Method and Parameter Reliability of In-situ Direct Shear Test on Vibroflotation Gravel Piles

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Abstract. Vibroflotation gravel piles can improve the antisliding stability of soft soil foundations. However, in-situ direct shear tests have rarely been conducted on vibroflotation gravel piles, and the understanding of the test methods and the reliability of the derived parameters varies. Three groups of in-situ direct shear tests on vibroflotation gravel piles for a reservoir bank treatment project are conducted. The design diameter of the piles is 100 cm, and the pile filler is artificial sandstone gravel with a maximum particle size of 15 cm. The specimens are formed by applying a steel square frame to encase the top section of the piles. For the specimens of groups S1 and S2, only the section above the shear plane is encased with the frame. The shear surface of S1 contains a small amount of soft soil while that of S2 contains no soft soil. For group S3, two pile sections above and below the shear plane are encased with the frame, and a steel support is set upon the lower frame in the opposite direction of the shear force. The shear failure of the piles is a type of plastic shear contraction failure mode with more than 4.5% shear strain and no peak shear stress. The internal friction angles and cohesion values are 20.8° and 20 kPa for S1, 30.5° and 21 kPa for S2, and 35.0° and 30 kPa for S3. Test results show that the restraint of the steel frame enhances the bite force between the gravel particles in the shear process. Moreover, the rigid shear displacement constraint set upon the lower frame makes the test pile cut along the control shear surface, thereby increasing the shear strength. Therefore, the test parameters are higher than those under working conditions. To ensure consistency between the test conditions and the working conditions of the piles, this work proposes the following suggestions for improving test measures: 1) the pile section should not be encased below the shear plane and should be kept buried in soil; 2) some soil should be retained around the pile in the steel frame above the shear plane.

Keywords: vibroflotation gravel pile; in-situ direct shear test; test method; internal friction angle; composite foundation
1. Introduction

The treatment method for vibroflotation gravel piles for soft soil foundations involves replacing the part of soft soil with gravel materials to form a composite foundation. This method can improve bearing capacity, reduce settlement, and enhance the antisliding stability of the foundation while using gravel piles as antisliding piles.

The Pribe\textsuperscript{[1]} formula and the formulas recommended by relevant domestic codes\textsuperscript{[2,3]} are widely applied in engineering practice to calculate the equivalent internal friction angles and cohesion of composite foundations on the basis of the internal friction angles and cohesion of pile materials and soil between piles, as well as the load ratio of pile to soil or stress concentration factors. Li Jinyuan\textsuperscript{[4]}, Zhang Wei\textsuperscript{[5]}, and Fan Guangli\textsuperscript{[6]} applied recommended formulas to calculate the shear strength parameters of composite foundations and analyzed foundation stability using parameters. Chen Jian\textsuperscript{[7]} applied the Pribe formula to calculate the parameters and analyzed its reliability by conducting numerical simulation and in-situ deformation monitoring. The differences in the shear strength parameters of pile fillers and finished piles and the compatibility of pile–soil shear deformation are important factors affecting the reliability of this type of formula. Jiang Jiwei\textsuperscript{[8]} performed triaxial tests on a vibroflotation pile model and gravel filler pile and deduced that the shear strength parameters of the pile were lower than those of the gravel filler. Wang Jiahui\textsuperscript{[9]} conducted direct shear tests with overlapping rings on a gravel pile composite foundation and deduced that the shear deformations of the pile and soil were consistent with the process of shearing with large deformation.

As a result of the complexity of granular materials’ shear strength, the calculation of the shear strength parameters of composite foundations using existing formulas involves a certain degree of uncertainty. Therefore, in-situ shear tests on gravel pile composite foundations offer a promising solution. However, such type of test has rarely been applied, and the test method and the reliability of the derived parameters are not clearly understood. Wang Shengyuan\textsuperscript{[10]}, Chen Xinhua\textsuperscript{[11]}, and Fan Guangli\textsuperscript{[6]} performed in-situ tests to determine the shear strength parameters of composite foundations. Wang Shengyuan used a concrete plate to restrict the shear displacement of a pile or pile–soil compound body below the shear plane and then obtained the internal friction angles of the pile and compound body as $47^\circ$ and $42^\circ$, respectively; the latter was only 8.5% lower than the former. Chen Xinhua obtained the parameters as $37.2^\circ$ and $22^\circ$ and explained that the parameter of the compound body was too small because the shear strain of the soil was far greater than that of the pile, which cannot easily play its due role as a result.

