Article

Not peer-reviewed version

Numerical Modelling of the Geotechnical and Structural Strengthening of Quay Structures with a Case Sudy

Selçuk Bildik *

Posted Date: 19 September 2023

doi: 10.20944/preprints202309.1218.v1

Keywords: quay wall; piles; port; finite element analysis; case study

Article

Numerical Modelling of the Geotechnical and Structural Strengthening of Quay Structures with a Case Sudy

Selçuk Bildik

Department of Civil Engineering, Nisantasi University, Istanbul, Turkey; selcuk.bildik@nisantasi.edu.tr

Abstract: In recent years, with the increase in maritime trade, the necessity of increasing the capacities of the ports has emerged. However, while it is planned to increase the capacities of the ports, it is important that the port continues to operate at the same time. In this respect, the old port structures should not be damaged during the capacity increase. In this study, the strengthening of a port in Guinea is discussed as a case study. In the study, the existing quay wall was evaluated, and geotechnical and structural alternatives of the new structure to be built for capacity increase were evaluated. A combined system was designed as a pile foundation and a reinforced foundation with plastic piles so as not to damage the existing quay wall. The pile capacities obtained as a result of the analyses were verified by loading tests.

Keywords: quay wall; piles; port; finite element analysis; case study

---

1. Introduction

In recent years, maritime transportation has also developed rapidly with the increase in trade volume around the world. Accordingly, the need to transport high tonnage loads and the use of larger ships made it necessary to increase the capacities of the ports. Increasing port capacities is possible with the alternatives of demolishing and rebuilding the port completely or strengthening the old port structure. However, considering the continuity of the operation of the ports, it is preferred to strengthen the old port structure.

When the current state of the quay structures is examined today, it is seen that they were built with stone fortifications in the past. The tides occurring in the sea cause the cement between the stone fortifications to be lost over time and thus to the emergence of stability problems. It is important to protect the quay structure in order to increase the capacity of such quay structures without hindering the operation of the port.

There are many studies on the behavior of quay structures in the literature (Yang et al., 2001; Alyami et al., 2009; Zekri et al., 2015; Habets et al., 2015; Hamed et al., 2017; Tan et al., 2018; Skopal et al., 2019; Aldelfee et al., 2020; Elshafey et al., 2021; Nguyen et al., 2021).

Alyami et al. (2009) investigated the deformation behavior of quay structures under earthquake loads with numerical analysis. In the study, the analyses were carried out in two dimensions using the effective-strain finite element procedure and elastoplastic material properties. Various monotonous and cyclic triaxial paths were simulated to see the model's capabilities. Back analysis of a damaged port structure in the 1995 Hyogoken-Nanbu earthquake was performed for FEM validation. According to the results obtained, liquefaction occurs in the backfill, and with the improvement of the backfill, the vertical displacements in the quay structure decrease by 200% and the horizontal displacements by 350%.

Tan et al. (2018) investigated the performance of the quay walls with anchored sheet piles with separate pile-supported platforms through field testing and numerical analysis. The structural features, construction processes, and instrumentation of the quay system were also taken into account in the study. Considering the results obtained, it has been observed that the platform behind the sheet piles significantly reduces the deflection of the wall. When compared with the numerical results,

 doi:10.20944/preprints202309.1218.v1

Coulomb's earth pressure theory significantly overestimates the earth pressure acting on the sheet pile wall at higher elevations and underestimates the values at lower elevations.

Adelfee et al. (2020) investigated the seismic performance of quay walls using the finite element method. A real gravity quay wall (Kobe Harbor Quay Wall) was simulated with single-frequency earthquake motion using Plaxis 2D program. The acceleration and displacement behavior of the quay wall were investigated numerically. The experimental results obtained from the shaking table in dry and saturated conditions were compared with the numerical results. The results show that the Plaxis-2d finite element program is an effective tool to predict the seismic performance of the quay wall and there was clear agreement between both the results of experimental results and numerical modeling.

Elshafey et al. (2021) numerically investigated the use of earth-reinforced walls with geogrids as quay walls. In the numerical analysis, the model was validated using the finite element program in the first stage, using the results of a full-scale laboratory experiment. In the second stage, the model was expanded by considering the parameters that will affect the performance of the soil wall in the marine environment. When the obtained results were evaluated, it was seen that the geogrid stiffness significantly affected the horizontal movement of the earth-reinforced walls as the quay wall. In the study, it has been verified that the geogrid length should be at least equal to the wall height to ensure the overall external stability of the walls used for the purpose of the quay wall. In addition, numerical results show that geogrids with smaller spacing should be used in the upper layers to minimize the lateral and vertical deformation of the quay wall.

Nguyen et al. (2021) examined the performance of the quay structure in case of increasing the water depth in front of it. In case of an increase in water depth, the improvement of the rubble mound under the caisson toe by injection was optimized and suggestions were made. They also performed analyses to predict the behavior of the quay wall and grouted rubble mound using a finite element program. The results show that the contact stress between the caisson and the rubble mound increases sharply after improvement. In addition, when the Hardening Soil (HS) model was used, the deformations on the quay wall were found to be higher than the Mohr-Coulomb (MC) soil model.

In literature studies, a model has been verified numerically, and parametric studies on quay structures have been carried out. In these studies, mostly geotechnical parameters were emphasized. In this study, a case study is discussed, different from the literature. In the case analysis discussed, the situation of increasing the capacity without damaging the existing quay structure has been taken into consideration. Geotechnical and structural analyzes were carried out, and performance tests were also carried out after the reinforcement works were decided according to the results.

