

Seismic response and postseismic stability of an investigation-constrained loess–mudstone slope

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Research Article

Keywords: loess–mudstone slope, seismic response, site investigation, permanent deformation, strain softening, postseismic stability

Posted Date: August 19th, 2026

DOI: <https://doi.org/10.21203/rs.3.rs-10640168/v1>

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Additional Declarations: No competing interests reported.

Version of Record: A version of this preprint was published at Transportation Infrastructure Geotechnology on October 17th, 2026. See the published version at <https://doi.org/10.1007/s40515-026-01071-7>.

Seismic response and postseismic stability of an investigation-constrained loess–mudstone slope

Abstract: A dry loess–mudstone slope in Tianshui, Gansu, was constrained by 1:500 topography, boreholes, in-hole shear-wave measurements and seismic CT to build an investigation-derived 3D model that delineated loess thickness and three mudstone integrity domains. Under a 0.30 g El Centro record with loess strain softening and postseismic strength reduction, the static factor of safety was 1.83. Downslope PGA exhibited a non-monotonic profile: 0.54 g at the toe (amplification 1.00), 1.29 g at midslope (2.40) and 0.94 g at the crest (1.76). The midslope peak arose from 3D wave focusing at changes in slope geometry and stiffness, whereas upper-slope yielding dissipated energy, limiting crest acceleration while permanent deformation accumulated. Post-shaking residual downslope displacement increased from 0.34 m at the toe to 1.15 m at the crest; the mean accumulated plastic shear strain was 0.0127, and 15.9% of the loess zones reached residual strength, forming two discrete plan-view clusters rather than a continuous basal band. Mild softening reduced the postseismic factor of safety to 1.74. These results link non-monotonic amplification, directional deformation and post-peak degradation to investigation-derived geological structure; under the investigated dry condition, the mapped lithologic contact did not behave as a controlling weak plane. The findings provide an investigation-based basis for seismic assessment and postseismic management of loess–mudstone slopes along transportation corridors.

Keywords: loess–mudstone slope; seismic response; site investigation; permanent deformation; strain softening; postseismic stability

1 Introduction

Earthquake-induced deformation is a recurrent hazard for loess-covered slopes on the Chinese Loess Plateau. Whether shaking produces transient vibration, cumulative permanent deformation or rapid failure depends on slope geometry, material contrast, antecedent moisture and structural discontinuities [1–4]. Numerical analyses are most informative when geological structure is constrained by investigation rather than inferred from topography alone.

Loess-over-mudstone slopes are often assumed to be inherently susceptible to sliding along the lithologic contact. This assumption is plausible because loess is porous and collapsible, mudstone may soften or disintegrate when wet, and impedance contrasts can amplify motion within the cover [5–12]. However, experimental and field studies distinguish an ordinary lithologic contact from a mechanically developed slip zone. Shaking-table models that contain a fault zone or a high-water-content interlayer commonly develop contact-controlled failure [7–10]. In contrast, fissured loess slopes without a persistent weak layer may fail within the loess or near the shoulder as cyclic damage accumulates [5]. A numerical interface therefore represents a specific hypothesis about discontinuity, strength, stiffness and possible separation. It should not be introduced solely because two lithologies meet.

The site is described here as a loess–mudstone slope in Tianshui. The investigation identified a thin loess cover over weathered mudstone across terrain with approximately 130 m of overall relief. Boreholes constrained cover thickness and weathering depth, in-hole tests provided shear-wave velocities, and seismic computed tomography (CT) resolved three mudstone velocity–integrity domains. The investigation reported no groundwater table, old landslide, fault, through-going fissure or persistent weak interlayer within the study area. These observations support a bonded-continuum baseline in which localisation develops from the measured geometry and assigned stress–strength field.

This study had three objectives. First, we examined how topography, boreholes, in-hole wave velocities and seismic CT can jointly constrain a three-dimensional geological model. Second, we quantified acceleration, directional displacement and permanent deformation under design-level shaking. Third, we assessed how post-peak softening affects postseismic stability and whether the response requires a contact-controlled failure hypothesis. The principal contribution lies in linking investigation evidence to the spatial response, rather than in the numerical procedure itself.

2 Investigation framework and geological constraints

2.1 Regional engineering-geological setting

The slope occupies a loess ridge-and-gully landscape. Within the analysed domain, the ground surface has approximately 120 m of relative relief. Bedrock is exposed on steeper slopes, whereas loess is concentrated on gentler ridge and shoulder positions. The model extends 223.26 m by 187.26 m in plan. For analysis and presentation, the minimum elevation was translated to a relative datum of $z = 0$. This translation does not alter self-weight stress because body force is integrated over material thickness. It does, however, avoid confusion between absolute elevation and overburden depth.

The reported design ground-motion parameters are seismic intensity VIII, a basic acceleration of 0.30 g, a characteristic period of 0.40 s and design group 2. The site is class II, with an effective shear-wave velocity of 376.7 m/s and an equivalent cover thickness of 11.7 m. The adopted 0.30 g excitation therefore represents a site-design case rather than an arbitrary scaling level [13].

No persistent groundwater table was encountered during the investigation. Seasonal fracture water may occur after rainfall, but no saturated layer was identified along the lithologic contact. The slope was assessed as generally stable under the surveyed dry condition. Nevertheless, the investigation recommended surface and subsurface drainage, staged earthworks, prompt support of disturbed ground and deformation monitoring. The numerical results are therefore interpreted as a dry-condition hazard assessment, not as evidence of long-term stability. Representative investigation evidence is shown in Fig. 1.



Fig. 1 Field evidence used to constrain the model: (a) borehole drilling on the natural slope, (b) in-hole wave-velocity testing

2.2 Stratigraphy, tomography and wave velocity

Boreholes indicated a residual-colluvial veneer generally 0.5–1.0 m thick, loess 0.8–6.4 m thick and strongly weathered mudstone 13–42 m thick. Moderately weathered mudstone extended beyond the investigated depth. The veneer is thinner than the approximately 5 m horizontal grid spacing, and explicit representation would require highly flattened zones. It was therefore homogenised with the loess. On steep bedrock-forming slopes, the equivalent surface-soil thickness was tapered to 1.0 m. This treatment avoids extending ridge-top loess thickness uniformly across bedrock outcrops.

Weighted mean shear-wave velocities were 190.3 m/s for loess, 620.4 m/s for strongly weathered mudstone and 1602.0 m/s for moderately weathered mudstone. Seismic CT divided the mudstone into three domains. The upper domain had a P-wave velocity of 1450–1800 m/s and an integrity index of 0.34–0.52. The intermediate domain had values of 1800–2000 m/s and 0.52–0.64, whereas the deeper domain had values of 2000–2400 m/s and 0.64–0.92. These domains were interpreted as strongly weathered, broken moderately weathered and relatively intact moderately weathered mudstone, respectively. The CT boundaries were not treated as mathematical discontinuities. Instead, they were interpolated as material-domain surfaces constrained by boreholes and profile geometry. Figure 2 and Table 1 summarise the investigation-to-model traceability.

