Field direct shear tests on a backfill soil slope and finite element analysis of its reinforcement schemes

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Abstract. The field direct shear test was used to analyze the backfill soil strength of a filling slope, and the slope controlled by different reinforcement conditions was simulated by finite element method based on the shear strength by the field test. The results show that the shear strength parameters of the backfill soil are friction angle $\phi = 15^\circ$ and cohesion $c = 25$ kPa. Reinforced by anti-slide pile with different lengths, the maximum horizontal displacement of the slope can be reduced by 79.7% ~ 81.3% compared with that of the unreinforced slope. The maximum shear strain of the soil is also reduced to some extent, and the sliding surface characteristics and the strain amplitude are weakened.

1. Introduction

The field direct shear test ensures the natural state of the soil by performing shear test directly on the in-situ rock and soil mass and measuring the stress-strain relationship, and it can apply vertical load according to the weight of the overlying soil layers. At the same time, the shear area of the sample is much larger than that of the indoor experiments, so it is closer to the actual situation.

Hu Wei et al [1] developed the in-situ direct shear test equipment and applied it to two landslides in Xiangjiaba reservoir area. Tang Jinsong et al [2] studied the shear strength index of the gravel soil. Liu Sihong et al. [3] conducted the direct shear tests on rock fill materials in the reservoir area. Yang Jihong et al. [4] conducted the direct shear tests on landslide accumulations, and applied the measured shear strength parameters to the stability evaluation. Xing HaoFeng et al [5] conducted large-scale shear tests on slate and sandstone splint rock in the South-to-North Water Transfer Project. Lu Zude et al [6] carried out direct shear tests on the strongly weathered rock mass in natural state and saturated state. Xu Xiaobin et al [7] conducted direct shear tests on highly weathered granite. In addition, many scholars studied the improvement and application of field direct shear equipments [8-10]. For the backfill soil which is difficult to sample and inconvenient to be taken back to laboratory for indoor experiment, field direct shear test is one of the most suitable means to obtain its strength parameters. Therefore, in this paper, field direct shear test is used to analyze the backfill soil strength of a filling slope, and then the strength parameters obtained from the test are used to simulate the support scheme by LS-DYNA program so as to obtain approximate reinforcement effect in advance and provide reference for the subsequent engineering design.
2. Project overview
See Figure 1 for slope overview. The survey site is a backfilled open space (Figure 1 (a)), with an area of about 10170.48 m². The backfill mainly consists of residual soil and weathered soil excavated near the hillside, containing about 5-15% fragmental stones (weathered and moderately weathered), mainly composed of tuff lava, with a particle size of 2-20cm. The filling time is about 4 years, with some consolidation. The average thickness of the backfill soil is 19.0m. Cracks, slippages and slumps appear at the edge of the slope (Figure 1(b)).

Since the backfill was not compacted during filling, the soils are relatively loose, and it is relatively difficult to take intact samples, so it is difficult to obtain the parameters accurately only through lab tests. However, if empirical parameters are completely adopted, the calculation results are inaccurate and too conservative, which will affect the reasonable design. Therefore, field direct shear test is used to analyze the backfill strength parameters so as to check the indoor and outdoor test results and obtain the design parameters more accurately. It provides a more rigorous reference basis for the landslide control design.

3. Field direct shear test

3.1. Test overview
Two key areas near the edge of the slope are selected as the test points. Four groups including 12 in-situ direct shear tests were carried out in total. The test procedure is shown in Figure 2. Firstly, the soil mass is cut into a block with a size of 0.5m × 0.5m × 0.25m. Then install the shear box, pressure plate, horizontal and vertical loading device, respectively. Finally, connect the displacement and Jack load measurement system to the acquisition instrument. The applied normal stress is 50 ~ 245 kPa. The horizontal shear force is applied by gradation loading. The peak shear stress or the shear stress under a shear displacement of 5 cm (i.e. 10% shear strain) is taken as the shear strength. Then, the Mohr-Coulomb failure curve can be obtained by linear fitting of several groups of "normal stress-shear strength". The angle of inclination is the internal friction angle, and the intercept with the longitudinal axis is the cohesion.
3.2. Result analysis
The test results are shown in Table 1. After a series of pre-tests, some abnormal data were eliminated, and four groups of effective test results were selected. The obtained strength parameters were \(c=25.12\) kPa, \(\phi=19\), \(c=26.14\) kPa, \(\phi=16\) and \(c=35.6\) kPa, \(\phi=15.3\), \(c=43\) kPa, \(\phi=13.9\). In consideration of the large difference in the backfill properties and referring to the relevant engineering experience and regional experience, it is recommended to use \(c=25\) kPa, \(\phi=15^\circ\) as the design parameters for landslide reinforcement.

