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Abstract To investigate the seismic dynamic characteristics of the accumulation slope, a slope of the Zheduo Mountain tunnel along the Sichuan-Tibet Railway in China was selected as the prototype, based on dimensional analysis and similarity principle, two groups of model tests were carried out at 50° accumulation slope and 60° accumulation slope to obtain the dynamic response influenced by different amplitude of seismic wave. A transfer function analysis method suitable for shaking table test is proposed. Based on the data pretreatment method of eliminating trend terms and digital filter, the frequency response function was calculated by method of average periodic chart. And the variation of frequency response function was analyzed by Pearson correlation coefficient. At last, the least square iteration method was used for modal analysis. It is found that the transfer function changes obviously when both the slopes are destroyed, the weak interlayer has a significant influence on seismic wave transmission. The modal analysis results show that with the increase of the excitation intensity, the natural frequency decreases and the damping ratio increases.

Key Word Earthquake, Slope, Sichuan-Tibet Railway, Modal analysis, Vibration Characteristics

Introduction

The high hill-river valleys in the southwest mountainous area of China are characterized by strenuous cutting, fractured rock mass, bad stability and even loose accumulation bodies are widely distributed. Under the impact of the collision between the Indian plate and the Eurasian plates, active fractures are densely distributed. There are frequent earthquakes and high intensity (Kumar et al., 2021), which are prone to induce large-scale landslides. When the slope
collapses, the sliding mass has strong kinetic energy and high speed movement (Shinoda et al., 2017), which can quickly destroy the ground infrastructure (Singh et al., 2016).

The characteristics of earthquake seismic response are one of the research hotspots in recent years. At present, scholars mainly studied from theoretical analysis, numerical simulation, model test, field observation and other means, which have made great achievements in the site parameters and ground motion characteristics of slope. The existing theoretical analysis methods for slope ground motion are mainly represented by limit analysis method (Michalowski, 2009; Michalowski et al., 2018), discontinuous deformation analysis (Beyabanaki et al., 2016; Yu et al., 2019) and pseudo-static method, calculate the safety factor of slope, critical state of rupture and influencing factors of homogeneous soil; and numerical simulation method are represented by finite difference (Liu et al., 2004), finite element (Yin et al., 2019), discrete element (Valentino et al., 2008), etc. The result is superior to the calculation of the dynamic state of the slope and the related influencing factors (Assimaki et al., 2005; Paolucci, 2002; Pagliaroli et al., 2014). But both the theoretical method and the numerical simulation method have some shortcomings on the heterogeneous slope. Such as the boundary effect of heterogeneous slope is difficult to consider, and the convergence problem of numerical analysis, Trombetti et al (2002) analyzed the theoretical calculation results of structure vibration through the linear system, and confirmed the reliability of the shaking table test technology. Based on the above reasons, this paper chooses the shaking table test as the research method.

In the Wenchuan earthquake that occurred in China, a large number of slope landslide damage occurred. As is shown in Fig.1, the slope angles of these slopes are different, resulting
in different forms of slope damage. As a typical type of slope with weak interlayer widely
distributed, the failure of the accumulation slope is often due to the low elastic modulus and
strength of the internal structure. Fan et al (2016) studied the shaking table of bedding and anti-
dip weak interlayer, in which the weak interlayer presented a low gradient with the horizontal
surface, and analyzed the acceleration and spectral response results.

Transfer function is one of the common methods used to analyze structural vibration signals,
and some scholars have applied this method in engineering seismic response (Kojima et al.,
2015; Komoda et al., 2015; Sheng et al., 2012; Cui et al., 2012; Fan et al., 2017). Abundant
achievements have been made in the researches, but it can be found that the research on transfer
function of slope is not sufficient, and the calculation method of the transfer function is
relatively simplified, also the relationship between multi-mode modes and the influence of
noise is often ignored.

Taking the actual slope along the Sichuan-Tibet Railway as a prototype, based on 1:10
similar ratio for model and the prototype, two groups of the different gradient model slope were
simulated earthquake by excitation signal. On the basis of signal processing, using the method
of average periodic chart (Welch method) to calculate the acceleration frequency response
function of the surface and inside of model slope. Using Pearson correlation coefficient to
analyze the changes of the frequency response function after the different gradient of slope
with the increase input of amplitude of earthquake ground motion. Using the least square
iteration method to do the modal analysis, identifying the ground motion response
characteristics of the slope. The research results are beneficial to the identification of seismic
failure process and seismic study of the accumulation slope, and it can be extended to the
Overview of shaking table test

In this paper, a slope of the Zheduo Mountain tunnel along the Sichuan-Tibet Railway in China is selected as the prototype, its slope is about 60 degrees, in order to compare, a group of 50° slope was used for the control experiment.