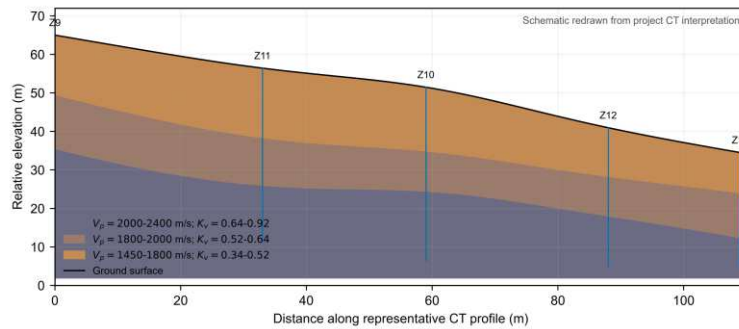


Fig. 2 English-language schematic redrawn from the representative seismic-CT interpretation. V_p denotes compressional-wave velocity, and K_v is the reported integrity index

Table 1 Investigation evidence and its use in the numerical model

Evidence source	Reported value or observation	Use in the model	Confidence or limitation
Topographic survey	1:500 contours; relief about 130 m	Terrain surface and model boundary	High within the surveyed area
Boreholes	Veneer 0.5-1.0 m; loess 0.8-6.4 m; strongly weathered mudstone 13-42 m	Controls on thickness and weathering surfaces	Interpolation required between boreholes
In-hole V_s	190.3, 620.4 and 1602.0 m/s	Reference for dynamic stiffness and impedance	Weighted means; strain dependence was not measured
Seismic CT	V_p bands of 1450–1800, 1800–2000 and 2000–2400 m/s	Definition of three mudstone integrity domains	Profiles constrain material domains, not interface strength

Evidence source	Reported value or observation	Use in the model	Confidence or limitation
Hydrogeology	No groundwater encountered; seasonal fracture water possible	Dry baseline without pore-pressure generation	Wet-season conditions were not represented
Geomorphology	No mapped landslide, fault or continuous fissure within the study area	No pre-defined weak interface	Absence of evidence does not establish perfect bonding

3 Investigation-constrained model and response measures

3.1 Geometry, mesh and material zoning

The terrain and subsurface boundaries interpreted from the investigation were converted into a structured three-dimensional mesh. Interpolation followed the ground surface, the base of the loess and two seismic-CT velocity boundaries. Each material transition was therefore traceable to a measured or interpreted investigation surface. The mesh contained 34,200 zones and 37,674 grid points. An approximately 5 m horizontal spacing balanced the survey resolution against the shortest retained seismic wavelength.

Four investigation-based domains were retained: loess incorporating the thin surface veneer, strongly weathered mudstone, broken moderately weathered mudstone and intact moderately weathered mudstone. The CT-derived mudstone classes remained separate even where they shared laboratory strength parameters. This preserved the geological interpretation and allowed subsequent calibration without remeshing. Their spatial distribution is shown in Fig. 3.

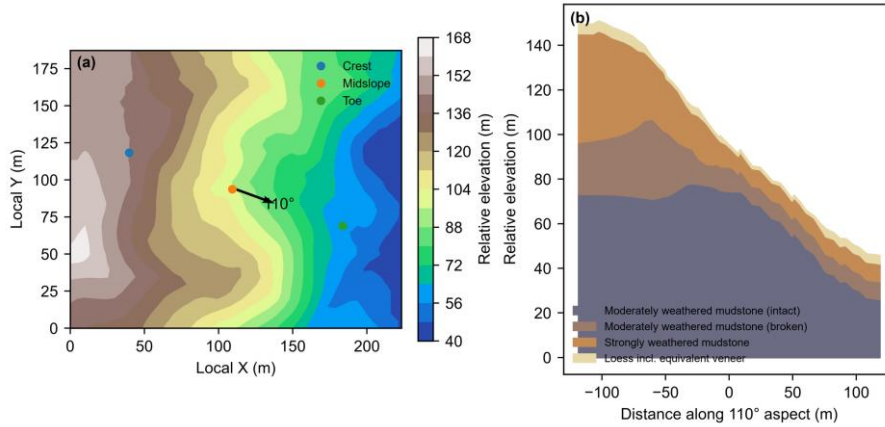


Fig. 3 Numerical model: (a) terrain expressed in relative elevation, monitoring locations and the downslope projection direction; and (b) representative stratigraphic section through the structured mesh

3.2 Constitutive formulation

The baseline model used an elastic-perfectly plastic Mohr–Coulomb formulation with a tensile cut-off and non-associated flow. This deliberately parsimonious description reflects the available investigation data, which did not include site-specific cyclic-degradation or tensile-fracture parameters. Effective-stress shear resistance was defined as

$$\tau_f = c + (\sigma_n - u) \tan \varphi \quad (1)$$

Here, τ_f is the limiting shear stress, c is cohesion, σ_n is the total normal stress, u is pore-water pressure, and φ is the friction angle. Pore pressure was not generated in the dry baseline. Elastic constants were derived from the adopted bulk and shear moduli and checked against the in-hole wave-velocity scale.

$$G = \frac{E}{[2(1+\nu)]} \quad (2)$$

$$K = \frac{E}{[3(1-2\nu)]} \quad (3)$$

Here, G is the shear modulus, K is the bulk modulus, E is Young's modulus, and ν is Poisson's ratio. In-hole shear-wave velocities provided an independent stiffness scale. The working moduli were kept below the small-strain maxima to avoid applying seismic-test stiffness as a constant large-strain modulus. Table 2 lists the adopted material parameters.

Table 2 Material parameters adopted for the baseline analysis

Material domain	Density (kg m ⁻³)	Bulk modulus (Pa)	Shear modulus (Pa)	c (kPa)	φ (°)	Dilation (°)
Loess, including veneer	1820	1.428×10^8	6.590×10^7	10.0	17.5	0
Strongly weathered mudstone	2080	3.737×10^9	8.007×10^8	120.0	22.0	0

Material domain	Density (kg m ⁻³)	Bulk modulus (Pa)	Shear modulus (Pa)	<i>c</i> (kPa)	φ (°)	Dilation (°)
Broken moderately weathered mudstone	2220	1.709×10^{10}	5.697×10^9	201.5	35.5	0
Intact moderately weathered mudstone	2220	1.709×10^{10}	5.697×10^9	201.5	35.5	0

3.3 Initial stress, boundaries and dynamic input

The z-axis is positive upwards, and gravity acts in the negative z direction. Static equilibrium was established using a fixed base, lateral rollers and an initial horizontal-to-vertical stress ratio of 0.5. Equilibrium was accepted when the average mechanical force ratio fell below 1.0×10^{-5} .

During dynamic loading, free-field lateral boundaries and a quiet compliant base were used to minimise artificial wave reflections. The horizontal El Centro record was baseline-corrected, band-limited to 0.10–3.50 Hz and scaled to 0.30 g. Incident velocity was converted to compliant-base shear stress using

$$\tau(t) = 2\rho C_s v_{(in)}(t) \quad (4)$$

Here, τ_t is the tangential stress applied at the compliant base, ρ is mass density, C_s is shear-wave velocity, ρC_s is shear impedance, and v_{in} is the incident particle velocity [14–15]. The 36.16 s motion was applied along the terrain-derived downslope direction under large-strain kinematics. Figure 4 shows the processed input motion.