This study introduces the methods of the in-situ direct shear test on vibroflotation gravel piles in the soft soil foundation of a reservoir bank treatment project and the influence of test conditions on test parameters. The reliability of the parameters is analyzed, and suggestions for improving test measures are proposed.

2. Foundation soil and gravel pile

2.1. Foundation soil

Three groups of in-situ direct shear tests on vibroflotation gravel piles, namely, S1, S2, and S3, were performed. Each group comprised five specimens. The characteristics of the foundation soil in each test area were as follows:

Area S1: Silty clay, brown gray–tawny–brown, soft plastic–plastic, some sand and mudstone; called low liquid limit clay according to particle size distribution. Dry density, 1.55 g/cm$^3$; corresponding compactness, 0.774.
Area S2 and S3: Muddy silty clay, brown gray–light gray, flow plastic–soft plastic, recent alluvial deposit, relatively pure quality; called silty sand according to particle size distribution. Dry density, 1.88 g/cm$^3$; corresponding compactness, 0.878.

2.2. Gravel piles

The characteristics of the gravel piles were as follows: design diameter of 100 cm, artificial stone filler, and maximum design particle size of 15 cm. For S1, the characteristics were pile spacing of 2 m, dry density of 2.026 g/cm$^3$, and corresponding compactness of 0.901. For S2 and S3, the characteristics were pile spacing of 1.5 m, dry density of 2.031 g/cm$^3$, corresponding compactness of 0.897. The grain gradation is shown in Table 1.

| Test Number | Mass Percentage of Each Particle Size Range (%) |
|-------------|-------------------------------------------------|
| Test Number | 100–150   60–100   40–60   20–40   10–20   5–10   2–5   1–2   0.5–1   0.25–0.5   0.1–0.25   <0.1 |
| S1          | 27.90     28.92    9.36     10.67    7.83     6.03     2.27   1.26   0.78     2.23   1.85     0.91 |
| S2, S3      | 30.54     30.15    10.09    10.60    5.54     3.82     1.59   1.32   0.61     1.92   2.99     0.84 |

3. Test method

3.1. Specimen

More than 15 days after the completion of the gravel pile construction, the pile body and the soil between the piles within a 2 m depth range were excavated to form the test pit. A water well was dug at the bottom of the pit to pump out the seepage water and keep the test pit free of ponding.

The piles of S1 and S2 were encased with one steel square frame. For S1, the frame had a height of 50 cm and side length of 100 cm. For S2, the frame had a height of 40 cm and side length of 70 cm. The top of the piles was covered with the frame, and excavation was performed manually along the pile circumference. The frame was pressed down simultaneously such that it sunk gradually until it covered the pile body. Then, the soil around the frame was removed to form a specimen.

For S3, the piles were encased with two steel square frames that were stacked up and down. The upper and lower frames were 40 and 20 cm tall, respectively, and had side lengths of 70 cm. A wide gap of 3.8–5 cm between the two frames was reserved.

3.2. Test equipment and installation

A hydraulic jack was used to apply the load, and a large range dial indicator was used to measure displacement.

The platform was built as a reaction body of normal pressure, and the buttress of the platform was used as the reaction body of the shear force.

The shear jacks of S1 and S2 were installed at the lower edge of the frame, and the pile below the shear plane was buried in the soil in its original state.
The shear jacks of S3 were installed at the lower edge of the upper frame. In the opposite direction of the shear jacks, a steel support was set upon the lower frame to avoid the horizontal displacement along the shear direction during the shear process. The equipment installation of S3 is shown in Figure 1.

![Figure 1. Test equipment installation of S3](image)

(a) Sketch map  (b) Real scene

1–pile; 2–liquid jack; 3–dial indicator; 4–roller row; 5–steel frame;
6–I-beam; 7–concrete counterweight; 8–concrete buttress; 9–steel support

3.3. Loading

The maximum normal pressures of S1, S2, and S3 were 520, 510, and 612 kPa, respectively. The shear force was applied step by step after the positive pressure was applied for 2 h, and the stable time of each step was 5 min.

