

Integrated Geotechnical and Seismic Stability Assessment of Tropical Residual Slopes in Sitinjau Lauik, West Sumatra, Indonesia

Okri Asfino Putra¹, Hamdeni Medriosa², Fajar Nugroho², Muhammad Hamzah Fansuri¹, Pungky Dharma Saputra¹, Anasya Arsita Laksmi³

¹*Department of Civil Engineering, Republic of Indonesia Defense University, Bogor 16810, Indonesia.*

²*Department of Civil Engineering, Padang Institute of Technology, Padang 25173, Indonesia.*

³*Department of Water Resource Engineering, Republic of Indonesia Defense University, Bogor 16810, Indonesia*

Abstract

Sitinjau Lauik, a national road corridor in West Sumatra, Indonesia, is one of the most landslide-prone segments of the Bukit Barisan volcanic belt. This study assesses the geotechnical and seismic stability of three tropical residual soil slopes at this location by combining cone penetration testing (CPT), laboratory index testing, consolidated-drained direct shear testing (DST), and Morgenstern–Price limit-equilibrium analysis under static and pseudo-static seismic loading. CPT profiles revealed a weak layer ($q_c = 0.4\text{--}0.6$ MPa) at 1.2–2.8 m depth in Slope 1. This slope also has a high plasticity index ($PI = 39.7\%$), a high void ratio ($e = 1.50$), and a low dry density ($\gamma_d = 10.06$ kN/m³), resulting in lower shear-strength parameters ($c' = 17.8$ kPa, $\phi' = 19.2^\circ$) than Slopes 2 and 3 ($PI < 10\%$). Static factors of safety (FS) were 1.08, 1.54, and 1.49 for Slopes 1, 2, and 3, respectively. A pseudo-static coefficient of $kh = 0.15$, consistent with regional peak ground acceleration zoning, reduced the FS values to 0.77, 1.23, and 1.17, making Slope 1 unstable during earthquakes. At $kh = 0.20$, none of the slopes met the minimum FS of 1.1 required by the Indonesian National Standard (SNI) 8460:2017. High plasticity, a high void ratio, and low CPT resistance distinguished the critical Slope 1 from the other slopes. Because only three slopes were examined, these findings are site-specific; nevertheless, they provide a basis for prioritizing slope remediation and early warning along this corridor.

Keywords: Tropical residual soil, Slope stability, Cone penetration test, Seismic loading, Sitinjau Lauik, Shear strength

1. Introduction

Landslides are among the most destructive natural hazards in tropical mountain regions, causing significant loss of life, infrastructure damage, and economic disruption each year [1]. In Indonesia, the convergence of steep topography, intense seasonal rainfall, deeply weathered volcanic profiles, and high seismic activity creates conditions that are exceptionally conducive to slope instability [2], [3]. West Sumatra Province lies within one of the world's most seismically active corridors, bounded to the west by the Sunda Megathrust

subduction zone and traversed along its entire length by the Sumatran Fault System [3]. The combination of these tectonic forces with deeply weathered residual soils derived from Quaternary volcanic material constitutes a persistent geohazard that challenges engineers across the region.

The Sitinjau Lauik route, a critical segment of the Sumatra National Road connecting Padang to Solok, has experienced repeated slope failures that periodically disrupt transportation and commerce [4]. The road traverses a series of steep, naturally formed slopes

comprising tropical residual soils developed from andesitic and basaltic parent rocks of the Bukit Barisan volcanic arc [3]. Previous failures at this location have ranged from shallow translational slides to deep rotational slumps, with many triggered by prolonged rainfall or earthquake shaking [1], [5]. Despite the site's strategic importance, systematic geotechnical investigations integrating field testing, laboratory characterization, and quantitative seismic stability evaluation remain limited in the published literature.

Tropical residual soils exhibit behaviour that is distinctly different from transported (alluvial or marine) soils of comparable index properties [3]. The in-situ fabric inherited from parent-rock structure, the presence of partial saturation [6], [7], variable degrees of cementation at inter-particle contacts, and the non-linear relationship between plasticity and shear strength all contribute to complexity in characterisation and stability prediction [8]. The cone penetration test (CPT) has been shown to be particularly effective in profiling these soils because it provides continuous, high-resolution measurements of tip resistance (q_c) and sleeve friction (f_s) that reflect variations in weathering grade and density with minimal sample disturbance [9].

Seismic slope stability analysis for residual soil slopes in Sumatra requires careful consideration of local ground-motion hazard. Following the 2009 Padang earthquake (moment magnitude, M_w 7.6), updated national seismic hazard maps for Indonesia [10] revised Peak Ground Acceleration (PGA) values for West Sumatra significantly upward, yet many existing infrastructures assessments pre-date these revisions. The pseudo-static method [5], [11], while conservative, remains the most widely adopted approach for preliminary and intermediate-level seismic stability screening, particularly where site-specific ground-response data are unavailable [5]. Previous research on Indonesian slope stability has largely focused on either empirical correlation between rainfall and failure [1], [2] or simplified stability assessments based on laboratory strength data alone [4], [12]. Few studies have integrated CPT profiling, grain-size characterisation, plasticity measurements, and direct shear test results within a unified framework that also addresses seismic loading. This gap is particularly significant for the Sitinjau Lauik area, where slope geometries, soil heterogeneity, and seismic exposure collectively elevate the risk of catastrophic failure.

The present study addresses this gap by (i) characterising the geotechnical properties of three representative residual soil slopes at Sitinjau Lauik using CPT and laboratory testing; (ii) determining Mohr–Coulomb shear-strength parameters from consolidated-drained direct shear tests; (iii) evaluating static and pseudo-static seismic slope stability using the Morgenstern–Price limit-equilibrium method; and (iv) identifying the soil characteristics that most strongly control slope susceptibility. The findings are intended to support evidence-based decisions on prioritising slope remediation and implementing early-warning systems along this nationally significant corridor.

2. Materials and Methods

2.1. Regional geology and tectonic setting

West Sumatra lies within the Sunda Arc, one of the world's most seismically and volcanically active tectonic settings. The province is bounded to the southwest by the Sunda Megathrust — the subduction interface between the Indo-Australian and Eurasian plates — and is transected by the right-lateral Sumatran Fault System (SFS), which runs parallel to the volcanic arc along the spine of the Bukit Barisan range [3]. The SFS accommodates approximately 23 mm/year of strike-slip motion distributed across multiple fault segments, several of which pass within 50 km of Padang [3], [10]. Historical seismicity records include the 1926 Padang Panjang (surface-wave magnitude, M_s 7.0), 2007 Bengkulu (M_w 8.4), and 2009 Padang (M_w 7.6) events, all of which caused widespread slope failures in West Sumatra [10].

