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Seismic Stability Evaluation of Benched Bedding Rock Slopes with Varying Landforms

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Abstract: Seismic stability of rock slopes is a critical issue in geotechnical engineering, particularly in seismically active regions where benched bedding slopes are susceptible to dynamic failure. This study explores the dynamic response and progressive damage mechanisms of benched bedding rock slopes under seismic excitation through shaking table tests and numerical simulations. Specifically, four slope models with varying bench widths are analyzed, focusing on key factors such as acceleration amplification coefficients (AAC), earth pressure distribution, and failure modes. The results reveal that AACs at the internal corners of benches are lower than those at the external corners. Moreover, it is observed that slope stability improves as the bench width increases, as evidenced by a reduction in failure occurrences for slopes with uniform bench widths of 2 m, 4 m, and 6 m. In contrast, the slope with uneven bench widths exhibits more severe failure. Furthermore, numerical simulations confirm that slopes with wider platforms at lower elevations are more prone to bench-scale failure. To quantify these effects, a novel dynamic stability equation, S_{qt} , validated by test data, classifies stability into four levels: $S_{qt} < 1.0$ (unstable), $1.0 < S_{qt} < 1.3$ (less stable), $1.3 < S_{qt} < 1.5$ (basically stable), and $S_{qt} > 1.5$ (stable). These findings provide a necessary basis for understanding the shattering cumulative damage mechanism and seismic fortification of complex rock slopes, and offer valuable insights for the design of cutting slopes in high-intensity seismic areas.

Keywords: benched bedding rock slope; dynamic response; seismic stability evaluation; shaking table tests; numerical simulations.

34 **Research Highlights:**

- 35 • Investigated dynamic responses of benched bedding rock slopes with varied landforms
- 36 • Revealed the impact of slope geomorphology on seismic damage and failure modes
- 37 • Developed and validated the dynamic stability equation S_{qt} for predicting slope stability
- 38 • Provided practical insights for designing safer rock slopes in seismic regions

1. Introduction

In recent years, major earthquakes in China have led to catastrophic geological disasters. Notably, the M_w 7.9 Wenchuan earthquake in Sichuan Province on May 12, 2008, triggered over 15,000 landslides and 619 debris flows, causing widespread devastation (Dong et al., 2022; Fan et al., 2016; He et al., 2021). Among these, the Daguangbao landslide remains the largest earthquake-induced landslide ever recorded globally, with an astounding volume of 742 million cubic meters. This landslide formed a dammed lake that soared to a height of 690 meters (Huang et al., 2012; Feng et al., 2022). The M_w 7.1 Yushu earthquake on July 1, 2010, resulted in over 2,036 landslides in Qinghai Province, claiming the lives of 2,220 people and leaving 70 unaccounted for. Similarly, the Lushan earthquake in Ya'an on April 20, 2013, resulted in a total of 3,000 geological disasters. More recently, on September 5, 2022, a magnitude of M_w 6.8 earthquake struck Luding County in Garze Prefecture, Sichuan Province, initiating 565 geological hazards in Luding and Shimian counties. These included 331 collapses and 234 landslides, and exacerbated the deformation of 81 existing geological hazards. In Western China, where earthquake intensity is generally high, the construction of large-scale roads often requires the formation of numerous cutting slopes (Xiao et al., 2010; Xin et al., 2024; Zhao et al., 2024). Additionally, in open-pit mining, the extraction of orebodies at different strata levels frequently results in the creation of benched landforms. A tragic example occurred on February 22, 2023, when a multi-bench landslide at Xinjing Coal Mine in Alxa Left Banner, Inner Mongolia, resulted in 63 deaths and the movement of nearly 5 million cubic meters of earth (Yi et al., 2023). These incidents underscore the critical need for enhanced stability assessments of bench slopes in seismically active regions.

Recent research has led to substantial advancements in both methodological approaches and mechanistic understanding of slope dynamic response and stability. With regard to dynamic response mechanisms, Jafarzadeh et al. (2015) systematically investigated the generation of Rayleigh waves in terrain amplification effects through a combination of shaking table tests and ABAQUS finite element simulations. Their findings quantitatively demonstrated the topographic amplification can significantly increase induced accelerations at the slope surface, particularly at the crest. Furthermore, they introduced the concept of the “effective depth of surficial amplification,” providing new insight into the influence of low-frequency seismic waves on deep slope responses. Song et al. (2019) employed a methodological framework integrating three-dimensional finite element modal analysis with shaking table testing to identify, for the first time, the typical vibration modes (shear, torsional, and flexural) and associated characteristic frequencies (12–29 Hz) of complex-structured rock slopes. Their results indicated that high-frequency components tend to cause localized surface damage, whereas low-frequency components are more likely to initiate progressive overall sliding failures. In terms of the geological structure's role in disaster control, Yang et al. (2025) conducted shaking table tests based on a

comparison of different weak interlayer designs (homogeneous weak layers and those with discontinuous interfaces). Using digital image correlation (DIC) full-field strain monitoring technology, they revealed the key influence of the development degree of weak interlayer structures on peak ground acceleration (PGA) distribution: continuous weak layers significantly increase PGA at the slope crest, aggravating back-edge tension cracking, while discontinuous interfaces suppress the amplification effect at the slope crest but lead to shear stress concentration at the slope toe. Another study on anti-dip bench rock slopes (Yang et al., 2025) used large-scale shaking table tests and numerical simulations to find that the bench geometry parameters (width and slope angle) regulate the failure mode by altering the stress transfer path. The widest bench effectively blocks internal deformation of the slope, and the displacement baseline offset ratio (DBOR) indicator was introduced for the first time to quantify the plastic zone expansion process. Regarding innovations in stability assessment methods, Maheshwari et al. (2023) established a small-scale physical model and PLAXIS HS small model coupling platform to accurately quantify the sensitivity of seismic settlement to slope angle and base edge distance. Pradhan et al. (2020) employed the Phase2D finite element platform, integrating the nonlinear Generalized Hoek-Brown (GHB) criterion and the Geological Strength Index (GSI) to establish a classification system for the stability of Himalayan jointed rock slopes. Their analysis confirmed that slopes with a factor of safety (FoS) ≤ 1 represent a critical state of instability and demonstrated the superior applicability of the GHB criterion over the Mohr-Coulomb model in cases of high joint density. Zirsang et al. (2024) further applied the Morgenstern-Price algorithm of the Slope/W limit equilibrium module, using 50 parameterized slope models. Their study verified that step-back setback configurations for buildings can enhance the seismic safety factor, thereby providing a quantitative foundation for disaster mitigation design in urban hilly terrains.

Despite these advances, three key limitations persist in the existing literature: 1) Object limitation: Current studies predominantly address homogeneous slopes or single-bench configurations, with limited investigation into the seismic response transmission mechanisms and overall system stability of benched rock slopes; 2) Methodological gap: Conventional static indicators (such as FoS) fail to capture dynamic cumulative damage. Although dynamic indicators like DBOR have been proposed, no unified assessment framework integrating geological structures, geometric morphology, and dynamic parameters has been established; 3) Engineering applicability: Existing models lack quantitative analysis of the synergistic effect of benched geometric parameters (overall slope angle α , bench angle β , and inter-ramp angle θ), thereby limiting their practical utility in dynamic control during slope construction.

This research examines the dynamic response and stability of slopes with varying topographies using a series of shaking table tests. It analyzes acceleration and earth pressure to evaluate the amplification effects and distribution of these factors at different platform positions of benched rock slopes. Moreover, the study also investigates the geometric characteristics of multi-bench slopes and establishes relationships between the overall slope angle (α), bench angle (β), and inter-ramp angle (θ).

The equation for calculating the slope stability, based on geological conditions (e.g., dip of structural surfaces), is proposed. The equation's applicability is verified through shaking table tests and numerical simulations. A dynamic correction coefficient (ζ) is introduced to refine slope stability analysis, and a new stability judgment equation incorporating this coefficient is developed. This study fills a gap in large-scale rock slope stability assessment, particularly regarding dynamic response analysis and slope angle adjustment. The proposed dynamic stability judgment equation offers a more accurate representation of the rock mass's mechanical behavior, providing geotechnical engineers with a precise and practical tool for real-time slope stability analysis and adjustments during construction, with significant engineering application potential.

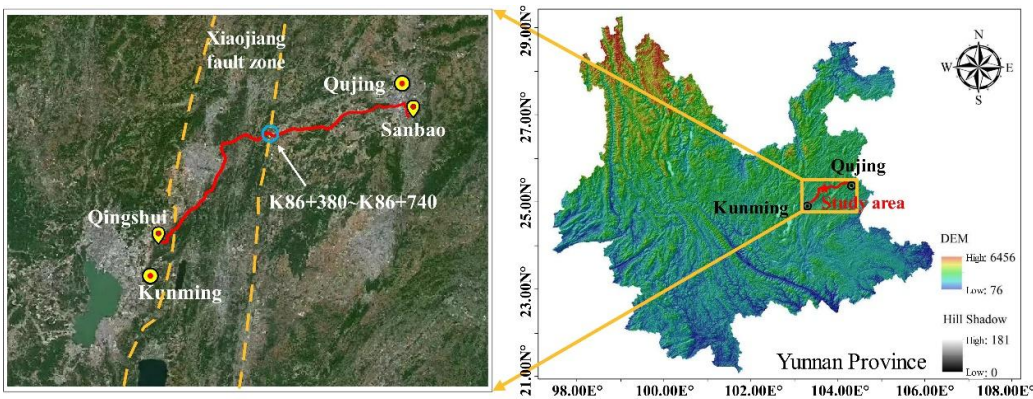
2. Geological Setting of the Study Slope

The study area is located in the Dongtai fold belt in Yunnan Province, China, known for its complex geological structures and well-developed faults. The project site is situated near the boundary between the Sichuan-Yunnan and South China fault blocks, traversing the active Xiaojiang fault zone. This zone, characterized by strong tectonic activity with features of inheritance and multi-stage reactivation, has a significant impact on the proposed road construction. The Xiaojiang fault, a northeast-trending Holocene active fault, features a left-lateral strike-slip movement and spans a structural width of 20 km. The exposed geological strata in the area include Quaternary (Q4) of the Cenozoic era, and Lower and Middle Cambrian Series (ϵ_1 and ϵ_2), respectively. In this series of shaking table tests, the cutting slope from Sanbao section to Qingshui section (K86+380 to K86+740) was selected as the prototype. Drill core tests were conducted at three locations along this segment (as shown in **Figure 1(b)**). The results from borehole 1 indicate the presence of strongly weathered siltstone, borehole 2 revealed highly weathered sandstone, and borehole 3 displayed moderately weathered sandstone.

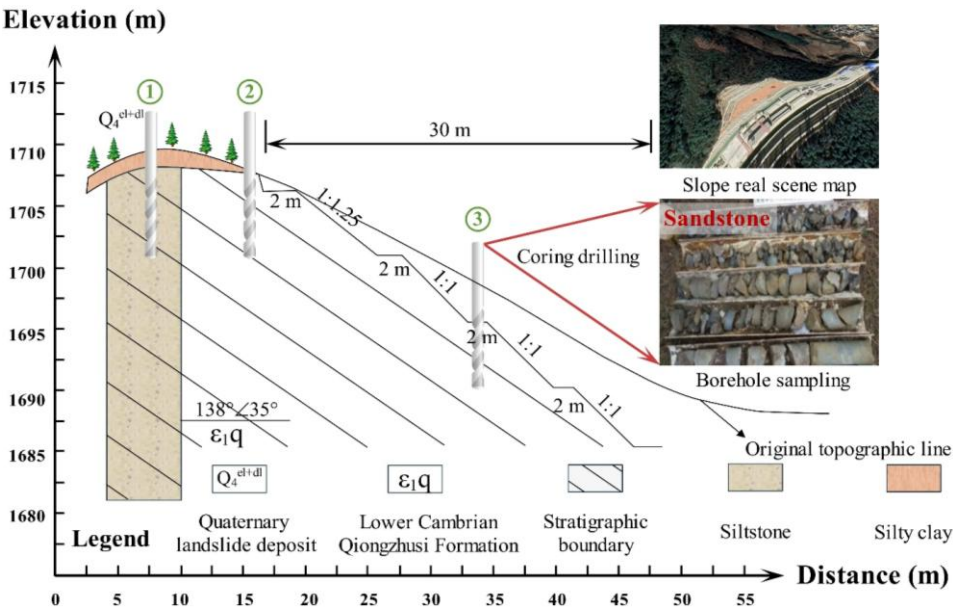
Under the influence of the geological structure of the study area, the highway cutting slope exhibits a layered structure with fully developed structural planes. The distribution and development degree of these structural planes, as well as their physical and mechanical properties, play a key role in determining the deformation, strength and stability of the slope. Therefore, accurately determining the mechanical parameters that reflect the actual engineering conditions is essential. Field investigations, sampling and in-situ testing were conducted on the cutting slope. Borehole shear tests were employed to enhance understanding of the in-situ geological condition and to retrieve rock mass samples, including rock blocks and structural planes. In order to establish the physical and mechanical parameters of the prototype, a comprehensive series of tests were conducted. These included in-situ shear tests, indoor direct shear tests, and uniaxial compression tests. The uniaxial compression and direct shear tests aimed to determine the physical and mechanical properties of rock masses and structural planes. Ring knife tests were conducted to obtain the density of the rock mass and structural plane, while direct shear tests

provided data on the cohesive force (c) and internal friction angle (ϕ). Additionally, uniaxial compression tests were performed to ascertain the elasticity modulus (E) and Poisson ratio (μ).

Site investigations and borehole sampling reveal that the engineering geological profile of the slope section primarily consists of siltstone and sandstone, characterized by argillaceous and layered structures. These rocks exhibit developed joint fissures and a loosely bonded structure, resulting in low mechanical strength. The geological formation of the slope is characterized by a monoclinical structure with strongly weathered sandstone bedrock, with a rock occurrence of $138^{\circ}\angle 35^{\circ}$. This orientation, combined with the slope direction, predisposes the slope to downslope movements and potential bedding plane sliding. The slope contained multiple structural planes with a dip angle of 35° , composed of clay with the thickness of the structural plane ranging from 0.1 to 0.2 m. Given the steep slope terrain and a natural inclination of approximately 30° , it is recommended to perform bench excavation of the slope. In the preliminary design phase, the maximum excavation height for this section is estimated at 29.5 m, to be accomplished in three benches. The excavation will follow a slope ratio of 1:1 for the first two benches and 1:1.25 for the third bench as shown in **Figure 1(b)**.



(a)



(b)

Figure 1 The location and geological profile of the study area. (a) location of the study area; (b) geological profile of the slope.

3. Shaking Table Model Test

3.1. Test Device

The shaking table test system employed in this research was provided by the Seismic Simulation Shaking Table Laboratory of the State Key Laboratory of Geohazard Prevention and Geoenvironment Protection, Chengdu University of Technology (see **Figure 2**). The shaking table equipment, produced by SERVOTEST company, UK, includes a shaking table measuring 6 m×4 m (Length × Width). This table is operated by four vertical actuators and four horizontal actuators, complemented by a 1500L@28Mpa oil source, an oil source control system, and a pulsar vibration input control system. The maximum load capacity of the shaking table is 4.0×10^4 kg. The actuators allow the table to achieve a full load horizontal and vertical acceleration of 1.5 g and 1.0 g, respectively, with corresponding maximum strokes of 300 mm and 150 mm.

