

Stability Assessment of Cut Slopes Along Kasara Ghat on National Highway 3

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Abstract

The Western Ghats are a highly intricate and diverse terrain located within the Indian subcontinent. The region's unique topography, characterized by steep rock slopes, poses inherent challenges, including the risk of slope destabilization. The excavation made during the construction of the National Highway has been predominantly responsible for the future destabilization. A field-intensive study was conducted for the selected area with laboratory testing. Different vulnerable location-based samples of rocks and soils were tested in the laboratory to check their properties and strength. Out of 11 slopes, 7 slopes were considered for evaluation, and 1 slope (Location 6) had already failed (FoS was 0.86) in the last rainy season (in the month of September 2025). 2 slopes (Location 1 Slope 1, Location 2 Slope 2) were severely fractured (FoS less than 1) and thus need immediate attention. The remaining 4 slopes are highly weathered (expected to fail in the next few years). Rock sample results are compared with the minimum unconfined compressive strength requirement of the rock mass for stability of cut slopes. The calculated FoS by numerical analysis indicates high susceptibility to failure of rock slopes in the selected area. The analysis highlights the risk of rock fall in the future at different locations along the highway.

Keywords: Cut Slopes, Factor of Safety (FoS), Rock Mass Characterization, Slope Destabilization.

1. Introduction

The Western Ghats, renowned for their ecological significance and diverse landscapes, host several critical transportation corridors, among which Kasara Ghat stands as a pivotal link connecting Mumbai and Nashik [1]. The Western Ghats region of India represents a significant global hotspot regarding landslide occurrences. A comprehensive literature review focused on landslides in the Western Ghats has been instrumental in accurately identifying hazardous locations at risk of these natural phenomena [2]. The geological characteristics, land use patterns, rainfall data, and other pertinent factors contributing to the destabilization of rock slopes provide a comprehensive understanding of the underlying causes [3]. Unfortunately, the rainfall in the Western Ghats during August is very high in magnitude, resulting in numerous landslides. However, by applying statistical and other methodologies, the accurate prediction is confined to only 18 localized landslides, which is roughly 4.67%. The problem does not lie here; the actual number of landslides in August 2019 in Karnataka was recorded as 431 landslides against a predicted value of 80 [4]. The cited 4.67% prediction accuracy pertains to regional statistical hazard-zonation studies that rely on broad

environmental and meteorological datasets without site-specific geotechnical verification. The present study addresses this limitation by combining detailed field geometry and in-situ geotechnical sampling at 11 discrete locations specific to the Kasara Ghat, thereby improving the reliability of a slope-specific stability study.

One of the main problems with existing landslide studies in this region is that they do not provide comprehensive spatial prediction scenarios that incorporate environmental, geological, and meteorological parameters, along with local data. Thus, they all failed to show the actual scenario of the spatial distribution of landslides in the region. Secondly, the longer return periods for failure return dates, where the landslide occurrences took place in hours rather than days, were selected in computing the Landslide Hazard Index, which restricted the accuracy of the sub-catchment division for the determination of a particular portion to which the landslide occurred in the spatial distribution.

Previous landslide susceptibility assessments along Western Ghats highway corridors have largely relied on regional-scale statistical-based hazard zonation. These integrate broad

environmental and atmospheric variables but achieve limited site-specific predictive accuracy (as low as 4.67% in comparable studies) and do not provide slope-specific engineering parameters (factor of safety, required remedial measures) usable by highway authorities. The present study addresses this gap by: (i) conducting a targeted field survey and landslide inventory of 11 cut-slope locations along the Kasara Ghat section of NH-3; (ii) carrying out site-specific geotechnical laboratory testing at each location; (iii) performing both graphical (Bishop's simplified method) and numerical (finite element) stability analyses for the seven most critical slopes; and (iv) validating the analysis against an actual slope failure event observed during the September 2025 monsoon.

The objectives of the study are therefore to (a) quantify the current stability status of critical cut slopes along this corridor, (b) identify the dominant factors controlling instability, and (c) recommend site-specific remedial measures. This will provide a methodology replicable for similar hill-road corridors in the Western Ghats.

2. Study Area

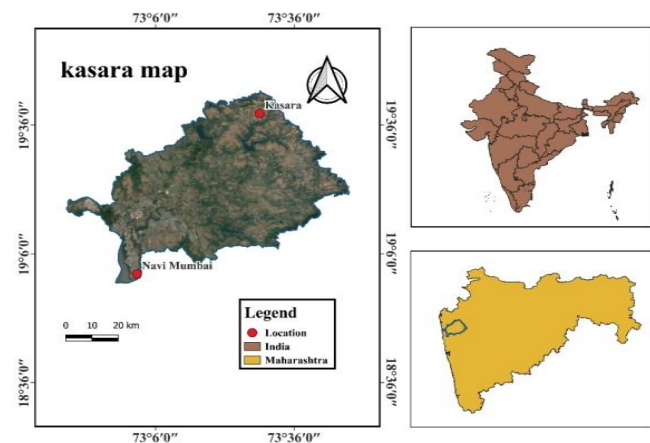
The study area is located along National Highway NH-3, which connects Nashik to Mumbai, and features the steepest slope in India, with a gradient of 1 in 37. This study focuses on the Nashik-Mumbai Road segment known for its multiple landslide-prone locations. The region is bounded by latitude $19^{\circ} 40' 12.00''$ N and longitude $73^{\circ} 28' 48.00''$ E. The Bhatsa River originates near Igatpuri, a hill station in Nashik district. The area has witnessed extreme weather events, such as the July 26, 2005 storm, which led to severe flash floods in the Mumbai Metropolitan Area around 2:00 PM. On that day, the Indian Meteorological Department (IMD) station in Santacruz recorded 944 mm (37.2 inches) of rainfall, the highest ever recorded in Mumbai. Spanning approximately 79 km, the study area experiences an average elevation change of 552 meters from Nashik to Kalyan. The region is characterized by shear zones, damaged areas, and multiple thrust zones, making it highly susceptible to landslides [5]. This susceptibility is primarily due to tectonic disturbances, which have resulted in weathered high-grade igneous rock formations. Additionally, intense rainfall further increases the landslide risk by saturating and weakening these fractured rock surfaces, leading to slope failures.

2.1. Geological Framework of Area

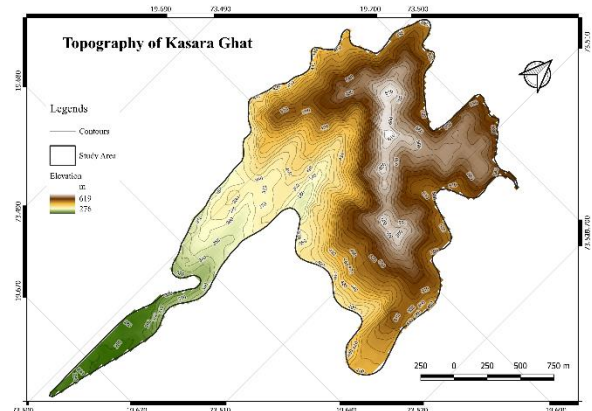
2.1.1. Lithology and weathering grade

The cut slopes investigated along Kasara Ghat are formed in a sequence of igneous and metasedimentary rocks comprising decomposed basalt overlain by a laterite cap with localized exposures of fine-to-medium-grained sandstone and schist near the base of several slopes [3]. The basalt unit beneath the laterite cap is pervasively decomposed as a result of tropical chemical weathering consistent with field observations of friable discolored rock at multiple sampling locations. Based on field exposure and the laboratory uniaxial compressive strength (UCS) results obtained in this study the rock mass at the majority of sampled locations is classified as Grade IV-V (highly to completely weathered) per the ISRM weathering

classification [6] with measured UCS values ranging from approximately 0.60 MPa to 4.46 MPa equivalent to only 3% to 22% of the 20 MPa benchmark generally considered necessary for a stable unsupported rock cut slope [5]. This weathering-grade classification directly explains the low cohesion and friction angle values measured at several locations (e.g., L1S1: $\phi = 11.86^{\circ}$) and is the primary factor underlying the low factor-of-safety values obtained for these slopes in Section 4.5/4.7. The location of the study area along NH-3 and the topographic setting of Kasara Ghat are illustrated in Fig. 1(a) and Fig. 1(b), respectively.



(a). Location of Study Area.



(b). Topography of Kasara Ghat.

