

THE STABILITY OF SOIL-ROCK MIXTURE SLOPES UNDER COMBINED EFFECTS OF RAINFALL AND TUNNEL EXCAVATION

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Abstract: Because of their heterogeneous composition, soil-rock mixture (SRM) slopes respond strongly to both rainfall infiltration and tunnel excavation. In the model test, heavy rainfall was applied while two tunnels were excavated, and water content, deformation, and earth pressure were monitored continuously. Settlement accumulated rapidly at the beginning of the test as infiltration increased the soil mass weight and reduced shear strength. The subsequent increase became slower with further excavation. Lateral displacement, by contrast, showed a distinct sudden change before entering a period of gradual development. Earth pressure passed through three stages: relative stability, rapid change, and subsequent adjustment. The test results show that rainfall-induced weakening and excavation-related stress redistribution acted together to control the slope response, providing experimental support for stability assessment under similar conditions.

Keywords: Soil-rock mixture; Rainfall; Tunnel excavation; Coupling effect; Slope stability

1 INTRODUCTION

In highway and railway construction, tunnels are commonly adopted to maintain route alignment and reduce engineering costs, particularly in mountainous terrain. In many projects, the tunnel-to-bridge ratio exceeds 50% and may reach more than 70% [1]. Such regions also contain extensive natural slopes composed of soil-rock mixtures (SRMs). An SRM generally consists of relatively strong rock blocks, a fine-grained soil matrix, and voids, with the proportion of rock fragments varying among deposits [2]. Because the soil and rock components differ considerably in particle size, strength, and stiffness, the mechanical response of an SRM cannot be regarded simply as that of homogeneous soil or intact rock [3,4]. The soil matrix is usually weaker and more deformable than the rock blocks, while the soil-rock interfaces may become preferential zones for shear deformation under external loading. This heterogeneous structure also makes SRM slopes sensitive to changes in their hydrological environment. Changes in rainfall and groundwater conditions can affect both stress redistribution and deformation within the slope, with the response varying spatially and over time [5,6].

For an SRM slope, this behavior is closely related to the properties of the soil and rock materials, how they are distributed, and the surrounding environmental conditions [7,8]. Rainfall infiltration increases pore water pressure [9], as a result, effective stress decreases and the soil matrix provides less shear resistance [10,11]. Tunnel excavation, in contrast, disturbs the original in-situ stress field and redistributes stress around the opening, thereby changing the mechanical equilibrium of the slope [12,13].

Rainfall-induced moisture migration and excavation-related stress changes are therefore considered two important causes of slope instability [14,15]. Zhang et al. combined physical model tests with numerical simulations and reported that excavation-induced slope deformation may extend to about 1.5 times the tunnel span [16]. Hou et al. used a fluid-solid coupling model to examine twin-heading excavation under different infiltration durations, focusing on surrounding-rock stress, primary-support stress, and deformation [17]. Wang et al. developed an experimental system to study rainfall-related failure of tunnel surrounding rock under different rainfall intensities and durations [18]. Luo et al. focused on transient seepage in unsaturated landslides and discussed how it contributes to rainfall-induced slope failure [19]. Using physical model tests, Wei et al. monitored earth pressure and internal forces at different tunnel sections under rainfall conditions, and analyzed the coupled development and failure characteristics of tunnel-landslide systems [20].

Limit equilibrium analysis, finite element methods, and numerical simulation are commonly adopted for slope stability studies because they can capture both complex material behavior and progressive failure [21,22]. Ng and Pang examined the influence of stress state on soil-water characteristics and slope stability [23]. Nordas et al. derived an analytical solution for undrained ground response curves that accounts for excavation-induced desaturation [24].

Numerical work by Tsaparas et al. showed that permeability and saturation coefficient had a greater effect on the stability of unsaturated slopes than water content or rainfall pattern [25]. Aswathy assessed tunneling-induced surface settlement using a linear regression method [26]. Air pressure was included in the seepage analysis of Montrasio et al., where finite element and limit equilibrium methods were combined to study the corresponding changes in seepage and slope stability

[27]. Das et al., meanwhile, used logistic regression together with rock-mass classification to evaluate landslide susceptibility and slope stability [28].

Despite these advances, most previous studies have examined rainfall and tunnel excavation separately, while their coupled hydro-mechanical effects on SRM slopes remain less well understood [29]. To address this issue, the present study uses a physical model of an SRM slope subjected to heavy rainfall and twin-tunnel excavation. Moisture content, displacement, and earth pressure are monitored during excavation to characterize the coupled response of the tunnel-slope system.

2 ENGINEERING BACKGROUND

2.1 Geological Conditions

The tunnel site is located in northeastern Sichuan Province and is characterized by hilly terrain. The mountain ridges generally extend in a SW–NE direction, with higher elevations in the northeast and lower elevations toward the southwest. Ridges and valleys alternate across the site, and the terrain is marked by gentle natural slopes of about 10–25°, narrow winding gullies, and pronounced local relief (Figure 1).