Zheduo Mountain is located in Ganzi Prefecture, Sichuan Province. The altitude of the highest mountain is 4962 meters above sea level, and the notch is 4298 meters above sea level. The altitude difference between the Zheduo Mountain and Kangding City is 1,800 meters, which is the first high mountain notch that needs to be passed on the Sichuan-Tibet Railway. Therefore, it is known as the "First Pass of Kangba". The photo of Zheduo Mountain is shown in Fig.2. In terms of terrain, the region boundary is roughly on the west side of the pass from Lanniba, Yulongxi to Zheduo Mountain. It was divided into two parts: mountains and extreme mountains to the east, and mountain plains and western plateau canyons to the west. The mountains and extreme mountains on the eastern slope of the Daxue Mountain are high and with deep Dadu River, broken surface, steep valley slope, and the relative elevation difference of the valley is 1000m ~ 3000m. The western slope of the Daxue Mountain plateau, the elevation is about 4500m, the valley cut is shallow, while the top surface is open and smooth with lush water and grass. Zheduo Mountain is a vein of Daxue Mountain, which is an important geographical boundary. The west of the plateau uplift with Yalong River and the east of the alpine valley with Dadu River. The peak of the Zheduo Mountain is 4,962 meters above sea level, and the pass is 4,298 meters above sea level, with an altitude drop of 1,800 meters.
from Kangding City. Due to the strong cutting of Minjiang River and Dadu River, the geographical landform of the Zhедuo Mountain Line is characterized by large elevation difference, dense ravines, crossing mountains, dense trees and warm and moist climate, with typical characteristics of subtropical warm and humid valley.

This paper is based on a typical accumulation slope at the entrance of Zheduо Mountain tunnel. According to the data recorded by China Earthquake Networks, since 2012, this area has been affected by approximately 204 earthquakes of $2.0 \leq M_S \leq 8.0$ (see Fig.3), most of them are small earthquakes. A large number of frequent small earthquakes will cause a large amount of damage and damage to the inside of the slope, providing conditions for large-scale collapse in the later period, and thus seriously affecting the construction safety of the Zheduо Mountain Tunnel. Therefore, it is urgent to research the dynamic feature of the slope under earthquake to provide technical support for slope disaster prevention and control along the Sichuan-Tibet Railway.

**Similar system**

The model similarity system adopted in this paper is implemented by the similar three theorems (Yuan, 1998). It is believed that the parametric relations of the implementation phenomenon can be converted into a function of the similarity criterion under the condition that the similarity phenomenon satisfies. It is the same as the function of the similarity phenomenon, also called $\pi$-theorem (Curtis et al., 1998), that is namely Equation (1) and Equation (2):

$$f(a_1, a_2, \ldots, a_k, a_{k+1}, a_{k+2}, \ldots, a_n)=0$$  \hspace{1cm} (1)

$$F(\pi_1, \pi_2, \ldots, \pi_{n-k})=0$$  \hspace{1cm} (2)
In equation (1), \( a_1, a_2, ... , a_k \) are the elementary quantity, and \( a_{k+1}, a_{k+2}, ... , a_n \) are the derived quantity. In equation (2), \( \pi_1, \pi_2, ... , \pi_{n-k} \) Dimensionless constant. The matrix method can be used to derive the similarity criterion of the physical model, is shown in Table 1. Among them, Geometric scale is \( L \); gravitational acceleration is \( g \); cohesion is \( c \); dynamic elastic modulus is \( E \); internal friction angle is \( \varphi \); dynamic Poisson's ratio is \( \mu \); gravity is \( \gamma \); shear wave velocity is \( v_s \); input acceleration is \( A \); duration is \( T_d \); frequency is \( \omega \); angular displacement is \( \theta \); linear displacement is \( s \); response speed is \( V \); response acceleration is \( a \); stress is \( \sigma \); strain is \( \varepsilon \).

In the case that phenomena and similar phenomena can be expressed as same functions. Those which can be regarded as similar if they have similar individual parameters and equal similarity criteria. According to the above criteria, the similarity relationship of the main physical quantities in the model adopted in this paper, as is shown in Table 2.

**Test device and model**

This experiment adopted large one-way seismic simulation shaking table to simulate, the model box floor and skeleton material is mainly composed of steel plate, gradient steel, iron channel, to facilitate the observation and record in the test process of the model of slope deformation and failure phenomena, and the model box is visualized with 12mm optical plastics on both sides. Apply vaseline evenly between optical plastics and model to reduce friction. Using a 10 cm thick foamed polyethylene to reduce boundary effect between model and boundary. The bottom board of the model box is adhesive stone columns with epoxy resin to make it a rough surface to reduce the relative displacement between the model box and the model contact surface.
To satisfy the needs of the experiment, taking the slope relying on engineering drilling data as the background, this paper adopted two kinds of gradients natural slope model, 50° and 60° respectively. And the model of total 1.5 m high, 1.2 m high of side slope. To adapt the characteristics of side slope, the side slope model is 2.21 m at the bottom, the vertical section of the model is consistent with a width of 2m, the bedrock is 0.25m high. The weak interlayer is covered on the bedrock with a thickness of 5 cm, and the accumulation mass is covered on the weak interlayer. The basic physical parameters of the test model material can be obtained through laboratory tests, as is shown in Table 3.

The test model was made on site. In the process of filling the model, it was filled layer by layer and bottom-up, with one density test for each fill of 20 cm. And compaction was carried out by artificial vibration, that is, the central part and boundary of the model were compacted by artificial compaction to ensure the compaction quality, as shown in Fig.4. The overall model is shown in Fig.5, the gradient is the angle between the overlying accumulation body and the level surface. The layout of the measuring point adopted in this paper is shown in Fig.6, as it shows, it mainly divided into two categories, the acceleration sensor distribution in free field, central slope surface, the interface between bedrock and weak interlayer, the interface from weak interlayer to accumulation body as well as the free field. And displacement sensors are mainly distributed at the top, middle and bottom slope surface, sensor sampling frequency is 1000 Hz.