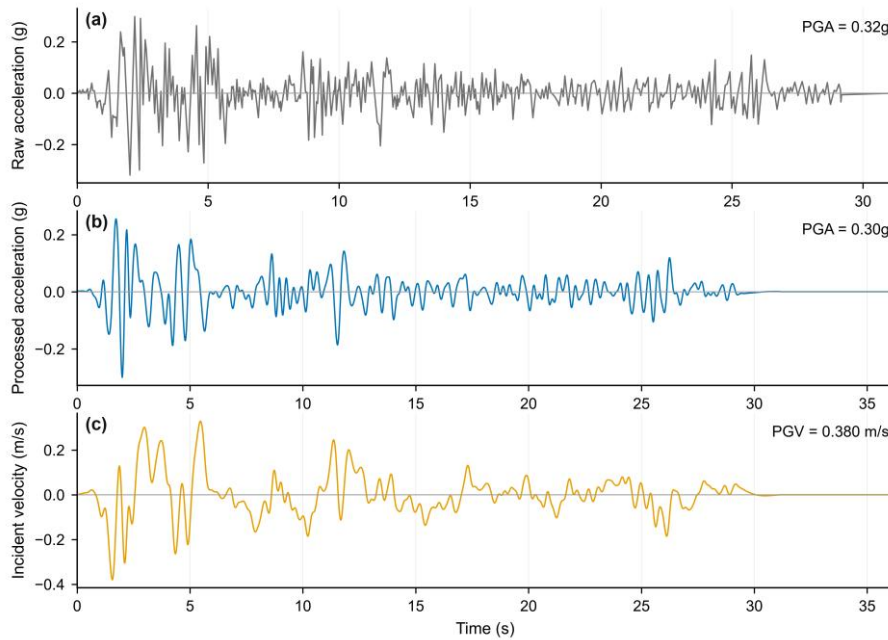


Fig. 4 Processing of the earthquake input: (a) raw El Centro NS acceleration; (b) baseline-corrected, band-limited acceleration scaled to 0.30 g; and (c) integrated incident velocity used for compliant-base stress input

Table 3 Numerical configuration and verification criteria

Item	Adopted value or setting	Purpose
Coordinate datum	$z_{rel} = \text{elevation} - 1150 \text{ m}$	Clarifies self-weight interpretation; coordinate translation preserves geometry and stress
Static convergence	Average force ratio $\leq 1.0 \times 10^{-5}$	Establish gravitational equilibrium before dynamic loading
Dynamic sides	FLAC3D free-field boundaries	Limit lateral wave reflection and represent far-field response
Dynamic base	Quiet boundary with stress input	Apply compliant-base excitation and absorb reflected waves
Kinematics	Large-strain mode enabled	Update geometry as permanent deformation accumulates
Mesh checks	$J_{min} = 3.056$; non-positive zones: 0	Identify inverted or degenerate zones

Item	Adopted value or setting	Purpose
Group checks	No missing or duplicate material assignments	Ensure that each zone belongs to one material domain

3.4 Monitoring and response measures

Acceleration and displacement were monitored at the base, toe, midslope, crest and a diagnostic surface location. Horizontal response was projected onto the terrain-derived downslope unit vector n . Displacement relative to the base was used to remove compliant-base translation:

$$u_{(rel,i)(t)} = n \cdot [u_{i(t)} - u_{(base)(t)}] \quad (5)$$

Here, $u_{(rel,i)(t)}$ is the relative downslope displacement at monitoring location i and time t , n is the downslope unit vector, $u_{i(t)}$ is the displacement vector at location i , and $u_{(base)(t)}$ is the base displacement vector. Peak ground acceleration (PGA) amplification was referenced to the recorded base response. Displacement-threshold times were used only to compare the sequence of deformation onset. A contact-mobilisation ratio would require measured interface shear and normal strengths and may be expressed conceptually as

$$R_m = \frac{|\tau_{contact}|}{[c_i + (\sigma_n - u) \tan \varphi_i]} \quad (6)$$

Here, R_m is the contact-mobilisation ratio, $\tau_{contact}$ is the mobilised contact shear stress, c_i is contact cohesion, σ_n is the total normal stress, u is pore-water pressure, and φ_i is the contact friction angle. Because the investigation provided neither these quantities nor evidence of a persistent slip layer, no numerical interface was introduced and no contact-specific factor was reported.

3.5 Mesh resolution and numerical stability

Mesh resolution was assessed from the shortest retained wavelength, $\lambda_{min} = \frac{V_{s,min}}{f_{max}}$, where λ_{min} is the minimum resolved wavelength, $V_{s,min}$ is the minimum shear-wave velocity, and f_{max} is the maximum retained frequency.

For the minimum measured shear-wave velocity of 190.3 m/s and a maximum retained frequency of 3.50 Hz, the shortest wavelength was 54.4 m. The approximately 5 m horizontal cell size provided about 10.9 cells per wavelength. The thin near-surface interval was discretised more finely.

The explicit time step was controlled by the smallest zone dimension and the highest local wave speed.

The solver selected the stable time step automatically, and no mass scaling was applied. Table 3 summarises the implementation and verification settings. All calculations were performed in FLAC3D 6.0 [16].

3.6 Strain-softening sensitivity and postseismic strength reduction

Because the baseline retains peak cohesion and friction after yielding, a supplementary strain-softening analysis was used to bound post-peak degradation in the loess. The mudstone domains retained their investigation-based baseline properties. Peak and residual values followed the selection principles of current Chinese landslide-design standards [17-18]. However, the residual values were treated as sensitivity parameters rather than universal code values. Under the mild law, c decreased from 10 to 9 and 8 kPa. Over the same accumulated plastic shear-strain intervals of 0, 0.005 and 0.020, φ decreased from 17.5° to 17.0° and 16.5°.

For a generic strength parameter p , the piecewise-linear softening law was written as

$$p(\kappa_s) = p_p + \frac{(p_r - p_p)\kappa_s}{\kappa_r}, 0 < \kappa_s < \kappa_r; p = p_r, \kappa_s \geq \kappa_r \quad (7)$$

Here, κ_s is the accumulated plastic shear strain, p_p and p_r are the peak and residual strengths, respectively, and κ_r is the plastic shear strain at residual strength. After excitation, the degraded model was re-equilibrated while retaining the stress, displacement and softening fields.

For postseismic strength reduction, the degraded c and φ in each zone were frozen as a heterogeneous Mohr–Coulomb field before a trial reduction factor F was applied. This procedure followed the conventional shear-strength-reduction (SSR) framework [19-20].

$$c_F = \frac{c_{post}}{F}, \tan \varphi_F = \frac{\tan \varphi_{post}}{F} \quad (8)$$

Here, c_F and φ_F are the trial cohesion and friction angle at reduction factor F , whereas c_{post} and φ_{post} are the corresponding postseismic values before reduction. The postseismic factor of safety was obtained from the limiting stable–unstable bracket. A lower-bound law used $c_r = 6$ kPa, $\varphi_r = 15^\circ$, and $\kappa_r = 0.010$. This case developed illegal geometry at 6.032754 s and was therefore retained as a dynamic-instability bound, not interpreted as a completed response.

4 Results

4.1 Mesh and static-state verification

Independent checks confirmed all 34,200 zones and 37,674 grid points. No missing references, unassigned zones, duplicate material memberships or non-positive centre Jacobians were identified. One low-quality zone on the lateral boundary supports only one overlay. It is remote from the principal response sector and does not create an internal free face. Nevertheless, this local defect should be refined before higher-resolution construction-stage analyses.

