Slope stability analysis and protection of deep accumulation excavation foundation pit in the Sichuan Bank of the Kahalo Jinsha River Bridge

Liu Tianxiang1, Du Zhaomeng1* & These authors contributed equally to this work and should be considered co-first authors. Cheng Qiang1, Wu Yunling1, Lei Hang1

1 Sichuan Highway Planning, Survey, Design and Research Institute Ltd, Chengdu, Sichuan 610041, China
duzhaomeng@schdri.com

Abstract. The Kahalo Jinsha River Super Large Bridge of the Sichuan Riverside Expressway spans the deep canyon of the Jinsha River and is situated in a high-intensity, complex, and dangerous mountainous area. Long-term deformation and stability of the gravity anchor slope are the main controlling factors affecting bridge safety. The maximum slope height formed by the excavation of the gravity anchor foundation pit on the Sichuan Bank of the Kahalo Jinsha River Bridge is 110 m, and the anchorage area is located on the Qionghai thick accumulation with a thickness of more than 170 m. First, the important soil mechanical parameters, such as shear strength, deformation modulus and bearing capacity eigenvalues of rock and soil were obtained by conducting field shear tests, deformation tests, and load tests. On this basis, the pseudo-static method was used to simulate the seismic conditions, and a three-dimensional numerical analysis and calculations of the potential instability range of the high slope of the gravity anchor foundation pit excavation were conducted. The results reveal that the slope of the super-deep foundation pit with a huge thick mixed accumulation layer will deform and become unstable under the influence of an earthquake, with a maximum deformation of about 20 m in the slope body. A two-level earthquake prevention and disaster-reduction target for the bridge foundation pit and slope engineering protection is proposed based on the two-level seismic fortification requirements of super-long-span suspension bridges. An active reinforcement scheme of excavation pre-reinforcement, step-by-step excavation, and zoning protection is designed and proposed based on this target and the deformation and stability analysis results of the field test and three-dimensional numerical calculation.
The dynamic time history analysis method was used to verify the deformation and stability of the reinforced foundation pit and slope. The results revealed that the maximum displacement of the reinforced foundation pit and the high slope was 0.223 m after a 30 s Wenchuan Earthquake wave, and the slope deformation was effectively controlled. According to the completion of the foundation pit construction, the maximum slope displacement is 8.5 mm, which further verifies the effective prevention and control of the sliding instability of the excavated temporary foundation pit slope and permanent slope, which can ensure the long-term safety of the super bridge project during operation, and provides a reference for the analysis, calculation, and reinforcement design of similar projects in the future.

Preface
Highway project constructions in western mountainous areas require large-span bridges that span deep gorges. Suspension bridges have become the preferred option because of their strong spanning abilities, low cost, and lightweight. Anchorages play a vital role in the stability of suspension bridges. They are the core component for supporting the main cable and ensuring the stability of the main structure of the bridge. Gravity anchors are more widely used than tunnel anchors because they have fewer requirements for geological conditions. However, there are few studies on the stability of gravity anchorage slopes of suspension bridges built on huge accumulations both home and abroad.

Li [1] and Li et al. [2] studied the stability of an anchorage foundation pit using model tests. Zhao [3] expounded on the current state of mechanical analysis of the anchorage system and the mechanical mechanism of foundation instability. Sun et al. [4] used an artificial intelligence prediction method to conduct dynamic prediction research on the deformation of an anchorage foundation pit during construction. Most of the previous studies used model test, which is expensive, and the traditional mechanical analysis method, which also has its limitations in the analysis of the stability of anchorage foundation pits. The rapid development of numerical simulation has provided a viable solution for resolving the foundation pit slope stability problem. Relevant scholars have conducted many numerical simulation studies on slope stability: Tian et al. [5] used the limit equilibrium method to design and reinforce a high and steep highway slope, which was then analyzed using the finite element method. Li et al. [6] and Zou et al. [7] used numerical simulation to analyze the stability of bridge slope and proposed a reinforcement scheme. However, there are few studies on using the numerical simulation method to analyze the excavation stability of the ultra-deep foundation pit slope exceeding 100 m.