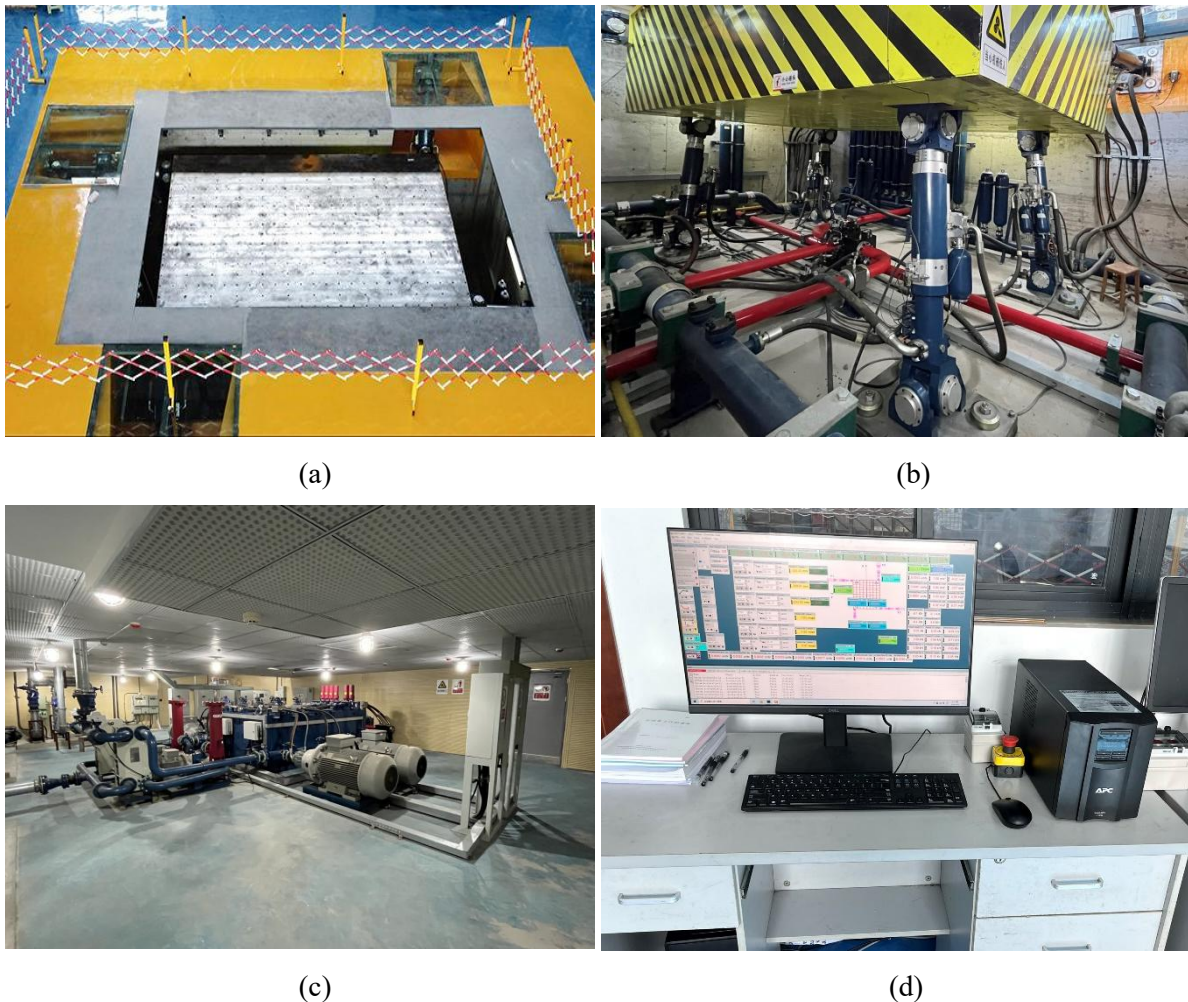


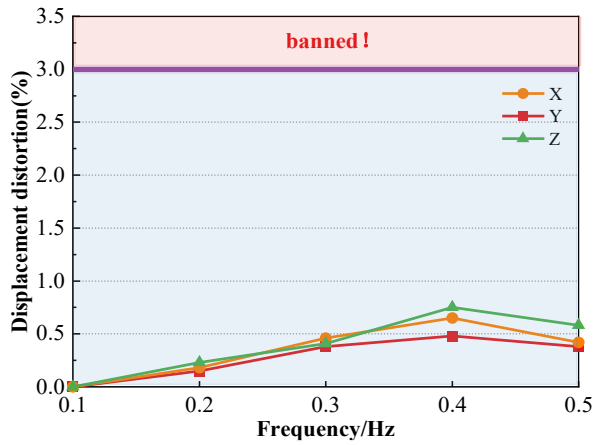
Figure 2 The shaking table system. (a) shaking table; (b) hydraulic actuator; (c) hydraulic oil source system; (d) shaking table controlling center.

The seismic simulation shaking table system can experience waveform distortion due to various factors, such as insufficient connection stiffness, high harmonics from the localized shaking on the table, nonlinearity of the servo valve, and the interactive effects between the specimen and the table. The degree of waveform distortion can be measured by the distortion degree, which comprises displacement waveform distortion and acceleration waveform distortion. The equation for calculating the distortion degree is given in **Equation (1)**. The distortion of displacement waveform is assessed in the maximum displacement range, by applying sinusoidal vibration to an inertia specimen at 50%~80% of the maximum displacement level. The system undergoes testing at five different frequencies (0.1, 0.5, 1, 3, and 5 Hz), with each test lasting a minimum of five minutes. The shaking table system is required to maintain a maximum displacement distortion of less than 3% for sinusoidal vibrations below 5 Hz in each degree of freedom. As illustrated in **Figure 3(a)**, the test results demonstrate that the displacement distortion values in the X/Y/Z directions are all below the 3% threshold, indicating minimal waveform distortion under displacement control has been achieved.

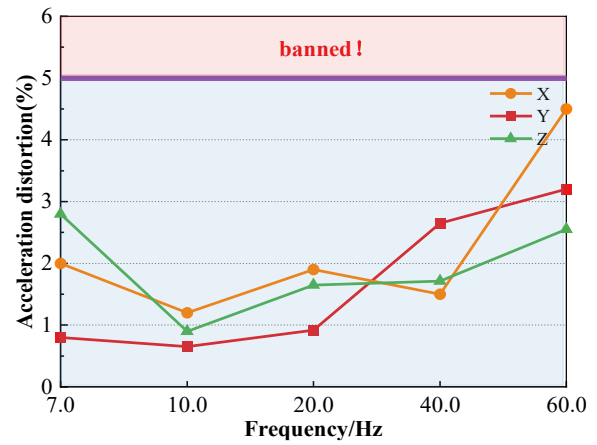
$$d = \frac{\sqrt{\sum_{i=2}^{11} D_i^2}}{D_1} \times 100\% \quad (1)$$

where D_1 is fundamental amplitude, and D_i denoted harmonic amplitude. The distortion degree of acceleration waveform is tested in the velocity and acceleration section with the maximum function. Under the action of inertia load, acceleration control mode is implemented for testing. The testing process focuses on five key frequencies (7, 10, 20, 40, and 60 Hz), and the acceleration distortion degree is required to remain below 5% before the iteration. The test results are presented in **Figure 3(b)**. It can be seen that with the use of acceleration control, the distortion levels for the X/Y/Z axes remain less than 5%, indicating the suitability of the shaking table system for high-precision testing applications.

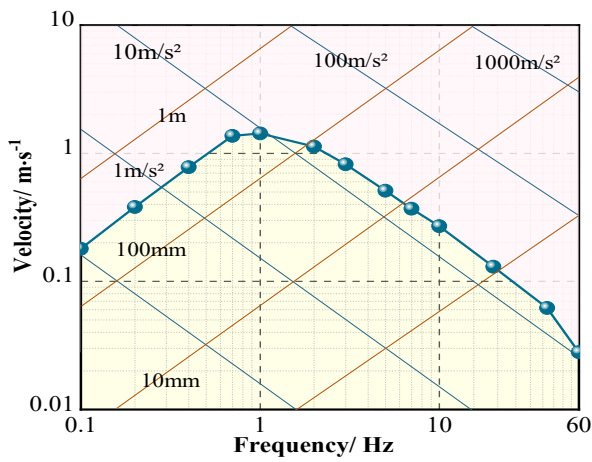
The maximum performance curve refers to the ultimate performance parameters of the seismic simulation shaking table, including its maximum acceleration, displacement, and velocity. During the measurement of the table's maximum capabilities, a sinusoidal wave with a fixed frequency is applied, and the curve is measured in three segments. The maximum performance curves of the shaking table system in the X and Z directions are shown in **Figures 3(c) and 3(d)**. The composition of the whole seismic simulation shaking table system is shown in **Figure 3(e)**.



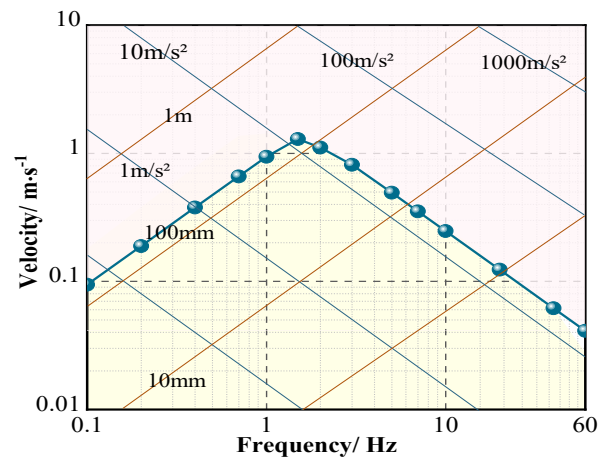
(a)



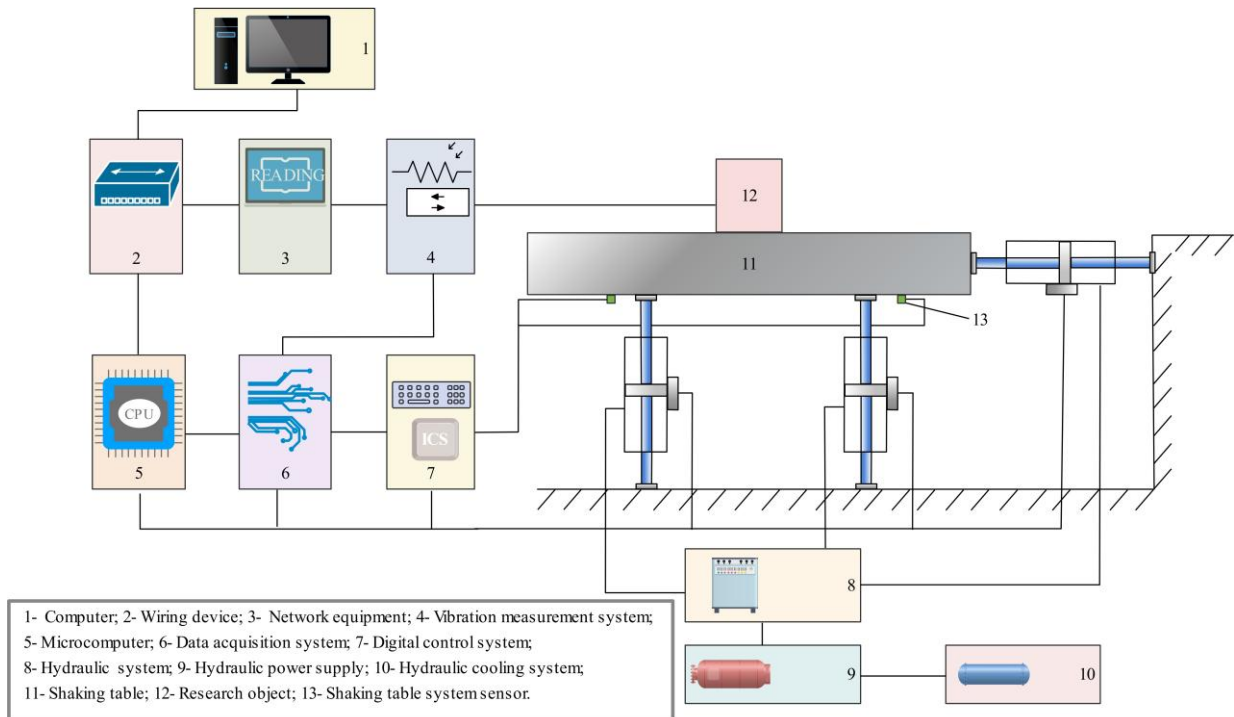
(b)



(c)



(d)



(e)

Figure 3 Three-dimensional six-degree-of-freedom seismic simulation shaking table system. (a) displacement distortion curve; (b) acceleration distortion curve; (c) maximum performance curve in the X direction; (d) maximum performance curve in the Z direction; (e) shaking table system diagram.

3.2. Scaled Model

Based on the dimension of the shaking table, a steel structure model container measuring $3.6\text{ m} \times 1.5\text{ m} \times 2.1\text{ m}$ (Length $\times$ Width $\times$ Height) was designed, as depicted in **Figure 4(a)**. The model container was composed of steel plate, section steel, and plexiglass. To minimize seismic wave reflections on the model container boundaries and ensure continuous deformation between the slope model and the container, flexible materials were employed to absorb seismic waves during the shaking table tests. The properties of these materials, including elastic modulus, density, and damping ratio were carefully determined to identify the material thickness that best approximates the free-field dynamic response. Following this analysis, an optimal thickness for the flexible material was established as 20 cm (Xin et al., 2019; Zhang et al., 2018). Consequently, a 20 cm polystyrene foam board was installed inside the model container to simulate the absorbing material in the horizontal vibration direction. Plexiglass was also installed along the sidewalls of the model container parallel to the excitation direction, as illustrated in **Figure 4(b)**.



Figure 4 The model container and the measures of reducing boundary effect. (a) the model container; (b) the measures of reducing boundary effect.

To ensure that the model provides an accurate representation of the prototype, shaking table tests must adhere to similarity relationships under both static and dynamic conditions. In this study, the Buckingham π theorem was employed, which utilizes dimensional analysis to determine the geometric similarity ratio between the model and prototype. Thirteen key parameters were considered for the shaking table test, including geometric dimensions L , mass density ρ , and seismic acceleration a . These were designated as the fundamental control quantities with scaling $C_L=25$, $C_\rho=1$, and $C_a=1$, respectively. The similarity relationships and dimensional representations of the shaking table test are detailed in **Table 1**.

To accurately determine the physical and mechanical parameters of the model material, a series of direct shear tests, and uniaxial compression tests were conducted on rock mass samples. These tests were aimed at assessing the strength parameters and mechanical properties of the rock mass and structural planes. From the results of orthogonal tests on the similar material ratios for the slope model, an optimal mass ratio of 10:35:25:14:2 for gypsum, barite powder, quartz sand, water, and glycerin were selected for the slope model rock mass. Here, barite powder and quartz sand served as fine and coarse aggregates, respectively, while gypsum and water served as binding agents, and glycerin aided in water retention. The deformation, strength and stability of rock slopes were governed by the distribution, degree of development and mechanical properties of the internal structural planes. Therefore, the careful selection and proportioning of similar materials for structural planes are crucial for obtaining accurate test results (Qi et al., 2022; Song et al., 2018; Xin et al., 2024). Based on the results of laboratory physical and mechanical tests, clay, barite powder, and glycerol were chosen as similar materials for the structural plane, with a mass ratio of 10:25:1. **Table 2** presents the target values, actual measured values, and the degree of similarity for physical quantities such as density, elastic modulus, and cohesion of the slope model. The degree of similarity is defined as the ratio between the actual measured values from the slope model and the target values of the model.

Table 1 The similarity relationship and similarity ratios.

Number	Parameter	Similarity relation	Similarity ratio
1	Geometric dimension (L)	Dominating variable	$C_L = 25$
2	Acceleration (a)	Dominating variable	$C_a = 1$
3	Density (ρ)	Dominating variable	$C_\rho = 1$
4	Elastic modulus (E)	$C_E = C_L C_\rho C_a$	$C_E = 25$
5	Poisson's ratio (μ)	—	$C_\mu = 1$
6	Cohesion force (c)	$C_c = C_L C_\rho C_a$	$C_c = 25$
7	Internal friction angle (φ)	1	$C_\varphi = 1$
8	Stress (σ)	$C_\sigma = C_L C_\rho C_a$	$C_\sigma = 25$
9	Strain (ε)	1	$C_\varepsilon = 1$
10	Time (t)	$C_t = C_L^{1/2} C_a^{-1/2}$	$C_t = 5$
11	Displacement (d)	$C_d = C_L$	$C_d = 25$
12	Velocity (v)	$C_v = C_L^{1/2} C_a^{1/2}$	$C_v = 5$
13	Damping ratio (ζ)	—	$C_\zeta = 1$

Table 2 The physical mechanical parameters of model slopes.

