In this paper, the failure mechanisms of large geotextile mats over soft soil are carried out through finite element analyses. A finite element model is generated and validated against centrifuge testing data and previously published data of numerical simulation. Parametric study is then carried out to investigate the geotextile tension distribution and the arrangement of crashed stone. Based on the parametric study, an optimized design considering the arrangement of rock berm and a special arrangement of large geotextiles was proposed to enhance the performance of the geotextile mats. The findings of this study can provide an engineering guidance for this new technique.

1. Introduction

In the wake of increasing incidents of structure failures at rivers and coastlines, construction of cofferdams has become an important disaster mitigation strategy (see Figures 1(a) and 1(b)) [1]. With the advantages of fast construction, high global stability, and strong adaptability to the environment, geotextile structures have been applied in the construction of dikes, coastal protection, flood control, and land reclamation projects [2].

Failure mechanisms of geotextile tubes or mats over soft clay have been investigated by analytical solution, numerical modeling, and field tests. A few analytical solutions have been derived to investigate the configurations and tensile forces of the geosynthetic containers (Cantrè et al. [3], Plaut [4], and Ghavanloo et al. [5]). Yan and Chu [6] carried out a preliminary design method for the geotextile mat cofferdam and assessed the stability of the dike before construction using both numerical simulation and centrifuge test. Guo et al. [7–10] proposed analytical methods to analyze the behavior of geosynthetic mattresses resting on rigid or deformable foundation. These studies primarily focused on materials and deformations of geosynthetic tubes and mats but ignored the settlement of underlying soils. Zhu et al. [11] carried out the parametric study to evaluate the slope stability of stacked geotextiles. A series of slope stability charts were derived for rapid evaluation of the feasibility of stacked geotextiles (Figure 1(c)).

Numerical analysis was also widely employed to investigate the properties of the geotextile container. Kim et al. [12, 13] investigated the stability of geotextile tube-reinforced reclamation embankments subjected to scouring with and without additional applied ground base modifications by the finite element method (FEM). It was found that riprap protection offered the maximum improvement to the overall performance of the stability of the geotextile tube embankment system. Sun et al. [14] investigated the impact of two types of wedges on the lateral stability of geomembrane tubes with PFC 2d and provided the optimize shape and size of the wedge.
Pavanello et al. [15] studied the tensile stress of stacked geological tubes under immersion conditions by the FE method and analyzed the influence of different factors on the shape and tensile stress of geotextiles. Górniak [16] utilized the FEM-DEM method to analyze the behavior of geosynthetics when the geotextile tube in the local sinking environment was subjected to applied load and displacement. The loading capacity and the apparent stiffness of the tube were found to increase with the filling rate.

Field tests and model tests have also been carried out. To study the characteristics of failure zones of the underlying soil for flexible foundation, Zhou et al. [17] revealed the deformation form of soft soil foundation during and after the construction of cofferdam. A new failure zone was proposed by Zhou et al. Stability analysis was carried out by Peng et al. [18] on two engineering cases. It was reported that the failure of one project was the sandbags were stacked too fast leaving the soft soil underneath not able to carry out the necessary reinforcement within the timeframe. Vahedifard et al. [19] reported a research on the utilization of geo-tubes filled with stabilized dredged sediments, in which a rock revetment was placed on top of the slope to prevent future damage.

In current engineering practice, almost all geotextile cofferdams are designed with the same thickness [10]. However, due to the large upper load, the tensile stress of the bottom geotextile mat is significantly larger than that of the upper mat which is apparently over designed. It is found that the reinforcement effect of geotextiles depends not only on the performance of geotextiles but also on the number and spacing of geotextiles [20]. A new optimized design method is proposed in this study. FE analysis is employed to investigate the geotechnical behavior of the geotextile mat cofferdams, where the effect of the crushed stone is rigorously explored. The failure mode of the cofferdam using the existing design method is analyzed, and the failure mechanism is studied.