The site area of the large bridge over the Kahalo Jinshajiang River on the Yanjiang Expressway in Sichuan province is a strong earthquake area. Since the Quaternary, there have been noticeable changes in the activity of regional large faults in the area. According to relevant records, since 1917, there have been more than 20 destructive earthquakes, six of which are above degree VIII. The pseudo-static method, which is simple and practical, is often used for slope stability analysis during earthquakes [8,9]. Michalowski et al. [10] used the pseudo-static method to draw a three-dimensional slope stability curve. Hack et al. [11] used the pseudo-static method to analyze the influence of earthquakes on slope stability. Adr et al.
studied the seismic response of slopes before and after reinforcement using a combination of numerical simulation with the pseudo-static method. However, it is difficult to use the pseudo-static method to consider the response of a slope during an earthquake. Dynamic time history analysis can better analyze the response of a slope to ground motion; however, there are few corresponding theoretical results. The dynamic time history analysis of a slope during an earthquake can now be conducted because of the rapid development of the numerical simulation method. Zhao et al., Li et al., and Mansour used the dynamic time history analysis in numerical simulation to analyze the seismic response and the seismic stability of a slope. Additionally, Zhang et al. used the dynamic time history analysis in numerical simulation to study the stability of a geogrid-reinforced soil slope. Qu et al. used the dynamic time history analysis in numerical simulation to study the stability of a slope reinforced by a sheet pile wall. However, there have been few previous studies on the dynamic stability of ultra-deep foundation pits. Therefore, it is necessary to study the stability of ultra-deep foundation pits of suspension bridges using both the pseudo-static method and the dynamic time history analysis.

This study is based on the ultra-deep foundation pit project of the gravity anchorage of the Kahalo Jinsha River Bridge. The slope stability analysis model of the foundation pit excavation process was established using the finite difference numerical simulation method based on geological investigation and field tests. The high and steep soil slope formed on the thick layer accumulation as well as the excavation process under strong earthquake conditions were determined and analyzed. This study serves as an important basis for slope reinforcement design and construction scheme.

1. Project overview
The proposed Kahalo Jinsha River Bridge is located at the junction of Yuanbaoshan Township, Leibo County, Liangshan Prefecture, Sichuan Province, and Huanghua Township, Yongshan County, Zhaotong City, Yunnan Province. The entire length of the bridge is 1817 m, with a span layout of $11 \times 41$ m T-girder. It is a 1030 m single-span steel truss girder suspension bridge $(+11 \times 41$ m T-girder), which has a frame-based gravity anchorage at the Sichuan bank.

1.1 Topography and lithology
The Jinsha River valley and its surrounding areas are located in the transition zone between the southeast edge of the Qinghai-Tibet Plateau and the Yunnan-Guiyang Plateau and in the middle section of the Hengduan Mountain. It is a high mountainous area with erosion and tectonic denudation. The terrain is generally higher in the northwest and lower in the southeast. The Sichuan quayside bridge is located in the middle and lower part of the left slope of the convex bank of the Jinsha River. There are secondary valleys on both sides and in front of the strip-shaped ridge, which is gentle in the lower part and steep in the upper part. The top of the ridge is relatively narrow. The slope near the highway line is about 15°~25°, the upward slope is gradually steeper, and the local slope is steep. Gravity, geology, and the Jinsha River water combine to form an accumulation in the middle and lower parts of the slope. The bridge axis crosses the deep-cut gorge of the Jinsha River at a large angle, and the thickness of the overburden on the slope surface gradually increases from bottom to top.
The overlying soil on the Sichuan quay bridge site is the Cenozoic Quaternary Holocene collapsed slope layer (Q₄e–d), which contains crushed silted clay, breccia, and gravel, as well as the Pliocene and Pleistocene mixed accumulation layer (Q₃h), which contains block stones and breccia, with a thickness of more than 170 m. The underlying bedrock consists mainly of the Paleozoic Silurian Lower Longmaxi Formation (S₁l) mudstone, marl, shale, and siltstone as well as the Paleozoic Ordovician middle-upper system (O₂–3) limestone.