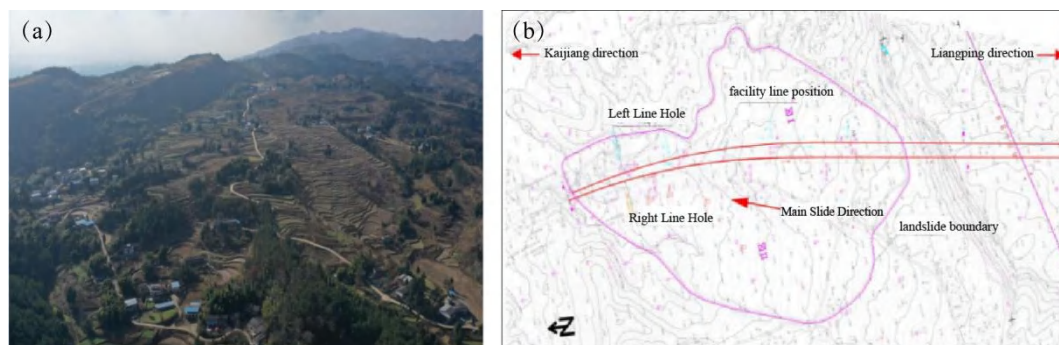


Figure 1 Landform Map. (a) Tunnel Location Map (Aerial photograph taken by the authors); (b) Map of Mixed Soil and Rock Slopes (Drawn by the authors using Adobe Illustrator based on the aerial photograph)

The tunnel entrance is situated on a sloping terrace with a natural gradient of 10–25°. Parts of this area have been reclaimed as paddy fields, while the remaining surface is barren. At the tunnel exit, the natural slope generally ranges from 20° to 45°, although some local terraces are considerably steeper. These terraces are about 6–10 m high, with slope angles of approximately 55–75°. Overall, the site is bounded by higher ground to the north and south and descends toward the east and west. The highest elevation is 730.1 m along the ridge crossed by the tunnel, whereas the lowest point is 459.1 m near the foot of the exit slope, giving a relative elevation difference of more than 279.0 m.

2.2 Strong Rainfall Excavation

The project area lies within the mid-subtropical humid climate zone of the Sichuan Basin. The climate is generally mild, with distinct seasons, little frost or snow in winter, rapid warming in spring, abundant summer rainfall, and gradual cooling in autumn. Annual precipitation exceeds 1,000 mm and is concentrated mainly between May and September, when heavy rainfall frequently occurs.

Owing to construction schedule requirements, tunneling through the soil-rock mixture (SRM) slope may need to continue during periods of intense rainfall. The local SRM is strongly affected by leaching, while its relatively high clay content gives the soil a strong capacity to retain water but limits aeration. Together with fractured rock, this condition restricts drainage and favors local water accumulation. During prolonged or intense rainfall, infiltration can therefore aggravate slope instability and increase the possibility of landslides, collapses, and debris flows. The heterogeneous composition and relatively strong cohesiveness of the SRM further complicate tunnel construction under these conditions [29].

3 SIMILARITY CRITERIA AND MATERIAL SELECTION

3.1 Methods for Determining Similarity Ratios

The similarity constant (C) is defined as the ratio of a physical quantity in the prototype to its corresponding value in the model. Key physical quantities considered in the experimental system, governed by dimensional analysis principles, are summarized in Table 1.

$$C = \frac{P}{M} \quad (1)$$

where: C —similarity ratio; P —physical quantity in prototype; M —corresponding physical quantity in model.

Table 1 Expression of Test Physico-Mechanical Parameter Scale

physical quantity	Symbol	dimension
Geometry	l	L
Density	ρ	ML^{-3}
Gravity acceleration	g	LT^{-2}
Displacement	d	L
Cohesion	c	$ML^{-1}T^{-2}$
Rainfall intensity	q	LT^{-1}
Angle of internal friction	φ	1
Poisson's ratio	μ	1
Modulus of elasticity	E	$ML^{-1}T^{-2}$
Stress	σ	$ML^{-1}T^{-2}$
Strain	ε	1
Water content	ω	1

This experiment established similarity relationships based on three fundamental scaling ratios: geometric (C_l), gravitational (C_g), and density (C_ρ). Through dimensional analysis, the similarity constants for key physical and mechanical parameters were derived as follows:

$$C_E = C_c = C_\sigma = C_d = 100 \quad (2)$$

$$C_q = C_l^{1/2} C_g^{1/2} = \sqrt{100} = 10 \quad (3)$$

3.2 Slip Similar Material Selection

The experimental soil preparation employed the equivalent mass replacement method, maintaining constant content of soil particles (<5 mm) while proportionally replacing particles >20 mm with 5-20 mm particles of equal mass. Figure 2 presents the comparative grain size distribution between engineering soils and experimental mixtures, quantifying the percentage content of different particle size fractions.

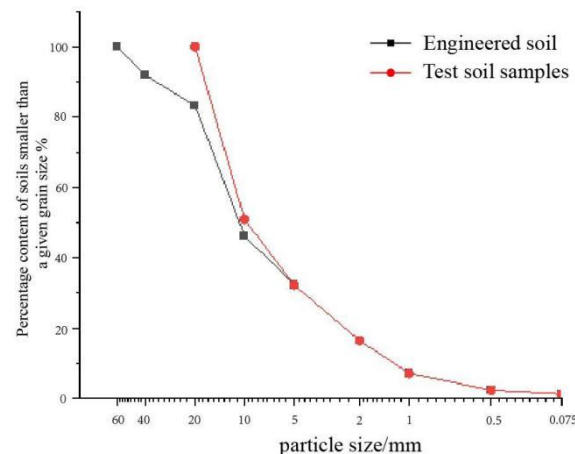


Figure 2 Comparison of Percentage Content of Different Grain Sizes in Engineering and Experimental Soil Samples

This model experiment simulates coupled rainfall infiltration and tunnel excavation conditions, requiring the similarity materials to satisfy scaling relationships for both natural and saturated states relative to in-situ soils. Based on field investigation data and literature, the mechanical parameters of the prototype slope are summarized in Table 2 [30].

Table 2 Physical and Mechanical Parameters of Soils at the Site

Parameter status	Cohesion/kPa	Angle of internal friction/°
Natural state	28.56	28.05
Saturated state	24.66	25.56

The model material consisted of silty clay, yellow sand, and crushed rock, with the proportions adjusted to match the target particle-size distribution. Candidate mixtures were evaluated by direct shear testing. The crushed-rock fraction was determined according to the scaled gradation, while the initial water content was set at 9% for the natural state and 13% for the saturated state. Three clay-to-sand ratios, 1:2, 1:1, and 2:1, were considered based on commonly used proportions for soil-rock mixtures.