Figure 1. The study area

2.1.2. Structural geology and discontinuities

The dominant structural control on slope stability in the study area is a near-vertical NNW-SSE-trending fault that cuts through the granite/basalt sequence and is traceable at the surface by the alignment of local drainage networks [5]. This fault is associated with a series of subsidiary thrust and shear zones [2] which introduce discontinuities oriented unfavorably with respect to several of the steeper cut-slope faces surveyed in this study, particularly at Location 2 (slopes L2S1-L2S3 measured cut angles of 50° - 58° ; Table 1). The south-western valley slope is further affected by active deep-seated landslides associated with reactivated Quaternary tectonics linked to the regional Konkan stress field [7]. The presence of this deep-seated failure history indicates that failure surfaces at some

locations may extend below the depth sampled by hand auger (0.5-1.5 m) in this study; this limitation is addressed further in the Discussion (Section 4.8) and is relevant to the numerical

slip-surface geometry adopted for Location 6 and the L2-series slopes in Section 4.5, where the modeled failure plane is consistent with the orientation of this fault system.

Table 1. Details of geometry of cut slopes at various locations.

Sr. No.	Location	Slope	Vertical Angle	Base length	Drop	Height	Total Height	Recommended Stable Angle Range*	Remark
1		S1	44°59'24"	14.9	1.1	14.90	16.00	35° - 45°	Within/near upper limit - monitor
2	L1	S2	41°44'24"	15.1	1.2	13.48	14.68	35° - 45°	Acceptable, weathering-sensitive
3		S3	39°58'12"	15.4	1	12.91	13.91	35° - 45°	Acceptable
4		S1	50°4'36"	14.7	0.6	17.57	18.17	35° - 45°	Exceeds recommended - critical
5	L2	S2	54°16'36"	13.9	0.5	19.31	19.81	35° - 45°	Exceeds recommended - critical
6		S3	57°38'00"	14.5	0.8	18.31	19.11	35° - 45°	Exceeds recommended - critical
7	L7	S1	39°2'46"	16	0.5	12.97	13.47	35° - 45°	Acceptable
8	L9	S1	30°7'00"	9.5	0.5	5.52	6.02	30° - 40°	Acceptable

*Indicative recommended stable cut-slope angle ranges for weathered basaltic/igneous rock masses based on IS 14458 (Part 1) guidance for hill-road cut slopes [10].

2.1.3. Hydrogeology

The hydrogeological regime of Kasara Ghat is characterized by a shallow water table that closely tracks the ground surface, generating rapid surface runoff during the monsoon, while the underlying basement exhibits a low infiltration capacity measured at less than 3 mm/hr. from field observation. This low permeability restricts the dissipation of pore water pressure once the near-surface soil/rock mass becomes saturated, as runoff is unable to drain vertically and instead accumulates at pre-existing discontinuities, joints, and the soil-rock interface [1].

2.1.4. Climatic and rainfall context

The Kasara Ghat region experiences a strongly seasonal monsoon climate with documented extreme rainfall events. The Indian Meteorological Department (IMD) station at Santacruz recorded 944 mm of rainfall within a single day on 26 July 2005, the highest single-day total ever recorded in the Mumbai region [8]. Rainfall of this intensity acting on the low-permeability basement described in Section 2.1.3 is capable of generating a rapid rise in pore water pressure at shallow depth, which is identified in Section 4.8 as the dominant short-term trigger for the failures observed at Location 1 (L1S1) and Location 6 (L6) during the September 2025 monsoon.

The infiltration capacity in the study area is reported to be low, indicating that the rock/soil mass drains very slowly once saturated. Under such conditions, subsurface drainage measures (e.g., horizontal drains, weep holes) have limited effectiveness in lowering the regional water table as flow towards the drains is restricted by the low hydraulic conductivity. Surface drainage measures - including catch-water drains at the slope crest, benching with lined chutes, and toe drains - are therefore expected to be more effective in this setting as they intercept and divert surface runoff before infiltration occurs.

Slope movements escalate if hazards remain unchecked; felled trees can exacerbate or mitigate debris spread, and various measures are implemented to restore stability and safeguard infrastructure against rainfall-induced failures. Swelling masses of monsoon rain saturate exposed surfaces, pooling at pre-existing faults, joints, and fractures.

2.2. Site Visit and Soil Sample Collection

The site visit to Kasara Ghat for the investigation and analysis of cut slope proved to be a comprehensive and insightful endeavor, offering valuable data for geotechnical and geological assessments. The Ghat, situated in a topographically challenging region, demanded a meticulous examination due to its susceptibility to geological hazards, particularly in the form of cut slopes along the roadway. Upon arrival, the geological setting of the site was immediately apparent, characterized by diverse rock formations and a complex terrain. The outcrops observed during the visit provided essential information about the lithological variations, structural features, and potential zones of weakness that could influence slope stability. The identified cut slope sections surveyed during the field visit are shown in Fig. 2.



Figure 2. Identified Cut Slope Sections.

Field measurements were conducted using instruments including a nautical sextant to assess the inclinations and potential movements of the cut slopes as shown in Fig. 3. Readings revealed subtle but noteworthy changes in slope angles at specific locations, indicating potential areas of concern. Field photographs documenting the condition of the investigated slopes along NH-3 are presented in Fig. 4. This information will be pivotal in the subsequent slope stability analysis.



Figure 3. Field measurements using a nautical sextant.

Furthermore, soil and rock samples were collected at various depths using soil augers, allowing for laboratory testing of geotechnical properties. These tests will provide essential data on soil cohesion, internal friction, and other parameters critical for slope stability analysis. The site visits also facilitated interaction with local authorities, residents, and stakeholders, providing valuable insights into historical events, local perceptions of slope stability, and any past incidents of slope failures. Such qualitative information supplements the quantitative data gathered during the site visit and contributes to a more comprehensive risk assessment.

In conclusion, the site visit to Kasara Ghat was an integral part of the investigation and analysis of cut slope stability. The diverse geological, hydrogeological, and environmental factors observed during the visit will serve as a foundation for a robust analysis aiding in the formulation of effective mitigation

strategies and engineering solutions to enhance the safety and sustainability of this critical transportation corridor. The multidimensional data acquired during the visit will inform subsequent geotechnical studies and contribute to a broader understanding of slope stability in complex terrains such as Kasara Ghat, as shown in Fig. 4.

2.3. Cut Slope Geometry

The cross-sectional geometry of the three cut slopes (S1, S2 and S3) at Location 2 is illustrated in Fig. 5. Slope angle and height are key inputs to rock-mass and slope-mass rating schemes used to assess road-cut slopes [9]. Table 1 summarizes the geometric parameters of all surveyed cut slopes, including vertical angle, base length, drop, total height, and recommended stable angle range.

3. Methodology

The vulnerable slopes along NH-3 in Kasara ghat can be identified in available literature, landslide inventory, and reconnaissance field survey. A detailed field survey was conducted to record geometrical, geological, structural, and geotechnical parameters with respect to slope stability analysis. Furthermore, laboratory experiments were carried out for the determination of geomechanical properties of the intact rock. The data obtained from the field were integrated with the laboratory results, which were used to perform numerical modeling for stability analysis.

The 11 locations sampled in this study were selected on the basis of a reconnaissance field survey targeting cut slopes showing visible signs of distress, prior failure, or steep/unsupported geometry along the Kasara Ghat corridor. This represents a targeted risk-based sampling strategy rather than a uniform or random sampling of the entire corridor. Accordingly, the results and FoS classifications presented should be interpreted as representative of the critical/high-risk zones identified during reconnaissance rather than as a statistically representative sample of the entire highway alignment. The overall procedure adopted for geotechnical investigation and stability analysis in this study is presented as a flowchart in Fig. 6.

3.1. Geotechnical Analysis

Soil parameters are determined through Laboratory tests [5]. Moisture-density relations are established by Standard Proctor tests [11]. Sieve analysis and Atterberg-limit tests provide particle-size distribution, liquid limit, plastic limit, and plasticity index [12], [13]. Cohesion and friction angles are obtained from direct shear, unconfined compressive, and triaxial tests [14], [15]. Powdery clay was allowed to settle in water for two weeks before sampling to shape undisturbed specimens for undisturbed-strength assessment.

To determine whether the slope material is coarse-grained or fine-grained, which has a direct effect on drainage and stability. Determines the susceptibility to erosion, piping, and wash by surface water; these are problems very common in ghat sections. In Kasara Ghat, residual soils from basalt generally exhibit spatial variation in grain size. Areas with a higher content of fines are especially prone to rain-induced failures.