Projects	Parameters	Density ρ (kg/m ³)	Elastic modulus E (Gpa)	Cohesion c (Mpa)	Internal friction angle φ (°)	Poisson's ratio μ
Rock mass	prototype value	2680	5.233	2.562	28	0.22
	target value	2680	0.2093	0.1152	28	0.22

	measured value	2520	0.1962	0.1025	25	0.18
	similarity degree	94.03%	93.74%	88.98%	89.29%	81.82%
	prototype value	1910	0.7815	0.6831	25	0.36
Structural	target value	1910	0.0313	0.0317	25	0.36
plane	measured value	1860	0.0256	0.0273	22	0.35
	similarity degree	97.38%	81.79%	86.12%	88.00%	97.22%

The construction processes of the benched bedding rock slope model were divided into three primary components: rock mass, structural plane and base. Initially, the outline of the slope, including the positions of the interlayer and base, was marked on the acrylic plates and affixed inside the plexiglass model container as depicted in **Figure 5(a)**. For the model base, M5 cement mortar and bricks were used to precisely control the angle of slope rock mass and structural plane. During the filling process, slope materials were thoroughly mixed in the designated proportions and layered from bottom to top following the volume control principle, with each layer's filling thickness maintained at about 5 cm (see **Figure 5(b)**). The surface of each rock mass and interlayer layer was compacted and smoothed using wooden boards to ensure consistency throughout the construction process as shown in **Figure 5(c)**. The density of each layer was verified using the ring knife method to ensure the uniformity of the model rock mass. Accelerometers and earth pressure cells were strategically embedded at specific positions within the model as per the design scheme (see **Figure 5(d)**). To safeguard the functionality of the sensors, they were encased with cling film and sealed with adhesive tape for waterproofing. After assembly, the model was allowed to cure to ensure the materials reached their design strength prior to testing. Gridlines spaced at 10 cm intervals were drawn on the completed bench slope surface and reflectors were placed at grid intersections to facilitate the collection of displacement data during seismic activity as depicted in **Figure 5(e)**. The entire construction process for the benched bedding rock slope model is shown in **Figure 5**.

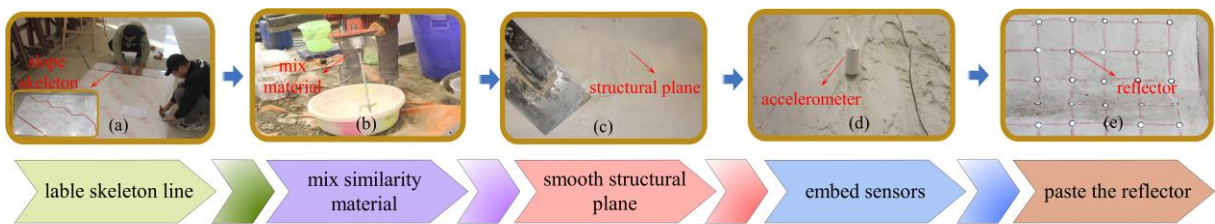
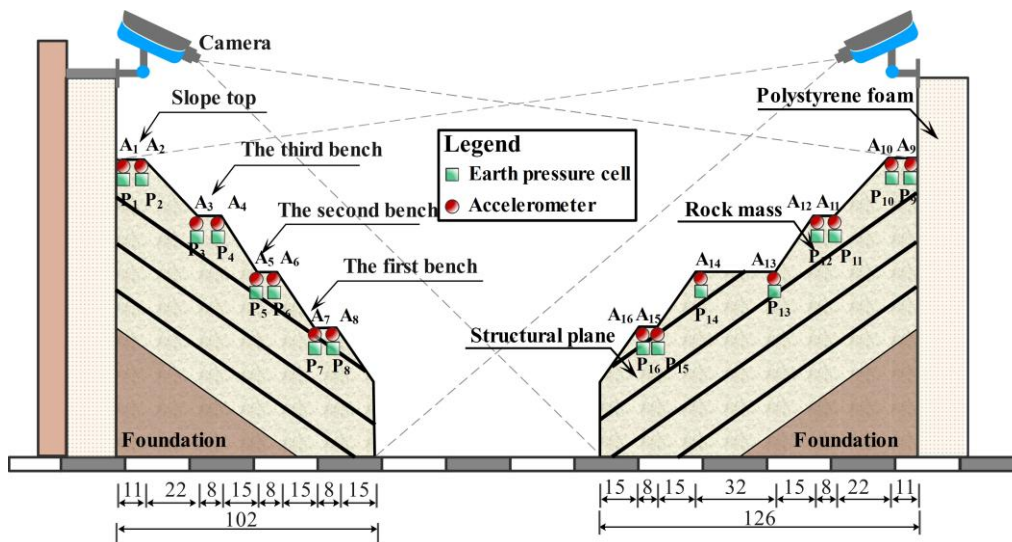


Figure 5 Construction process of the model slopes in the shaking table test. (a) lable skeleton line; (b) mix similarity material; (c) smooth structural plane; (d) bury sensors; (e) paste the reflector.

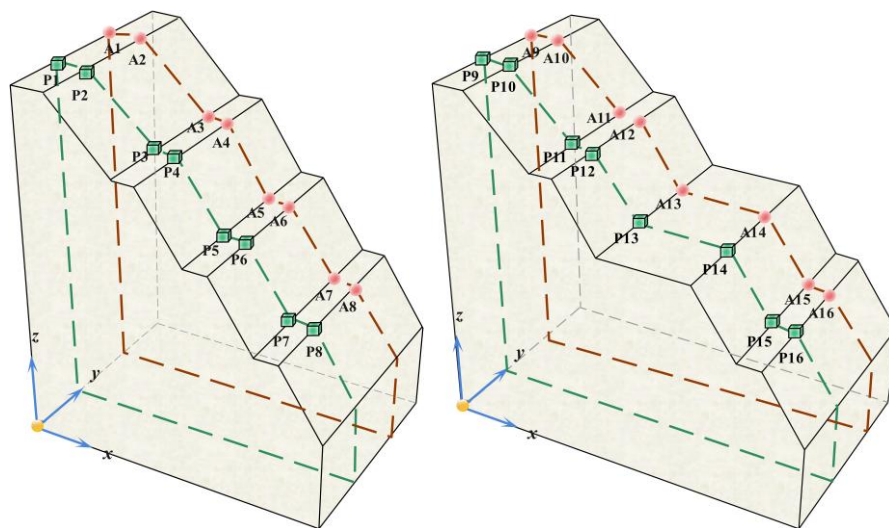
3.3. Measuring Systems

Following the geological mapping of the slope, a benched bedding rock slope with a structural plane was identified for the study. Four bench slopes with different landform patterns were selected for examination, including slopes with uniform bench widths of 2 m, 4 m, and 6 m, and a slope with an uneven bench width of 8 m. These widths comply with the requirements of the *Specifications for Design of Highway Subgrades* (JTGD30-2015) in China, which specifies the dimensions and traffic functionality of the bench slope platform. Adhering to the geometric similarity ratio of 1:25 established for the test, the width of the model slopes with uniform bench were set to 8 cm, 16 cm, and 24 cm, respectively. For the model with an uneven bench width, the second platform was set to 32 cm, while the width of the other benches was set to 8 cm. Each of the four model slopes featured three benches, with each bench standing at 22 cm high, culminating in a total height of 118 cm. Scheme 1 was applied to the model slopes with 8 cm uniform bench width and the uneven bench width, while Scheme 2 was designated for model slopes with uniform bench widths of 16 cm and 24 cm (as shown in **Figure 6**). In addition, the model slope incorporated a structural plane with a thickness of 0.5 cm, positioned at an angle of 35° to the horizontal plane, and spaced at 15 cm intervals.

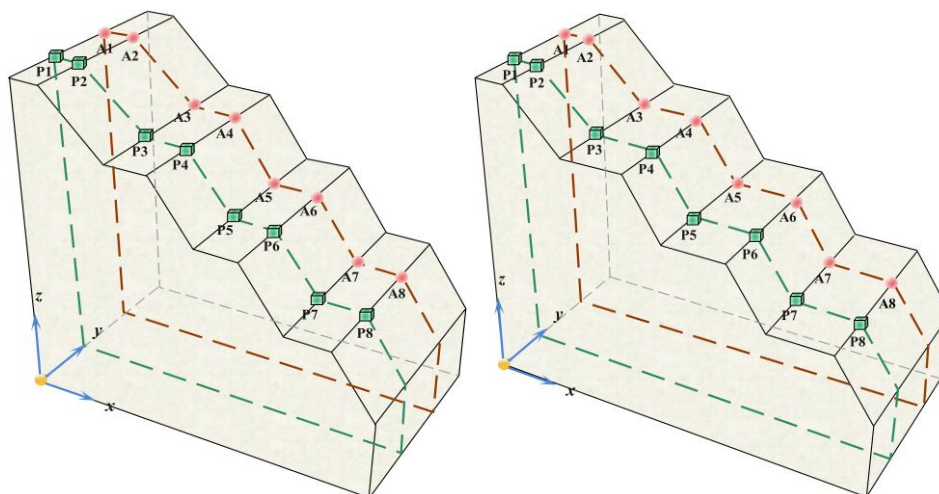
To fulfil the objectives of the test, accelerometers and earth pressure cells were strategically positioned at both the internal corners and external corners of each platform within the model slope (as depicted in **Figure 6**). These sensors were utilized to analyze the dynamic response of the slope at various elevations when subjected to ground motion, aiming to ascertain the deformation and failure laws of the slope. **Table 3** shows the sensor types used in this study. During the loading process, the dynamic behavior of the slope model was captured in real-time using a high-definition camera. For precise real-time monitoring of the surface displacement, reflectors were affixed to the slope surface. These reflectors were then tracked using a laser displacement meter, allowing for accurate measurement of movement during the tests.



(a)



(b) (c)



(d) (e)

Figure 6 The schematic diagram of the model slope and the monitoring points. (a) the schematic diagram of the model slope; (b) the arrangement of sensors in the slope with 2m uniform bench width; (c) the arrangement of sensors in the slope with uneven bench width; (d) the arrangement of sensors in the slope with 4m uniform bench width; (e) the arrangement of sensors in the slope with 6m uniform bench width. (Unit: cm)

Table 3 Types of Sensors.

Type	Number	Illustration	Note
Accelerometer	16	Monitoring acceleration at different positions of the slope	



3.4. Testing Sequences

The shaking table test was conducted by selecting the east-west (E-W) component of the strong earthquake recorded at Wolong Station, near the fault in the Wenchuan earthquake, to serve as the dynamic load input. The seismic excitation was horizontally applied in the X-direction. The time history and Fourier spectra are shown in **Figures 7(a) and 7(b)**. Spectrum analysis revealed that seismic wave energy was primarily concentrated in the first 10 s in the time domain and 12 Hz in the frequency domain (see **Figure 7(c)**). To evaluate the progressive failure of the model slope subjected to varying excitation amplitudes, six distinct loading conditions were tested: 0.1 g, 0.2 g, 0.4 g, 0.6 g, 0.8 g, and 1.0 g. Following each loading, the deformation and failure of the model slope were observed and recorded at 20-minute intervals. Before each excitation, a 0.05 g white noise was also applied to obtain the natural frequency of the model slope. The test loading scheme is illustrated in **Figure 7(d)**.

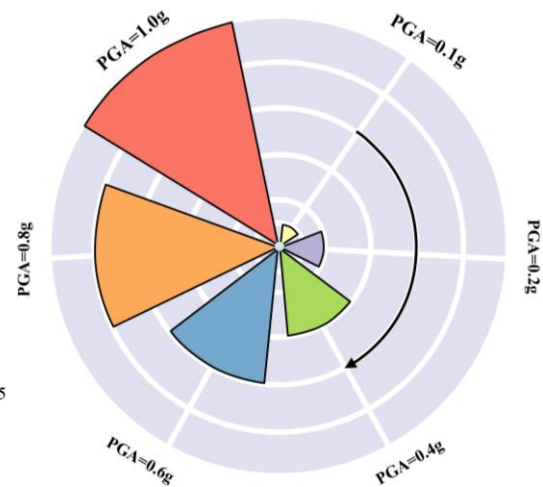
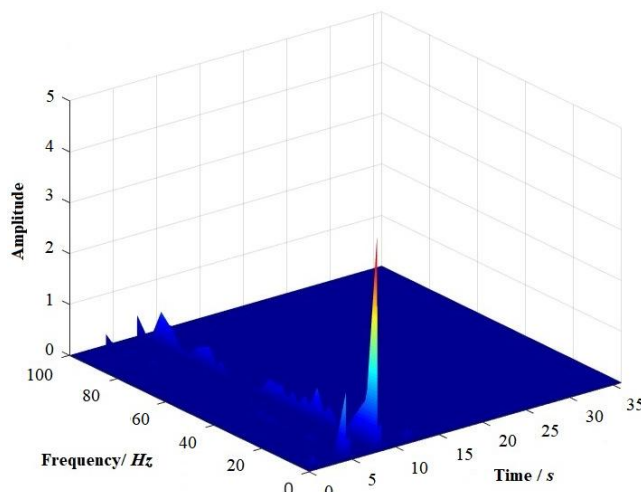
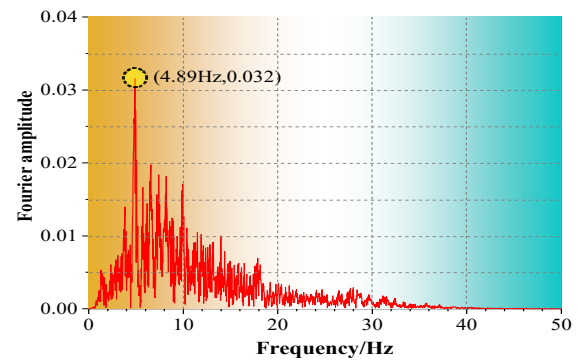
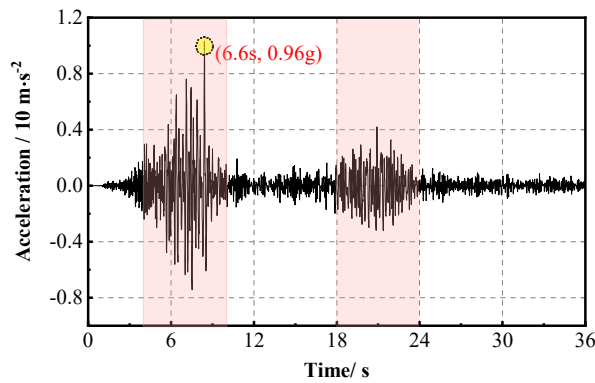


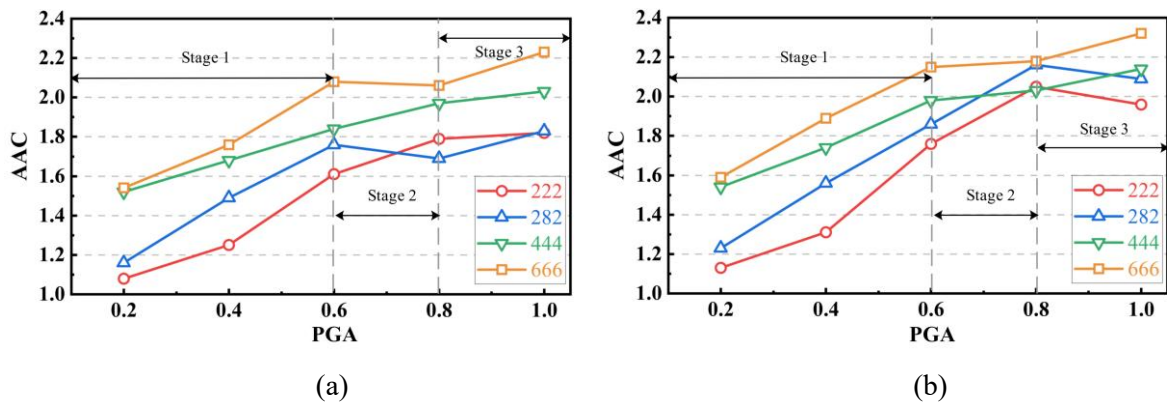
Figure 7 The Wolong wave characteristics and loading scheme. (a) the time history of E-W component; (b) Fourier spectra of E-W component; (c) time-frequency distribution of seismic wave; (d) loading scheme in the shaking table test.

