Research Article

Qingsong Zheng*

Embedded deformation of the rubble-mound foundation of gravity-type quay walls and influence factors

<https://doi.org/10.1515/geo-2020-0301>

received January 25, 2021; accepted September 20, 2021

Abstract: In this study, the embedded depths of the rocks of the single-layer placement group and the multilayer riprap group were compared through a self-made test device. Besides, a finite element model of a single layer of regular rocks was developed to probe into the factors that impact the embedded deformation of the rubble-mound foundation of a gravity-type quay wall, including the cohesion and angle of internal friction of foundation soil, as well as the particle size of and the interval between rocks. Research results indicated that it was feasible to replace multilayer randomly dumped rocks with single-layer regularly placed rocks. The embedded depth decreased as the cohesion and angle of internal friction of soil increased. The influence of load on the embedded depth was small when the cohesion was less than 10 kPa or the angle of internal friction was smaller than 40°. When the load was lower than 80 kPa, the impact of the particle size of rocks on the embedded depth would be negligible, and such impact grew along with the load. When the load was below 100 kPa, the interval between rocks exerted relatively minor influence on the embedded depth: under a constant porosity, the embedded depth would decrease as the interval increased, and this correlation became more significant when the load exceeded 800 kPa; under varied porosities, a larger interval would lead to an increased embedded depth but the increase would be limited within a range of 10% of the original depth.

Keywords: rubble bedding, embedded depth, model test, computing method, cohesion

1 Introduction

A rubble-mound foundation is composed of rock particles at specific gradation and the pores between these particles. It is prone to settlement and deformation under load due to sliding, rolling, crushing, gap filling, and reorganization of rock particles. What's worse, the thicker the rubble-mound foundation, the more the rock particles and the more obvious the embedded deformation caused by the above microscopic changes in rock particles. Such deformation seriously affects the stability of the deep foundation and the safety of the superstructure, haunting project investigators and designers. To find solutions, scholars in China and abroad have conducted extensive research on the deformation characteristics of the rubble-mound foundation from two aspects: the compression deformation of the foundation and the deformation of the rocks.

With regard to the deformation of the foundation, Araei et al. [1] discussed the stress–strain characteristics of rockfill materials under the conditions of monotonic and cyclic loading. Fu et al. [2–4] revealed the residual strain of rockfill materials under different initial stress and dynamic loading and deemed that the creep characteristics of rockfill materials should not be ignored when estimating the deformation of the dam body. Moreover, Zhang et al. [5] proposed the size effect of rockfill materials on embedded depth. Li et al. [6] built a riprap made of randomly dumped rocks and performed a numerical simulation of the bearing capacity of the bed. Additionally, Zhang et al. [7] analyzed the influence of a soft-rock riprap on the stress and deformation of a rockfill dam with a core height. Huo et al. [8] simulated the deformation and stress rules of a rockfill dam at the stages of completion and water retention.

Researchers have also probed into deformations of rock particles. Xiao et al. [9–12] explored the crushing characteristics of a single rock particle and the component of rockfill materials through crushing and compression

* **Corresponding author: Qingsong Zheng**, General Institute of Fujian Communications Planning & Design Institute Co., Ltd., Fuzhou, Fujian, 35004, China, e-mail: qingsongz01@outlook.com

tests and deliberated over the impact of the intermediate principal stress on the intensity and expansion characteristics of rockfill materials. Yu and Su [13] simulated the process of particle crushing arising from high-pressure soil shear. Besides, Gupta [14] described the influence of the dimension and confining pressure of rockfill materials on the shear deformation of particles during crushing. Chen [15] delved into the study on the crushing pattern, deformation degree, and influence of the deformation of the corner angle. Han et al. [16] conducted a numerical simulation of a biaxial compression test of rockfill materials under the stress path. In addition, Zhu et al. [17] created a generalized plastic model and discussed the crushing characteristics of rock particles at different gradations. Guo et al. [18] established a prototype model for compression tests, observed the crushing of rocks by applying load hierarchically, and finally obtained the stress–strain curve and deformation parameters of rocks [9–18].

However, for a gravity-type quay wall with a deep foundation trench, rocks at the bottom of the rubble-mound foundation are under high loads. Meanwhile, the contact area of a single irregular rock and the foundation soil is small, resulting in local stress concentration. Consequently, foundation soil is partially unbalanced and destroyed before being squeezed out. Rocks “penetrate” the foundation soil, leading to embedded deformations. Such embedded deformation changes the stiffness coefficient of the rubble-mound foundation, resulting in the redistribution of the inner stress of the foundation. Yang [19] pointed out that embedding rocks of the rubble-mound foundation into the foundation soil are similar to the process of dumping stones to squeeze mud out for reinforcement. Yan et al. [20,21], based on finite element simulation and the ultimate balance theory, explored the relationship between the thickness of mud and the height of rocks and deduced a formula to calculate the embedded depth of rocks.

