

# Detailed Investigation and Integrated Slope Stabilization of a Progressive Deep-Seated Landslide on the Damchu-Chukha Highway (Km 19.800), Bhutan: A Case Study

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## ABSTRACT

A progressive landslide at Km 19.800 on the Damchu-Chukha section of the Phuentsholing-Thimphu highway (AH-48), Bhutan, evolved from pavement cracking, embankment settlement and gabion-wall distress into complete loss of the original carriageway and subsequent deformation of an upslope temporary diversion. The failure history indicated that the instability was not confined to the road embankment but was consistent with a deeper soil-weathered-rock mass movement. A scientific investigation was therefore undertaken using field reconnaissance, one borehole, laboratory characterization, seismic refraction tomography (SRT), rock-mass classification using RMR and GSI, and two-dimensional limit-equilibrium analyses under static, rainfall-saturated and pseudo-static seismic conditions. The interpreted profile comprised approximately 6.0-6.5 m of soil/fill over thick highly and moderately weathered rock of poor rock-mass quality. Unreinforced analyses indicated inadequate stability, with rainfall saturation producing the most adverse response. The proposed mitigation combines a Shored Reinforced Soil (SRS) wall with soil nails/rock bolts, mechanically connected geosynthetic reinforcement, high-tensile mesh, erosion-control treatment, and integrated surface and subsurface drainage. Final analyses reported overall factors of safety of 1.709, 1.300 and 1.241 for static, rainfall and seismic cases, respectively, satisfying the case-specific acceptance criteria adopted for the respective loading conditions. The case demonstrates a diagnostic transition from symptom-based retaining-wall repair to mechanism-based stabilization of the complete slope-road system. It further illustrates the application of a mechanically coupled SRS-soil-nail system where conventional reinforced-soil geometry is constrained by steep Himalayan terrain. The limited analytical reserve under saturated conditions makes drainage performance, construction-stage verification and post-construction monitoring integral to the design philosophy.

**Keywords:** deep-seated landslide; Himalayan highway; slope stabilization; shored reinforced soil wall; soil nailing; seismic refraction tomography; rainfall-induced instability.

## INTRODUCTION

Mountain highways in the Himalayas are exposed to steep relief, fractured and weathered rock masses, intense seasonal rainfall, active tectonics and difficult drainage conditions. Landslide classification and process-based interpretation are therefore fundamental to selecting appropriate mitigation (Cruden and Varnes 1996). For transportation corridors, the engineering problem is intensified where cut slopes, historic fill placement, widening and retaining structures modify natural slope geometry while monsoonal infiltration changes pore-water pressure and effective stress. Risk-informed slope management consequently requires consideration of both instability likelihood and the consequences of failure (Fell et al. 2008).

Geotechnical case studies are particularly valuable when field observations, investigation data and analysis are assembled into a clear account of why a failure occurred and how the adopted intervention addresses the governing mechanism (Marr et al. 2014). For active or progressive landslides, this distinction is important because failure of a visible retaining structure may be only the surface expression of a much larger ground movement. Soil nails and mechanically stabilized earth systems have been successfully combined for landslide stabilization where in-situ reinforcement and reconstructed fill must act together (Turner and Jensen 2005), but

the applicability of such systems depends strongly on-site geometry, groundwater, reinforcement bond and construction sequence.

The Damchu-Chukha road forms part of AH-48, the principal highway connection between Phuentsholing and Thimphu. At Km 19.800, distress first manifested as carriageway settlement, longitudinal cracking and deformation of a valley-side gabion retaining system. The failure subsequently resulted in loss of the original carriageway. Continued deformation of a temporary diversion constructed upslope was diagnostically significant because it showed that instability extended beyond the original retaining-wall footprint and road embankment. These observations prompted a detailed geotechnical investigation and development of a permanent stabilization scheme (Shelke 2026).

The technical contribution of this case study is twofold. First, it demonstrates how progressive distress of a highway retaining system can be differentiated from a larger rainfall-sensitive slope failure through integrated interpretation of failure chronology, borehole information, SRT and limit-equilibrium analysis. Second, it documents the application of a mechanically coupled Shored Reinforced Soil-soil nail system for restoration of roadway width where the reinforcement length required for a conventional reinforced-soil wall is constrained by steep Himalayan terrain. The paper presents the failure evolution, investigation strategy, parameter derivation, stability assessment, mitigation concept, design uncertainties and practical implications for mechanism-based management of recurrent mountain-road failures.

## **SITE CONDITIONS AND FAILURE EVOLUTION**

### **Terrain, geology and hydrological setting**

The site lies on a steep valley flank in western Bhutan, where the road formation requires cutting into the hill side and support of valley-side fill. The regional setting comprises metamorphic Himalayan rocks affected by weathering, fracturing and discontinuities. At the failure location, the road had been formed partly over fill supported by gabion retaining structures. The combination of steep geometry, weak near-surface material, fractured weathered rock and limited valley-side toe support created a slope sensitive to changes in groundwater and imposed loading.

The corridor is strongly influenced by southwest monsoon rainfall. Infiltration through pavement cracks, fill and rock discontinuities can raise pore-water pressure, reduce effective stress and increase the unit weight of the mobilized mass. Preferential seepage paths are particularly relevant in heterogeneous fill and fractured weathered rock. The observed acceleration of distress during wet periods and the stability analyses discussed subsequently are consistent with rainfall-sensitive behaviour. In this paper, rainfall influence is represented through the design phreatic-surface assumptions contained in the project analysis; no site-specific transient seepage calibration is claimed.