4. Analysis of the Test Results

4.1. Seismic response characteristics of slope model

4.1.1. Distribution law of slope's acceleration

The inertial force generated by seismic acceleration is a critical factor for slope deformation and instability under an earthquake action. Consequently, variations in acceleration are essential for evaluating the dynamic response of the slope (Wu et al., 2020; Zhou et al., 2022). Therefore, this section focuses on the horizontal acceleration amplification coefficient of the benched bedding rock slope with structural planes at different platform positions as illustrated in **Figure 8**. In this figure, legend “222” implies that the actual widths of the third, second, and first benches of the slope are all 2 m, whereas “282” signifies that the actual widths of the third, second, and first benches are 2 m, 8 m, and 2 m, respectively. The acceleration amplification coefficient (AAC) is defined as the ratio of the peak horizontal acceleration at each measuring point to the peak acceleration at the base of the shaking table. The acceleration responses at the internal and external corners of the first and second benches were analyzed under various excitation amplitudes. The results indicate that the acceleration response of all four slope models significantly increases as the excitation amplitude rises, particularly up to 0.6 g. Among these, the slope with 6 m uniform bench width displays the highest acceleration values. When the excitation amplitude ranges from 0.6 g to 0.8 g, the acceleration response at the internal corners of the first and second benches of the slope with uneven bench width decreases significantly (as shown in **Figures 8(a)** and **8(c)**). This reduction is attributed to the extensive propagation of internal cracks within the slope when the excitation amplitude exceeds 0.6 g. These cracks absorb seismic wave energy, thereby reducing the monitored acceleration at the internal corners.



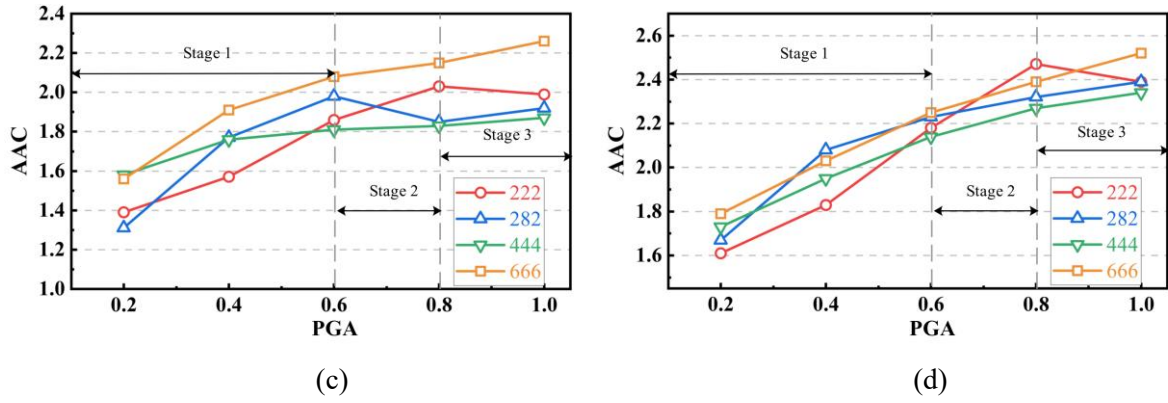
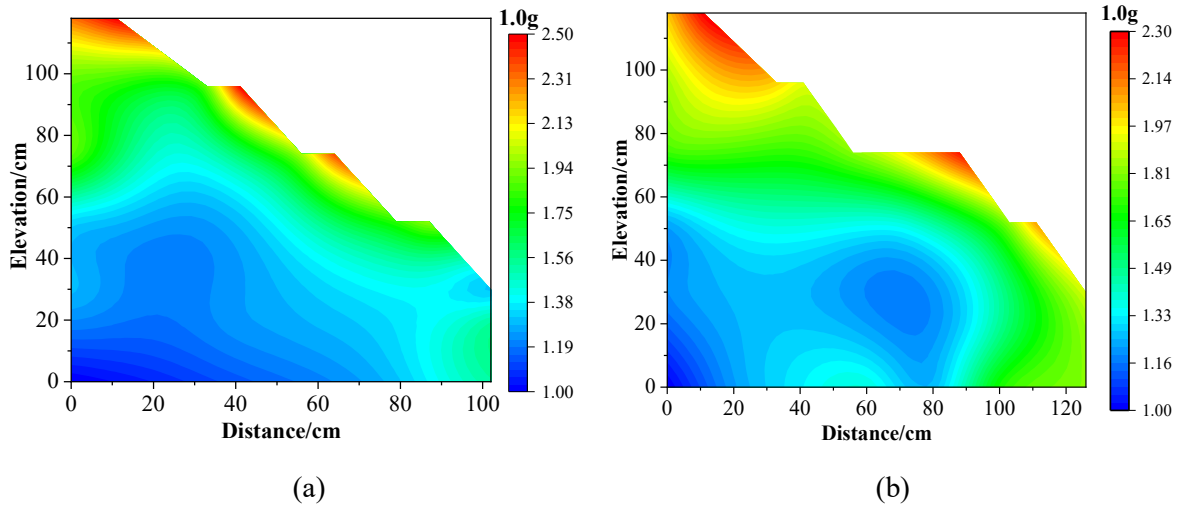


Figure 8 Distribution of acceleration amplification coefficients of four slope models under different excitation amplitudes. (a) the AAC at the internal corner of the first bench; (b) the AAC at the external corner of the first bench; (c) the AAC at the internal corner of the second bench; (d) the AAC at the external corner of the second bench.

As seen in **Figure 9**, the AAC gradually increases from the bottom to the top of the slope and reaches the maximum values at the slope top regardless of the slope surface patterns. This trend highlights the elevation amplification effects (EAE) of seismic-induced acceleration responses in bedding rock slopes. The test results reveal that AACs at the internal corners of benches are consistently lower than those at the external corner. This disparity can be attributed to the fact that the variations of slope gradients, which lead to conspicuous stress concentration at the internal corners of the benches. Moreover, compared to the interior of the slope, the surface is more sensitive to seismic excitation due to the lack of internal constraints, making it more prone to loosening and collapse.



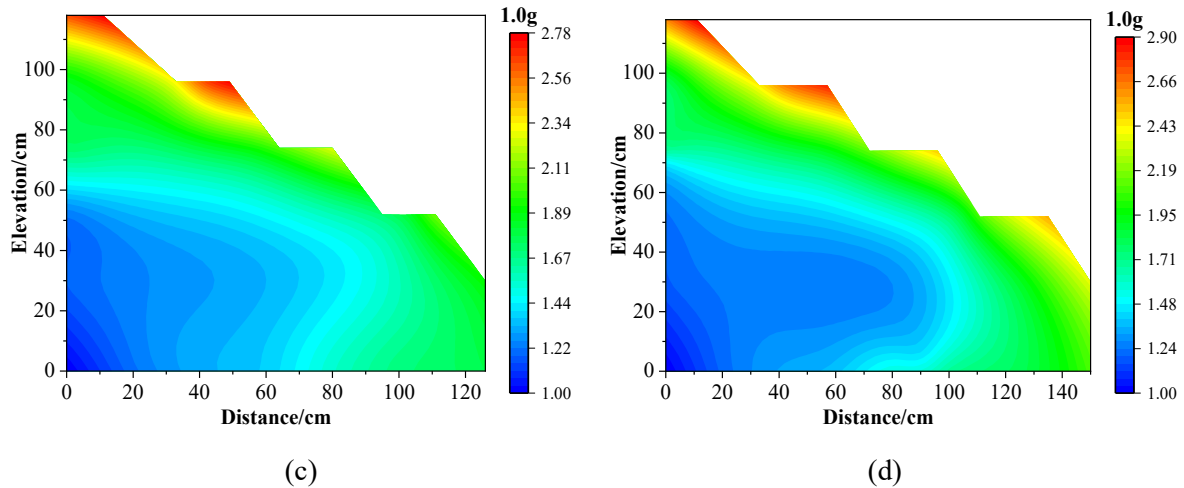


Figure 9 The AAC distribution of the four slopes at 1.0g. (a) the AAC distribution of the slope with 2 m uniform bench width; (b) the AAC distribution of the slope with uneven bench width; (c) the AAC distribution of the slope with 4 m uniform bench width; (d) the AAC distribution of the slope with 6 m uniform bench width.

4.1.2. Distribution law of slope's earth pressure

The stability assessment of slopes after an earthquake primarily depends on understanding the stress changes induced by seismic excitation and the subsequent stress redistribution. These factors are crucial for determining the deformation mechanisms and damage modes of slopes during multi-stage seismic events. For example, benched bedding slopes experience damage due to the combined effects of dynamic and residual earth pressures. During seismic excitation, both dynamic and residual earth pressures develop. While dynamic earth pressure dissipates after excitation, residual earth pressure persists, causing further deformation until significant cracking occurs. This section analyzes the peak earth pressure responses at the internal corners of the first and second benches in four slope models to explore the pressure response patterns.

As shown in **Figure 10**, the earth pressure increases with excitation amplitude, ranging from 0.2g to 0.6g. This is attributed to enhanced particle interaction within the slope material, leading to higher contact forces, internal friction, and shear strength. However, when the excitation amplitude exceeds 0.6g, the earth pressure in the slope generally decreases with increasing amplitude. Among the four slope models, the slope with uneven bench width exhibits the largest reduction in earth pressure. At the internal corner of the first bench, the earth pressure reduction is 67.28% and at the second bench, it is 33.47%. Combined with failure observations from the shaking table tests, when the excitation amplitude exceeds 0.6g, severe damage occurred at the first and second benches of the slope with uneven bench width. Extensive crack propagation absorbs seismic energy, resulting in reduced earth pressure.

Crack damage in the rock mass limits the accumulation of residual earth pressure. Existing cracks hinder further increases in residual pressure, causing a nonlinear response. Additionally, residual earth

pressure at the external corner of the second bench is mainly due to seismic inertial forces acting from the third bench and slope top during excitation. In conclusion, the gradual increase of residual earth pressure induced by earthquakes is significant and should not be overlooked. Real-time monitoring of earth pressure responses, especially at internal corners, is essential for slope engineering.

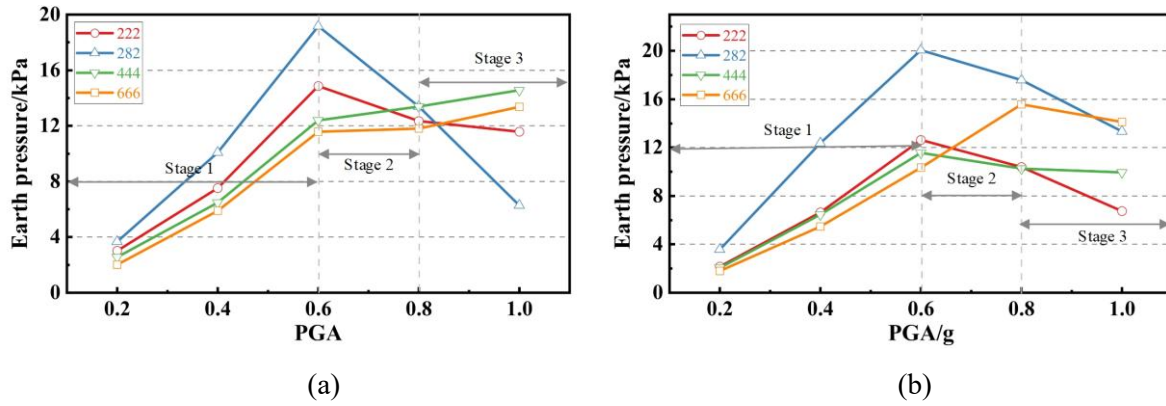


Figure 10 The earth pressure distribution at the internal corner of the first bench and the second bench in slope model with different excitation amplitudes. (a) the earth pressure at the internal corner of the first bench; (b) the earth pressure at the internal corner of the second bench.

4.1.3. Failure process analysis

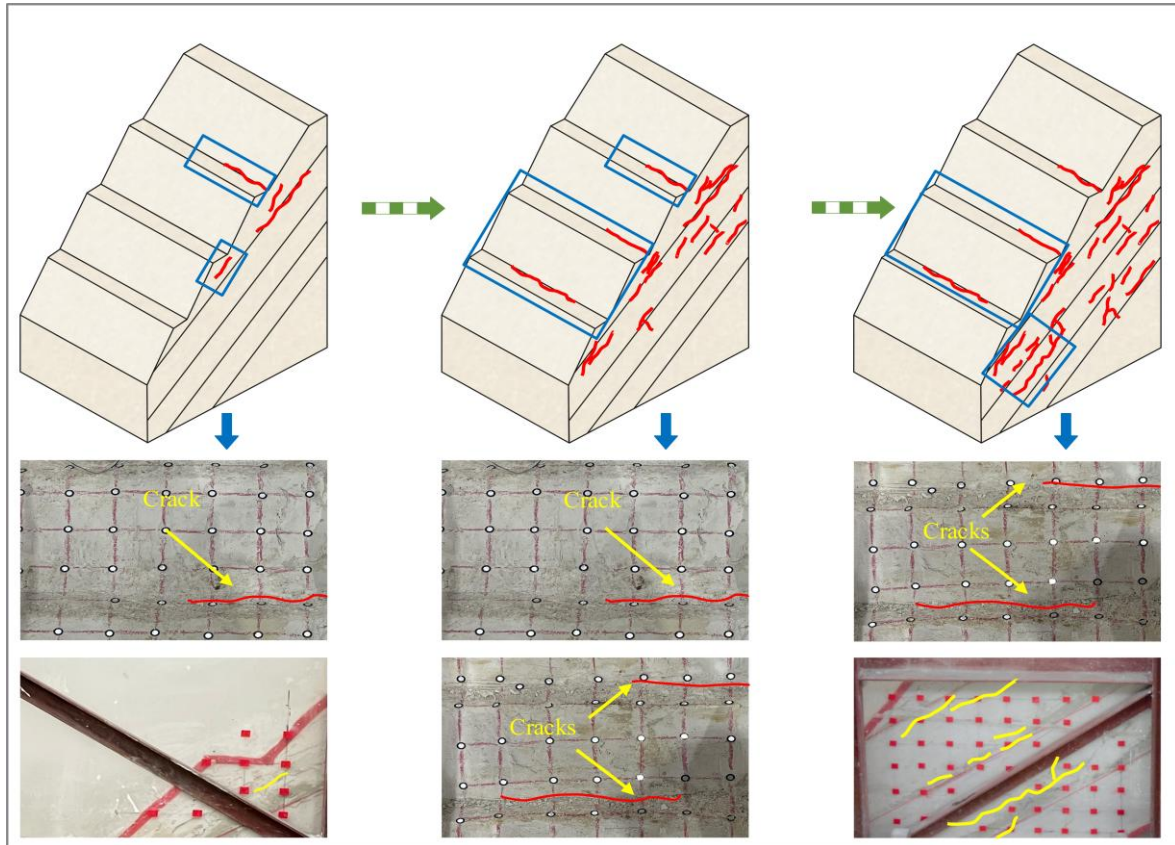
Figure 11 illustrates the failure phenomena of four slope models subjected to multi-stage seismic excitations. In the scheme 1 of tests, a slope with 2 m uniform bench width exhibited no significant failure when the acceleration amplitude was below 0.6 g. However, when the amplitude increased to 0.6 g, an initial crack appeared at the internal corner of the third bench. Under repeated tension-shearing action of seismic waves, tensile cracks appeared at the internal corners of all benches, and numerous tension-shearing cracks parallel to the interlayer appeared on both sides of the structural plane near the slope surface at an acceleration amplitude of 0.8 g. At this stage, the shear stress became more concentrated at the internal corners, diminishing the shear strength and stability of the slope. At an acceleration amplitude of 1.0 g, the surface cracks of the slope propagated and became deeper and wider, while the density of cracks on the side of the slope increased. From the front to the rear edge of the slope, the density of cracks decreased successively, indicating that the transverse shear action of seismic waves had a significant impact on the transformation of the rock mass at the bottom. This also implies a propensity for the slope to slide down along the structural plane. Despite the presence of cracks throughout the slope body under multi-stage earthquake action, no complete collapses were observed. The pre-existing cracks dissipated a portion of the seismic energy during the vibration excitation process, enhancing the filtering effect of the rock mass and resulting in failure phenomena characterized by produced cracks on the slope body without a loss of overall stability (Liu et al., 2024; Lu et al., 2015; Wang et al., 2024).