In short, currently, only two types of deformations are considered in riprap design and calculation. However, little attention has been paid to why rocks are embedded into the bottom of the rubble-mound foundation and lead to deformation and how to calculate such deformation. A few literature articles studied the influence of rocks parameters on embedding deformation. Therefore, this study integrated model tests and theoretical analysis to probe into the embedding of the rubble-mound foundation as well as the influences of the intensity of foundation soil and the shape of rocks on the embedded depth.

2 Materials and methods

2.1 Lab test

2.1.1 Materials

A multilayer riprap group and a single-layer placement group were designed in accordance with the number of rock layers. For the multilayer riprap group, rocks weighing 10–100 kg each were randomly dumped. The porosity of rocks was between 30 and 40%. This group was designated to simulate the embedded deformation of the rocks at the bottom of the rubble-mound foundation during actual engineering. For the single-layer placement group, rocks were placed on soil in accordance with certain rules. This group aimed to identify the influences of the porosity of rocks and load on the embedded deformation of rocks. Each sample consisted of the bottom and the top. The bottom was sand with a thickness of 1–1.2 m, simulating the natural foundation, and its physical and mechanical properties are shown in Table 1; the top contained 1–2 layers of rocks, simulating the rocks at the very bottom of the rubble-mound foundation. See Figure 1 for the two sample models.

2.1.2 Instruments

2.1.2.1 Lab device

A multifunctional structural test device was independently designed commensurate with the proposed test, as shown in Figure 2. This test device was mainly made of steel, including a base, pillars, a pulley system, a capping beam, a barrel, and a load basket. Its maximum loaded pressure was up to 300 kN and the highest dimension was 2.5 m × 2.0 m × 1.5 m. Featured a clear principle of measurement, simple operation, reliable measuring results, and low costs, this test device could be used to simulate the stress of the rubble-mound foundation of a gravity-type quay wall and measure the mechanical and deformation parameters of rocks.

2.1.2.2 Displacement gauge

An omnidirectional displacement gauge was customized by Hunan Beidou Xingkong Automation Technology Co.,

Table 1: Physical and mechanical properties of sand

Internal friction angle Φ_c (°)	Nonuniformity coefficient C_u	Curvature coefficient C_c	Median particle size D_{50} (mm)	Maximum dry density ρ_{dmax} (g cm ⁻³)	Minimum dry density ρ_{dmin} (g cm ⁻³)	Maximum void ratio e_{max}	Minimum void ratio e_{min}	Water content ω (%)
28	5.5	1.122	0.61	2.0	1.763	1.495	0.750	9.63

Figure 1: Samples of the (a) multi-layer riprap group and (b) single-layer placement group.

as shown in Figure 3. Its works in the temperature range- from -20 to $+60^{\circ}\text{C}$. The resolution was $0.1\text{--}500\text{ mm}$. Characterized by low power consumption, high sensi- tivity, real-time reporting, and a solid structure, the device is applicable to the detection of displacement of soil and layered settlement of the profile.

2.1.2.3 Jacks

STB100-57 manual hydraulic jacks produced by Jiangsu Yangzi Hydraulic Machinery Manufacture Co. were adopted, as shown in Figure 4. The jack could be perfectly used in a narrow place, thanks to its characteristics of a new structure, superior quality, a great jacking force, and small volume. Its parameters included the following: elevating capacity, 100 t ; stroke, 57 mm ; inner and outer diameters of the cylinder, 180 and 140 mm , respectively; working pressure, 63 MPa ; and the outer diameter of the piston, 100 mm .

2.1.3 Experimental process

2.1.3.1 Preparation and placement of samples

First, coarse sands were bagged and dumped in the barrel before saturated with water and being compacted. Its thickness was measured and recorded. Then, we num- bered, weighed, and recorded the rocks before dumping (the multilayer riprap group) or placing (the single-layer placement group) them on the sands. The porosity of rocks was adjusted by changing the number and interval of rocks. Last, the initial thickness of the rocks was mea- sured and recorded.

Figure 2: Schematic diagram of test device: (a) schematic diagram of the multifunctional structural test device and (b) photo of the multifunctional structural test device.

Figure 3: Displacement gauge.