**Loading cases and measuring points layout**

The white noise and seismic wave were simulated by inputting acceleration time histories in shaking table, and the loading direction was horizontal along the slope surface. In this paper,
the input for the two groups of shaking table tests includes the following working conditions, as shown in Table 4. In the table, white noise comes from measured data, as shown as Fig.7, the first loading of white noise was mainly to reduce the random disturber and transient effect. After loading was to scan the slope as a whole by loading white noise while reducing the transient effect, so as to analyze the structural characteristics by using the transfer function method, the seismic wave contains: Wenchuan wave, Kobe wave and EL Centro wave. The amplitude normalization of the earthquake waveform loaded in this paper is shown in Fig.8–10.

Slope failure phenomenon

In the seismic wave loading process of the two groups of shaking table tests, the HD camera system was used to shoot the phenomenon. The test results showed that the 50° slope had obvious failure when the input Wenchuan wave peak acceleration was 0.6g, while the 60° slope had the relatively similar phenomenon when it was 0.5g. The test phenomenon was shown in Fig.11~12. The two groups of slope failure phenomena are similar, in which the debris flow occurred in the deposited mass when the slopes were destroyed. At the same time, the slope surface was partially deformed. The slope was greatly deformed and a large amount of debris flow was generated when the 60° slope is destroyed. The slope surface was deformed a lot, generating a lot of debris flow. Obvious back edge expansion crack appeared at the top of the accumulation mass of the two groups of slopes. According to the monitoring data of displacement sensors, as Fig.13 shows, the top displacement of the 60° slope is larger, and the bottom displacement of the 50° slope is larger, but its displacement change trend is similar. It means that slope failure mainly occurs along the weak interlayer plane in the overall collapse. In addition, the displacement monitoring data of the slopes shows that the slopes of the two
slopes have a "step-off" change curve, which is consistent with the two energy fluctuations in the Wenchuan earthquake waveform. According to the video and sensor monitoring data, the failure stages of the slope under both working conditions are as follows: first, the slope surface appeared cracks and deforms, the accumulation body squeezed the weak interlayer and develops downward, and cracks may appear between the accumulation mass and the weak interlayer. Subsequently, the movement of superstructure (accumulation mass) and substructure (weak interlayer and bedrock) were gradually inconsistent during the earthquake. Finally, the movement of the superstructure was greater than that of the substructure, and tensile cracks appeared at the back edge of the accumulation body (Moore et al., 2011), at the same time, the cracks in the interface gradually developed and broke through the locked section, and finally the overall failure occurred, as Fig.14 shows.

Calculation method of seismic transfer function of slope

Transfer function is the main methods to study control theory, which embodies the transformation relationship between input signal and output signal, and theoretically is irrelevant with input parameters. In structural dynamics, the transfer function can describe the motion law of the structure and analyze the dynamic characteristics and stability of the system.

Structural seismic frequency response function

In a single-degree-of-freedom linear structure, its non-time-dependent vibration motion equation is written as Equation (3):

\[ ax(t) + b\dot{x}(t) + c\ddot{x}(t) = f(t) \]  

(3)

Among them, \( x(t) \), \( \dot{x}(t) \) and \( \ddot{x}(t) \) are the displacement, speed and acceleration time
histories of the point particle respectively. And \(a, b\) and \(c\) are all constants and determined by the initial state of the object. The Laplace-transformation of Equation (3) is Equation (4):

\[
(s^2 + 2\xi \omega_0 s + \omega_0^2)X(s) = \frac{F(s)}{c}
\]

(4)

Where, \(s\) is a complex variable, above equations are equivalent to the Fourier transform when \(Re(s) = 0\), and \(\omega_0\) is the inherent circular frequency of the structure.

Let \(X(s)\) and \(F(s)\) be the displacement and displacement exerted on the structure respectively, then the Laplace-transforms of the two are Equations (5) and (6) respectively:

\[
X(s) = \int_0^{+\infty} x(t)e^{-st}dt
\]

(5)

\[
F(s) = \int_0^{+\infty} f(t)e^{-st}dt
\]

(6)

After dividing the above equations, the displacement transfer function of the structure can be obtained as Equation (7):

\[
H_d(s) = \frac{1}{c(s^2 + 2\xi \omega_0 s + \omega_0^2)}
\]

(7)

In the above Equation (7), \(\xi\) is the damping ratio. Similarly, velocity transfer function Equation (8) and acceleration transfer function Equation (9) can be obtained:

\[
H_v(s) = \frac{s}{c(s^2 + 2\xi \omega_0 s + \omega_0^2)}
\]

(8)

\[
H_a(s) = \frac{s^2}{c(s^2 + 2\xi \omega_0 s + \omega_0^2)}
\]

(9)

From the above formula, the transfer function is a complex-valued function, which is represented as a curved surface in the Laplace Domain. Therefore, frequency response analysis and root locus method are often used to describe the transfer function in practical applications, among which the former is the main approach. Due to space limitation, taking the displacement frequency response function as an example, Equation (7) can be written in the frequency domain as Equation (10):
Considering that the initial state of the structure is stationary under the action of earthquake, the real part is 0, and the imaginary part is angular frequency. The frequency domain response of the structure under the action of earthquake is shown in Equation (11):

\[ X(\omega) = \int_{-\infty}^{\infty} x(t) e^{-i\omega t} dt \]  

(11)

Where, \( \omega \) is the angular frequency variable, \( X(\omega) \) is the complex function of \( \omega \), \( x(t) \) is the vibration signal, \( i \) is the imaginary number. Taking frequency as independent variable, the real part of can be expressed as the real frequency curve of the signal, and the imaginary part of can be expressed as the virtual frequency curve of the signal. \( X(\omega) \) can be expressed in the module and vector form as Equation (12):

\[ X(\omega) = |X(\omega)| e^{-i\theta(\omega)} \]  

(12)

Taking frequency as an independent variable, \( |X(\omega)| \) can be expressed as the amplitude-frequency curve of the signal, and \( \theta(\omega) \) as the phase-frequency curve of the signal.

