

Article

Investigation of Degradation Mechanism of Unsaturated Shear Strength at Geogrid–Sandy-Soil Interface Under Rainfall Infiltration

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Abstract

Reinforced-soil structures in rainfall-prone regions may deform or fail when infiltration weakens the geogrid–soil interface. This study quantifies the degradation of unsaturated shear strength at a geogrid–sandy-soil interface during rainfall infiltration. A large-scale direct shear apparatus was retrofitted with a controllable rainfall system and real-time water-content monitoring. Interface shear tests were conducted under different normal stresses, rainfall intensities, infiltration durations, and shear rates. Peak interface shear strength increased approximately linearly with normal stress and remained about 50% higher than that of unreinforced sand. Rainfall infiltration caused pronounced strength loss; at $120 \text{ mm}\cdot\text{h}^{-1}$, extending infiltration from 10 to 30 min reduced apparent cohesion by ~56% and friction angle by ~23%. Cohesion decayed exponentially, whereas friction angle decreased nearly linearly, and faster shearing intensified both reductions. Response-surface regression further indicates that degradation is most severe under low normal stress, high rainfall intensity, and long infiltration duration. Water-content profiles reveal a persistent moisture-enriched zone adjacent to the shear plane (~3.4% higher than at 30 mm depth), implying reduced matric suction and promoting shear-band localization that accelerates interface weakening. These findings provide quantitative input for evaluating rainfall-induced performance loss of geogrid-reinforced soil structures.

Keywords: geogrid; geogrid–sandy-soil; rainfall infiltration; large-scale direct shear; unsaturated shear strength; response surface; water-content distribution

1. Introduction

In rainfall-prone regions with frequent extreme storms, reinforced retaining walls, embankments, and slopes are commonly exposed in service to the combined effects of prolonged infiltration and short-duration intense rainfall. Engineering statistics indicate that strength deterioration induced by rainfall infiltration and inadequate drainage is a key contributor to excessive deformation and instability of reinforced-soil structures [1,2]. The geogrid–soil interface is a critical weak link governing stress transfer and deformation compatibility; its strength degradation directly reduces reinforcement effectiveness and shifts the position of the potential slip surface.

Importantly, the interface response is not equivalent to shear failure within a homogeneous backfill. The interface constitutes a thin, heterogeneous contact layer where load

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transfer is mediated by particle–rib contact, geometric roughness, and mechanical interlocking and where local hydraulic conditions can deviate from the bulk soil. In particular, the geogrid ribs promote interlocking while the apertures provide preferential seepage and redistribution pathways in the vicinity of the interface, making local wetting and suction decay potentially more pronounced than in unreinforced soil. Therefore, incorporating the geogrid–soil interface is essential for quantifying rainfall-induced degradation mechanisms and for translating laboratory observations into design-relevant interface parameters for reinforced-soil structures.

Mechanistically, rainfall infiltration alters the water-content and pore-water-pressure fields near the interface, reducing matric suction and unsaturated effective stress while changing the particle–geogrid contact state. Studies on unsaturated shear strength have established effective-stress-based frameworks. Building on Bishop’s effective stress concept [3], Fredlund et al. [4–8] formulated shear strength using net normal stress and matric suction as independent stress-state variables and developed SWCC-based parameterized prediction methods. Khalili and Khabbaz [9] proposed a unified relationship between χ and the suction ratio, whereas Lu and Likos [10] introduced suction stress (σ_s) and the suction-stress characteristic curve to describe unsaturated effective stress. Related theories and reviews indicate that suction-related strength components exhibit pronounced non-linearity and hysteresis during wetting [11–13]. However, their applicability to the heterogeneous contact layer at the geogrid–soil interface—and the associated parameter identification—still lacks adequate experimental validation.

Numerous direct shear and pullout studies have investigated geogrid–soil interface behavior. Recent large-size direct shear evidence on soil–geosynthetic interfaces further highlights the importance of interface strength characterization for reinforced-soil design [14]. Large shear-box testing of soil–constructions provides complementary context for selecting interface parameters under different contact conditions [15]. Under dry or constant water content conditions, studies typically examine how soil properties, geogrid type, and shear rate affect interface friction and interlocking and have proposed various empirical strength models. Regarding moisture effects, Ziaie-Moayed and Kamalzare [16] used a large-scale direct shear apparatus to examine cohesive soil–geosynthetic interfaces and found that, as water content approaches the plastic limit, both cohesion and friction angle decrease markedly, leading to peak-strength reductions of 40–60%. Chen et al. [17] reported for a sand–geogrid system that increasing water content reduces interface strength and dilatancy and that the match between particle size and geogrid aperture is critical. For unsaturated interfaces, Hamid [18] and Jotisankasa [19] showed that interface shear strength varies nonlinearly with matric suction.

However, most prior work was conducted under constant water content or simplified hydraulic boundary conditions, which cannot capture the transient development of water-content gradients and pore-water pressure buildup during rainfall events; quantitative characterization of local moisture-enriched zones near the interface also remains limited. In recent large-scale direct shear tests, Mirzaeifar et al. [20] compared sand–geogrid interfaces using shear boxes of different sizes and found that larger boxes yield systematically lower measured interface friction angles; this scale effect is more pronounced in dense sands and across different geogrid configurations. Lin and Wang [21] investigated geogrid–soil interface strength under rainfall infiltration and reported that rainfall markedly reduces interface cohesion and alters the failure mode.

Overall, quantitative characterization of near-interface moisture-enriched zone formation and the associated evolution of interface strength parameters under rainfall-driven transient infiltration remains insufficient. The present work provides (i) an integrated large-scale interface direct shear platform that couples controllable rainfall infiltration with real-time, multi-depth moisture monitoring in the vicinity of the geogrid–soil

interface, enabling transient hydro-mechanical observations beyond constant-water-content tests; (ii) a systematic multi-factor dataset covering the coupled effects of normal stress P , rainfall intensity R , infiltration duration T , and shear rate v , including unreinforced controls to decouple reinforcement gain from rainfall-induced weakening; (iii) reproducible evidence and quantification of a persistent moisture-enriched zone adjacent to the shear plane, together with an SWCC-based conversion from $\Delta\omega$ to $\Delta\psi$ to link local wetting to suction decay; and (iv) engineering-oriented models, including response-surface characterization of $\tau_p(P, R, T)$ and the distinct functional evolutions of $c(T)$ and $\phi(T)$, interpreted mechanistically in terms of suction loss, lubrication, and interlocking-to-sliding transition. A complete list of symbols and notations is provided in Appendix A.

2. Test Materials and Experimental Design

Research Scope and Objectives. To directly address the above objectives, a large-scale direct shear system was integrated with a controllable rainfall simulator and in situ water-content sensors positioned above the interface to track the development of vertical moisture gradients during wetting. The experimental program was designed by varying normal stress, rainfall intensity, infiltration duration, and shear rate, while measuring the corresponding changes in peak/residual interface strength and the derived Mohr–Coulomb parameters. This design enables a direct linkage between rainfall-induced near-interface wetting (and suction reduction) and the observed strength degradation and failure characteristics at the geogrid–soil interface.

2.1. Test Materials and Physical Properties

Quartz sand with 10% fines (Henan Chuguo Environmental Protection Technology Co., Ltd., Henan, China) was used as the fill material and is hereafter termed sandy soil, as shown in Figure 1. The fines had particle sizes <0.075 mm, a specific gravity of 2.71, and a plasticity index of 18. The physical properties of the sandy soil are summarized in Table 1, and the particle-size distribution curve is shown in Figure 2. The tested sandy soil exhibited relatively uniform gradation and consisted primarily of quartz sand with 10% fines [22]. In the dry state, load transfer was governed mainly by interparticle friction; after wetting, shear strength became sensitive to variations in water content. To represent the medium-dense condition commonly encountered in engineering practice, specimens were prepared by layered placement and vibration compaction to a dry density of approximately $1700 \text{ kg}\cdot\text{m}^{-3}$ and a relative density D_r of 80% [23]. Prior to testing, the initial water content was maintained at 7%, which is sufficiently low to be treated as a dry condition.



(a)



(b)



Figure 1. Experimental materials. (a) Quartz sand; (b) fine-grained soil; (c) specimen; (d) geogrid.

Table 1. Physical indices of the sandy soil.

Sandy Soil Type	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	Maximum Void Ratio (e_{max})	Minimum Void Ratio (e_{min})	Specific Gravity	Coefficient of Uniformity (C_u)	Coefficient of Curvature (C_c)
Sandy soil	0.075	0.835	1.064	0.67	0.44	2.656	14.18	8.74

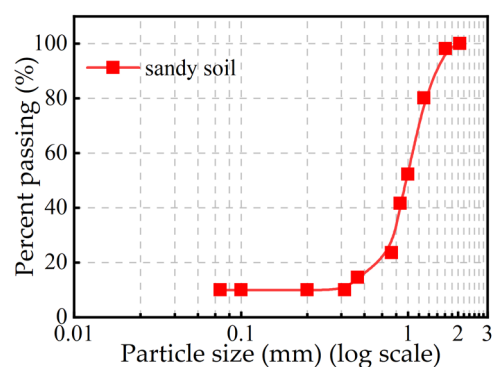


Figure 2. Particle-size distribution curve of the tested sandy soil.

A biaxial polypropylene geogrid (Wuhan Changfei General Cable Co., Ltd., Wuhan, China) was used, and its physical properties are listed in Table 2. The geogrid had a mass per unit area of approximately $0.33 \text{ kg}\cdot\text{m}^{-2}$ and an aperture size of about $35 \text{ mm} \times 35 \text{ mm}$; the ultimate tensile strengths in the machine and cross-machine directions were approximately 45.8 and $46.3 \text{ kN}\cdot\text{m}^{-1}$, respectively, and the ultimate elongation was 9.8 – 9.9% . The geogrid ribs exhibit high stiffness and a pronounced cross-section, which promotes mechanical interlocking with surrounding sand particles. The relatively large apertures facilitate stable seepage pathways for rainfall near the geogrid–soil interface and allow the deformation during shearing to better represent stress transfer in reinforced-soil masses in practice.