### **Progressive failure sequence and interpreted mechanism**

During 2024, the principal warning signs were differential settlement of the carriageway, longitudinal and transverse cracking, outward deformation of the valley-side embankment, distress in the gabion wall and localized seepage. The simultaneous movement of the road and retaining structure suggested that wall distress was a consequence of movement in the supporting ground rather than an isolated structural failure. In October 2025, continued deterioration compounded by prolonged rainfall culminated in collapse of the valley-side support and washout of the original carriageway. The affected zone extended beyond the wall limits and developed a pronounced failure morphology.



Fig. 1. Pre-failure field evidence showing tension cracking and progressive valley-side distress at Km 19.80.

Traffic was restored through a temporary upslope diversion. Subsequent deformation of this diversion provided a critical diagnostic observation: moving the road platform a short distance uphill had not escaped the active landslide mass. By December 2025, continued erosion, downslope movement and deterioration of the diversion confirmed progressive instability. The combined field evidence and analytical results were therefore interpreted as being consistent with a deep-seated, predominantly rotational/global failure mechanism involving soil/fill and weathered rock rather than a shallow embankment failure (Shelke 2026). The actual slip surface was not independently instrumented; the term 'deep-seated' is used here as an engineering interpretation supported by the observed movement pattern and analytical model.

This interpretation governed design philosophy. Reconstructing a larger gravity or gabion wall at the original location could have restored local support but would not, by itself, intercept a failure surface extending behind and beneath the road. The engineering objective therefore became restoration of the original alignment within a stabilized composite slope, with separate measures assigned to global stability, local surface instability, roadway support and groundwater control.

## GEOTECHNICAL INVESTIGATION AND GROUND MODEL

### Investigation programme

The investigation combined direct and indirect methods because the site geometry and geological variability made reliance on a single technique inappropriate. Inputs included field reconnaissance, one borehole at the slope location, laboratory characterization, SRT and representative critical cross-sections. SRT was used to extend interpretation laterally beyond the discrete borehole and to identify contrasts associated with soil/fill and different weathering grades. The final ground model reconciled geophysical response with borehole information and visible field conditions rather than treating seismic velocity as a stand-alone lithological classifier.

Table 1. Geotechnical Investigation Programme, Purpose and Design Outputs for the Km 19.800 Landslide

| Investigation input | Extent available in project records | Primary purpose                  | Output used in design            |
|---------------------|-------------------------------------|----------------------------------|----------------------------------|
| Field               | Repeated observations               | Failure morphology, seepage, and | Failure-mechanism interpretation |

|                             |                                                                                    |                                                |                                              |
|-----------------------------|------------------------------------------------------------------------------------|------------------------------------------------|----------------------------------------------|
| reconnaissance              | during failure evolution                                                           | constructability                               |                                              |
| Borehole                    | One borehole at the landslide site                                                 | Direct stratigraphy and rock recovery          | Weathering profile, core recovery and RQD    |
| Laboratory characterization | Representative recovered rock samples; test count not stated in manuscript records | Intact rock strength                           | Representative UCS values                    |
| SRT                         | Survey lines across the landslide zone; exact line lengths not reproduced here     | Lateral subsurface interpretation              | Weathering/stratigraphic boundaries          |
| Rock-mass classification    | HWR, MWR and SWR horizons                                                          | Engineering characterization                   | RMR and GSI                                  |
| 2-D stability model         | Critical section near Ch. 19+975                                                   | Evaluate unreinforced and reinforced stability | FoS under static, rainfall and seismic cases |

A representative critical section near Ch. 19+975, with an overall vertical extent of about 60 m, was selected for detailed stability assessment. The section included the upper hill-side slope, the central road-restoration zone and the lower valley-side slope. This zoning was necessary because potential failure surfaces could pass through or beneath more than one treatment zone.

### Spatial representativeness and uncertainty of the ground model

The subsurface characterization relies on one borehole supplemented by seismic refraction tomography (SRT), surface observations and rock-mass classification. Although the borehole establishes a direct vertical record of weathering, recovery and rock quality, it cannot independently resolve lateral variations in stratigraphy, discontinuity persistence, groundwater conditions or the three-dimensional geometry of the potentially mobilized mass. SRT extends interpretation between investigated locations, but seismic velocity is not a unique indicator of material strength or lithological boundaries, particularly in fractured and variably saturated rock. Accordingly, the interpreted critical section is treated as a design-stage engineering ground model rather than a fully verified three-dimensional representation of the landslide. The uncertainty is particularly relevant to the depth of possible slip surfaces, the continuity of weaker horizons and the hydraulic response of the weathered rock. Geological and groundwater conditions encountered during reinforcement drilling and excavation should therefore be systematically recorded and reconciled with the model; material departures should prompt reassessment of the reinforcement and drainage details.

### Subsurface model and rock-mass characterization

Table 2. Interpreted Subsurface Stratigraphy and Rock-Mass Recovery Characteristics at Km 19.80

| Engineering stratum             | Approx. thickness | Core recovery | RQD          |
|---------------------------------|-------------------|---------------|--------------|
| Topsoil/fill                    | 6.0-6.5 m         | Not reported  | Not reported |
| Highly weathered rock (HWR)     | 15-17 m           | 0-25%         | 0%           |
| Moderately weathered rock (MWR) | 14-15 m           | 25-50%        | 0%           |
| Slightly weathered rock (SWR)   | Continuous below  | Not specified | 0%           |

The interpreted profile shows that the road was not founded immediately on competent rock. A thick weak near-surface zone was underlain by substantial highly and moderately weathered rock. The reported zero RQD in recovered rock zones indicates very poor rock-mass quality despite increasing intact strength with depth. Representative intact UCS values were 8, 22 and 24 MPa for HWR, MWR and SWR, respectively. Corresponding RMR values were 23, 27 and 30, classifying the horizons as poor rock; GSI values adopted in the design were 18, 22 and 25.