2. Methodology

2.1. Modeling Details. In this study, the commercial software PLAXIS 2D is employed. A geotextile mat cofferdam with two berms on both sides is considered. The cofferdam and the berms sit on homogeneous soft clay. The dimension is 600 m in the horizontal direction and 150 m in the vertical direction, which eliminates the influence of boundary conditions (see Figure 2). Both the cofferdam and berms have a slope ratio of 1:2, and the widths of the cofferdam and berms are listed in Table 1. Figure 3 presents the FEM meshes used in this study, in which the underlying soil and fill material are modeled using 15-node triangular elements. In the previous studies by Kim et al. [12, 13], it is found that considering the thickness of each geo-mat and step loading of construction mesh size does not significantly influence the behavior of the dam. In this study, the mesh was generated using a fine global coarseness and locally encrypted at the cofferdam and the rockfill toes. Both the vertical and horizontal displacements are restrained at the bottom, and the horizontal displacement from the left to the right boundaries is also fixed.
2.2. Constitutive Model. The linear-elastic-perfect-plastic Mohr–Coulomb model available in PLAXIS 2D is adopted with continuum elements for soil and interface elements. This model follows Hook’s law for the linear-elastic part, and the Mohr–Coulomb failure criterion is considered for the perfectly plastic region [21]. A uniform stiffness ratio of \( E_s / E_s \) of 500 is used for the clay. The stiffness ratio is within the range commonly adopted for soft clays, but the precise value has a negligible effect on the results presented. All the analyses simulated the undrained conditions. A Poisson’s ratio \( v = 0.49 \) is used, and the friction and dilation angles are \( \phi = \psi = 0 \).

The geotextile material is modeled using the geogrid element, which only requires elastic axial stiffness (\( EA \)) as the material property. The interaction between soil and geotextile is modeled using the interface element with an interface reduction factor (\( R_{int} \)) of 0.6 to simulate the interaction between soil and geotextile [21]. It means that the limiting shear force along the interface equals \( s_n \times R_{int} \). Table 1 summarizes the parameters of the numerical model. All crashed stones and soil properties are based on results from experimental tests.

2.3. Model Validation. The FE model of this study is validated against Yan and Chu’s study [6]. The thickness of the three layers of clay from top to bottom is 5 m, 10 m, and 5 m in prototype size, respectively, with corresponding undrain shear strengths are 16 kPa, 16 kPa, and 55 kPa for each layer. The modules of the filled sand are \( E_s = 20 \text{ MPa} \), and the frictional angle of sand is \( \varphi = 30^\circ \) (\( W = 18 \text{ m}; J = 140 \text{ kN/m} \)). Other details can be found in reference [1]. Figure 4(a) compares Yan and Chu’s results and the predicted results using the numerical modeling method of this study, from which good agreements can be found in terms of the maximum displacements of locations.

The current FE model is further validated with Yan and Chu’s centrifuge testing data. A model box of 685 mm × 400 mm × 200 mm was constructed to study the failure mechanism of dike on soft soils [1]. Figure 4(b) compares the displacements between the numerical prediction using the current modeling method and the centrifuge testing results, where good agreement can be found.
The above comparisons demonstrate that the FE model of this study can capture the flow mechanisms and the potential sliding failure surface of the geotextile mat dikes over layered soils.

3. Results and Analysis

3.1. Uneven-Thicknesses Reinforcement of Cofferdams. Uneven-thicknesses reinforcement is proposed which uses an interval thickness due to continuous thickening of each layer. The first three layers of geotextile mats have a thickness of 0.2 m. The thickness then gradually increases until 1 m.

To explore the differences between uniform reinforcement and uneven-thicknesses reinforcement, a group of numerical modeling cases are simulated with cofferdam base width varying as $W_{cd} = 30, 50, 60, 80, 100$ m and with the homogeneous soil strength $s_u = 6$ kN·m. In these cases, the cofferdam slope is kept constant as $k = 1/2$, and the soil strength of foundation is $s_u = 6$ kPa.