Limits of Atterberg describe consistency as well as the plasticity characteristics of fine-grained soils. Show sensitivity to moisture and a reduction in shear strength when saturated. High values of the liquid limit and plasticity index indicate high compressibility potential for strength loss during monsoon rains. The shear strength increased with better compaction of soil, and permeability decreased as well. Direct shear parameters find application in Bishop's method, Janbu's method, and Morgenstern–Price method for slope stability assessment applicable to ghat sections [5]. UCS represents short-term undrained shear strength immediately after excavation or during heavy rainfall. Useful for total stress

analysis of slopes in saturated cohesive soils. A uniaxial compressive strength test on rock samples assesses the intact rock's strength and deformability. It assesses the strength of basaltic rock exposed in cut slopes and the degree of weathering, supporting rock mass classification systems such as RMR, Q-system, and GSI, with a combined interpretation from soil and rock test results to identify dominant failure mechanisms and select appropriate shear strength parameters for effective drainage stabilization measures designed to rely heavily on an integrated approach for reliable cut slope analysis at Kasara Ghat, where highly variable geological conditions exist alongside hydrological conditions.



Figure 4. Field photographs depicting the condition of investigated slopes along NH3 (S1, S2, etc. Names of Slopes for field identification).

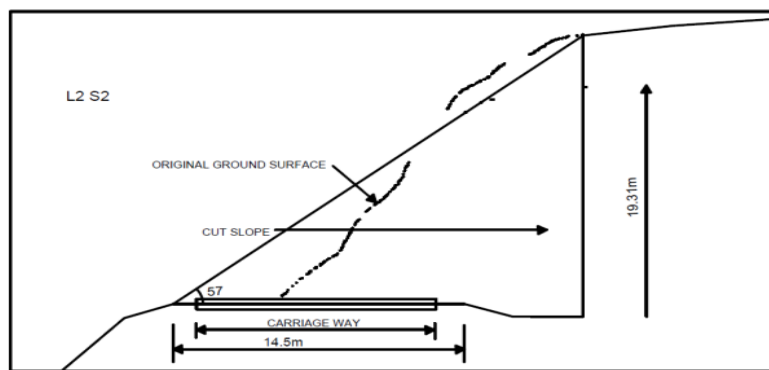


Figure 5. Cross Section of cut slopes 1, 2, 3 at location 2.

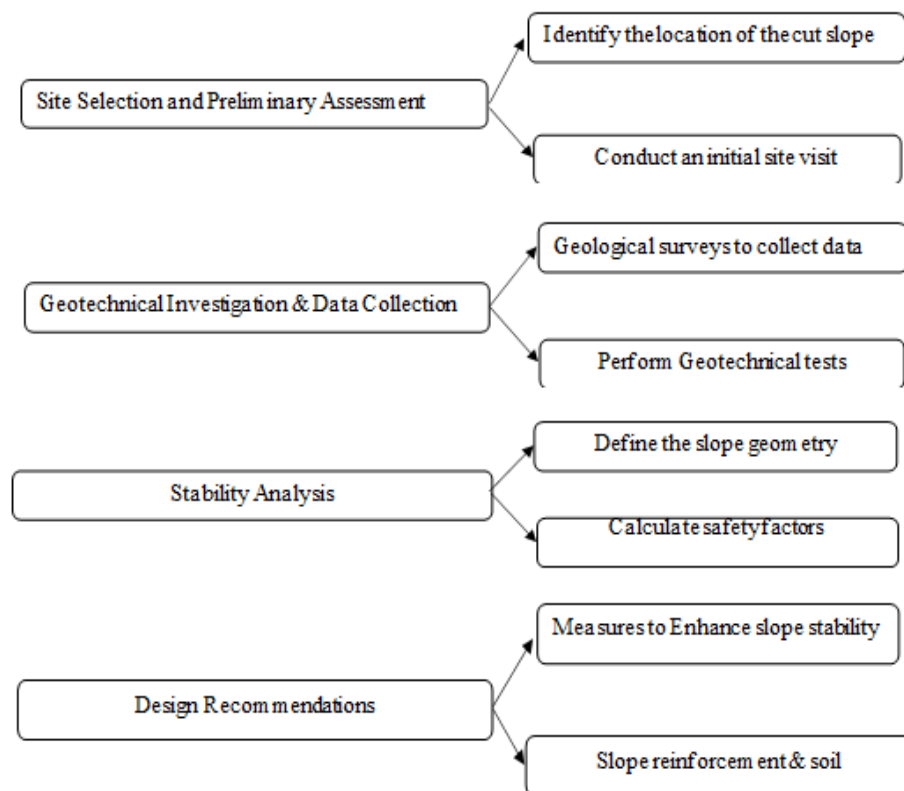


Figure 6. Procedure of geotechnical investigation and analysis.

Soil and rock samples were collected from each of the 11 locations using hand augers and by manual extraction of representative rock fragments from exposed cut faces at depths of approximately 0.5-1.5 m below the existing slope surface. At locations where multiple sub-samples were collected (e.g., L1S2, L3S1), each sub-sample was tested independently, and the resulting range of values (Table 2) illustrates the local variability in material properties attributable to heterogeneous weathering as observed in Fig. 4.

Uniaxial compressive strength (UCS) of rock samples was determined in accordance with IS 9143 (equivalent to ASTM D2938) [15] on prepared cubical specimens loaded axially to failure at a controlled loading rate. The measured UCS values

are compared against a minimum benchmark of 20 MPa representing the lower-bound compressive strength generally recommended for rock masses forming stable unsupported or lightly supported cut slopes in the height range encountered in this study. Measured UCS values are reported in both kN/m^2 and MPa ($1 \text{ MPa} = 1000 \text{ kN/m}^2$); the percentage of the 20 MPa benchmark achieved by each sample is also tabulated to allow rapid identification of locations with critically low rock mass strength. The applicable testing standards, sample preparation procedures and number of specimens tested for each laboratory test are listed in Table 2.

Table 2. Standards adopted for the various laboratory tests.

Test	Applicable Standard	Sample Preparation	No. of Specimens
Grain Size Analysis	IS 2720 (Part 4)	Air-dried, sieved per standard gradation	3 per location (11 locations)
Atterberg Limits	IS 2720 (Part 5)	Air-dried, passed through 425-micron sieve	3 per location (11 locations)
Direct Shear Test	IS 2720 (Part 13)	Remolded to field density, tested at field moisture content	6 per location (11 locations)
Uniaxial Compressive Strength (UCS)	IS 9143 / ASTM D2938	Cylindrical/block specimens cut from rock fragments, ends ground parallel	3 per location (11 locations)
Standard Proctor test	IS 2720 (Part 7)	Tested for optimum moisture content and maximum dry density	3 per location (11 locations)

3.2. Numerical Modeling

The numerical stability analyses were performed using Plaxis 3D, a three-dimensional finite element modeling platform employing the Mohr-Coulomb elasto-plastic constitutive model for all material zones. Critical slope locations were modeled in full three dimensions, enabling the capture of lateral confinement effects, three-dimensional stress redistribution, and non-planar failure surface geometries.

The three-dimensional model domain for each location was constructed based on the field-surveyed slope geometry (Table 1), with the model width in the out-of-plane direction taken as sufficient to eliminate boundary effects on the computed failure surface. The domain was discretized using a triangular-quadrilateral hybrid finite element mesh, with mesh refinement applied in the vicinity of the slope face and the anticipated failure zone to improve resolution of shear strain localization. Boundary conditions comprised full fixity at the model base and horizontal restraint on the lateral boundaries perpendicular to the slope alignment, and roller conditions on the front and rear faces of the 3D model block. Gravity loading was applied as the initial stress condition. A cut slope was formed at the location during the construction of the highway. The slope height at the location is approximately 20 m. The failing slope is located at chainage 123.035 km, and the site lies between latitudes 19.3835987 and 19.3853563 and longitudes 73.3458295 and 73.3467069. The reduced loading configuration estimates factors of safety. Slope geometries evolve as filling and excavation modify the field. The factor of safety and increase in performance window remain closely linked, guiding the evolution of engineering responses.

In the present numerical models, vehicular loading was represented as a static surcharge applied at the road shoulder together with the self-weight of the soil/rock mass. Dynamic/cyclic loading arising from heavy truck movement was not explicitly modeled. However, for slopes already close to or below the critical FoS of 1 (L1S1, L1S2, L1S3, L2S1, L2S2, L2S3, L5, L7), repeated traffic-induced vibration may contribute to progressive degradation of shear strength along discontinuities and crack propagation over time, effectively reducing the FoS further. Traffic-induced vibration could act as an additional trigger, potentially advancing the timing of failure relative to what a static analysis would suggest. This study

should therefore be interpreted as a relative susceptibility ranking under static conditions [16]; the absolute timing of collapse for near-critical slopes is governed by additional transient factors (rainfall intensity, traffic-induced vibration) not captured in the present static models and which are recommended for inclusion in future dynamic analyses.