The Bukit Barisan volcanic chain consists of Quaternary andesitic to basaltic stratovolcanoes and their associated pyroclastic deposits, lahar sequences, and interbedded tuffaceous sediments [3]. Prolonged tropical weathering under a humid climate (annual rainfall $> 4,000$ mm in the upland zone) has produced deeply developed lateritic and saprolitic residual soil profiles that may extend to depths exceeding 20 m before unweathered bedrock is encountered. These profiles are characterised by high void ratios, elevated fine-grained fractions, variable plasticity, and strong sensitivity to pore-pressure changes induced by rainfall or earthquake shaking [1], [3], [5]. Figure 1 shows the geological map of the Padang and Solok region.

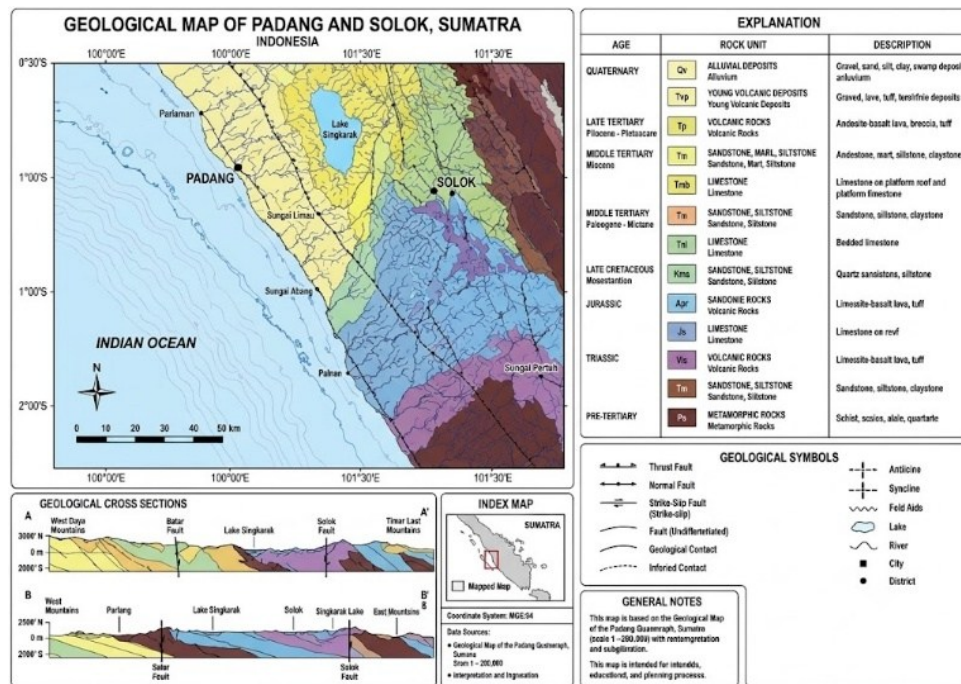


Figure 1. Geological map of the Padang and Solok region.

2.2. Site description

The three investigation slopes (Slope 1, Slope 2, Slope 3) are located along the Sitinjau Lauik road corridor between approximate coordinates 0°56'S, 100°28'E, at elevations ranging from 780 to 950 m above sea level. Slope 1 is situated on the lower section of the corridor near a historically active failure zone, with a measured inclination of 34° and an estimated height of 18 m. Slope 2 is located approximately 1.2 km uphill, with an inclination of 29° and height of 22 m; the face is partially vegetated with pioneer shrubs following a previous cut. Slope 3 is the uppermost of the three, with an inclination of 31° and height of 20 m, and comprises natural ground with sparse vegetation. All three slopes exhibit surface signs of previous shallow movement, including tension cracks, irregular micro-topography, and disturbed drainage patterns consistent with creep and prior translational sliding [13].

2.3. Field investigation: Cone penetration testing

Cone penetration tests (CPT) were performed at each slope using a 10-cm² standard electric cone at a penetration rate of 20 mm/s in accordance with ASTM International (formerly the American Society for Testing and Materials, ASTM) standard D5778. At each location,

one CPT sounding was advanced to refusal or a practical termination depth of 8 m; a single sounding per slope was adopted as a rapid screening approach, co-located with the corresponding trial pit and direct-shear samples so that the CPT, index-property, and strength data at each slope refer to the same soil mass. Given the known spatial heterogeneity of tropical residual soils, a single sounding cannot by itself establish the lateral continuity of the identified weak layer across the full slope height, and this is treated explicitly as a study limitation (Section 4). Continuous profiles of cone tip resistance (q_c) and sleeve friction (f_s) were recorded at 2-cm intervals, and the friction ratio ($Fr = f_s/q_c \times 100\%$) was calculated at each depth. Soil behaviour type (SBT) was classified using the measured, non-normalised q_c - Fr relationship against the classification chart of Robertson [9]; because the investigated depths were shallow (≤ 8 m) and the corresponding overburden stresses low, the non-normalised chart was judged adequate for this application, and fully normalised parameters (normalised cone tip resistance, Q_{tn} , incorporating in-situ vertical effective stress) were not computed. This is noted as a simplification relative to Robertson's more recent normalised soil behaviour type (SBT_n) procedure. Weak zones were identified as intervals where q_c fell below 1.0 MPa. This threshold is not proposed as a universal criterion for tropical residual soils; it was adopted here on

the basis of the sharp, internally consistent contrast between the 0.4–0.6 MPa interval and the surrounding 1.5–8 MPa profile at Slope 1, and is broadly consistent with the association between $q_c < 1.0$ MPa and highly compressible, low-strength residual material reported for comparable tropical weathering profiles [14], [15].

2.4. Laboratory testing

Disturbed and undisturbed block samples were retrieved from trial pits excavated adjacent to each CPT location, with one undisturbed block and a companion disturbed bulk sample obtained from each slope at the depth representative of the identified weak layer (0.5–2.5 m). Undisturbed blocks (approximately 200 mm × 200 mm × 200 mm) were hand-trimmed, wrapped in cling film, wax-sealed, and labelled in the field immediately after excavation to preserve natural water content and structure during transport, following standard practice for tropical residual soils; samples showing visible disturbance during extraction were discarded and re-sampled. Natural water content (w_n , mass of water as a percentage of dry soil mass) was determined for each sample immediately upon unwrapping (ASTM D2216). All laboratory tests were performed in accordance with the relevant ASTM standards. Specific gravity (G_s , ratio of soil-solid density to the density of water) was determined by the pycnometer method (ASTM D854). Bulk density (γ_b , total unit weight) and dry density (γ_d) were calculated from measured mass and volume of the undisturbed samples. Void ratio (e , ratio of void volume to solid volume) was computed from G_s and γ_d . Liquid limit (LL) and plastic limit (PL) were determined by the Casagrande cup and thread-rolling methods (ASTM D4318), respectively, and plasticity index (PI) was derived as $PI = LL - PL$. Particle-size distribution was determined by combined sieve and hydrometer analysis following the procedures of ASTM D6913 (sieve) and ASTM D7928 (hydrometer), which superseded the withdrawn ASTM D422 method used in earlier practice.

