Experimental Study of Bridge Foundation Reinforced with Front and Back Rows of Anti-Slide Piles on Gravel Soil Slope under El Centro Waves

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Featured Application: The research conclusions could be used as the reference for engineering designs, especially for the seismic reinforcement design of a bridge foundation reinforced with front and back rows of anti-slide piles on a gravel soil slope. The supposed method of two-dimensional equipotential maps used in model test analysis are visual and convenient.

Abstract: A shaking table test for a bridge foundation reinforced with the front and back rows of anti-slide piles on a gravel soil slope was designed. The test results were obtained by loading El Centro waves with different peak accelerations. It was not an advantage for the deformation of bridge pile foundation while the distance between the front-row anti-slide piles and pier was large. The back-row anti-slide piles played a major role in seismic reinforcement, and the peak bending moment of the pile shaft and the peak earth pressure behind the pile had a triangular distribution. The distance from the crack to the sliding surface of the anti-slide pile was approximately one fifth of the length of the anchoring section. As the crack propagated, the bearing capacity of the pile shaft decreased gradually. Since the influence of pier inertia force and soil horizontal thrust, a peak negative bending moment and a peak positive bending moment were observed near the pile top and the sliding surface respectively. The rate of attenuation of the bending moment from the top of the pile along its depth was related to the resistance of the soil around the pile. The stress-induced deformation of the pile foundation behind the pier was larger than that in front of the pier. The peak ground acceleration (PGA) amplification factor of the slope had a vertical amplification effect and a layered distribution. The acceleration responses of the sliding section and the steep slope section were strong, while the acceleration responses of the region between the bridge pier and the back-row anti-slide piles were weak. With the increase in the vibration intensity, the soil damping ratio increased and the PGA amplification factor decreased. The feasibility of analyzing the acceleration response of the slope model by the two-dimensional equipotential map was experimentally verified.

Keywords: slope; bridge pier foundation; anti-slide pile; shaking table test; two-dimensional equipotential map
1. Introduction

In mountains and valleys frequently prone to earthquakes, many bridge pile foundations are constructed on potential landslide slopes and need to be reinforced through the design of seismic support structures. As one of the common structural forms, anti-slide piles have been extensively used to reinforce the bridge foundations on slopes. Thus far, fruitful research had been conducted on the dynamic response of slopes reinforced with front and back rows of anti-slide piles. Liu et al. [1] performed a shaking table test to study a cut slope of the Yuxi–Mengzi railway which was reinforced by double-row anti-slide piles. It was revealed that the existence of anti-slide piles reduced acceleration response. The earth pressure of the lower-row pile increased higher than the upper one, which increased with the earthquake magnitude. Lai et al. [2] pointed out that the soil acceleration would change suddenly when the slope was cracked or near to failure in a model test, suggesting that the strength and stiffness of the upper part of the anti-slide piles in the back row should be strengthened. Ye [3] and Xiao [4], respectively, analyzed the design method of a strengthening slope with the front and back rows of anti-slide piles, and the numerical simulation and elastic foundation beam model analysis provided a beneficial reference for seismic reinforcement design. Knappett and Madabhushi [5] carried out theoretical and experimental studies on the characteristics of force and deformation of slope bridge foundation. The results showed that the P-Δ effect should be considered in the calculation of bridge pile deflection, otherwise the calculated displacement of the bridge foundation is smaller. Lin et al. [6] established the static differential equations of bridge pile foundation considering the effect of slope, and determined the calculation formulas of force and deformation of the pile foundation according to the finite difference method. Gong et al. [7] designed bearing tests of bridge pile foundation on three different slopes, and analyzed the distribution law of the internal force of the pile foundation as well as the soil pressure distribution under horizontal and vertical loads. On this basis, Lei et al. [8] carried out a shaking table test of a bridge foundation reinforced by the rear anti-slide piles in a small distance. The pattern of the dynamic interaction between the bridge foundation and the rear anti-slide piles was analyzed, the spectral response characteristics subjected to seismic waves was summarized, and the variation pattern of the peak ground acceleration (PGA) amplification factor and the landslide thrust of a silty clay slope was obtained. In high-intensity earthquake areas where the rock strata exhibit a fractured occurrence, there are numerous engineering cases of using the front and back rows of anti-slide piles to reinforce a bridge foundation on a gravel soil slope, yet the related experimental research lags behind engineering practice.