Direct shear tests were performed in a 200 mm × 200 mm × 200 mm shear box. For each mixture, four specimens were prepared and tested under normal stresses of 50, 100, 150, and 200 kPa. The measured shear strength was then plotted against normal stress (Figure 3), cohesion and internal friction angle were obtained from the intercept and slope of the fitted strength envelope, respectively, and the results are listed in Table 3.

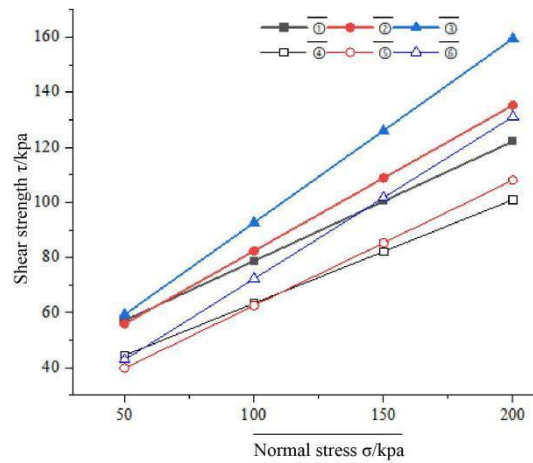


Figure 3 Normal Stress vs. Shear Strength Curve

Table 3 Straight Shear Test Data

Soil to sand ratio	Natural state mechanical parameters			Saturated state mechanical parameters		
	Group	Cohesion/kPa	Angle of internal friction/°	Group	Cohesion/kPa	Angle of internal friction/°
1:2	①	35.43	23.45	④	25.73	20.63
1:1	②	29.47	27.89	⑤	17.13	24.45
2:1	③	25.86	33.76	⑥	13.47	30.48

Combining the mechanical parameters of the above three proportioning tests and the mechanical parameters of the prototype soil samples in the field, the material with a soil-to-sand ratio of 1:1 was finally selected as the similar material, and the proportion of the similar material soil samples was 50% for 10-20 mm gravel, 20% for 5-10 mm gravel, 15% for chalky clay, and 15% for sand.

3.3 Selection of Similar Materials for the Lining

The prototype of this test tunnel employs a C30 reinforced concrete lining structure, while the lining model utilizes PLA material. When the printing filling rate of the material is 75%, its physical and mechanical parameters generally meet the similarity relationship requirements outlined in the Code for Design of Highway Tunnels. The tunnel lining model is shown in Figure 4.

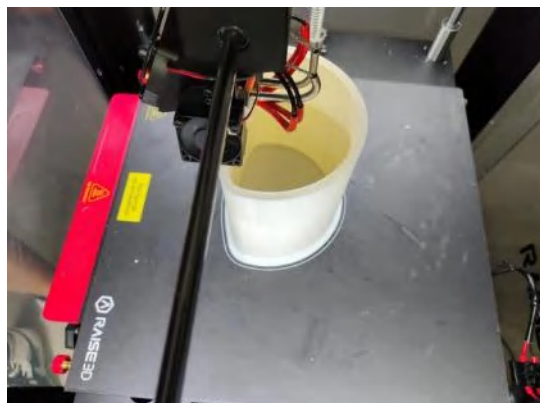


Figure 4 Tunnel Lining Model

4 TEST SYSTEM AND EXPERIMENTAL STEPS

4.1 Test System Components

The physical model testing system consisted of three primary components: a model test chamber, a simulated rainfall system, and a data monitoring system.

Model Test Chamber. The model test chamber (Figure 5) has overall dimensions of 2.7 m (L) × 1.8 m (W) × 1.2 m (H), with an internal usable space of 2.6 m × 1.7 m × 1.2 m. The chamber was constructed from high-strength steel sections assembled with bolted connections.

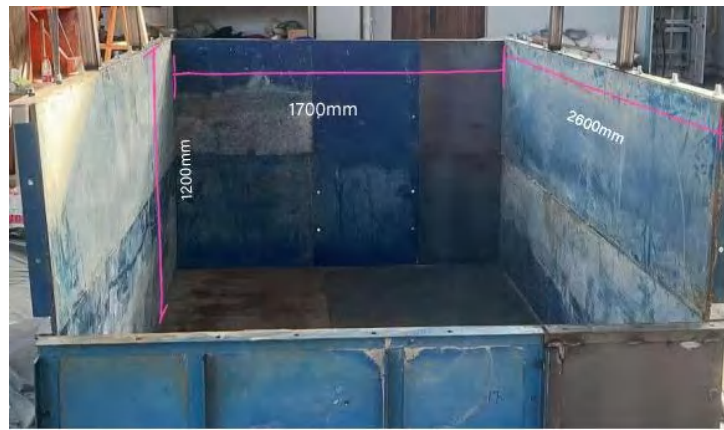


Figure 5 Model Test Chamber

Data Monitoring System. The monitoring system comprised a vibrating wire data acquisition (VDA) device (Figure 6), earth pressure cells, moisture content sensors, and a data logger.



Figure 6 VDA Equipment

Rainfall Simulation System. The rainfall simulation system consisted of a rain gauge, flow meter, flow control valve, rainfall sprinklers, and water storage tank. Prior to testing, the system was calibrated for rainfall uniformity. Calibration was performed using rain gauges to measure the intensity and spatial distribution of simulated rainfall, with the target intensity set to 30 mm/h. Nine measurement points were arranged within the model chamber to determine rainfall intensity. The rainfall uniformity was calculated as follows:

$$U = 1 - \frac{\sum |R_i - \bar{R}|}{n\bar{R}} \quad (4)$$

where: R_i —The rainfall depth measured at monitoring point i within the simulated rainfall area during the specified time period. n —Number of measurement points. $\bar{R}$ —Average rainfall for the same time period within the rainfall range. The rainfall intensity was maintained at 30 mm/h with uniformity exceeding 80% by adjusting both the positions of the rainfall sprinklers and the opening of the flow control valves.