Consolidated-drained direct shear tests (DST) were conducted on undisturbed specimens trimmed to 60 mm × 60 mm × 20 mm (ASTM D3080). Prior to shearing, specimens were inundated and allowed to saturate under a small seating load for at least 24 h, with free vertical drainage provided through saturated porous stones at the top and base of the shear box; the applied displacement

rate of 0.05 mm/min was selected to be slower than the rate implied by the consolidation characteristics of the finer, higher-plasticity Slope 1 material, following the drained-rate guidance associated with ASTM D3080, so as to maintain fully drained conditions in all specimens. Three specimens from each slope were sheared under normal stresses (σ) of 25, 50, and 75 kPa — one specimen per normal stress, with no replicate specimens at a given stress level, which limits assessment of specimen-to-specimen variability and is noted as a study limitation (Section 4) — yielding nine tests in total (DST1–DST9). Specimens were consolidated for a minimum of 24 h under the applied normal stress (σ), with consolidation judged effectively complete when the rate of vertical settlement became negligible, before shearing to a maximum horizontal displacement of approximately 8 mm or until a clear post-peak reduction in shear resistance was observed, whichever occurred first. Peak shear stress (τ_{peak} , the maximum shear stress recorded during each test) was used to define the Mohr–Coulomb failure envelope via linear regression, and the effective cohesion (c') and effective friction angle (ϕ') were extracted; residual strength was not evaluated. These parameters represent the best-fit linear Mohr–Coulomb criterion over the applied normal-stress range. The schematic diagram of the direct shear test apparatus is illustrated in Figure 2.

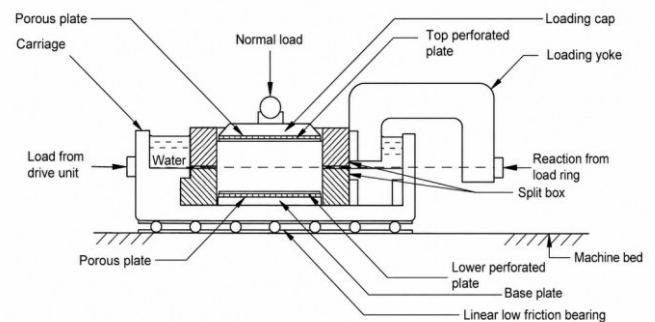


Figure 2. Schematic diagram of the direct shear test apparatus.

2.5. Slope stability analysis

Two-dimensional limit-equilibrium (LE) slope stability analyses were performed using SLOPE/W (GeoStudio) with the Morgenstern–Price method [16], which satisfies both force and moment equilibrium and is considered among the most rigorous of the LE procedures for general-shaped slip surfaces [8]; unlike the simplified Bishop method [17], which satisfies moment equilibrium

only, it does not require the slip surface to be circular. Model geometry for each slope was constructed from the field-measured slope height and inclination (Section 2.2: Slope 1, 18 m at 34°; Slope 2, 22 m at 29°; Slope 3, 20 m at 31°), extended by a crest and toe berm of approximately one slope height in each direction to avoid boundary effects on the searched slip surfaces. Soil parameters used in each analysis corresponded to the slope-specific values listed in Table 1 and Table 2, applied as a single homogeneous layer per slope consistent with the sampled depth range. Groundwater conditions were represented by a piezometric surface set at 1.0 m below the slope face; this depth was assumed from field observations of shallow seepage and saturated soil at the base of the trial pits during the wet-season investigation, rather than derived from installed piezometric instrumentation, and its influence on the reported factor of safety (FS) values was not tested through sensitivity analysis and Road-traffic surcharge (see Limitations). The critical (minimum) FS under static loading was determined for each slope.

Table 1. Physical properties of residual soil samples from the three investigated slopes.

Parameter	Slope 1	Slope 2	Slope 3
Specific Gravity, G_s	2.56	2.59	2.56
Bulk Density, γ_b (kN/m ³)	17.59	16.75	16.34
Dry Density, γ_d (kN/m ³)	10.06	11.66	10.81
Void Ratio, e	1.50	1.18	1.32
Natural Water Content, w_n (%)	38.6	33.4	33.7
Plasticity Index, PI (%)	39.7	7.35	8.27

Table 2. Mohr–Coulomb shear-strength parameters derived from direct shear test results (c' = effective cohesion; ϕ' = effective friction angle; R^2 = coefficient of determination of the linear regression).

Parameter	Slope 1	Slope 2	Slope 3
Effective Cohesion, c' (kPa)	17.8	20.7	18.8
Effective Friction Angle, ϕ' (°)	19.2	31.7	30.2
R^2 of Linear Fit	0.9991	0.9999	0.9999

The critical slip surface of each slope was located by an automatic search using the entry-and-exit technique in

SLOPE/W. Entry points were defined along the crest and upper slope face and exit points along the lower slope face and toe, and trial circular slip surfaces were generated between every combination of entry and exit points over a range of radius increments. Each trial sliding mass was divided into 30 vertical slices, and a half-sine interslice force function was adopted in the Morgenstern–Price formulation, with the scaling factor λ (the multiplier applied to the interslice force function) iterated until force and moment equilibrium converged within a tolerance of 0.01. The slip surface with the lowest FS was taken as critical, and it was checked that this surface did not lie on the boundary of the search zone; where it did, the entry and exit ranges were extended and the search repeated. The same search settings were used for the pseudo-static cases, so that the critical surface could change position under seismic loading.

Seismic stability was evaluated using the pseudo-static method [5], [11], in which an equivalent horizontal body force equal to $k_h W$ (where k_h is a dimensionless horizontal seismic coefficient and W is the weight of the sliding mass) is added to the equilibrium equations. Two values of k_h were applied: 0.15 and 0.20. These values were selected to bracket the design PGA for the Sitinjau Lauik area given in the 2017 Indonesian Seismic Hazard Map [10], combined with a site-amplification factor appropriate for soft residual soil (Site Class D). A vertical seismic coefficient $k_v = 0$ was adopted, consistent with common pseudo-static practice for preliminary assessments [5].