For the seismic condition of bridge pier located on a gravel soil slope, the pile foundation is subjected to a non-negligible horizontal thrust. Therefore, the resulting horizontal deformation of the pile shaft should be controlled to ensure the serviceability of bridges. In current engineering practice, the front and back rows of anti-slide piles are mostly designed on the basis of the existing experience or a simple theoretical calculation. The rationality of the design remains open to question and hence calls for experimental verification.

The Chengdu–Lanzhou Railway crosses the eastern side of the Qinghai–Tibet Plateau. The seismic fault zone along the railway has a complex structure [9], and earthquakes are prone to occur frequently in this region, as shown in Figure 1. Through preliminary investigations, the pier pile foundation of the site of Jiuzhaigou Bridge was reinforced by front-back row anti-slide piles. The volume of gravel soil landslide is about 840,000 m$^3$, the peak value of regional ground motion acceleration is 0.2 g, and the characteristic period of the ground motion response spectrum is 0.45 s. It is a typical landslide with a high risk of geological disasters. This study took this site as a prototype, and a shaking table model test was conducted on the bridge foundation reinforced with the front and back rows of anti-slide piles on the gravel soil slope. In order to simulate the actual seismic conditions, the El Centro wave, which recorded the complete time history curve of acceleration for the first time in history, was used as the input vibration wave. Yang [10] analyzed the influencing factors of rock slope dynamic response by inputting El Centro waves. As the slope angle increased, the vertical magnification effect of acceleration increased, but the acceleration of the slope direction did not change significantly. By loading the El
Centro waves with different peak acceleration, Ye et al. [11] found that the earth pressure distribution on the rear side of cantilever piles was in parabolic form, with the peak acceleration of El Centro waves increased and the acceleration amplification factor decreased. On this basis, this study explored the distribution characteristics of the landslide thrust of stacked gravel soil by loading different peak accelerations of El Centro waves. The stress-induced deformation patterns of the pier pile foundation as well as the front and back rows of anti-slide piles, in addition to the stress-induced deformation compatibility of the three, were analyzed. The variation pattern of the PGA amplification factor of the gravel soil slope under seismic waves was investigated, the evolution of the anti-slide pile damage and slope failure was summarized, and a new method for analyzing the acceleration response of the slope model by using a two-dimensional equipotential map was developed. The results revealed the dynamic response characteristics of the front and back rows of anti-slide piles, pier pile foundation, and gravel soil slope, and provided the necessary technical references for the seismic reinforcement design of a bridge foundation on a gravel soil slope.

Figure 1. Seismic distribution and fault zones of the Chenglan railway.

2. Shaking Table Test Design

2.1. Overview of Worksite

The worksite for this study is situated in the county of Jiuzhaigou, Sichuan Province, China. It is a colluvial slope and has a fan-shaped distribution. The slope has a main axis in the north-south direction and is 260 m long and 30–160 m wide. The main pier of the bridge is located in the anti-sliding section of the slope. The pile foundation has three rows, which have pile lengths of 40 m, 32.5 m, and 27.5 m, respectively. The landslide is mainly reinforced with anti-slide piles. In order to consolidate the seismic reinforcement effect, a composite retaining structure of soil nail walls and anti-slide piles is added between the main bridge pier and the back row of anti-slide piles to increase the horizontal load bearing capacity. The slope is composed of layers of gravel soil, weakly weathered (W2) sandstone, slate, and limestone mixture from the top to the bottom. The profile of the slope along the line is shown in Figure 2.
2.2. Overview of Test Equipment

The test was performed on a unidirectional electro-hydraulic servo shaking table independently designed by Southwest Jiaotong University [12,13]. The main structural components are shown in Figure 1a. The vibrating table was 4 m × 2 m in size, and its basic parameters are shown in Table 1. To facilitate construction and observation, a geotechnical model was studied in the rigid model container. As shown in Figure 1b, the container was 3.7 m × 1.5 m × 2.1 m (length × width × height) in size. The inner side of the wall was filled with an 8-cm-thick polyethylene foam board and rubber pad [14] to reduce the reflection effect of the seismic waves [15,16].

| Performance Indices          | Specific Parameters       |
|------------------------------|---------------------------|
| Shaking table size           | 4 m × 2 m                 |
| Maximum load                 | 25 t                      |
| Maximum acceleration         | 1.2 g                     |
| Maximum displacement         | ±100 mm                   |
| Output waveform accuracy     | 3%                        |
| Control mode                 | Acceleration, displacement|
| Loading waveforms            | Regular/seismic waves     |

