coordinates 78.035705°E and 30.482769°N. It is predominantly underlain by the Krol Formation, characterised by thickly bedded dolomitic limestone. The focus of this investigation is a 150 m wide corridor encompassing three representative road-cut slopes along NH-707 A in Mussoorie. The selected corridor along NH-707 A is of major strategic importance, as it serves as the primary access route to Kempty Fall and other key urban and tourist zones. Continuous road widening and blasting activities have modified the slope geometry, often without adequate geotechnical evaluation, leading to repeated rockfalls and localised failures that threaten transportation safety and infrastructure. Despite such high risk, slope stability studies in Mussoorie remain limited, particularly those incorporating advanced numerical deformation analysis and the influence of joint stiffness parameters on road-cut rock slopes. The geological framework of the area, derived from Geological Survey of India data and processed using Quantum Geographic Information System (QGIS), is illustrated in Fig. 1, while the three critical slopes examined in this study are depicted in Fig. 2.

3 Analysis methodology

The evaluation of slope stability in this study employs a comprehensive and multi-method approach designed to capture both field-based conditions and advanced numerical responses. The methodology integrates empirical rock mass classification schemes, kinematic analysis, and numerical simulations, ensuring a robust and cross-validated assessment of the three selected road-cut slopes. The sequential framework adopted for the investigation is illustrated in Fig. 3, which presents the workflow of the analysis.

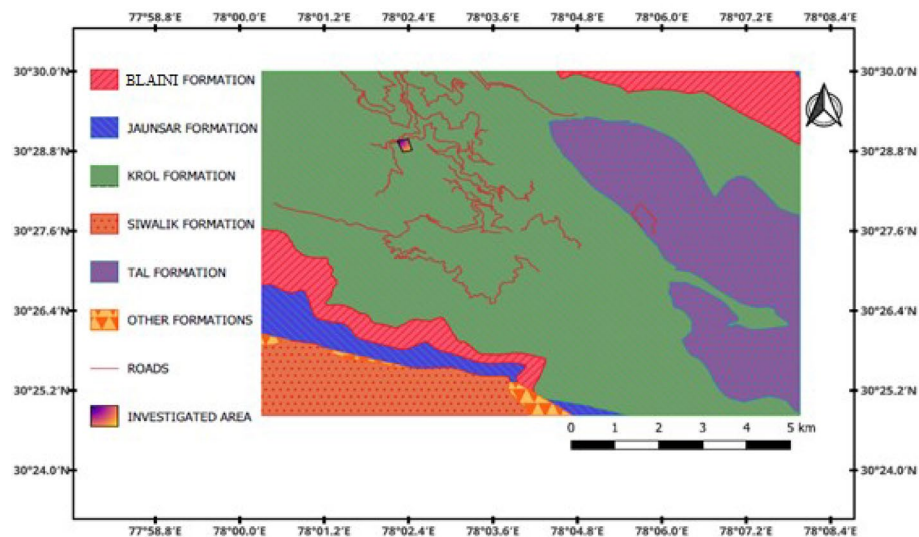


Fig. 1 The geological setup of the area, modified after Rupke [41]

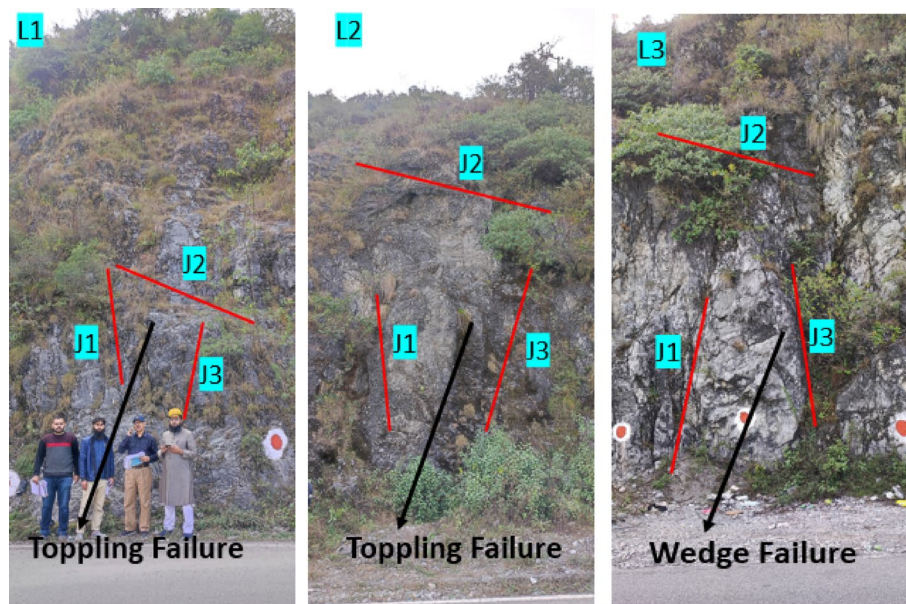


Fig. 2 Road-cut slope locations L1, L2, and L3 depicting the type of failure

3.1 Field visit

A detailed geological and geotechnical survey was performed at three representative road-cut slopes in Mussoorie. Slope geometry, including height and inclination, was measured using a laser rangefinder and a Brunton compass. Discontinuity orientations were documented at multiple stations along each slope face to capture spatial variability, and key structural features such as persistence, aperture, roughness, weathering condition, and infilling characteristics were recorded following the recommended procedures [26]. Schmidt hammer rebound tests (20 readings per location) were carried out on fresh, dry surfaces to estimate the Uniaxial Compressive Strength (UCS) using an established correlation [53]. Joint friction angles were obtained through tilt testing on representative blocks and verified against field observations of surface roughness. These