![Diagram](image)

**Figure 1.** Typical section of the excavation slope of the anchorage foundation pit on the Sichuan bank

1.2 Earthquake

The peak acceleration of the ground motion in the bridge site area is 0.15 g, indicating that the area is prone to strong earthquakes. Since the Quaternary, the activities of regional large faults in the region have been significantly different. The Ebian-Jinyang fault was active from the early Pleistocene to the late Pleistocene but has been weak since the late Pleistocene, and the southern end is weaker than the northern end. The Lianfeng fault had many periods of activity in the early and middle Cenozoic, with stable creep characteristics. The most recent active period was from the Middle Pleistocene to the beginning of the Late Pleistocene, and there has been no evidence of activity since the late Pleistocene.

According to geological surveys, the soil at the Sichuan bank is complex, and it includes breccia, crushed rocks, boulders, pebbles, and silty clay. According to the drilling shear wave logging results, the soil shear wave velocity (Vs) is 262.5–676.2 m/s, the overburden is deep, and the drilling depth without overburden exposure is 71.3 m. The overburden is a Class II engineering site [10]. This section of the site is a slope landform, which is spread along the trailing edge of the slope, with a slope angle of about 15°–35° and signs of slumping. Earthquakes that induce slumping hazards may cause the foundation soil of the site to lose stability. Therefore, it is an earthquake-resistant section that has a significant effect on bridges.

In summary, because of the large-scale excavation of foundation pits for the anchorage on the Sichuan bank and because the anchorage area is a strong earthquake area, the slope is steep and the overburden thickness is huge. The stability analysis method and the support scheme for high and steep slopes with slipping instability have been studied.

2. In situ mechanical test of the soil on the gravity-anchored slope on the Sichuan bank
In this study, an in situ soil mechanical test was conducted on the upper Pleistocene mixed accumulation (Q^3h) soil slope where the gravity anchor is located. The test was conducted in Kahalo Yuanbaoshan Township on the Sichuan bank of the Kahalo Jinsha Bridge. The test flat hole is 25 m long, 1.8–2 m wide, and 1.8 m high. The test content includes two groups of 8-point field shear tests, 3-point load tests, 3-point deformation tests, 11-point rock and soil density tests, and particle analysis tests. The rock and soil mechanical parameters, such as shear strength, deformation modulus, and characteristic values of bearing capacity of the rock and soil, were determined based on the test results.

2.1 Soil density
The field density test adopts the irrigation method. A test pit with a diameter of 80 cm and a depth of 60 cm was manually excavated. The volume of the test pit was measured using the collar irrigation method, which calculates the volume of the test pit before and after pilot excavation. After the test pit excavation sample was fully weighed, the pilot density was calculated. The natural density of the soil was 2.34–2.46 g/cm³, the moisture content was 4.47%–7.53%, and the dry density was 2.20–2.31 g/cm³.

2.2 In situ shear test
A shear box with a diameter of 1 m was selected. The side length of the sample is 80–90 cm and the height is 50 cm. Each group of the 4-point shear test and a total of two sets of the 8-point shear test were used. The tests adopt the horizontal push method. Figures 2 and 3 depict the test devices. Table 1 shows the results of the in situ shear test. According to the test results, the recommended value range of the friction angle (φ) in the soil is 24°–35°, and the value range of the cohesion (c) is 88–128 kPa.