2.3. Similarity Design

Compliant with the geometry of the rigid model box, the prototype slope was scaled down by 70 times. The length, unit weight, and loading acceleration of the geotechnical model were the main
control factors. According to the similarity theorem [17,18], the similarities of the main physical quantities were derived. C was defined as the similar ratio of physical quantities which included length, stress, and so on. The similarity criteria are shown in Table 2. The similarity constants of the test model were derived according to the dimensional analysis [19]. Using mass (M), length (L), and time (T) as the basic dimensions, we obtained the geometric, stress, and density similarity constants. Under the same gravity field, the design similarity constant of acceleration was 1. The shaking table model test was divided into three sub-systems, namely, the geotechnical model, the pier pile foundation and anti-slide piles, and the seismic wave loading. The corresponding similarity constants are shown in Table 3, Table 4, Table 5, respectively.

Table 2. Similarity criteria of physical quantities.

| Physical Quantity       | Similarity Relationship |
|-------------------------|-------------------------|
| Strain                  | $C_\varepsilon = 1$     |
| Internal friction angle | $C_\phi = 1$            |
| Poisson’s ratio         | $C_\mu = 1$             |
| Displacement            | $C_s = C_l$             |
| Cohesion                | $C_c = C_l C_\gamma$    |
| Acceleration            | $C_a = C_l C_\omega^2$  |
| Stress                  | $C_\sigma = C_l C_\gamma$ |
| Gravity acceleration    | $C_g = C_l C_\omega^2$  |
| Frequency               | $C_\omega = C_f^{-1}$   |

Table 3. Similarity constants of geotechnical model.

| Physical Quantity       | Symbol | Formula | Similarity Constant |
|-------------------------|--------|---------|---------------------|
| Length                  | l      | $C_l$   | 70                  |
| Stress                  | $\sigma$ | $C_\sigma = C_l$ | 70 |
| Unit weight             | $\gamma$ | $C_\gamma$ | 1 |
| Gravity acceleration    | $g$    | $C_g$   | 1                   |
| Shear modulus ratio     | $G/G_{max}$ | $C_{G/G_{max}} = 1$ | 1 |
| Shear strain            | $\gamma$ | $C_\gamma = 1$ | 1 |
| Damping ratio           | $\lambda$ | $C_\lambda = 1$ | 1 |

Table 4. Similarity constants of bridge pile foundation and anti-slide piles.

| Physical Quantity       | Symbol | Formula | Similarity Constant |
|-------------------------|--------|---------|---------------------|
| Length                  | l      | $C_l$   | 70                  |
| Unit weight             | $\gamma$ | $C_\gamma$ | 1 |
| Gravity acceleration    | $g$    | $C_g$   | 1                   |
| Strain                  | $\varepsilon$ | $C_\varepsilon = 1$ | 1 |
| Stress                  | $\sigma$ | $C_\sigma = C_l$ | 70 |
| Displacement            | $S$    | $C_S = C_l$ | 70 |
| Shear force             | $F$    | $C_f = C_\sigma^2 C_f$ | 70$^3$ |
| Bending moment          | $M$    | $C_M = C_\sigma C_f^2$ | 70$^4$ |

Table 5. Similarity constants of seismic wave loading.

| Physical Quantity       | Symbol | Formula | Similarity Constant |
|-------------------------|--------|---------|---------------------|
| Length                  | l      | $C_l$   | 70                  |
| Unit weight             | $\gamma$ | $C_\gamma$ | 1 |
| Gravity acceleration    | $g$    | $C_g$   | 1                   |
| Acceleration            | $a$    | $C_a$   | 1                   |
| Time                    | $T$    | $C_T = C_l^{0.5}$ | 8.37 |
| Frequency               | $a_1$  | $C_{a_1} = C_l^{-0.5}$ | 0.12 |
2.4. Test Model Construction

According to the in situ geological survey data, artificially prepared geotechnical materials were used for the model test. The gravel soil was simulated by the mixture of wetted gravel and pea gravel. The physical and mechanical parameters of the soil are shown in Table 6, and the gradation curve is shown in Figure 4. A mixture of bentonite and fine sand with a mass ratio of 1:2 was used to simulate the potential sliding surface [20,21]. The bedrock was a stirred mixture of different materials, including red clay, fine sand, cement, and water with a mass ratio controlled as 1.055:1:0.25. It was constructed using the layer-filling method and compacted in layers. The elastic modulus of the hardened rock mass reached 16.4 GPa, which provided the necessary strength and stiffness.