| Group | Vertical loading (kPa) | Peak shear stress (kPa) | Cohesion (kPa) | Internal friction angle (°) | Remark |
|-------|------------------------|------------------------|----------------|-----------------------------|--------|
| 1     | 50                     | 37.68                  | 25.12          | 19                          | Test point 1 |
|       | 100                    | 69.08                  |                |                             |        |
|       | 150                    | 72.22                  |                |                             |        |
| 2     | 100                    | 56                     | 26.14          | 16                          |        |
|       | 200                    | 79                     |                |                             |        |
|       | 245                    | 99                     |                |                             |        |
| 3     | 50                     | 48                     | 35.6           | 15.3                        | Test point 2 |
|       | 100                    | 65.6                   |                |                             |        |
|       | 150                    | 75.4                   |                |                             |        |
| 4     | 50                     | 52                     | 43             | 13.9                        |        |
|       | 100                    | 73                     |                |                             |        |
|       | 200                    | 91                     |                |                             |        |
4. Finite Element Simulation of Landslide Reinforcement

4.1. Proposal of governance scheme and finite element model

Pile foundation joist retaining wall and local anchor cable is recommended to reinforce the slope. After simplification, the finite element models before and after reinforcement are shown in Figure 3. The layers with colors from light to deep are backfill soil, residual cohesive soil, loose strongly weathered tuff lava and fragmental strongly weathered tuff lava, respectively.

The diameter of the anti-slide piles in the benchmark model is 1 m, the pile length is 7 m, and the pile spacing is 3 m. The length of anchor cable is 22 m, and the anchoring section is located in the fragmentary strongly weathered tuff lava, with an anchoring length of 10 m. A 3 m section is selected for simulation. The finite element model has a length of 70 m and a height of 45 m. In order to fully study the effect of different reinforcement schemes, parametric analysis is carried out on pile length. See Table 2 for specific calculation conditions.

![Figure 3. Finite element model of the slope before and after reinforcement](image)

![Figure 3. Finite element model of the slope before and after reinforcement](image)

Table 2. Working conditions of numerical model

| Working condition | Research object                    |
|-------------------|------------------------------------|
| 1                 | No governance                      |
| 2                 | Pile length 7 m (reference model)  |
| 3                 | Pile length 5 m                    |
| 4                 | Pile length 10 m                   |

4.2. Material parameters of rock and soil

See Table 3 for the parameters of the Mohr-Coulomb constitutive model for rock and soil materials. The backfill is gravel soil with good water permeability and gradation, the cohesion is 0, and the internal friction angle is 42°.

The anchor cable is simulated by beam element with a diameter of 100 mm, the calculation parameters are as follows: elastic modulus 200 GPa, Poisson's ratio 0.3, density 7800 kg/m³.

The retaining wall and anti-slide pile are made of concrete material, the calculation parameters are as follows: elastic modulus 25 GPa, Poisson's ratio 0.2, density 2500 kg/m³.
Table 3. Parameters of soil and rock

| Soil type                  | Parameters                      |
|----------------------------|---------------------------------|
|                            | Gravel soil | Backfill soil | Residual cohesive soil | Loose strongly weathered rock | Granular strongly weathered rock |
| Density (kg/m³)            | 2000        | 1700          | 1710                   | 2100                         | 2200                             |
| Shear modulus (MPa)        | 18          | 6             | 8                      | 18                           | 20                               |
| Poisson's ratio (°)        | 0.2         | 0.3           | 0.3                    | 0.3                          | 0.3                              |
| φ (°)                      | 42          | 15            | 12.10                  | 30                           | 45                               |
| c (kPa)                    | 0           | 25            | 10.93                  | 32                           | 38                               |
| Dilatation Angle(°)        | 0           | 0             | 0                      | 0                            | 0                                |

4.3. Result analysis

4.3.1. Horizontal displacement. The horizontal displacement nephogram is shown in Figure 4. In order to show the comparison results more clearly, the deformation is magnified by 2 times. The comparison of the maximum horizontal displacement is shown in Table 4. It can be found that:

1) Before the slope is reinforced, the maximum horizontal displacement of Condition 1 reaches 180 cm (Figure 4(a)), the deformation mainly concentrated at the foot of the slope. The top of the slope is sinking violently, and the whole slope is almost collapsing. Therefore, it is necessary to take remedial measures immediately.