The gravity stage converged to an average force ratio of approximately 1.0×10^{-5} , with a maximum static displacement of about 4.21 mm. Zone-centre vertical stress ranged from approximately -6.43 kPa near the free surface to -3.04 MPa at depth, with compression reported as negative. This distribution is consistent with a traction-free surface and increasing overburden. Absolute elevation therefore did not create 1150 m of fictitious overburden. FLAC3D integrates body force over material thickness, not coordinate magnitude. The relative datum improves interpretability but does not materially reduce the cycle count or alter the stress field.

A separate built-in SSR analysis was restored from the converged gravity state. Cohesion and friction were reduced simultaneously, without an interface contribution, until stable and unstable brackets were obtained. The resulting static factor of safety was 1.83. The dry natural slope therefore retained appreciable static reserve, although this did not prevent cumulative deformation under design-level shaking.

4.2 Acceleration response

Downslope PGA was 0.54 g at the base, 0.52 g at the toe, 1.29 g at midslope and 0.94 g at the crest. Relative to the recorded base, the corresponding amplification factors were 1.00, 0.97, 2.40 and 1.76. Because the midslope maximum exceeded the crest maximum, the response could not be described by monotonic amplification with elevation.

Three complementary mechanisms explain this ordering. First, the midslope instrument lies near changes in local inclination and cover thickness. Incident, reflected and refracted waves may therefore focus within the low-velocity loess above weathered mudstone. Second, the three-dimensional ridge-and-gully geometry permits lateral radiation and modal antinodes away from the geometric crest [21]. Third, the strain-softening results show that accumulated plastic strain is greatest in the upper-slope loess. Nonlinear yielding and hysteretic dissipation can limit peak crest acceleration while permanent displacement and Arias intensity continue to accumulate [22]. The crest consequently had lower PGA than midslope but greater residual displacement and cumulative shaking energy. PGA, energy demand and permanent deformation must therefore be considered together.

The recorded base PGA exceeded the nominal 0.30 g used to process the incident motion. This difference does not indicate an input error. The base history represents the response of the compliant boundary and contains both incident and reflected components, whereas the nominal motion is the incident target. Accordingly, all amplification ratios were normalised by the recorded base response. The resulting pattern agrees qualitatively with experiments showing spatially non-uniform amplification in loess–mudstone slopes where stiffness contrasts interact with topography [7-11]. Figure 5 and Table 4 present the monitoring histories and response metrics, respectively.

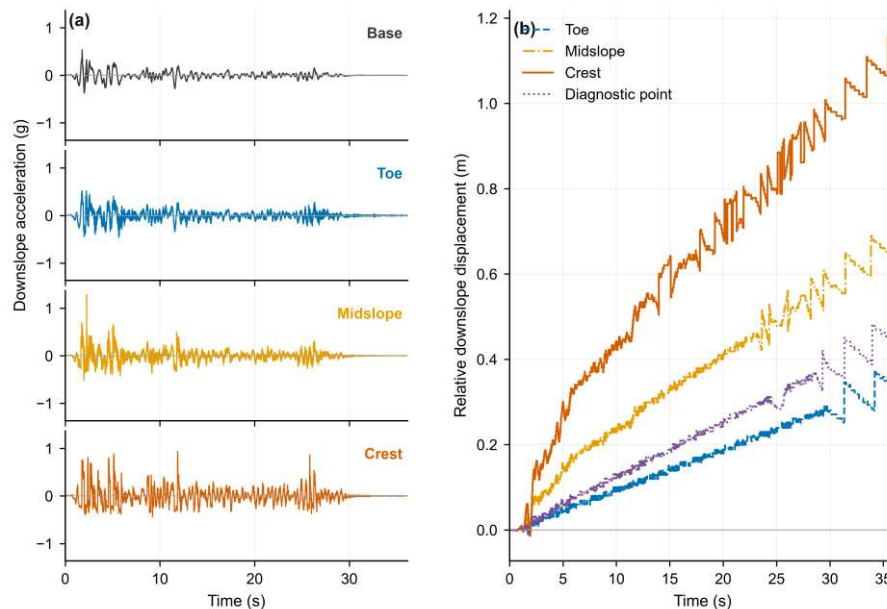


Fig. 5 Computed downslope response histories: (a) station-specific acceleration histories plotted on a common scale and (b) displacement relative to the recorded base. All FLAC3D history samples are retained; the stepwise accumulation reflects permanent deformation under large-strain updating

4.3 Permanent deformation and onset

Peak relative downslope displacement was 0.37 m at the toe, 0.69 m at midslope and 1.16 m at the crest. The corresponding residual values were 0.34, 0.64 and 1.15 m. The close agreement between peak and residual values

shows that permanent deformation dominated the final response. The diagnostic location retained a residual relative displacement of 0.44 m, smaller than that at the crest. Figure 6 summarises the spatial ranking.

Residual vertical displacement was -0.54 m at the toe, -0.09 m at midslope, +0.20 m at the crest and -0.37 m at the diagnostic point. The mixed signs reflect three-dimensional deformation and geometric updating. Settlement and outward movement developed at the toe, whereas the crest underwent relative heave and rotation in the adopted coordinate system. These values indicate severe dynamic deformation or local instability under the design record. They do not imply that the slope would travel as an intact block, nor are they suitable for runout zoning after substantial mesh distortion.

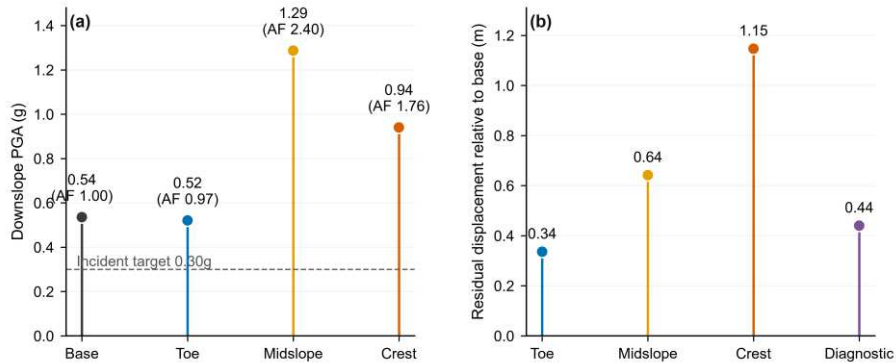


Fig. 6 Summary of the dynamic response: (a) recorded downslope PGA and amplification factor (AF), normalised by the recorded base and compared with the 0.30 g incident target; and (b) residual downslope displacement relative to the base. Points are deterministic outputs for the analysed record and are not replicate statistics

The 0.10 m relative-displacement threshold was exceeded first at the crest after 2.1 s. It was then exceeded at midslope after 3.9 s, at the diagnostic point after 7.5 s and at the toe after 10.0 s. Only the crest reached 1.0 m, at 30.0 s. Midslope reached 0.50 m at 24.0 s, whereas the toe and diagnostic point did not. This sequence indicates early upper-slope damage followed by progressive accumulation, rather than simultaneous basal release. Figure 7 and Table 5 show the onset sequence.