3.3. Finite Element Method

FEM is an elastoplastic technique that has been extensively used in geotechnical analyses. It does not necessitate any prior assumptions about the position of the slip surface, and its solutions are inherently nonlinear and iterative [17]. The method is cost-effective and time-saving, with iteration capabilities making it a trustworthy technique for gaining optimal and efficient answers to geotechnical issues. In FEM, failure happens spontaneously along paths where the existing shear stresses exceed the material's shear strength without any pre-set assumptions [18]. For assessing slope stability using numerical approaches, factor of safety (FoS) is probably the most straightforward and quick way to assess how close a slope may be to failure [19]. This quantitative evaluation gives the user or reader an easy summary; it can be defined as the ratio between the actual shear strength of the material forming the slope and the least shear strength necessary to keep it from falling. In shear strength reduction (SSR) analyses, many simulations are run over a series of FoS trials [18]. During successive iterations, the material's shear strength is reduced until failure takes place [18].

3.4. Graphical Method

Bishop's simplified graphical method was adopted to evaluate the stability of cut slopes at Kasara Ghat, where failures are predominantly rotational and triggered by monsoon-induced saturation. The method is well suited for soil-dominated slopes, as it considers inter-slice normal forces and allows inclusion of pore water pressure effects while maintaining computational simplicity [20]. The analysis was carried out using geotechnical parameters obtained from laboratory testing of soil samples collected at Location 1 – Slope 1 (L1S1). A circular slip surface passing through the toe of the cut slope was assumed based on field observations. The soil mass above the slip surface was divided into vertical slices, and stability was evaluated by satisfying moment equilibrium.

$$X_i = C * l + (w * \cos\alpha - u * l) * \tan\phi \quad (1)$$

$$Y_i = w \sin\alpha \quad (2)$$

$$FoS = X_i/Y_i \quad (3)$$

4. Result and Discussion

Since cut slopes are exposed to weathering and erosion, they are more vulnerable to landslides than natural slopes [21]. Field observations of cut slopes along the studied stretch indicate that debris accumulation near the toe occurred only during the rain; without drainage, mud and water trickled down from the top of the slope, and more frequent inspections during the monsoon might help. Despite concerns regarding road safety and increasing traffic along the Kasara Ghat, frequent monitoring and maintenance of cut slopes remain essential [22].

4.1. Geotechnical Aspect

Cut slopes are often affected by saturation and weathering, causing extensive slab or planar-type failures [23]. The geotechnical investigation highlighted a potential for failure due to shear along a horizontal plane similar to previous

landslides. The road traverses the lower Western Ghats comprising basaltic rocks and potential landslide-prone slopes [2]. Therefore, interpreting location-based test results is a key geotechnical aspect of the cut slope study.

Grain size analysis classified the soil at location L1S1 as well-graded gravel (GW), a material that would typically be expected to exhibit a friction angle in the range of 30-40 degrees in a dense in-situ state. However, the direct shear test on this sample yielded a markedly lower friction angle of 11.86 degrees (Table 3). This is attributed to: (i) disturbance of natural particle interlock and density during auger sampling, reducing the dilatant component of shear resistance; (ii) high in-situ moisture content at the time of sampling (post-monsoon), which can lubricate particle contacts, particularly where weathered-rock fines are present within the gravel matrix. The measured value of 11.86 degrees should be treated as a conservative lower bound for the disturbed state. The soil properties used as input parameters for the graphical stability analysis using Bishop's simplified method are given in Table 3. A sample table illustrating the calculated values of each term in Bishop's formula for each vertical slice of the slope is shown in Table 4.

Table 3. Soil properties used for the preparation of graphical solution.

Parameter	Symbol	Value	From
Unit weight of soil	γ	20.3 kN/m ³	Standard Proctor test
Cohesion (drained)	c	39.22 kPa	Direct shear test
Angle of internal friction	ϕ	11.86°	Direct shear test
Undrained shear strength	c_u	≈ 26 kPa	UCS test
Pore water pressure	u	Considered for monsoon condition	Field assumption

Table 4. Sample table of values of different terms in the formula suggested by Bishop for Location 6.

Slice	Area	Angle α	Weight (A X γ)	X_i	Y_i
1	49.1	4.9	1104.75	393.25	94.36
2	114.4	5.3	2574	700.34	237.76
3	109.7	15.7	2468.25	661.1	667.9
4	94.8	26.7	2133	595.67	958.39
5	70	38.9	1575	419.51	989.04
6	29.6	54.1	666	244.11	539.48

The location-based soil properties, including soil type, fines content, maximum dry density (MDD), optimum moisture content (OMC), cohesion, and friction angle, identified at each location along NH-3 are presented as follows: Location 1 (Slopes 1 and 2) in Table 5, Location 2 (Slopes 1 and 2) in Table 6, Location 3 in Table 7, Locations 6 and 7 in Table 8, and Location 8 in Table 9.

Gravelly material is strong, but the measured ϕ is low, so a conservative design is needed. Stability mainly depends on friction, not cohesion. Due to the high LL of the fines portion, strength may drop during saturation, which is critical in monsoon conditions. Saturated conditions still reduce friction significantly (confirmed by shear test). However, direct shear tests show a significantly reduced friction angle, likely due to the disturbed state and high moisture.

The friction angle obtained from direct shear testing of the L1S1 sample was relatively low ($\phi = 11.86$ degrees). This is attributed

in part to disturbance of the sample during extraction and transport and to the elevated in-situ moisture content at the time of sampling (samples were collected during/after the monsoon period). Both factors are known to reduce the measured peak friction angle relative to the in-situ, less-disturbed condition.

Indicates low true cohesion, consistent with gravelly soil behavior. Shear strength is predominantly friction-controlled. Soil is non-expansive and moisture-insensitive. Exhibits good drainage and interlocking characteristics. Low UCS values confirm a non-cohesive to weakly cohesive nature. Moisture Sensitivity: Low to moderate; suitable for monsoon conditions with proper drainage. Soil exhibits high frictional resistance, consistent with gravelly material.

Table 5. Location-based soil properties identified along the NH-3 (Location 1)

Location 1 Slope 1	
Parameter	Value
Soil Type	Well-graded gravel (GW)
Fines content	2.56%
MDD	20.3 kN/m ³
OMC	20%
UCS_avg	52 kPa
Undrained su	26 kPa
Cohesion c	39.22 kPa
Friction angle ϕ	11.86°

Location 1 Slope 2	
Parameter	Value
Soil Type	Well-graded gravel with sand
Fines content	3.05 %
MDD	16.6 kN/m ³
OMC	24.5 %
UCS_avg	50 kPa
Undrained su	25 kPa
Cohesion c	37 kPa
Friction angle ϕ	10.2°

Table 6. Location-based soil properties identified along the NH-3 (Location 2)

Location 2 Slope 1	
Parameter	Value
Soil Type	Well-graded gravel with sand
Fines content	1.90 %
MDD	19.9 kN/m ³
OMC	25.4 %
UCS_avg	20 kPa
Undrained su	10 kPa
Cohesion c	10 kPa
Friction angle ϕ	17°

Location 2 Slope 2	
Parameter	Value
Soil Type	Well-graded gravel with sand
Fines content	1.95 %
MDD	17.6 kN/m ³
OMC	20.7 %
UCS_avg	20 kPa
Undrained su	10 kPa
Cohesion c	10 kPa
Friction angle ϕ	18°

Table 7. Location-based soil properties identified along the NH-3 (Location 3)

Parameter	Value
Soil Type	Gravel–sand mixture
Fines content	0.28 %
MDD	16.5 kN/m ³
OMC	23.5 %
UCS_avg	29.5 kPa
Undrained su	14.8 kPa
Cohesion c	18 kPa
Friction angle ϕ	32°

Table 8. Location-based soil properties identified along the NH-3 (Locations 6 and 7)

Location 6	
Parameter	Value
Soil Type	Gravel–sand mixture
Fines content	2.05 %
MDD	20.5 kN/m ³
OMC	19 %
UCS_avg	31.5 kPa
Undrained su	15.8 kPa
Cohesion c	11 kPa
Friction angle ϕ	35°

Location 7	
Parameter	Value
Soil Type	Gravel–sand mixture
Fines content	2.22 %
MDD	16.4 kN/m ³
OMC	23.3 %
UCS_avg	34 kPa
Undrained su	17 kPa
Cohesion c	10 kPa
Friction angle ϕ	25°

The soil at Location 6 is a coarse-grained gravel–sand mixture with negligible fines, characterized by low plasticity, high dry density, and high frictional strength. The soil at Location 7 is classified as well-graded gravel with sand (GW), exhibiting negligible fines, very low plasticity, and favorable compaction characteristics.