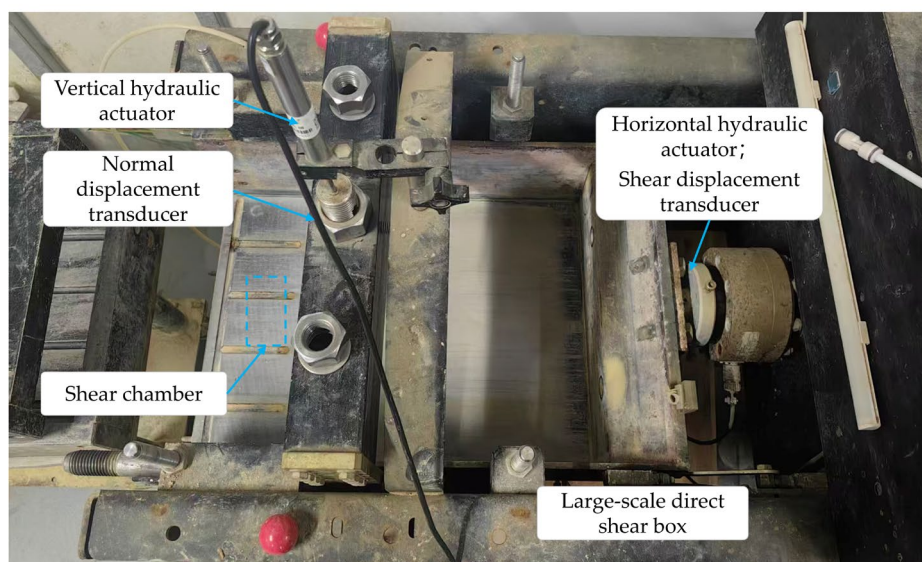
Table 2. Technical indices of the geogrid.

Geogrid Type	Mass per Unit Area ($\text{kg}\cdot\text{m}^{-2}$)	Aperture Size (mm)	Ultimate Elongation (%)		Ultimate Tensile Strength ($\text{kN}\cdot\text{m}^{-1}$)	
			Cross-Machine Direction	Machine Direction	Cross-Machine Direction	Machine Direction
polypropylene geogrid	0.33	35×35	9.8	9.9	45.8	46.3

2.2. Experimental Apparatus and Rainfall Simulation System

The interface direct shear tests were performed using a HUMBOLDT large-scale multifunctional direct shear apparatus (Humboldt Mfg. Co., Elgin, IL, USA). The upper and lower shear boxes had internal dimensions of 305 mm × 305 mm × 100 mm and 405 mm × 305 mm × 100 mm, respectively, corresponding to a total shear area of approximately 0.093 m². The apparatus can apply a normal stress of 0–800 kPa with a shear displacement stroke of 0–100 mm, and the shear rate can be continuously adjusted from 0.1 to 10.0 mm·min^{−1}. The system integrates normal and shear load sensors, displacement transducers, and a data acquisition module, enabling synchronous recording of the shear stress–displacement response and the normal deformation.

To simulate rainfall infiltration (Figure 3), the large-scale direct shear apparatus was retrofitted with a rainfall simulator. As shown in Figure 3c, four atomizing nozzles were mounted symmetrically above the loading assembly with overlapping spray coverage. Rainfall water was sprayed directly onto the perforated loading plate, which served as the imposed infiltration boundary. The plate contains 10 evenly distributed holes within the shear area, and the sprayed water infiltrated into the specimen through these perforations, thereby wetting the zone above the shear plane without being blocked by the solid portion of the plate. To minimize localized ponding and to promote redistribution among perforations, a ~5 mm highly permeable layer was placed immediately beneath the perforated plate. This layer promoted rapid vertical infiltration and more uniform redistribution of water, thereby preventing a persistent water film or localized accumulation. Two water-content sensors (Weihai Jingxun Changtong Electronic Technology Co., Ltd., Weihai, Shandong, China) were installed at 10 and 30 mm above the shear plane. They measured volumetric water content θ with a nominal accuracy of $\pm 0.5\%$. This configuration enabled continuous monitoring of near-interface moisture evolution during infiltration and shearing. The retrofit preserved the loading stiffness and bearing capacity required for large-scale direct shear testing.



(a)

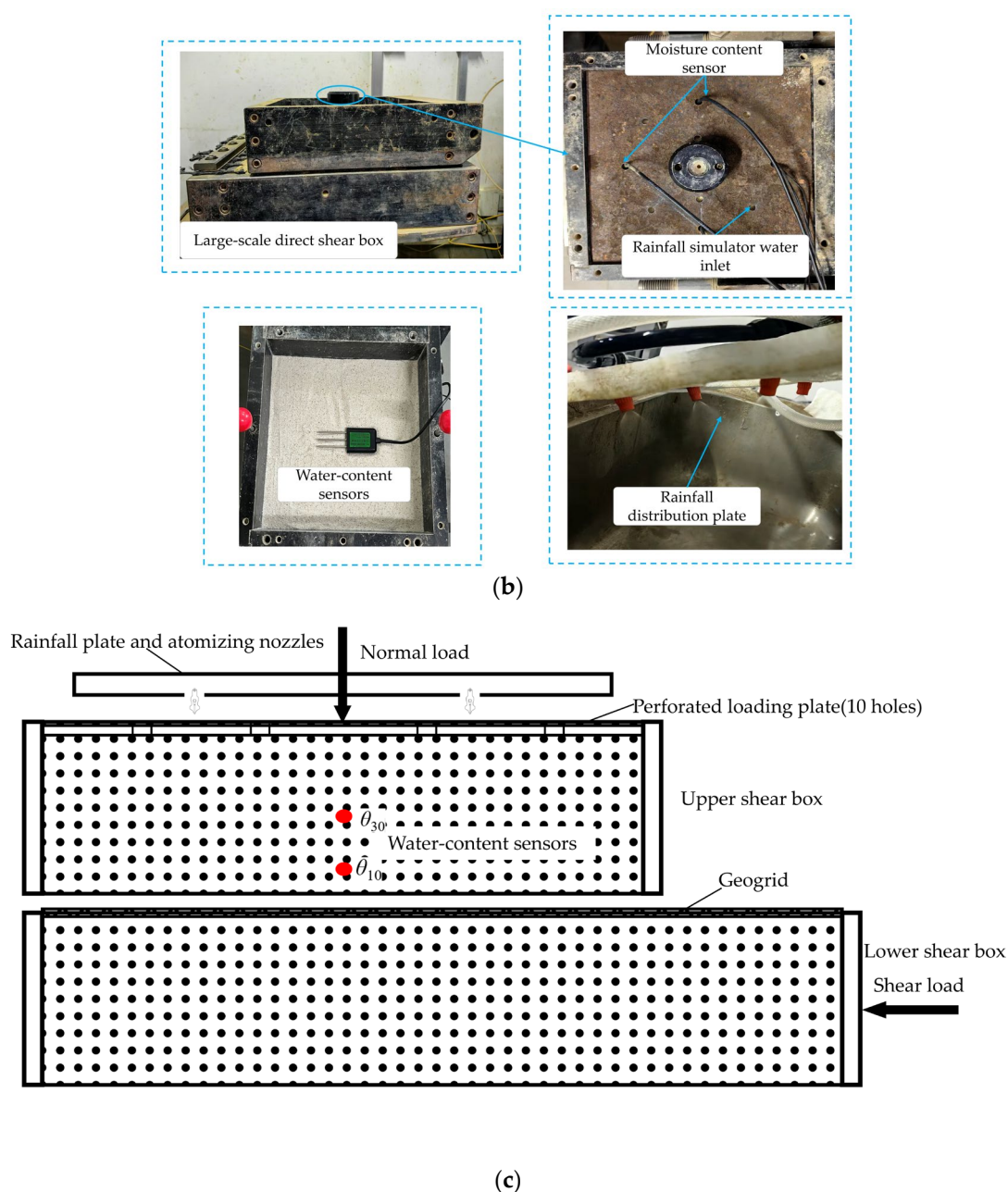


Figure 3. Test equipment and schematic diagram. (a) Large direct shearing machine; (b) shear box, rainfall distribution plate, and moisture content sensor; (c) schematic diagram of the test profile.

To ensure comparability across test conditions and to support subsequent numerical analyses, all tests employed identical initial and boundary conditions. The initial dry density and initial water content were kept constant for all specimens to eliminate the influence of initial state on interface strength. The shear-box sidewalls were rigid and impermeable, imposing no-flow lateral boundaries ($q_x = q_y = 0$). Rainfall was applied at the specimen surface as a prescribed infiltration-flux boundary ($q_z \approx R$), whereas drainage holes and an outlet at the base provided a vertical outflow (free drainage) boundary (approximately $p \approx 0$). With a specimen thickness of $H = 100$ mm and a characteristic plan dimension of $L = 305$ mm ($L/H \approx 3.05$; half-width/ $H \approx 1.53$), the configuration was designed to promote a predominantly vertical, quasi-one-dimensional seepage path for systematic comparison of the effects of R and T .

2.3. Experimental Program and Loading Procedure

The experimental program followed the Standard for Geotechnical Testing Method (GB/T 50123–2019) [24]. Interface direct shear tests were performed in strain-controlled mode [25]. Normal stresses were set to $P = 50, 100, \text{ and } 150 \text{ kPa}$. Shear rates were set to $v = 1.2 \text{ and } 2.0 \text{ mm}\cdot\text{min}^{-1}$ [26]. Following the Precipitation Intensity Grades standard [27], hourly rainfall intensities were used as proxies for daily intensity grades [28]. Rainfall intensity was set to $R = 0, 60, 120, \text{ and } 180 \text{ mm}\cdot\text{h}^{-1}$, where $R = 0 \text{ mm}\cdot\text{h}^{-1}$ served as the dry control. Infiltration duration was set to $T = 10, 20, \text{ and } 30 \text{ min}$. An unreinforced sand control group was included to separate rainfall-induced weakening from reinforcement-induced strengthening. The test matrix is summarized in Table 3. Each condition was tested in triplicate, and the results reported herein represent the arithmetic mean of the three runs. Because the individual-run datasets were not retained after post-processing, variability metrics (e.g., standard deviation/variance) cannot be reported for the present dataset.

Table 3. Test condition scheme.

Reinforcement	Rainfall Intensity R ($\text{mm}\cdot\text{h}^{-1}$)	Infiltration Duration T (min)	Normal Stress P (kPa)	Shear Rate v ($\text{mm}\cdot\text{min}^{-1}$)
none	0	0	50, 100, 150	1.2, 2.0
none	120	10	50, 100, 150	1.2, 2.0
geogrid	0	0	50, 100, 150	1.2, 2.0
geogrid	60	10, 20, 30	50, 100, 150	1.2, 2.0
geogrid	120	10, 20, 30	50, 100, 150	1.2, 2.0
geogrid	180	10, 20, 30	50, 100, 150	1.2, 2.0

The loading procedure was identical for all tests. The target normal stress P was applied and held for 10 min to complete initial consolidation. The simulated rainfall system was then activated to impose infiltration according to the prescribed R and T . After infiltration for T , horizontal shearing was initiated while maintaining P and R constant. The specimen was sheared at the prescribed rate v until failure or a stable residual stage was reached. Throughout the test, shear stress, shear displacement, and normal displacement were recorded continuously, and the evolution of water content at 10 and 30 mm above the interface was monitored simultaneously.