The damage criteria were as follows: The shear force could not be maintained, or the shear displacement reached 1/10 of the side length of the shear surface (10 cm for S1 or 7 cm for S2 and S3).

4. Test results

4.1. Displacement

The average shear displacements before the last shear load step was applied were 4.5 cm for S1, 4.2 cm for S2, and 3.8 cm for S3; the average shear strains were 4.5%, 6.0%, and 5.4%, respectively.

An example of the displacement curve is shown in Figure 2.

![Figure 2. Example of displacement curve](image)
4.2. Shear strength parameters

The plastic shear contraction failure characteristics did not show an obvious peak value for shear stress. The maximum shear stress was taken as the shear strength. The shear strength vs. normal pressure curve is shown in Figure 3. The Mohr–Coulomb parameters are shown in Table 2.

![Figure 3. Shear strength vs. normal pressure curve](image)

**Table 2. Shear strength parameters**

| Test Number | Friction Coefficient | Internal Friction Angle, $\phi(^\circ)$ | Cohesion, $C$ (kPa) |
|-------------|----------------------|----------------------------------------|--------------------|
| S1          | 0.39                 | 22.4                                   | 20                 |
| S2          | 0.59                 | 30.5                                   | 21                 |
| S3          | 0.70                 | 35.0                                   | 30                 |

5. Influence of test conditions

5.1. Influence of steel frame

The hollow cross sections of the frames of S1 and S2 measure 100 cm × 100 cm and 70 cm × 70 cm, respectively. The design diameter of the piles is 100 cm. Thus, the four corners of the shear surface of S1 contain soil mass, and the average area ratio of gravel to shear surface is 87%; meanwhile, the surface of S2 contains no soil mass.

Under the assumption that gravel bears the entire load and that soil stress is 0, the normal pressure and the corresponding shear strength of the pile are calculated on the basis of the gravel area. Then, the modified shear strength parameters of S1 are obtained as $\phi = 22.4^\circ$ and $C = 20$ kPa, which are
27% and 5% lower than those of S2, respectively. This result reveals that the soft soil wrapped in the frame causes the gravel filler to loosen in the shear process, weakens the bite force between the gravel particles, and reduces the shear strength. Meanwhile, the steel frame increases the shear strength of the pile.

5.2. Influence of shear displacement constraint

The S2 pile below the shear plane is buried in soil without a steel frame while the pile section below the shear plane of S3 is encased with a steel frame and rigidly supported in the opposite direction of the shear force. The other test conditions of S2 and S3 are the same. The internal friction angle and cohesion of S2 are 13% and 30% lower than that of S3, respectively. In addition to the steel frame enhancing the bite force between the gravel particles, the differences in parameter values are also explained as follows: for S2, the shear displacement occurs in a certain depth range below the shear plane; for S3, the pile is cut along the control shear surface. This result reveals that the shear strength increases when the pile section below the shear plane is constrained by shear displacement.

The two failure modes are shown in Figure 4.

(a) S2: shear displacement occurs in a certain depth range
(b) S3: cut along the control shear surface

Figure 4. Failure mode diagram

6. Conclusions and suggestions

Three groups of in-situ direct shear tests on vibroflotation gravel piles are conducted. The pile diameter is 100 cm, and the pile filler is artificial sandstone gravel with a maximum particle size of 15 cm. The specimens are formed by applying a steel square frame to encase the top section of the piles. The conclusions and suggestions are as follows:

1) The shear failure of the piles is a type of plastic shear contraction failure mode with more than 4.5% shear strain and no peak shear stress.

2) The internal friction angles and cohesion of the three groups are as follows: 22.4° and 20 kPa for S1, 30.5° and 21 kPa for S2, and 35.0° and 30 kPa for S3.

3) As the piles encased by the steel frame and the rigid shear displacement constraint applied to the section below the shear plane increase the shear strength of the piles, the parameters derived from the tests are higher than those under working conditions.

4) For objective test results, the pile section below the shear plane should not be encased by a steel frame and should be kept buried in soil. Moreover, a small amount of soil around the pile should be
retained in the steel frame above the shear plane (assuming that the stress of the soil is nil during calculation).

Acknowledgments
The first author acknowledges the support from The Basic Research Fund for Central Research Institutes of Public Causes (No. CKSF2019434/SL) and The National Natural Science Foundation of China (No. 51779018).

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