4.2 Sensor Arrangement

The sensors used in the experiment and their corresponding displacement monitoring diagrams are shown in Figure 7 and Figure 8:

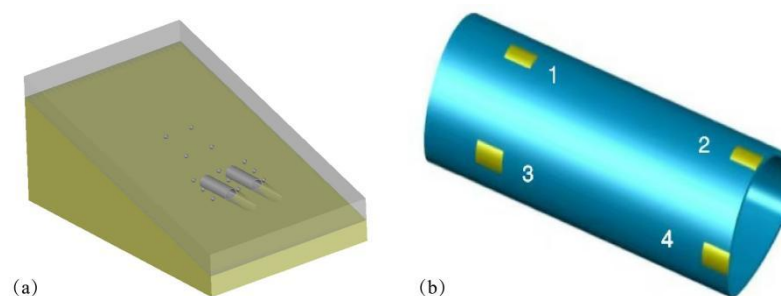


Figure 7 Sensor Layout Diagram. (a) Layout of Earth Pressure Sensors; (b) Strain Gauge Layout

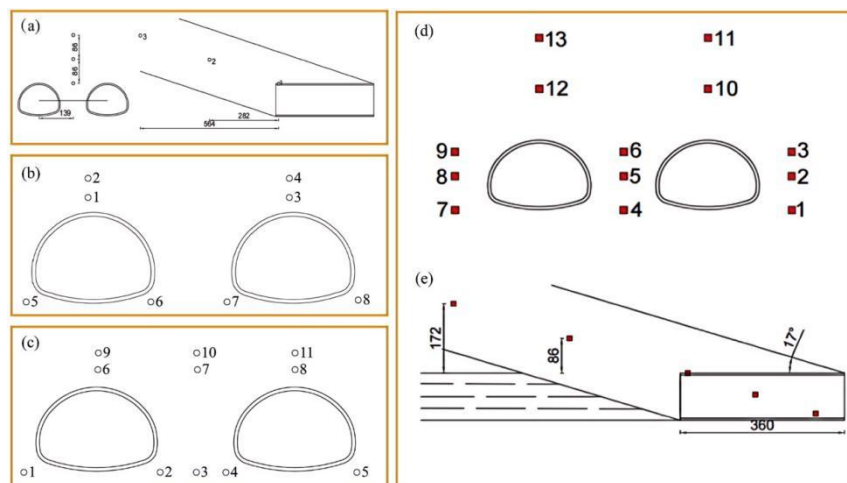


Figure 8 Various Displacement Monitoring Diagrams. (a) Layout of Moisture Content Sensor; (b) Schematic Diagram of Locations for Monitoring Settlement and Displacement of the Vaults; (c) Schematic Diagram of Lateral Displacement and Slope Slip Monitoring Points; (d) Main View of Soil Pressure Sensor; (e) Left View of Soil Pressure Sensor

4.3 Test Procedure and Excavation Program

4.3.1 Test procedure

Soil sample preparation: The soil was first crushed and sieved to achieve uniform particle size distribution. Distilled water was then added to reach the target moisture content, followed by thorough mixing. The prepared soil was cured for 24 hours to ensure homogeneous moisture distribution. The final moisture content was verified through laboratory testing prior to placement.

Model construction: The soil was compacted in 10 cm lifts to achieve uniform density. The slope angle was set at 17° . A 5 cm thick sliding zone was created between the bedrock and sliding mass using a homogeneous mixture of talcum powder and water (2:1 ratio by mass). The mixture was uniformly placed in the model chamber. All sensors were installed at predetermined positions during the filling process.

Data Acquisition System: After completing sensor installation, all sensors were connected to the data logger, which was then interfaced with a computer. The entire system was inspected to verify proper wiring connectivity and ensure operational reliability.

Testing Procedure: Under simulated rainfall conditions (30 mm/h intensity for 170 minutes), tunnel excavation was conducted in 5 cm increments. Each excavation step was followed by a 10-minute stabilization period. The left and right tunnels were excavated alternately, with a total of 8 cycles (40 cm cumulative advance per tunnel).

4.3.2 Tunnel excavation program

Excavation Procedure: The tunnel was excavated in 16 stages, alternating between the left and right tunnels. After an initial two-stage excavation of the left tunnel, the subsequent stages were performed alternately between the two tunnels. Each stage involved a 5 cm advance, with a 10-minute interval between consecutive excavation stages. The detailed excavation sequence is presented in Table 4.

Table 4 Tunnel Excavation Options

Excavation times	Excavation time/min	Cumulative excavation length/cm
1	20	Left hole 5cm
2	30	Left hole 10cm
3	40	Right hole 5cm
4	50	Left hole 15cm
5	60	Right hole 10cm
6	70	Left hole 20cm
7	80	Right hole 15cm
8	90	Left hole 25cm
9	100	Right hole 20cm
10	110	Left hole 30cm
11	120	Right hole 25cm
12	130	Left hole 35cm
13	140	Right hole 30cm
14	150	Left hole 40cm
15	160	Right hole 35cm
16	170	Right hole 40cm

4.4 Analysis of the Phenomenon

During tunnel excavation, the tunnel face after the initial excavation of the left hole is shown in Figure 9(a). At this stage, low rainfall allows rainwater to primarily infiltrate the shallow layer of the slope, exerting limited influence on the deeper structure. As a result, both the tunnel wall and face remain dry and smooth, contributing to overall stability.

As the excavation process progresses and rainfall continues, the tunnel face and walls gradually become wet, leading to local sliding phenomena (Figure 9(b-d)). Rainfall significantly increases soil moisture content and notably decreases soil strength.

