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Numerical analysis of static behavior of caisson-type quay wall deepened by grouting rubble-mound

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Abstract

Caisson type gravity quay wall is a common structure used in the coastal regions. However, many of the existing quay walls constructed in the past are becoming obsolete. Therefore, the main goal of this study is to enhance the performance of these quay walls by increasing the front water depth. To deepen the water depth, a special grout type is ejected to solidify the rubble mound under the caisson toe, then excavating a part of the rubble placed in front of the caisson to the designed level. Various cases with different shapes and dimensions are proposed to optimize the grouted area. Based on the examination of stability and construction feasibility, the reasonable geometry and area of grouted rubble can be selected. In addition, the numerical analysis is performed by the Finite Element Method (FEM) program (PLAXIS 2D) to expect the behavior of the quay wall and grouted rubble. The results demonstrate that after upgrading, the maximum contact stress between caisson and rubble mound increases sharply, but the stress at the bottom of grouted rubble does not change in comparison prior to innovation. The analysis also indicates that when the Hardening Soil (HS) model is applied, the displacement of the quay wall is higher than that of the Mohr–Coulomb soil (MC) model.

Keywords: Caisson type quay wall, Deepening, Front water depth, Grouted rubble, Upgraded quay wall

Introduction

Quay walls are earth retaining structures at which ships can berth. They are usually equipped with bollards to provide moorings for ships and fendering to absorb the impacts of the vessels. The quay walls are used for the transshipment of goods by cranes or heavy equipment that moves alongside the ships [1]. There are various types of the quay wall structures, but in general they can be classified into four basic groups: gravity walls, sheet pile walls, structures with relieving platform and open berth quays as shown in Fig. 1.

Nowadays, the demand for cargo transportation via waterways are rising significantly with the rapid increase of the large tonnage ships. Therefore, the deep-water ports become more and more necessary to accommodate these ships. However, many

quay walls have been used for a long time becoming out-date and cannot meet current demands. However, due to the high costs and the environmental boundaries, the complete demolition of these existing quay walls and replacing them with new structures are not preferred. Thus, upgrading the existing quay walls are giving great challenging for engineers and researchers.

Regarding this issue, many case histories about the deepening and upgrading of the existing quay walls have been reported in the literature. Elskén and Bols [2] indicated that combining the techniques of the very high-pressure grouting, installation of ground anchors and drains had already proven to be an economical and technical solution for deepening the quay walls. Bauduin et al. [3] proposed some general guidelines for an integrated design and construction approach, combining risk fault tree analyses, robust and flexible design, construction methods and monitoring methods. Examples of projects in several countries (Europe and Africa) where different types of existing quay walls were strengthened and deepened illustrate the guidelines. Ko et al. [4] conducted a basic study on wharf deepening for the improvement of harbor facilities. Oung and Brassinga [5] discussed widely the risks of upgrading the existing quay walls such as deepening in front of quay walls and increasing the loads on the quay surface. Ruggeri et al. [6] summarized the main issues involved and experiences in geotechnical design for upgrading six quay walls to meet new demands in Italy. To enhance these quay walls, some methods were adopted in which they focused on using jet grouting and micro-piles to increase bearing capacity and front water depth of berth. Galal [7] carried out the numerical analysis to upgrade the container terminal with an open berth on pile type structure in Egypt. To increase water depth, the selected rehabilitation technique consisting of new

fender piles and new box sheet pile panels were installed at the toe of the berth. Cornell et al. [8] presented the solutions and experience to innovate the berth structure in Panama Canal. The structure was strengthened by an additional structure system including piles and sheet pile before excavating the seabed so to increase water depth. ElGendy et al. [9] evaluated the influence of the deepening in front of the Port Said East Port container terminal located on the Mediterranean Sea on the north of Egypt. Both Mizutani et al. and Oh et al. [10, 11] used grouting as a method to strengthen rubble mound of gravity quay wall before deepening the water depth. The new material for rubble mound grouting was developed in these studies. Various numerical analyses and some dynamic tests such as the shaking table or centrifuge tests were conducted to access static and seismic behavior of the caisson type quay wall after enhancement. Douairi and De Gijt [12] introduced series of concepts for creating the water dept of sheet pile wall such as using extra grout anchor, grouting in front of the quay wall, using additional sheet pile, etc. El-naggar [13] suggest how to enhance steel sheet walls using grouted anchors using FEM and evaluated the effect of sheet pile geometry, inclination and location, length of grout, excavated depth, internal friction angle of backfill in upgrading this type of wall.