The soil flow patterns are shown in Figures 5(a) and 5(b), where two different failure mechanisms can be observed. It is apparent that with the increase in the width of cofferdam, the soil failure mechanism transits from local failure to global failure, while the local failure mechanism shows up in a larger width of cofferdams with uneven-thicknesses reinforcement.

For flexible foundations, the greater the rigidity of the foundation, the greater the deformation range of the underlying soil caused by the foundation can be found. Due to the differences in interfacial contact stresses, the settlement of the cofferdam experiences an apparent increase, which leads to the failure of the cofferdam [22]. Compared with the uniform-reinforced cofferdams, the stiffness of bottom layers of the uneven-thicknesses reinforced cofferdam increases owing to the intensive reinforcement. Then, a large plastic zone is developed [23]. This helps to explain the formation of local failure modes with a narrower width of cofferdam.

3.2. Effect of Stiffness of Geotextiles. To further explore the effect of uneven-thicknesses reinforcement on the rigidity of the cofferdam, two groups of cases are modeled by varying the geotextile tensile stiffness of the cofferdams with uneven-thicknesses reinforcement with $J = 140, 500, 1000, 2000, 4000, 8000$ kN/m. In these cases, the cofferdam slope is kept constant as $k = 1/2$, the soil strength of the foundation is $s_u = 6$ kPa, the cofferdam base width is $W = 80$ m, and the internal friction angle of the sand fill $\phi = 30^\circ$.

Figure 6 depicts the variation of the limiting stack height and safety factor with the tensile strength of the geotextiles. The limiting stacked height could be effectively improved when the tensile strength of the geotextile increases. For example, the limiting stacked height of an 80 m wide cofferdam increases from $3.24$ m to $3.55$ m with the tensile strength of the geotextiles rising from $140$ kN/m to $1000$ kN/m. When the tensile strength of the geotechnical bag exceeds $1000$ kN·m, the limiting stack height of the cofferdams does
not increase significantly. Similar phenomenon can be seen from the results of effects on the safety factor, and the only difference is that the inflection point position is different.

As the stiffness of the geotextile increases, the restraining effect on the filler is also enhanced. The increase in the tensile strength of the geotextile is beneficial to resist the vertical deformation of the cofferdam. When the load exceeds the maximum threshold of the contact surface, the shear capability of the contact surface is invalid resulting in the slippage of the fill along the surface of the geotextile. When the load exceeds the threshold of the contact surface, the shear capacity of the contact surface will be invalid, resulting in the slippage of the fill along the surface of the geotextile. The effect of the geotextile generating the tensile force against deformation is no longer improved. Even if the tensile strength of the geotextile continues to increase, the limit of the cofferdam is not improved. Therefore, there is a threshold for the selection of the tensile strength of large sandbags rather than indefinite increase. Similar results were reported by Noorzad et al. [24].

![Figure 5: Soil heaving mechanism of different widths of large-scale sandbag cofferdams.](image)

![Figure 6: Effect of stiffness of geotextiles on limiting stack height of cofferdams.](image)
3.3. Effect of Stabilizing Berms. As mentioned in the introduction, riprap berms are often used in the construction of large sandbag cofferdams to improve the stability of the large sandbag slope [12, 19]. In order to analyze the influence of back pressure on the limit height and safety factor of cofferdams with different failure modes, a group of numerical modeling cases with varying cofferdam width $W_{cd}$ = 30, 50, 60, 80, and 100 m with and without rockfill berms are simulated. Figure 7 shows the soil failure mechanisms of these two situations. From the numerical simulation, it can be found that for the cofferdams with different foundation failure modes, the limiting stack height and safety factors of the cofferdams with crashed stone are significantly improved. It is proved that the back pressure has obvious effect on the height limit and safety factor of the cofferdam.

3.4. Influence of Berm Size and Form. As can be seen from the above, the stable berm plays a significant role in balancing the cofferdam load and increasing the limiting stack height and safety factor of the cofferdam. However, the influences of berm size and form to the behavior of the cofferdam are not known, which are investigated herein.