![Figure 2. Site installation photos of the direct shear test](image1)

![Figure 3. Pilot τ1-3 soaked in water for 10 h](image2)
Table 1. Results of the in situ shear test

| Pilot No. | moisture condition          | Normal stress σ (MPa) | Peak shear stress τ (MPa) | tanφ  | Cohesion c (kPa) |
|-----------|-----------------------------|------------------------|---------------------------|-------|-----------------|
| τ2-3      | Natural                     | 0.75                   | 0.49                      |       |                 |
| τ2-4      | Natural                     | 1.00                   | 0.57                      |       |                 |
| τ1-1      | Natural                     | 0.25                   | 0.24                      | 0.47  | 128             |
| τ1-2      | natural                     | 0.51                   | 0.42                      |       |                 |
| τ1-3      | soaked in water for 10h     | 0.73                   | 0.50                      |       |                 |
| τ1-4      | soaked in water for 10h     | 1.01                   | 0.57                      | 0.44  | 88              |
| τ2-1      | soaking                     | 0.25                   | 0.19                      |       |                 |
| τ2-2      | soaking                     | 0.50                   | 0.30                      |       |                 |

2.3 Deformation test and load test

The deformation test adopts the rigid bearing plate method. The test equipment, which is the same as that of the load test, has a circular bearing plate with a diameter of 80 cm and an area of 5000 cm², which is loaded in the horizontal direction. The design bearing capacity is 600 kPa, and the maximum load bearing capacity is 2 MPa. The deformation test results are shown in Table 2. In the natural state, the deformation modulus of the coarse-grained soil under horizontal load is 149.75–696.39 MPa, while the elastic modulus is 338.15–886.58 MPa. The dispersion in the test results of the three pilots is relatively large, and it tends to increase as the depth of the flat hole increases. Since the gravity anchor frame foundation is closer to the inner side of the mountain and the buried depth is greater than the test point of the flat cave, the test recommended value of the horizontal deformation modulus for the coarse-grained soil is 149.75–696.39 MPa, and the test recommended value of the elastic modulus is 338.15–886.58 MPa.

![Figure 4. Installation diagram of the test site](image)

Table 2. Deformation test results

| Pilot No. | From the entrance of the cave (m) | Maximum stress (MPa) | Deformation modulus (MPa) | Elastic modulus (MPa) | Remark          |
|-----------|-----------------------------------|----------------------|---------------------------|-----------------------|-----------------|
| E1        | 15.8                              | 1.0                  | 149.76                    | 338.15                | Natural state   |
| E2        | 19                                | 1.0                  | 328.16                    | 579.69                | Soaked in water |
| E3        | 23                                | 1.0                  | 696.39                    | 886.58                | Natural state   |

The load test results are shown in Table 3. The maximum load applied in each pilot test did not reach the failure limit load. There was no sharp increase in settlement during the test,
and there was no noticeable sign of extruding, cracking, or uplifting in the surrounding soil of the bearing plate. P1-1, P1-2, and P1-3 were determined, and the characteristic values of the bearing capacity were 0.8 MPa, 1.05 MPa, and 1.0 MPa respectively. The experimental bearing capacity characteristic value range was 0.25 MPa, which is not more than 30% of its average value. It meets the requirements of the “Specifications for Design of Foundation of Highway Bridges and Culverts.” The average value of 0.95 MPa was determined as the characteristic value of the bearing capacity of the coarse-grained soil.

Table 3. Load test results

| Pilot No. | From the entrance of the cave (m) | Maximum load (MPa) | Settlemnt (0.01 mm) | Proportion limit value (MPa) | Ultimate load (MPa) | Characteristic value of bearing capacity (MPa) | Whether it meets the design requirements | Damage condition |
|-----------|----------------------------------|--------------------|--------------------|-----------------------------|--------------------|-----------------------------------------------|----------------------------------------|------------------|
| P1-1      | 15.8                             | 1.6                | 741                | 1.6                         | 1.6                | 0.8                                           |                         | undamaged        |
| P1-2      | 19                               | 2.1                | 407                | 2.1                         | 2.1                | 1.05                                          | Yes                      | undamaged        |
| P1-3      | 23                               | 2.0                | 204                | 2                           | 2.0                | 1.0                                           |                         | undamaged        |