Table 6. Physical and mechanical properties of gravel soil.

| Stratum       | Density (kg/m³) | Cohesion (kPa) | Inner Friction Angle (°) | Water Content (%) | Elastic Modulus (MPa) |
|---------------|-----------------|----------------|--------------------------|-------------------|-----------------------|
| Gravel soil   | 2100            | 0              | 38.6                     | 7.6               | 150                   |

![Grading curve of gravel soil](image)

Figure 4. Grading curve of gravel soil.

On the basis of the condition of the prototype worksite, the test model was appropriately simplified in the design of the front and back rows of anti-slide piles to reinforce the bridge foundation on the gravel soil slope. According to the reinforcement ratio and the stirrup ratio of the prototype structure, the reinforcement cages of the pier pile foundation and anti-slide piles were tied and formed, and micro-concrete was poured to construct a reduced scale model, as shown in Figure 5a,b. The geometry and concrete grades of the front-row and back-row anti-slide piles as well as the pier pile foundation are shown in Table 7. Findings from the existing research have shown that the well-graded micro-concrete has similar stress-induced deformation characteristics as ordinary concrete [22]. A steel box with a sliding track was welded on the top of the pier, and two counterweights were hung on the track to simulate the dynamic response of the adjacent box girders during vibration [23], as shown in Figure 5c. The weights and the spacing of the counterweights were determined on the basis of the similarity relationships. The photograph of the full test model is given in Figure 5d.

Table 7. Structural dimensions of bridge foundation and anti-slide piles.

| Structure             | Length (cm) | Sectional Dimension (cm) | Pile Spacing (cm) | Concrete Grade |
|-----------------------|-------------|--------------------------|-------------------|----------------|
| Front-row stabilizing pile | 38.6        | 2.9 × 4.3                | 8.6               | C35            |
| Back-row stabilizing pile | 65.7        | 3.6 × 5.0                | 8.6               | C35            |
| Bridge pile foundation | 57.1, 46.4, 39.3 | Diameter 2.9            | 6.6 or 9.3        | C30            |
| Cushion cap           | 5.7         | 23.1 × 30.9              | —                 | C30            |
2.5. Layout of Measurement Points

All of the measurement points were located along the central axis of the slope model, as shown in Figure 6. Nine horizontal accelerometers were placed inside the rock and soil mass to facilitate the dynamic response analysis of the slope. The accelerometer A10 was located on the surface of the vibrating table and monitored the seismic waveform output. The accelerometer A9 was located on the pier cap to acquire the acceleration response of the cap. In all, 16 earth pressure cells were buried near the pier and anti-slide piles to monitor the horizontal thrust on the pile shafts. Four horizontal displacement transducers were fixed on the welded angle steel bracket. The measurement points included key positions such as the slope toe, top of the front and back rows of anti-slide piles, and the cushion cap. The horizontal displacement values were analyzed to determine the flexural deformation of the pile shafts as well as the failure state of the slope. In total, 15 pairs of strain gauges were attached symmetrically to the front and back sides of the pier pile foundation and the anti-slide piles to measure the flexural deformation of the pile shaft. The technical parameters of the test elements are shown in Table 8.

Figure 5. Photographs of model construction. (a) Reinforcement cage; (b) bridge pile foundation; (c) box girder simulation; (d) test model.

Figure 6. Longitudinal section of test model.
2.6. Seismic Wave Loading

The El Centro wave was used as the input seismic wave, and the seismic waveform was continuously loaded with a stepwise increase in the peak acceleration, as shown in Figure 7. The time history of the El Centro wave, which had a duration of 20.47 s, is shown in Figure 8. Before the start of the seismic wave loading, a 0.08-g Gaussian white noise wave was applied for the excitation test. The Fourier spectrum of the time history curve of the accelerometer A4 in Figure 9 showed that the slope had a natural frequency of 17.1 Hz, while the Fourier spectrum of the El Centro wave as shown in Figure 10 indicated that the energy of the seismic wave was mainly concentrated in the low-frequency band below 12 Hz. Because the frequency response ranges of the two were different, there was no frequency coupling distortion for the slope model in the test.