3.5. Effect of Width of Crashed Stone. To study the influence of the berm width on the limiting stack height of the cofferdam, five cofferdam widths from 30 m to 100 m with different berm widths (0 m, 1 m, 2.5 m, 5 m, and 10 m) are modeled to analyze its influence to the limiting stack height of the cofferdam. Figures 8(a) and 8(b) illustrate the results of numerical modeling. Within a certain range, as the berm width increases, the limiting stack height and safety factor of the cofferdam increase linearly.

As shown in Figures 8(c) and 8(d), when the berm is constructed at the toe of the cofferdam, the averaged effective force under the toe of the cofferdam increases due to the downward gravity of the berm, which leads to a higher shear strain required for the foundation to yield. Under the same load, the plastic shear strain of foundation decreases, and the range of the plastic zone increases when the critical failure occurs, which makes the cofferdam more stable. Meanwhile, the wider the height of the berm, the lower the plastic shear zone under the toe is developed. This is because the mean effective stress is larger at the toe as well as the shear strain levels at which soil becomes plastic [25].

3.6. Effect of Crashed Stone Height. To illustrate the different failure modes of the soft clay with different heights of crashed stone, a group of numerical modeling cases are performed with the width of crashed stone as a constant of 10 m. The height of the berm is varied at 0.6 m, 0.8 m, 1.0 m, 1.5 m, 2 m, and 2.5 m. The safety factor of the cofferdam, the variation of the ultimate fill height, and the vector diagram of the soil displacement in typical failure modes under different width berms are shown in Figure 9.

With the increase in crashed stone height, the effect of restraining heave and preventing slip becomes more obvious until the effect reaches the optimal value. When the thickness of the berm increases further, the berm will also cause damage to the foundation soil since the berm is also a loose accumulation body built on soft soil. The final safety factor of the cofferdam is reduced. Considering the trends of the maximum height and safety factor of the cofferdam, the optimum thickness of the berm is 1 m within the scope of this study.

3.7. Distance between Cofferdam and Berms. By increasing the area of the plastic zone and limiting the movement of the soil by gravity, the berm increases the limiting stack height and safety factors of the large-scale sandbag cofferdams. Numerical simulation is performed to study the influence of the position of berm on the stability and safety of the cofferdam. Without losing generality, a berm with a thickness of 1 m and a width of 10 m is modeled. The distance between the cofferdam and the berm is gradually increasing to study the effect of the back pressure position on the performance of the cofferdam.

Figure 10 shows the effect of the distance between the cofferdam and the back pressure on the limit height and safety factor of the cofferdam; it can be seen that there is an optimal spacing (around 4 m) resulting in the maximum height and safety factor of the cofferdam. And the failure mechanisms of these cases are shown in Figure 11. It is needed to pointed that the normal method is that the case with one berm.

3.8. Combination of Rockfill Berms. As it is found above, the limiting stack height and safety factor of cofferdam are increased by the effect from the rockfill berms that can prevent the flow of soil and expand the range of plastic zone. It indicates that when the same volume of crashed stones is used to set the berms, it is more helpful to increase the stability of the cofferdam by adopting berm combinations which has a small volume and a large amount rather than only berm.

To investigate the effect of multiberms combination on the stability and safety of cofferdam, numerical analysis is carried out with varying volume ratio (inside: outside) to 4:6, 5:5, 6:4, and 7:3. Figure 12 compares the limiting stack height and the safety factor with different volume ratios. The following observations can be made: (1) as a result of preventing effect from rockfill berms, there is a threshold for the spacing of the two sections of the cofferdam; (2) the larger volume ratio (inside/ outside), the stronger the preventing effect from rockfill berms between the cofferdam and the berms exists and the greater the spacing of the limiting stack height and safety factor for the cofferdam.

As illustrated in Figure 12(c) through plotting the instantaneous (resultant) velocity vectors, due to the segmentation and the interval of berms, the soil flow mechanisms are limited by the berms. If the berms closed to the cofferdam are narrower, it is apparent that the movement of the foundation soil presents two different states. The soil movement in the area
Figure 7: Soil heaving mechanism of large-scale sandbag cofferdams (a) without rockfill berms or (b) with rockfill berms.