Through literature reviews, it can be observed that the number of researchers working on how to upgrade the gravity quay walls are still limited, especially considering the detailed behavior of the structures and rubble mound. Therefore, the contribution given in this study are relevant. As mentioned above, this research suggests deepening the water depth of the caisson type quay wall by cutting the rubble mound apart in front of the structure. Besides, in order to guarantee the stability of the quay wall, the grouting of rubble is conducted beneath the front caisson toe before removing the rubble. The present study firstly focuses on optimizing the area and shape of grouted rubble. The geometric type and dimensions of grouted rubble can be determined by examining the slip, overturning, slope stabilities, and considering construction ability. Secondly, the study assesses the overall stability of the quay wall after upgrade using numerical analysis by the PLAXIS 2D program. The displacements of the quay wall during construction and operation are estimated from both the MC model and the HS model to evaluate the influence of the soil model on the estimation of quay wall displacement. Finally, from the numerical analysis results, the change of stress in the rubble before and after innovation can be evaluated.

Case study

Cross-section of the quay wall

The typical cross-sections of the caisson type quay wall examined in this paper before and after the upgrade are shown in Fig. 2. The wall consists of a caisson having a height of 17.5 m and a width of 10 m. It rests on a 10 m thick foundation layer of rubble mound and is underlain by bedrock. Behind the caisson, there is a rubble prism having slope 1:1. The retained backfill has a thickness of 23.5 m by sand. The water depth of the existing quay wall is 13.5 m, and it can receive the 40,000 DWT tonnage ships. After deepening, the front water depth rises to 15.5 m. The performance of the quay wall also increases significantly, and 70,000 DWT tonnage ships can berth.

Stages of construction

The construction process of the quay wall upgrade is divided into four steps as shown in Fig. 3. Step 1 is the current state of the quay wall without any external loads. Step 2, the uniform load of 20 kN/m^2 is assigned on the ground surface. This load is due to the machines and materials located on the ground surface during construction. After that, the grout is injected to improve the rubble mound at the bottom of the caisson toe in step 3. Finally, a part of rubble in front of the quay wall is excavated up to -15.5 m deep at step 4. It is noticed that the uniform load of 20 kN/m^2 is due to the construction loads, it remains stable from step 2 until the end of the construction process.

Soil models and parameters

Mohr–Coulomb model

The Mohr–Coulomb model is a simple and well-known linearly elastic, perfectly plastic model, which can be used as a first approximation of soil behavior. The linear elastic part of the Mohr–Coulomb model is based on Hooke’s law of isotropic elasticity. The perfectly plastic part is based on the Mohr–Coulomb failure criterion, formulated in a non-associated plasticity framework. The basic principle of elastoplastic is that strains and strain rates are decomposed into an elastic part and a plastic part as shown in Fig. 4.

The linearly elastic, perfectly plastic Mohr–Coulomb model requires a total of five parameters, which are generally familiar to most geotechnical engineers and which can be obtained from basic tests on soil samples. These basic parameters including Young's modulus (E), Poisson's ratio (ν), cohesion (c), friction angle (ϕ), dilatancy angle (ψ). Table 1 shows the properties of the soil used in this study.