Table 8. Sensor parameters.

| Classification          | Size/mm | Technical Parameters                |
|-------------------------|---------|-------------------------------------|
| Earth pressure cell     | Φ 67 × 13 | Accuracy: <±0.5% FS, range: 0.1 MPa |
| Horizontal accelerometer| Φ 24 × 40 | Acceleration range: ±50 g, voltage sensitivity: 100 mV/g |
| Strain gauge            | 10 × 3  | Resistance value: 120.1 ± 0.1 Ω, sensitivity coefficient: 2.20% ± 1% |
| Displacement transducer | Φ 35 × 300 | Accuracy: (0.05–0.04%) FS |

Figure 7. Loading steps.

Figure 8. Time history curve of El Centro wave.
piles were not very effective as a seismic support structure because of their large distance from the pier, and the landslide thrust was mainly resisted by the back-row anti-slide piles.

Under the El Centro wave with a peak acceleration of 0.1 g to 0.3 g, the surface soil of the slope had a strong dynamic response. Figure 11a shows that the slope surface exhibited the phenomenon of delamination, the soil particles moved and slid within a small range, and the gravel soil tended to slide and collapse. Figure 11b shows that under the 0.4 gravitational acceleration El Centro wave, the steep section of the slope, which had poor stability, collapsed locally and tension cracks occurred on the slope surface. This failure phenomenon appeared symmetrically and simultaneously on both sides of the central axis of the model, proving the rationality of the slope filling. Figure 11c shows that under the 0.5 g El Centro wave, the chain effect of the slope failure caused a substantial loss of soil in the lower part of the slope, and the gravel soil continued to accumulate near the slope toe, reducing the horizontal resistance of the soil in front of the back-row anti-slide piles and exposing the pile foundation in front of the pier, thereby causing the loss of soil resistance to some extent. As shown in Figure 11d, when the peak acceleration of the seismic wave reached 0.6 g, a large portion of the soil in the sliding section of the slope slid, the slope bed near the crest of the slope was exposed, and the sliding mass formed a significant accumulation behind the back-row anti-slide piles. During the vibration process, the gravel soil constantly showed the phenomenon of slide and shear at the top of the back-row anti-slide piles, and the back-row anti-slide piles played a role in the seismic reinforcement. The front-row anti-slide piles were not very effective as a seismic support structure because of their large distance from the pier, and the landslide thrust was mainly resisted by the back-row anti-slide piles.
Figure 11. Development process of landslide under the action of vibration wave. (a) soil slip on the slope surface; (b) local failure of the slope body; (c) failure of the lower half of the sliding body; (d) the test model was completely destroyed.

3.2. Horizontal Displacement of Measurement Points

Figure 12 shows that as the peak acceleration of the seismic wave increased, the horizontal displacements of the measurement points at the top of the anti-slide piles and at the bottom of the slope increased, but the horizontal displacement of the cushion cap was zero, and the failure of the slope did not cause the pier pile foundation to have a large flexural deformation. The slope of the horizontal displacement curve at the top of the front-row anti-slide piles was small and developed linearly. With the increase in the vibration intensity, the horizontal displacement curve at the top of the back-row anti-slide piles changed from linear to non-linear. When the peak acceleration of the seismic wave reached 0.4 g, the slope of the curve increased significantly, cracks occurred in the anchoring section of the pile shaft, and cracks continued to propagate under the seismic waves, leading to a substantial increase in the horizontal displacement of the top of the back-row anti-slide piles. The crack locations of 17 back-row anti-slide piles were statistically analyzed. As shown in Figure 13, the blue horizontal line was the position of the sliding surface, and the black curves marked the cracks. The distance between the crack and the sliding surface was approximately one fifth of the anchorage length of the anti-slide piles. The crack location was under the combined action of a large bending moment and shear, resulting in a common form of stress-induced failure of elastic piles.