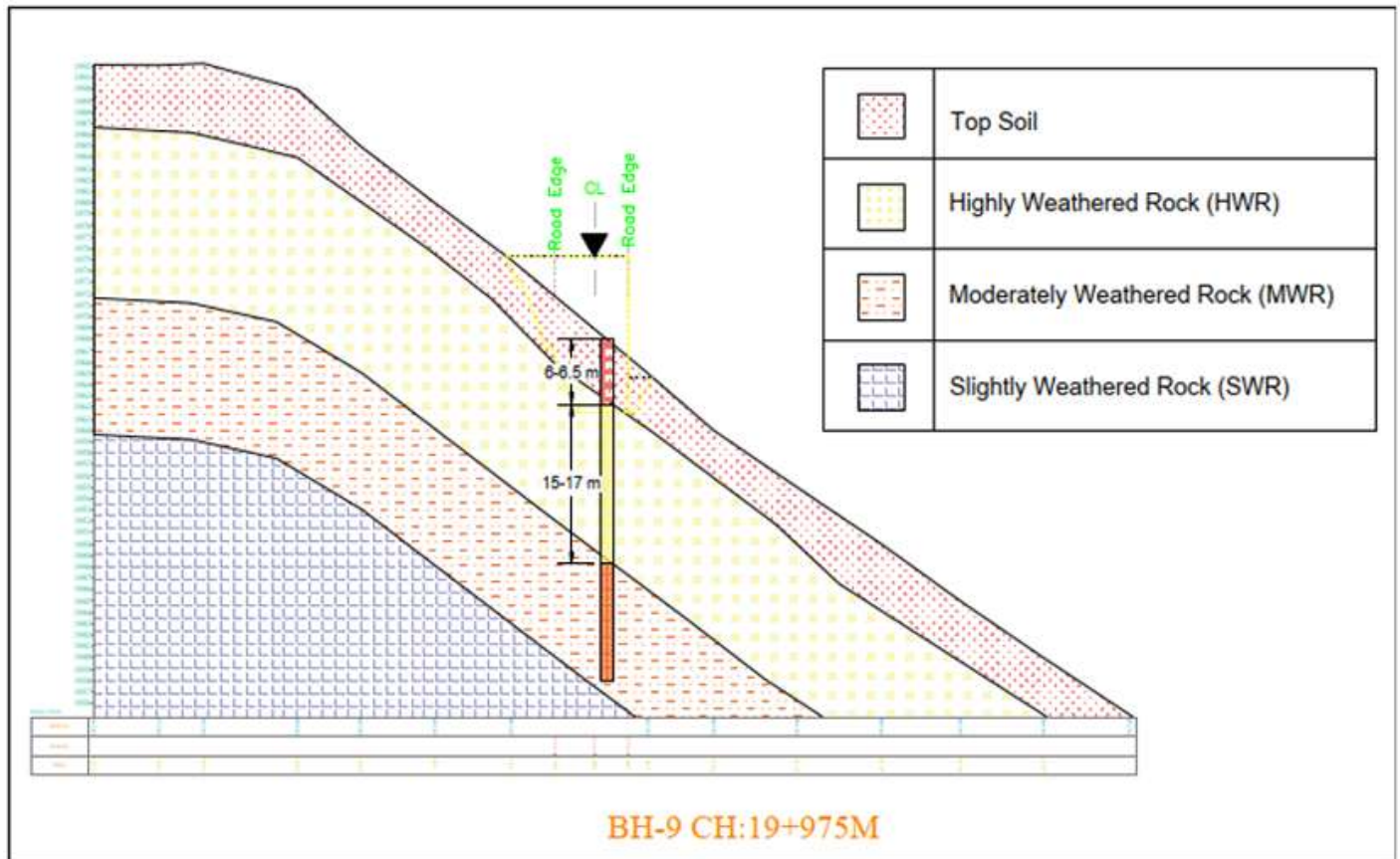


Fig. 2. Interpreted subsurface stratification developed from borehole data and SRT at the critical section (adapted from Shelke 2026).

### Derivation and traceability of design parameters

Equivalent rock-mass strength parameters were developed from intact strength, GSI, material constants and unit weight using Hoek-Brown rock-mass relationships implemented in RSData. The Hoek-Brown framework provides a means of relating intact rock strength and rock-mass quality to non-linear rock-mass strength and, where required by limit-equilibrium software, to equivalent Mohr-Coulomb parameters over a specified stress range (Hoek and Brown 1997; Hoek et al. 2002). In the project model, the derived friction angles were conservatively capped at 35 degrees while the corresponding cohesion values were retained. The soil/fill parameters were typical adopted values rather than results of direct shear or triaxial testing; this is an important uncertainty in the available dataset.