In contrast, the slope with uneven bench width exhibited more severe failure phenomena (as shown in **Figure 11(b)**). At an excitation acceleration amplitude of 0.6 g, initial tensile cracks appeared at the internal corner of the second bench due to the seismic dynamics. Concurrently, the slope was affected by the elevation amplification effects (EAE), resulting in shear cracks parallel to the structural plane's strike in the rock mass with high elevations on the slope, with the rock mass tending to slide along the structural plane under seismic action. When the acceleration amplitude reached 0.8 g, cracks at the external corner of the second bench rotated outward from slope due to the synergistic effect of the free surface amplification and seismic inertia force, manifesting as the propagating of cracks along the slope's free surface in the reciprocating vibration process. The tension cracks, generally wider at the top and narrower at the bottom, gradually extended downward. After multi-stage seismic excitations, some cracks interconnected with shear cracks parallel to the interlayer strike, splitting the slopes into fractured rock mass and increasing the wave filtering effects of the slope bodies. At an excitation acceleration amplitude of 1.0 g, the second wider bench tended to slide down along the structural plane under the strong seismic inertia force, with the upper part of the slope structure compressing the lower part repeatedly during the earthquake dynamic action. Eventually, the first bench of the slope was completely squeezed and slid down quickly along the structural plane and out of the slope. It is worth noting that the fractured rock fell into the crevice, forming a wedge splitting effect that further expanded the crack downward at the moment of crack opening. After test loading, the rear edge of the slope was 8 cm away from the absorption boundary, and a settlement of 9 cm occurred, indicating that the upper structure of the slope slid along the structural plane. The position of each platform on the slope model showed significant misalignment compared to its initial state after the earthquake.

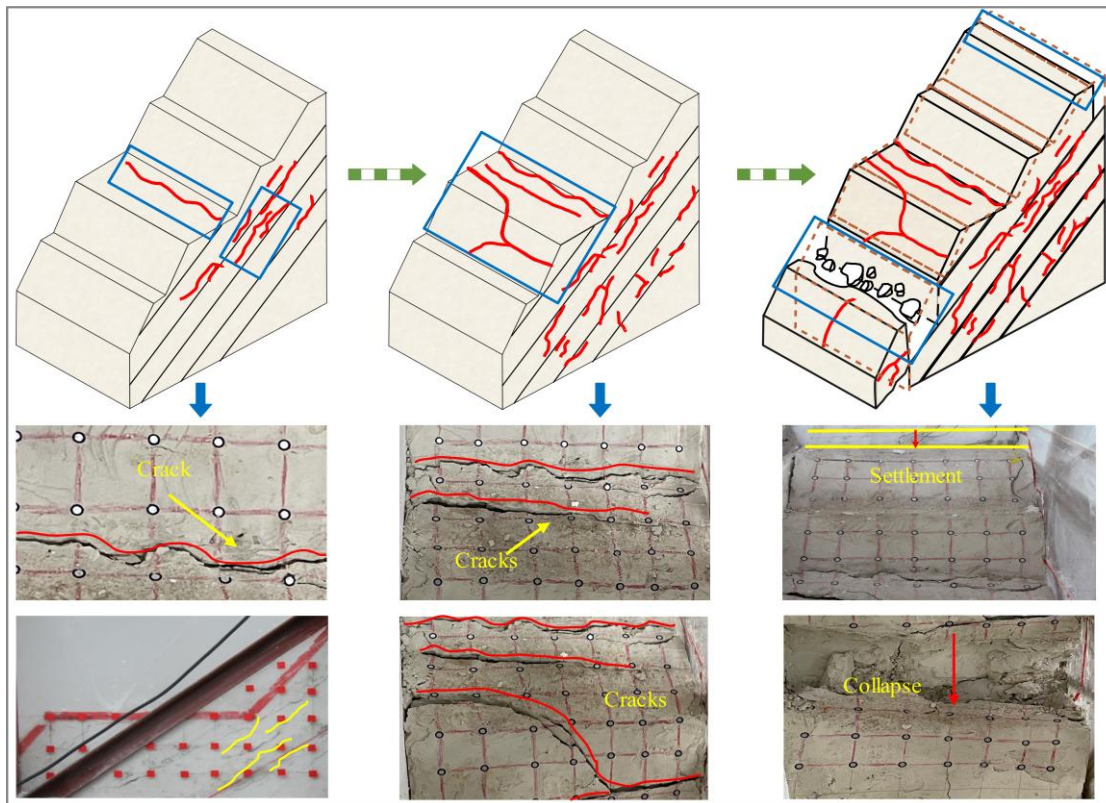
Compared to scheme 1, the slope failures in scheme 2 were relatively minor (as shown in **Figure 11(c)** and **11(d)**), with only tensile cracks at the internal corner and a few tensile-shear cracks along the slope side. As the bench width of the slope increased from 2 m to 6 m, both the degree of failure and the development of cracks gradually reduced. These observations suggest that the stability of the slope enhanced with the widening of the slope platform width, as evidenced by the reduction in failure phenomena compared to slopes with uniform bench widths of 2 m, 4 m, and 6 m.

Following the completion of the loading in scheme 1, the slope was dissected layer by layer along the structural plane, revealing irregular tensile cracks perpendicular to the structural plane and compressive-shear cracks parallel to it on the first and second structural surfaces (see **Figure 12**). A comparison between **Figure 12(a)** and **Figure 12(b)** indicates that cracks were densely developed in the middle and lower parts of the slope, mainly due to the tendency of the rock mass to slide downward along the structural plane under seismic excitations. As the seismic intensity increased, this sliding tendency became more pronounced, resulting in differential deformation across each structural plane and scratches on the interlayer surfaces within the slope. This confirmed the relative displacement between layers in the slope during seismic excitations. It was noteworthy that the slope with uneven

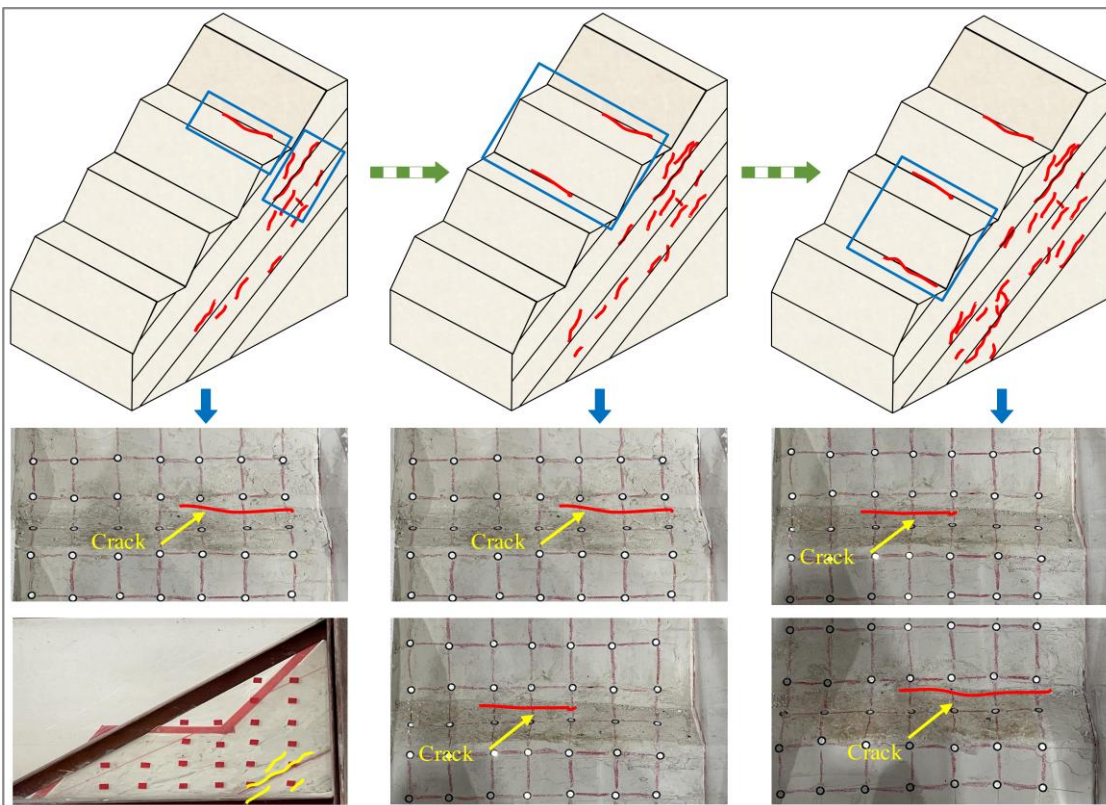
bench width was dissected and exhibited cracks spanning the entire width of the second bench at the first and second interlayer planes (as depicted in **Figures 12(c) and 12(d)**). This pattern was consistent with the penetrating cracks of the second bench at the internal corners of the slope surface induced by seismic waves. These observations indicated the differential deformation of rock masses between layers in the slope, with the interlayer serving as a potential sliding surface. Under intense seismic excitations, the slope might slide along the structural plane due to the continuous tensile action of seismic waves.



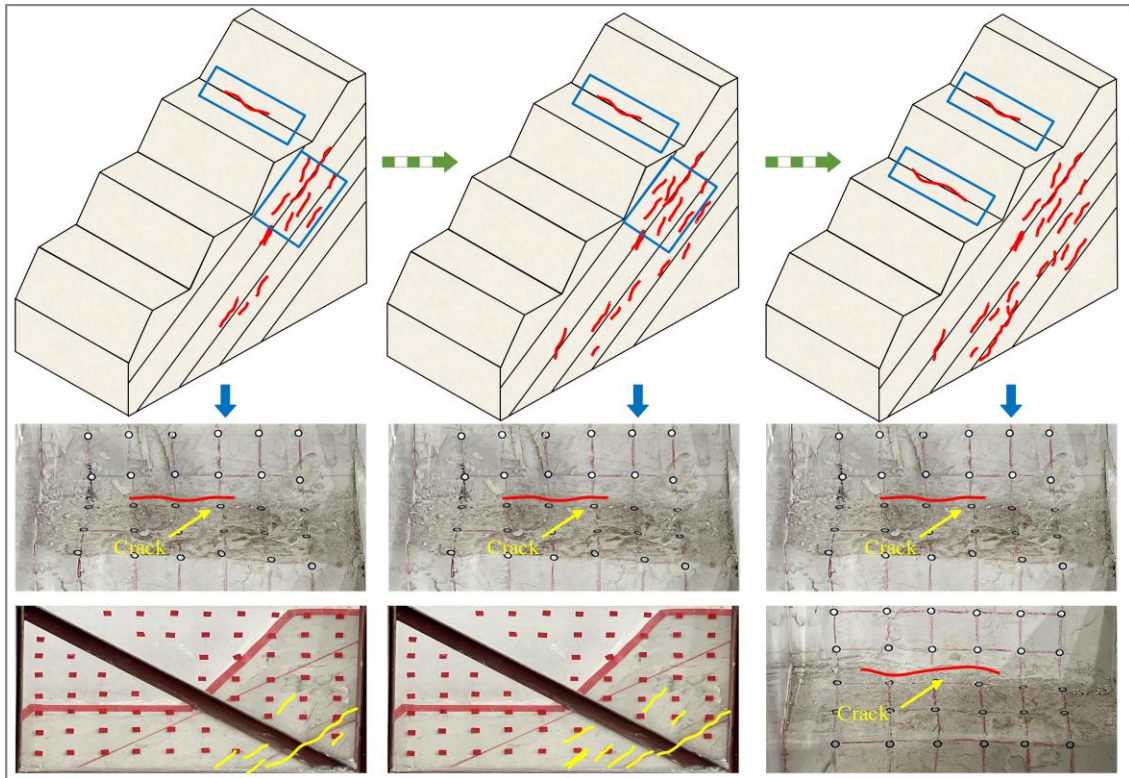
(a)



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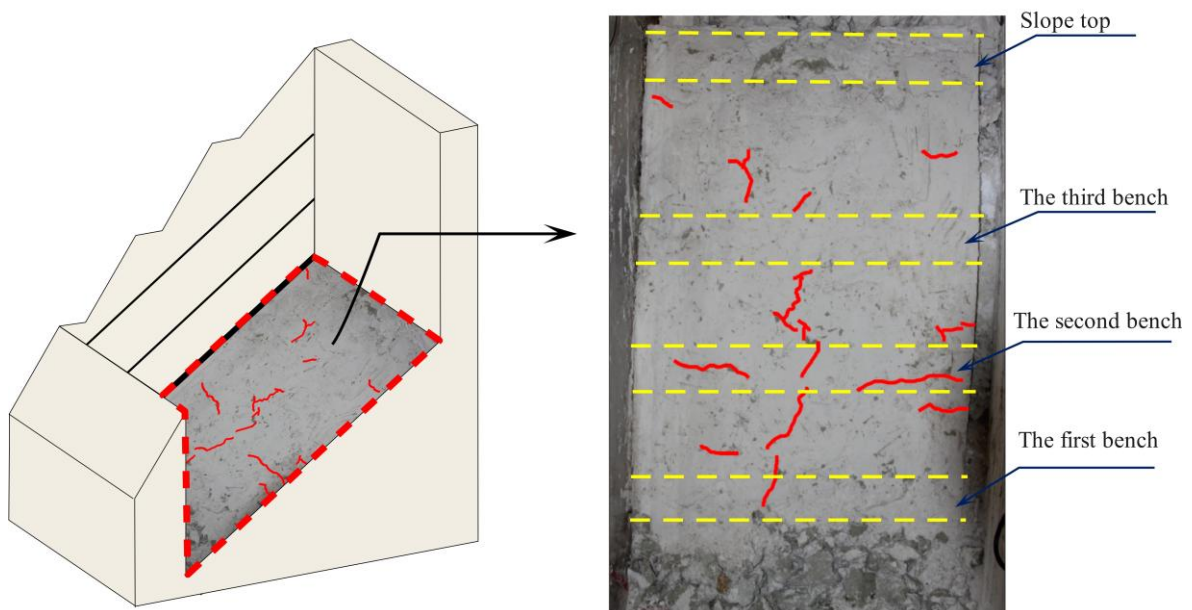


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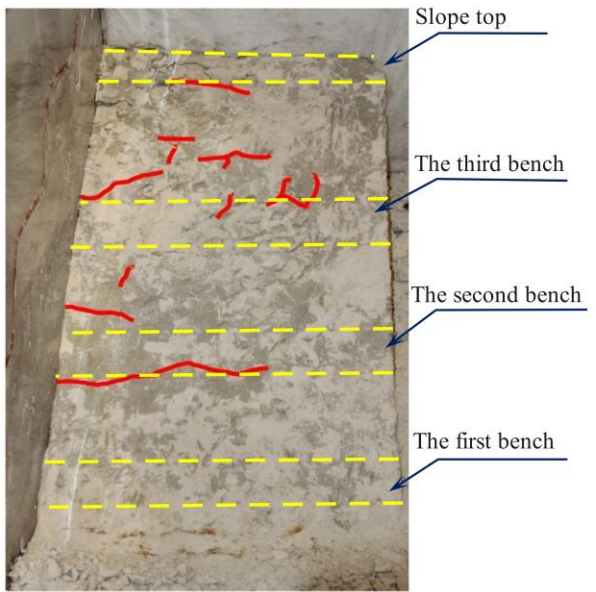
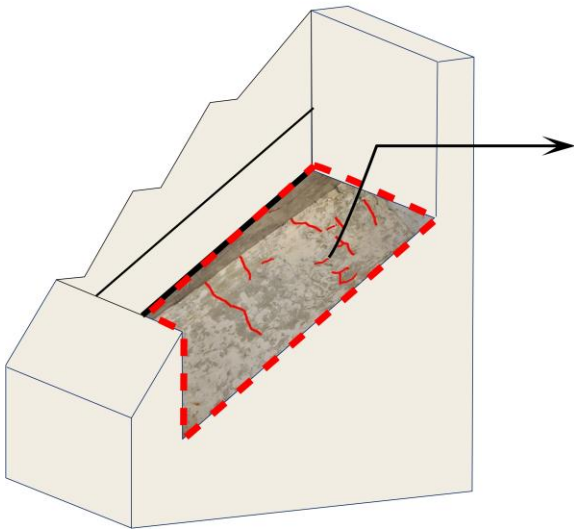