The percentage reduction in FS due to seismic loading was computed to quantify the seismic vulnerability of each slope. Following the Indonesian National Standard (Standar Nasional Indonesia, SNI) 8460:2017 on geotechnical design requirements, a minimum FS of 1.1 is adopted here as the design acceptance criterion for pseudo-static slope stability, distinct from the theoretical static-equilibrium failure boundary of $FS = 1.0$; this distinction is applied consistently when interpreting the seismic results below.

3. Results and Discussion

3.1. CPT profiles

Figure 3 presents the qc profiles obtained at the three slope locations.

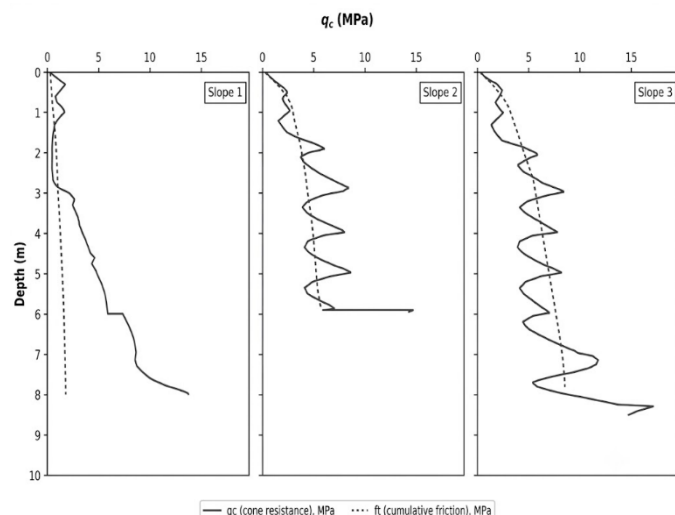


Figure 3. CPT profiles obtained at the three slope locations.

At Slope 1, q_c values were consistently below 1.5 MPa throughout the upper 4 m, with a pronounced minimum of approximately 0.4–0.6 MPa between depths of 1.2 and 2.8 m. This interval coincides with a highly weathered, clay-rich horizon that was also identified visually in the trial pit as a plastic, dark-brown material with slickensided surfaces indicative of previous displacement. Friction ratios in this interval exceeded 4%, further confirming the presence of a soft to firm clayey layer (SBT Zone 3, Robertson [9]). Below 4 m, q_c increased progressively toward values of 5–8 MPa at 7 m depth, representing the transition from residual soil to partially weathered parent rock.

At Slopes 2 and 3, q_c profiles showed higher and more uniform resistances throughout the investigated depth. Mean q_c values in the critical upper 3 m were approximately 2.8 MPa (Slope 2) and 2.3 MPa (Slope 3), consistent with a medium-density, silty sand to sandy silt residual soil (SBT Zone 4–5). No persistent weak layer comparable to that at Slope 1 was identified, though isolated intervals of slightly reduced q_c (1.0–1.5 MPa) were noted at 1.5–2.0 m depth in Slope 3, likely reflecting localised variations in weathering intensity.

3.2. Physical properties

Table 1 summarizes the physical properties of the three soil samples. Slope 1 is distinguished by a substantially higher plasticity index ($PI = 39.7\%$) compared with Slopes 2 ($PI = 7.35\%$) and 3 ($PI = 8.27\%$). According to the Unified Soil Classification System (USCS), Slope 1 soil classifies as a high-plasticity clay (group symbol CH),

whereas Slopes 2 and 3 classify as low-plasticity silt or silty sand (group symbols ML/SM, respectively). The void ratio of Slope 1 ($e = 1.50$) is markedly higher than those of Slopes 2 ($e = 1.18$) and 3 ($e = 1.32$), indicating a looser, more compressible fabric. The dry density of Slope 1 ($\gamma_d = 10.06 \text{ kN/m}^3$) is the lowest of the three, consistent with its high void ratio and clay-rich composition [3]. Specific gravity values (2.56–2.59) are typical of iron-oxide-stained tropical residual soils derived from andesitic parent material. Table 1 also lists the natural water content (w_n) for Slopes 1, 2, and 3, respectively.

3.3. Grain-size distribution

Particle-size analysis (Figure 4) revealed that Slope 1 contains a significantly higher fines content (clay + silt < 0.075 mm) of approximately 71%, with a clay fraction (< 0.002 mm) of 38%. The median particle size D_{50} of Slope 1 is approximately 0.008 mm (fine silt). In contrast, Slopes 2 and 3 possess fines contents of approximately 42% and 48%, respectively, with D_{50} values of 0.032 mm (Slope 2) and 0.024 mm (Slope 3). The high clay fraction in Slope 1 is consistent with its elevated plasticity and cohesive behaviour, and contributes to reduced hydraulic conductivity, which promotes pore-pressure accumulation during rainfall infiltration events [1], [18]. The sandy silt gradations of Slopes 2 and 3 indicate better drainage capacity and are generally associated with higher friction angles and lower susceptibility to pore-pressure-induced instability [5].

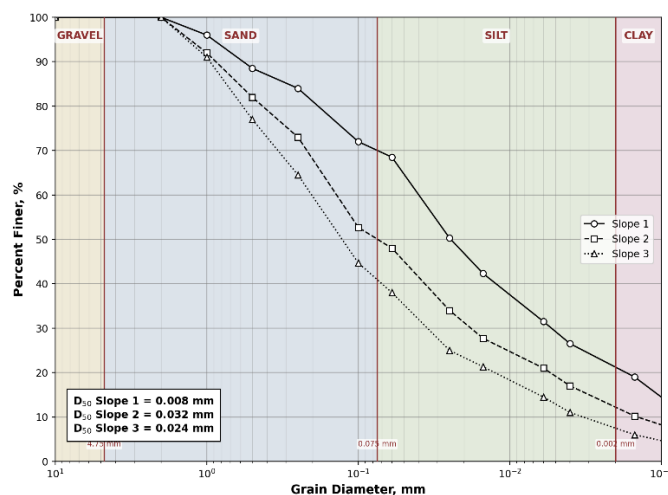


Figure 4. Particle-size distribution of the three residual soil samples.

3.4. Direct shear test results and shear strength parameters

The shear-strength parameters obtained in this study are broadly consistent with those reported for tropical residual soils in comparable settings. Rahardjo et al. [2] reported ϕ' values of 22–30° for Singapore residual soils of comparable plasticity; the ϕ' values for Slopes 2 and 3 (31.7° and 30.2°) lie slightly above the upper end of this range, whereas that for Slope 1 (19.2°) falls well below it, consistent with its markedly higher plasticity index.