2.4. Hydraulic Boundary Conditions: Verification and Field Implications

The imposed near-1D boundary condition is primarily intended for controlled mechanistic comparison; however, it does not fully represent the hydraulic complexity of reinforced slopes and retaining walls. In the field, lateral drainage may occur through drainage layers, weep holes, toe drains, or along slope faces, while preferential flow paths (e.g., along heterogeneities, cracks, or construction interfaces) may lead to spatially non-uniform wetting. These processes can either (i) dissipate infiltrating water laterally and reduce the persistence of moisture gradients near the reinforcement interface or (ii) locally concentrate wetting in limited zones. Therefore, the moisture enrichment and strength degradation observed in the laboratory should be interpreted as most representative of conditions with limited lateral drainage (e.g., poorly drained systems or near-imperious facings) and may be conservative relative to well-drained structures. For engineering application, we recommend combining site-specific seepage assessment (or monitoring) with the proposed degradation trends to relate the local wetting/suction evolution near the interface to the controlled laboratory boundary condition.

During testing, the specimen surface was continuously inspected, and no evident overflow, persistent ponding, or visible runoff paths along the boundaries were observed under any tested combination of R and T . Because surface runoff requires the formation

of a persistent water film/ponding, we provide a conservative upper-bound estimate for potential (undetected) runoff using specimen geometry and the known rainfall input. Let A denote the infiltration area ($A \approx 0.093 \text{ m}^2$). To reflect uncertainty in the visually detectable ponding threshold, we consider a sensitivity range of ponding thickness $h_p = 0.3\text{--}1.0 \text{ mm}$. The corresponding maximum surface water volume is $V_p = Ah_p$, giving $V_p = 2.79 \times 10^{-5}\text{--}9.30 \times 10^{-5} \text{ m}^3$ (27.9–93.0 mL). The rainfall input is computed as $V_{in} = R \cdot A \cdot (T/60)$. Under the most runoff-prone condition in this study ($R = 180 \text{ mm} \cdot \text{h}^{-1}$, $T = 30 \text{ min}$), $V_{in} = 8.37 \text{ L}$, yielding an upper-bound runoff ratio $C_{r,max} = V_p/V_{in} = 0.33\text{--}1.11\%$. This indicates that any potential runoff, if present, would remain small relative to the imposed rainfall input. Therefore, the applied rainfall intensity R was taken as the infiltration-flux boundary condition in the subsequent analyses.

3. Analysis of Test Results

3.1. Evolution and Vertical Distribution Characteristics of Water Content

To facilitate SWCC-based theoretical analysis, the measured volumetric water content θ was converted to gravimetric water content ω using Equation (1):

$$\omega = \theta \cdot \frac{\rho_w}{\rho_d} \quad (1)$$

In Equation (1), ρ_d is the dry density and ρ_w is the density of water. Unless otherwise specified, “water content” hereafter refers to the gravimetric water content ω .

Figures 4 and 5 present gravimetric water content (ω)–displacement curves at monitoring points 10 and 30 mm above the interface under $R = 120 \text{ mm} \cdot \text{h}^{-1}$. The tests cover different combinations of infiltration duration T and normal stress P . At the end of infiltration, ω at 10 mm increased rapidly and then approached a plateau. In contrast, the increase at 30 mm was smaller. During shearing, ω at 10 mm remained high and showed only minor displacement-related fluctuations. Water content at 30 mm changed more gradually. For $P = 50 \text{ kPa}$, the difference between the two depths was $\Delta\omega = 3.4\%$. This gradient was reproducible across different combinations of P , T , and v . It indicates a stable moisture-enriched zone adjacent to the shear plane, with higher saturation and lower matric suction. Because the enriched zone coincides with the interface shear plane and c and φ decrease markedly with increasing T , local moisture enrichment is inferred to be a primary driver of interface strength degradation and preferential sliding. Under low P , high R , and long T , sustained high ω near the interface reduces suction and weakens friction and interlocking. This promotes shear localization within the enriched zone. As a result, peak shear stress can drop sharply and residual strength may become very low under extreme conditions. Time-history data further show that ω at 10 mm rises quickly during early infiltration, whereas the increase at 30 mm is slower. After shearing begins, small oscillations at 10 mm likely reflect drainage and redistribution within the shear band. Changes at 30 mm remain minor, suggesting that shear deformation is confined to a thin zone near the interface. Across different combinations of P and T , $\Delta\omega$ remains relatively stable. Even at higher P , the 10 mm point typically remains $\sim 3\text{--}4\%$ higher than the 30 mm point, indicating limited drainage of the enriched zone.

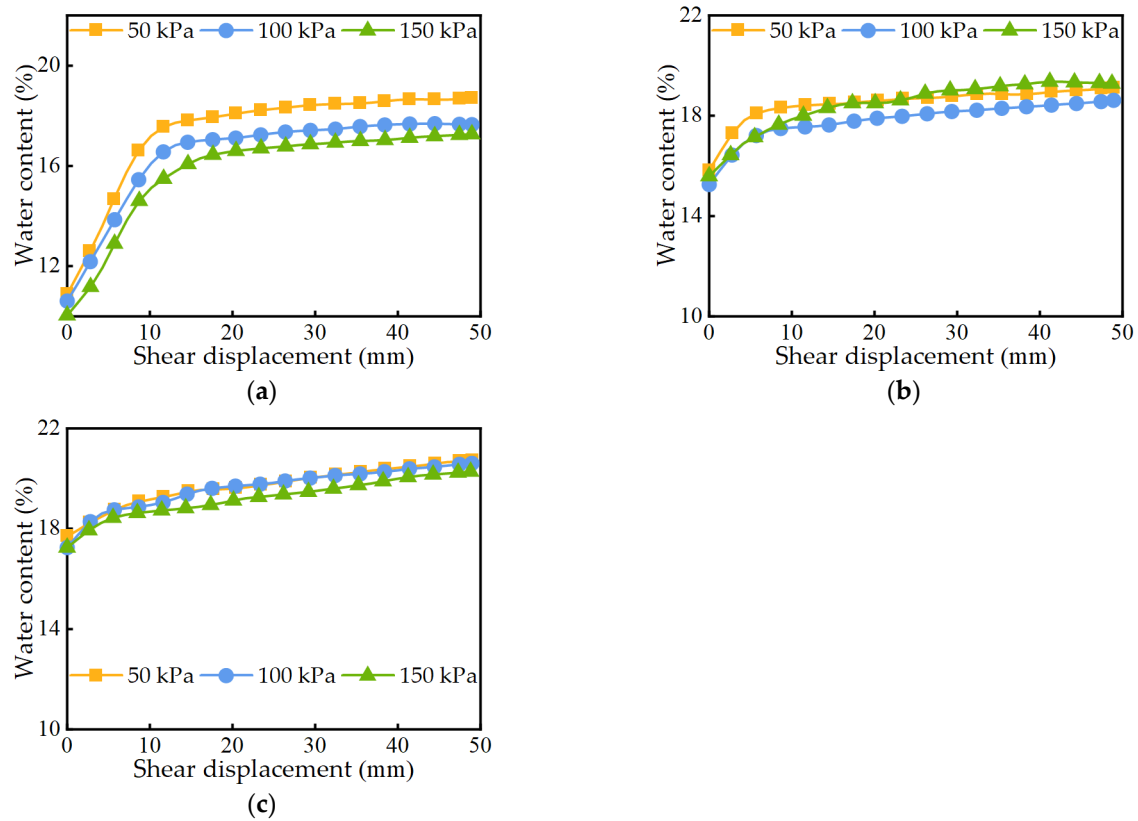


Figure 4. Moisture content–shear displacement curves at 10 mm from the interface of reinforced sand. (a) $T = 10$ min; (b) $T = 20$ min; (c) $T = 30$ min.

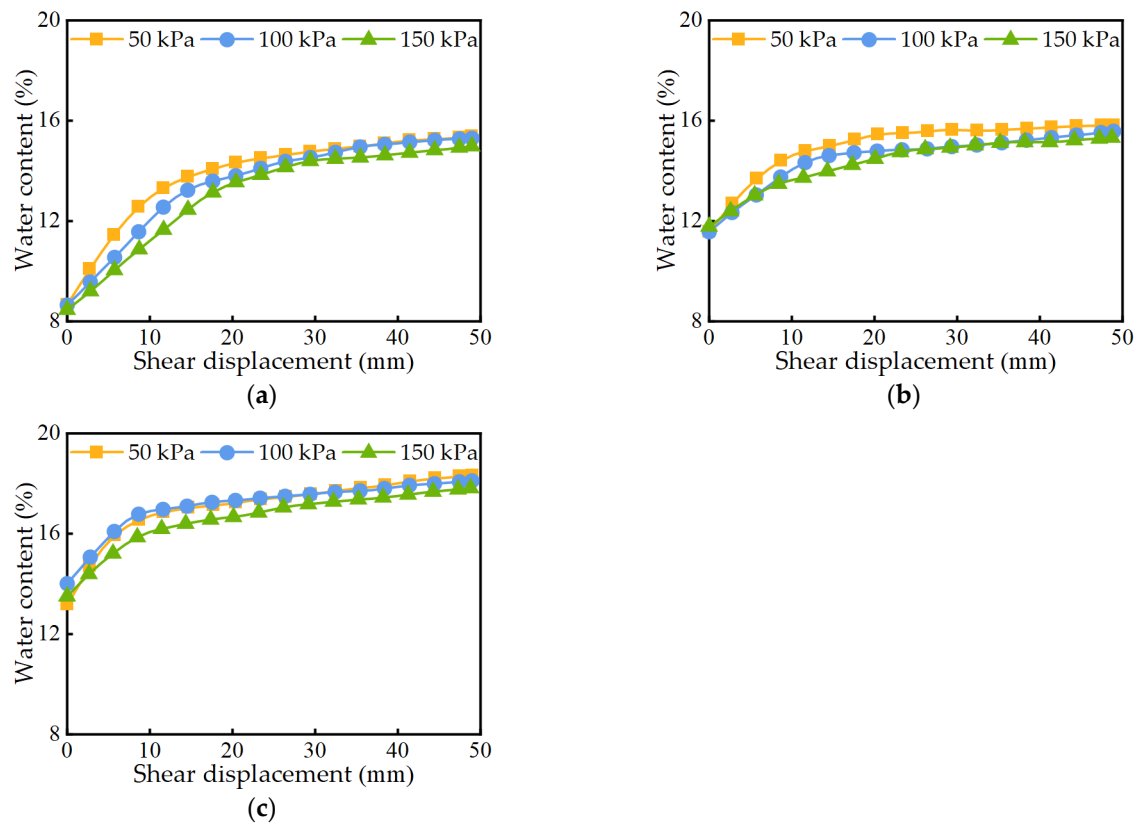


Figure 5. Moisture content–shear displacement curves at 30 mm from the interface of reinforced sand. (a) $T = 10$ min; (b) $T = 20$ min; (c) $T = 30$ min.