Table 3. Rock-Mass Characterization and Design Parameters Adopted in the Limit-Equilibrium Stability Analyses

| Material     | $\gamma$ (kN/m <sup>3</sup> ) | UCS (MPa) | RMR | GSI | mi | c (kPa) | $\phi$ (°) | Basis                               |
|--------------|-------------------------------|-----------|-----|-----|----|---------|------------|-------------------------------------|
| Topsoil/fill | 20                            | -         | -   | -   | -  | 3       | 32         | Adopted typical values              |
| HWR          | 24                            | 8         | 23  | 18  | 15 | 74      | 35         | Rock-mass derivation; $\phi$ capped |

|     |    |    |    |    |    |     |    |                                     |
|-----|----|----|----|----|----|-----|----|-------------------------------------|
| MWR | 24 | 22 | 27 | 22 | 15 | 144 | 35 | Rock-mass derivation; $\phi$ capped |
| SWR | 24 | 24 | 30 | 25 | 18 | 213 | 35 | Rock-mass derivation; $\phi$ capped |

The table provides an audit trail from observed/tested rock-mass descriptors to the parameters used in stability analysis. It should not be interpreted as eliminating uncertainty: RMR/GSI assignment, equivalent Mohr-Coulomb conversion, spatial variability and the adopted soil/fill parameters all influence calculated FoS. For this reason, the design relies on construction-stage verification of actual weathering, groundwater and reinforcement bond rather than treating the numerical model as a complete representation of field behaviour.

## STABILITY ASSESSMENT

### Analysis framework

Two-dimensional limit-equilibrium analyses were undertaken in Rocscience SLIDE 2D using the Bishop simplified method for rotational failure surfaces (Shelke 2026). Bishop's method satisfies overall moment equilibrium and is widely used for circular slip surfaces (Bishop 1955); the broader limitations of slice-based methods and sensitivity to assumptions regarding interslice forces are well recognized (Fredlund and Krahn 1977; Duncan et al. 2014). Three principal loading conditions were considered: (1) static condition with an interpreted normal water table; (2) rainfall/near-saturated condition represented by a raised phreatic surface; and (3) pseudo-static seismic condition using a horizontal coefficient  $k_h = 0.2$ . The model used the representative critical section and the material parameters summarized above.

The project records available for this manuscript do not provide sufficient detail to reproduce the complete slip-surface search settings, number of trial surfaces or all software search parameters; accordingly, these are not inferred here. The paper reports the documented analysis outcomes and focuses on their engineering interpretation. As target factors of safety vary with loading condition and the specific stability check being performed, FoS values achieved are evaluated against the corresponding case-specific acceptance criteria rather than being interpreted as universally applicable design requirements.

### Method selection, search domain and analytical verification

The Bishop simplified method was adopted for the principal analyses because the observed movement pattern, including road settlement, valley-side displacement and distress extending upslope of the original alignment, was consistent with a predominantly rotational or compound failure mechanism. Bishop's method provides a well-established moment-equilibrium solution for circular slip surfaces (Bishop 1955). It does not, however, demonstrate that the critical surface is circular. In weathered and discontinuous rock, non-circular or compound sliding along stratigraphic and weathering interfaces remains plausible; generalized methods such as Spencer or Morgenstern-Price are therefore appropriate for complementary verification (Fredlund and Krahn 1977; Duncan et al. 2014).

The modeled cross-section near Ch. 19+975 includes the upper hill-side slope, restored road formation and lower valley-side slope. A defensible search domain must permit potential surfaces to enter behind the observed upper distress and exit on the valley-side slope, including surfaces passing beneath the road-restoration zone. The precise entry/exit coordinates, search-grid dimensions, trial-surface density, refinement settings and convergence tolerances are not documented in the records available for this manuscript and are therefore not reported as verified model inputs. Reproducibility requires extraction of these settings from the original SLIDE 2D files, together with a plan/section showing the search boundaries. A comparison with a generalized non-circular method is recommended before final design verification; no such additional numerical comparison is claimed here.

### Unreinforced response

The analyses confirmed that the observed failure could not be addressed as a local wall problem. A local excavated/unreinforced configuration produced a reported FoS of 0.561 against a target of 1.40. For the upper

slope, the reported FoS values were 1.042 under static groundwater, 0.633 under rainfall saturation and 0.742 under seismic loading, all below the respective criteria used in the design. A separate overall-slope stage gave 1.222, 1.035 and 0.934 for static, rainfall and seismic conditions. These values correspond to different model stages and should not be interchanged, but collectively they demonstrate inadequate stability without structural inclusions.

The reduction from 1.042 to 0.633 in the upper-slope model when the phreatic surface was raised is particularly instructive. It provides an analytical explanation for the observed association between wet conditions and accelerated movement. Drainage was therefore treated as a primary stabilization mechanism rather than an ancillary detail.

### Reinforcement modelling and final stability

The principal reinforcement comprised 25-mm-diameter grouted bars. For slope reinforcement, a tensile resistance of 147.79 kN per nail was considered. A grout-to-ground bond value was adopted from FHWA guidance for comparable materials and was required to be verified by field pull-out testing (FHWA 2015). This verification is essential because bond resistance in fractured weathered rock depends on drilling, hole cleaning, grout placement, saturation and local discontinuity conditions.