3. Stability analysis of the excavated foundation pit slope

The Sichuan bank slope is mainly composed of a Quaternary Cenozoic Quaternary Holocene Collapsible layer (Qc0–d3), an upper Pleistocene mixed accumulation layer (Q2p) gravel, and block stones. The overburden is thick. The underlying silty mudstone of the Silurian Longmaxi formation (Si1) has good integrity. The strongly weathered zone has a small thickness. Drilling has revealed that the maximum thickness of the overburden within the range of the Sichuan bank anchorage exceeds 170 m. The survey showed no signs of slope cracking and deformation. The current stability is good, but under the action of the excavation of the anchored foundation pit, the excavation height of the small pile of the side foundation pit is up to 111 m, and the stability of the slope after excavation is poor. There is a large slip and instability. The construction of anchors and bridge structures will be jeopardized by risks. In this study, the pseudo-static method was used based on the original test parameters to carry out three-dimensional numerical analysis and calculations for the high slope formed by the gravity anchor foundation pit on the Sichuan bank.

3.1 Calculation model

The finite difference calculation software was used for the numerical simulation analysis, and the three-dimensional calculation model of the slope before the excavation of the foundation pit is shown in Figure 5. The calculation model includes 4 strata: stratum 1 is a Qc0–d3 gravel-sand mixture, stratum 2 is a Q2p gravel-sand mixture, stratum 3 is Qc0–d3 gravelly clay, and stratum 4 is a Q2p rubble-sand mixture. The model is 700 m long and 620 m wide, and it uses tetrahedral elements for meshing. The model has 706794 elements and 131086 nodes. In the calculation and analysis, an ideal elastoplastic constitutive model was used to describe the stress–strain relationship of each layer, and the yield condition is described using the classic Mohr–Coulomb model. The physical and mechanical parameters used in each area based on the field test results are shown in Table 4.

According to the “Specifications for Seismic Design of Highway Bridges” (JTG B02-2013), the E1 seismic action, which is a design earthquake, is an earthquake with a
recurrence period of 475 years, and the E2 seismic action, which is a rare earthquake, is a seismic action with a recurrence period of 2000 years. Under different earthquake actions, the anti-seismic fortification targets of the foundation pit slope project are calculated as follows: the E1 seismic action has a seismic acceleration of 0.185 g, and the slope protection engineering seismic importance factor is 1.7; the E2 seismic has a seismic acceleration of 0.36 g. The importance coefficient is 1.0. For this calculation, the main control condition is the E2 seismic condition.

![Figure 5. Computational model](image)

| area                  | Elastic modulus (MPa) | Poisson's ratio | Cohesion (kPa) | Internal friction angle (°) | Bulk density (kN m$^{-3}$) |
|-----------------------|-----------------------|-----------------|----------------|---------------------------|--------------------------|
| Q$_4^{w+dl}$ gravelly clay | 300                   | 0.35            | 125            | 24                        | 23                       |
| Q$_4^{w+dl}$ gravel-sand mixture | 500                   | 0.33            | 100            | 28                        | 23                       |
| Q$_3^{h}$ gravel-sand mixture | 600                   | 0.31            | 115            | 33                        | 24                       |
| Q$_3^{h}$ rubble-sand mixture | 800                   | 0.3             | 110            | 35                        | 24                       |

3.2 Numerical calculation results and analysis

This section uses numerical simulation to analyze the possible plastic deformation zone, shear strain zone, and slope displacement of the foundation pit and slope after the excavation of the Sichuan bank anchorage and uses this as a basis to determine the deformation of the foundation pit and the excavated slope as well as the scope of potential instability damage.