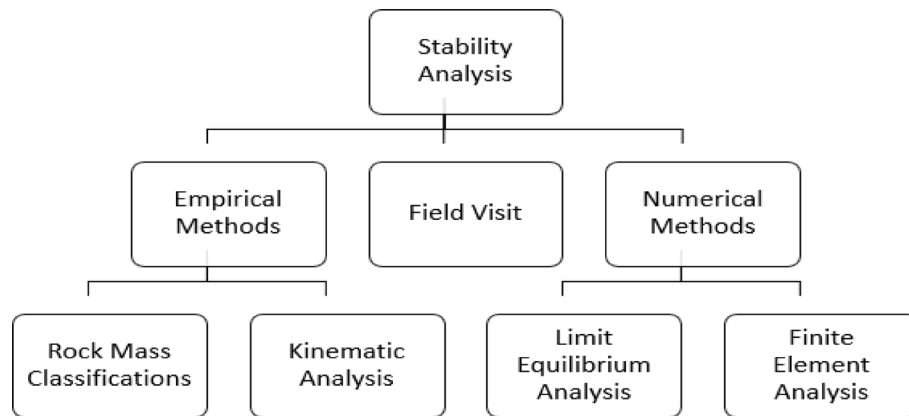


Fig. 3 Flow chart representing various techniques used for stability analysis

measured parameters formed the input basis for empirical classification and numerical model calibration.

3.2 Empirical methods

Empirical slope stability assessment commonly relies on rock mass classification systems along with kinematic analysis to evaluate potential instability in jointed rock slopes. In this study, three widely used empirical systems, Rock Mass Rating (RMR), Slope Mass Rating (SMR), and Geological Strength Index (GSI), were applied in conjunction with kinematic evaluation to develop a comprehensive understanding of the in-situ stability conditions. These methods incorporate parameters such as lithology, uniaxial compressive strength, joint spacing and persistence, surface condition, groundwater influence, and excavation orientation, all of which exert a strong control on the mechanical behaviour of the rock mass and its susceptibility to failure.

3.2.1 Slope kinematics assessment

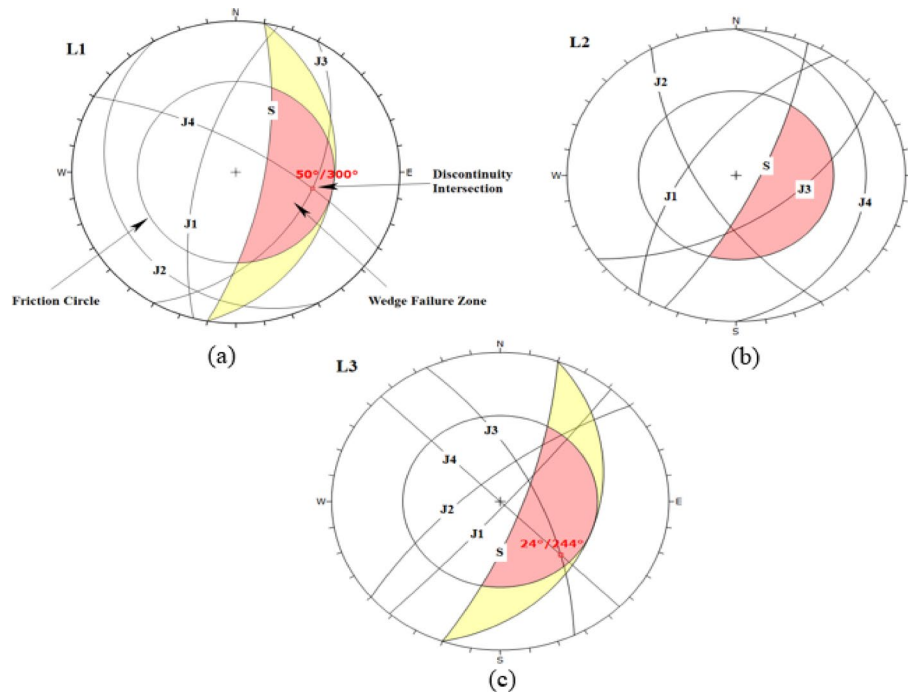
Kinematic analysis is a fundamental approach for determining the likelihood of structurally controlled failures in rock slopes. It assesses the geometric relationships between slope faces and discontinuity orientations to predict the occurrence of planar, wedge, or toppling failures [44, 52]. In the present study, discontinuity orientations and slope face attitudes were systematically measured in the field using a Brunton compass. Joint friction angles were obtained through tilt tests conducted on representative blocks collected from the slope faces. The collected data were analysed using Rocscience DIPS software, where stereographic projections were generated to examine discontinuity intersections and their position relative to friction cones and slope-face limits. These projections enabled the identification of the dominant structural failure mechanisms at each slope and guided the subsequent numerical modelling scenarios. The orientation data used in this assessment are summarised in Table 1, and the generated stereonets for the three locations are presented in Fig. 4.

3.2.2 Geomechanical characterisation of rock masses

A detailed characterisation of the rock masses was conducted to quantify the mechanical condition of the slopes and to generate reliable input parameters for both empirical and numerical analyses. Three widely recognised classification approaches, Rock Mass

Table 1 Kinematic analysis data obtained from field survey

Location	Rock	Joints friction angle	Slope (dip/dip direction)	Joints (dip/dip direction)			
				J1	J2	J3	J4
L1	Limestone	28°	70°/100°	65°/285°	15°/240°	40°/120°	70°/30°
L2	Limestone	30°	75°/115°	80°/305°	70°/240°	55°/145°	15°/90°
L3	Limestone	30°	75°/110°	86°/131°	72°/320°	66°/64°	90°/225°

**Fig. 4** Stereographic projections of the three locations: **a** L1, **b** L2, and **c** L3, showing slope orientations, joint set orientations, and failure zones highlighted in red

Rating (RMR), Slope Mass Rating (SMR), and Geological Strength Index (GSI), were applied to translate field observations into engineering-relevant measures of strength and stability.

The RMR system was used as the foundational index, integrating UCS, rock quality designation (RQD), joint spacing, discontinuity surface condition, and groundwater influence [6]. To minimise subjectivity and enhance data consistency, the updated rating charts were adopted, enabling direct graphical selection of parameter values [32]. UCS was derived indirectly through 20 Schmidt hammer rebound readings at each site using the established correlation [53], as expressed in Eq. 1.

$$UCS (MPa) = 0.0028H_r^{2.584} \quad (1)$$

In which, H_r Is the average Schmidt hammer rebound value.