Table 9. Location-based soil properties identified along the NH-3 (Location 8)

Location 8	
Parameter	Value
Soil Type	Gravel–sand mixture
Fines content	2.88 %
MDD	18.6 kN/m ³
OMC	22.6 %
UCS_avg	32 kPa
Undrained su	16 kPa
Cohesion c	9.5 kPa
Friction angle ϕ	14°

Exhibits excellent drainage and good particle interlocking. Soil is non-expansive with low moisture sensitivity. Moderate to high dry density achievable with standard compaction effort. The soil test results for the slope at Location 6, including sieve analysis, liquid limit, moisture-density relationship, normal stress versus shear stress plot, and UCS test, are presented in Fig. 7.

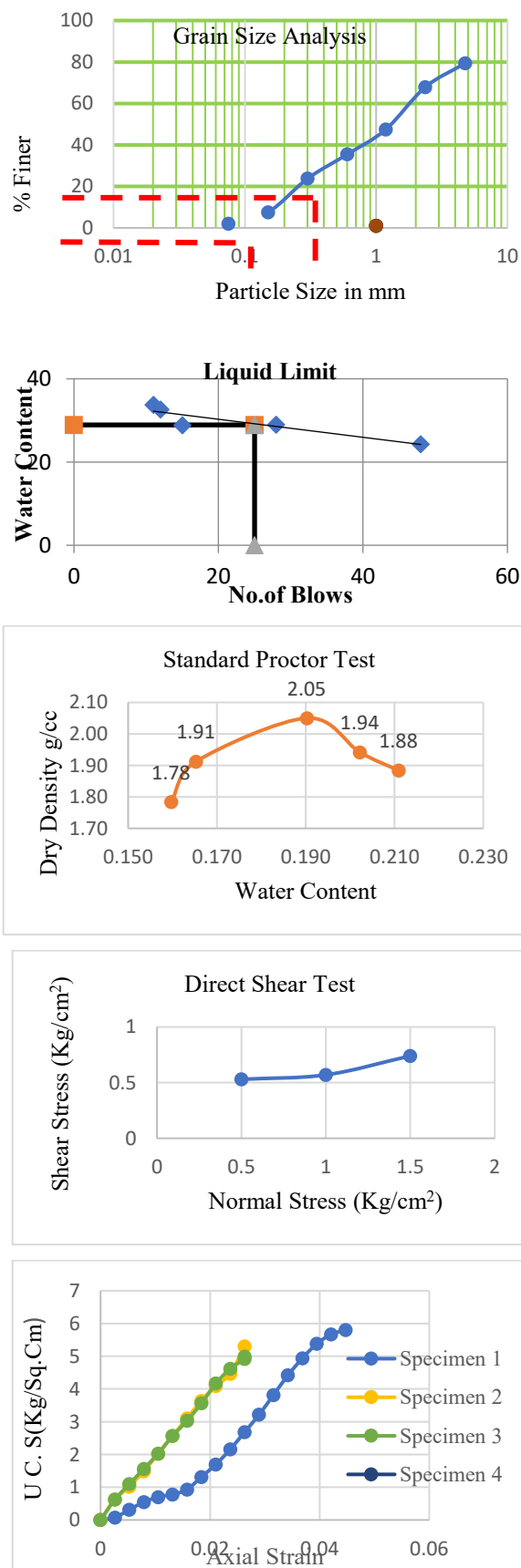


Figure 7. Soil test results of slope at location 6: a) Sieve Analysis, b) Liquid Limit, c) MDD Vs OMC, d) Normal Stress Vs Shear Stress, e) UCS Test.

4.2. Rock Sample Test

Table 10 presents the uniaxial compressive strength (UCS) of rock samples collected from all locations, along with water absorption, specific gravity, and the percentage of remaining strength compared to the 20 MPa minimum benchmark.

4.3. Rock Type and Compressive Strength

For intact competent rock, uniaxial compressive strength (UCS) of 20 MPa (megapascals) (20000 kN/m²) or higher is generally considered adequate for moderately steep cut slopes. For weaker rocks or highly weathered rock masses, stabilization measures such as retaining structures or slope reinforcement may be needed even if the UCS exceeds 20 MPa (20000 kN/m²), because weathering and saturation reduce rock strength and slope stability [24].

4.4. Slope Angles

Shallower slope angles require lower compressive strength for stability [9]. For steep slopes (greater than 45 degrees), UCS values of 40–50 MPa (50000 kN/m²) or higher may be necessary, especially in areas prone to heavy rainfall or seismic activity.

While UCS is a critical parameter, slope stability is determined by the shear strength of rock and discontinuities. Groundwater conditions (e.g., water pressure reducing effective stress).

Table 10 shows that measured compressive strength values for most rock samples are substantially below the 20 MPa benchmark generally considered adequate for moderately steep cut slopes in competent rock. As noted in the Conclusion, this corresponds to roughly 3 to 22% of the strength expected of the parent rock attributed to tropical chemical weathering of the basaltic lithology [24] (L4 55.3% is the exception). The present study did not directly measure the rate of UCS decrease over time, as this would require repeat sampling of the same rock faces over multiple monsoon seasons. Given the rapid weathering evident from the already-low UCS values, it is recommended that compressive strength testing be repeated at 3–5-year intervals at each site.

4.5. Finite Element Method

Finite element method (FEM) stability assessments show a range of slope conditions across elements, with a distinctly low degree of stability for the selected slope, consistent with FEM-based assessments of road slopes in the Himalaya [25]. The lowest safety factor computed, 0.86, confirms a highly unstable situation [5], while another presents an alternative phase with ongoing instability. Numerical outcomes also suggest the condition degrades following rainfall, exacerbating the hazard. The numerical analysis presented represents the rock and soil mass using equivalent material properties. This approach is appropriate where the rock mass behaves approximately as a continuum but may not fully capture the kinematics of highly fractured rock masses such as those associated with the NNW-SSE fault and thrust zones described in Section 2.1, where failure may instead occur through sliding and toppling of discrete blocks bounded by discontinuities.

Table 10. Compressive Strength of Rock Sample.

Location	Dimension		Area (cm ²) L*B	Load kN	Compressive strength kN/m ²	Compressive strength MPa	Water absorption	Specific gravity	% of remaining Strength	Remark
	L	B								
L1S1	18.6	15.6	290.16	17.44	601.05	0.60	1.49	2.64	3.0%	Highly deficient
L1S2 S	13.5	10.3	139.05	37.66	1297.90	1.30	1.2	2.59	6.5%	Highly deficient
L1S2B	15	14.8	222	69.86	2407.64	2.41	1.15	2.6	12.0%	Highly deficient
L1S3	13.5	13.3	179.55	23.34	804.38	0.80	1.45	2.55	4.0%	Highly deficient
L2S1	15.8	13.9	219.62	129.4	4459.61	4.46	1.59	2.58	22.3%	Deficient
L2S2	17.2	16.5	283.8	25.86	891.23	0.89	1.54	2.59	4.5%	Highly deficient
L2S3	15	13.9	208.5	53.14	1831.40	1.83	1.32	2.7	9.2%	Highly deficient
L3S1 Y	14.8	14.4	213.12	46.3	1595.67	1.60	1.28	2.6	8.0%	Highly deficient
L3S1	19.3	13.9	268.27	98.34	3389.16	3.39	0.83	2.64	17.0%	Deficient
L3S1 S	16.5	11.1	183.15	38.94	1342.02	1.34	0.98	2.59	6.7%	Highly deficient
L3S1 B	21.4	18.5	395.9	81.7	2815.69	2.82	0.96	2.6	14.1%	Deficient
L4	14.8	11.4	168.72	321.1	11066.31	11.07	1.92	2.65	55.3%	Deficient
L5	18.5	16.6	307.1	70.28	2422.11	2.42	0.97	2.6	12.1%	Highly deficient
L6	15.7	11.4	178.98	33.04	1138.68	1.14	1.43	2.7	5.7%	Highly deficient (failed)
L8	13.7	11.3	154.81	54.92	1892.75	1.89	1.24	2.64	9.5%	Highly deficient

For locations exhibiting visibly blocky or jointed rock exposures, Discrete Element Method analysis is recommended as a complementary approach to better represent block-scale movement [26], [27].