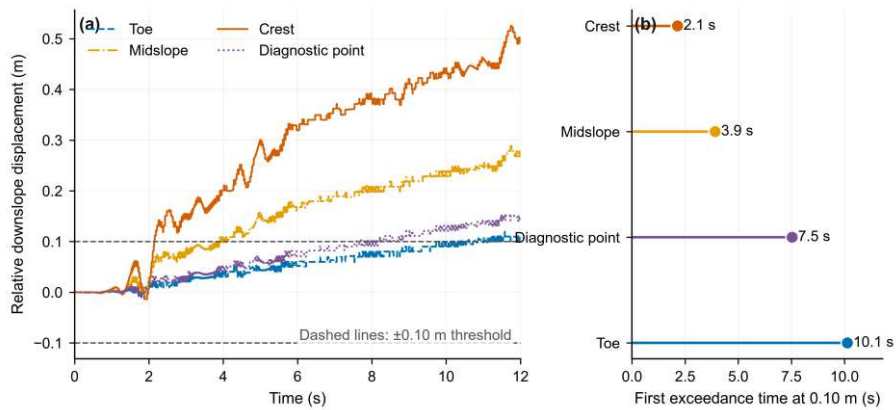


Fig. 7 Onset of deformation: (a) relative downslope displacement during the first 12 s and (b) first exceedance time for an absolute threshold of 0.10 m

4.4 Energy and spectral characteristics

Arias intensity reached 3.05 m/s at the base, 4.93 m/s at the toe, 5.85 m/s at midslope and 14.30 m/s at the crest. The crest value was approximately 4.68 times the base value. Cumulative shaking energy was therefore amplified more strongly near the upper slope than the PGA ranking alone suggests. The 5%-damped response spectra also showed broad upper-slope amplification at short-to-intermediate periods rather than a single narrow resonance. Figure 8 presents the energy and spectral measures.

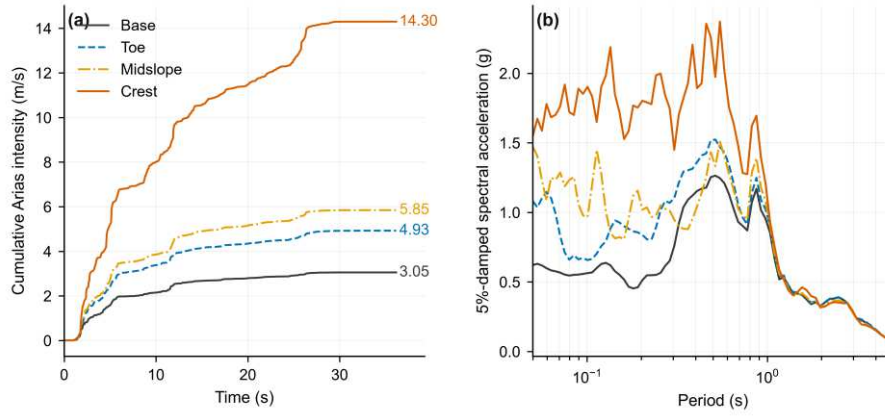


Fig. 8 Energy and spectral response: (a) cumulative Arias intensity and (b) 5%-damped acceleration response spectra

Table 4 Dynamic response at the monitoring locations

Monitoring location	Downslope PGA (g)	PGA amplification	Peak u_{rel} (m)	Residual u_{rel} (m)	Residual u_z (m)
Base	0.536	1.000	0	0	not recorded
Toe	0.521	0.972	0.374	0.336	-0.536
Midslope	1.287	2.403	0.689	0.642	-0.094
Crest	0.941	1.756	1.159	1.147	0.196
Diagnostic location	0.775	1.447	0.482	0.440	-0.373

Table 5 First-exceedance time for relative downslope displacement

Monitoring location	0.01 m	0.05 m	0.10 m	0.50 m	1.00 m
Toe	1.7 s	4.3 s	10.0 s	Not reached	Not reached
Midslope	1.4 s	2.1 s	3.9 s	24.0 s	Not reached
Crest	1.4 s	1.6 s	2.1 s	12.0 s	30.0 s
Diagnostic location	1.5 s	3.8 s	7.5 s	Not reached	Not reached

4.5 Strain-softening response and postseismic stability

The mild-softening model completed 691,903 dynamic cycles and reached 36.16003 s without developing illegal geometry. Its maximum main-grid displacement magnitude was 2.7479 m, 22.4% greater than the baseline maximum of 2.2442 m. All 1,710 loess zones in the main model accumulated non-zero plastic shear strain. Of these, 272 zones (15.9%) reached the prescribed residual-strength plateau. The postseismic mean accumulated plastic shear strain was 0.01273, and the raw zone maximum was 0.19294. The FLAC3D contour was capped at 0.02 for display, so red denotes values at or above the residual-strain threshold rather than exactly 0.02. Mean loess strength decreased to $c = 9.113$ kPa and $\phi = 17.057^\circ$, with minima of 8 kPa and 16.5° . Figure 9 shows the adopted softening law and the quantitative plan-view degradation.

Static re-equilibration required 2,216 cycles and ended at an average force ratio of 9.9841×10^{-6} . SSR applied to the frozen degradation field gave $F_s = 1.74$. The corresponding values were 1.83 before shaking and 1.82 after shaking in the non-softening baseline. Earthquake loading combined with softening therefore reduced capacity by 4.9% relative to the pre-earthquake state. The reduction was 4.4% relative to the non-softening postseismic state, whereas the effect on permanent displacement was larger.

Zones at residual strength formed two principal plan-view clusters. The largest connected cluster contained 206 zones in the high rear and upper model (approximately $x = 3.9 - 53.9$ m and $z = 135.6 - 159.3$ m). A second cluster contained 57 zones in the lower opposite-side sector ($x = 157.7 - 212.2$ m and $z = 64.9 - 84.9$ m). Figure 10a shows that these high-strain regions are separated by lower-strain sectors rather than connected by a continuous plan-view trace. The intervening cyan-green belts represent strain gradients and redistribution around topographic breaks, not a uniformly mobilised rigid block.

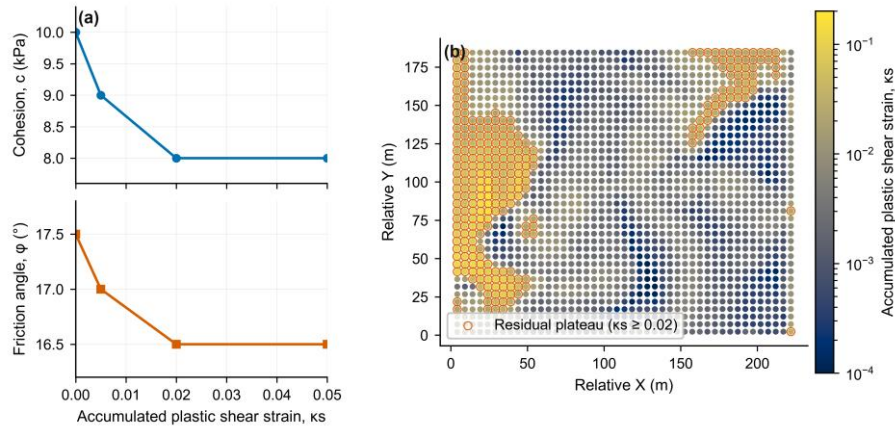


Fig. 9 Mild dry-loess strain-softening law and quantitative postseismic distribution of accumulated plastic shear strain: (a) degradation of cohesion and friction angle on aligned axes and (b) plan-view loess degradation. Orange outlines identify zones that reached the residual-strength plateau ($\kappa_s \geq 0.02$)

The two representative longitudinal sections provide a more discriminating test of the failure mechanism (Fig. 10b,c). Accumulated plastic shear strain reached the 0.02 display cap mainly within the crest and upper-shoulder loess. Local green–cyan zones occurred at slope breaks and near the toe, whereas most of the middle and lower sections remained below approximately 0.006. Neither section contained a continuous high-strain band along the full loess–mudstone contact. In the bonded dry model, this pattern is consistent with distributed upper-slope degradation rather than coherent basal sliding. It does not establish that the contact is intrinsically strong. Because the loess is represented by only one vertical zone layer and no interface element is present, internal localisation cannot be clearly separated from contact-parallel shear. These sections therefore describe the response of this model and should not be generalised to wet-season conditions.