The low ϕ' value for Slope 1 (19.2°) is at the lower end of the reported range for high-PI tropical clays and underscores the critical nature of this slope. The shear stress–time response for each specimen is illustrated in Figure 5; these curves are presented to demonstrate that a stable peak shear stress was reached and sustained (rather than continuing to rise at the end of the test, which would indicate an underestimated peak) and to confirm the

absence of testing artefacts such as premature slippage or irregular load-cell response.

For each specimen, the peak shear stress used in the Mohr–Coulomb regression (Table 3) was taken as the maximum recorded shear stress on this curve, occurring at or before the target displacement noted in Section 2.4.

Table 3. Direct shear test results: specimen identification, applied normal stress, and measured peak shear stress (τ_{peak}).

Test ID	Slope	σ (kPa)	τ_{peak} (kPa)
DST1	1	25	26.8
DST2	1	50	35.1
DST3	1	75	44.3
DST4	2	25	36.2
DST5	2	50	51.5
DST6	2	75	67.1
DST7	3	25	33.4
DST8	3	50	47.9
DST9	3	75	62.5

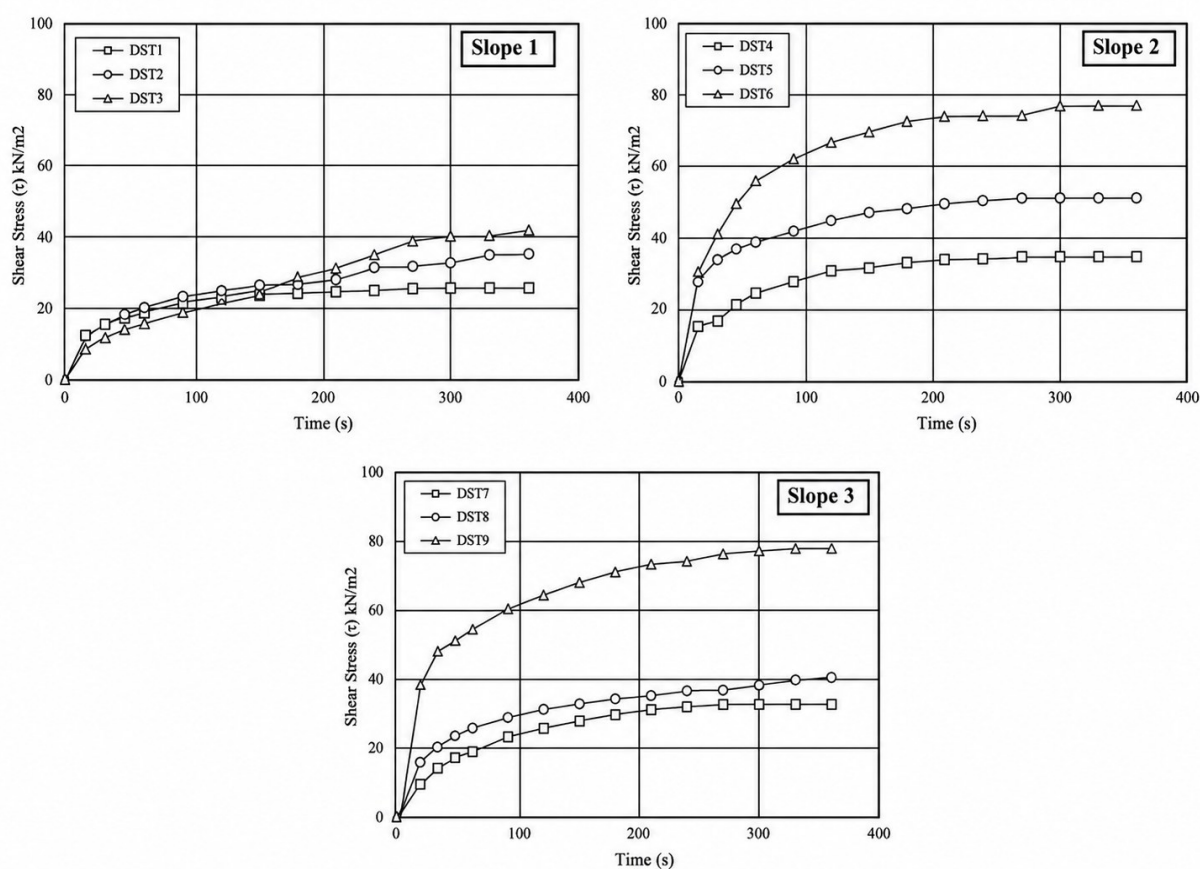


Figure 5. Relationship between shear stress and time for each slope.

3.5. Slope stability analysis results

Table 4 presents the computed factors of safety. The FS values (1.08–1.54 static) are consistent with those reported by Suryo et al. [12] and Pramusandi et al. [4] for natural cut slopes along Indonesian national highways, confirming that marginal static stability with high seismic vulnerability is a recurrent pattern for this class of slope.

Table 4. Summary of static and pseudo-static seismic factors of safety for the three slopes (FS = factor of safety; kh = horizontal seismic coefficient; Δ FS% = percentage reduction in FS relative to the static FS).

Slope	FS (Static)	FS (kh=0.15)	FS (kh=0.20)	Δ FS% (0.15)	Δ FS% (0.20)
1	1.08	0.77	0.71	28.7	34.3
2	1.54	1.23	1.08	20.1	29.9
3	1.49	1.17	1.04	21.5	30.2

Under static conditions, Slope 1 is marginally stable (FS = 1.08), while Slopes 2 and 3 show adequate static stability (FS = 1.54 and 1.49, respectively). Application of a seismic coefficient $kh = 0.15$ reduces the FS of Slope 1 to 0.77 — below the theoretical static-equilibrium threshold of 1.0, indicating failure [5] — whereas Slopes 2 and 3 retain FS values of 1.23 and 1.17, respectively. Applying the SNI 8460:2017 pseudo-static design acceptance criterion of FS > 1.1 rather than the theoretical FS = 1.0 boundary, Slopes 2 and 3 (FS = 1.23 and 1.17) satisfy the code criterion, Slope 3 by a narrow margin. At $kh = 0.20$, Slope 2 (FS = 1.08) and Slope 3 (FS = 1.04) both fall below the SNI 8460:2017 criterion of FS > 1.1, although both remain above the theoretical failure threshold of FS = 1.0.