RQD is evaluated using the joint count methodology [37] using Eq. 2.

$$RQD = 115 - 3.3J_V \quad (2)$$

In which, J_V is the number of joints in a unit rock mass volume.

The remaining parameters, including discontinuity condition (persistence, roughness, aperture, surface characteristics, and weathering condition) and groundwater

characteristics, were assigned based on field surveys and visual observation, as summarised in Table 2. The field surveys involved a systematic set of in-situ measurements, including (i) geological mapping of lithology and structural features, (ii) measurement of joint orientations using a Brunton compass, (iii) Schmidt hammer rebound tests (20 readings per site) for indirect estimation of UCS, (iv) tilt tests to determine joint friction angles and (v) persistence, aperture, roughness, weathering condition, and infilling characteristics were recorded following the recommended procedures [26].

The RMR method does not explicitly account for the critical influence of discontinuity orientation on rock slope stability. To address this limitation, the Slope Mass Rating (SMR) system was employed, which was developed to incorporate slope–joint orientation relationships [40]. This approach incorporates four additional adjustment factors F_1 , F_2 , F_3 , and F_4 (Eq. 3) that specifically account for slope and discontinuity orientation within the rock mass, as well as the excavation method. By integrating these parameters, the SMR system offers a more comprehensive assessment of slope condition (Eq. 3). The parameters utilised in this classification are illustrated in Table 2.

$$SMR = RMR_{Basic} \pm (F_1 \times F_2 \times F_3) + F_4 \quad (3)$$

For heavily jointed and blocky rock masses, the RMR and SMR classification systems often fall short in accurately representing the true condition of the slope. To overcome this limitation, the Geological Strength Index (GSI) was introduced as a field-based method for characterising both massive and highly fractured rock masses, enabling a more realistic estimation of rock mass properties [20]. The rock mass structure is categorised into four groups, ranging from blocky with three orthogonal joint sets to

Table 2 RMR and SMR data for different road-cut slope locations

Parameter	L1	L2	L3
UCS (MPa)	35	37	31
Rating	4	4	3.5
RQD (%)	52	62	59
Rating	10.5	12.5	12
Mean discontinuity spacing (m)	0.32	0.38	0.46
Rating	10	11	12
Persistence (m)	13	14	12
Rating	1	1	1
Separation (mm)	3	0.5	4
Rating	1	4	1
Roughness	Rough	Slight Rough	Slight Rough
Rating	5	3	3
Infilling (mm)	3	None	< 5 mm
Rating	2	6	4
Weathering	Moderate weathered	Moderate Weathered	Moderate Weathered
Rating	3	3	3
Groundwater condition	Dry	Dry	Damp
Rating	15	15	10
RMR value	51.5	59.5	49.5
F1	0.85	0.70	0.15
F2	1.0	1.0	0.4
F3	−25	−25	−60
F4	8	8	8
SMR Value	38.25	50	45.1

crushed rock with poor interlocking, while the surface condition ranges from rough, unweathered, and interlocked to slicken side with clayey coatings or thicker clay filling. The advantage of GSI is that it solely evaluates the strength of rock mass through field observation and does not depend on laboratory test results, making it less time-consuming and economical than other methods. Also, the value of GSI is used in numerical modelling when the GHB model is used as a material model for performing the analysis.

The conventional GSI chart provides a broad range of values for a given rock mass condition. However, numerical modelling requires a single representative value to adequately characterise the ground behaviour. This limitation can be effectively addressed by adopting the refined estimation procedure developed for GSI selection [49]. This approach incorporates two key parameters: Structure Rating (SR) and Surface Condition Rating (SCR), which jointly determine the GSI value. The *SR* parameter hinges on the rock mass's blockiness (as per Eq. 4), whereas the *SCR* factor considers surface attributes such as roughness, weathering, and the presence of infilling materials (as per Eq. 5). The conclusive GSI values for the three investigated slopes are shown in Table 3.

$$SR = -17 \ln J_v + 79.8 \quad (4)$$

$$SCR = R_r + R_w + R_f. \quad (5)$$

In which, J_v denotes the volumetric joint count, R_r is the roughness factor, R_w is the weathering factor, and R_f is the infilling factor.

3.3 Numerical modelling

Numerical modelling was employed to quantify the factor of safety, stress–strain response, and failure development in the jointed rock slopes, and to assess the influence of discontinuities on overall stability. Two complementary approaches were adopted: Limit Equilibrium Analysis (LEA) to establish baseline global stability conditions, and Finite Element Analysis (FEA) to capture progressive deformation and structurally controlled instability. All modelling parameters were derived from field measurements and rock mass characterisation results, ensuring alignment between site conditions and simulation inputs.

3.3.1 Limit equilibrium analysis

The Limit Equilibrium Analysis (LEA) is among the most widely adopted approaches for stability assessment of the slope, primarily due to its straightforward concept and ease of implementation. In this approach, a potential slip surface is assumed, and the slope is discretised into multiple slices (typically around 30) to compute the factor of safety by satisfying equilibrium conditions of forces, moments, or a combination of both. Earlier

Table 3 GSI values for different locations

Parameter	L1	L2	L3
J_v	19	16	17
R_r	5	3	3
R_w	3	3	3
R_f	2	6	4
SR	30	33	32
SCR	10	12	10
GSI	40	45	42

simplified formulations focused on limited equilibrium conditions [8, 28], whereas later advances accounted for both interslice normal and shear forces, thereby improving solution accuracy [35, 50]. The most general form, known as the General Limit Equilibrium (GLE) method, was developed to simultaneously satisfy both force and moment equilibrium for any assumed interslice force function [16]. The GLE formulation is based on two safety factor equations, F_m (Moment Equilibrium) and F_f (Force Equilibrium), which provide a range of interslice shear-normal force conditions.