The gravity anchor foundation pit was excavated in 11 levels, and the excavation depth of each level was 10 m. The partial schematic of the completed model of the 11th foundation pit excavation level is shown in Figure 6. To ensure the overall stability of the foundation pit, the method of excavation and support should be adopted. Here, the stability and instability failure mode of the excavated foundation pit during an earthquake were analyzed. The strength reduction method was used to analyze the state of the slope when each level of the foundation pit was excavated without support, and it provides a reference for the reinforcement design of excavation pits, which is described below.
3.2.1. Displacement analysis.

Figure 7 shows the displacement field near the excavated foundation pit and its vector diagram calculated using the pseudo-static method after the 7th level foundation pit excavation until the final completion. It can be observed that as the excavation depth increases, the maximum deformation of the pit gradually increases, and the area where the deformation is greater than 10 m gradually increases. The maximum deformation occurs at the trailing edge of the slope after the excavation of the eleventh-grade slope is completed, reaching nearly 20 m, indicating poor stability of the unsupported pit. The foundation pit must be supported in time after excavation.

3.2.2. Plastic analysis.

Figure 8 shows the distribution of plastic zones after various levels of excavation of the foundation pits. It can be observed that as the excavation progressed, the plastic zone of the slope gradually expanded, and the shear plasticity in the plastic zone accounts for the majority. However, after the excavation was completed, the range of the stretched plastic state increased and mostly appeared on the trailing edge of the slope. The plastic zone always appears on one slope because of the use of the pseudo-static method for analysis, and the actual seismic waves reciprocate in positive and negative directions. The analysis results in one direction can also be applied to the analysis of the horizontal seismic waves in other directions.
3.2.3. Strain analysis.
Figure 9 shows the increase in shear strain and its stability coefficient at each stage of the foundation pit excavation. The top of the slope on the left is the interface of the stratum, where local stress concentration occurs in the calculation. Therefore, the increase in the shear strain is always large. It is worth noting that as the excavation depth increases, the maximum shear strain increment of the most dangerous slope gradually increases. However, the stability coefficient of the foundation pit does not decrease significantly as the excavation depth increases. This also shows that the number of slopes that need to be improved with supporting measures is the same at all levels, providing a basis for using the same value as the anchor cable spacing and length on all the slopes in the design.

4. Reinforcement design of the excavation foundation pit slope
4.1 Two-level seismic fortification requirements and reinforcement ideas
This paper proposes two-level earthquake prevention and disaster-reduction targets for the protection of foundation pit slope engineering, based on the two-level seismic fortification requirements of super-long-span bridges and combined with the engineering characteristics of ultra-deep foundation pit slopes. In the first level, the foundation pit engineering is under the action of the E1 earthquake; the reinforced slope is not damaged, and it can be used normally after partial repair or without repair. In the second level, the slope project will locally damage
the reinforced slope project under the action of the E2 earthquake, but it can be used after repair and reinforcement.

According to the two-level seismic fortification requirements of the foundation pit engineering of the super-long-span suspension bridge as well as the slope deformation range determined using the three-dimensional numerical simulation during the excavation of the foundation pit, this study proposes that the reinforcement design idea of the ultra-deep foundation pit slope is to adopt a comprehensive treatment method, which is mainly anchoring combined with anti-slide piles and interception and drainage systems.

4.2 Reinforcement plan
Anchor shotcrete technology and frame anchoring structure are particularly suitable for high slope protection projects. They are deep reinforcement treatment technologies that can solve the stability problem of high slopes. The traditional single anchoring technology faces challenges in ensuring the stability of the slope while aiming at the ultra-deep foundation pit excavated on the huge accumulation layer. To achieve the purpose of eradicating the instability of high slopes, a comprehensive treatment plan of active reinforcement of pre-reinforcement of excavation, graded excavation, and zoned protection was used in this study. The details are as follows:

(1) Frame beam + pressure grouting anchor rod (anchor cable) reinforcement + hanging net and grass protection: A frame beam and a pressure grouting anchor rod (an anchor cable) were used to strengthen the excavation slope of the 6th to 11th level on the small pile side of the foundation pit with hanging nets and planting grass protection. The length of the anchor rod is 12–27 m, and the length of the anchor cable is 30–32 m.