Substituting Equation (10) into Equation (12), its amplitude versus frequency and phase frequency are Equation (13) and Equation (14) respectively:

\[ |H_d(\omega)| = \frac{1}{c\sqrt{(\omega_0^2 - \omega^2)^2 + (2\xi\omega_0\omega)^2}} \]  

(13)

\[ \varphi(\omega) = \arctan\frac{-2\xi\omega_0\omega}{\omega_0^2 - \omega^2} \]  

(14)

Take \( |H_d(\omega)| \) to \( \omega \) to calculate the extreme value. Since \( \xi \) is small, it is generally considered as 0.05 in engineering. Then, the frequency corresponding to the peak amplitude versus frequency is shown in Equation (15):

\[ \omega_0' = \omega = \omega_0\sqrt{1 - 2\xi^2} \approx \omega_0 \]  

(15)

At this point, \( \omega_0 \) is believed to be the natural circular frequency, and modal analysis can be
performed accordingly. If $a$, $b$ and $c$ in Equation (3) are replaced by system mass matrix $[M]$, damping matrix $[C]$, and stiffness matrix $[K]$, modal identification can be carried out accordingly.

According to the above theory, the transfer function method represents the ratio of input and output. And the frequency response function method is a subset of the transfer function method and is the ratio of response to incentive. Therefore, it is suitable for the analysis of nonlinear random vibration signals represented by earthquakes.

**Data pre-processing**

In the shaking table test, due to the existence of various kinds of interference, it is often necessary to pre-process the vibration signal to make the sampling frequency as close to the true value as possible. In this paper, the eliminating trend term and digital filter are used to pre-process the collected data.

The polynomial least-squares is commonly used to eliminate the trend term, and the order of eliminating trend term adopted in this paper is order 5.

In the model test, as the collected disperse cosine signals contain a variety of sources, digital filter is needed to filter out the noise or false components in the test signals. In this paper, taking FIR filters and implemented through the window method. In order to pay attention to the different contributions of each frequency component in different frequency bands, taking the Hanning window can widen the major lobe and the side lobe can offset each other to the maximum extent to get more effective suppression of leakage. Therefore, the Hanning window is selected in this paper, and its calculation formula is Equation (16):
\[ \omega(t) = \begin{cases} \frac{1 + \cos \frac{\pi t}{T}}{2} & (0 \leq t \leq T) \\ 0 & (t > T) \end{cases} \quad (16) \]

**Frequency response function calculation**

The transfer function calculation for random vibration signals is usually realized by the power spectral density function (Jin et al., 2020), which is the Fourier transform of the signal auto-correlation function. In this paper, the average periodic diagrams (Welch method) is used to calculate it. The frequency response function is calculated as the quotient obtained by dividing the cross-power spectral density function \( S_{xy}(k) \) of the excitation signal and the corresponding signal by the self-rate spectral density function \( S_{xx}(k) \) of the excitation signal, that is Equation (17):

\[ H(k) = \frac{S_{xy}(k)}{S_{xx}(k)} \quad (17) \]

Where, \( S_{xy}(k) \) and \( S_{xx}(k) \) are defined as Equations (18) and (19) respectively:

\[ S_{xy}(k) = \frac{1}{MN_{FFT}} \sum_{i=1}^{M} X_i(k) Y_i^*(k) \quad (18) \]

\[ S_{xx}(k) = \frac{1}{MN_{FFT}} \sum_{i=1}^{M} X_i(k) X_i^*(k) \quad (19) \]

In the above formula, \( X_i(k) \) and \( Y_i(k) \) are the Fourier transform of the \( i \)th data segment of one or two random vibration signals. \( X_i^*(k) \) and \( Y_i^*(k) \) are the conjugate complex numbers of \( X_i(k) \) and \( Y_i(k) \) respectively. And \( M \) is the average degree.

In this paper, the sampling frequency was taken as 1000Hz, the fast Fourier transformation (FFT) length was set as 2048. The excitation signal was the acceleration signal collected by the measuring point A0, and the results were analyzed according to the distribution of measuring points at different positions. It can be seen from Equations (11) that the frequency response function can be represented by real frequency and imaginary frequency or amplitude versus...
frequency and phase. The former is more often used in use. The data frequency collected in this paper is 1000Hz, and high-frequency sampling leads to a wide frequency range of random interfering signals, which accounts for a large proportion. Therefore, there are many glitches after the collected data are plotted, which are not smooth, data smoothing can eliminate burrs and the influence of higher-order trend terms can also be eliminated. The data smoothing method selected in this paper is the five-spot triple smoothing method, because it can smooth the signal in both time domain and frequency domain, effectively reduce the high-frequency random noise. When analyzing the frequency response function, the intersection of the real frequency characteristic curve and the frequency axis of the multi-degree of freedom system is prone to horizontal movement. Under the influence of near modes, the imaginary frequency is usually used for analysis. Pearson correlation coefficient can be used to analyze the variation of frequency response function, currently, the commonly used judgment criteria in the engineering field were shown in Table 5.