In this study, Limit Equilibrium Analysis (LEA) was performed in Slide 2D (Rocscience) using the Generalised Limit Equilibrium (GLE) formulation, ensuring simultaneous satisfaction of both force and moment equilibrium for improved accuracy. To realistically capture the mechanical behaviour of the jointed dolomitic limestone, the Generalized Hoek–Brown (GHB) failure criterion was adopted, as it more accurately represents non-linear rock mass strength compared to the simplified linear Mohr–Coulomb approach. The governing form of the GHB criterion applied in the analysis is presented in Eq. 6:

$$\sigma_1 = \sigma_3 + \sigma_{ci} \left(m_b \frac{\sigma_3}{\sigma_{ci}} + s \right)^a \quad (6)$$

In which, σ_1 and σ_3 Represent major and minor principal stresses, respectively, while σ_{ci} is the uniaxial compressive strength of the rock mass. m_b , s and a (Eqs. 7, 8 and 9) are material parameters that can be determined using the following equations:

$$m_b = m_i \exp \left(\frac{GSI - 100}{28 - 14D} \right) \quad (7)$$

$$s = \exp \left(\frac{GSI - 100}{9 - 3D} \right) \quad (8)$$

$$a = \frac{1}{2} + \frac{1}{6} e^{GSI/15} - e^{-20/3} \quad (9)$$

The geometrical configuration of the model across three locations is illustrated in Fig. 5. Material constants for GHB were computed using UCS and GSI values derived from field and empirical classifications. To avoid bias in slip surface geometry, potential circular and non-circular failure surfaces were determined automatically via a global search algorithm. Model geometry was based on field-measured slope height and inclination, and unit weight values were derived from published ranges for the Krol limestone strengthened through local observations. The initial values of the Hoek–Brown material constant m_i were obtained from published reference tables for intact limestone, the representative lithology of the study slopes [24]. The disturbance factor D was assigned based on the degree of blasting, stress relief, and construction activity, following recommended guidelines for rock masses affected by excavation [25]. The specific values of these parameters across various locations are shown in Table 4.

3.3.2 Finite element analysis

Numerical modelling has become a vital tool in geotechnical engineering and is widely employed to simulate complex slope stability problems. Rock mass behaviour can be represented through three primary numerical approaches: continuous, discontinuous,

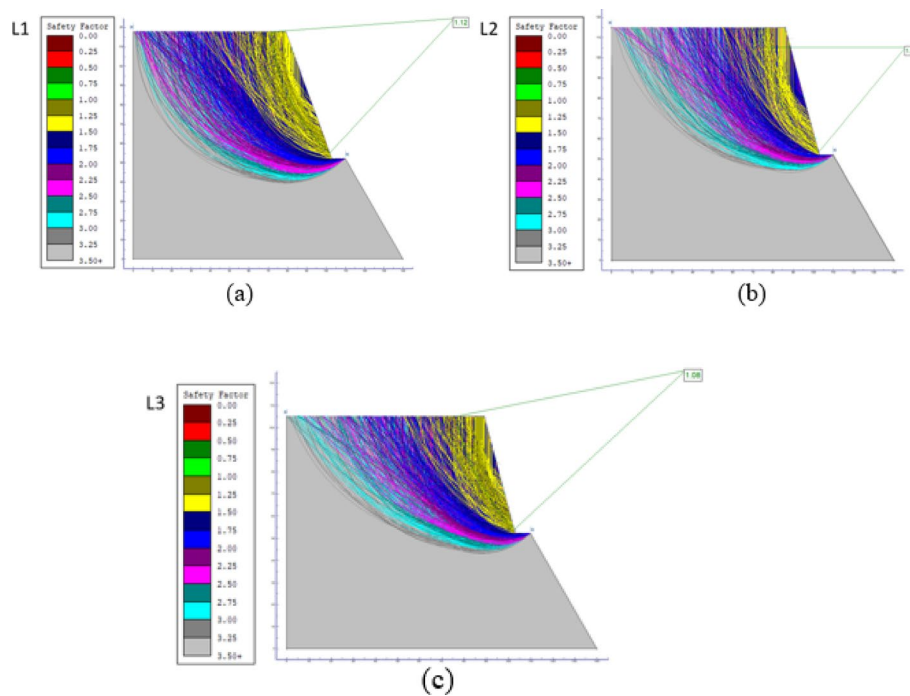


Fig. 5 Geometry of the three locations L1, L2, L3 showing different safety factor contours in Slide 2D

Table 4 Parameters used for limit equilibrium analysis in Slide 2D

Parameters	L1	L2	L3
Height (m)	66	63	53
Inclination (°)	70	75	75
Unit weight (KN/m ³)	27	27	27
UCS (MPa)	35	37	31
GSI	40	45	42
mi	8	8	8
mb	0.296	0.390	0.330
s	0.0001673	0.0003453	0.0002236
a	0.5114	0.5081	0.5099
D	0.7	0.7	0.7

and hybrid methods, each offering different capabilities in capturing the mechanical response of jointed rock masses [29]. For slope stability assessment, continuum-based approaches are commonly preferred as they treat the slope as a single deformable medium and allow efficient computation. This category includes finite element, finite difference, finite volume, boundary element, and mesh-free methods [54]. Among these, the Finite Element Analysis (FEA) is extensively used due to its ability to quantify stress redistribution, deformation patterns, and progressive failure without requiring any pre-defined slip surface [17].

In FEA-based stability evaluation, failure is considered to occur when mobilised shear stresses exceed the available shear strength of the material, and the resulting stability is expressed in terms of the FOS. The Shear Strength Reduction (SSR) technique is widely adopted for this purpose, wherein shear strength parameters are incrementally reduced until numerical non-convergence or a well-defined failure mechanism emerges. SSR provides a realistic representation of slope behaviour, enabling deformation assessment

and explicit consideration of discontinuity effects, capabilities that are largely absent in conventional LEA.

In jointed rock masses, discontinuities exert a significant influence on slope stability. Their shear strength and morphological characteristics govern rock mass behaviour and must be integrated into the numerical model. The Barton–Bandis joint shear strength criterion is well-suited for such conditions and incorporates five key parameters: Joint Compressive Strength (JCS), Joint Roughness Coefficient (JRC), residual friction angle, joint normal stiffness (K_n), and joint shear stiffness (K_s) [2].