(2) Net sprayed concrete + cushion pier + prestressed anchor cable protection: The 5th level excavation slope of the small pile side of the foundation pit was closed and reinforced by the suspended concrete sprayed concrete and a cushion pier anchor cable. The length of the prestressed anchor cable is 30–34 m. The design anchoring force is \( P_t = 560 \text{ kN} \), and the tension locking load is 670 kN.

(3) The hanging net sprayed concrete + cushion pier + pressure grouting anchor rod protection: Hanging net sprayed concrete, cushion pier, and pressure grouting anchor rods were used for the excavation slopes of the 1st to 5th level around the anchorage foundation pit (except the side of small pile numbers of the foundation pit). The grouting bolt was closed and reinforced, and the length of the bolt was 6–27 m.

(4) Anti-slide piles: A row of circular rotary anti-slide piles were installed on the first-level platform on the small pile side of the foundation pit to support the slope. The pile diameter was 2.0 m, the pile length was 16 m, and the pile center spacing was 5.0 m

(5) Intercepting and drainage system: An I-type intercepting ditch was set up around the designed excavation groove line to allow surface water to be discharged uniformly to the bottom of the slope.

5. Validation analysis of slope reinforcement of the excavated foundation pit
5.1 Dynamic time history analysis method
In this study, the finite difference software was used to verify the ultra-deep foundation pit reinforcement scheme, and the dynamic time history analysis method was used. The specific process is as follows: a finite difference method numerical model was established, the model
material parameters and static boundary conditions were set, and the model's self-weight stress analysis was carried out. The initial stress field of the model was generated using a static analysis under the field. Figure 10 shows the foundation pit model when the step-by-step excavation and step-by-step support were completed. The support measures of the foundation pit adopt the reinforcement scheme in section 4.2 of this article, which are the middle and lower parts. The combined supporting measures mainly consist of anchoring shotcrete, middle and upper frame anchor rods (cable), and slope foot anti-slide piles, as shown in Figure 11. The dynamic constitutive model and the material dynamic parameters were set based on this. The dynamic boundary condition was the free field boundary, and the 30 s seismic wave of the Wenchuan earthquake was input to conduct seismic dynamic time history analysis. In the analysis process, the method of applying real-time acceleration time history data at the bottom of the model was used to simulate the seismic action, and Rayleigh damping and free field boundaries were used. The slope materials are all Mohr–Coulomb materials, and the non-associative flow rule was adopted.

![Figure 10. Dynamic time history analysis model](image)

![Figure 11. Supporting measures for the excavation of the foundation pits](image)

Figure 12 shows the displacement of the foundation pit and its vector diagram after the earthquake. Figure 13 shows the global displacement of the slope foundation pit after the earthquake. It can be observed that the maximum displacement after the 30 s Wenchuan earthquake wave appeared in the upper part of the highest slope of the foundation pit. The maximum displacement of the slope is 0.223 m, which is significantly less than that of the
unsupported pit after excavation, and the deformation of the slope is effectively controlled. The support measures did not fail. Overall, the active reinforcement scheme proposed in this paper can still maintain the overall stability of the ultra-deep foundation pit and the excavated high slope, effectively control the deformation of the slope, and realize temporary excavation even under the influence of a major earthquake. The slippage and instability of foundation pit slopes and permanent slopes were effectively prevented and controlled. At the same time, this demonstrates that numerical simulation based on the finite difference method can be used to guide the design of foundation pit support under strong earthquakes and that the method is feasible and effective.