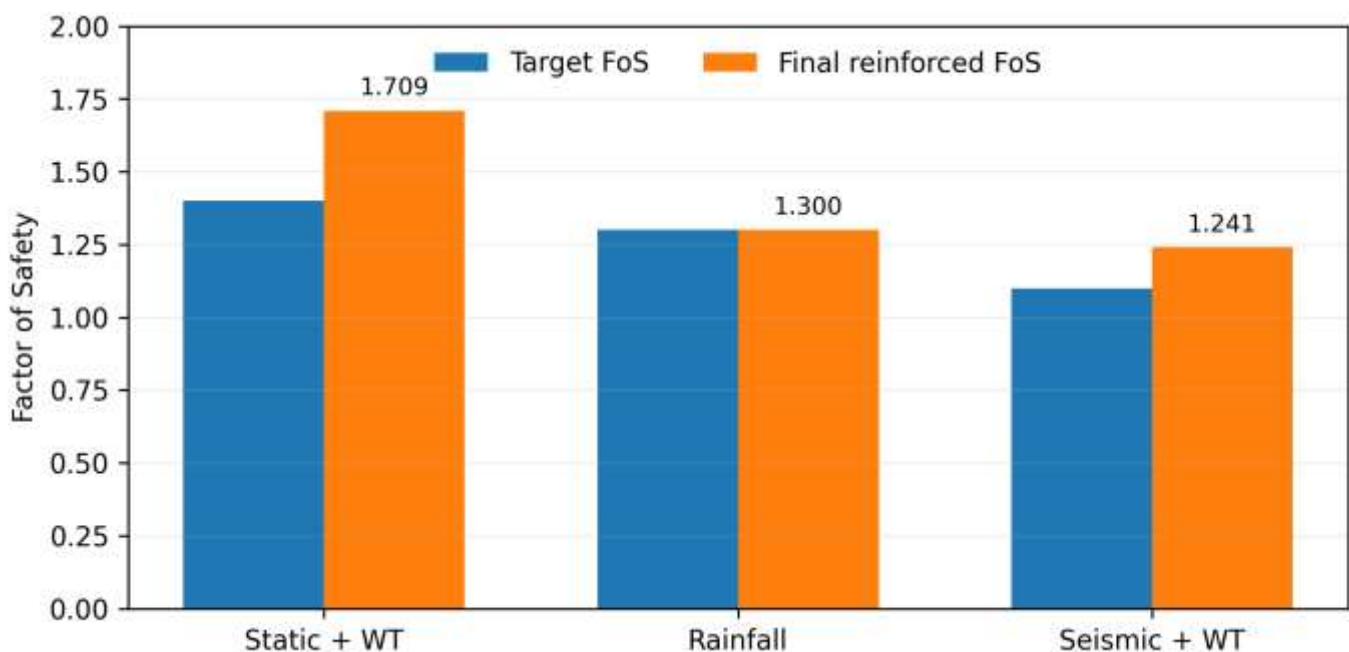


Fig. 3. Case-specific target factors of safety and final reinforced-slope results reported for the critical section.

The reinforced model satisfied the stated criteria under all three loading cases. However, the rainfall FoS equals the adopted target of 1.30, leaving essentially no numerical reserve. The implications of uncertainty in groundwater, soil/fill strength and reinforcement bond are therefore examined through the sensitivity-assessment framework described below. The present paper reports the documented reference FoS values only; independent perturbation results are not available and are not represented as completed calculations.

### Parametric sensitivity assessment: framework and required verification

The principal uncertain inputs affecting the reinforced-slope response are the rainfall-condition phreatic surface, shear strength of the soil/fill and mobilized grout-ground bond resistance of the nails/bolts. The documented reinforced rainfall case (FoS = 1.300) is the reference configuration. A one-factor-at-a-time sensitivity programme is recommended in which the groundwater surface is displaced by -2, -1, +1 and +2 m relative to the adopted rainfall surface, where geometrically and hydraulically feasible; soil/fill cohesion is varied around the adopted 3 kPa (for example, 2-5 kPa); soil/fill friction angle around the adopted 32° (for example, 28-34°); and nail-ground bond resistance is examined at 60%, 80% and 120% of the design value.

These are proposed perturbations, not measured variability or results of completed model runs. The other parameters, geometry, reinforcement layout and analysis settings should remain unchanged for each individual run.

For each case, the minimum computed FoS, critical slip-surface location and change relative to the reference model should be recorded, with particular attention to whether FoS falls below the rainfall acceptance criterion of 1.30. Groundwater sensitivity is best expressed as change in FoS per metre of phreatic-level shift; normalized sensitivity may be used for positive-valued material and bond parameters. A summary table and sensitivity plot should be included once the original SLIDE 2D model has been rerun. Because no rerun outputs are available in the present source records, quantitative sensitivity rankings or revised FoS values cannot responsibly be inferred. Thus this subsection documents the required analytical programme and its significance but does not claim that the editorial request for a completed parametric study has been fully met.

### **Rainfall infiltration and transient seepage considerations**

Rainfall-induced slope response depends on rainfall intensity and duration, antecedent moisture, hydraulic conductivity, water-retention characteristics, fracture-controlled flow and drainage efficiency. Infiltration through cracked pavement, heterogeneous fill and fractured weathered rock may produce localized or delayed pore-pressure increases along potential failure surfaces. At Km 19.800, the temporal association of prolonged rainfall with accelerating distress supports a hydrological contribution, but the available field records do not establish a calibrated rainfall-pore-pressure relationship.

The adverse rainfall analysis used an elevated phreatic surface in the two-dimensional limit-equilibrium model. This is an assumed groundwater-loading scenario, not a transient infiltration simulation, and it does not identify the rainfall duration or intensity required to produce the assumed pore pressures. A more rigorous extension would involve transient finite-element seepage analysis with rainfall flux, surface-runoff conditions, initial groundwater levels, saturated/unsaturated hydraulic conductivity and soil-water retention inputs, followed by transfer of time-dependent pore-water pressures to stability analyses (Fredlund and Rahardjo 1993; Iverson 2000). Such modelling should be calibrated or bounded using site-specific rainfall and piezometric observations; absent those inputs, numerical predictions of rainfall-trigger timing would imply unjustified precision.

Pending such analysis, the raised-phreatic-surface case should be interpreted as a design-stage adverse scenario and tested through groundwater-level sensitivity calculations. Given that the final rainfall FoS equals its target, drain continuity, outlet maintenance, groundwater observation and post-storm deformation surveillance are design-critical measures rather than routine ancillary activities.