Figure 4: Jack schematic.

2.1.3.2 Loading and data measurement

First, the barrel was covered with a bearing plate and the capping beam was placed on the bearing plate. Then, we got the displacement gauge fixated to the edge of the barrel and read the initial height of the bearing plate.

Next, four 100 t manual hydraulic jacks were symmetrically placed between the capping beam and the bearing plate. The capping beam was put down by pulling the hand-drive block and the steel wire ropes in the pulley block were pulled taut. The load basket was mounted to begin loading. Here we lifted up and pushed down the rocking bar of the hydraulic oil pump and added oil into the jacks for pressurization, which then jacked the beam up and drove the pulley block fixated to the beam to lift the load basket. At this moment, we stopped the oil and recorded the reading on the oil pump for 10 min. Now, Level 1 loading was completed. Then, we recorded the reading on the oil pump and the displacement gauge. The difference between this reading and the previous one gave the embedded depth of the rocks at Level 1 loading. The next-level loading could be prepared after the embedded depth was recorded. The above steps were repeated till loading was fully completed.

2.1.3.3 Rocks were taken out to measure the depth of the pit

Upon completion of loading, the rocks in the barrel were taken out. For the multilayer riprap group, rocks were reweighed and recorded. But, this step was unnecessary for the single-layer placement group. The depth of the rocks embedded in the sand (i.e., depth of the pit) was measured, while the rocks were taken out.

2.1.4 Experimental content

Since multiple layers of rocks are randomly dumped in real-world scenarios, factors like porosity are difficult to

Table 2: Test content

Group	Multilayer riprap group	Single-layer placement group
Test process	Separate compression test Thickness of the sand layer: 1 m; thickness of rocks: 0.565 m; porosity of rocks: 37% The embedded depth of rocks was calculated	Separate compression test Thickness of the sand layer: 1 m; thickness of rocks: 0.4 m; porosity of rocks: 37%
Test results	The embedded depths of the two groups were compared	Overall compression test Hierarchical loading

control. Therefore, the two groups were designed to simplify the model and keep influence factors under control and analyze the feasibility of replacing complicated multilayer randomly dumped rocks with simple single-layer regularly placed rocks. First, a separate compression test was performed on the layers of rocks and sands, respectively. Second, an overall compression test was conducted on the two groups. Third, the embedded depths of the rocks of the two groups were compared, as shown in Table 2.

2.1.5 Embedding test of multilayer randomly dumped rocks

2.1.5.1 Compression test

(1) Compression test of rocks. We randomly selected 10–100 kg of rocks for the rubble-mound foundation of a gravity-type quay wall and numbered them before being put in the barrel of the test device, as shown in Figure 5(a). The initial average height of the rocks prior to the test was 1.721 m. A hierarchical loading test was performed to record the displacement.

(2) Compression test of the sand bed. Sands were put in the barrel and leveled. The height was 1 m, as shown in Figure 5(b). A hierarchical loading test was performed to record the displacement.

(3) Overall compression test of the multilayer riprap group. As shown in Figure 5(c), sands were put in the barrel and leveled. Then, we placed the rocks neatly after the thickness of the sand bed reached 1 m. A hierarchical loading test was carried out when the average thickness of the rocks turned 0.564 m. The displacement was recorded.

2.1.5.2 Embedded depth

For the multilayer riprap group, the total displacement measured included three parts: the volume of compression of the rock layer, the embedded depth of rocks into the sand bed, and the volume of compression of the sand bed. As shown in Figure 6, H_A is the total height of the sample, H_B is the height of rocks, H_C is the height of the sand layer, and H_D is the embedded depth of rocks. The relationship between these parameters is as follows:

$$H_D = \Delta H_A - \Delta H_B - \Delta H_C, \quad (1)$$

where ΔH_A is the total volume of compression of the sample (mm); ΔH_B is the volume of compression of

Figure 5: Schematic diagram of the compression test: (a) block stone stacking, (b) sand stacking, and (c) overall material stacking.

Figure 6: Schematic diagram of the overall compression test of rocks and the sand bed (a) before loading and (b) after loading.

rocks (mm); and ΔH_C is the volume of compression of the sand layer (mm).

2.1.6 Embedding test of single-layer regularly placed rocks

In the overall compression test of rocks and the sand bed, the randomness of dumping of multilayer of rocks resulted in varied parameters of rocks, such as porosity, and made it difficult to analyze the data. Hence, multilayer randomly dumped rocks were replaced with single-layer regularly placed rocks.