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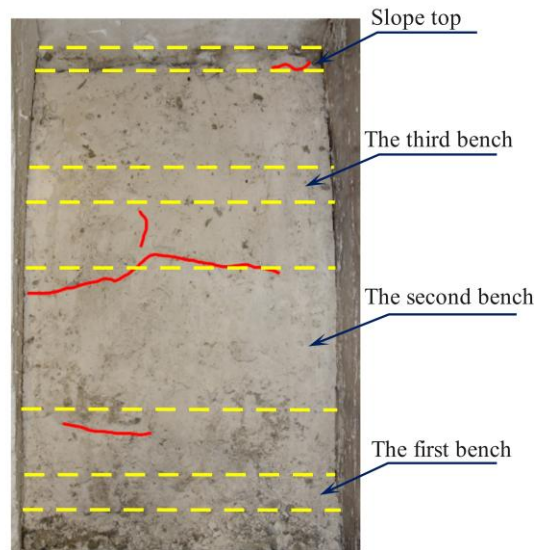
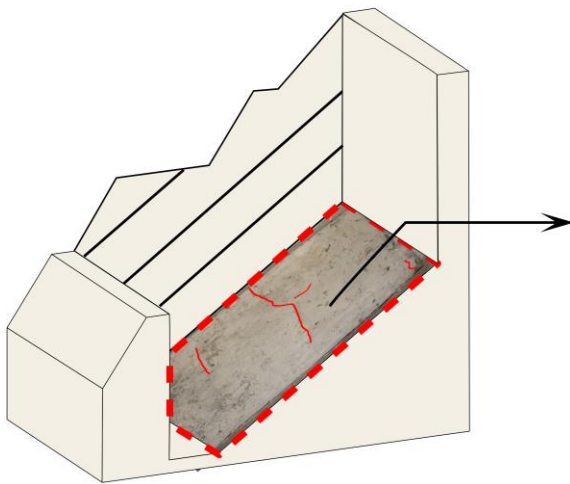
Figure 11 The failure phenomena of four slope models subjected to multi-stage seismic excitations. (a) failure phenomena of slope with 2m uniform bench width after seismic excitations; (b) failure phenomena of slope with uneven bench width after excitations; (c) failure phenomena of slope with 4m uniform bench width after seismic excitations; (d) failure phenomena of slope with 6m uniform bench width after seismic excitations.



(a)



(b)



(c)

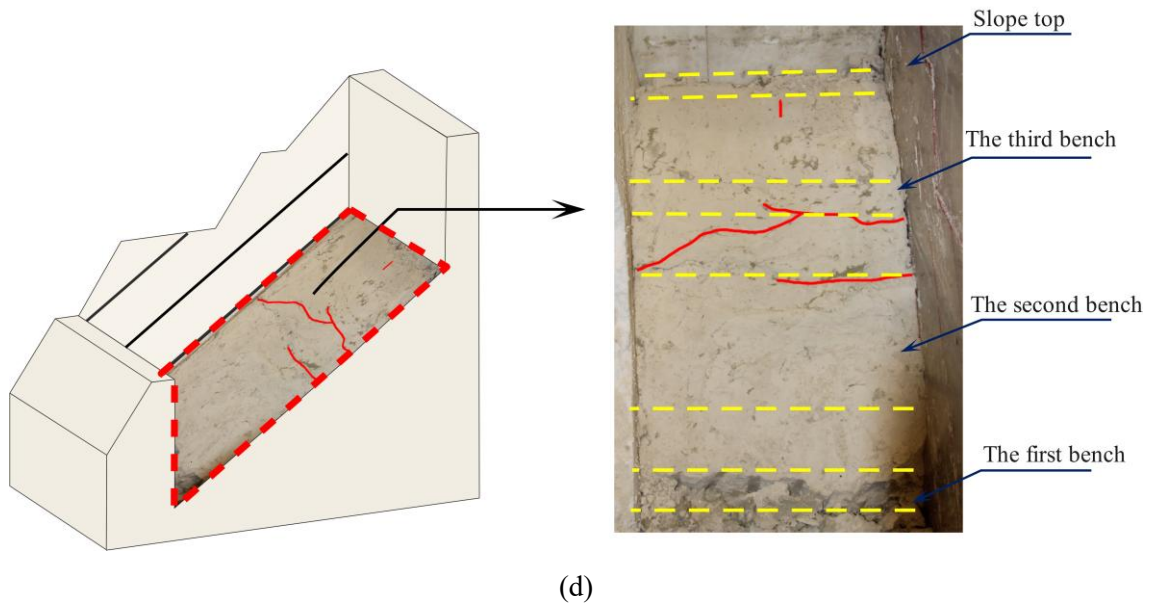


Figure 12 The failure phenomena of two slope models subjected to multi-stage seismic excitations. (a) cracks distribution of the first structural plane of the slope with 2 m uniform bench width dissected after seismic excitations; (b) cracks distribution of the second structural plane of the slope with 2 m uniform bench width dissected after seismic excitations; (c) cracks distribution of the first structural plane of the slope with uneven bench width dissected after seismic excitations; (d) cracks distribution of the second structural plane of the slope with uneven bench width dissected after seismic excitations.

4.2 Slope stability analysis

In the field of rock slope failure prevention and control, the primary task is to calculate and evaluate slope stability (Hu et al., 2024; Pradhan et al., 2020; Song et al., 2021). Prior to conducting slope stability analysis, it is necessary to assess the potential failure modes, corresponding failure directions, failure scale, and impact scopes under geotechnical engineering geological conditions, as outlined in Section 5.2.1 of the *Technical Specifications for Construction Slope Engineering* (GB50330-2013) of China. Currently, the stereographic projection method is widely adopted for qualitative analysis to evaluate the influence of structural planes on bench slopes. This method allows direct determination of the relationship between the structural plane and the slope body. However, most analyses conventionally consider the slope angle of a single bench as the boundary for determining local failure, where the boundary is defined as the free surface of the bench slope. Thus, the results of such analyses can only represent the failure mode of a single bench rather than the failure mode of multi-bench combination. Since the characteristics of the bench slope are not examined in detail, the corresponding boundary conditions remain ambiguous. Therefore, to conduct a comprehensive qualitative analysis of the impact of structural planes on slope stability, it is imperative to analyze the geometric characteristics of multi-bench combination slopes (Dong et al., 2025; Li et al., 2023; Song et al., 2020).

Assuming the geometrical configuration of the bench slope as depicted in **Figure 13**, where each bench exhibits an identical height (h) and bench angle (β), the main geometric parameters entail the overall slope height (H), the number of platforms (n), the count of uniform platforms underneath the wider platform (m), the uniform platform width (a), the wider platform width (b), the distance from the crest to the rear edge of the slope (c), and the length from the rear edge to the slope toe defined as the overall length (L). The angles involved are the overall slope angle (α), defined as the angle between the line connecting the slope toe to the crest and the horizontal plane, the bench angle (β), which is the angle between each bench and the horizontal plane, and the inter-ramp angle (θ), which is the angle between the slope toe, the external edge of the wider bench, and the horizontal plane. The following relationships between these parameters are valid:

$$h = \frac{H}{n} \quad (2)$$

$$L = c + (n-2)a + b + nh / \tan \beta \quad (3)$$

$$\tan \alpha = \frac{H}{L-c} = \frac{H}{(n-2)a + b + H / \tan \beta} \quad (4)$$

$$\tan \theta = \frac{\frac{m+1}{n}H}{ma + \frac{m+1}{n \tan \beta}H} \quad (5)$$

Hereon, we can calculate $\frac{1}{\tan \beta} = C_1 + \frac{1}{\tan \alpha}$, $\frac{1}{\tan \beta} = C_2 + \frac{1}{\tan \theta}$ respectively from **Equations (4)** and **(5)**, where the constant $C_1 = -\frac{(n-2)a+b}{H}$, $C_2 = -\frac{nma}{(m+1)H}$.

Finally, both overall slope angle (α) and inter-ramp angle (θ) can be represented by bench angle (β) as follows:

$$\alpha = \arctan \frac{1}{1 / \tan \beta - C_1} \quad (6)$$

$$\theta = \arctan \frac{1}{1 / \tan \beta - C_2} \quad (7)$$

By applying the equation $\frac{1}{\tan \beta} = C_1 + \frac{1}{\tan \alpha}$, $\frac{1}{\tan \beta} = C_2 + \frac{1}{\tan \theta}$, we can derive the relationship $C_2 - C_1 = \frac{1}{\tan \alpha} - \frac{1}{\tan \theta}$.

Accordingly, the final equation can be expressed as follows:

$$\sin(\theta - \alpha) = (C_2 - C_1) \sin \theta \sin \alpha \quad (8)$$

$$\cos(\beta - \alpha) = 2 \sin \alpha \sin \beta - C_1 \sin \alpha \cos \beta \quad (9)$$

Furthermore, it is evident that angles α and β must comply with the criterion of $\beta > \alpha$, wherein angle β varies from 0 to 90°. Based on the **Equation (6)** between α and β , it is apparent that as angle β increases, angle α increases proportionally. Similarly, the **Equation (7)** between θ and β demonstrates that as angle β expands, angle θ increases comparatively.

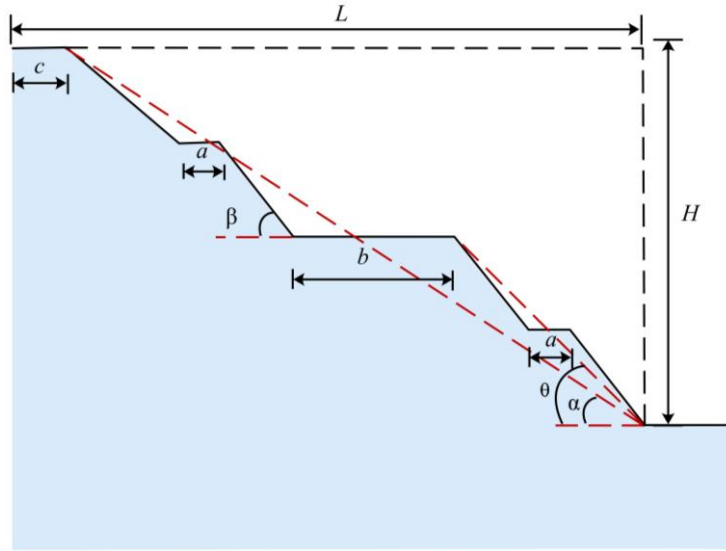


Figure 13 Geometric parameters of bench slope.

In a series of shaking table tests, four slope models with different bench widths were considered, representing a three- bench slope. These can also be viewed as mining slopes consisting of two safety platforms and one clean-out platform. The varying bench widths correspond to different slope angles. Four key angles in the slope model were defined: overall slope angle (α), bench angle (β), inter-ramp angle (θ), and structural plane dip angle (δ), as illustrated in **Figure 14**. In the shaking table tests, the slope model had a ratio of 1:0.7, and the overall slope angle (α), bench angle (β), and inter-ramp angle (θ) could all be determined by the bench height and platform width, as given in **Equations (10)–(12)**.

$$\tan \alpha = \frac{4h}{4l + 2a + b} \quad (10)$$

$$\tan \beta = \frac{h}{l} \quad (11)$$

$$\tan \theta = \frac{2h}{2l + a} \quad (12)$$

where l is defined as the projection length of each bench on the horizontal plane.

Based on previously derived equations, the relationships between the geometric characteristics of the bench slope have been established. Additionally, considering the inherent geological conditions of the slope (i.e., the dip angle of the structural plane), a series of geometric calculations involving the

overall slope angle (α), bench angle (β), inter-ramp angle (θ), and structural plane dip angle (δ) were performed to define the slope stability assessment equation, as shown in **Equation (13)**.

$$S_q = \sqrt{\frac{|\cos(\theta + \delta) \sin(\alpha + \delta)|}{\cos(\beta - \alpha) \sin(\theta - \alpha)}} \quad (13)$$

Hereon, the denominator of the equation is dictated by the values of $\beta - \alpha$ and $\theta - \alpha$, both of which range from 0° to 90° , with the requirement that θ cannot be equal to α . Regarding the numerator of the equation, the values of $\theta + \delta$ and $\alpha + \delta$ range from 0° to 180° . Nevertheless, it is crucial to emphasize that $\theta + \delta$ cannot be equal to 90° , as this would lead to a singularity in the calculation of the stability equation S_q . The angle values are calculated by substituting each of the four slope models, and the corresponding results are presented in **Table 4**. It is observed that the slope with uneven bench width exhibits the smallest S_q , indicating the lowest stability and highest potential for failure along the structural plane. Contrarily, with the increase of bench width, the S_q value of the slope with uniform bench width grows, and the stability is progressively enhanced. This law is in accordance with the damage observed in the four types of slopes in the shaking table model tests.

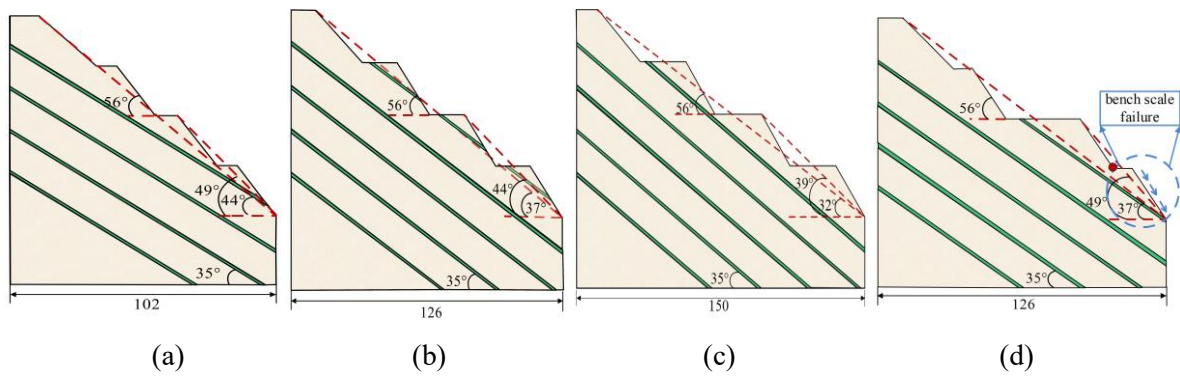


Figure 14 The angle schematics of the four slope models in the shaking table tests. (a) angle diagram of slope with 2m uniform bench width; (b) angle diagram of slope with 4m uniform bench width; (c) angle diagram of slope with 6m uniform bench width; (d) angle diagram of slope with uneven bench width.

Table 4 The S_q value results of four slopes in shaking table test.

Slope type	$\tan\alpha$	$\tan\beta$	$\tan\theta$	δ	$S_q = \sqrt{\frac{ \cos(\theta + \delta) \sin(\alpha + \delta) }{\cos(\beta - \alpha) \sin(\theta - \alpha)}}$
2 2 2	1.028	1.429	1.134	35	1.511
2 8 2	0.803	1.429	1.134	35	0.808
4 4 4	0.803	1.429	0.940	35	1.617
6 6 6	0.659	1.429	0.803	35	1.726

4.3 Plastic zone of the bench rock slope

FLAC 3D is a three-dimensional numerical simulation software that is widely used for analyzing the seismic stability of slopes. The software is capable of constructing three-dimensional geological models, accommodating various material behaviors (such as elastic, elastoplastic, and plastic models), simulating seismic loads, and analyzing the impact of varying slope landforms. It also facilitates the visualization and interpretation of stress and displacement data, allowing for precise assessments of slope stability under seismic conditions and enabling detailed sensitivity analyses through parameter adjustments (Alsardi et al., 2023; Lu et al., 2022; Song et al., 2024).