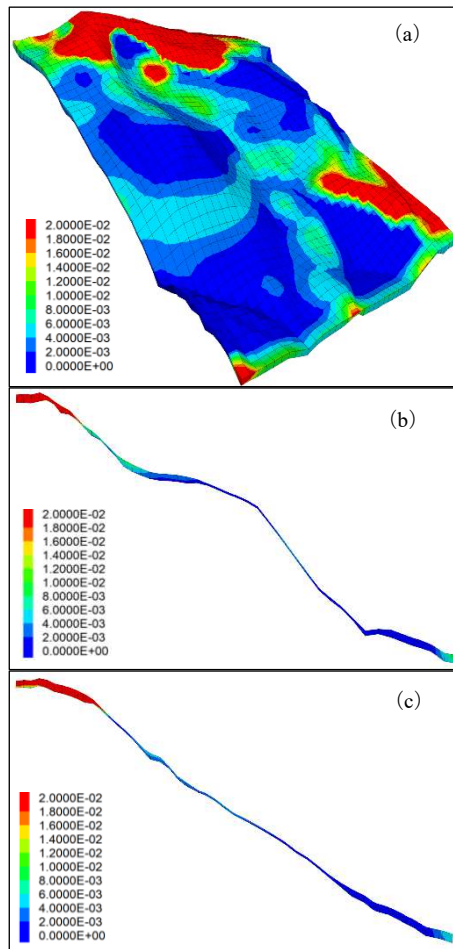


Fig. 10 Postseismic accumulated plastic shear strain in the loess: (a) three-dimensional distribution and (b,c) two representative longitudinal sections. Contours use volumetric averaging and are capped at $\kappa_s = 0.02$; red therefore denotes zones at or above the prescribed residual-strength plateau. The raw maximum zone value is 0.19294

4.6 Directional displacement fields

Figure 11 shows the total-displacement fields at the end of shaking. These fields include compliant-base translation and should not be interpreted directly as slope-relative deformation. Across the main grid, total displacement ranged from 0.8448 to 2.7206 m in X, -0.7464 to -0.1050 m in Y and -0.8370 to 0.6614 m in Z. Displacement magnitude ranged from 0.8693 to 2.7481 m. After subtracting the base displacement vector (0.8707, -0.1646, -0.0841 m), the maximum differential displacement magnitude was 1.865 m. The total fields therefore indicate spatial gradients and directionality, whereas the monitoring histories and differential values quantify deformation relative to the base.

The X component closely matched the magnitude field and formed a broad maximum over the rear and upper shoulder. This confirms that deformation was dominated by the imposed downslope direction. Y displacement was smaller and negative throughout the main grid, with a local minimum on the upper flank. It therefore indicates asymmetric lateral distortion rather than coherent transverse translation. The Z field changed sign, with local uplift upslope and increasingly negative movement towards lower terrain. This juxtaposition is consistent with rotation, local heave and downslope settlement. The component extrema did not coincide, and the displacement magnitude lacked a uniform plateau. The deforming mass therefore did not behave as a single rigid block.

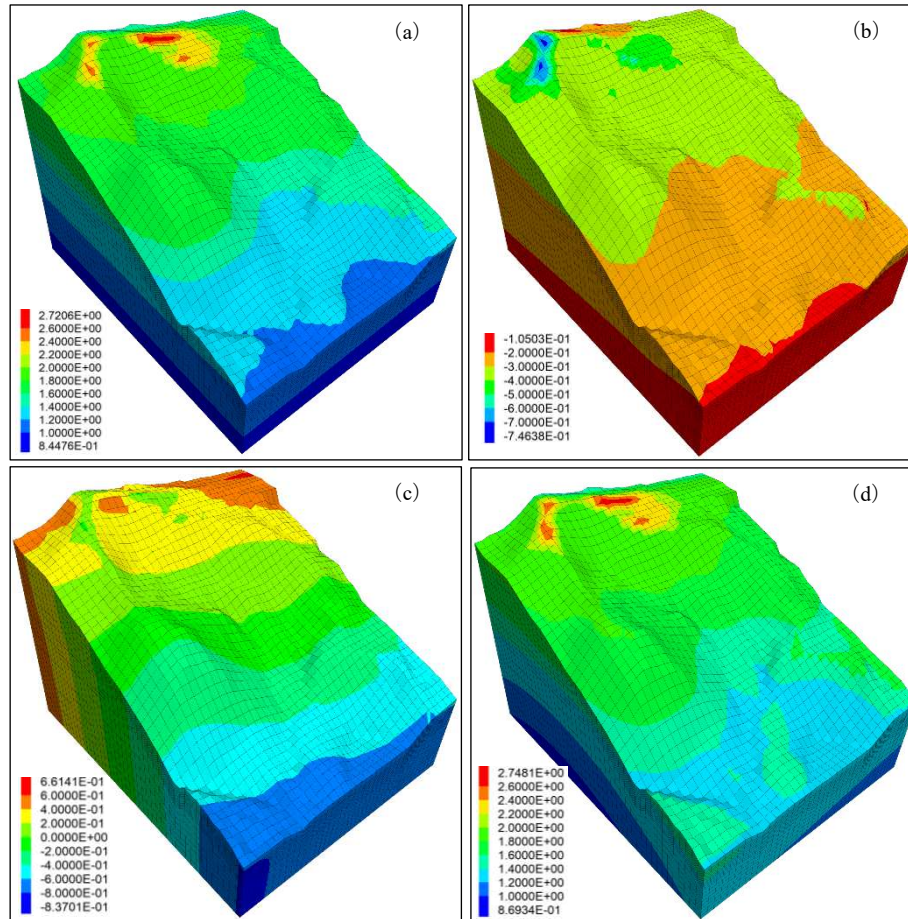


Fig. 11 Total-displacement fields at the end of shaking: (a) X, (b) Y, (c) Z and (d) magnitude. All fields include compliant-base translation. After subtraction of the reference-base vector, the maximum differential displacement magnitude is 1.865 m

The closest stable SSR state displayed a trial factor of 1.73, whereas the stable–unstable bracket gave a limiting factor of safety of 1.74. At this near-limit state, X displacement reached 22.605 m. Y displacement ranged from -7.906 to 3.255 m, and Z displacement ranged from -8.946 to 0.661 m (Fig. 12). These large values are numerical signatures of the trial failure mechanism, not postseismic permanent displacements. Localisation occurred in several upper and lateral sectors, with sign reversals between displacement components. This pattern is inconsistent with uniform translation of a rigid block along the mapped lithologic contact.