2.1.6.1 Compression test

Regular rocks, similar to a quadrangular shape, with a height of 400 mm were utilized. The thickness of the sand bed was still 1 m, and the schematic diagram and actual arrangement of materials are shown in Figure 7. As with the multilayer riprap group, the porosity was controlled

to be the same as that of multilayer randomly dumped rocks by adjusting the interval of rocks (37%). Then, a hierarchical loading test was carried out and the displacement was recorded.

2.1.6.2 Embedded depth

There was basically no crushing or displacement, attributable to the big elasticity modulus of the rocks. For the single-layer placement group, changes in H_B were negligible, as shown in Figure 8. Thus,

$$H_D = \Delta H_A - \Delta H_C. \quad (2)$$

2.2 Finite element simulation

2.2.1 Coupled Euler–Lagrange (CEL) method

ABAQUS is powerful finite element software. For multi-component problems, it can define material parameters and divide meshes for each component and then

Figure 7: Arrangement of single-layer regularly placed rocks: (a) schematic diagram of arrangement and (b) actual arrangement.

Figure 8: Schematic diagram of the embedded depth of the single-layer placement group in the compression test (a) before loading and (b) after loading.

compose them into a complete model. Therefore, ABAQUS is used to establish the finite element of the cylindrical boulder embedded in the soil. Because the parameters involved in the Mohr–Coulomb model are simple and easy to determine, and can better reflect the different yield and failure strengths of soil materials under tension and compression conditions, the Mohr–Coulomb model is selected as the constitutive model of the soil in this article.

The traditional method of Lagrange multipliers relies on mesh deformation. Mechanical results are solved based on the displacement of nodes. However, convergence is difficult in the analysis of large deformations. With regard to the Euler method, nodes are fixated on the space, and materials flow in a unit that does not deform. Thus, the material boundary and the unit boundary are usually inconsistent and should be calculated in each increment. The coupled Euler–Lagrange method combines the advantages of both methods above. Leveraging the advantages of the Euler method that the mesh is fixed and materials freely flow in the mesh, we can build a model to address the problem of large deformation and material destruction. The stress–strain response of the structure is solved through the Euler–Lagrange contact algorithm and the Lagrange mesh.

Figure 9: Schematic diagram of the arrangement of rocks at different porosities.

Figure 10: Finite element model and meshing (unit: m): (a) overall structure of the finite element model and (b) meshing of the model.

2.2.2 Geometric dimension

The calculation model was a 3D finite element model, including a bearing plate, rocks, and a sand layer. In the model, the bearing plate was a cylinder with a basal diameter of 1.5 m and a height of 0.2 m. The foundation (sand layer) was simulated by a cylinder with a basal diameter of 1.5 m and a height of 1 m. The pile of rocks was simulated as a small cylinder with a basal diameter of 0.28 m and a height of 0.4 m, as shown in Figure 9, 19, 18, 16, and 12 small cylindrical rocks would be used in the model, when the porosity (n) was 35, 37, 47, and 59%, respectively.

2.2.3 Meshing of the model and boundary conditions

Figure 10(a) shows the overall structure of the finite element model. Models of rocks and the sand layer employed C3D8R and EC3D8R, respectively. The meshing adopted both the structured and sweep meshing techniques. Rocks and the bearing plate with large stiffness and small deformation were sparsely meshed, while the sand layer, were meshed densely, so as to increase the computational efficiency. Figure 10(b) shows the meshing scheme.

Table 3: Physical and mechanical parameters of each layer of materials

Model	Density (g cm^{-3})	Elasticity modulus (MPa)	Poisson's ratio	Cohesion, c (kPa)	Angle of internal friction, φ ($^{\circ}$)
Bearing plate	7.830	2.06×10^5	0.304		39
Sand layer	1.300	30	0.25	0.1	28
Rocks	2.286	3×10^4	0.3		45

As the deformation of the steel barrel on the testbed was extremely small and could be ignored, the barrel was replaced with boundary conditions to restrict the horizontal displacement of each layer of materials. For the side of the bearing plate, rocks, and the sand layer, the boundary speeds along the x - and y -axis were set as 0, while that along the z -axis was not limited. For the bottom of the foundation, the boundary speed along the z -axis was set as 0, while those along the x - and y -axis were unlimited.

2.2.4 Setting of parameters and the contact surface

With respect to the traditional method of Lagrange multipliers, it is difficult to achieve convergence in the analysis of large deformations. The CEL method integrates the strengths of both the Lagrange and Euler meshes, thus effectively tackling large deformation and material destruction. As a result, it was employed to simulate the embedding of rocks into the sand bed. Meanwhile, the Mohr–Coulomb model was used to describe the stress–strain relation of soil. Table 3 presents the parameters of all materials.