To explore how the position of wider platforms influences slope failure modes, the single-factor analysis method was employed in this section. A numerical simulation model was established using finite difference software *FLAC 3D* based on the actual engineering slope dimensions, as shown in **Figure 15**. Generally, in order to accurately represent the wave propagation characteristics, the maximum mesh size in the numerical model should be smaller than $1/10 \sim 1/8$ of the wavelength of the input seismic motion in dynamic numerical simulations, expressed as: $\Delta l \leq (1/10 \sim 1/8)\lambda$. Herein, l represents the mesh size of the numerical model; λ refers to the minimum wavelength of seismic motion. Thus, the maximum mesh size of the elements was set to 0.8 m in this series of numerical simulations. Additionally, the dynamic boundary condition was another crucial issue in dynamic numerical simulation. The seismic wave would reflect along the boundary of the numerical model, subsequently influencing the accuracy of wave propagation. To mitigate this issue, a free field was implemented to minimize the wave reflection in the dynamic numerical simulation. Furthermore, the local damping with a value of 0.157 was adopted in this study. The loading method employed in the dynamic numerical simulation mirrored that of the shaking table test. The model was subjected to the Wulong seismic wave as a horizontal seismic input, and dynamic response simulation was conducted to supplement and compare with shaking table test results. Specifically, four numerical models were developed to investigate the relationship between the position of wider platforms on the slope and the slope failure modes. Two specific types of slopes were analyzed: the slope with 2 m uniform bench width and uneven bench width. The results of these numerical simulations were compared with findings from shaking table tests, showing remarkable consistency in slope failure modes, characterized by tension failure at the inner corner of the bench and shear failure along the structural plane. Additionally, two supplementary models with wider platforms placed at the first bench and third bench were constructed to explore the influence of wider platform position on slope failure mechanisms and stability. These additional simulations served both as an extension of the shaking table test results and as a confirmation of the stability equation S_g .

The distribution of plastic zones of the slope under seismic excitation is shown in **Figure 16**, where “shear” refers that the yield shear stress of the rock material exceeds the shear strength, and shear failure occurs in the rock mass. On the other hand, “tension” denotes that the tensile stress of the rock material

exceeds the yield tensile strength. The suffix “n” indicates that the rock material is undergoing failure, the suffix “p” means that failure has occurred, and “None” indicates that the rock is in an elastic state. It is evident that, compared to other slopes, the slope with 2m uniform bench width experienced minor damage, primarily in the form of shear damage at the interlayer and tension damage at the rear edge of the slope and the internal corner of the bench (as illustrated in **Figure 16(a)**). Notably, no sliding surface was observed extending from the slope surface to the internal interlayer. Conversely, the wider platform located at the second bench of the slope, showed the initiation of a tensile plastic zone at the internal corner of the second bench and the rear edge of the slope when subjected to seismic waves. The resulting damage propagated inward and connected to the structural plane in the slope. The slope was susceptible to experiencing shear sliding along the interlayer, as illustrated in **Figure 16 (b)**. In the case of a slope with a wider platform on the first bench (see **Figure 16(c)**), the shear plastic zone initially appeared on the structural plane when the loading time was 10s (see **Figure 16(e)**), due to its inherently low shear strength. Furthermore, at the lower part of the slope, the tensile stress exceeded the rock’s tensile strength on both sides of the interlayer, leading to the formation of a tensile plastic zone. By $t = 24$ s, this zone expanded downward on both sides and appeared at the internal corner of the first bench (as depicted in **Figure 16(f)**). Concurrently, a tensile plastic zone was developed on the rear edge of the slope, leading to a downward sliding tendency along the structural plane. At $t = 36$ s, the tensile plastic zone connected with the interlayer, which led to a high susceptibility of the first bench to slide away from the slope along the interlayer. In the case of the wider platform located on the third bench of the slope, the damage to the slope is shown in **Figure 16(h)** after the seismic wave loading was completed. Extensive tensile plastic damage to the rock body was noted at the rear edge of the slope and both sides of the interlayer with higher elevation, resulting in a tendency for the slope to slide down along the structural plane. Nevertheless, the overall stability of the slope remains maintained.

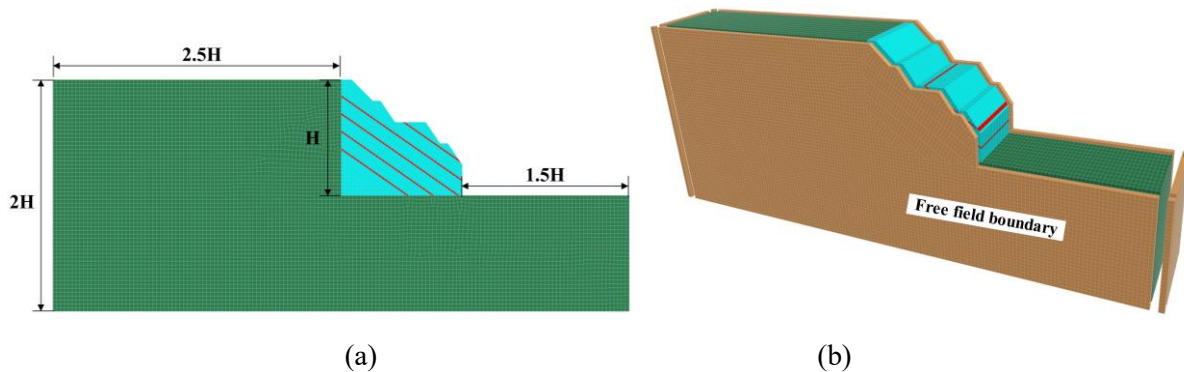
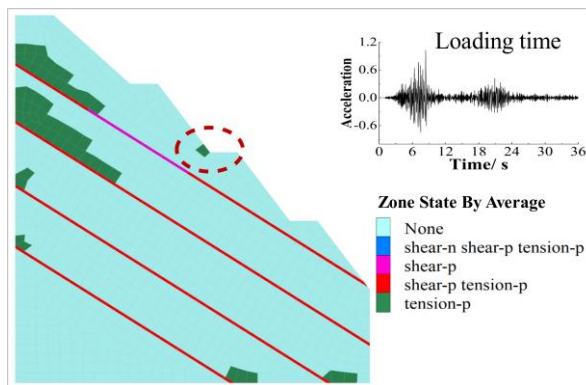
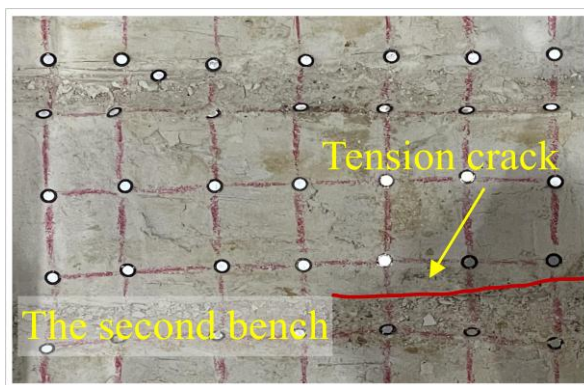


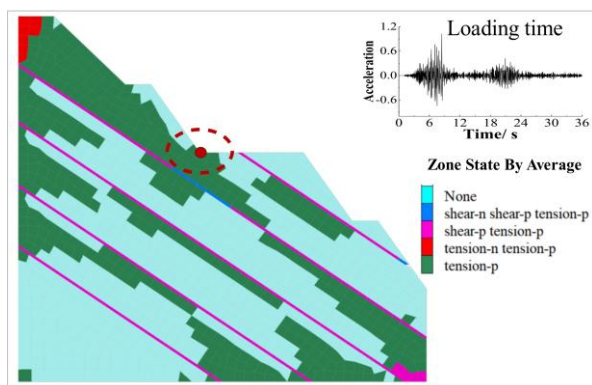
Figure 15 Numerical models and boundary conditions. (a) the slope model geometry in numerical simulation; (b) the free field boundary generation in numerical simulation.



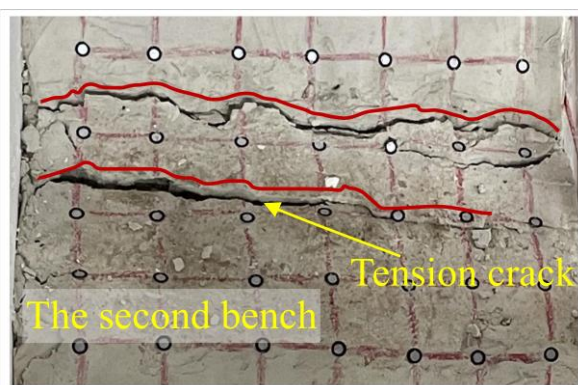
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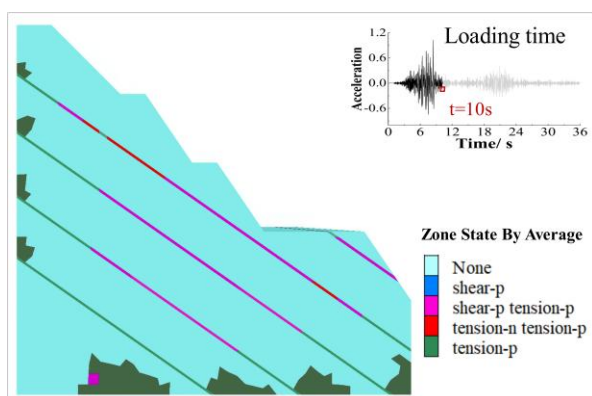
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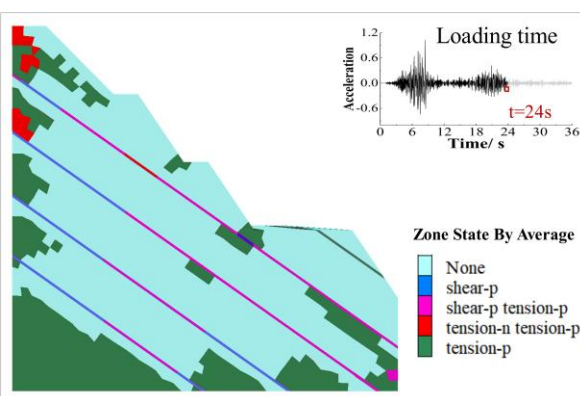
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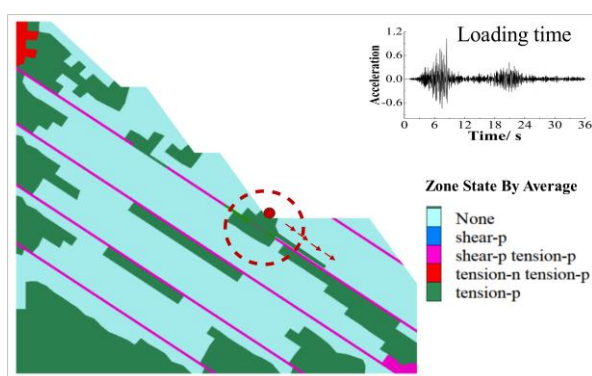
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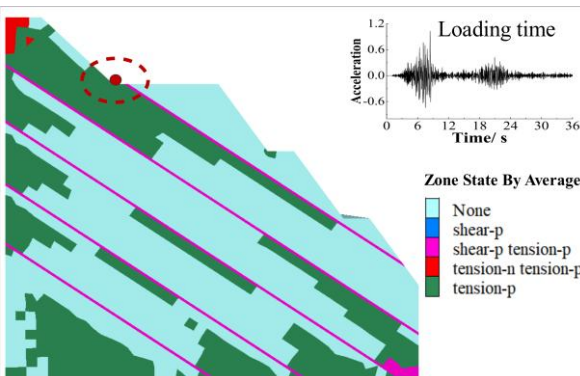
(e)



(f)



(g)



(h)

Figure 16 The distribution of plastic zones of the slope and the failure phenomena under seismic excitations. (a) the distribution of plastic zone in the slope with 2m uniform bench width; (b) failure phenomena of slope with 2m uniform bench width after excitations; (c) the distribution of plastic zone in the slope model of the wider platform on the second bench; (d) failure phenomena of slope with uneven bench width after excitations; (e) the distribution of plastic zone in the slope of the wider platform on the first bench at 10 s of loading; (f) the distribution of plastic zone in the slope of the wider platform on the first bench at 24 s of loading; (g) the distribution of plastic zone in the slope of the wider platform on the first bench; (h) the distribution of plastic zone in the slope of the wider platform on the third bench.

5. Discussion

5.1 Slope deformation damage process

Shaking table tests are a crucial tool in earthquake engineering, enabling precise simulation of seismic parameters (e.g., acceleration, frequency, and duration) and revealing structural dynamic responses under repeatable loading conditions (Chen et al., 2023; Yang et al., 2022). Unlike numerical simulations, these tests allow direct observation of structural damage, providing empirical validation for theoretical models. However, they have two main limitations: size constraints, as large or complex structures require scaled models that may introduce size effects or boundary distortion, and discrepancies in simulating seismic motion, as real earthquake waves exhibit spatial non-uniformity and multi-dimensional coupling, which are challenging to fully replicate, especially with respect to randomness and local geological effects.

Despite these challenges, the shaking table used in this study (6.0 m × 4.0 m) offers a favorable specimen-to-table mass ratio, mitigating boundary distortion from scaled model effects. Displacement and acceleration distortions in the X, Y, and Z directions remain within acceptable limits. Therefore, the shaking table meets high precision standards, and the data obtained are reliable and accurate.

The cumulative failure of the slope during seismic excitations can be divided into four distinct developmental stages, as illustrated in **Figure 17**. These stages include shallow creep (0.1g-0.2g), local tension (0.2g-0.6g), accelerated deformation (0.6g-0.8g), and overall instability (0.8g-1.0g). The model test results show that the deformation damage mode is generally expressed as a slip-tension type.

The shear fault momentum in rock mass results from the accumulation of fault momentum in a specific direction under the influence of multi-stage earthquakes (Liao et al., 2015; Tai et al., 2024). This cumulative damage, similar to material fatigue damage, is a function of earthquake duration and magnitude, and it increases with both parameters. This analysis is consistent with the regional phenomenon of opening joints observed on the deep surface during the investigation of slope damage caused by the Wenchuan earthquake. In actual slope engineering, attention should be paid to the “dynamic” stability evaluation of the slope. Here, the term “dynamic” encompasses several aspects: the

slope deformation develops dynamically over time, and its stability must be continuously tracked and monitored. Moreover, the stability evaluations require constant revision of the calculated conditions and parameters. Therefore, the evaluation methods and means are also undergoing dynamic development. Consequently, the dynamic evaluation of slope is a continuous cycle process of theoretical calculation and field monitoring “interaction”.

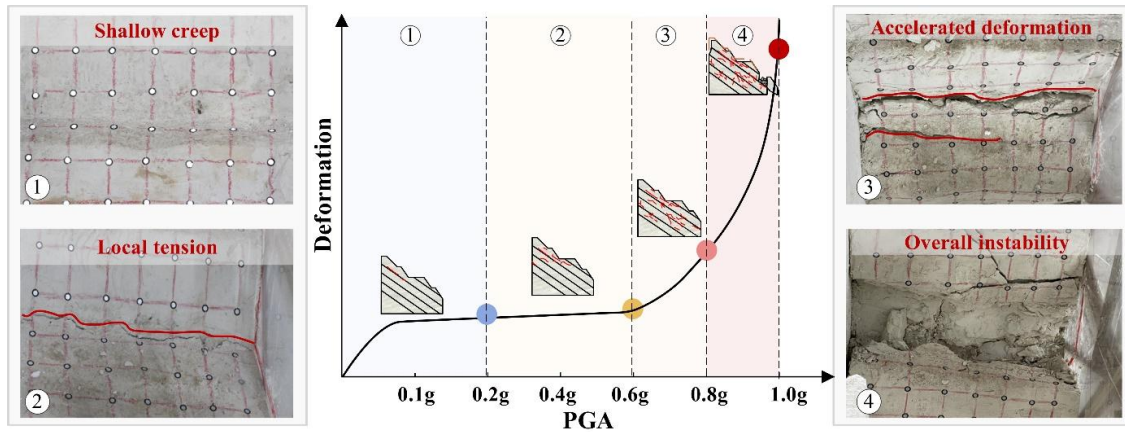


Figure 17 The cumulative failure process of slopes under seismic excitations.