During subsequent excavation stages, prolonged rainfall and water infiltration triggered frequent peeling and sand shedding at the tunnel face (Figure 9(e-f)). Concurrently, muddy water seepage emerged from the face, collectively indicating progressive deterioration of the tunnel's self-stabilization capacity as the experiment progressed.

During the final excavation stages penetrating the rear sliding mass (Figure 9(g-h)), tunnel wall peeling and sand shedding intensified significantly, accompanied by increased volume and velocity of muddy water seepage. These phenomena demonstrated substantial degradation of the excavated section's stability. Notably, despite these destabilizing effects, the lining-supported segments maintained structural integrity without collapse, confirming the reinforcement system's effectiveness under extreme hydro-mechanical loading.

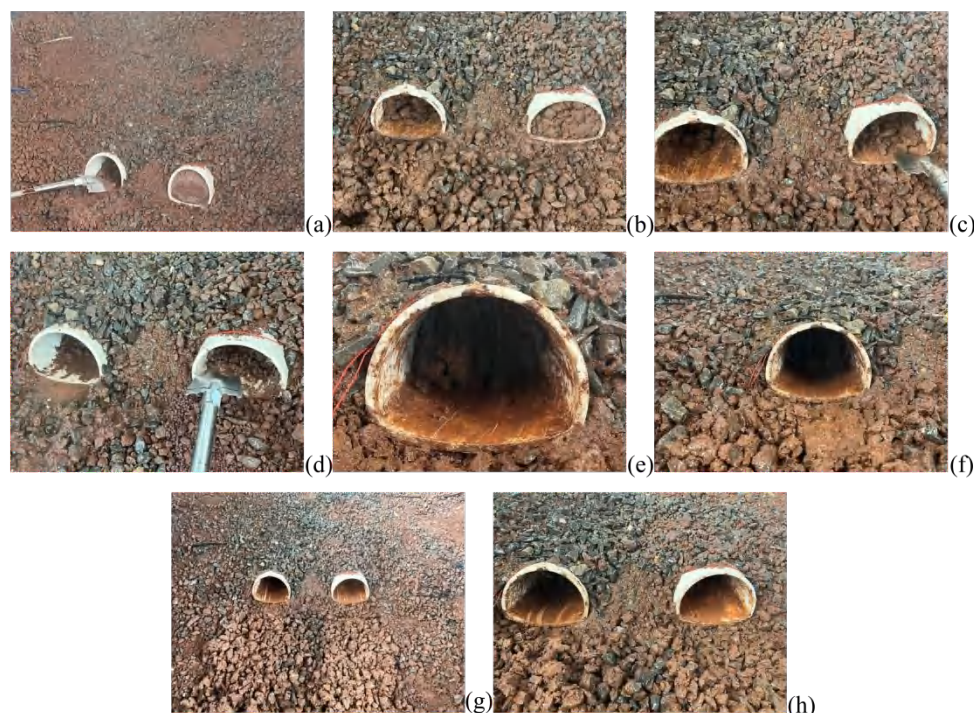


Figure 9 Process of Excavating. (a) First Excavation of the Left Hole, (b) Second Excavation of the Left Hole, (c) First Excavation of the Right Hole, (d) Second Excavation of the Right Hole, (e) Fifth Excavation of the Left Hole, (f) Fifth Excavation of the Right Hole, (g) Eighth Excavation of the Left Hole, (h) Eighth Excavation of the Right Hole

5 SLOPE WATER CONTENT

The moisture content sensors were installed as shown in Figure 8(a). As evident from Figure 10, the slope's moisture content increased rapidly during the initial 30 minutes of testing, rising from 20% (pretest baseline) to 43%. Notably, Monitoring Point 1 recorded a peak moisture content of 50%, demonstrating significant spatial variability in hydration response.

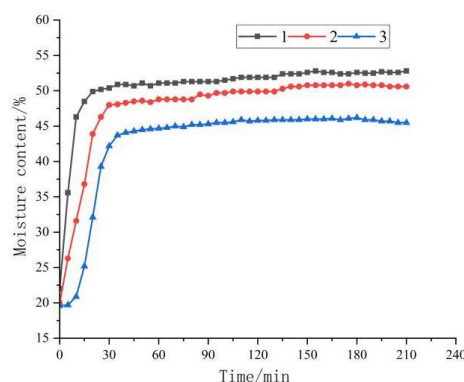


Figure 10 Plot of Moisture Content with Time

The three sensors exhibited distinct moisture response patterns due to their different topographic positions. Sensor 1, located at the slope toe, recorded the highest peak moisture content (50%) as it continuously received infiltrated water from up-slope areas. Sensor 2, positioned at the mid-slope, showed an intermediate peak of 46%, reflecting both downward percolation and lateral flow redistribution. Sensor 3, situated at the upper slope, demonstrated the lowest peak (43%) due to rapid drainage through both vertical infiltration and surface runoff.

The temporal progression of peak moisture content revealed a distinct top-to-bottom saturation pattern: Sensor 1 (slope toe) reached peak saturation first, followed sequentially by Sensor 2 (mid-slope) and Sensor 3 (upper slope). This demonstrates that hydro-logical response times in the slope decreased progressively with elevation, with the lowest elevation zone achieving full saturation 27% faster than the uppermost zone.

In summary, the change in water content of the slope body under continuous heavy rainfall conditions is closely related to depth. The deeper the slope body, the higher the soil water content and the earlier the peak is reached; conversely, the shallower the slope body, the lower the soil water content and the later the peak is reached.

6 DEFORMATION ANALYSIS OF SLOPE SURFACE

6.1 Slope Displacement Pattern Analysis

Settlement, transverse displacement and outboard slip of the slope were monitored via VDA equipment. Settlement monitoring points are shown in Figure 8(b), while the locations of the transverse displacement and outboard slip monitoring points are illustrated in Figure 8(c).