Hard contact was adopted as the normal contact of rocks and the sand bed during embedding. The penalty function was used for the tangential contact. The friction coefficient between the sand bed and rocks was 0.6.

3 Results and discussions

3.1 Comparison of the multilayer riprap group and single-layer placement group

According to equation ((1)) and equation ((2)), the embedded depths of the multi-layer riprap group and single-layer placement group were calculated and compared (the porosity is the same), as shown in Figure 11. It can be seen from the figure that the embedding depth increased with the increase of load. When the load was 200 kPa, the embedded depth of the multilayer riprap group was approximately 20 mm, while that of the other group doubled to 40 mm. The main reason

is that the overall compression of the single-layer placement group was mainly derived from the crushing of rocks. In contrast, almost no rock of the single-layer placement group was crushed. Hence, the multilayer riprap group had a higher embedded depth at the beginning of loading and there was a big difference in the test results of the two groups. That said, as the load augmented, the consistency between the overall trends of the two curves gradually increased, and the final error was less than 10%. The primary cause was that the overall compression was mainly caused by embedding after the crushing of rocks in the multilayer riprap group. Hence, under the same porosity, it was reasonable to replace the multilayer randomly dumped rocks with the single-layer regularly placed rocks in the embedding test as the load grew. Single-layer regular rocks were adopted in the numerical simulation to simplify the model.

3.2 Precision verification of numerical simulation

3.2.1 Precision verification of the compression test of the sand bed

The vertical displacement in the numerical simulation and single-layer placement test results (see Figure 12)

Figure 11: Comparison of embedded depths of the multilayer riprap group and the single-layer placement group ($n = 37\%$).

Figure 12: Comparison between the test and simulated results of the sand bed.

of the sand bed were compared. Figure 12 demonstrates that the test result tallied with the simulated result, with the maximum difference less than 3%.

3.2.2 Precision verification of the embedded depth at varied porosities

Figure 13 shows the comparison between the simulated value of the finite element model and the embedded deformation of the test with different porosities.

As shown in Figure 13, the increase in the simulated embedded depth was divided into two stages as well, namely, the stage of rapid growth and the stage of slow growth. In the first stage, the simulated value was larger than the experimental value when the load was less than

Figure 13: Comparison between the simulated value of the finite element model and the embedded depth of the test. (a) $n = 35\%$, (b) $n = 37\%$, (c) $n = 47\%$, and (d) $n = 59\%$.

250 kPa. Especially in the stage of just applying the load, the simulated value of the embedded depth increased rapidly, while the growth rate of the experimental value was relatively small. This is mainly because the numerical simulation had ideal conditions and the stones were placed in order. Under the test conditions, the rock had a recombination process when the load was just applied, and various uncertain factors caused the simulation value of the embedded depth to increase at a greater rate than the test value. When the recombination was stable, the test value and the simulation value change law tended to be consistent, and the difference in the embedding depth between the two was small, which was basically kept within 10%. Therefore, the finite element model can be considered to be reliable. The above results can also show that both the numerical simulation and the self-made test device can form mutual verification, and the self-made multifunctional structure test device can be used to test the deformation of the rubble bed approximately.

3.3 Analysis of influence factors of embedded deformation

As mentioned above, the embedded depth of rocks is influenced by porosity, which is associated with the particle size and the interval of rocks. Besides, when rocks are embedded into the foundation soil, the corresponding amount of the foundation soil will be squeezed out (see Figure 14). Thus, the angle of internal friction and cohesion of the foundation soil also affect the embedded deformation of rocks.

The vertical displacement–load curve was calculated, based on the finite element technology to analyze the influences of factors such as the intensity of the foundation soil, the particle size, and the interval of cylindrical rocks on the embedded depth.

Figure 14: Schematic diagram of rocks being embedded into foundation soil: (a) before embedding, (b) during embedding, and (c) after embedding.

3.3.1 The cohesion of the foundation soil

Consider, for instance, a porosity of 37% (see Figure 9 for the schematic diagram of rock arrangement). The cohesion (c) values of the foundation soil were set as 0, 5, 10, 15, and 30 kPa, respectively, to perform five groups of analog computation. The values of other parameters are the same as those in Table 1. Figure 15 shows the changes in the embedded depth with the load at different cohesion values.