JCS was estimated from Schmidt hammer rebound values using the established hardness correlation chart [15]. JRC was derived by comparing joint surface morphology with standard roughness profiles [3]. Residual friction angles were determined through tilt tests performed on representative joint samples. The joint stiffness values were evaluated using empirical relationships developed for rock discontinuities under varying stress conditions [4].

The empirical relationship governing normal stiffness (K_n) is illustrated in Eq. 10, as follows:

$$K_n = \frac{E_i E_{rm}}{L (E_i - E_{rm})} \quad (10)$$

where, E_i is the intact rock mass modulus, E_{rm} is rock mass modulus, and L is the mean spacing of the joint. The value of E_{rm} was estimated using the empirical correlation proposed for rock masses of different qualities [21], as given in Eq. 11.

$$E_{rm} = 100,000 \frac{1 - D/2}{1 + e^{(60+15D-GSI)/11}} \quad (11)$$

The empirical relationship for Shear stiffness (K_s) is (Eq. 12):

$$K_s = \frac{G_i G_m}{L (G_i - G_m)} \quad (12)$$

Where, G_m is the shear modulus of rock mass, and G_i is the shear modulus of intact rock. Due to the challenges associated with conducting in situ direct shear tests, an approximate relationship is adopted in which the normal stiffness value is reduced by a factor of ten to obtain shear stiffness for jointed rock masses [36]. Furthermore, the estimation of Poisson's ratio (Eq. 13) is accomplished through a best-fit regression of recommended values originating from the GSI classification system [23].

$$\nu_{rm} = 0.32 - 0.0015GSI \quad (13)$$

Finite Element Analysis (FEA) of the three road-cut slopes was carried out using RS2 (Rocscience) [39], in which the model domain was discretised with six-node triangular elements to ensure enhanced accuracy in capturing stress concentrations near the slope face and discontinuity zones. The boundary conditions included full constraint of the base in both horizontal and vertical directions, while the lateral boundaries were restrained against horizontal movement but allowed vertical displacement to permit natural deformation of the model; the slope face was left unconstrained to simulate free movement. Two analysis scenarios were evaluated to investigate the role of structural features on slope performance: first, an intact rock mass without any structural planes,

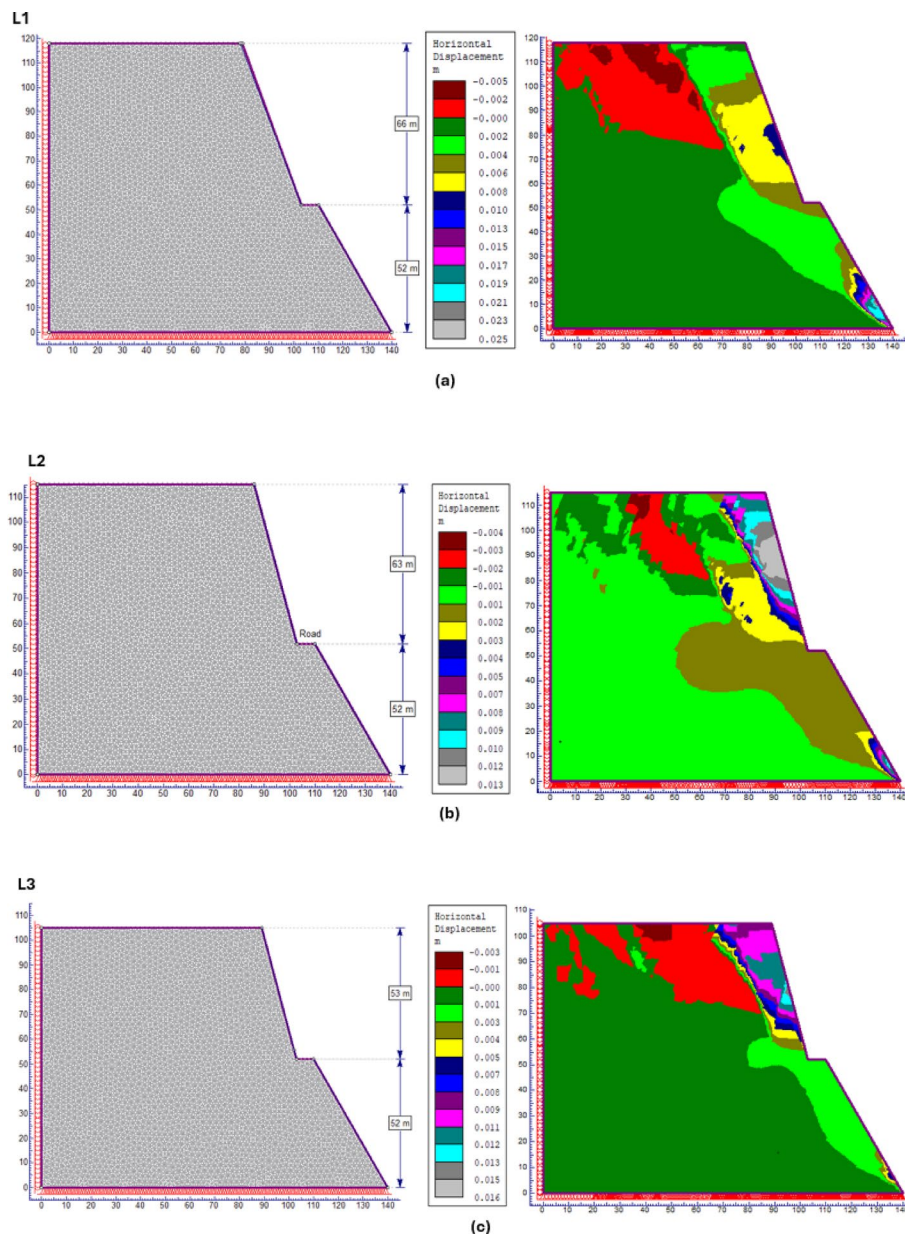
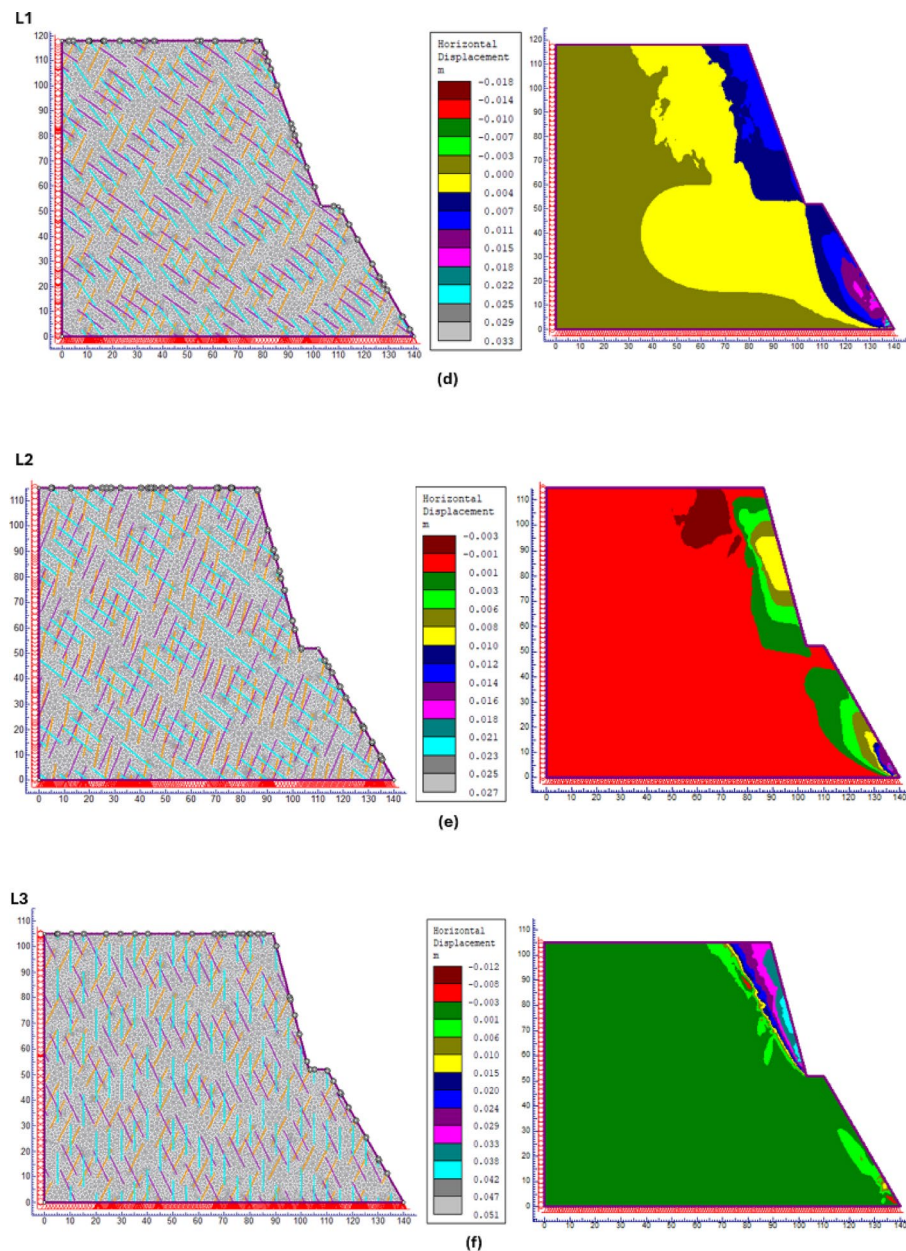