To ensure valid monitoring, VDA angles were adjusted to match the slope, making its observation surface parallel to the slope for measuring outward surface deformation. The cloud diagram of outward slope displacement is given in Figure 11.

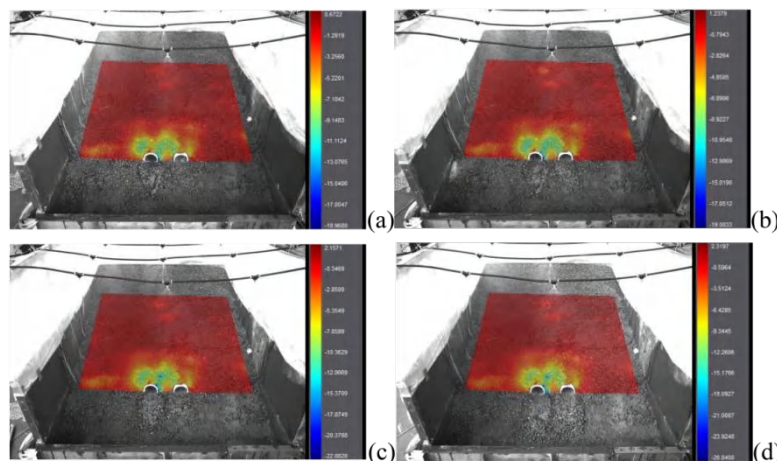


Figure 11 Cloud Map of Outward Displacement of Slope Surface. (a) Fourth Excavation, (b) Eighth Excavation, (c) Twelfth Excavation, (d) Sixteenth Excavation

6.2 The Settlement and Displacement Law of the Vault

As can be seen from the analysis in Figure 12, the settlement displacements at each monitoring point increased significantly during the early stages of excavation and exhibited an alternating upward trend. This was primarily due to reduced cohesion of the slope caused by rainwater infiltration, as well as disturbances to the slope during tunnel excavation, resulting in substantial settlement displacements at all monitoring points.

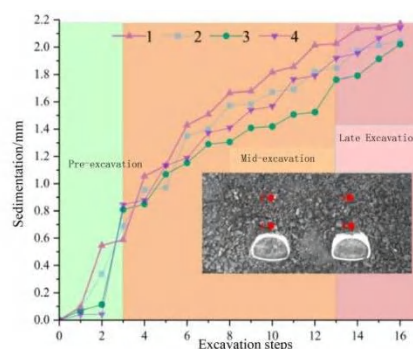


Figure 12 Variation Rule of Vault Settlement Displacement with the Number of Excavations

During mid-excavation, the settlement displacements at each monitoring point continued to rise, but the rate of increase was significantly slower than in the early stages of excavation. This slowdown can be attributed to the self-stabilizing ability of the soil-rock mixed slope. Even as the soil was eroded by rainfall and began to sink, the interlocking of crushed rocks effectively suppressed settlement and prevented large displacements. Additionally, the left tunnel experienced more disturbance than the right tunnel during excavation. As a result, the settlement displacement at monitoring point No. 1, located at the top of the left tunnel arch, was greater than that at monitoring point No. 4, at the top of the right tunnel arch, even though both are at the same horizontal height. Similarly, the settlement displacement at monitoring point No. 2 on the left tunnel arch was larger than that at monitoring point No. 3 on the right tunnel arch. Furthermore, the settlement displacements at monitoring points No. 1 and No. 4 were greater than those at monitoring points No. 2 and No. 3, due to the closer proximity of No. 1 and No. 4 to the tunnel body, which subjected them to greater excavation disturbances. At the end of the excavation, the growth trend of the monitoring points was relatively flat. This observation is attributed to the water content within the slope body being close to saturation, which gradually weakens its mechanical properties due to continuous rainfall, leading to slope stabilization.

As a result, the settlement displacements at each monitoring point exhibited slow growth. Upon completing the 16th excavation, the settlement displacements at each monitoring point reached their maximum values: monitoring point No. 1 recorded a maximum settlement of 2.17 mm, monitoring point No. 2 recorded 2.04 mm, monitoring point No. 3 recorded 2.02 mm, and monitoring point No. 4 recorded 2.14 mm.

6.3 The Settlement Displacement Law of the Arch Foot

As shown in Figure 13 prior to the second tunnel excavation, the infiltration of rainwater reduced the slope's cohesive strength. Combined with the disturbance caused by tunnel excavation, significant settlement displacement occurred at the arch feet of both the left and right tunnels.

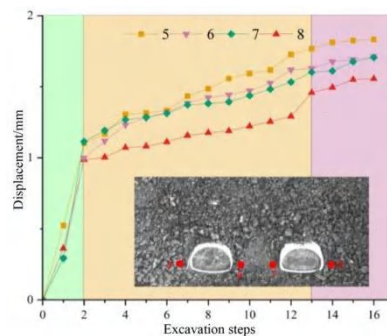


Figure 13 The Pattern of Change of Arch Foot Settlement Displacement with the Number of Excavations

During the 2nd to 13th excavation stages, the growth rate of settlement displacement at the arch feet significantly decreased. After experiencing substantial initial deformation, the soil-rock mixture fully mobilized its self-stabilizing capacity, thereby mitigating further slope settlement.

After the 13th excavation stage, the slopes internal moisture content reached saturation. Additionally, as the tunnel excavation face was sufficiently distant from the monitoring points, further excavation had negligible influence on displacement deformation. Consequently, only minor increases in settlement were observed at the monitoring points.

Analysis of the crown and arch foot settlement displacements in Figure 12 and Figure 13 reveals that the crown settlement exceeds that of the arch foot. Prior to the second excavation stage, monitoring points at the arch foot exhibited significantly greater displacement than those at the crown, indicating that the arch foot is more susceptible to the combined effects of rainwater infiltration and excavation disturbance. Following the second excavation, crown displacement entered a phase of decelerated growth, whereas the arch foot displacement growth rate began decreasing after the first excavation. This demonstrates that the arch foot possesses superior self-stabilization capacity compared to the crown.