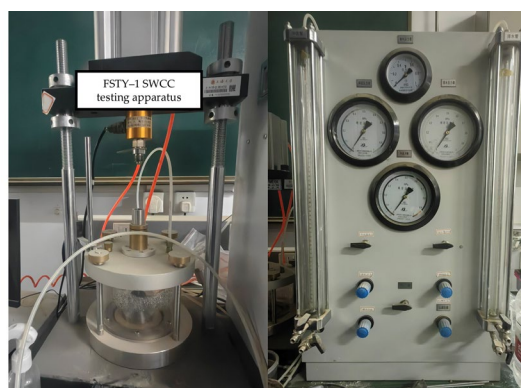
To relate the stable water-content gradient $\Delta\omega$ (Figures 4 and 5) to the strength-degradation mechanism within an unsaturated effective stress framework, the soil–water characteristic curve (SWCC) was used to map $\Delta\omega$ to the matric suction difference $\Delta(\mu_a - \mu_w)$. This mapping converts the qualitative statement that moisture enrichment reduces suction into a quantified magnitude of suction decay. For sand–clay mixtures, Elkady et al. [29] obtained the drying SWCC using a hanging column ($\psi < 5$ kPa) and a pressure plate ($\psi = 5$ –1500 kPa). In this study, the wetting SWCC of the specimens was measured using an FSTY-1 soil–water characteristic testing apparatus (Liyang, Jiangsu, China) (Figure 6a) to establish the relationship between gravimetric water content ω and matric suction $\psi = (\mu_a - \mu_w)$. The Van Genuchten water-retention model [30] was fitted (Figure 6b) by least squares to obtain the empirical ω – ψ conversion used herein:

$$\omega(\psi) = \omega_r + \frac{\omega_s - \omega_r}{\left[1 + (\alpha\psi)^n\right]^m}, \quad m = 1 - \frac{1}{n} \quad (2)$$

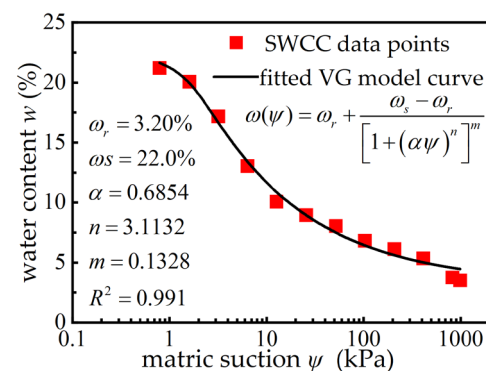
In Equation (2), $\psi = (\mu_a - \mu_w)$ is the matric suction (kPa). ω is the gravimetric water content (%) and is consistent with the water content defined above. ω_s , ω_r , α , n , and m are fitting parameters. Table 4 summarizes the fitted parameters used to construct the continuous SWCC.

Table 4. Van Genuchten fitting parameters for sand with 10% fines.

Parameter	Value
ω_s (%)	22.00
ω_r (%)	3.20
α (kPa ^{−1})	0.6854
n	3.1132
m	0.1328



(a)



(b)

Figure 6. Reference SWCC for sand with 10% fines and Van Genuchten fitting. (a) FSTY-1 SWCC testing apparatus; (b) measured wetting SWCC and Van Genuchten fitting.

Based on the water contents under $P = 50$ kPa in Figures 4a and 5a, $\omega = 18.7\%$ at 10 mm above the interface and $\omega = 15.3\%$ at 30 mm, giving $\Delta\omega = 3.4\%$. Back-calculation using the fitted ω – ψ relationship in Table 4 yields $\psi = 2.14$ kPa near the shear plane and $\psi = 4.19$ kPa at 30 mm above the interface, $\Delta\psi = 2.05$ kPa. This suction decay is sufficient to markedly reduce the additional shear resistance contributed by the suction term in unsaturated effective-stress formulations, thereby accelerating interface strength degradation. From the perspective of unsaturated soil mechanics, the shear strength of the sandy soil within the shear band adjacent to the interface can be described using Bishop’s effective stress or equivalent formulations [9–13]:

$$\tau_f = c' + (\sigma - \mu_a) \tan \varphi' + \chi(\mu_a - \mu_w) \tan \varphi' \quad (3)$$

In Equation (3), $(\sigma - \mu_a)$ is the net normal stress, $(\mu_a - \mu_w)$ is the matric suction, and χ is a weighting function related to the degree of saturation [1–7,11]. In this study, the normal stress P governs $(\sigma - \mu_a)$, whereas rainfall infiltration and local moisture enrichment govern $(\mu_a - \mu_w)$ and $\chi(S_r)$ together; these terms determine the effective stress level and the suction contribution to interface strength [9–13]. The suction decay was concentrated within approximately 10 mm above the interface, indicating that infiltrating water preferentially accumulates within the interface contact layer and forms a locally weakened zone characterized by high water content and low suction.

It should be noted that the thickness of this moisture-enriched (weakened) zone cannot be uniquely determined from two discrete moisture measurements. Here we define an operational thickness δ_e as the near-interface region exhibiting moisture substantially higher than the background level. With sensors located at 10 and 30 mm above the shear plane, the present data constrain δ_e to be at least on the order of 10 mm while remaining less than ~30 mm for the tested cases, i.e., 10 mm captures the enriched core and 30 mm represents a less-affected zone. Moreover, δ_e is expected to evolve with infiltration; it develops rapidly near the interface during early wetting and can expand with increasing rainfall intensity R and infiltration duration T , consistent with the observed stronger degradation under high- R –long- T conditions. Once infiltration progresses sufficiently, the relatively stable $\Delta\omega$ during shearing suggests a quasi-stationary moisture gradient in the test timescale. A thicker/enhanced near-interface wet zone further reduces suction/effective stress and increases lubrication, thereby promoting preferential shear localization adjacent to the interface. A more definitive quantification of the enriched-zone thickness would require additional depth sensors or continuous moisture/suction profiling across the near-interface region.

Within this zone, the suction contribution term $\chi(\mu_a - \mu_w)$ becomes small and the effective stress level is reduced, so friction and interlocking at particle–rib contacts are preferentially weakened. This provides a hydro-mechanical basis for shear band localization near the interface. Furthermore, when the suction contribution is expressed in terms of Mohr–Coulomb parameters [1], it is manifested primarily as a reduction in apparent cohesion, with a more gradual decrease in the friction angle.

Note on χ evaluation. In the present study, χ was not calibrated as an independent constitutive parameter. Instead, the Mohr–Coulomb parameters (c and φ) regressed from the peak shear strength under each infiltration condition are treated as apparent parameters, within which the suction contribution $\chi(\mu_a - \mu_w)$ is implicitly embedded—primarily in the apparent cohesion term.

To provide an explicit linkage between measured water content, suction, and strength, we further adopt a commonly used wetting-path approximation, $\chi \approx S_r$. Assuming the void ratio remains approximately constant during infiltration, S_r can be estimated from gravimetric water content as $S_r \approx \omega G_s / e$, with e computed from the reported specimen state ($\rho_d \approx 1.700 \text{ kg}\cdot\text{m}^{-3}$; $G_s = 2.656$). For the representative measurements ($\omega = 18.7\%$ at 10 mm and $\omega = 15.3\%$ at 30 mm; $\psi = 2.14 \text{ kPa}$ and 4.19 kPa), the resulting $\chi\psi$ term decreases near the shear plane, which is consistent with the rapid decay of the apparent cohesion observed during infiltration. This estimate is intended to improve transparency of the hydro-mechanical interpretation; full calibration of $\chi(S_r)$ would require additional independent measurements beyond the scope of this work.

3.2. Shear Stress–Displacement Curves Under Different Normal Stresses and Reinforcement Effects

Taking $v = 1.2 \text{ mm}\cdot\text{min}^{-1}$ as an example, Figures 7 and 8 present shear stress–shear displacement curves for geogrid-reinforced and unreinforced sandy soil under rainfall

infiltration conditions at different normal stresses P . The figures show that, under dry and relatively weak rainfall conditions, unreinforced specimens exhibit a friction-dominated strain-hardening response with an indistinct peak, where shear stress increases gradually with displacement and approaches a plateau. In contrast, reinforced specimens generally exhibit a pronounced peak, followed by post-peak softening; shear stress rises rapidly at small displacements, reaches the peak τ_p at a displacement of 10–20 mm (3–7% of the shear-plane length), and then decreases toward a residual level, indicating interface sliding and shear band localization. Under a given rainfall condition, τ_p increases approximately linearly with P , consistent with the Mohr–Coulomb criterion [1]. Compared with unreinforced specimens, τ_p of reinforced specimens is ~50% higher across the tested P and rainfall conditions, with a relatively stable gain at $P = 100$ kPa. Under rainfall infiltration, for the same P and R , τ_p of reinforced specimens remains higher than that of unreinforced specimens, indicating a net strength gain despite degradation. Increasing P elevates the overall stress level and delays the onset of softening. Under high P and moderate rainfall, the reinforced sandy soil retained more than 70% of its dry-state strength. Under low P , intense rainfall combined with longer T leads to pronounced reductions in τ_p and, in some cases, the residual shear stress can become very low, indicating rapid loss of interface load capacity. Mechanistically, under dry conditions, interface resistance is provided by interparticle friction and interlocking among sand grains, augmented by geometric roughness and mechanical interlocking mobilized by the geogrid ribs. The pronounced peak and post-peak softening suggest that interlocking is mobilized at small displacements and then transitions toward residual frictional control as local sliding develops. Rainfall infiltration increases ω near the interface and reduces matric suction; concurrently, water-film lubrication weakens interparticle friction and reduces the mobilization efficiency of interlocking, promoting a shift from interlocking to sliding and causing concurrent reductions in peak and residual strengths under low P or long T . In contrast, higher P can partially offset the adverse effects of lubrication and suction reduction by increasing contact normal stress and confinement, thereby maintaining a net reinforcement gain even under intense rainfall. To facilitate direct comparison, Figure 9 overlays the reinforced and unreinforced stress–displacement curves under identical P for the dry condition ($R = 0$ mm·h^{−1}, $T = 0$ min) and a representative infiltration condition ($R = 120$ mm·h^{−1}, $T = 10$ min). This consolidated plot enables an immediate assessment of the reinforcement-induced peak and softening behavior and its reduction under infiltration. Accordingly, the presence of a geogrid interface introduces additional degradation mechanisms (interlocking loss and interface-localized weakening) that should be explicitly considered when assessing rainfall-induced performance of reinforced-soil structures.

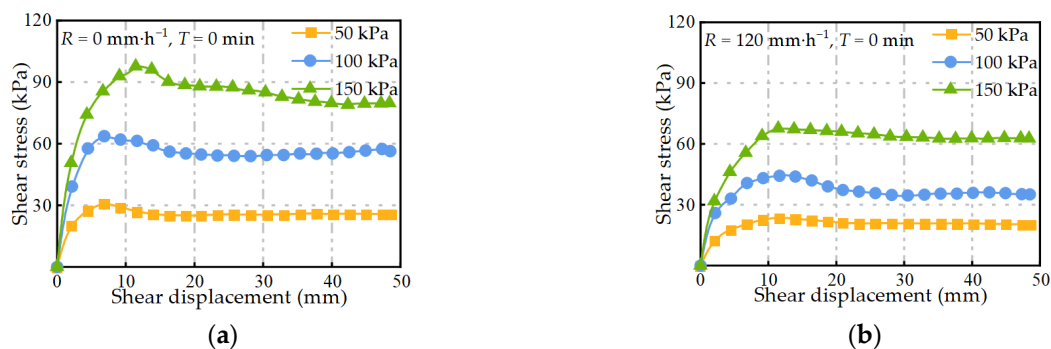


Figure 7. Shear stress–shear displacement curves of sand under different normal stress. (a) $R = 0$ mm·h^{−1}, $T = 0$ min; (b) $R = 120$ mm·h^{−1}, $T = 10$ min.