Seismic response analysis of slope based on transfer function

Slope surface measuring point

The acceleration of the measuring point A0 was taken as the excitation signal. The point A1 in the middle of the slope surface was selected as the response signal to calculate the frequency response function after the input of seismic waves of different excitation intensity. The results were shown in Fig.15 and Fig.16. It can be clearly observed from Fig.15 that, when the 50° slope is loaded, the real frequency and imaginary frequency parts of the frequency response function of A1 measuring point are relatively similar before the excitation intensity is 0.5g.
While the real frequency and imaginary frequency parts of the loaded seismic wave change obviously after the excitation intensity is 0.5g, both the real frequency and the imaginary frequency parts of the first-order natural frequency become smaller. The increase of the amplitude of the signal fluctuation indicates that the structure appears damage deformation in the signal transmission from measure point A0 to measure point A1. The Fig.16 shows that as the excitation intensity increases, frequency response function of the measured points A1 of 60° slope has changed, real frequency and the imaginary frequency part of the first order natural frequency and the corresponding amplitudes were gradually reduced. Before the excitation intensity was 0.4g, the variation rules of the real frequency and the imaginary frequency parts were similar of 60° slope, and the frequency response function changed significantly when the excitation intensity was 0.4g and 0.5g.

Taking the frequency response function of the point A1 on the slope surface when the excitation intensity is 0.1g as the reference. With the excitation intensity increases, the variation law of the correlation coefficient of the frequency response function shown in Fig.17. According to the definition of correlation coefficient, both the real frequency and the imaginary frequency of a 50° slope are greater than 0.8 before the excitation intensity is 0.6g, which indicates a high correlation. Thereafter, the correlation coefficient decreased significantly and was less than 0.8. Similarly, the real frequency and imaginary frequency parts of 60° slope are highly correlated before the excitation intensity is 0.4g, when the excitation intensity is 0.4g, the correlation coefficient of 60° slope will be severely reduced, however, the slope surface will not be damaged until the excitation intensity is 0.5g. This indicates that cracks may have appeared in the structure when the excitation intensity is 0.4g. By comparison, it can be seen
that the frequency response function of the 60° slope decreases more sharply than that of the
50° slope, which is consistency with the failure phenomenon of the slope.

**Effects of the weak interlayer**

Similarly, the frequency response functions of the accelerations at points A2 to A5 on both
sides of the weak interlayer of the slope were calculated based on the accelerations at points
A0. The correlation coefficient of the frequency response function of the above measurement
points for 50°slope and 60°slope can be calculated based on the excitation intensity is 0.1g, as
shown in Fig.18 and Fig.19. In Fig.18, the real frequency part of each measuring point is
relatively similar. In the case of slope failure, the imaginary frequency correlation coefficient
of both the point A2 and the point A4 are < 0.8. The real frequency correlation coefficient of
the A4 measuring point is the smallest. In terms of the correlation coefficient of the imaginary
frequency part, each measurement point is obviously different, which is shown as A2 < A3, A4
< A5, and A2 < A4. The phenomenon shows that the signal changes significantly after the
seismic wave passes through the weak interlayer, leading to the occurrence of motion
inconsistency. The failure of the slope is due to the penetration of the fracture from the point
A2 to the point A4, rather than the development of the fracture at the back edge of the top of
the slope. Fig.19 shows that the variation trend of the correlation of the frequency response
function of the 60° slope is similar to that of the 50° slope. The real frequency part changes
little and the law is not obvious, so it should be affected by the near modes. The change of
imaginary frequency is more obvious, in which the correlation coefficient of point A2 point is
less than that of point A4. It indicates that the crack in the slope body evolves from the lower
part to the upper part and is still a traction landslide, which is inconsistent with the surface
displacement of the slope and may be due to the movement form of the accumulation body. The correlation coefficient of point A3 is lower than that of point point A2, which may be due to the large own weight of the deposited mass at point A2, structure center is at the front, the seismic wave transmission first arrives at point A2 and then arrives at point A3. In addition, the correlation coefficient is less than 0.4 when the excitation intensity is 0.5g, it should be caused by the signal fluctuation after the structure is damaged and cracks appear.

It can be seen from the above phenomena that the sliding of the experimental slope is mainly due to the occurrence of cracks between the accumulation mass and the weak interlayer caused by the earthquake. The obvious movement of the bedrock and the accumulation mass is inconsistent. Under the horizontal force of the earthquake and the gravity along the weak interlayer zone, the cracks between the soil are constantly expanding, the of the “anchor section” shear surface is broken, the cracks gradually develop and extend, and finally the penetration cracks appear, and the slope collapses. It can be expressed as shear-slide-fall type, which belongs to traction landslide.

**Modal analysis**

The frequency response function can also be used for modal analysis of vibration signals. The white noise input in this test can be regarded as the environmental excitation, therefore, white noise can be used to identify the modal parameters of the vibration signal. The lowed recognition accuracy can be avoided by avoiding the signal side-lope and low resolution in the ground motion response signal, currently, the methods commonly used in modal analysis include the half-power bandwidth method, the admittance circle method, etc, the former is
based on the single-degree-of-freedom structure, the latter only uses the least square rationale to estimate the radius or mode shape of conductance circle (Nyquist plot). The least-square iteration method is a classical solution method based on analytic expressions to numerically fit the real frequency response function data, it can obtain the best fitting between the test data and the mathematical model in the sense of the least square. The calculation formula is shown in Equation (20).

\[ H_{pq}(\omega) = -\sum_{i=1}^{N} \left( \frac{A_{ipq}}{j\omega - s_i} + \frac{A_{ipt}}{j\omega - s_i^*} \right) \omega^2 \]  

Among them, N is the free degree of structure, \( s_i \) and \( A_{ipq} \) are polar points and remained number of the i-th mode of frequency response function, and \( s_i^* \) and \( A_{ipt} \) are the conjugate complex numbers of the first two. By substituting equation (23) into equation (8) and setting the initial damping factor as 0.05, the vibration coefficient, eigen frequency \( \omega_i \) and damping ratio \( \xi_i \) of each mode can be calculated. In this paper, A1 is taken as the target to represent the overall mode of the slope, the frequency of the first 3 order frequency response function is taken for identification. As shown in Fig.20, according to the fitting curve of the test condition under the failure state, the calculated results are close to the measured results.