Fig. 6 Slope profile of the different locations without joints (a, b, c), incorporating three joint sets (d, e, f) and their corresponding horizontal displacement profile

and second, a jointed rock mass incorporating three representative joint sets to reflect in-situ geological conditions. This enabled the evaluation of the influence of structural discontinuities on slope behaviour.

The slope geometry and resulting deformation patterns are shown in Fig. 6. The rock mass strength was represented using the Generalised Hoek–Brown (GHB) criterion, while discontinuities were modelled with Barton–Bandis nonlinear joint elements. The discontinuity surfaces were partially infilled with clay-rich gouge and weathered calcite, which reduces joint wall strength and significantly contributes to deformability. These infill conditions were incorporated into the Barton–Bandis joint model through reduced JCS and stiffness inputs. The stability was assessed using the Shear Strength Reduction

**Fig. 6** (continued)

technique, and all geotechnical input parameters used in the numerical simulation are summarised in Table 5.

3.3.3 Model validation and uncertainty assessment

Uncertainty in rock mass input parameters can strongly influence numerical stability predictions, especially in jointed limestone, where discontinuity behaviour controls deformation mechanisms. To evaluate this influence, a sensitivity assessment was carried out by adjusting the joint normal and shear stiffness (K_n and K_s) within ± 10 – 20% of their field-derived values. The analysis revealed a clear trend: variations in stiffness notably affected the Strength Reduction Factor (SRF), with a 10 – 20% decrease in K_n leading to a 7 – 18% drop in SRF, while the same reduction in K_s resulted in a comparatively

Table 5 Parameters used for performing Finite Element Analysis

Parameter	L1	L2	L3
Height (m)	66	63	53
Inclination (°)	70	75	75
Unit weight (KN/m ³)	27	27	27
u_{rm}	0.26	0.25	0.26
UCS (MPa)	35	37	31
GSI	40	45	42
mi	8	8	8
mb	0.296	0.390	0.330
s	0.0001673	0.0003453	0.0002236
a	0.5114	0.5081	0.5099
D	0.7	0.7	0.7
Ei (MPa)	31,500	33,300	27,900
Erm (MPa)	1834	2606	1822
Kn (MPa/m)	6086	7440	4237
Ks (MPa/m)	609	744	423
JCS (MPa)	75	78	68
JRC	11	12	8
Residual Friction (°)	28	30	30

lower decline of 5–12%. This greater sensitivity to Kn highlights the critical role of normal stiffness in limiting joint opening and volumetric dilation, which directly contributes to toppling and wedge-controlled instabilities. These results are consistent with earlier findings [43], reinforcing the importance of accurately characterising joint stiffness in rock slope assessments.