5.2 Identification of the slope stability

To examine the impact of varying widths of wider platforms (i.e., different b values) and their positions on slope stability, we developed several slope models with uneven bench widths (where $a/b \neq 1$). The parameters α , β , θ , and δ were extracted from these models, and the S_q values for each slope model were calculated and are presented in **Table 5**. Notably, when $b/a = 2$ and the wider platform is placed on the third bench, the S_q value is 1.266; when $b/a = 3$ and the platform is on the third bench, the S_q value is 1.003. As the b/a ratio increases, the S_q value decreases, indicating a reduction in slope stability. The S_q distribution for the 8 2 2, 2 8 2, and 2 2 8 slopes follows the trend observed in numerical simulations. This confirms that the proposed S_q stability equation is consistent with both model test results and numerical simulations, demonstrating its strong applicability to benched bedding rock slopes. Additionally, as shown in **Table 5**, regardless of platform width, if the platform is positioned on the first bench, the S_q value remains around 0.2, indicating extremely poor stability. Consequently, these slopes are highly prone to bench-scale damage at the internal corner of the first bench. Therefore, reinforcement of these slopes, particularly at the internal corner of the wider bench, is essential to prevent bench-scale failure and maintain slope stability.

Table 5 The S_q of different slope models.

Slope type	b/a	α	β	θ	δ	$\theta-\alpha$	$\beta-\alpha$	$\theta+\delta$	$\alpha+\delta$	$S_q = \sqrt{\frac{ \cos(\theta+\delta)\sin(\alpha+\delta) }{\cos(\beta-\alpha)\sin(\theta-\alpha)}}$
4 2 2	2	42	56	47	35	5	14	82	77	1.266
2 4 2	2	42	56	49	35	7	14	84	77	0.928
2 2 4	2	42	56	56	35	14	14	91	77	0.269
6 2 2	3	39	56	47	35	8	17	82	74	1.003
2 6 2	3	39	56	49	35	10	17	84	74	0.778
2 2 6	3	39	56	56	35	17	17	91	74	0.245
8 2 2	4	37	56	47	35	10	19	82	72	0.898
2 8 2	4	37	56	49	35	12	19	84	72	0.808
2 2 8	4	37	56	56	35	19	19	91	72	0.232
10 2 2	5	36	56	47	35	11	20	82	71	0.856
2 10 2	5	36	56	49	35	13	20	84	71	0.684
2 2 10	5	36	56	56	35	20	20	91	71	0.227
12 2 2	6	34	56	47	35	13	22	82	69	0.789
2 12 2	6	34	56	49	35	15	22	84	69	0.638
2 2 12	6	34	56	56	35	22	22	91	69	0.217

Note: Slope type column “4 2 2” in the table indicates that the widths of the third, second and first benches of the actual three-bench slope are 4 m, 2 m and 2 m respectively.

The natural frequency and damping ratio are critical dynamic parameters of the slope model, which can be derived from the acceleration time history curves at various monitoring points. When the slope is subjected to seismic excitations, the slope model will generate stress and strain during the vibration process (Wu et al., 2024; Zhang et al., 2025; Zhao et al., 2023). These forces cause deformation and damage to the slope’s internal structure, leading to slope deterioration. Changes in the natural frequency and damping ratio indicate the extent of internal damage to the slope model. By analyzing the acceleration time history curves, the slope’s dynamic parameters can be calculated. These parameters quantify the slope’s vibration characteristics, and by comparing them with those of an undamaged slope, the degree of damage and deformation can be assessed, providing insight into the progression of slope

degradation. As damage accumulates, the natural frequency decreases due to a reduction in stiffness and damping capacity caused by slope deformation and failure.

According to the loading scheme in the shaking table test, a 0.05 g white noise scan lasting 60 s is applied before each seismic excitation (as shown in **Figure 18(a)**). Each slope model undergoes seven white noise scans, labeled sequentially as WN-1, WN-2, ..., WN-7. In this study, the absolute transfer function at each monitoring point is determined using the tfestimate function in the MATLAB toolbox. The equation for tfestimate in MATLAB is as follows:

$$T_{xy}(f) = \frac{P_{yx}(f)}{P_{xx}(f)} \quad (14)$$

where, $T_{xy}(f)$ represents the transfer function between the input signal x (acceleration at the table measurement point) and the output signal y (acceleration at the slope measurement point); $P_{yx}(f)$ denotes the cross-correlation spectral density function between signals x and y ; $P_{xx}(f)$ is the autocorrelation spectral density function of signal x , and f is the signal frequency.

This series of shaking table tests includes four slope models. Before seismic excitation, each slope model undergoes a white noise scan, and the transfer function of the slope is plotted using Matlab. Taking the slope with uneven bench width as an example, the transfer function curve is shown in **Figure 18(b)**. The frequencies corresponding to the peak points of the transfer function curve can be approximated as the natural frequencies of the slope model.

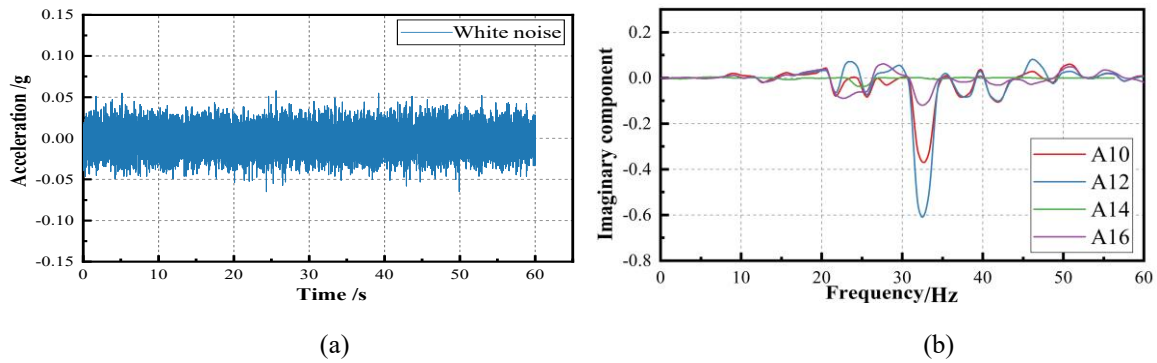


Figure 18 White noise time history curve and slope model under WN-1 condition of the transfer function curve of each measuring point. (a) white noise time history curve; (b) transfer function curve of each measuring point.

The transfer function of each slope model before each seismic excitation in the shaking table tests is calculated using the tfestimate function in MATLAB. The first-order natural frequencies of the four slope models under each white noise condition are statistically analyzed, as shown in **Table 6**. The natural frequencies of all slope models decreased with increasing excitation amplitudes. In the shaking table tests, cumulative damage to the slopes increased under multiple seismic excitations, with some slopes showing significant damage and instability. For the slope with 2 m uniform bench width, the first-order natural frequency decreased from 29.98 Hz under WN-4 to 26.79 Hz under WN-5, a reduction of

10.64%. For the slope with 4 m uniform bench width, the frequency decreased from 30.45 Hz under WN-4 to 28.64 Hz under WN-5, a 5.94% reduction. The slope with 6 m uniform bench width showed a decrease from 30.78 Hz under WN-4 to 28.94 Hz under WN-5, a 5.98% reduction. For the slope with uneven bench width, the frequency decreased from 28.76 Hz under WN-4 to 26.13 Hz under WN-5, a 9.14% reduction.

Minimal changes in the first-order natural frequencies were observed between WN-1 and WN-3. However, the largest decrease occurred between WN-4 and WN-5, with some slopes experiencing a sudden drop in frequency between these conditions. WN-4 and WN-5 correspond to excitation amplitudes of 0.4 g and 0.6 g for the slope, respectively. The results from the shaking table tests indicate that after experiencing these two higher excitation amplitudes, the slope exhibited severe damage. When subjected to lower excitation amplitudes, the first-order natural frequencies showed little change, as the slope could withstand smaller stresses, maintaining stable dynamic characteristics. However, at higher excitation amplitudes, the increased stresses led to greater damage and deformation, resulting in a significant decrease in the first-order natural frequency. This suggests that abrupt changes in natural frequency may serve as indicators of severe damage, deformation, or instability in the slope models.

The *Technical code for building slope engineering* (GB50011-2010) in China outlines guidelines for slope safety classification. Section 3.2 specifies that classification should consider factors such as potential damage severity, slope type, and height, categorizing slopes into three safety levels: Class I, Class II, and Class III. Section 5.3 outlines the slope stability evaluation criteria, with Subsection 5.3.2 defining the slope stability safety factor (F_{st}). For slopes under “normal conditions”, the F_{st} for Class I slopes is 1.35, while under “seismic conditions”, the F_{st} is 1.15. For Class II slopes, the F_{st} under “normal conditions” is 1.30, and under “seismic conditions”, it is 1.10. For Class III slopes, the F_{st} under “normal conditions” is 1.25, and under “seismic conditions”, it is 1.05. This standard suggests that the stability safety factor is reduced under seismic conditions, reflecting a reduction coefficient applied to the “normal conditions” safety factor. This concept is incorporated into the stability equation presented in this paper. As shown by the transfer function analysis, slope failure is a dynamic process involving cumulative damage, necessitating the inclusion of a dynamic correction coefficient, ζ , which accounts for slope displacement and crack propagation rate. Larger displacements and faster crack propagation negatively impact stability.

Table 6 First order natural vibration frequency of slope model.

Slope type	White noise condition	first-order natural vibration frequency	Slope type	White noise condition	first-order natural vibration frequency
2 2 2	WN-1	31.89	4 4 4	WN-1	32.22
	WN-2	31.66		WN-2	32.14
	WN-3	30.99		WN-3	31.89

6 6 6	WN-4	29.98	2 8 2	WN-4	30.45
	WN-5	26.79		WN-5	28.64
	WN-6	26.76		WN-6	27.98
	WN-7	26.53		WN-7	27.75
	WN-1	32.45		WN-1	32.03
	WN-2	32.24		WN-2	31.89
	WN-3	31.99		WN-3	30.23
	WN-4	30.78		WN-4	28.76
	WN-5	28.94		WN-5	26.13
	WN-6	28.37		WN-6	25.99
	WN-7	28.03		WN-7	25.74

Based on the failure processes observed during shaking table tests, and using displacement sensors, the displacement of slopes during seismic excitation was measured to determine the dynamic correction coefficient for each slope. The equation for evaluating dynamic stability, incorporating the dynamic correction coefficient, is proposed, with ζ values provided in **Table 7**.

Table 7 The S_{qt} of slope model considering dynamic coefficient.

Slope type	$S_q = \sqrt{\frac{ \cos(\theta + \delta)\sin(\alpha + \delta) }{\cos(\beta - \alpha)\sin(\theta - \alpha)}}$	dynamic coefficient ζ	$S_{qt} = \zeta \sqrt{\frac{ \cos(\theta + \delta)\sin(\alpha + \delta) }{\cos(\beta - \alpha)\sin(\theta - \alpha)}}$
2 2 2	1.511	0.9	1.360
4 4 4	1.617	1	1.617
6 6 6	1.726	1	1.726
2 8 2	0.808	0.7	0.566

The *Code for investigation of geotechnical engineering* (GB50021-2001) of China specifies that for newly designed slopes, the stability coefficient K should range from 1.30 to 1.50 for important projects, 1.15 to 1.30 for general projects, and 1.05 to 1.15 for minor projects. Similarly, the *Technical code for building slope engineering* (GB50330-2013) of China stipulates that for slope engineering stability verification, the stability coefficient should be no less than the safety factor requirement K_s (ranging from 1.20 to 1.35) to avoid necessitating slope reinforcement. In Queensland, Australia, *Geotechnical Design Standard—Minimum Requirements* states that stability analysis for geotechnical design of a cutting slope must adhere to a minimum factor of safety (FOS) of 1.30 during construction (short-term). This study classifies slope stability based on the dynamic stability equation, S_{qt} (as shown in **Figure 19**). The dynamic stability is considered basically stable if S_{qt} falls within the range of 1.3 to

1.50, with potential for localized damage. When S_{qt} exceeds 1.5, the slope is deemed fully stable. A S_{qt} value between 1.0 and 1.3 indicates less stable, while a value below 1.0 suggests very poor stability and a high likelihood of bench-scale failure. Consequently, the dynamic stability S_{qt} of slopes should not be lower than 1.0. The dynamic stability equation provides geotechnical engineers and rock engineers with a practical means to assess the dynamic stability of multi-bench rock slopes on-site. It also allows for potential adjustments to the slope angle as the rock mass conditions evolve during slope excavation. This approach represents a convenient and effective method for evaluating slope stability.

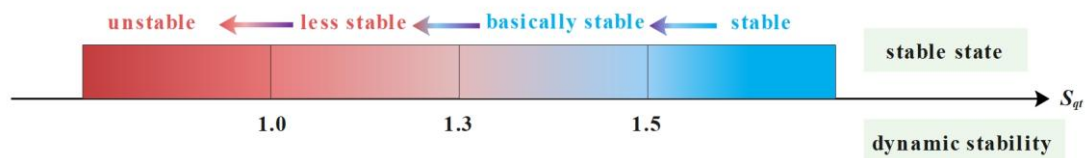


Figure 19 Slope dynamic stability classification.

6. Conclusion

The seismic response of benched bedding rock slopes was investigated in this study through shaking table tests and complimentary numerical simulations. Four types of slopes with diverse bench widths were examined, utilizing real-time test data to analyze the distinct damage phenomena under seismic excitation, acceleration distribution laws, and earth pressure responses across different platform positions. Moreover, a dynamic stability equation S_{qt} was proposed as an index for assessing slope stability. The key findings are summarized as follows:

(1) The test outcomes show that the AACs at the internal corners of benches are lower compared to those at the external corners. This phenomenon is attributed to the considerable stress concentration induced by the variations of slope gradients, where the internal corner of the bench serves as the stress concentration zone. Additionally, the external surfaces of the slope, being less constrained, are more susceptible to seismic excitations. Hence, mitigating steep changes in slope topography is a primary focus in actual slope projects. Implementing rounded internal corners helps alleviate stress concentration and reduces the likelihood of potential damage.

(2) The benched bedding rock slopes experience shattering damage due to the combined influence of dynamic earth pressure and residual earth pressure. While dynamic earth pressure dissipates after the excitation ends, residual earth pressure persists within the slope, leading to further deformation until noticeable cracks emerge. Consequently, the progressive growth of earthquake-induced residual earth pressure should not be overlooked, and real-time monitoring of earth pressure response is vital for slope engineering, especially at the internal corners.

(3) The stability of the slope increases with the widening of the slope platform, as evidenced by the reduction in failure phenomena compared to slopes with uniform bench widths of 2 m, 4 m, and 6 m. However, the slope with uneven bench width exhibits more severe failure phenomena. Therefore, slopes

with uniform bench widths generally demonstrate better seismic stability than those with uneven bench widths.

(4) The numerical simulation results show that slopes with wider platforms at lower elevations (e.g., the first bench) exhibit a higher density of shear plastic zones compared to those with platforms at higher elevations (e.g., the third bench), making the slope more prone to sliding along the structural planes. Additionally, when the wider platforms are located at lower elevations of the slope, the slope is more likely to experience bench-scale failure.

(5) The S_{qt} effectively measures the dynamic stability of the slope. The calculated results are consistent with the damage law observed in shaking table model tests and align well with numerical simulations, demonstrating the high applicability of the equation for benched bedding rock slopes with varying geological structures and landform patterns.

Declaration of Competing Interest

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

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874 **List of symbols/notations or Abbreviations and symbols**

d	Displacement distortion
D_1	Fundamental amplitude
D_i	Harmonic amplitude
L	Geometric dimension
a	Acceleration
ρ	Density
E	Elastic modulus
μ	Poisson's ratio
c	Cohesion force
φ	Internal friction angle
σ	Stress
ε	Strain
t	Time
d	Displacement
v	Velocity
ζ	Damping ratio
S_q	Stability
S_{qt}	Dynamic stability
α	Overall slope angle
β	Bench angle
θ	Inter-ramp angle
δ	Interlayer angle
AAC	Acceleration Amplification Coefficient
EAE	Elevation Amplification Effects

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