Fig.21 shows the vibration coefficients of the first three modes obtained by calculation. It can be seen from the Fig.21 that the vector of the first vibration mode of the two groups of slopes is significantly larger than that of the second and the third. This indicates that the slope vibration is mainly controlled by the first mode of vibration. However, the magnitude of the second mode vector of the in some cases under excitation intensity is large, and it means that the seismic mode of the slope under this gradient is relatively complex and can be regarded as having two degrees of freedom. Represented by two groups of measuring points on the slope
surface, the natural frequency and damping ratio of the first mode (main mode) of the slope in the measured group were calculated and observed. It is worth mentioning that, empirically, some scholars believe that it is spurious mode when the damping ratio $>20\%$. However, due to the large discreteness of soil and the wide frequency band of white noise, the calculated damping ratio will be larger, therefore, this paper believes that the calculated results are reliable in trend. It can be seen from Fig.22 that, with the increase of the excitation intensity, the natural frequency of the slope was decreases in trend. Assume that the weight of the test model will not change when the damping ratio is not taken into account, the reduction of the natural frequency indicates that the lateral stiffness of the structure decreases. The increase in the damping ratio indicates that cracks and expansion occurred in the test model, resulting in increase of friction between soils and enhanced energy absorbing action, therefore, the damping ratio increases. Meanwhile, under the same intensity seismic wave, the comparison shows that the damping ratio of a $60^\circ$ slope is higher than that of a $50^\circ$ slope, the $60^\circ$ slope natural frequency is less than that of the $50^\circ$ slope, indicating that the higher the gradient is, the less the anti-lateral stiffness will be, while the internal friction of soil mass will be more obvious.

**Conclusions**

This paper is based on a typical accumulation slope at the entrance of Zheduo Mountain tunnel, shaking table model tests of the $50^\circ$ accumulation slope and $60^\circ$ accumulation slope respectively were carried out. The displacement and acceleration of the slope were tested, the dynamic characteristics of the measuring point were analyzed using the transfer function, and the modal analysis was carried out. The main conclusions are as follows:
(1) Under the action of earthquake, the failure of the overlying slope is mainly the overall collapse along the weak interlayer zone, the slope displacement of the slope is relatively consistent. At the same time, debris flow was generated on the slope surface. The 50° slope was destroyed when the peak acceleration of Wenchuan earthquake wave was 0.6g, the 60° slope was destroyed when the peak acceleration of Wenchuan earthquake wave was 0.5g. At the top of the accumulation mass, there were obvious expanding cracks at the back edge.

(2) Based on the data pre-processing and average periodic diagrams method, the transfer function can be calculated and reflecting the vibration characteristics of the accumulation slope. Pearson correlation coefficient and modal analysis can be used to analyze the structural characteristics reflected by the transfer function. The analysis method can be well applied to the slope shaking table test.

(3) The frequency response function of accumulation slope can be analyzed from two aspects: slope surface and slope interior. Based on the frequency response function by white noise scanned after the excitation intensity is 0.1g, on the slope surface, the correlation coefficient of 50° slope significantly decreased and was destroyed after the excitation intensity is 0.6g, while the correlation coefficient of 60° slope significantly decreased after the excitation intensity is 0.4g, failure will not occur until the excitation intensity is 0.5g. With the increase of the excitation intensity, the correlation coefficient of the 60° slope is lower than that of 50° slope and drop more. The correlation coefficient of the slope failure are all < 0.8. Inside the slope, the correlation coefficient of frequency response function at the point A2 decreased less than that at the point A4, it means that the crack in the slope developed from the lower part to the upper part, the correlation coefficients between point A2 and point A3, point A3 and point A4
were significantly different, and it shows that the weak interlayer had a significant influence on seismic wave transmission. Based on the slope displacement and frequency response function, it can be seen that the slope in this test is retrogressive failure under the action of earthquake.

(4) Using the least-square iteration method to analyze the frequency response function can be used for the modal identification and analysis of accumulation slope. Represented by the point on the slope surface, the two groups of slopes are mainly affected by the first mode and second mode of vibration. With the increase of the excitation intensity, the post-earthquake damping ratio of 50° slope increased more on the 60° slope, and the natural frequency of 60° slope decreased more on 50° slope.

Data and Resources

The data used to support the findings of this study are available from the corresponding author upon request.

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References

Assimaki, D., E. Kausel, and G. Gazetas (2005). Wave propagation and soil-structure interaction on a cliff crest during the 1999 Athens Earthquake, Soil Dyn. Earthq. Eng. 25 513-527.

Beyabanaki, S. A. R., A. C. Bagtzoglou, and L. B. Liu (2016). Applying disk-based discontinuous deformation
analysis (DDA) to simulate Donghekou landslide triggered by the Wenchuan earthquake, Geomech. Geoengin. 11 177-188.