### **Integrated Slope Stabilization Scheme**

#### **Design concept and treatment zoning**

The proposed solution comprises three interacting treatment zones. The upper hill-side slope is reinforced using long soil nails/rock bolts, high-tensile mesh, drainage and erosion protection. The central zone restores the road using a Shored Reinforced Soil (SRS) wall mechanically connected to the reinforced in-situ slope. The lower valley-side slope is stabilized using long reinforcement, mesh, drainage and erosion-control measures. This arrangement is intended to intercept potential failure surfaces that could bypass a treatment confined to the roadway.

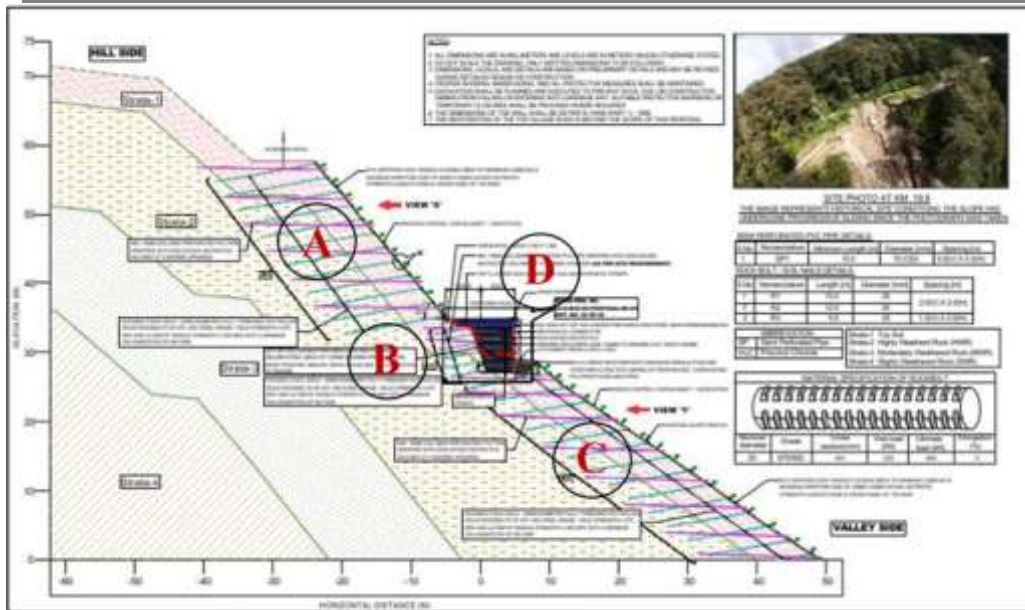


Fig. 4. Concept cross-section for the integrated mitigation scheme: A = hill-side soil nails/rock bolts and protective mesh; B = reinforcement at the SRS wall zone; C = valley-side soil nails/rock bolts and protective mesh; D = Shored Reinforced Soil wall (adapted from Shelke 2026).

### Engineering rationale for the SRS wall

A conventional reinforced-soil wall generally requires a reinforcement length that is a substantial fraction of wall height; IRC:SP:102-2014 provides Indian highway guidance for design and construction of reinforced soil walls (IRC 2014), while IS 18591:2024 provides a current Indian code of practice for geosynthetic reinforced soil structures (BIS 2024). At Km 19.800, steep terrain and restricted working width limited the space available for a conventional configuration. The SRS concept addresses this constraint by constructing a relatively narrow reinforced-fill zone in front of a nailed/bolted backslope and providing a positive mechanical connection between geosynthetic reinforcement and in-situ reinforcement. The new fill and stabilized backslope can therefore act compositely while reducing excavation into the unstable slope.

The wall system includes galvanized mesh facing, selected granular backfill, polyester geosynthetic reinforcement straps, grouted bars, mechanical connectors, filter geotextiles and internal/rear drainage. The facing retains the reinforced fill locally and controls deformation; the geosynthetic straps transfer tensile load through the fill and, through the connector, into the reinforced in-situ ground. The connector is therefore part of the structural load path, not merely a detailing accessory. This distinction is central to the proposed system.

### Functional comparison with conventional treatments

Table 4. Functional Comparison of Alternative Slope-Retention and Stabilization Systems for the Km 19.800 Site

| System                            | Primary function                         | Limitation at Km 19.800                                 | Role in proposed scheme                                       |
|-----------------------------------|------------------------------------------|---------------------------------------------------------|---------------------------------------------------------------|
| Gabion/gravity wall               | Local retention and toe support          | Does not by itself intercept a deep/global slip surface | Not relied upon as the principal global stabilization measure |
| Conventional reinforced-soil wall | Retained fill and formation widening     | Full reinforcement width constrained by steep backslope | Design concept modified to a shored/narrow configuration      |
| Conventional soil-nailed slope    | In-situ reinforcement of existing ground | Does not independently create required road             | Provides reinforcement to upper/lower slope and               |

|                   |                                                | formation width                                 | backslope                                 |
|-------------------|------------------------------------------------|-------------------------------------------------|-------------------------------------------|
| SRS + nails/bolts | Coupled road support and in-situ reinforcement | Requires reliable connectors, bond and drainage | Principal composite stabilization concept |

### Soil nails, rock bolts and surface stabilization

The SRS wall zone uses 25-mm bars approximately 6 m long at variable horizontal spacing, while the upper and lower slope zones use approximately 15-m-long bars at about 2 m × 2 m spacing. The longer reinforcement is intended to cross potential failure surfaces and develop resistance in deeper material. Actual bonded length and capacity must be confirmed against ground encountered during drilling and by suitability/proof tests. FHWA soil-nail guidance emphasizes the importance of construction control and verification testing because assumed bond values may not represent site-specific conditions (FHWA 2015).