The Hardening soil model

The Hardening Soil model is an advanced model for simulating the behavior of different types of soil, both soft soils and stiff soils [18]. When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. In the special case of a drained triaxial test, the observed relationship between the axial strain and the deviatoric stress can be well approximated by a hyperbola. The yield function of Hardening soil model is given by Eq. (1):

$$\bar{f} = \frac{2}{E_i} \frac{q}{1 - q/q_a} - \frac{2q}{E_{ur}} - \gamma^p \quad (1)$$

where q_a is the asymptotic value of the shear strength, E_i the initial modulus, E_{ur} is unloading/ reloading modulus, γ^p is the function of plastic strain.

$$q_a = \frac{q_f}{R_f}, \quad q_f = (c \cot \varphi - \sigma'_3) \frac{2 \sin \varphi}{1 - \sin \varphi} \quad (2)$$

$$E_i = \frac{2E_{50}}{2 - R_f}, \quad E_{50} = E_{50}^{ref} \left(\frac{c \cot \varphi - \sigma'_3 \sin \varphi}{c \cot \varphi + p^{ref} \sin \varphi} \right)^m \quad (3)$$

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cot \varphi - \sigma'_3 \sin \varphi}{c \cot \varphi + p^{ref} \sin \varphi} \right)^m \quad (4)$$

R_f is the failure ratio, normally smaller than or equal to 1; E_{50} is secant modulus at 50% failure load, m is the material parameter, typical in the range $0.5 < m < 1.0$; E_{50}^{ref} , E_{ur}^{ref} is the reference secant modulus and reference unloading/reloading modulus respectively, corresponding to the reference pressure p^{ref} . The meaning of the parameters appearing in Eq. (1) is illustrated in Fig. 5.

The function of plastic strain is estimated by Eq. (5):

$$\gamma^p = -(2\varepsilon_1^p - \varepsilon_v^p) \quad (5)$$

Table 1 Properties of soil for the MC model [15–17]

Material	γ_{unsat} (KN/m ³)	γ_{sat} (KN/m ³)	E (kPa)	ν	C (kPa)	ϕ (°)	ψ
Backfill sand	18.0	20.0	24,000	0.3	~	33	3
Rubble	18.0	20.0	36,000	0.3	20	35	5
Alluvium	17.0	19.0	22,000	0.3	20	30	0

Table 2 Parameters of soil for the HS model [15–17]

Material	γ_{unsat} (KN/m ³)	γ_{sat} (KN/m ³)	E_{50}^{ref} (kPa)	$E_{\text{oed}}^{\text{ref}}$ (kPa)	$E_{\text{ur}}^{\text{ref}}$ (kPa)	m	C (kPa)	ϕ (°)	ψ
Backfill sand	18.0	20.0	24,000	24,000	72,000	0.5	~	33	3
Rubble	18.0	20.0	36,000	36,000	108,000	0.5	20	35	5
Alluvium	17.0	19.0	22,000	22,000	66,000	0.5	20	30	0

where ε_1^p and ε_v^p are the major principal and volumetric components of plastic strain respectively.

The flow rule adopted in HSM is characterized by a classical linear relation, with the mobilized dilatancy angle given by:

$$\sin \psi_m = \frac{\sin \phi_m - \sin \phi_{cv}}{1 - \sin \phi_m \sin \phi_{cv}} \quad (6)$$

where ϕ_{cv} is the critical state friction angle, being an inherent material property independent of density, and ϕ_m is the mobilized friction angle

$$\sin \phi_m = \frac{\sigma'_1 - \sigma'_3}{\sigma'_1 + \sigma'_3 - 2c \cot \phi} \quad (7)$$

The basic parameters of the Hardening soil model include reference secant modulus at 50% failure load (E_{50}^{ref}), reference tangent modulus for primary oedometer loading ($E_{\text{oed}}^{\text{ref}}$), reference unloading/ reloading modulus ($E_{\text{ur}}^{\text{ref}}$), power for the stress-level dependency of stiffness (m), the Mohr–Coulomb failure criterion parameters (c , ϕ , ψ). Table 2 shows the soil parameters used in this paper.

Fig. 6 Model of quay wall in Plaxis-2D

Fig. 7 Stability of rubble mound without grouting

Method and the fem model used in this study

In general, the calculations and criteria conform to the requirements in [15]. The analytical method is used to check the sliding, overturning, and circular slip stabilities, while the Finite Element Method (PLAXIS program) is applied to determine the stress and displacement of the soil and structure.