The reduced loading configuration estimates factors of safety. Slope geometries evolve as filling and excavation modify the field. The factor of safety and increase in performance window remain closely linked, guiding the evolution of engineering responses. Across the full range of slope specifications, safety exceeds the critical terrain threshold, revealing capacity to accommodate substantial hydraulic enlargements. For Location 6, the FEM analysis produced an FoS of 0.88, close to the graphical-method value of 0.86, and the deformed-mesh output (Fig. 8 and 9) showed a continuous zone of shear-strain localization forming an unbroken slip path from the slope crest to the toe, an indicator commonly associated with imminent failure. This combination of (i) FoS below 1.0 in both methods and (ii) a fully developed shear band was consistent with the slope failure subsequently observed in September 2025 (see Section 4.8), providing ground validation of the approach. At other locations with FoS close to but not below 1.0 (e.g., L1S2: 1.0-1.13; L1S3: 0.99; L2S1: 0.91-0.97; L2S2: 0.90; L2S3: 0.98; L5: 0.95; L7: 0.92) a similar near-critical FoS does not guarantee imminent failure since failure additionally requires a triggering mechanism that may or may not occur within a given season. These locations should therefore be regarded as having

an elevated probability of failure rather than a certainty, and the possibility of false positives cannot be excluded. Continued monitoring and re-assessment after each monsoon season is recommended to track changes in FoS at these locations.

4.6. Graphical Method

A graphical method for slope stability analysis was used to determine the factor of safety of the modeled cut slopes along Kasara Ghat [2].

$$Xi = C * l + (w * \cos\alpha - u * l) * \tan\Phi = 88.25 * 37.2 + (1104.75 * \cos 4.9 - 67.5 * 37.2) * \tan 11.86 = 393.25 \quad (4)$$

$$Yi = w \sin\alpha = 1104.75 * \sin 4.9 = 94.36 \quad (5)$$

$$FoS = Xi/Yi = 502.33/581.15 = 0.86 \quad (6)$$

The principal effective stress (σ'_3) contours and the deformed mesh output from the Plaxis 3D analysis for Location 6 are presented in Fig. 8 and 9. The plastic point history and tension cut-off zones developed during the numerical analysis of Location 6 are illustrated in Fig. 9. The deformed soil model and factor of safety (FoS) convergence curve obtained from the numerical analysis for cut slope L1S2 are presented in Fig. 10. The deformed soil model and FoS convergence curve from the Plaxis 3D numerical analysis for Location 6 (L6) are shown in Fig. 11. The numerical analysis output including deformed soil model and FoS curve for cut slope L2S1 is presented in Fig. 12.

The deformed soil model and factor of safety convergence curve for cut slope L5, obtained from the Plaxis 3D analysis are shown in Fig. 13. The numerical analysis results for cut slope L7 including the deformed mesh and FoS convergence curve are presented in Fig. 14.

The factor-of-safety values computed for each slope from the finite-element numerical analysis using Plaxis 3D are summarized in Table 11. The detailed cross-sectional drawing of the slope at Location 6 used for the graphical stability analysis is shown in Fig. 15.

4.7. Location 6: Properties of Soil and Slope

The geotechnical properties of the soil at Location 6 and the computed slice-by-slice values of X_i and Y_i used in the graphical analysis are presented in Table 12. The factor-of-safety values obtained for each evaluated slope using the graphical method (Bishop's simplified approach) are summarized in Table 13. A comparison of the factor-of-safety values obtained from both the graphical (Bishop's simplified method) and numerical (Plaxis 3D) methods for all evaluated slopes is presented in Table 14.

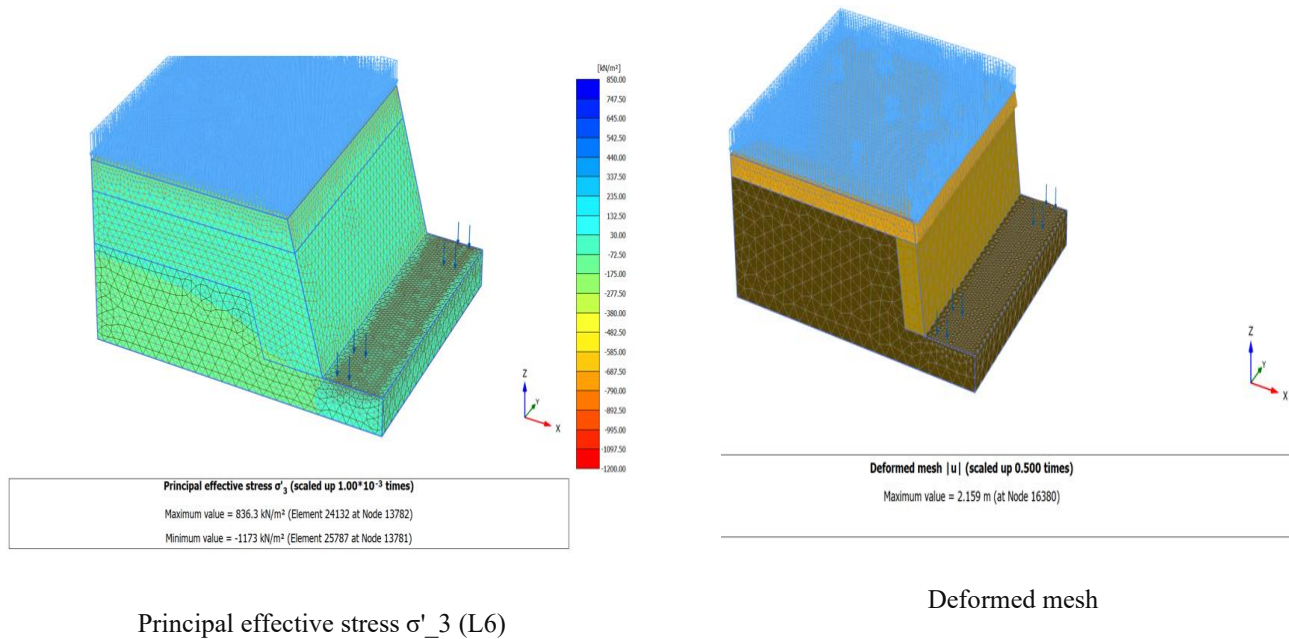


Figure 8. Representation of Principal effective stress and deformed mesh.

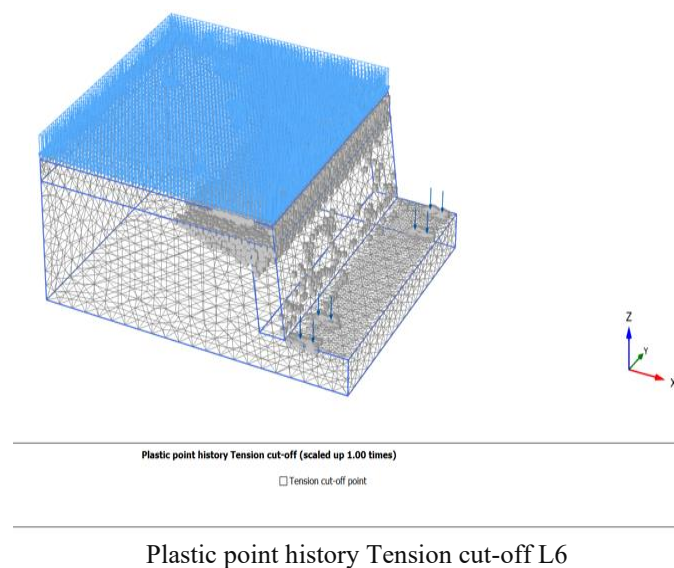
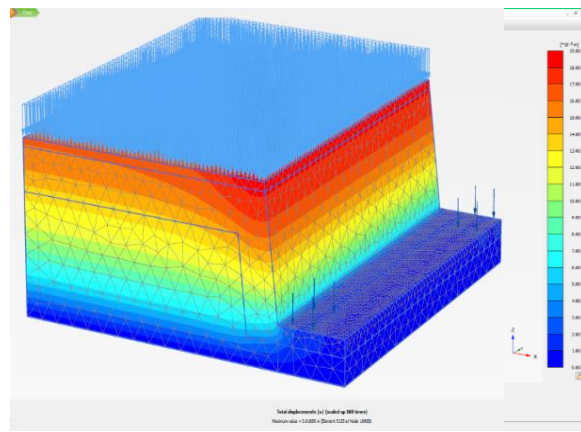
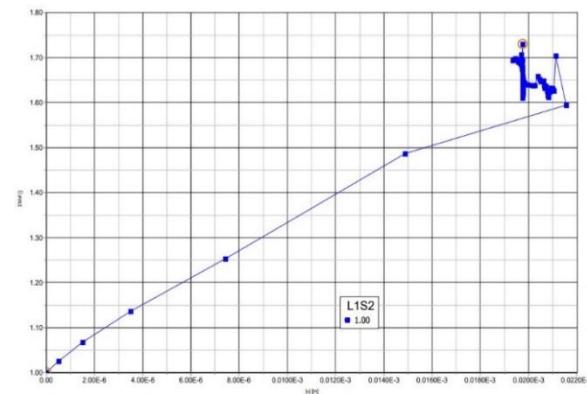


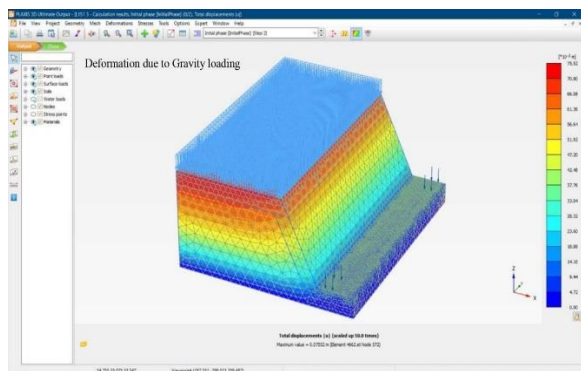
Figure 9. Representation of Plastic point history, tension cut-off L6.