6.4 Lateral Displacement Change Rule Analysis

The sign convention for lateral slope displacement is defined as follows: rightward displacement is positive, while leftward displacement is negative.

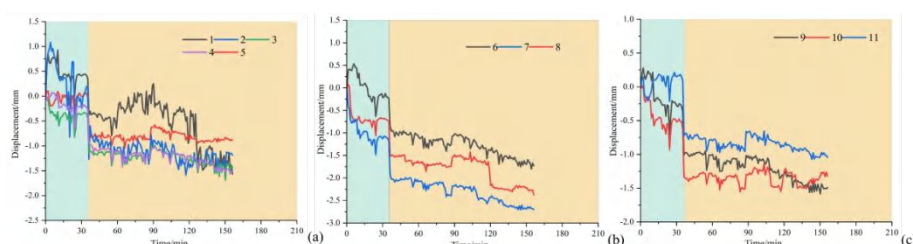


Figure 14 Change Rule of Transverse Displacement with Time

Analysis of Figure 14(a) reveals that the lateral displacement at the tunnel arch foot generally increased over time. During the first 35 minutes of testing, the left tunnel arch foot exhibited slightly greater lateral displacement than the right tunnel arch foot due to more significant excavation effects. Since Monitoring Point 3 was closer to the left tunnel, its displacement similarly exceeded that of the right arch foot.

After 35 minutes, all arch foot monitoring points experienced abrupt increases in lateral displacement. Following this sudden change, the displacement rate decreased markedly and eventually stabilized. Throughout the remainder of the test, lateral displacements at all monitoring points remained relatively constant, maintaining values between 1.0 and 1.5 mm. Analysis of Figure 14(b-c) reveals that all monitoring points immediately detected surface lateral displacement at the beginning of the test, exhibiting an overall increasing trend. During the first 35 min of the test, lateral movement was detected at all monitoring locations, although the increase remained relatively limited. A clear change occurred at about 35 min, when the displacement curves at nearly all points rose sharply. The increase became much slower thereafter, and the curves gradually approached a relatively stable trend. Among the monitoring points near the tunnel crown, Points 6–8 showed comparatively large lateral movements, with the displacement ranked as Point 7 > Point 8 > Point 6. The upper monitoring points, Points 9–11, were farther from the excavation face and showed smaller but still measurable responses. Their displacement followed the order Point 9 > Point 10 > Point 11.

This variation can be related to the combined development of rainfall infiltration and excavation disturbance. In the early stage, rainfall gradually increased the water content and self-weight of the surrounding soil-rock mass, so the lateral displacement developed slowly. At 30 min, another excavation step was carried out. At nearly the same time, the water content increased rapidly and the material approached a highly saturated state. The resulting increase in self-weight, together with the reduction in shear resistance, made the slope more sensitive to excavation disturbance. The pronounced displacement change observed around 35 min therefore reflects the superimposed effects of rainfall infiltration and tunneling.

With further excavation, the excavation face moved progressively away from the monitored slope surface, and its disturbance effect weakened. Meanwhile, the water content had already approached saturation, so the subsequent increase in the self-weight of the soil-rock mass became limited. Under these conditions, lateral displacement continued to accumulate, but the rate of increase was much lower than that observed during the earlier stage. No comparable abrupt change occurred during the later part of the test.

Overall, the monitoring results show a similar temporal response at most measurement points. Lateral deformation developed from an initially gradual increase to a short period of rapid change, followed by a much slower accumulation stage. This behavior suggests that excavation had a stronger influence on the slope during the early excavation process, whereas its contribution to surface lateral deformation became less evident as the tunnel advanced. The later displacement was therefore characterized mainly by progressive accumulation rather than repeated sudden increases.

6.5 The Change Rule of Surface Slippage

Figure 15 shows that the development of surface slip can be roughly separated into two periods. A rapid response was observed immediately after the test started.

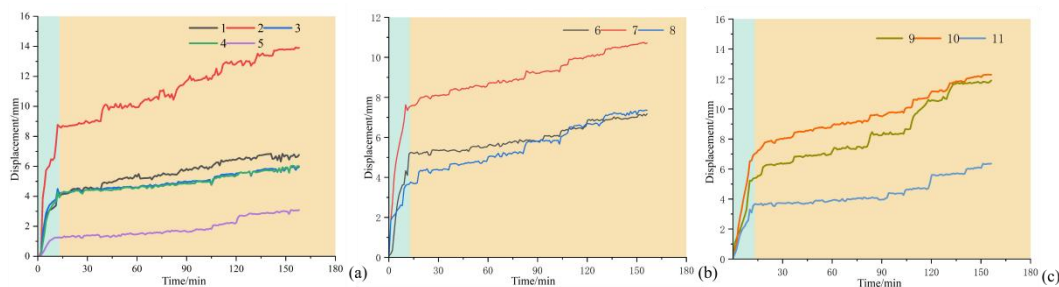


Figure 15 Plot of the Pattern of Change in Surface Slip over Time

During the first 13 min, slip increased quickly at all monitoring points. At this stage, rainfall infiltration added to the self-weight of the soil mass while weakening its shear resistance, which favored outward movement of the slope surface. After about 13 min, the measured slip continued to accumulate, but the increase became much slower.

The slower development in the following period suggests that the slope response was changing as rainfall continued. Although water infiltration did not stop, the soil-rock mass gradually approached a relatively stable condition, and the additional surface movement accumulated at a lower rate than during the initial stage.