Figure 6 shows the model of the quay wall simulated in the PLAXIS. The height of the model from the bedrock surface to the ground surface is 27.5 m, while the length is 250 m. The analyses are conducted and compared with two of soil models: the MC model and the HS model which available in the material models library of the program. The concrete caisson and grouted rubble are simulated using the elastic model. The elastic modulus and Poisson ratio are 3.0×10^7 kPa and 0.2 for caisson material [19], 3.0×10^6 kPa and 0.2 for grouted rubble [10] respectively. All the quay wall, soil, and rubble mound are modeled with 15 node triangular plane strain elements. The fixed condition ($x=0$, $y=0$) is applied at the bottom boundary, while at the left and right boundaries, horizontal fixed condition ($x=0$) is used. The interface friction angles are assumed to be 15° and 30° at the back and base of the wall, respectively. The initial horizontal effective stresses are set equal to 0.5 times the initial vertical effective stresses. Moreover, the water level at ± 0.0 m is used in the analysis, and the uniform load 20 kN/m^2 is assigned to the ground.

Results and discussion

Selection of the optimum area of grouted rubble

Firstly, the excavation of the rubble mound without any improvement methods are considered. Figure 7 shows the result of the slope stability examination of the rubble mound at the bottom of the caisson. According to the requirements mentioned in [15], all slope surfaces with safety factors less than 1.2 are not accepted. As can

be seen from the figure, if the rubble mound is not improved before cutting, all rubble around 3.0 m from the front edge of the caisson slips down. This can lead to the collapse of the rubble mound and also the quay wall. Thus, it is necessary to have a solution solidifying the rubble mound foundation at the bottom of the front toe of the caisson.

Figure 8 suggests four cases for the grouting with different shapes and dimensions. To optimize the improved area, the area of the grouted rubble, A , is increased slowly from 4 m^2 (case 1) to 11 m^2 (case 4). Besides, in order to consider the effect of the shape of the grouted rubble on its stability and construction ability, the study proposed two types of shape: In the case 1 and case 2, the rubble mound is excavated vertically; while in the case 3 and case 4, the grouted rubble are designed with an oblique edge in front.

The area and shape of grouted rubble are determined firstly based on its stability and construction ability. The grouted rubble must meet the requirements of the sliding, overturning, and circular slip stabilities. The process of checking these problems is shown in Fig. 9. In the first step, from the design conditions such as the geometric, material properties, loads, water level, the earth pressure behind the caisson can be calculated. Next step, the contact pressure between the caisson and the rubble mound is determined, and it is also the load impacting on the rubble mound surface. After that, the earth pressure owing to the rubble on the back of grouted rubble is calculated. Finally, from the loads and the earth pressure calculated above, the safety factors of sliding, overturning, circular slip stability can be determined.

Figure 10 shows the change of the sliding, overturning, and circular slip stability factors with the grouted area. The criteria of the safety factor equal to 1.2 for all cases according to Design Standard for Harbor and Fishing Port—Korea [15]. In general, the stability safety factor, F_s , increases with an increase in the area of grouted rubble. Moreover, for all cases, the F_s satisfied the requirements mentioned in the standard. In the case 1 ($A = 4 \text{ m}^2$) the F_s of the sliding stability is 1.9, and then it rises significantly to 2.8, 3.3 and 3.7 in the case 2 ($A = 6 \text{ m}^2$), case 3 ($A = 8 \text{ m}^2$) and case 4 ($A = 11 \text{ m}^2$), respectively. Similarly, for the circular slip stability, F_s of case 1 is 1.43, after that, it increases slightly to 1.55 at case 2, case 3, and 1.57 at case 4. Differently, the F_s of the overturning stability fluctuates around 3.0 and ranges from 2.27 in case 2 to 4.04 in case 3.