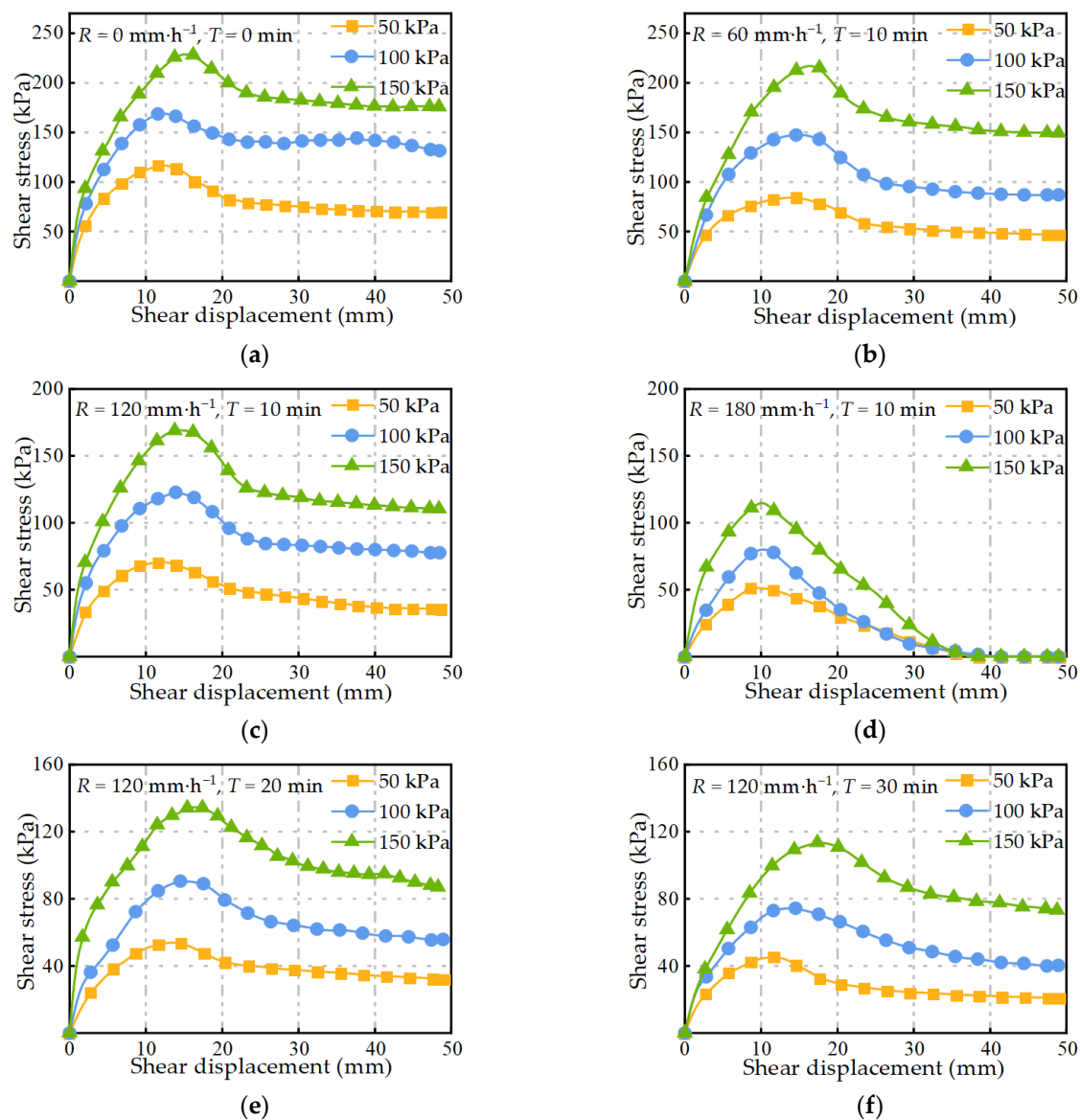
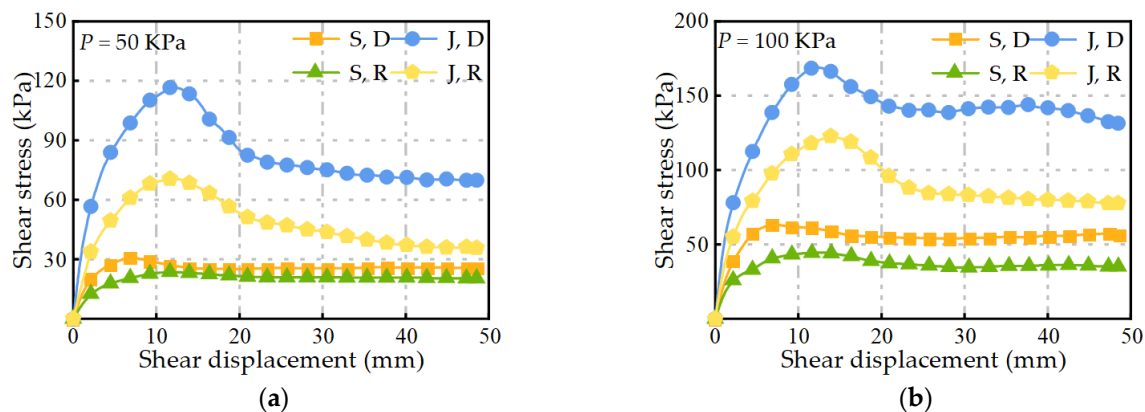


Figure 8. Shear stress–shear displacement curves of reinforced sand under different normal stress. (a) $R = 0 \text{ mm}\cdot\text{h}^{-1}$, $T = 0 \text{ min}$; (b) $R = 60 \text{ mm}\cdot\text{h}^{-1}$, $T = 10 \text{ min}$; (c) $R = 120 \text{ mm}\cdot\text{h}^{-1}$, $T = 10 \text{ min}$; (d) $R = 180 \text{ mm}\cdot\text{h}^{-1}$, $T = 10 \text{ min}$; (e) $R = 120 \text{ mm}\cdot\text{h}^{-1}$, $T = 20 \text{ min}$; (f) $R = 120 \text{ mm}\cdot\text{h}^{-1}$, $T = 30 \text{ min}$.



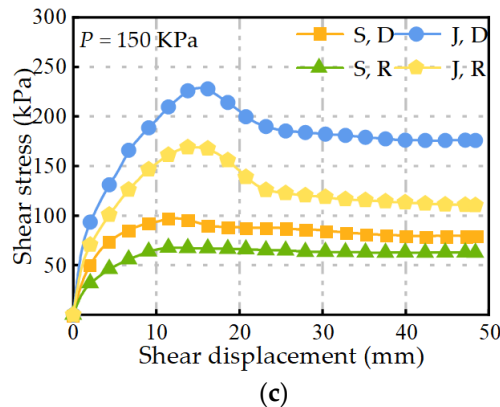


Figure 9. Direct comparison of shear–stress displacement responses with and without geogrid reinforcement under matched conditions at $v = 1.2 \text{ mm} \cdot \text{min}^{-1}$. In the figure, S represents unreinforced, J represents reinforced, D represents dry condition, and R represents $R = 120 \text{ mm} \cdot \text{h}^{-1}$. (a) $P = 50 \text{ kPa}$; (b) $P = 100 \text{ kPa}$; (c) $P = 150 \text{ kPa}$.

3.3. Evolution of Strength Parameters Under the Coupled Effects of Infiltration Duration and Shear Rate

Figure 10 shows shear stress–shear displacement curves of geogrid-reinforced sandy soil under $R = 120 \text{ mm} \cdot \text{h}^{-1}$ for different infiltration durations T . With increasing T , the response evolves from a distinct peak with pronounced softening to an indistinct peak with low residual strength, indicating progressive interface strength degradation. Under $P = 50 \text{ kPa}$, the curve at $T = 10 \text{ min}$ shows a clear peak at small displacement with limited post-peak softening. When T increases to 30 min , the peak level decreases markedly and post-peak stress drops abruptly; the residual stress can become very low, exhibiting brittle-like softening. Under $P = 100$ and 150 kPa , shear stress also decreases with increasing T , but the residual level remains finite; notably, at $P = 150 \text{ kPa}$ the curve exhibits an extended plateau, indicating that higher confinement mitigates strength degradation. As T increases from 10 to 30 min , the peak displacement shifts slightly to the right, particularly at higher P and v , which may suggest wetting-induced structural relaxation near the interface and shear band widening, requiring larger displacement to mobilize residual strength. Regarding shear rate v , Figure 10b,d,f show that curves at $v = 2.0 \text{ mm} \cdot \text{min}^{-1}$ are generally lower than those at $v = 1.2 \text{ mm} \cdot \text{min}^{-1}$, with reduced peak and residual stresses and steeper post-peak softening. This suggests that, under the same infiltration condition, a higher v may limit local drainage and redistribution, thereby amplifying strength reduction and interface degradation. Notably, pore-water pressure was not monitored; hence any pore-pressure-related explanation is inferential. Considering the impermeable sidewalls and basal drainage configuration, together with the shortened shearing duration at higher v , the tests at $v = 2.0 \text{ mm} \cdot \text{min}^{-1}$ are more consistent with a partially drained to near-un drained interface response, especially under wetter (high- R /long- T) conditions.