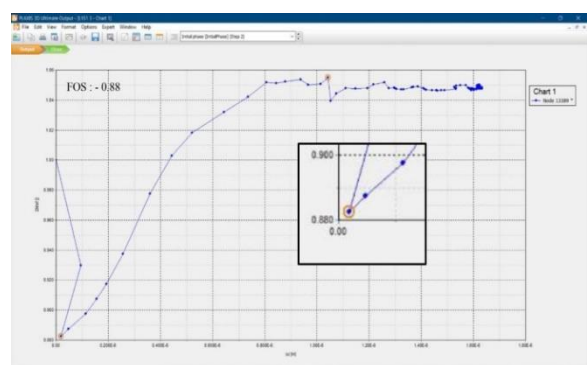
Deformed Soil Model (Output) (L1S2)



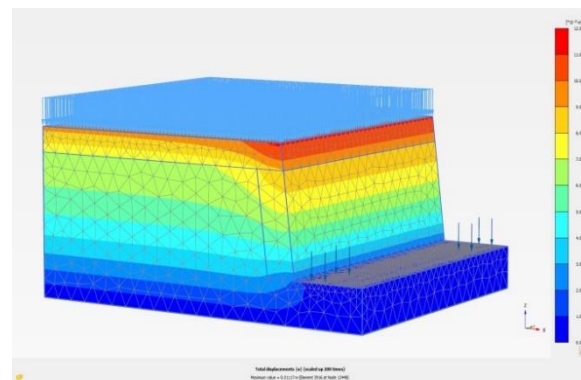
FoS Curve for Soil Model (L1S2)

Figure 10. Representation of numerical analysis output for the cut slope L1S2.

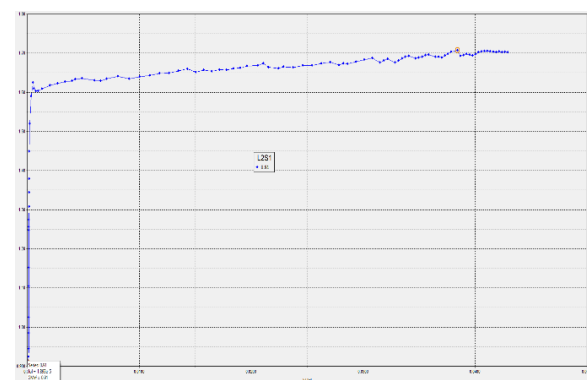
Deformed Soil Model (Output) (L6)



FoS Curve for Soil Model (L6)

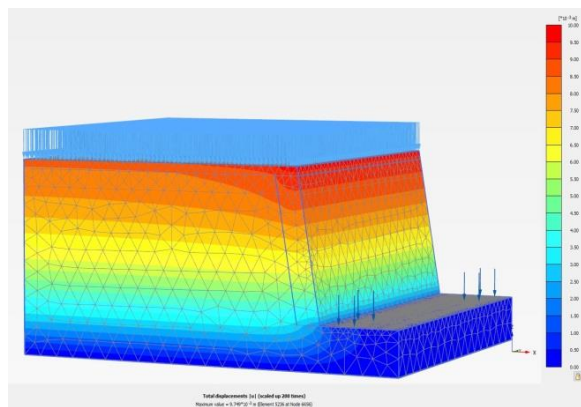
Figure 11. Representation of numerical analysis output for the cut slope L6.

Deformed Soil Model (Output) (L2S1)

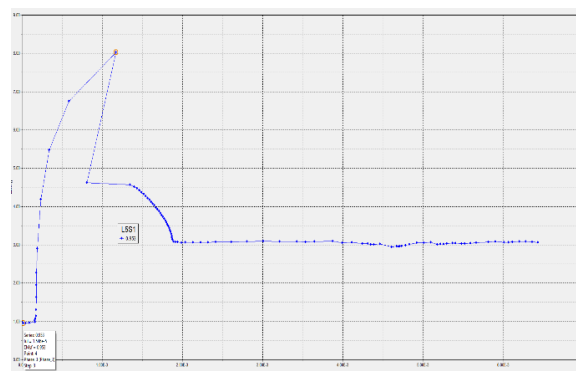


FoS Curve for Soil Model (L2S1)

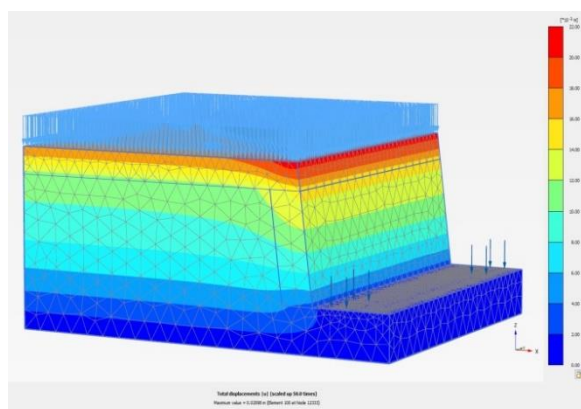
Figure 12. Representation of numerical analysis output for the cut slope L2S1.



Deformed Soil Model (Output) (L5)



FoS Curve for Soil Model (L5)

Figure 13. Representation of numerical analysis output for the cut slope L5.

Deformed Soil Model (Output) (L7)



FoS Curve for Soil Model (L7)

Figure 14. Representation of numerical analysis output for the cut slope L7.**Table 11.** Summary of FoS as per the numerical method.

Sr. No	Location	Slope	FoS	Status
1	L1	S1	0.65	Failed
2		S2	1.0	Severely fractured
3		S3	--	--
4	L2	S1	0.91	Severely fractured
5	L5	-	0.95	Severely fractured
6	L6	-	0.88	Failed
7	L7	-	0.92	Severely fractured

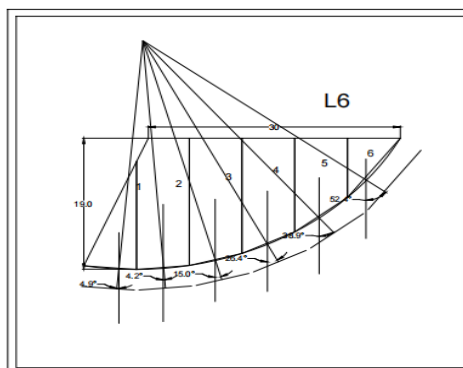
**Figure 15.** Drawing of L6.

Table 12. Properties of soil at slope L6.

C	88.25 kN/m ²	γ_{sat}	23.15 kN/m ³	γ_b	22.5 kN/ m ³
Φ	11.86°	γ_d	21.85 KN/ m ³	μ	67.5 kPa
L	37.2 m				
Slice	Area	Angle α	Weight (A X γ)	Xi	Yi
1	49.1	4.9	1104.75	393.25	94.36
2	114.4	5.3	2574	700.34	237.76
3	109.7	15.7	2468.25	661.1	667.9
4	94.8	26.7	2133	595.67	958.39
5	70	38.9	1575	419.51	989.04
6	29.6	54.1	666	244.11	539.48
		Total		502.33	581.15

Table 13. Summary for FoS as per graphical method.

Sr. No.	Location	Slope	FoS	Status
1		S1	0.86	Failed
2	L1	S2	1.13	Severely Fractured
3		S3	0.99	Severely Fractured
4		S1	0.97	Nearly Failed
5	L2	S2	0.90	Severely Fractured
6		S3	0.98	Severely Fractured
7	L6	-	0.86	Failed

Table 14. Comparison of FoS.