The amount of slip also varied with monitoring position. Points 7 and 10, located in the central region between the two tunnels, generally showed larger movements than Points 6, 8, 9, and 11 on the two sides. This spatial difference indicates that the zone between the tunnels was more strongly affected by the combined action of rainfall and excavation. By the end of the test, however, the slip values measured at Points 9 and 10 were relatively close. The left tunnel was subjected to stronger excavation disturbance during the test, which enhanced the local slip response around these monitoring locations; the final value at Point 9 was slightly larger than that at Point 10.

7 SOIL PRESSURE CHANGE ANALYSIS

The earth pressure sensors were arranged according to Figure 8(d-e). As shown in Figure 16, the measured earth pressure did not vary uniformly throughout the test. Instead, its evolution can be described by three successive stages: an initial period with only small fluctuations, a period of pronounced change, and a final stage in which the readings gradually stabilized.

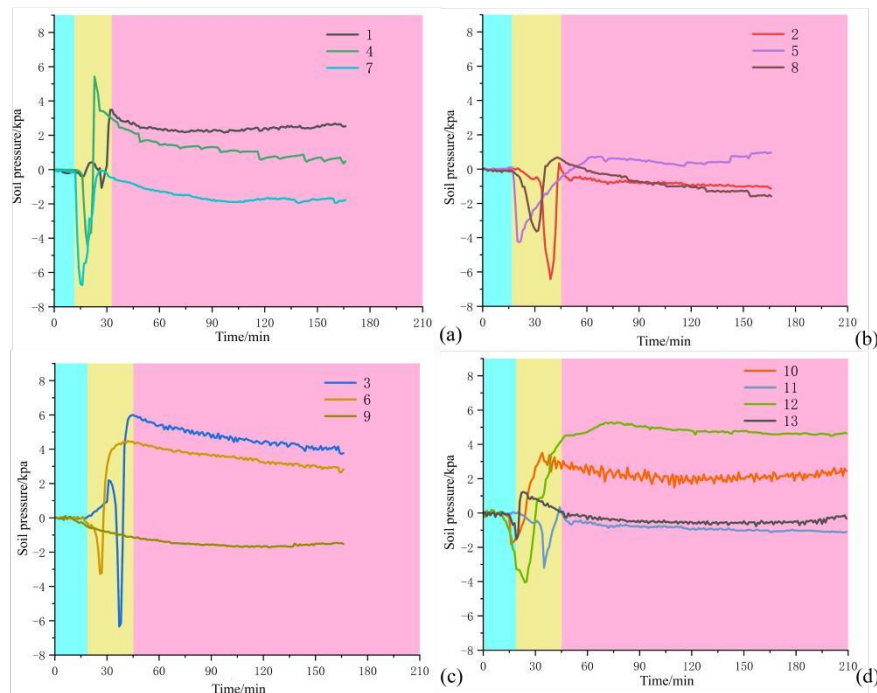


Figure 16 Soil Pressure Change Pattern with Time

At the beginning of rainfall, the earth pressure recorded by the sensors changed only slightly. The short infiltration duration at this stage meant that only a limited amount of water had entered the soil mass, and the degree of saturation remained low. Therefore, the increase in soil self-weight was still small, while the frictional resistance within the soil was not yet substantially affected. These conditions explain why the earth pressure remained nearly unchanged during the early part of the test.

With continued rainfall, water progressively infiltrated the slope and the soil approached saturation. The associated reduction in shear strength caused a much stronger response in the earth pressure measurements. At the same time, tunnel excavation disturbed the original stress field and produced stress redistribution in the surrounding soil-rock mass. The combined hydraulic and excavation effects led to the marked pressure variations observed during the intermediate stage. After further adjustment, the magnitude of the earth pressure fluctuations gradually decreased. Redistribution of stress around the tunnel and the accompanying adjustment of pore water pressure allowed the soil mass to approach a new equilibrium condition. Once this balance was established, the earth pressure curves became relatively stable. This final stage indicates that the internal stress state of the slope had largely completed its adjustment under the combined influence of rainfall infiltration and tunnel excavation.

8 CONCLUSIONS

Physical model tests were carried out to examine the response of an SRM slope subjected to rainfall infiltration and twin-tunnel excavation. The main conclusions are as follows.

Crown settlement was most sensitive during the early excavation stages. The first two excavations produced the main increase in crown settlement, while later excavation caused progressively smaller changes. A similar trend was observed at the arch foot, where the first excavation produced the most evident response. The arch foot showed stronger self-stabilizing behavior than the crown, and the final crown settlement was greater under the combined effects of rainfall and excavation.

Lateral deformation showed an initial increase, followed by an abrupt change and then slower accumulation. Excavation disturbance dominated the early response, whereas its influence weakened as excavation advanced. In the later stage, creep contributed more strongly to the continuing deformation.

Most of the rapid surface-slip development occurred within the first 13 min, indicating a strong early response to rainfall. The increase slowed markedly afterward. Monitoring points in the central region between the two tunnels generally recorded larger slip than those on the outer sides, reflecting stronger combined effects of rainfall infiltration and excavation disturbance.

Earth pressure developed through three successive periods: a relatively stable stage, a rapid-change stage, and an adjustment stage. During the initial period, limited rainfall infiltration caused only minor pressure changes. As infiltration

progressed, soil softening and excavation-induced stress redistribution produced pronounced fluctuations. These fluctuations gradually decreased as pore water pressure and internal stress were readjusted, and the soil-rock mass eventually approached a new equilibrium state.

COMPETING INTERESTS

The authors have no relevant financial or non-financial interests to disclose.

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AUTHOR CONTRIBUTION

HongYun Yang: Writing – review & editing, Writing – original draft, Investigation. LinFei Xiong: Writing – review & editing. YaKe Ma: Formal analysis. Yong Li: Conceptualization. Zhi Lin: Writing – original draft, Resources. Qing Liu: Validation. Xiang Chen: Software.

All authors critically reviewed and approved the final version of the manuscript and agreed to be accountable for all aspects of the work.

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