High-tensile galvanized wire mesh is proposed over exposed upper and lower slope faces to restrain shallow ravelling and detached weathered blocks. An erosion-control blanket beneath or in conjunction with the mesh limits migration of fine material and supports revegetation. These surface systems are not substitutes for deep reinforcement: they address shallow detachment and erosion, whereas global stability is provided principally by soil/rock reinforcement, the composite SRS system and pore-pressure control.

### Surface and subsurface drainage

The drainage concept combines interception of upslope runoff, controlled road drainage, horizontal/sub-horizontal drains and drainage layers or geocomposites behind the reinforced system. The engineering objective is to prevent uncontrolled infiltration and reduce pore-water pressure within the soil-weathered-rock mass. Drain outlets must remain inspectable and maintainable. Because the rainfall analysis provides the smallest reserve, blockage or deterioration of drainage is a geotechnical performance issue rather than a routine maintenance defect. The emphasis on drainage and slope-specific mitigation is consistent with the broader hill-road and landslide-management framework of IRC:SP:48-2023 and IRC:SP:106-2015 (IRC 2015, 2023).

### Construction sequence, road restoration and verification

Slope reinforcement is intended to proceed in controlled stages, generally stabilizing exposed slope portions before extensive excavation and constructing the SRS wall progressively from the bottom upward. Critical controls include borehole depth and inclination, ground conditions encountered, grout quality and take, bar centralization, corrosion protection, pull-out testing, backfill gradation and density, geosynthetic placement, connector capacity and drainage continuity. Temporary excavation geometry and plant surcharge require separate attention because construction stages may be less stable than the completed configuration.

Restoration of the original alignment requires the stabilized slope and reconstructed pavement to be treated as a single serviceability problem. Differential settlement at the transition between existing formation, reinforced fill and rigid drainage or pavement elements can reintroduce cracking that subsequently becomes an infiltration path. Road-restoration works should therefore include controlled removal of loose material, careful preparation of interfaces, selected fill in thin compacted lifts, positive drainage away from the pavement edge and monitoring of initial post-construction settlement.

Construction itself should be treated as an additional investigation stage. Nail drilling exposes actual weathering, groundwater and discontinuity conditions; pull-out tests provide site-specific bond evidence; and grout take may identify fractured or open ground. These observations should be compared with the design ground model. Material deviations should trigger review of reinforcement length, spacing, drainage or construction sequence before the relevant work is concealed.

## Field verification of grout-ground bond resistance

The reinforcement design depends on the mobilization of adequate bond resistance between grouted bars and the highly to moderately weathered rock. A staged testing programme is recommended in accordance with the principles of the FHWA Soil Nail Walls Reference Manual (FHWA 2015). Before production installation, sacrificial verification nails should be constructed using the proposed drilling and grouting methods in representative ground horizons, with the test bond length sufficiently isolated to permit meaningful interpretation of grout-ground resistance. The previously proposed minimum of three preliminary tests may serve as an initial project-specific provision, but the final number must reflect ground variability and the approved design/testing specification; it is not presented as a universal FHWA minimum.

Verification testing should apply controlled incremental loads, record load-displacement response and include appropriate load-hold/creep observations and unloading measurements. Acceptance should be assessed against the project-specified design bond resistance and displacement/creep criteria, using calibrated equipment and documented procedures. During production, proof tests at a frequency specified in the approved quality-assurance plan should confirm workmanship and bond performance. Installation logs should document hole depth and inclination, encountered strata, groundwater strikes, grout take, bar protection and connector details. Where tests fail or encountered ground differs materially from the design model, bonded length, spacing or drilling/grouting procedures should be reviewed before proceeding. No site pull-out test results are reported in the present paper; the programme is a construction-stage verification recommendation.

## ENGINEERING DISCUSSION

### From local retaining failure to global slope instability

The principal engineering lesson from Km 19.800 is that visible failure of a retaining structure does not necessarily identify the governing geotechnical mechanism. Gabions remain appropriate for many local mountain-road applications; their distress at this site does not imply a general deficiency in gabion technology. Rather, the sequence of pavement cracking, embankment settlement, retaining-wall deformation, complete loss of the road and subsequent movement of the upslope diversion indicated instability extending into a larger soil-weathered-rock mass. Rebuilding the same structural form without addressing that mechanism would therefore have treated a symptom.

The SRS solution is technically attractive because it couples a road-support structure with in-situ slope reinforcement where full conventional reinforcement length is unavailable. Similar case-study literature demonstrates the broader feasibility of combining soil nails with MSE-type systems for active landslide stabilization (Turner and Jensen 2005). At Km 19.800, effectiveness depends on continuity of the load path from facing to geosynthetic strap, connector, nail/bolt and stable ground. Loss of capacity at an interface can alter composite behaviour, so specification and quality control must address the system rather than individual products in isolation.

### Hydrological control and lifecycle performance

The analyses reinforce the governing role of hydrology. Raising the phreatic surface produced the most adverse response in the unreinforced upper-slope model, and the final rainfall FoS only meets the stated criterion. This supports a lifecycle view of drainage: catch drains, horizontal drains and outlets require pre-monsoon cleaning, post-storm inspection and recording of discharge. A stabilized slope with blocked drainage no longer corresponds to the groundwater condition assumed in design.