5.2 Slope displacement monitoring results
By June 2021, the foundation pit and the upper slope were fully excavated in place, the deformation was controllable, and the protective measures were effective. The site map is shown in Figure 14. According to the field monitoring data, the maximum displacement of the excavated slope appeared at the trailing edge of the eleventh-grade slope, which is consistent with the maximum deformation position of the numerical simulation results. Table 5 shows the monitoring results of the displacement monitoring points located at the trailing edge of the eleventh-grade slope. The results reveal that the displacement of the slope within 8 months following the completion of the excavation and support is within 8.5 mm, which is very small. The protection ideas and measures have been confirmed to be effective, which is based on the numerical simulation results of the maximum displacement of the supporting slope of 9.3 mm under a static state.

Figure 12. Displacement of the foundation pit and its vector diagram after the earthquake
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Figure 13. Displacement of the whole situation after the earthquake

Figure 14. Site drawing of the foundation pit and slope protection

Table 5 Displacement curve of the eleventh-grade slope trailing edge

6. Conclusion
This study used the Sichuan bank foundation pit of the Kahalo Jinsha River Bridge on the Sichuan Yanjiang highway as an example of a typical bridge foundation pit project in the high-intensity, complex, and dangerous mountainous area. The earthquake action was analyzed based on the results of on-site shear tests, deformation tests, and load tests. The instability failure mode of the rock–soil mass of the slope was determined by the stability of the ultra-deep foundation pit and the stability of the rock–soil mass of the high slope.
underneath the huge thick layer of accumulation. Furthermore, the pseudo-static method is proposed for analyzing the stability of the rock and soil mass of high slopes under the influence of an earthquake, which will be used to guide the design work. A reinforcement plan is also proposed. The dynamic time history analysis method was used to verify the stability of the reinforced slope. The main conclusions are as follows:

(1) The ultra-deep foundation pit excavated by the gravity anchor on the Sichuan Bank of the Kahalo Jinsha River Bridge is located on the upper Pleistocene mixed accumulation (Q₃h) thick layer, with a thickness of more than 170 m. The excavation formed a maximum slope height of about 110 m.

Three-dimensional numerical analysis and calculations of the potential instability range of the high slope formed by the excavation of the foundation pit were carried out using the pseudo-static method. The results show that under the action of an earthquake, the slope of the ultra-deep foundation pit with a huge thick mixed accumulation layer will deform and lose stability. The layer’s thickness is about 15~23 m, indicating that the high-steep soil slope formed by the excavation of the foundation pit is particularly susceptible to deformation, slip instability, and damage during the excavation process as well as strong earthquakes during operation.

(2) The recommended range of the mechanical parameters of the upper Pleistocene mixed accumulation (Q₃h) soil, which is composed of gravel, block stone, breccia, and gravel sand, were obtained via on-site shear tests, deformation tests, and load tests. The natural density was 2.34~2.46 g/cm³, the moisture content was 4.47%~7.53%, the dry density was 2.20~2.31 g/cm³, the internal friction angle of the soil was 24°~35°, the cohesive force was 88~128 kPa, the horizontal deformation modulus was 149.75~696.39 MPa, the elastic modulus was 338.15~886.58 MPa, and the average characteristic value of the bearing capacity was 0.95 MPa.

(3) Two-level earthquake prevention and disaster-mitigation targets are proposed for the bridge foundation pit engineering protection, according to the two-level seismic fortification requirements of the anchorage foundation pit of the super-long-span suspension bridge, and the specific targets for the large-thick layer accumulation in the strong earthquake area are provided. The active reinforcement plans for ultra-deep foundation pits are pre-reinforcement by excavation, stepwise excavation, and partition protection.

(4) The dynamic time history analysis method was used to study the deformation and stability of the reinforced ultra-deep foundation pit. The numerical analysis results reveal that the reinforced foundation pit and the excavated high slope were stable and that the deformation was controllable, indicating that slippage instability of the excavation temporary foundation pit slope and the permanent slope was effectively prevented. This will ensure long-term safety during the operation period of the extra-large bridge project and provide a reference for the analysis, calculation, and reinforcement design of similar projects in the future.

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