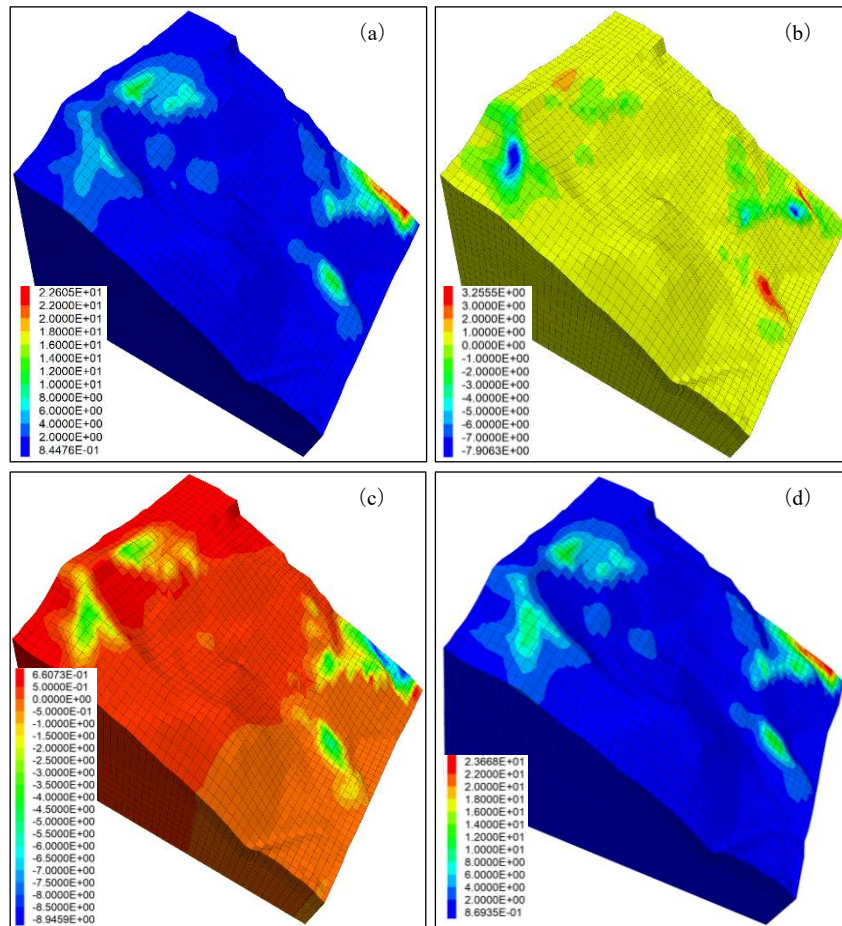


Fig. 12 Directional displacement fields in the closest stable postseismic SSR state: (a) X, (b) Y and (c) Z displacement. The displayed trial factor is 1.73, whereas the limiting factor of safety from the stable–unstable bracket is 1.74

5 Discussion

5.1 Investigation-based interpretation of lithologic contact behaviour

A lithologic contact can become a slip surface when three conditions coincide. It must be geometrically persistent and kinematically adverse, have lower shear resistance than the surrounding materials, and experience a stress or hydraulic path that mobilises that resistance. The site provides evidence for a continuous geological boundary, but not for the second or third conditions. No polished shear zone, clay seam, fault gouge, high-water-content layer or persistent groundwater was reported. Assigning low cohesion and friction to an interface would therefore prescribe the mechanism that the model was intended to test.

The numerical evidence instead supports distributed upper-slope deformation in the bonded dry-condition model. First, the 0.10 m threshold was reached earliest at the crest and midslope, not at the toe. Second, residual relative displacement increased from toe to crest rather than remaining uniform, as expected for rigid-block translation on a basal plane. Third, PGA was greatest at midslope because topographic focusing and impedance contrast need not peak at the crest. Fourth, the postseismic strain field comprised separate upper and lower clusters. Neither longitudinal section showed a continuous high-strain band along the mapped contact. Finally, FLAC3D state bits showed that 33,911 zones yielded at some point during shaking, but only 12 of 34,200 zones remained active at the terminal state. None of the 1,076 zones in the representative central section remained active (Fig. 13). The widespread past-yield flag records cyclic excursions, whereas the terminal and accumulated-strain fields do not require a single contact-controlled mechanism. Table 6 summarises this evidence test.

Comparison with published interface-controlled experiments clarifies this distinction. Huang et al. [7] introduced a bedding fault zone, Yu et al. [8] an anti-dip fault zone and Chen et al. [10] a high-water-content interlayer. In each case, the weak feature was represented as a distinct material rather than merely as a lithologic boundary. The dry baseline used here more closely represents an intact contact. It does not show that contact sliding is impossible. Instead, it shows that the available investigation does not justify a weak-interface model and that the computed response does not require one.

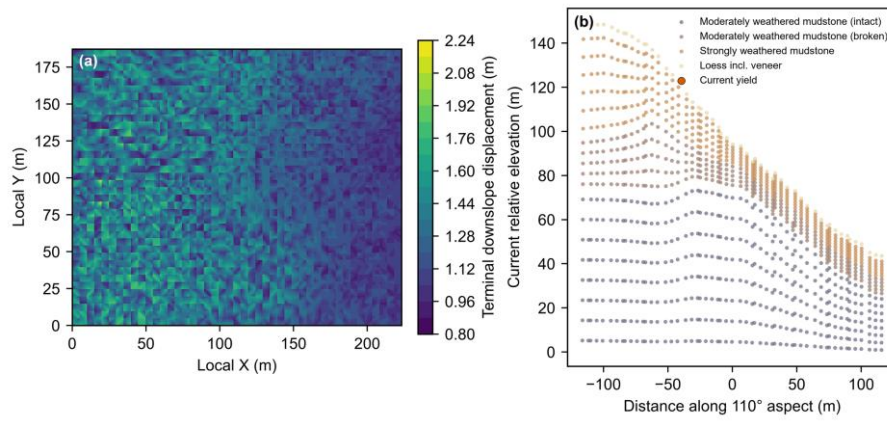


Fig. 13 Terminal response: (a) surface downslope displacement and (b) central stratigraphic section with currently yielding zones highlighted. Past-yield flags are omitted because they record cumulative cyclic excursions; no zone in the central section was actively yielding at the terminal state

Table 6 Evidence used to assess a contact-controlled mechanism

Required indicator	Site or model evidence	Assessment
Persistent weak seam or fault gouge	Not reported in boreholes, CT interpretation or geomorphological mapping	Not demonstrated
Low contact shear strength	No direct- or ring-shear data for the contact	Unknown; cannot be assigned defensibly
Elevated pore pressure/wet interlayer	No groundwater encountered; only seasonal fracture water was considered possible	Absent from the dry baseline; wet-season sensitivity is required
Kinematic alignment	The contact follows weathering geometry, but no slip-line orientation was measured	Locally possible, but insufficient on its own
Uniform block-like displacement above contact	Residual relative displacement increases from toe to crest	Not supported
Terminal contact-focused shear	12 of 34,200 zones remained active; none of the 1,076 central-section zones was active	No active contact-parallel band

The softening analysis strengthens, but does not prove, the interpretation that dry-condition deformation need not be controlled by a discrete loess–mudstone interface. In the unstable SSR state, 1,198 of 1,710 loess zones and 1,634 of 10,260 strongly weathered mudstone zones were actively yielding. The broken and intact mudstone units also participated. This multi-material mechanism is inconsistent with slip confined solely to the contact. However, because the model contained no interface, the mesh enforced displacement continuity and could not simulate opening or relative tangential slip along the geological contact.

5.2 Engineering implications for loess-mudstone slopes

The design-level analysis produced permanent deformation that warrants drainage, staged earthworks and multilevel monitoring. Response was concentrated in the upper and middle slope, where acceleration amplification and displacement gradients were greatest. The results do not imply movement of the entire slope as an intact block. Instead, they identify sectors where cyclic distortion, cracking and differential movement are most likely.