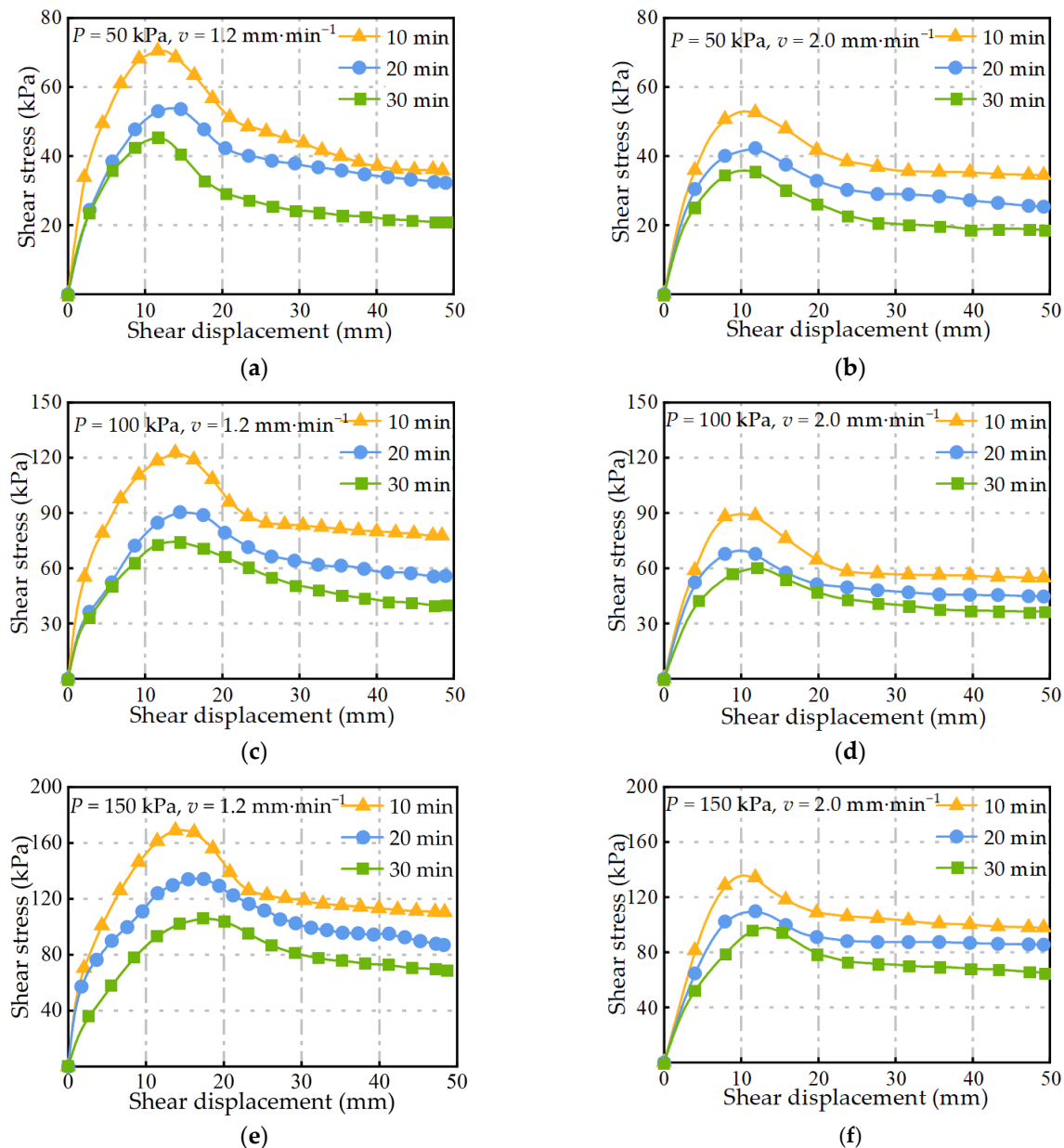


Figure 10. Shear stress–shear displacement curves of reinforced sand under different infiltration duration. (a) $P = 50$ kPa, $v = 1.2$ mm·min⁻¹; (b) $P = 50$ kPa, $v = 2.0$ mm·min⁻¹; (c) $P = 100$ kPa, $v = 1.2$ mm·min⁻¹; (d) $P = 100$ kPa, $v = 2.0$ mm·min⁻¹; (e) $P = 150$ kPa, $v = 1.2$ mm·min⁻¹; (f) $P = 150$ kPa, $v = 2.0$ mm·min⁻¹.

Based on the relationships between peak shear stress τ_p and normal stress P at different infiltration durations T , the interface cohesion c and friction angle φ were obtained by linear regression using the Mohr–Coulomb criterion [1]:

$$\tau_p = c + \sigma' \tan \varphi \quad (4)$$

In Equation (4), τ_p is the peak shear stress and P is the normal stress. The regression yields cohesion c and friction angle φ for each infiltration duration T . Figure 11 and Table 5 summarize the parameters and goodness-of-fit metrics for different T under $R = 120$ mm·h⁻¹. At $v = 1.2$ mm·min⁻¹, c decreases from 22.24 kPa ($T = 10$ min) to 12.46 kPa ($T = 20$ min) and then to 9.80 kPa ($T = 30$ min). Over the same T range, φ decreases from 44.63° to 38.87° and then to 34.22°. The coefficients of determination R^2 range from 0.985 to 0.998, indicating excellent linear fits. Therefore, increasing T from 10 to 30 min reduces c by ~56.3%

and ϕ by $\sim 22.5\%$, showing that c is more sensitive to infiltration. At $v = 1.2 \text{ mm}\cdot\text{min}^{-1}$, c equals 9.57, 6.63, and 2.30 kPa for $T = 10, 20$, and 30 min , respectively. The corresponding ϕ values are $39.00^\circ, 34.02^\circ$, and 32.17° . For a given T , both c and ϕ are generally lower than those at $v = 1.2 \text{ mm}\cdot\text{min}^{-1}$. This suggests that higher v limits local drainage/redistribution and amplifies strength reduction. When $T = 30 \text{ min}$, the reduction in c exceeds 75% .

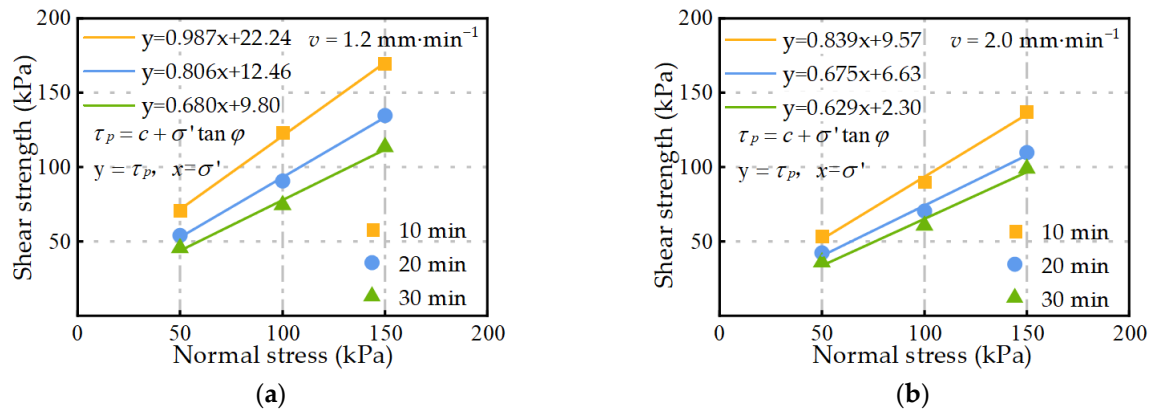


Figure 11. Shear strength curves of sandy soil with different infiltration duration. (a) $v = 1.2 \text{ mm}\cdot\text{min}^{-1}$; (b) $v = 2.0 \text{ mm}\cdot\text{min}^{-1}$.

Table 5. Strength indices with different infiltration duration.

Infiltration Duration T (min)	Cohesion c (kPa)	Friction Angle ϕ ($^\circ$)	Coefficient of Determination R^2	Fitted Curve	Shear Rate v ($\text{mm}\cdot\text{min}^{-1}$)
10	22.24	44.63	0.998	$y = 0.987x + 22.24$	1.2
20	12.46	38.87	0.994	$y = 0.806x + 12.46$	
30	9.80	34.22	0.985	$y = 0.680x + 9.80$	
10	9.57	39.00	0.989	$y = 0.839x + 9.57$	2.0
20	6.63	34.02	0.982	$y = 0.675x + 6.63$	
30	2.30	32.17	0.968	$y = 0.629x + 2.30$	

Based on the peak shear stress–normal stress relationship for each infiltration duration, a linear Mohr–Coulomb regression was performed within the tested stress range ($\sigma_n = 50\text{--}150 \text{ kPa}$) to obtain equivalent apparent parameters (c and ϕ). While unsaturated shear strength can be nonlinear with respect to suction and degree of saturation, under the rainfall-infiltration conditions examined here the matric suction adjacent to the interface is on the order of only a few kPa (e.g., $\psi \approx 2.14\text{--}4.19 \text{ kPa}$), which is small relative to σ_n in the tested range; thus, curvature in the $\tau_p\text{--}\sigma_n$ response is not pronounced in this window. The resulting regressions show high goodness of fit (Table 5), and residual diagnostics (e.g., Figure A1) do not indicate systematic stress-dependent deviations, supporting the applicability of linear envelopes for $\sigma_n = 50\text{--}150 \text{ kPa}$. Given that only three normal-stress levels were tested, potential curvature at lower σ_n ($<50 \text{ kPa}$)—which may be more evident under intense wetting/high rainfall—requires further verification with expanded stress levels; therefore, no extrapolation is made beyond the tested range.

To facilitate engineering calculations and parameter extrapolation, $c(T)$ and $\phi(T)$ were fitted within the tested range; the fitted parameters and coefficients of determination are given in Equation (5):

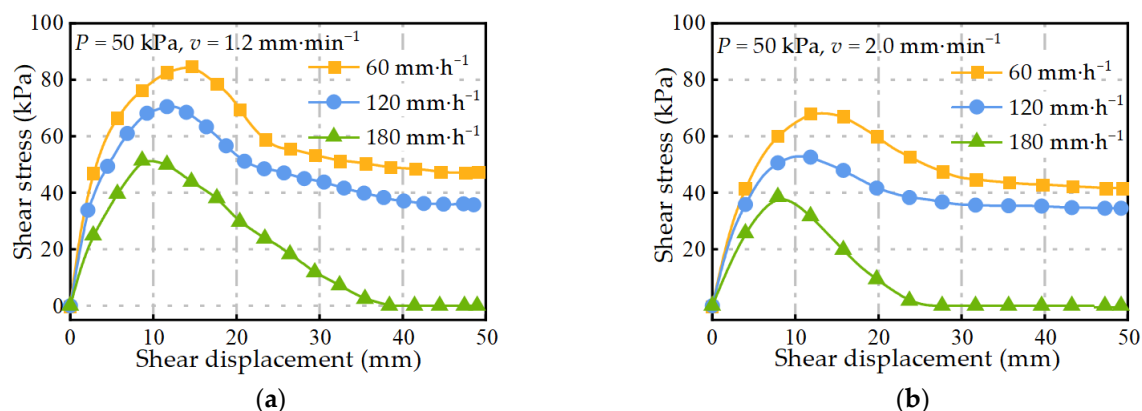
$$c(T) = c_0 e^{(-\alpha T)}, \phi(T) = \phi_0 - \beta T \quad (5)$$

Under the present test conditions, $c_0 \approx 22.5 \text{ kPa}$, $\alpha \approx 0.03 \text{ min}^{-1}$, $\phi_0 \approx 45^\circ$, and $\beta \approx 0.35^\circ \text{ min}^{-1}$, with $R^2 \approx 0.95$. Combined with the short-term suction decay near the interface

(Section 3.1), the c and φ values derived from Mohr–Coulomb regression [1] can be regarded as apparent parameters reflecting the coupled evolution of unsaturated effective stress and contact structure. Accordingly, the fitted cohesion c should be interpreted as an apparent cohesion incorporating the suction contribution $\chi(\mu_a - \mu_w)$, rather than a purely saturated-material parameter. Within Bishop’s effective stress framework [9–13], the suction term can be expressed as an apparent cohesion, $c_a = \chi(\mu_a - \mu_w)\tan\varphi'$. Along the wetting path, decreases in ψ and saturation-dependent changes in χ jointly cause c_a to decay rapidly during early infiltration, resulting in an exponential decrease in $c(T)$; as the interface approaches a high-water-content state, ψ tends toward a residual level and the decay rate of c correspondingly slows. In contrast, the approximately linear decrease in $\varphi(T)$ more directly reflects gradual weakening of interface roughness and interlocking; thickening water films and lubrication at particle–rib contacts increase the proportion of sliding, leading to a slow reduction in the equivalent friction angle. When v increases, local drainage and redistribution within the shear band may become insufficient and may be accompanied by pore-water pressure changes that reduce the net normal stress, yielding smaller c and φ at the same T . This reflects coupled amplification between rainfall infiltration and shear drainage conditions.