Figure 12. Horizontal displacement curves.
was distributed in an inverted triangle pattern. The gravel soil started to fail from the relatively steep peak acceleration of the seismic wave increased, the peak earth pressure gradually increased. Because peak earth pressure behind the back-row anti-slide piles. In Figure 14b, the peak earth pressure in the process of the continuous loading of the seismic waves, the soil behind the back-row anti-slide piles gradually slid until a large number of sliding masses accumulated behind the back-row anti-slide piles, and the load state of the pile shaft was aggravated continuously, leading to an increase in the peak earth pressure behind the back-row anti-slide piles. In Figure 14b, the peak earth pressure in front of the back-row anti-slide piles had an inverted trapezoidal distribution. Because the deflection at the top of the pile was the largest, the soil was compressed, causing the peak earth pressure to be relatively large. In Figure 14c, the peak earth pressure behind the pier had an approximately rectangular distribution, and the distribution pattern of the horizontal earth pressure between the pier and the back-row anti-slide piles gradually changed from an inverted trapezoid to a rectangle, lowering the center of gravity of the horizontal load. In Figure 14d, the peak earth pressure in front of the pier was distributed in an inverted triangle pattern. The gravel soil started to fail from the relatively steep surface of the slope, leading to a decrease in the peak earth pressure along the burial depth. As the peak acceleration of the seismic wave increased, the peak earth pressure gradually increased. Because the front-row anti-slide piles were in the anti-sliding section of the slope, the landslide thrust behind the piles was very small, as shown in Figure 14e. When the peak acceleration of the seismic wave reached 0.6 g, the peak earth pressure was only approximately 600 Pa. The above analysis showed that the stress-induced deformation of the back-row anti-slide piles was larger than that of the front-row anti-slide piles, which was basically consistent with the conclusion drawn by Shen et al. [24].
3.4. Distribution Characteristics of Pile Shaft Bending Moment

The concrete was tested using strain gauges in a Wheatstone quarter bridge circuit. The bending moment of the pile shaft was calculated according to the tensile and compressive strain values, using the following formula, where \( E \) is the elastic modulus of the micro-concrete, \( I \) is the moment of inertia of the section, \( EI \) is the flexural stiffness of the pile shaft, \( \epsilon_+ \) and \( \epsilon_- \) are the micro-strains on the tension side and the compression side, respectively, and \( h \) is the linear distance between the measurement points on the tension side and the compression side, respectively, of the same section.

\[
M = \frac{EI \cdot (\epsilon_+ + \epsilon_-)}{h}
\]  

When the test model was loaded with El Centro waves with different peak accelerations, the bending moment of the back-row anti-slide piles was as shown in Figure 15a. As the peak acceleration of the seismic wave increased from 0.1 g to 0.3 g, the bending moment of the back-row anti-slide piles gradually increased, with the maximum bending moment of the pile shaft occurring near the sliding
was crushed. Figure 15b shows that the dynamic load of the superstructure caused the peak negative moment of the pile shaft continued to decrease until the tensile steel bars yielded and the concrete was crushed. Figure 15b shows that the dynamic load of the superstructure caused the peak negative bending moment to occur at the top of the pile foundation behind the pier. Under the soil resistance around the pile, the negative bending moment along the pile depth gradually changed to a positive bending moment, which reached its peak value at the bottom of the sliding mass, which restrained the flexural deformation of the pile shaft. Figure 15c shows that the variation pattern of the bending moment of the pile foundation in front of the pier was similar to that behind the pier; because of the steep slope in front of the pier, the soil around the pile had limited resistance during vibration, causing the attenuation rate of the negative bending moment along the pile depth to be smaller. The front-row anti-slide piles were located in the anti-sliding section of the slope. As shown in Figure 15d, the bending moment of the pile shaft above the sliding surface was very small. Therefore, the seismic reinforcement design of the front-row anti-slide piles needs to be further optimized.

![Figure 15. Distribution of bending moment. (a) Back-row anti-slide pile; (b) back-row pile of bridge foundation; (c) front-row pile of bridge foundation; (d) front-row anti-slide pile.](image)