Model validation was carried out through two complementary approaches. First, the initial numerical modelling configuration, including geometry representation, boundary settings, non-linear GHB material parameters, and SSR-based stability evaluation, was developed following the established modelling framework [38], which has been successfully applied to jointed Himalayan rock slopes. After benchmarking this setup, the same validated procedures were implemented for the three slopes studied here. Second, the predicted deformation and failure patterns were verified against observed field indicators such as block detachment, tension cracks, and debris accumulation near slope toes. Locations of maximum horizontal displacement in the jointed-slope simulations matched the actual instability zones identified during field inspections. The observed shift from marginal stability in LEA ($FOS \approx 1$) to unstable behaviour in joint-controlled FEA further aligns with ongoing localised failures in the study area.

4 Results and discussion

The stability evaluation of the three investigated road-cut slopes was carried out through a sequential framework integrating field investigations, empirical analyses, and numerical simulations. This multi-method approach enables cross-validation of results while also highlighting the unique strengths and limitations of each methodology.

4.1 Empirical analysis

In the empirical phase, Kinematic analysis using Rocscience DIPS identified distinct failure mechanisms at the three slope locations. At L1, joint set J1 (65°/285°) dips steeply opposite to the slope face (70°/100°), satisfying the geometric conditions for toppling. A similar anti-dip configuration occurs at L2, where J1 (80°/305°) is unfavourably oriented

relative to the slope face ($75^{\circ}/115^{\circ}$), again indicating toppling as the dominant mechanism. In contrast, L3 exhibits a wedge formed by the intersection of joint sets J3 ($66^{\circ}/64^{\circ}$) and J4 ($90^{\circ}/225^{\circ}$), producing an intersection line plunging 24° towards 244° that daylights on the slope face. These kinematic results align with field observations and confirm toppling failures at L1–L2 and wedge failure at L3.

Unlike previous slope assessments in the Mussoorie and broader Lesser Himalayan region, this study systematically establishes a one-to-one correspondence between slope-scale mechanism identification (via kinematics) and deformation-scale validation (via FEA). The explicit coupling of stereographic outputs with numerical displacement patterns represents a methodological advancement over studies that rely solely on LEA or field classification, and it provides a deeper mechanistic understanding of how discontinuity architecture governs slope response in jointed dolomitic limestone.

The next phase of the empirical analysis involves the application of rock mass classification schemes. The computed Rock Mass Rating (RMR) values classify all three slopes within the fair rock mass category. The relatively low RMR values are primarily attributed to the high degree of jointing, which reduces the Rock Quality Designation (RQD), moderately weathered rock characteristics, and lower uniaxial compressive strength (UCS). Incorporating the effects of discontinuity orientation and excavation method, Slope Mass Rating (SMR) assessments classify L1 as unstable, while L2 and L3 are deemed partially stable. The advantage of such classifications is their wide acceptance, simplicity, and suitability for rapid hazard zonation, making them valuable in preliminary stability screening. On the downside, these systems often generalise rock mass behaviour into discrete categories, potentially overlooking non-linear responses of jointed rock masses under varying stress paths, particularly in seismically active or heavily weathered terrains like the Himalayas.

4.2 Numerical modelling

The numerical simulation phase offered a more rigorous quantification of stability conditions. Stability analyses for the three road-cut slopes were carried out using Rocscience RS2 and Slide 2D. For Slide 2D, Limit Equilibrium Analysis (LEA) was applied with the Generalised Hoek-Brown (GHB) criterion as the model for the material, employing the General Limit Equilibrium (GLE) technique to ensure a comprehensive assessment. The results indicate that all three slopes exhibit factors of safety (FOS) close to 1, classifying them as partially stable, as shown in Fig. 5. This reduced FOS is mainly due to the steep topography of the slopes, combined with relatively low uniaxial compressive strength (UCS) and Geological Strength Index (GSI) values, highlighting the critical influence of rock mass quality and slope geometry on stability. The key strength of LEA is its simplicity and efficiency in estimating global slope stability, especially when coupled with the GLE method, which accommodates different failure surface assumptions. Nonetheless, LEA's inability to simulate stress-strain behaviour and progressive failure limits its capacity to capture the full complexity of rock slope mechanics.

FEA in RS2 introduces the deformation-based dimension necessary to decode structurally controlled failures (Fig. 6). When joints are excluded, numerical outputs mirror LEA results, indicating marginal stability. However, once the three mapped joint sets are incorporated, all slopes exhibit pronounced reductions in Stability Reduction Factor

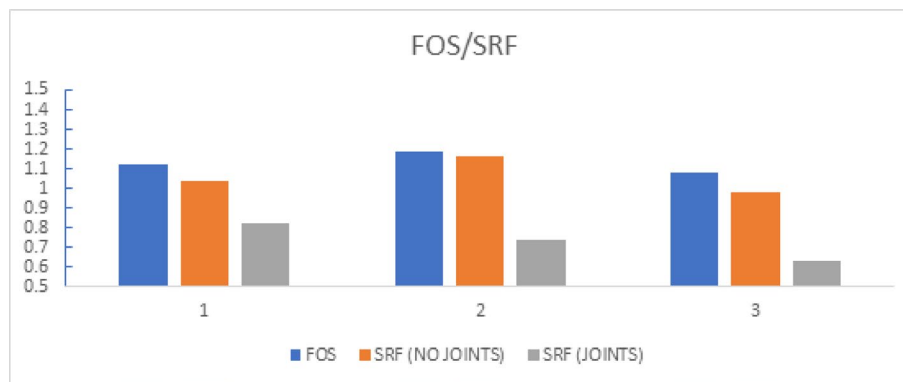


Fig. 7 Factor of safety (FOS) and strength reduction factor (SRF) bar chart for different locations with no joints and three sets of joints

(SRF), ranging from 20 to 40% as shown in Fig. 7, which shifts their classification from marginally stable to definitively unstable.