Under the most severe credible seismic scenario for this region, therefore, none of the three slopes meets the design stability criterion, but only Slope 1 (FS = 0.71) falls below the theoretical limit-equilibrium failure threshold. The percentage reduction in FS attributable to seismic loading ranges from 20.1% to 34.3%, with Slope 1 exhibiting the greatest proportional sensitivity. These reductions are consistent with those reported by Jibson [5] for pseudo-static analyses of natural slopes in high-seismicity regions. The model configuration and the critical slip surface obtained for each slope are shown in Figure 6.

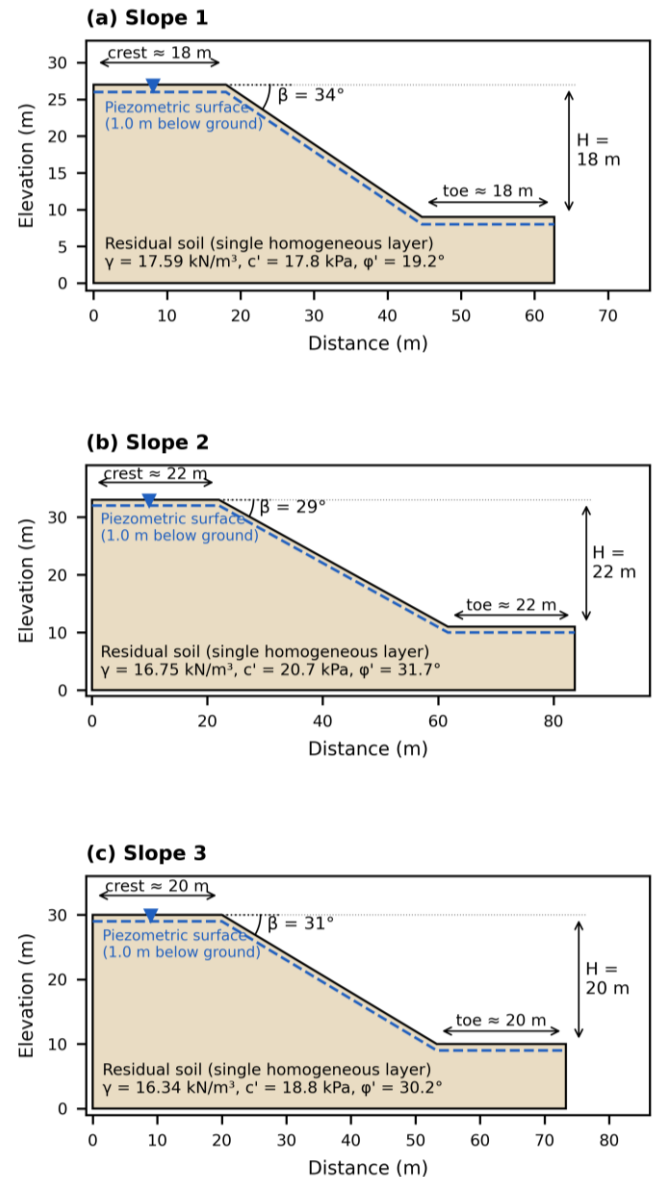


Figure 6. Slope-stability model configurations for (a) Slope 1, (b) Slope 2, and (c) Slope 3, showing the slope geometry, soil unit and parameters, and the assumed piezometric surface 1.0 m below the ground surface.

The large sensitivity of Slope 1 to seismic loading (28.7% FS reduction at $kh = 0.15$) relative to Slopes 2–3 (20–21%) reflects the interaction between the low static FS and the destabilising effect of the pseudo-static force. When the starting static FS is already close to 1.0, even a modest seismic coefficient can push the system below the failure threshold. This underscores the danger of evaluating slope stability under static conditions alone in high-seismicity regions such as West Sumatra [3], [5], [10]. The 2009 Padang earthquake, which reached intensities of VII–VIII on the Modified Mercalli Intensity (MMI) scale in the Sitinjau Lauik area, triggered numerous slope failures that matched this pattern: slopes

with marginal static stability became the primary failure locations [10], [19].

At $kh = 0.20$, Slopes 2 and 3 both fall below the SNI 8460:2017 design criterion of $FS > 1.1$ (although both remain above the theoretical $FS = 1.0$ threshold), implying that in a near-fault scenario or under site amplification conditions more severe than assumed, widespread instability across all three slopes could occur. This finding aligns with regional observations by Jibson [5] and Hung et al. [19], who emphasise that earthquake-triggered landslides disproportionately affect slopes with $FS < 1.5$ under static conditions. The adoption of $kh = 0.20$ as an upper-bound scenario is supported by the probabilistic seismic hazard analysis of Irsyam et al. [10] for the Padang region.

3.6. Influence of plasticity index on shear strength and stability

The most striking finding of this study is the contrast between Slope 1 and the other two slopes in virtually every measured parameter. Slope 1's PI of 39.7% places it firmly in the high-plasticity category, which is associated with the dominance of expansive clay minerals in the fine fraction; direct mineralogical analysis (e.g., X-ray diffraction) was not performed in this study, and this inference should therefore be treated as indicative rather than confirmed. High PI is well established as an inverse predictor of effective friction angle: as PI increases, the failure envelope rotates toward lower ϕ' values as plate-like clay particles align parallel to the shear plane, reducing inter-particle friction. The resulting $\phi' = 19.2^\circ$ for Slope 1 is substantially below the slope angle (34°), meaning that even a modest reduction in effective normal stress — through pore-pressure build-up, seismic shaking, or progressive strength loss — is sufficient to trigger failure [1], [2].

The elevated void ratio ($e = 1.50$) of Slope 1 further indicates a metastable, open fabric that is susceptible to collapse under undrained shearing or cyclic loading [20], [21]. This fabric is a characteristic feature of tropical residual soils that have developed in situ without significant compaction, and it contributes to the relatively low qc values observed in the CPT profile at this location. The low dry density ($\gamma_d = 10.06 \text{ kN/m}^3$) reduces the net stabilising weight component acting on potential slip

surfaces, further limiting FS. Collectively, these properties identify Slope 1 as a geotechnically high-risk slope that warrants priority attention.

3.7. CPT as an indicator of slope vulnerability

The CPT profiles corroborate the laboratory characterisation and provide additional spatial context. The persistent low-qc zone (0.4–0.6 MPa) between 1.2 and 2.8 m depth at Slope 1 delineates a specific layer that is both mechanically weak and likely to control the geometry of any potential failure surface. This layer coincides with the depth range from which DST samples were obtained and where slickensided surfaces were visually confirmed in the trial pit — physical evidence of previous shear displacement [14]. Robertson's [9] SBT framework classifies this interval as Zone 3 (clay to silty clay), consistent with the PI and grain-size data.