Mitigation should reflect both the investigation and the computed response. Surface water should be intercepted before reaching the loess–mudstone contact, and deformation monitoring should extend from crest to toe. Suitable instruments include three-component prisms or GNSS stations, inclinometers crossing the loess base, crack gauges and seasonal pore-pressure or suction sensors. Alert thresholds should be derived from monitored trends and construction-stage baselines, not copied directly from numerical displacement contours.

The model represents the investigated natural slope. Any future earthworks, support or drainage measures should therefore be introduced explicitly and analysed in stages. The retained geological ranges allow these extensions without discarding the investigation-constrained stratigraphic framework.

5.3 Limitations and falsifiable follow-up tests

The analysis used one horizontal ground-motion component scaled to a single intensity. It did not quantify record-to-record variability, vertical excitation, near-fault pulse effects or probabilistic demand. Damping and modulus reduction were not calibrated against site-specific resonant-column or cyclic-triaxial tests. The baseline Mohr–Coulomb model does not represent cyclic degradation, suction, collapse or reversible crack closure. Only the

supplementary loess case included a prescribed strain-softening law. These omissions concentrate uncertainty in acceleration peaks and permanent deformation after yielding.

Hydromechanical uncertainty is the principal limitation affecting the inferred mechanism. Wetting can reduce loess suction, soften mudstone and form a high-water-content layer along the contact. Experiments show that these processes can reverse the failure sequence and initiate sliding at the interface [10]. The conclusion is therefore conditional: the investigated dry contact was not compelled to slide. Wet-season and leakage scenarios require separate evaluation.

The interface hypothesis can be tested directly. Continuous core logging should document slickensides, gouge and remoulded seams. Undisturbed block samples should undergo constant-normal-stress or constant-normal-load direct shear at natural and saturated water contents. Field or laboratory measurements should constrain normal and shear stiffness, k_n and k_s . Nested piezometers should establish whether positive pore pressure develops along the contact. An explicit interface model should be accepted only if it reproduces observed relative displacement or contact-focused shearing more convincingly than the bonded baseline.

A continuum large-strain analysis remains a deformation model, not a robust tool for post-failure runout. Once mesh distortion becomes severe, the 2.24 m global maximum and 1.15 m crest-relative residual displacement indicate instability severity. They do not define travel distance. Quantitative runout analysis would require remeshing, material-point, coupled Eulerian–Lagrangian or discrete-element methods calibrated against post-peak strength and basal friction [23].

A further identifiability limitation arises from representing the merged loess veneer with only one vertical zone layer. Every loess zone is therefore adjacent to the loess–mudstone contact. An internal loess shear band cannot be separated geometrically from contact-parallel shearing. Local strain softening is also mesh-dependent. A definitive mechanism test requires finer vertical discretisation of the loess, acceptable zone aspect ratios and paired bonded and interface models. It also requires site-specific ring-shear or large-displacement direct-shear calibration at representative water contents and normal stresses.

5.4 Decision rule for an explicit contact

A future model should include an explicit interface only when doing so improves representation of the observed mechanics. A defensible workflow comprises four stages. Detection requires continuous core, outcrop mapping or geophysics to identify a persistent seam, fault, slickensided surface or remoulded band. Characterisation must constrain normal and shear stiffness, peak and residual strength, tensile capacity, dilation and hydraulic transmissivity. Calibration should reproduce interface-relative displacement, pore pressure or a mapped shear band without compensating changes to unrelated material properties. Validation should then predict an observation or loading stage not used during calibration.

Without this evidence, an interface model may be numerically precise but geologically underdetermined. For the investigated slope, the bonded-continuum model remains the reference hypothesis. Weak-contact sensitivity cases should be labelled as hypothetical. They should be evaluated using localisation depth, contact-relative displacement and pore-pressure demand, rather than global maximum displacement alone. Table 7 lists the principal uncertainties and the evidence required for further assessment.

Table 7 Principal uncertainties and recommended follow-up evidence

Uncertainty	Effect on interpretation	Recommended follow-up
Contact strength and stiffness	Determines whether the lithologic contact becomes a slip plane	Direct shear under natural and saturated conditions; logging of roughness and thickness
Water and suction	May soften mudstone and reduce effective stress in the loess	Seasonal piezometers, suction sensors and seepage analysis
Cyclic degradation	Influences PGA, damping and accumulated displacement	Cyclic-triaxial or resonant-column testing
Input variability	A single record cannot represent uncertainty in spectrum and duration	At least seven spectrally compatible records with intensity scaling
Construction disturbance	May alter the free face, stress path and support demand	Staged earthwork and support analysis
Post-failure runout	Continuum meshes lose predictive fidelity after localisation	MPM, CEL or DEM analysis calibrated against residual strength

6 Conclusions

Integrated 1:500 topography, borehole logs, in-hole velocity measurements and seismic CT constrained the three-dimensional geological model of a loess–mudstone slope in Tianshui. The investigation defined cover thickness, weathering depth and three mudstone integrity domains. It also found no persistent groundwater table, old landslide or

continuous weak interlayer. These observations reduced structural ambiguity and avoided assigning an unsupported weak-interface law to the lithologic contact.

The dry initial slope had a factor of safety of 1.83, yet the 0.30 g excitation produced a strongly non-monotonic response. Downslope PGA was 0.54 g at the base, 1.29 g at midslope and 0.94 g at the crest. The midslope amplification factor of 2.40 exceeded the crest value of 1.76. Three-dimensional topographic and impedance effects focused transient motion near slope breaks, whereas nonlinear upper-slope deformation dissipated energy. Lower PGA at the crest did not indicate less damage. Residual relative downslope displacement increased from 0.34 m at the toe to 0.64 m at midslope and 1.15 m at the crest.

Total displacement at the end of shaking was dominated by the X component, but included compliant-base translation. After subtracting base motion, the maximum differential displacement magnitude was 1.865 m. Localised Y displacement and the sign change in Z displacement indicated lateral distortion, settlement, heave and rotation. Because the component extrema occurred at different locations, the slope did not undergo uniform rigid-block translation.

Mild post-peak degradation increased the maximum displacement magnitude by 22.4% and reduced the postseismic factor of safety to 1.74. Mean accumulated plastic shear strain in the loess was 0.0127, and 15.9% of the loess zones reached residual strength. These zones formed two separate plan-view clusters. Longitudinal sections showed concentration at the crest and upper shoulder without a continuous basal high-strain band. A more severe residual-strength law caused dynamic instability at 6.03 s, demonstrating the sensitivity of permanent deformation to post-peak calibration.

Together, the investigation and response fields support distributed upper-slope deformation under the investigated dry condition. They do not show that contact sliding is impossible. A definitive test requires finer vertical discretisation of the loess, paired bonded and interface models, direct- or ring-shear testing at representative normal stresses, and seasonal pore-pressure or suction observations. The central conclusion is therefore conditional: a mapped loess–mudstone boundary is not, by itself, evidence of a controlling weak plane.

Statements and Declarations

Availability of data and materials

The data used for this study are available upon request from the corresponding author.

Competing interests

The authors declare that they have no competing interests.

Acknowledgements

The authors would like to express their gratitude to the editors and anonymous reviewers for their valuable comments and suggestions. We also thank our colleagues for their insightful discussions and technical support.

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