Sr. No.	Location	Slope	FoS (Graphical)	FoS (Numerical)	Status
1		S1	0.86	0.65	Failed
2	L1	S2	1.13	1.0	Severely Fractured
3		S3	0.99	--	Severely Fractured
4	L2	S1	0.97	0.91	Severely Fractured
5	L5	-	-	0.95	Severely Fractured
6	L6	-	0.86	0.88	Failed
7	L7	-	-	0.92	Severely Fractured

Comparison of factor of safety values from graphical method (Bishop's simplified graphical approach) and numerical method for selected cut slopes at various locations along Kasara Ghat. Results show predominantly low factors of safety for most investigated slopes, thereby confirming the inherently unstable nature of this terrain. For Location L1, Slope S1, graphical and numerical factor of safety values equal to 0.86 and 0.65, respectively, are both significantly less than unity, clearly indicating failure conditions. The lower FoS obtained from numerical analysis is an indication that the latter is more sensitive to stress redistribution, pore water pressure effects, and material nonlinearity. FoS values obtained through Numerical analysis will generally be lower or comparable; hence, they would be more applicable to critical design and hazard assessment. Comparison between graphical and numerical analyses shows consistently low factors of safety, with numerical methods tending to give lower values here, as also reported in comparisons of limit equilibrium and finite element analyses [28], [29], and closely matching the observed field failures, underscoring the critical instability of cut slopes in the Kasara Ghat region. The factor-of-safety values reported in Tables 13 and 14 represent a steady-state design-condition assessment intended for long-term prioritization of cut slopes for remedial action. Actual triggering of failure, as observed at Location 6 in September 2025, occurs over a much shorter

timescale (hours) during peak monsoon rainfall. The present FoS-based classification should therefore be interpreted as an indicator of relative susceptibility rather than a predictor of the exact timing of failure.

4.8. Ground Validation of Slope

Two visits were conducted at the same location subsequent to the identification of the cut slope. The initial visit was dedicated to collecting soil and rock samples for conducting the geotechnical investigation and numerical analysis. The factor of safety (FoS) computed from the analysis was markedly below the required value, registering 0.86 and 0.88 by the two methods employed. During the second visit, the slope failed in September 2025, thereby validating and supporting the analysis performed on the selected slope.

The geotechnical samples used in this study were obtained using soil augers, which sample near-surface soil and weathered rock to limited depths. However, Section 2.1 notes that the south-western slopes of the Kasara Ghat valley are affected by active deep-seated landslides associated with the regional NNW-SSE fault and multiple thrust zones. It is therefore possible that failure surfaces are positively indicative of the current situation of the strata at depth.

The study area has historically experienced extreme rainfall events, including the record 944 mm in a single day recorded at the Santacruz IMD station on 26 July 2005. For the severely weathered slopes presently classified with FoS between 0.90 and 1.13 (Table 14), a rainfall event of this magnitude would substantially push several of these slopes towards immediate failure within hours rather than the multi-year timeframe currently estimated under average monsoon conditions. The stability period of severely weathered slopes is therefore highly sensitive to single extreme rainfall events and cannot be considered constant over time.

Table 14 shows that in addition to Location 6 (FoS = 0.86-0.88 failed), slope L1S1 also returned an FoS below unity (0.65-0.86) and is recorded as Failed (where L1S1 and L6 both failed during the September 2025 rainfall). The remaining slopes at L1 (S2, S3) and L2 (S1, S2, S3) have FoS values ranging from 0.90 to 1.13, close to but not below unity in several locations, and are classified as Severely Fractured or 'Nearly Failed' rather than 'Failed'. A FoS marginally below or near 1.0 indicates a state of marginal stability under the assumed monsoon/saturated condition. Whether or not failure actually occurs depends on whether a sufficient triggering event occurs at that specific location and time.

Differences in slope geometry (Table 1; e.g., L2S3 has the steepest measured angle at 57°38') and in localized drainage/seepage conditions also mean that slopes with similar FoS values may respond differently to the same rainfall event. The slopes at L1 (S2, S3) and L2 (S1-S3) are therefore considered to be in a state of marginal stability and are consistent with the Abstract, requiring immediate attention, as

failure may occur during a future high-intensity rainfall event, even though it has not yet occurred.

The instability of the cut slopes investigated can be attributed to the interaction of three principal mechanisms. First, advanced tropical chemical weathering has reduced rock mass compressive strength to 3-22% of the 20 MPa benchmark (Table 10), directly reducing shear strength along potential failure surfaces. Second, the NNW-SSE fault and associated thrust zones introduce pre-existing discontinuities oriented unfavorably relative to several cut-slope faces (notably L2S1-L2S3, slope angles exceeding 50 degrees; Table 1), providing pre-defined failure planes. Third, the low basement infiltration capacity (<3 mm/hr) means intense monsoon rainfall - including events approaching the historical extreme of 944 mm/day - generates a rapid rise in pore water pressure at the soil-rock interface, reducing effective normal stress and acting as the proximate trigger for failure, as evidenced by the September 2025 failures at L1S1 and L6. These findings are broadly consistent with previous studies on slope stability in the Western Ghats. These similarly identify the combination of deep weathering profiles, structurally controlled discontinuities, and monsoon rainfall intensity as dominant controls [2], [3]. The present study extends this by quantifying these factors at individual cut-slope scale and validating the FoS classification against an observed failure.

Photographs of the slope at Location 6 taken before failure in February 2025 and after failure in September 2025 are presented in Fig. 16, providing direct ground validation of the stability analysis.

Slope at Location 6

Coordinates - 19°41'28.8"N 73°30'46.8"E



Date: 02/02/2025



Date: 14/09/2025

Figure 16. Photos of the slope at Location 6 before and after failure in the same year during the summer and rainy seasons.

5. Conclusion

A geotechnical and numerical stability assessment was carried out for cut slopes along NH-3 in the Kasara Ghat section of the Western Ghats. Field surveys, geotechnical laboratory testing, graphical (Bishop's simplified) analysis, and finite element numerical modeling were used to evaluate slope stability at 11 identified locations.

Most of the cut slopes in the study area are steep to near-vertical, with several locations (L2S1, L2S2, L2S3) exceeding the recommended stable angle range for weathered basaltic/igneous rock. Rock samples tested from these slopes show compressive strengths of only about 3- 22% of the 20 MPa benchmark generally required for stable cut slopes, confirming severe weathering-related strength loss attributed to tropical chemical weathering of the basaltic and granitic lithologies.

Combined graphical and numerical analyses indicate that slopes L1S1 and Location 6 (L6) have FoS values below unity (0.65–0.86 and 0.86–0.88, respectively) and both failed during the September 2025 monsoon. This provides direct ground validation of the analysis. Slopes L1S2, L1S3, L2S1, L2S2, L2S3, L5 and L7 have FoS values ranging from approximately 0.90 to 1.13, placing them in a state of marginal stability that warrants immediate remedial attention. Particularly given the sensitivity of pore water pressure (and hence FoS) to high-intensity rainfall events such as the 944 mm/day event recorded historically in this region.

Given the low permeability of the slope materials, surface drainage measures are likely to be more effective than subsurface drains alone in controlling pore water pressure build-up.

At locations where the rock mass has weathered to a soil-like consistency with very low UCS, rock bolts or spikes may not achieve adequate anchorage as there is insufficient competent rock to develop the required bond/pull-out resistance. For such locations, gravity or reinforced-concrete retaining walls at the toe combined with surface protection using shotcrete (sprayed concrete) incorporating weep holes or drainage openings are recommended as alternative stabilization measures. This approach is particularly relevant for the four highly weathered slopes identified in the Abstract as being at risk of failure within the next few years.

The present FoS values derived from models and static loading conditions with conservative shear strength parameters (due to sample disturbance and moisture) should be regarded as a relative susceptibility ranking for prioritizing intervention rather than a precise prediction of failure timing. Future work should incorporate dynamic/traffic-vibration loading and full three-dimensional and discrete element modeling for highly fractured locations. Overall, the findings confirm that cut slopes along the Kasara Ghat section of NH-3 are highly susceptible to rainfall-triggered failure, with at least two locations (L1S1 and L6) having already failed and several others in a state of marginal stability requiring urgent geotechnical intervention.

Use of LLM and AI

During the preparation of this work, the authors used Claude (Anthropic) to improve language, to assist in writing text, and to check references. The authors reviewed and edited all content and take full responsibility for the manuscript.

Data Availability

The data supporting the findings of this study are included in the article.

Funding information

The authors state that no funding was involved.

Author contributions

Both authors contributed equally to the manuscript.

Conflict of interest statement

Authors state no conflict of interest.

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