2) After adopting the reinforcement scheme of "pile foundation joist retaining wall and local anchor cable" (Condition 2), the maximum horizontal displacement is reduced to 25.17 cm (Figure 4(b)), which is 80.5% lower than that before reinforcement. The maximum horizontal displacement is mainly concentrated near the foot of the slope, and the area with higher displacement amplitude is also reduced, which fully demonstrates the effectiveness of the reinforcement scheme.

3) For the sensitivity analysis of pile length, the comparison between Figs. 5(b), (c) and (d) shows that when the pile length is reduced from the original 7 m (Condition 2) to 5 m (Condition 3), the maximum horizontal displacement increases to 26.11 cm (Figure 4(c)), which is 3.73% larger than Condition 2. However, when the pile length is increased from 7 m to 10 m (Condition 4), the maximum horizontal displacement is reduced to 24.04 cm (Figure 4(d)), which is 4.49% lower than Condition 2. At the same time, the slope area with higher displacement amplitude also decreases. This shows that the increase of pile length can reduce the deformation of the slope to a certain extent, but the effect is not significant for this case.

Table 4. Comparison of maximum horizontal displacement

| Working Condition | 1   | 2      | 3   | 4   |
|-------------------|-----|--------|-----|-----|
| Horizontal displacement (cm) | 128.8 | 25.17  | 26.11 | 24.04 |
| Compared to Condition 1 (%) | -    | -80.5% | -79.7% | -81.3% |
| Compared to Condition 2 (%) | -    | -     | 3.73% | -4.49% |
4.3.2. Slope shear strain. See Figure 5 for slope shear strain. The deformation characteristics and sliding surface of the slope can be analyzed by observing the local shear strain feature. It can be seen from the figure that the maximum shear under all reinforcement conditions except Condition 1 is about 5%, but the distribution of shear strain is quite different.

(1) The maximum shear strain of the unreinforced slope (Condition 1) reached 91%, and obvious sliding surface characteristics appeared in the shear strain concentrated area (Figure 5(a)). The sliding surface extends upward from the toe of the slope to the top of the slope, and the maximum shear strain occurs at the bottom.

(2) After different reinforcement schemes are adopted (Figs. 6(b)-(d)), due to the retaining wall and anti-slide piles, the maximum shear strain of the soil is reduced to different degrees, the overall strain amplitude of the slope is also significantly reduced, and the characteristics of the sliding surface are weakened.

It is worth noting that the sliding surface feature in Figs. 6(b) to 10(d) is more obvious than that in Figure 5(a), which is actually caused by different strain scales used in each figure.
5. Conclusion

Based on the reinforcement project of a backfill soil slope, the backfill strength parameters of the slope are analyzed by field direct shear test, and then the proposed reinforcement scheme is simulated and predicted by numerical calculation. The main conclusions are as follows:

(1) The field direct shear test results show that the strength parameter values obtained in the two selected test areas are $(c=25.12 \text{ kPa}, \phi = 19)$, $(c=26.14 \text{ kPa}, \phi = 16)$ and $(c=35.6 \text{ kPa}, \phi = 15.3)$, $(c=43 \text{ kPa}, \phi = 13.9)$, respectively. Considering the large difference in the properties of backfill soil itself, and for safety's sake and also referring to relevant engineering experience and regional experience, it is recommended to use $c=25 \text{ kPa}$ and $\phi = 15^\circ$ as the design parameters for landslide reinforcement.

(2) The finite element simulation of "pile foundation joist retaining wall and local anchor cable" supporting scheme is carried out for the slope, and parametric analysis is carried out for the pile length. The results show that the slope deformation decreases with the increase of pile length, and the horizontal displacement of the reinforced slope can be reduced by about 80% compared with that before reinforcement.

(3) The maximum shear strain of the unreinforced slope reached 89.7%, and obvious sliding surface characteristics appeared in the strain concentrated area, with sliding surface extending upward from the foot of the slope to the top of the slope. However, after different reinforcement schemes are adopted, the maximum strain is reduced to different degrees, the overall strain amplitude of the slope is also significantly reduced, and the sliding surface characteristics are weakened.

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