As shown in Figure 15, the embedded depth decreased as the cohesion increased. When the cohesion was below 10 kPa, its influence on the embedded depth was high. The embedded depth nearly halved by every 5 kPa increase in cohesion. When the cohesion was higher than 10 kPa, the differences in the embedded depth at different cohesion values were small. When cohesion remained constant, the embedded depth first surged along with the load, then slowed down, and finally, reached a stable value. A larger cohesion corresponded to a smaller load when the embedded depth reached a stable value. For instance, when the cohesion was above 10 kPa, the corresponding load was approximately 500 kPa; when the cohesion was 5 kPa, the corresponding load was approximately 600 kPa.

3.3.2 The angle of internal friction of the foundation soil

Consider, for example, a porosity of 37% (see Figure 9 for the schematic diagram of rock arrangement). The angles of internal friction, ϕ , of the foundation soil were set as 10°, 20°, 30°, 40°, and 45°, respectively, to conduct five groups of analog computation. The values of other parameters are the same as those in Table 1. See Figure 16 for the embedded depth–load curve based on the finite

Figure 15: Embedded depth–load curve at different cohesion values.

Figure 16: Stress-strain curve and the fitted curve.

Table 4: Number of different cylinders and porosities

Basal radius of the cylinder	30 cm	28 cm	26 cm	24 cm	20 cm
No. of cylinders	12	14	16	19	27
Porosity (%)	51.9	51.2	51.9	51.3	51.9

element calculation at different angles of internal friction of foundation soil.

Figure 16 shows that a higher angle of internal friction of the foundation soil corresponded to a smaller embedded depth of rocks. Meanwhile, the embedded depth turned less sensitive to the angle of internal friction. For example, when the angle of internal friction increased from 10° to 20° , the embedded depth would decrease 3 times. However, when the angle of internal friction increased from 40° to 50° , the decrease in the embedded depth would approach zero. When the angle of internal friction remained unchanged, the embedded depth would increase along with the load and finally become stable. When the load reached 1,000 kPa, the angle of internal friction of 10° would correspond to an embedded depth of 165 mm, while the angle of internal friction of 50° would correspond to an embedded depth of only 25 mm.

Figure 17: Schematic diagrams of the arrangements of cylindrical blocks with different diameters.

3.3.3 The particle size of rocks

The particle size of rocks was described with the basal radius of the cylinder. The basal diameters, d , of the cylinder were set as 30, 28, 26, 24, and 20 cm, respectively. See Table 1 for the values of other parameters, Table 4 for the number of cylinders with different diameters and porosities, and Figure 17 for the schematic diagram of the specific arrangement.

Figure 18 shows the changes in the embedded depth with the added loads at different particle sizes.

As shown in Figure 18, when the porosity remained constant, the higher the particle size, the higher the embedded depth and the influence of the particle size on the embedded depth became increasingly obvious as the load increased. That said, when the particle size was less than 28 cm, the particle size exerted a small influence on the embedded depth. When the load was around 80 kPa, a notable turning point appeared on the curve. The influence of the particle size of rocks on the embedded depth was basically negligible prior to the turning point. Yet, such influence became increasingly obvious after it. When the load reached 800 kPa, the maximum growth

Figure 18: Correlation between the embedded depth and the load at different particle sizes.

Figure 19: Schematic diagrams of rock arrangements at different intervals (under the same porosity).

Figure 20: Embedded depth-load curves at different intervals of rocks (under the same porosity).

rate of the embedded depth along with the particle size would surpass 30%. When the particle size of rocks remained constant, the embedded depth increased dramatically along with the load and then slowed down to increase steadily. The increase was almost linear.

3.3.4 Interval of rocks

3.3.4.1 Same porosity

Consider, for instance, a particle size, d , of 20 cm. The intervals, l , of cylindrical rocks were set as 2, 4, 6, 8, and

10 cm, respectively. The number of cylinders was 19 each. In other words, the porosity of the rocks was the same. The values of other parameters were identical to those in Table 1. Figure 19 shows the schematic diagrams of the specific arrangements.

Figure 20 demonstrates the embedded depth-load curve when the porosity was the same and the intervals of rocks were different.

As shown in Figure 20, when the porosity was constant, the embedded depth gradually decreased as the interval of rocks increased. Yet, the growth rate was small. The embedded depth-load curves were almost identical when the intervals of rocks were different. In addition, when the load was below 100 kPa, the impact of the interval of rocks on the embedded depth was basically negligible. When the load turned higher than 100 kPa, the impact became more obvious but was basically within 10%. Under the same interval of rocks, the embedded depth drastically increased along with the load, then slowed down, and finally, increased at a high growth rate. The curve was characterized by two turning points where the load reached 100 and 750 kPa, respectively.