3.5. Distribution of Peak Ground Acceleration (PGA) Amplification Factor of the Slope

According to the test results of the horizontal accelerometers in the test model, the peak acceleration in the time-history curve was extracted and divided by the peak table acceleration to obtain the PGA amplification factor \[22\], which was combined with the horizontal and vertical coordinates of the monitoring section to draw a two-dimensional equipotential map by conversion into the form of a digital matrix by using the Renka Cline random matrix generation method.
Figure 16 shows that the PGA amplification factor of the slope had a vertical amplification effect. As the height increased, the PGA amplification factor tended to increase and had a layered distribution. The sliding section of the slope and the steep section in front of the pier had relatively poor stability and a strong tendency for sliding failure, and the PGA amplification factor of the corresponding area was large. Because the back-row anti-slide piles withstood most of the landslide thrust, the horizontal inertial force of the soil mass behind the pier was limited. Therefore, the PGA amplification factor in the area between the pier foundation and the back-row anti-slide piles was relatively small, and the back-row anti-slide piles played their role in seismic reinforcement. The bedrock was artificially filled and compacted, resulting in a large elastic modulus and a weak PGA amplification effect. Because there were only three accelerometer points in the bedrock, there was a certain error in plotting the equipotential lines according to the digital matrix, leading to a larger PGA amplification factor in the upper area of the bedrock. As the peak acceleration of the seismic wave increased from 0.1 g to 0.6 g, the PGA amplification factor of the slope generally showed a decreasing trend. As the shear modulus of the soil decreased and its damping ratio increased, the frictional energy dissipation between the soil particles increased, and the energy loss of the upwardly propagating seismic wave increased, resulting in a reduction in the PGA amplification factor. When the peak acceleration of the seismic wave reached 0.6 g, the slope completely failed, and the gravel soil accumulated substantially behind the back-row anti-slide piles. The compactness of soil in this region increased to some extent, resulting in a weakened dynamic response and a smaller PGA amplification factor.

Figure 16. Cont.
completely failed, and the gravel soil accumulated substantially behind the back-row anti-slide piles. The compactness of soil in this region increased to some extent, resulting in a weakened dynamic response and a smaller PGA amplification factor.

Figure 16. Peak ground acceleration (PGA) amplification factor distribution equipotential maps of slope with different seismic inputs. (a) 0.1 g; (b) 0.2 g; (c) 0.3 g; (d) 0.4 g; (e) 0.5 g; (f) 0.6 g.

4. Conclusion

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4. Conclusions

A 1:70 scale model was designed and constructed to study the bridge foundation on a gravel soil slope reinforced with the front and back rows of anti-slide piles, and loaded with the El Centro wave in the laboratory shaking table test. The stress-induced deformation of the pier pile foundation and the front and back rows of the anti-slide piles as well as the acceleration response characteristics of the slope were analyzed. According to the experimental phenomena, the following conclusions were drawn:

(1) When the front and back rows of anti-slide piles were used to reinforce the bridge foundation on the gravel soil slope, the back-row anti-slide piles withstood most of the landslide thrust and played the main role in seismic reinforcement. The front-row anti-slide piles could provide horizontal resistance to the pier foundation, and the distance had to be small.

(2) The pier pile foundation had a large bending moment near the top and the sliding surface, where the piles had to be reinforced. The rate of attenuation of the bending moment from the top of the pile along its depth was related to the soil resistance, and the reasonable spacing of the back-row anti-slide piles and bridge foundation should be controlled. If the back-row anti-slide piles were too closed to the bridge foundation, the dynamic interaction was strong. If the spacing was large, the soil inertial force behind bridge pier increased and went against the deformation of bridge piles.

(3) The test results showed that the stress-induced deformation of the pile foundation behind the pier was larger than that in front of the pier. It is recommended that the pile foundation behind the pier act as the analysis subject in the seismic reinforcement design so as to control the stress-induced deformation of the pile shaft. The engineering design of lowering height and weight of the bridge is advantage for the forced deformation of the pile foundation.

(4) For a gravel soil slope, the peak earth pressure behind the back-row anti-slide piles had a triangular distribution. After the pile shaft cracked, it still had a certain horizontal load-bearing capacity. As the cracks propagated, the tensile steel bars gradually yielded, and the concrete was crushed, causing the bearing capacity of anti-slide piles to gradually decrease. The distance between the crack and the sliding surface was approximately one fifth of the length of the anchoring section of the anti-slide pile, where the pile was subjected to a combination of a large shear and bending moment.

(5) The PGA amplification factor of the slope had a vertical amplification effect. As the height increased, the PGA amplification factor tended to increase and had a layered distribution. The regions with a large PGA amplification factor were the sliding section and the steep section of the slope, while the region between the pier and the back-row anti-slide piles were kept with a small PGA amplification factor. The bedrock was an approximately rigid body and, thus, the increase in the PGA amplification factor was very small.

(6) The test results showed that it was feasible to analyze the distribution pattern of the PGA amplification factor of the slope by using the two-dimensional equipotential map, which could meet the basic requirements of the macro analysis of the slope acceleration response. The mapping accuracy was related to the number and the layout mode of the horizontal acceleration measurement points. For the key monitoring regions, the number of horizontal acceleration measurement points should be increased and a reasonable layout scheme should be formulated.

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