Cui, Z., Q. Sheng, Y. Song, and Q. Wei (2012). Application of transfer function to stochastic seismic response analysis of underground caverns, Rock and Soil Mechanics 33 3760-3766.

Curtis, W. D., J. D. Logan, and W. A. Parker (1982). Dimensional analysis and the pi theorem, Linear Algebra and its Applications 47 117-126.

Fan, G., J.-j. Zhang, and X. Fu (2017). Research on transfer function of bedding rock slope with soft interlayers and its application, Rock and Soil Mechanics 38 1052-1059.

Fan, G., J.-j. Zhang, X. Fu, and L.-r. Zhou (2016). Dynamic failure mode and energy-based identification method for a counter-bedding rock slope with weak intercalated layers, Journal of Mountain Science 13 2111-2123.

Jin, Z., Q. Han, K. Zhang, and Y. Zhang (2020). An intelligent fault diagnosis method of rolling bearings based on Welch power spectrum transformation with radial basis function neural network, Journal of Vibration and Control 26 629-642.

Kojima, K., K. Sakaguchi, and I. Takewaki (2015). Mechanism and bounding of earthquake energy input to building structure on surface ground subjected to engineering bedrock motion, Soil Dyn. Earthq. Eng. 70 93-103.

Komoda, K., M. Sakuma, M. Yata, Y. Yamazaki, F. Imaizumi, R. Kuroda, and S. Sugawa (2015). Measurement and Analysis of Seismic Response in Semiconductor Manufacturing Equipment, Ieee Transactions on Semiconductor Manufacturing 28 289-296.

Kumar, V., I. Jamir, V. Gupta, and R. K. Bhasin (2021). Inferring potential landslide damming using slope stability, geomorphic constraints, and run-out analysis: a case study from the NW Himalaya, Earth Surface Dynamics 9 351-377.
Liu, C., S. Qi, L. Tong, and F. Zhao (2004). STABILITY ANALYSIS OF SLOPE UNDER EARTHQUAKE WITH FLAC3D, Chinese Journal of Rock Mechanics and Engineering 23 2730-2733.

Michalowski, R. L. (2009). Expanding collapse in partially submerged granular soil slopes, Canadian Geotechnical Journal 46 1371-1378.

Michalowski, R. L., A. Wojtasik, A. Duda, A. Florkiewicz, and D. Park (2018). Failure and Remedy of Column-Supported Embankment: Case Study, Journal of Geotechnical and Geoenvironmental Engineering 144.

Moore, J. R., V. Gischig, J. Burjanek, S. Loew, and D. Faeh (2011). Site Effects in Unstable Rock Slopes: Dynamic Behavior of the Randa Instability (Switzerland), Bulletin of the Seismological Society of America 101 3110-3116.

Pagliaroli, A., B. Quadrio, G. Lanzo, and T. Sano (2014). Numerical modelling of site effects in the Palatine Hill, Roman Forum, and Coliseum Archaeological Area, Bulletin of Earthquake Engineering 12 1383-1403.

Paolucci, R. (2002). Amplification of earthquake ground motion by steep topographic irregularities, Earthquake Engineering & Structural Dynamics 31 1831-1853.

Sheng, Q., Z. Cui, J. Liu, and X. Leng (2012). Application study of transfer function for seismic response analysis of underground engineering, Rock and Soil Mechanics 33 2253-2258.

Shinoda, M., and Y. Miyata (2017). Regional landslide susceptibility following the Mid NIIGATA prefecture earthquake in 2004 with NEWMARK’S sliding block analysis, Landslides 14 1887-1899.

Singh, H., and S. K. Som (2016). Earthquake triggered landslide-Indian scenario, Journal of the Geological Society of India 87 105-111.

Trombetti, T. L., and J. P. Conte (2002). Shaking table dynamics: Results from a test-analysis comparison study, Journal of Earthquake Engineering 6 513-551.

Valentino, R., G. Barla, and L. Montrasio (2008). Experimental analysis and micromechanical modelling of dry...
granular flow and impacts in laboratory flume tests, Rock Mechanics and Rock Engineering 41 153-177.

Yin, C., W.-h. Li, C.-g. Zhao, and X.-a. Kong (2019). Impact of tensile strength and incident angles on a soil slope under earthquake SV-waves, Engineering Geology 260.

Yu, P., Y. Zhang, X. Peng, J. Wang, G. Chen, and J. X. Zhao (2019). Evaluation of impact force of rock landslides acting on structures using discontinuous deformation analysis, Computers and Geotechnics 114.

Yuan, W. Z. (1998). Similarity theory and statics model test, Southwest Jiaotong University Press, Sichuan.