The deformation patterns provide compelling validation of earlier empirical findings:

- L1 and L2: Localised horizontal and rotational displacement at the upper slope face ranging between 10 and 40 mm, as shown in Fig. 6, mirrors the toppling initiation zones mapped in the field and identified in stereonets.
- L3: Shear localisation precisely tracks the J3–J4 intersection, reproducing the wedge sliding geometry predicted by the kinematic analysis.

Incorporating joint mechanical parameters (JCS, JRC, Kn, Ks) further reveals that stability is governed by joint deformability rather than intact rock strength. Even modest reductions in normal and shear stiffness cause disproportionately large decreases in SRF, demonstrating that Himalayan slopes behave as stiffness-controlled systems.

A key contribution of this work is the first quantitative demonstration of the sensitivity of SRF to joint stiffness variation (± 10 – 20%) for Himalayan dolomitic limestone. This mechanistic linkage between joint deformability, structural geometry, and slope-scale failure provides a new framework for understanding instability in highly jointed rock masses.

4.2.1 Recommendations for enhancing the stability

The integrated empirical and numerical assessment confirms that instability across the investigated slopes is predominantly governed by joint-controlled deformation, further intensified by steep slope geometry and fair-to-poor rock mass conditions. For the slopes at L1 and L2, where kinematic and FE results identified toppling as the principal failure mechanism, displacement concentrations were observed near the upper slope face, aligning with field evidence of block detachment. To enhance rotational resistance in these locations, systematic rock bolting oriented approximately perpendicular to the critical joint sets is recommended, complemented by toe anchoring measures to restrain outward movement of destabilised blocks. In contrast, the slope at L3 exhibited deformation patterns and shear displacements characteristic of wedge sliding along intersecting joint planes. Therefore, stabilisation at this location should focus on improving block interlock through cable-lacing or shotcrete-with-mesh systems, supported by directional bolts installed parallel to the plunge of the defined wedge intersection line.

4.2.2 Comparison with previous studies

The results indicate that all three slopes are only marginally stable under LEA conditions ($FOS \approx 1$), but their stability reduces significantly when discontinuity behaviour is explicitly captured in FEA, with corresponding reductions in Safety Reduction Factor (SRF) ranging between 20 and 40%. This behaviour aligns with previous studies in the Himalayan region, where structural defects and blasting-induced damage were identified as major contributors to instability in rock slopes [38, 47]. Similarly, recent findings suggest that explicit inclusion of structural conditions in FEM leads to markedly reduced safety margins in northwestern Himalayan rock slopes [27]. Past investigations near Mussoorie have also reported strong displacement localisation in jointed rock masses, consistent with the deformation patterns obtained in the present analysis [19].

The observed toppling failures at L1 and L2 and wedge failure at L3 correspond well with classical kinematic interpretations for steeply bedded dolomitic slopes [42]. Furthermore, the sensitivity of stability response to relatively small variations in GSI and joint stiffness parameters supports earlier observations emphasising the influence of rock mass quality and joint deformability [36]. Unlike studies relying mainly on discontinuity orientation or linear Mohr–Coulomb assumptions, this work incorporates both the non-linear Hoek–Brown rock mass model and the Barton–Bandis joint model, providing a more realistic representation of instability. These insights strengthen the relevance of the findings for infrastructure corridors in the Lesser Himalayas, where excavation, slope modification, and blasting continue to alter geomechanical conditions.

4.3 Study limitations

While this study presents a robust multi-method framework for evaluating the stability of jointed rock slopes, certain limitations must be acknowledged. Field data were collected from accessible portions of the slope face, which may not fully represent subsurface variability in joint conditions, especially at greater depths. Some mechanical parameters, including UCS and joint stiffness values, were derived from empirical correlations rather than laboratory testing due to site constraints, introducing uncertainty in strength characterisation. In the numerical analyses, joints were modelled as planar features and groundwater influence was considered minimal, whereas actual seepage conditions may vary seasonally and influence stability more prominently during peak monsoon periods. The absence of dynamic loading effects also limits the ability to predict slope performance during seismic events, which are known to impact the Mussoorie region. Future studies incorporating detailed geophysical investigations, laboratory shear testing, transient groundwater modelling, and seismic loading simulations would help refine predictive accuracy and strengthen design recommendations for similar Himalayan infrastructure corridors.

5 Conclusions

This work establishes an integrated and mechanism-aware slope stability evaluation framework for jointed rock masses in the Lesser Himalayas by systematically combining field investigation, empirical rock mass classifications, kinematic assessment, and deformation-based finite element modelling. The key contribution of this study is the demonstration that joint mechanical properties, particularly normal and shear stiffness, are major catalysts for deformation accumulation and can substantially reduce stability

margins even when traditional limit-equilibrium estimates indicate marginal safety. The findings make clear that reliance solely on global FOS assessments may lead to underestimation of risk in structurally controlled slopes.

Field-validated FE simulations revealed that toppling or wedge failures can be triggered from localised deformation zones concentrated near steep slope faces, providing a more realistic depiction of failure development. This integrated approach therefore advances the regional understanding of how blasting-induced damage, high joint persistence, and stress redistribution interact to control stability along transportation corridors in the Mussoorie area.

Furthermore, the study delivers application-ready insights by connecting numerical outcomes to mechanism-specific stabilisation strategies. This ensures that reinforcement design around critical road infrastructure can be grounded in quantified deformation behaviour rather than generalised hazard assumptions. The framework developed here is readily transferable to other Himalayan corridors and similar mountainous regions where discontinuity-dominated slope systems, limited geotechnical data, and rapid anthropogenic modifications heighten landslide risk.

Future work should incorporate hydrological variability, seismic loading, three-dimensional geometry, and laboratory-derived joint shear parameters to improve predictive confidence and support dynamic mitigation planning.

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Author contributions

V.S.C. and M.A.F. performed data curation and numerical analysis. M.A.F., M.R.S., and M.M.A. supervised the research and contributed to the final editing of the manuscript. All authors (V.S.C., M.A.F., M.R.S., and M.M.A.) participated in comprehensive editing, assessment, and approval of the final version of the manuscript.

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Data availability

The data used in the analysis can be made available on request.

Declarations

Ethics approval and consent to participate

Not Applicable.

Competing interests

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