3.4. Rainfall Intensity and Multifactor Response Surface

Figure 12 presents shear stress–displacement curves at $T = 10$ min under different rainfall intensities R . At $R = 60 \text{ mm}\cdot\text{h}^{-1}$, shear stress increases over a relatively large displacement range and reaches a higher peak. Post-peak softening is mild, and the residual level remains high. This behavior is most evident at $P = 100$ – 150 kPa , where a distinct peak–plateau response is observed. When R increases to $120 \text{ mm}\cdot\text{h}^{-1}$, peak shear stress τ_p decreases markedly. The post-peak segment becomes steeper, and residual shear stress drops, indicating reduced interface stiffness and load-carrying capacity. At $R = 180 \text{ mm}\cdot\text{h}^{-1}$ and $P = 50 \text{ kPa}$ (Figure 12a,b), the peak decreases substantially and post-peak stress rapidly drops to a very low level. Residual strength is nearly lost. This indicates the formation of a pronounced weakened layer near the interface, which may rapidly evolve into a slip surface once shear localizes within it. At $P = 100$ – 150 kPa (Figure 12c–f), the curves retain a discernible peak even at $R = 180 \text{ mm}\cdot\text{h}^{-1}$. This suggests that higher normal stress partially offsets rainfall-induced degradation by enhancing particle–geogrid contact and interlocking. Shear rate v also modulates the response. Compared with $v = 1.2 \text{ mm}\cdot\text{min}^{-1}$, tests at $v = 2.0 \text{ mm}\cdot\text{min}^{-1}$ show lower peak stress, steeper post-peak softening, and smaller residual stress. A higher v likely limits local drainage/redistribution and promotes pore-water pressure accumulation, which further reduces effective stress and aggravates rainfall-induced weakening.



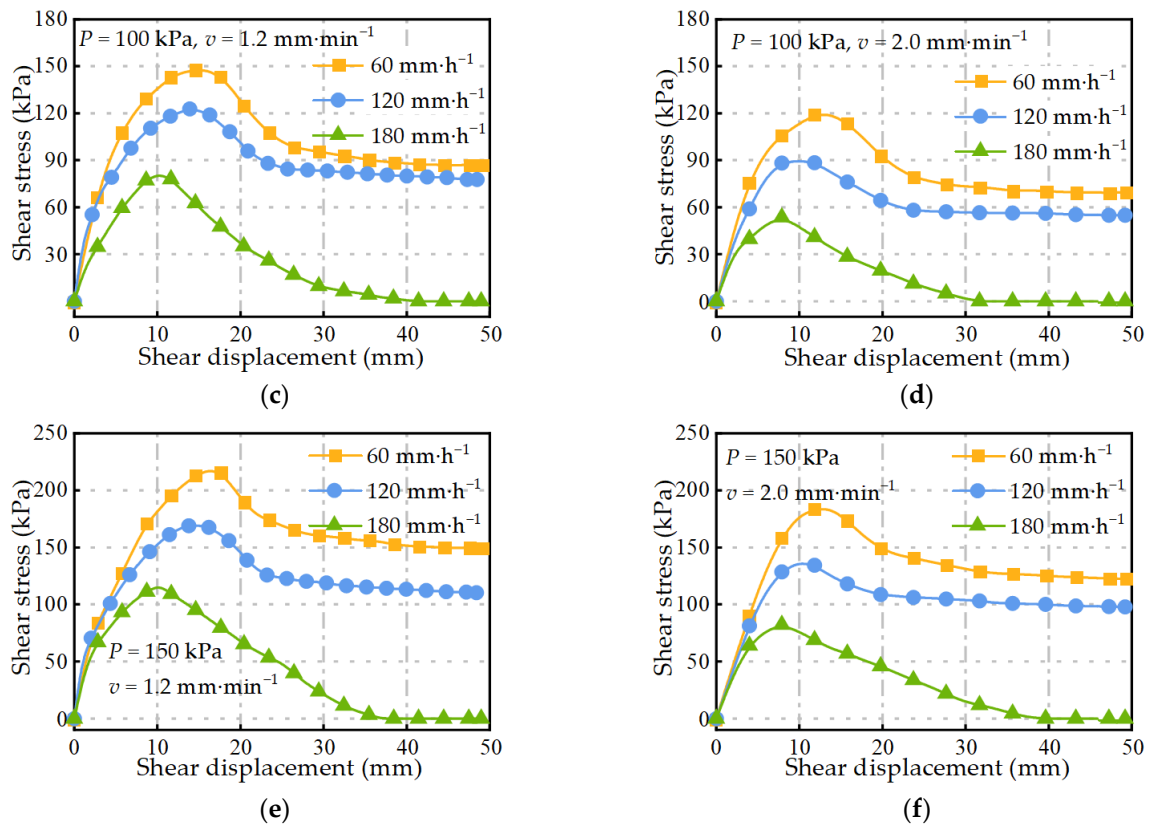


Figure 12. Shear stress–shear displacement curves with different rainfall intensity. (a) $P = 50$ kPa, $v = 1.2$ mm·min^{−1}; (b) $P = 50$ kPa, $v = 2.0$ mm·min^{−1}; (c) $P = 100$ kPa, $v = 1.2$ mm·min^{−1}; (d) $P = 100$ kPa, $v = 2.0$ mm·min^{−1}; (e) $P = 150$ kPa, $v = 1.2$ mm·min^{−1}; (f) $P = 150$ kPa, $v = 2.0$ mm·min^{−1}.

To quantify multifactor coupling, we developed a second-order (quadratic) response surface model using the measured peak shear stress τ_p . Normal stress P , rainfall intensity R , and infiltration duration T were used as predictors, and τ_p was the response. We fitted two-dimensional slices, $\tau_p(P, R)$ and $\tau_p(R, T)$, on the (P, R) and (R, T) planes, respectively. The model achieves $R^2 = 0.93$, indicating good agreement with the overall trend of τ_p .

Figure 13 quantifies how τ_p responds to P and R at different shear rates v . At both shear rates, τ_p increases monotonically with P and decreases monotonically with R . The curvature of the surfaces indicates strong coupling between these factors. In the low-rainfall-intensity regime, τ_p is highly sensitive to P , and the contours show a steep gradient along the P direction. Under $P = 100$ – 150 kPa, τ_p retained more than 70% of the dry-state strength in this study. In the high-rainfall-intensity regime, the marginal gain in τ_p diminishes even as P increases to 150 kPa. The contours flatten, suggesting that confinement alone cannot fully offset degradation under intense rainfall. Comparing Figure 13a,b shows that increasing v from 1.2 to 2.0 mm·min^{−1} shifts the entire surface downward. The downward shift is larger at high R , indicating that rainfall-induced weakening is amplified at higher shear rates.

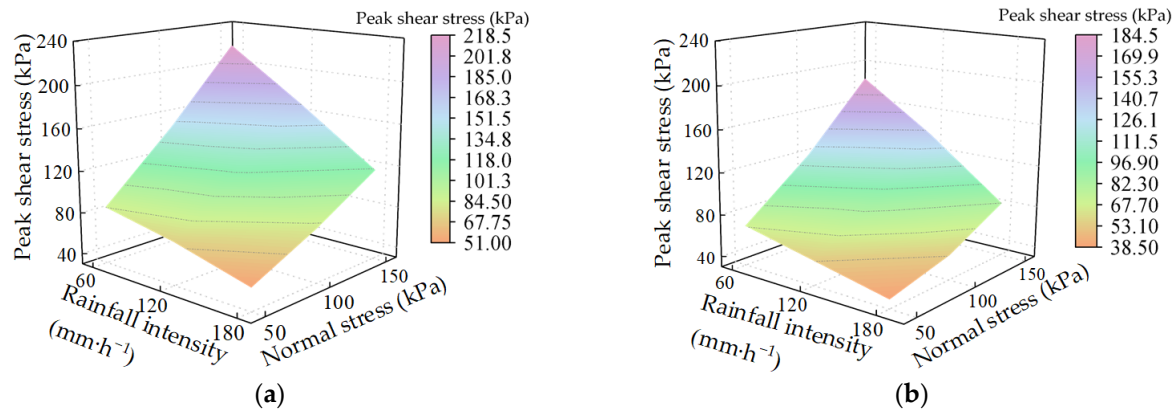


Figure 13. Rainfall intensity–normal stress–peak shear stress. (a) $v = 1.2 \text{ mm} \cdot \text{min}^{-1}$; (b) $v = 2.0 \text{ mm} \cdot \text{min}^{-1}$.

Figure 14 further quantifies the coupled effects of rainfall intensity R and infiltration duration T . For short infiltration durations, τ_p decreases markedly with increasing R . In this regime, the surface is relatively flat along the T direction, indicating that R dominates because infiltration has not yet produced a moisture-enriched weakened zone of appreciable thickness beneath the interface. When T increases to 20–30 min, the surface becomes strongly concave along both the R and T directions. Under $R = 180 \text{ mm} \cdot \text{h}^{-1}$ and $T = 30 \text{ min}$, τ_p decreases by 63% relative to the dry condition. This corner depression becomes more pronounced at higher v , indicating synergistic amplification between high rainfall intensity and long infiltration duration. This trend is consistent with increases in near-interface moisture and expansion of the high-saturation weakened zone. Comparing Figure 14a,b also shows that, at $v = 2.0 \text{ mm} \cdot \text{min}^{-1}$, the $\tau_p(R, T)$ surface is lower and steeper than at $v = 1.2 \text{ mm} \cdot \text{min}^{-1}$. The corner drop under long-duration, high-intensity rainfall is therefore more pronounced at the higher shear rate.