Though all the cases meet stability conditions, in terms of feasibility and construction ability, cases 1 and 2 are difficult to perform because the vertical cutting is not practicable for the rubble. In addition, from the cost-effective view, the case 3 is better than the case 4 due to the area of grouting of the case 4 ($A = 11 \text{ m}^2$) is larger than that of the case 3 ($A = 8 \text{ m}^2$). Therefore, it can be concluded that case 3 is selected as the most reasonable solution for all aspects including the design, the construction method, and the economic perfective. Note that the area of grouted rubble might be changed depending on the structure type, structure dimensions, design conditions, etc. Hence, further study is required for various structure types under different

conditions so as to recommend a suitable area of grouted rubble for each specific case.

The change of contact stress after the upgrade

After innovation, the surface level of the rubble foundation in front of the quay wall is lower by 2 m in comparison with the initial status. In addition, the rubble mound at the bottom of the front toe of the caisson is solidified by the grouting. The modulus of the grouted rubble is 3×10^6 kPa, which is approximately 100 times larger than that of the rubble mound, at 3.6×10^4 kPa. The change of cross-section of the structure and the increase in foundation stiffness lead to the redistribution of contact stress between the caisson and the rubble mound. Figure 11 compares the contact stresses (effective stress) before and after the upgrade. After deepening the maximum stress is 650 kPa which is more than double that of before, with about 310 kPa. In addition, the difference in the stiffness between grouted rubble and the rubble mound also leads to the concentration of contact stress around point O.

To assess the change in the contact stress (effective stress) distribution between the caisson and rubble mound, series of numerical analyses with different the grouted rubble stiffness is carried out. The results of maximum and minimum stresses are plotted in Fig. 12. As can be seen from the figure, the maximum stress increases with an increase in the value of modulus. At $E_g = 3.6 \times 10^4$ kPa, the maximum stress is 310 kPa. When the elastic modulus of the grouted rubble rises to $E_g = 2.0 \times 10^6$ kPa and 3.0×10^6 kPa, the maximum value soars to 595 kPa and 650 kPa, respectively. By contrast, the greater the modulus, the smaller is the minimum stress. There is a sharp decrease in the minimum stress from 245 kPa at $E_g = 3.6 \times 10^4$ kPa to 210 kPa and 186 kPa at $E_g = 2 \times 10^6$ kPa and 3×10^6 kPa, respectively.

Not only the contact stress between the caisson and the rubble foundation but also stress at the bottom of the grouted rubble should be considered. Figure 13a, b compare the distributions of stresses at the bottom of grouted rubble level before and after the upgrade. It can be seen that, although after innovation the stress at the top of the rubble mound around point O (Fig. 11) is higher than that the earlier one, the stresses at the sections A–A and A'–A' are similar for both cases. This result can be interpreted by comparison the stress distribution along section B–B and B'–B'. Before quay wall

Fig. 11 The contact stress under the front toe of caisson before and after grouting. **a** Before upgrade, **b** After upgrade

deepening, the total stress at the top of the rubble mound is about 449 kPa (section B–B) in comparison to about 650 kPa after upgrade (section B'–B'). However, while the total stress along section B–B reduce slightly with the depth, there is sharp decrease in the stress of section B'–B'. As a result, the total stresses at the level A–A and A'–A' almost the same, at 431 kPa for both cases. Therefore, effective stress at section A–A and A'–A' are also similar, with maximum value around 270 kPa.

Displacement of upgraded quay wall and the effect of constitutive model

In order to assess the stability of the quay wall during and after innovation, some numerical analyses with different constitute models are carried out to clarify displacements. The analyses are performed according to construction stages presented in “The Hardening soil model” section, include step 1: initial status, step 2: application of surcharge, step 3: grouting, and step 4: excavation. Figure 14 plots the relationship between displacements at the left top corner of the quay wall and the construction steps. The allowable displacement equals 10 cm calculated according to Harbor and Fishing Port Design Standard (Ministry of Ocean and Fisheries-Korea, 2018). As can be seen from the figure, the displacement estimated from the HS model is higher than that of the MC model, but all values meet the requirements in the standard. At step 1, the caisson is impacted by only the earth pressure of backfill, the horizontal displacement is 2.61 cm for the MC model and 6.75 cm for the HS model. When the surcharge is applied at step 2, the horizontal displacement increases to 3.03 cm and 8.03 cm for the MC model and HS model, respectively. After that, the horizontal displacements almost remain stable. At the end of the construction process (after excavation), the horizontal displacement rises slightly to 3.15 cm for the MC model and 8.26 cm for the HS model.