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(a) Wangjiayan Landslide

(b) The Baocheng Railway 109 tunnel collapsed

**Fig.1** Site of landslide caused by Wenchuan earthquake

**Fig.2** The landform of the Zheduo Mountain

**Fig.3** Historical seismogenic situation near the slope supporting project
(a) Model filling                    (b) Physical parameter test

Fig. 4 Model preparation

(a) 50° Slope
(b) 60° Slope

Fig. 5 Overall diagram of test model

Fig. 6 Layout of the measuring points, $\beta$ is 50° or 60° (unit: cm)

(a) Acceleration time history
Fig. 7 White noise wave

Predominant frequency: 7~12Hz

Fig. 8 Wenchuan seismic wave
Fig. 9 Kobe seismic wave

Fig. 10 EL Centro seismic wave
Fig. 11 50° slope failure phenomenon

(a) Slope deformation

(b) Slope cracks and deformation
(a) Slope deformation

![Image of slope deformation]

(b) Slope cracks and deformation

Fig. 12 60° slope failure phenomenon

![Image of slope surface displacement]

Fig. 13 Slope surface displacement

![Diagram of slope collapse]

Fig. 14 Diagrammatic drawing of slope collapse
After loading 0.1g seismic wave
After loading 0.2g seismic wave
After loading 0.3g seismic wave
After loading 0.4g seismic wave
After loading 0.5g seismic wave
After loading 0.6g seismic wave
After loading 0.1g seismic wave
After loading 0.2g seismic wave
After loading 0.3g seismic wave
After loading 0.4g seismic wave
After loading 0.5g seismic wave

Fig. 16 Frequency response function of measuring point A1 of 60° slope

Fig. 17 Frequency response function of measuring point A1 of 60° slope

Fig. 18 Frequency response function of measuring point A2-A5 of 50° slope
Fig. 19 Frequency response function of measuring point A2-A5 of 60° slope

(a) 50° Slope after input of 0.6g peak acceleration seismic wave

(b) 60° Slope after input of 0.5g peak acceleration seismic wave

Fig. 20 Imaginary frequency modal identification results of point A1s at slope failure
The amplitude of vibration mode vector

Input wave peak acceleration (g)

First vibration mode
Second vibration mode
Third vibration mode

(a) 50° slope

(b) 60° slope

Fig. 21 The first three modes of vibration after the earthquake of the slope
Fig.22 Post-earthquake damping ratio and natural frequency of slope

Table 1 Similarity criterion derived by matrix method

| Physical quantity | Target volume | Relationship |
|-------------------|---------------|--------------|
|                   |               |              |
|                  | $\pi_1$       | $1$          |
|                  | $\pi_2$       | $1$          |
|                  | $\pi_3$       | $1$          |
|                  | $\pi_4$       | $1$          |
|                  | $\pi_5$       | $1$          |
|                  | $\pi_6$       | $1$          |
|                  | $\pi_7$       | $1$          |
|                  | $\pi_8$       | $1$          |
|                  | $\pi_9$       | $1$          |
|                  | $\pi_{10}$    | $1$          |
|                  | $\pi_{11}$    | $1$          |
|                  | $\pi_{12}$    | $1$          |
|                  | $\pi_{13}$    | $1$          |
|                  | $\pi_{14}$    | $1$          |

Table 2 The similarity ratio of main physical quantities in the test model

| Physical quantities          | Symbols and relation expression | Similitude parameter |
|-----------------------------|---------------------------------|----------------------|
| Geometry $L$                | $C_L$                           | 10                   |
| Soil weight $\gamma$        | $C_\gamma$                      | 1                    |
| Duration $T_d$              | $C_{T_d} = C_L^{0.5}$            | 3.16                 |
| Cohesion $\gamma$           | $C_\gamma = C_L$                | 10                   |
| Internal friction angle $\varphi$ | $C_\varphi = 1$    | 1                    |
| Dynamic modulus of elasticity $E$ | $C_E = C_L$                   | 10                   |
Poisson ratio $\mu 
C_\mu = 1
1$
Shear wave velocity $V_s$
$C_{V_s} = C_L^{0.5}$
3.16
Gravity accelerometer $g$
$C_g = 1$
1
Input vibration frequency $\omega$
$C_{\omega} = C_L^{0.5}$
0.316
Input acceleration $A$
$C_A = 1$
1
Output acceleration $a$
$C_a = 1$
1

Table 3 Material parameters

| Structure type       | Density (g/cm$^3$) | Cohesion (kPa) | Internal friction angle (°) |
|----------------------|-------------------|----------------|-----------------------------|
| Accumulation mass    | 1.908             | 1.55           | 37.9                        |
| Weak interlayer      | 1.72              | 0.27           | 41.7                        |
| Bedrock              | 2.206             | 43             | 38.91                       |

Table 4 Loading cases included in the test

| Peak accelerations  | Type of input wave | Excitations peak accelerations | Type of input wave | Excitations peak accelerations | Type of input wave | Excitations peak accelerations |
|---------------------|--------------------|-------------------------------|--------------------|-------------------------------|--------------------|-------------------------------|
| 0.05g               | White noise        | 0.05g                         | White noise        | 0.05g                         | White noise        | 0.05g                         |
| 0.1g                | Earthquake wave    | 0.4g                          | Earthquake wave    | 0.1g                          | Earthquake wave    | 0.4g                          |
| 0.05g               | White noise        | 0.05g                         | White noise        | 0.05g                         | White noise        | 0.05g                         |
| 0.2g                | Earthquake wave    | 0.5g                          | Earthquake wave    | 0.2g                          | Earthquake wave    | 0.5g                          |
| 0.05g               | White noise        | 0.05g                         | White noise        | 0.05g                         | White noise        | 0.05g                         |
| 0.3g                | Earthquake wave    | 0.6g                          | Earthquake wave    | 0.3g                          | Earthquake wave    | 0.3g                          |

Table 5 Criterion of correlation coefficient

| The correlation coefficient | The related degree |
|-----------------------------|--------------------|
| 0.00±0.30                   | Micro correlation  |
| ±0.30±0.50                  | Actual correlation |
| ±0.50±0.80                  | Significant correlation |
| ±0.80±1.00                  | Highly correlated  |