3.3.4.2 Different porosities

Consider, for example, a particle size, d , of 20 cm. The interval, l , of cylindrical rocks was set as 2, 4, and 6 cm. Cylinders were placed to fill the barrel up. The corresponding porosities were 35, 47, and 58%, respectively. The values of other parameters were the same as those in

Figure 21: Schematic diagrams of rock arrangements at different intervals and porosities.

Figure 22: Correlations between the embedded depth and the load at different porosities and intervals of rocks.

Table 1. Figure 21 shows the schematic diagrams of the specific arrangements.

Figure 22 shows the embedded depth–load curves when both the porosity and interval of rocks are different.

As shown in Figure 22, when the porosities were different, the embedded depth increased along with the interval of rocks within the range of 10%, which was opposite to the situation when the porosity was constant (Figure 21). Besides, when the interval of rocks remained unchanged, the embedded depth increased significantly along with the load and slowed down when the load reached 200 kPa. Then, it maintained an approximately linear growth.

4 Conclusion

In this article, a multilayer random riprap group and a single-layer placement group experiment were designed to (1) measure the compression of the sample, the rock, and the sand layer; (2) compare the embedding depths of single-layer placement and multilayer random riprap test blocks; and (3) verify the feasibility of replacing multilayer randomly dumped rocks with simple single-layer regularly placed rocks. The calculation formula of the embedding depth was proposed according to the sample compression model. In view of the fact that the simulation model of blocks arranged regularly was easier to establish, the simulation models of single-layer placement were established using ABAQUS to verify the accuracy of the finite element model via the comparison of the simulation results and test results. Based on the verified finite element model, simulation models under different

cohesive forces, internal friction angles, rock pile particle sizes, and rock intervals were established to analyze the influence of different factors on the embedding deformation of the rubble-mound foundation of a gravity-type quay wall.

(1) When the porosity was the same, the embedded depths of the two groups were basically consistent. Therefore, it is strongly feasible to replace multilayer randomly dumped rocks with single-layer placement rocks for the embedding test.

(2) As the cohesion and the angle of internal friction of the foundation soil increased, both the embedded depth and the influence of the two factors on it decreased. When the cohesion was smaller than 10 kPa or the angle of internal friction was less than 40° , the influence on the embedded depth was high and the embedded depth kept increasing as the load increased; otherwise, the influence on the embedded depth was small. The embedded depth gradually turned stable as the load augmented.

(3) The embedded depth increased as the porosity and the particle size of rocks increased. When the load reached around 300 and 80 kPa, respectively, the embedded depth–load curves showed an apparent turning point, after which the slope of the curves was basically identical at different porosities. As the particle size increased, the slope of the curves gradually increased. When the particle size increased greater than 26 cm, the influence on embedded depth was obvious.

(4) The influence of the interval of rocks on the embedded depth was small. When the porosity was the same and the load was less than 100 kPa, the impact of stone spacing on the embedded depth could be basically ignored; when the load was greater than 100 kPa, the impact increased slightly but was basically kept within 10%. When porosities were different, the embedded depth increased along with the interval at a small range.

The purpose of this article is to reveal the influence of cohesion, internal friction angle, particle size, and the rock interval on the embedded depth under different loads. The comparison is only a means to verify the feasibility of numerical simulation. The contribution of this research is to demonstrate the feasibility of replacing multilayer randomly dumped rocks with simple single-layer regularly placed rocks for the embedding test; put forward a calculation method for the embedded depth of rocks in the foundation bed; and reveal the influence of cohesion, internal friction angle, particle size, and the rock interval on the embedded depth under different loads. Harbor, Waterway, and Coastal Engineering designers and scientific researchers may be interested in furthering this research. Future research studies should devise a large

number of instruments to collect data at the project site so that the conclusions of this research can be confirmed further.

Conflict of interest: The author states no conflict of interest.

Data availability statement: The data presented in this study are available on request from the corresponding author.