Long-term performance also depends on durability of buried and exposed components. Galvanized steel bars, mesh and connectors require appropriate corrosion protection for the humid mountain environment, while polymeric reinforcement requires consideration of installation damage, creep and environmental reduction factors. Filters and drainage geocomposites must retain soil while maintaining transmissivity. Construction records should identify material batches, installed locations, pull-out test results and any field damage or replacement, because embedded reinforcement becomes difficult to inspect after completion.

## Limitations, uncertainty and observational verification

The available investigation has important limitations. One borehole cannot fully characterize lateral variation or establish the three-dimensional geometry of the mobilized mass; SRT improves spatial interpretation but requires geological calibration. Soil/fill shear parameters were adopted rather than directly measured, rock-mass parameters were empirically derived, and groundwater was represented by assumed phreatic surfaces rather than measured transient pore pressures. The two-dimensional Bishop analysis addresses circular mechanisms but does not exclude non-circular or compound surfaces; the original search settings and independent generalized-method comparison have not been recovered. The actual slip surface was inferred rather than instrumentally mapped. These limitations do not negate the engineering interpretation but constrain the precision and transferability of the reported FoS values, especially the rainfall FoS of 1.300.

An observational approach is therefore appropriate during implementation. Drilling records, groundwater strikes, grout consumption during reinforcement installation, pull-out tests, survey movement and drain response should be compared continuously with design assumptions. Where the observed ground model differs materially, reinforcement or drainage should be reviewed before work proceeds. Monitoring should focus on quantities that test the design assumptions: displacement of the wall and slope, cracks beyond the treated zone, groundwater/piezometric response where instruments are feasible, and drain discharge during major rainfall. LiDAR and other remote-sensing techniques can complement field instrumentation for large unstable slopes (Jaboyedoff et al. 2012; Scaioni et al. 2014).

## ENGINEERING IMPLICATIONS FOR HIMALAYAN HIGHWAY PRACTICE

The case supports four transferable principles for highway agencies working in landslide-prone mountain terrain. First, recurrent distress requires an escalation criterion. Progressive longitudinal cracking, repeated settlement, retaining-wall deformation, new seepage, toe erosion or movement of a temporary diversion should trigger geotechnical investigation rather than repeated local repair. The diagnostic question is whether the visible defect is the failure mechanism or merely its surface expression.

Second, investigation effort should be proportional to failure scale and consequence. Boreholes provide direct information but sparse spatial coverage; geophysics can extend interpretation but requires calibration; geological mapping establishes structural controls; and groundwater observations constrain the hydrological model. The value comes from integrating these datasets into a testable ground model. Where uncertainty materially affects design, sensitivity analysis and field verification are preferable to false precision.

Third, mitigation should be assigned by function. Deep reinforcement intercepts potential slip surfaces; reinforced fill and retaining systems restore formation width; mesh controls shallow detachment; erosion blankets limit surface degradation; and drainage controls hydrological loading. A single measure should not be credited with functions it cannot physically perform. This functional separation improves design review and exposes single-point vulnerabilities in composite schemes.

Fourth, construction and maintenance must remain part of geotechnical risk management. Drilling and pull-out testing verify assumptions before reinforcement is concealed, while drainage inspection and deformation monitoring test whether the completed system continues to behave within the design envelope. For recurrent or high-consequence sites, a project-level geohazard inventory containing chainage, failure history, geology, investigation data, drainage, stabilization works and monitoring records would support risk-based prioritization. The transferable lesson is not that SRS walls should become a default treatment, but that stabilization technology should follow the identified failure mechanism.

## CONCLUSIONS

1. The Km 19.800 failure evolved from road and retaining-wall distress into progressive movement affecting both the original alignment and an upslope diversion. The governing problem was therefore interpreted as a larger soil-weathered-rock slope instability rather than a local wall failure.

2. Integrated interpretation of field observations, one borehole, SRT and rock-mass characterization identified approximately 6.0-6.5 m of soil/fill over thick poor-quality weathered rock. The combination of direct and indirect investigation was more informative than either dataset in isolation, although lateral variability remains an uncertainty.
3. Limit-equilibrium analyses showed inadequate unreinforced stability under static, rainfall and seismic cases. Rainfall saturation produced the most pronounced deterioration in the upper-slope model, establishing groundwater control as a primary design function.
4. The proposed stabilization combines an SRS wall, soil nails/rock bolts, mechanically connected geosynthetic reinforcement, high-tensile mesh, erosion control and surface/subsurface drainage. Final overall FoS values of 1.709, 1.300 and 1.241 for static, rainfall and seismic conditions met the adopted case-specific criteria, but the rainfall case has essentially no numerical reserve above its target.
5. The broader engineering implication is that recurrent mountain-road distress should trigger mechanism-based investigation and observational verification rather than repeated reconstruction of the visible failed element. At sites with limited analytical reserve, drainage maintenance, field verification of reinforcement bond and post-construction deformation monitoring are integral parts of the stabilization system.
6. The sensitivity of the proposed scheme to groundwater elevation, soil/fill shear strength and grout-ground bond resistance warrants explicit parametric verification using the original numerical model. The single-borehole ground model, simplified rainfall phreatic surface and circular slip-surface assumption further support construction-stage pull-out testing, observational verification and, where suitable hydraulic data become available, transient seepage assessment. These additional analyses and tests are recommendations; no unreported numerical or field results are implied.

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## Data and Source Statement

Case-specific numerical values, ground-model parameters, stability results and mitigation details reported in this paper are derived from the project investigation and design records cited as Shelke (2026). No additional laboratory or field data have been invented for this manuscript. Where the available records do not provide sufficient detail for independent reproduction, the limitation is stated explicitly.

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