For Slopes 2 and 3, the higher and more uniform qc profiles indicate better-consolidated, less plastic soil without a discrete weak layer. However, the slightly lower qc interval at 1.5–2.0 m depth in Slope 3 deserves monitoring attention, particularly if rainfall infiltration or seismic shaking should locally elevate pore pressures. These results demonstrate the value of CPT as a rapid, continuous, in-situ method for identifying failure-susceptible layers in tropical residual slopes, complementing laboratory testing by providing spatial continuity that point samples cannot supply [9].

3.8. Failure mechanism of tropical residual slopes at Sitinjau Lauik

The combined evidence from CPT profiles, physical properties, grain-size distributions, and stability analyses supports a failure mechanism characterised by shallow translational sliding along the weak, high-plasticity clay horizon identified at 1.2–2.8 m depth in Slope 1. Geometrically, this horizon lies at a depth-to-height ratio of approximately 0.07–0.16 within an 18 m slope inclined at 34° , consistent with a shallow, roughly slope-parallel failure surface rather than a deep-seated rotational mechanism; the low ϕ' (19.2°) relative to the slope angle (34°) means the horizon offers little resistance once it is even partially mobilised, so failure is expected to initiate locally within the weak layer and propagate laterally

along it rather than requiring a new surface to develop through the stronger overlying or underlying material. This mode of failure is consistent with the Varnes classification as updated by Hungr et al. [19] of shallow translational debris slides and with observations for tropical residual soil slopes in Southeast Asia. The failure mechanism is initiated by the reduction of effective normal stress through pore-pressure accumulation at the base of the low-permeability clay layer during intense rainfall events [1], [2], and can be instantaneously triggered or amplified by seismic shaking [5], [11]; because the static FS at Slope 1 (1.08) is already close to unity, either mechanism alone — or their combination, such as an earthquake occurring during or shortly after a period of sustained rainfall — is plausibly sufficient to trigger failure, which is consistent with the disproportionate seismic sensitivity of Slope 1 noted in Section 3.5.

The role of progressive weathering must also be considered. In situ tropical weathering progressively reduces the q_c , density, and friction angle of the soil over time [3], [9], implying that even slopes currently exhibiting FS slightly above 1.0 under static conditions may degrade toward marginal stability over engineering timescales without active remediation. This highlights the importance of long-term monitoring and periodic re-evaluation of the geotechnical conditions at Sitinjau Lauik, particularly as climate projections for West Sumatra indicate increasing intensity and variability of rainfall events [1], [18].

3.9. Engineering implications

The results of this study carry direct implications for slope management and hazard mitigation along the Sitinjau Lauik corridor. Slope 1 is identified as requiring immediate priority attention, given its marginal static stability (FS = 1.08), critically low seismic FS (0.77 at $k_h = 0.15$), and the presence of a discrete weak clay layer at shallow depth. Recommended engineering interventions for Slope 1 include: (a) installation of horizontal drain bores targeting the 1.2–2.8 m weak layer to lower the phreatic surface and reduce pore pressures within it; (b) construction of a toe berm or reinforced retaining structure (e.g., a soil-nailed or anchored wall) to increase the resisting moment on the critical slip surface; (c) installation of surface and subsurface drainage, including

crest interceptor drains, to reduce infiltration into the weak horizon; and (d) re-grading to reduce the effective slope angle where right-of-way permits, since the current inclination (34°) exceeds ϕ' by a wide margin. Because SNI 8460:2017 requires a design FS of at least 1.1 under pseudo-static loading, these interventions should be sized to raise Slope 1's pseudo-static FS above 1.1, not merely above the theoretical FS = 1.0 boundary. For Slopes 2 and 3, intermediate-priority measures such as slope re-grading, surface revegetation with deep-rooting species to enhance suction and reduce surficial erosion, and installation of shallow crack-seal grouting are advisable to arrest progressive deterioration and to raise FS above the SNI 8460:2017 criterion under the $k_h = 0.20$ scenario.

At the monitoring and early-warning level, the study recommends deployment of shallow piezometers within the identified weak layer at Slope 1 to provide real-time pore-pressure data, combined with inclinometers to detect sub-centimetre displacements indicative of precursory creep. Threshold pore-pressure and displacement values for escalating alert levels can be calibrated using the FS–pore-pressure relationships derived from the stability analyses presented here. Seismic monitoring using strong-motion accelerographs would allow real-time triggering of alerts when recorded PGA exceeds the critical threshold corresponding to FS < 1.0 for Slope 1. These measures, collectively, constitute a tiered risk management framework consistent with international best practice.

4. Conclusions

This study has presented an integrated geotechnical and seismic stability assessment of three tropical residual soil slopes at Sitinjau Lauik, West Sumatra. CPT profiling identified a persistent weak layer ($q_c < 0.6$ MPa) at 1.2–2.8 m depth in Slope 1, absent in Slopes 2 and 3, coinciding with a high plasticity index (PI = 39.7%), elevated void ratio ($e = 1.50$), and low dry density ($\gamma_d = 10.06$ kN/m³) at that slope. Direct-shear regression gives an effective friction angle $\phi' = 19.2^\circ$ for Slope 1, markedly lower than Slopes 2 ($\phi' = 31.7^\circ$) and 3 ($\phi' = 30.2^\circ$), consistent with the well-established inverse relationship between plasticity index and friction angle for cohesive soils.

Under static loading, Slope 1 is marginally stable ($FS = 1.08$) while Slopes 2 and 3 show higher static FS (1.54 and 1.49). Pseudo-static analysis at $kh = 0.15$ reduces the FS of Slope 1 to 0.77, below the theoretical static-equilibrium threshold of 1.0. Applying the SNI 8460:2017 pseudo-static design acceptance criterion of $FS > 1.1$, Slopes 2 and 3 satisfy the criterion at $kh = 0.15$ ($FS = 1.23$ and 1.17), whereas at $kh = 0.20$ none of the three slopes meets it ($FS = 0.71$, 1.08, and 1.04). Only Slope 1, however, falls below the theoretical failure threshold of $FS = 1.0$; the values of 1.08 and 1.04 for Slopes 2 and 3 are below the design criterion but not below 1.0.

Among the three slopes investigated, high plasticity, elevated void ratio, and low CPT resistance are the physical parameters that most clearly distinguish the critically vulnerable Slope 1 from Slopes 2 and 3; given the small number of slopes examined, this is presented as a site-specific finding rather than a generally applicable predictive rule (see Limitations). The disproportionately large seismic FS reduction at Slope 1 reflects the interaction between an already-marginal static FS and the destabilising pseudo-static force, underscoring the danger of relying on static-only assessment in a high-seismicity setting such as West Sumatra.