References

- [1] Araei AA, Razeghi HR, Ghalandarzadeh A, Tabatabaei SH. Effects of loading rate and initial stress state on stress-strain behavior of rock fill materials under monotonic and cyclic loading conditions. *Sci Iran*. 2012;19(5):1220–35. doi: 10.1016/j.scient.2012.08.002.
- [2] Fu Z, Chen S, Han H. Experimental investigations on the residual strain behavior of a rockfill material subjected to dynamic loading. *J Mater Civ Eng*. 2017;29(5):1–11. doi: 10.1061/(ASCE)MT.1943-5533.0001816.
- [3] Fu Z, Chen S, Zhong Q. Modeling interaction between loading-induced and creep strains of rockfill materials using a hardening elastoplastic constitutive model. *Can Geotech J*. 2019;56(10):1380–94. doi: 10.1139/cgj-2018-0435.
- [4] Milosevic J, Gago AS, Lopes M, Bento R. Experimental assessment of shear strength parameters on rubble stone masonry specimens. *Constr Build Mater*. 2013;47(oct):1372–80. doi: 10.1016/j.conbuildmat.2013.06.036.
- [5] Zhang Y-T, Chen P-S, Yong YU, Yang Z. Numerical Study on Treatment of Weak Groundsill with Method of Extruding Slit by Dumping Rockfill[C]//International Conference on Mechanics & Civil Engineering, 2014.
- [6] Li DJ, Liu WJ, Rao WS. Mechanical characteristics and dynamic compaction of rubble bed. *Sci Technol Eng*. 2019;19(15):253–7.
- [7] Zhang HH, Yuan YR. Study on effects of soft rock-contained rockfill for stress and deformation of high core rockfill dam. *Appl Mech Mater*. 2011;90–93:2516–20. doi: 10.4028/www.scientific.net/AMM.90-93.2516.
- [8] Huo XL, Wang B, Huang L. Research on deformation and stress of concrete face rockfill dam based on finite element analysis. *Adv Mater Res*. 2012;462:235–41. doi: 10.4028/www.scientific.net/AMR.462.235.
- [9] Xiao Y, Meng M, Daouadji A, Chen Q, Wu Z, Jiang X. Effects of particle size on crushing and deformation behaviors of rockfill materials. *Geosci Front*. 2018;11:375–88. doi: 10.1016/j.gsf.2018.10.010.
- [10] Xiao Y, Liu H, Chen Y, Chu J. Influence of intermediate principal stress on the strength and dilatancy behavior of rockfill material. *J Geotech Geoenviron Eng*. 2014;140(11):04014064.1. doi: 10.1061/(ASCE)GT.1943-5606.0001178.
- [11] Xiao Y, Sun Z, Stuedlein AW, Wang C, Wu Z, Zhang Z. Bounding surface plasticity model for stress-strain and grain-crushing behaviors of rockfill materials. *Geosci Front*. 2020;11(2):495–510. doi: 10.1016/j.gsf.2019.06.010.
- [12] Xiao Y, Liu H, Zhang W, Liu H, Yin F, Wang Y. Testing and modeling of rockfill materials: a review. *J Rock Mech Geotech Eng*. 2015;8(3):415–22. doi: 10.1016/j.jrmge.2015.09.009.
- [13] Yu F, Su L. Particle breakage and the mobilized drained shear strengths of sand. *J Mt Sci*. 2016;13(8):1481–8. doi: 10.1007/s11629-016-3870-1.
- [14] Gupta AK. Effects of particle size and confining pressure on breakage factor of rockfill materials using medium triaxial test. *J Rock Mech Geotech Eng*. 2016;3:378–88. doi: 10.1016/j.jrmge.2015.12.005.
- [15] Chen CP. Study on deformation of corner angle of rocks of deep foundation bed. Tianjin: Tianjin University; 2018.
- [16] Han H, Chen W, Huang B, Fu X. Numerical simulation of the influence of particle shape on the mechanical properties of rockfill materials. *Eng Comput*. 2016;34(7):2228–41. doi: 10.1108/EC-12-2016-0453.
- [17] Zhu S, Ning ZY, Zhong CX, Chu JW, Gao ZP. Study on particle crush and deformation characteristics considering rockfill gradation effect. *J Hydraulic Eng*. 2018;49(7):849–57.
- [18] Guo D, Zhu Z, Zhong C, Peng H, Wang A, Xu T, et al. Experimental study on compression characteristics and deformation parameters of the rubble bedding of gravity quays. *J Hydraulic Eng*. 2019;50(04):120–9.
- [19] Yang GX. Study on squeezing mud for reinforcement. *Chin J Geotech Eng*. 1992;14(2):72–6.
- [20] Yan H, Dong L. The impacts of land cover types on urban outdoor thermal environment: the case of Beijing, China. Calculatimethod model test penetratidepth squeezing soft clay (Stone) method. 2015;36:43–8. doi: 10.16285/j.rsm.2015.S1.008.
- [21] Yan SW, Chen J, Sun LQ, Lin S. Calculation method and model test on penetration depth of squeezing soft clay (with stone). *Rock Soil Mech*. 2015;S1:43–8.