Figure 15 shows the horizontal displacement contours of the quay wall at the end of the construction process for both soil models. The displacement of the caisson leads to the movement of the backfill. However, owing to the low value of quay wall displacement,

Fig. 13 Stress distribution in the rubble mound and grouted rubble. **a** Before upgrade, **b** After upgrade

Fig. 14 The development of displacements at the top of the quay wall with construction steps

the backfill rubble is rearranged and almost deformation occurs inside of the rubble. Therefore, the displacement decreases with increasing of distance from caisson back, and the displacement of the ground surface is lower than that of the caisson.

The horizontal displacements along the section C–C for the MC and HS soil model are shown in Fig. 16. For the MC model, the displacements are constant at any levels of the caisson, this means that the caisson tends to horizontal slip. Whereas, in the case of the HS model, the higher the caisson level, the greater is horizontal displacement. The maximum horizontal displacement occurs at the top of the caisson and reduces with an increase in the caisson depth so that the caisson tends to overturn.

The uniqueness of the results due to the different soil behavior depends on each model used. The MC model is a linear elastic perfectly plastic model, and it is suitable to describe the behavior of soil at the first stage. In the MC model, only the elastic modulus of the soil is defined. However, in reality the soil is not an elastic material, and the modulus of the soil is controlled by the stresses. The HS soil model is an advanced constitutive

model, and the relationship between the axial strain and the deviatoric stress is approximated by a hyperbole. In the HS soil model 3 stiffness values are defined: the secant modulus (E_{50}), tangent modulus for primary oedometer loading (E_{eod}), and reloading/unloading modulus (E_{ur}). This definition makes the behavior of the soil simulated by the HS model approach the real behavior. As a result, the displacements of the quay wall estimated from the HS soil model and the MC model are different.

Conclusions

The existing caisson-type quay wall can be upgraded by the solution in which the rubble mound beneath the caisson is solidified by grouting then cut to increase the front water depth. This paper provides the principle process to optimize the area of grouted rubble and static analysis of the upgraded quay wall using numerical analysis. The study also assesses the effect of the grout stiffness and the constitutive models on the static behavior of the quay wall.

After deepening, the change of structural cross-section and material properties especially stiffness of grouted rubble lead to the redistribution of contact stress between the caisson and rubble mound. The result demonstrates that the greater elastic modulus of grouted rubble, the higher is maximum contact stress. In this study, the maximum contact stress increases more than twice the value prior to the innovation.

Though the maximum contact stress transferring from caisson to grouted rubble increase significantly after the upgrade, it tends to concentrate on the top of grouted rubble and decreases sharply with the depth. The results show that, below 2 m depth from the top of the rubble mound, the total stress in the grouted rubble equals that in the rubble mound before upgrade at the same level. Similarly, the stress at the bottom of grouted rubble is almost the same prior to the upgrade.

The computed response of the wall and soil system is largely influenced by the choice of the soil model. The maximum displacement of the caisson determined from the HS soil model is approximately 2.5 times larger than that of the MC model. In addition, using the MC model, the horizontal displacements are constant regardless of the point in the caisson. Differently, the results from the HS model show that the horizontal displacement is highest at the top of the caisson and tends to decrease with an increase in the caisson depth.

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Authors' contributions

ADN conducted the analysis and wrote the manuscript. YSK supervised the research and revised the manuscript. GOK provided the data and revised the manuscript. HJK made some figures. All authors read and approved the final manuscript.

Competing interests

The authors declare that they have no competing interests.

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