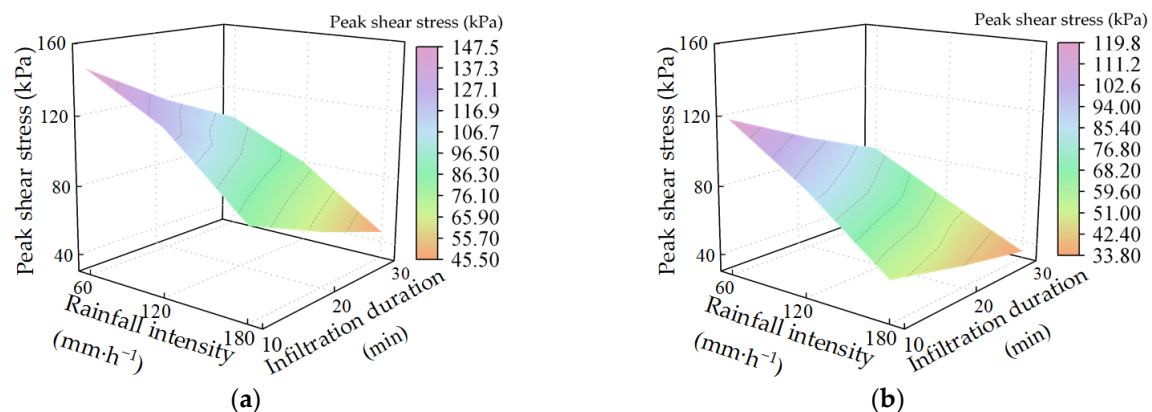


Figure 14. Rainfall intensity–infiltration time–peak shear stress. (a) $v = 1.2 \text{ mm} \cdot \text{min}^{-1}$; (b) $v = 2.0 \text{ mm} \cdot \text{min}^{-1}$.

The low- P –high- R –long- T corner depression on the response surface reflects a non-linear reduction in interface effective stress under coupled hydro-mechanical processes. Increasing R raises the infiltration flux and accelerates wetting-front arrival at the shear plane, which rapidly increases near-interface saturation. A longer T allows the moisture-enriched zone to continue growing and further diminishes the suction contribution term $\chi\psi$. When P is low, confinement is insufficient to maintain rib interlocking. The interface response therefore shifts from interlocking-dominated resistance to sliding control, and τ_p drops abruptly. Higher P mitigates this effect by increasing contact confinement and

suppressing the formation of a through-going shear band. Increasing v shortens the shearing duration, making it more difficult for moisture redistribution and suction recovery to reach a quasi-steady state during shearing, which is consistent with the downward shift of the response surface. Since pore-water pressure was not measured, the pore-pressure interpretation is inference based on boundary conditions and timescale arguments, suggesting a partially drained to near-undrained response at higher v . Overall, rainfall-induced degradation can be summarized as follows: near-interface moisture enrichment reduces $\chi\psi$ and, together with lubrication, weakens interlocking. This promotes shear localization near the interface and triggers sliding [31–33]. In practice, degradation can be delayed by increasing normal confinement and improving drainage and moisture-migration pathways.

4. Conclusions

This study used a retrofitted large-scale direct shear apparatus with multi-depth water-content monitoring near the interface. We systematically investigated the shear response and strength degradation of the geogrid–soil interface under different P , v , R , and T . The degradation mechanism was interpreted in terms of unsaturated effective stress and the evolution of the local moisture field. Within the tested sand–geogrid system and the investigated conditions, the following conclusions are drawn:

1. Normal stress and reinforcement effects.
2. Under rainfall infiltration, the peak interface shear stress τ_p increases approximately linearly with P , and higher P significantly mitigates rainfall-induced degradation. Compared with unreinforced specimens, τ_p of reinforced specimens is ~50% higher overall, indicating a net reinforcement gain under rainfall; however, this gain is locally reduced under intense rainfall and long infiltration due to the formation of a moisture-enriched zone. In the absence of a geogrid, rainfall-induced weakening is still expected due to suction decay and lubrication, but the degradation pathway is generally governed by bulk-soil frictional response rather than interlocking-to-sliding transition along a predefined interface.
3. Infiltration duration, shear rate, and evolution of strength parameters.
4. Under $R = 120 \text{ mm}\cdot\text{h}^{-1}$, increasing T from 10 to 30 min reduces cohesion c by ~56% and friction angle φ by ~22.5%, indicating that c is more sensitive to T . $c(T)$ can be approximated by an exponential decay, whereas $\varphi(T)$ decreases approximately linearly, reflecting combined effects of suction decay and deterioration of particle contact conditions. At $v = 2.0 \text{ mm}\cdot\text{min}^{-1}$, pore-water pressure is more difficult to dissipate, and the resulting c and φ under the same P , R , and T are lower than those at $v = 1.2 \text{ mm}\cdot\text{min}^{-1}$, indicating a significant shear-rate effect on degradation.
5. Water-content evolution and local moisture enrichment.
6. Real-time water-content monitoring indicates a stable vertical gradient near the interface; ω at 10 mm remains ~3.4 percentage points higher than at 30 mm, corresponding to a matric suction reduction of $\Delta\psi \approx 2.05 \text{ kPa}$ and forming a moisture-enriched layer characterized by high saturation and low suction. This behavior is reproducible across different P , T , and v , indicating that a moisture-enriched zone readily forms adjacent to the shear plane and constitutes a critical region where degradation and sliding preferentially occur.
7. Multifactor coupling and mechanistic implications.
8. Significant nonlinear coupling exists among P , R , T , and v . Response-surface analysis indicates that τ_p decays most severely under low P , high R , long T , and higher v , whereas relatively high strength can be maintained under high P , moderate R , and an appropriate v . In terms of unsaturated effective stress and local moisture-field evolution, degradation can be summarized as follows: local moisture enrichment causes suction decay and reduces frictional resistance, promoting shear band localization near the

interface; consequently, reinforcement effectiveness becomes constrained by the formation and evolution of a local weak interface. When applying these results to field structures, the influence of three-dimensional seepage, lateral drainage, and preferential flow paths should be considered, and site-specific seepage conditions should be used to relate in situ wetting to the laboratory boundary configuration.

Limitations of the present study. The quantitative results are obtained for a specific sand gradation and one geogrid type, and thus should be generalized to other soil–reinforcement combinations with caution. Laboratory hydraulic boundaries and discrete moisture measurements define the applicability range; suction is interpreted indirectly (e.g., via SWCC) rather than measured directly.

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Abbreviations

The following abbreviations are used in this manuscript:

1D	One-dimensional
DEM	Discrete Element Method
MC	Mohr–Coulomb criterion
MSE	Mechanically Stabilized Earth
SWCC	Soil–Water Characteristic Curve
VG	Van Genuchten model

Appendix A

Table A1. Nomenclature.

Symbol	Definition	Unit
C_c	Coefficient of curvature	
C_u	Coefficient of uniformity	
c	Interface cohesion from Mohr–Coulomb regression	kPa
c_0	Fitted initial parameter in $(c(T))$	kPa
D_{10}	Characteristic particle size	mm
D_{30}	Characteristic particle size	mm
D_{60}	Characteristic particle size	mm
D_r	Relative density	%
e_{max}	Maximum void ratio	
e_{min}	Minimum void ratio	
m	Van Genuchten fitting parameter	
n	Van Genuchten fitting parameter	
P	Normal stress	kPa
R	Rainfall intensity	mm·h ^{−1}
R^2	Coefficient of determination	

S_r	Degree of saturation	
T	Infiltration duration	min
v	Shear rate	$\text{mm}\cdot\text{min}^{-1}$
α (Equation (2))	Van Genuchten parameter in SWCC fitting	kPa^{-1}
α (Equation (5))	Decay coefficient in $(c(T))$ fitting	min^{-1}
β	Slope coefficient in $(\varphi(T))$ fitting	$^{\circ}\cdot\text{min}^{-1}$
δ_e	moisture-enriched (weakened) zone	mm
χ	Bishop's effective-stress parameter (function of saturation)	
$\Delta\psi$	Matric suction difference	kPa
$\Delta\omega$	Water-content difference (between depths)	%
μ_a	Pore-air pressure	kPa
μ_w	Pore-water pressure	kPa
ρ_d	Dry density	$\text{kg}\cdot\text{m}^{-3}$
ρ_w	Density of water	$\text{kg}\cdot\text{m}^{-3}$
σ	Total normal stress	kPa
$(\sigma - \mu_a)$	Net normal stress	kPa
σ_s	Suction stress	kPa
τ	Shear stress	kPa
τ_p	Peak shear stress	kPa
θ	Volumetric water content	(m^3/m^3)
φ	Interface friction angle from Mohr–Coulomb regression	$^{\circ}$
φ_0	Fitted initial parameter in $(\varphi(T))$	$^{\circ}$
ψ	Matric suction $(\mu_a - \mu_w)$	kPa
ω	Gravimetric water content	%
ω_r	Residual gravimetric water content (VG parameter)	%
ω_s	Saturated gravimetric water content (VG parameter)	%

Appendix B

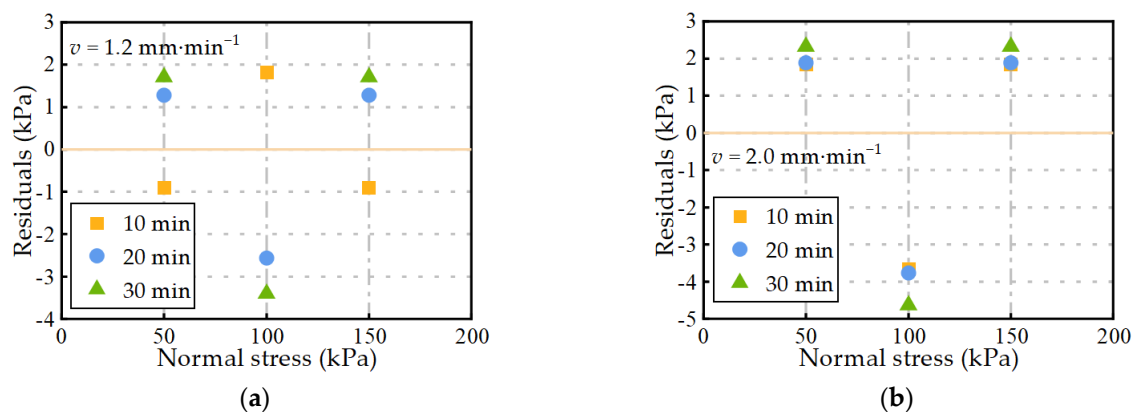


Figure A1. Residual plots for linear Mohr–Coulomb fits of peak interface shear strength under rain-fall infiltration ($R = 120 \text{ mm}\cdot\text{h}^{-1}$) at different infiltration durations ($T = 10, 20$, and 30 min). (a) $v = 1.2 \text{ mm}\cdot\text{min}^{-1}$; (b) $v = 2.0 \text{ mm}\cdot\text{min}^{-1}$.

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