Priority remediation should focus on Slope 1 via drainage improvement and structural reinforcement sized to achieve the SNI 8460:2017 pseudo-static criterion ($FS > 1.1$), not merely the theoretical $FS = 1.0$ boundary, and all three slopes should be equipped with real-time pore-pressure and displacement monitoring linked to an operational early-warning system, given that Slopes 2 and 3 also fail the code criterion under the more severe seismic scenario considered here. These findings provide a scientific basis for prioritising slope remediation and early-warning interventions along this corridor.

The pseudo-static method used here is a simplified, conservative approximation of true earthquake loading. No dynamic or time-history analysis, and no site-specific ground-response analysis, was performed; the kh values applied are based on the 2017 Indonesian Seismic Hazard Map combined with an assumed Site Class D amplification factor rather than a site-specific response study. In addition, each slope was characterised by a single CPT sounding and one set of three direct shear specimens without replicates, the soil was modelled as a single homogeneous layer, the piezometric surface was

assumed from field observations rather than measured and was not varied in a sensitivity analysis, road-traffic surcharge was not included, and clay mineralogy was not confirmed by X-ray diffraction. Future work should extend this integrated framework to additional slopes along the Sumatra National Road, incorporate site-specific ground-response and traffic-load sensitivity analysis, and add cyclic/dynamic testing to refine the seismic hazard assumptions used here.

Acknowledgments

The authors would like to express their sincere gratitude to all individuals and institutions that contributed to this research. Special appreciation is extended to colleagues and technical staff who assisted with field investigations, laboratory testing, and data processing.

Competing Interest Statement

The authors declare no known competing financial interests or personal relationships that could have influenced the work reported in this paper.

Data Availability Statement

The data supporting the findings of this study, including CPT logs, laboratory test results, and slope-stability model files, are available from the corresponding author upon reasonable request.

References

- [1] H. Rahardjo, E. C. Leong, and R. B. Rezaury, "Effect of antecedent rainfall on pore-water pressure distribution characteristics in residual soil slopes under tropical rainfall," *Hydrol. Process.*, vol. 22, no. 4, pp. 506–523, 2008.
- [2] H. Rahardjo, A. S. Nio, E. C. Leong, and N. Y. Song, "Effects of groundwater table position and soil properties on stability of slope during rainfall," *J. Geotech. Geoenviron. Eng.*, vol. 136, no. 11, pp. 1555–1564, 2010.
- [3] D. H. Natawidjaja and W. Triyoso, "The Sumatran fault zone — from source to hazard," *J. Earthq. Tsunami*, vol. 1, no. 1, pp. 21–47, 2007.
- [4] S. Pramandi, A. Rifa'i, and W. Rahayu, "Determination of unstable slope area and slope movement characteristics

- analysis on the national road segment Cisomang–Padalarang," *Procedia Eng.*, vol. 125, pp. 311–317, 2015.
- [5] R. W. Jibson, "Methods for assessing the stability of slopes during earthquakes — a retrospective," *Eng. Geol.*, vol. 122, no. 1–2, pp. 43–50, 2011.
- [6] D. G. Fredlund, N. R. Morgenstern, and R. A. Widger, "The shear strength of unsaturated soils," *Can. Geotech. J.*, vol. 15, no. 3, pp. 313–321, 1978.
- [7] C. W. W. Ng and Q. Shi, "A numerical investigation of the stability of unsaturated soil slopes subjected to transient seepage," *Comput. Geotech.*, vol. 22, no. 1, pp. 1–28, 1998.
- [8] J. M. Duncan, "State of the art: limit equilibrium and finite-element analysis of slopes," *J. Geotech. Eng.*, vol. 122, no. 7, pp. 577–596, 1996.
- [9] P. K. Robertson, "Interpretation of cone penetration tests — a unified approach," *Can. Geotech. J.*, vol. 46, no. 11, pp. 1337–1355, 2009.
- [10] M. Irsyam et al., "Development of new seismic hazard maps of Indonesia 2017," *J. Earthq. Spektra.*, vol. 36, no. 1s, pp. 112–136, 2020, doi: 10.1177/8755293020951206
- [11] A. Rifa'i and O. A. Putra, "Slope stability analysis due to infrastructure development of Ratu Boko site Yogyakarta under earthquakes loading," *J. Appl. Eng. Sci.*, vol. 19, no. 4, pp. 920–925, 2021, doi: 10.5937/jaes0-30070.
- [12] E. A. Suryo, B. Triasriputro, and C. Sulistiyowati, "Assessment of slope stability along the trans-Java toll road using numerical analysis," *IOP Conf. Ser.: Earth Environ. Sci.*, vol. 109, no. 1, Art. no. 012047, 2017.
- [13] S. Leroueil, "Natural slopes and cuts: movement and failure mechanisms," *Géotechnique*, vol. 51, no. 3, pp. 197–243, 2001.
- [14] A. Tohari, M. Nishigaki, and M. Komatsu, "Laboratory rainfall-induced slope failure with moisture content measurement," *J. Geotech. Geoenviron. Eng.*, vol. 133, no. 5, pp. 575–587, 2007.
- [15] R. P. Orense, S. Shimoma, K. Maeda, and I. Towhata, "Instrumented model slope failure due to water seepage," *J. Nat. Disaster Sci.*, vol. 26, no. 1, pp. 15–26, 2004.
- [16] N. R. Morgenstern and V. E. Price, "The analysis of the stability of general slip surfaces," *Géotechnique*, vol. 15, no. 1, pp. 79–93, 1965.
- [17] A. W. Bishop, "The use of the slip circle in the stability analysis of slopes," *Géotechnique*, vol. 5, no. 1, pp. 7–17, 1955.
- [18] P. Aleotti and R. Chowdhury, "Landslide hazard assessment: summary review and new perspectives," *Bull. Eng. Geol. Environ.*, vol. 58, no. 1, pp. 21–44, 1999.
- [19] O. Hungr, S. Leroueil, and L. Picarelli, "The Varnes classification of landslide types, an update," *Landslides*, vol. 11, no. 2, pp. 167–194, 2014.
- [20] K. Sassa, H. Fukuoka, G. Wang, and N. Ishikawa, "Undrained dynamic-loading ring-shear apparatus and its application to landslide dynamics," *Landslides*, vol. 1, no. 1, pp. 7–19, 2004.
- [21] O. A. Putra, H. Herman, H. Medriosa, and F. Nugroho, "Effectiveness of waste paper ash for stabilization on clay soil," *Rev. Ing. Constr.*, vol. 37, no. 2, pp. 253–259, Aug. 2022, doi: 10.7764/RIC.00030.21.