Figure 8: Continued.
Failure surface

\[ W_{berm} = 0 \text{ m} \]

\[ H_{max} = 2.63 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.073 \]

\[ W_{berm} = 1 \text{ m} \]

\[ H_{max} = 2.68 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.091 \]

\[ W_{berm} = 2.5 \text{ m} \]

\[ H_{max} = 2.73 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.115 \]

\[ W_{berm} = 5 \text{ m} \]

\[ H_{max} = 2.86 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.155 \]

\[ W_{berm} = 10 \text{ m} \]

\[ H_{max} = 3.06 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.231 \]

\[ W_{berm} = 15 \text{ m} \]

\[ H_{max} = 3.29 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.313 \]

Failure surface

\[ W_{berm} = 0 \text{ m} \]

\[ H_{max} = 2.91 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.164 \]

\[ W_{berm} = 0 \text{ m} \]

\[ H_{max} = 2.93 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.180 \]

\[ W_{berm} = 2.5 \text{ m} \]

\[ H_{max} = 2.97 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.206 \]

\[ W_{berm} = 5 \text{ m} \]

\[ H_{max} = 3.06 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.252 \]

\[ W_{berm} = 10 \text{ m} \]

\[ H_{max} = 3.23 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.331 \]

\[ W_{berm} = 15 \text{ m} \]

\[ H_{max} = 3.38 \text{ m} \]

\[ SF(2.4 \text{ m}) = 1.422 \]

Figure 8: Effect of width of berms: (a) effect on limiting stack height; (b) effect on safety factor; (c) soil heaving mechanism with global failure mechanism; (d) soil heaving mechanism with local failure mechanism.

Figure 9: Continued.
Figure 9: Effect of height of berms: (a) effect on limiting stack height; (b) effect on safety factor; (c) soil heaving mechanism with global failure mechanism; (d) soil heaving mechanism with local failure mechanism.

Figure 10: Effect of gap between cofferdam and berms on limiting stack height and safety factor of cofferdams.
affected by the ground and the inner back pressure is obviously larger than that affected by the lateral back pressure. When the total gravel volume is constant and the inner back pressure width is increased, the occurrence is obvious. The range of the moving soil expands and the foundation exhibits a single slip surface in the critical failure state.

**Figure 11:** Soil heaving mechanism of large-scale soil bag cofferdams with different distances between berm and cofferdam: (a) with normal crashed stone; (b–h) with different gap; (i) with no crashed stone.
Figure 12: Effect of height of berms: (a) effect on limiting stack height; (b) effect on safety factor; (c) soil heaving mechanism of large-scale sandbag cofferdams with different berm combinations.
Numerical simulations are performed to compare the improvement effects of cofferdam performance parameters under different berms modes. It is apparent that the limiting stack height of the cofferdam is only 0.14 m lower than that of the nonsegmented cofferdam of the same influence width, and the safety factor of the same height (2.4 m) is only 0.061 lower. This means the material cost in berms can be saved by 1/3 with the help of the berm-combination method. The same phenomenon happens in cofferdams with local failure mode.

4. Conclusion

Numerical modeling is performed in this study to investigate the effects of geotextile mats and rockfill berm on the behavior of underlying soil and the stability of cofferdams. The following conclusions could be drawn:

1. The limiting stack height and safety factor of the large sandbag cofferdam are affected by the width of the cofferdam, the shear strength of the foundation, the tensile strength of geotextile, the arrangement of berms, and the additional counter pressure platform.

2. For large-scale geotextile cofferdams with global failure mode and local failure mode, the rockfill berms affect the limit height and safety factor of cofferdams by influencing the stress distribution in the underlying soil and limiting the uplift of foundation.

3. An optimized design method for large geotextile mats over soft soil is proposed to enhance the stability with the same volume of berm.

Data Availability

Some data, models, or code generated or used during the study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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