2. Site Conditions

The site is located at a port in Conakry City in Guinea. Various renovation and construction activities are carried out at the port. Within the scope of these activities, new mobile cranes were procured in order to handle the general cargo dock in more modern conditions. In the renovation works, it is necessary to determine the effect of new crane loads on the existing clinker pool and to carry out the necessary strengthening works. For this purpose, first of all, the structural project of the existing quay wall was determined based on old data and field observations (Figure 1). Then, soil investigations were carried out in the field in order to determine the soil conditions, and the borehole locations are presented in Figure 2. The borehole depths vary between 10.00-30.00 meters. Standard Penetration Test (SPT) is performed at regular intervals of 1.5m for all the boreholes (Figure 3). The soil profile obtained from BH-1 and BH-1A boreholes is shown in Figure 4. The study area generally consists of sand gravel-laterite mixtures followed by yellowish-white, light brownish-red medium dense sand - green dark gray clay - brown clay soils. Laboratory experiments were carried out on the samples obtained from the boreholes carried out in the region where the problem was taken into account. According to the laboratory results, the cohesion value varies between c=60-88 kPa and the internal friction angle value varies between ϕ=2-7°. The groundwater level at the study area was observed between 2.50-2.90 meters according to the borehole locations.

Preprints (www.preprints.org) | NOT PEER-REVIEWED | Posted: 19 September 2023 doi:10.20944/preprints202309.1218.v1

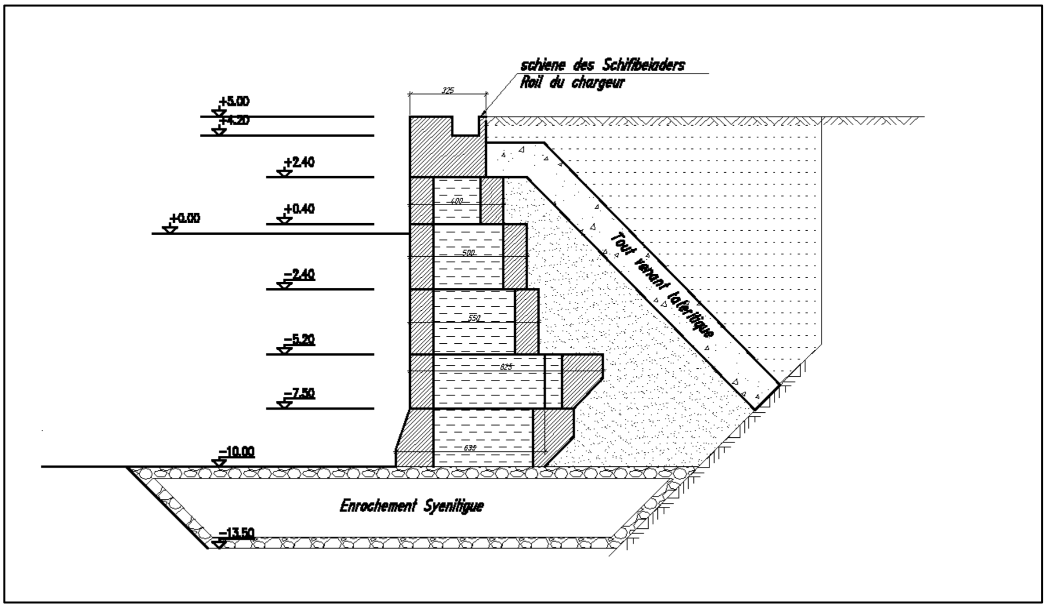
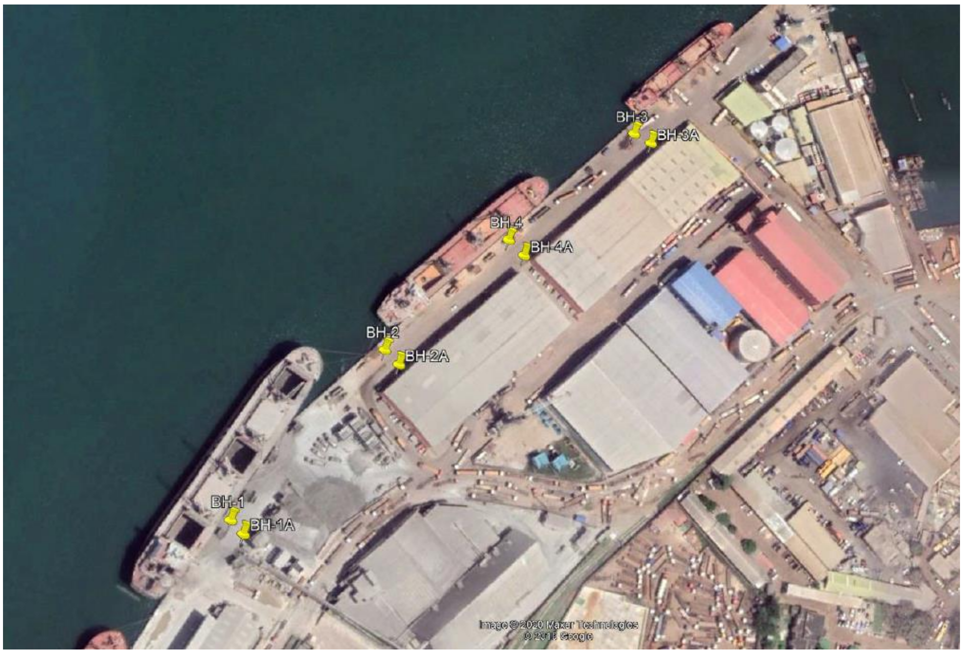


Figure 1. Structural profile of the existing quay wall.

(**a**) Boreholes location (first study)

Preprints (www.preprints.org) | NOT PEER-REVIEWED | Posted: 19 September 2023 doi:10.20944/preprints202309.1218.v1



(**b**) Boreholes location (second study).

Figure 2. Boreholes layout plan.

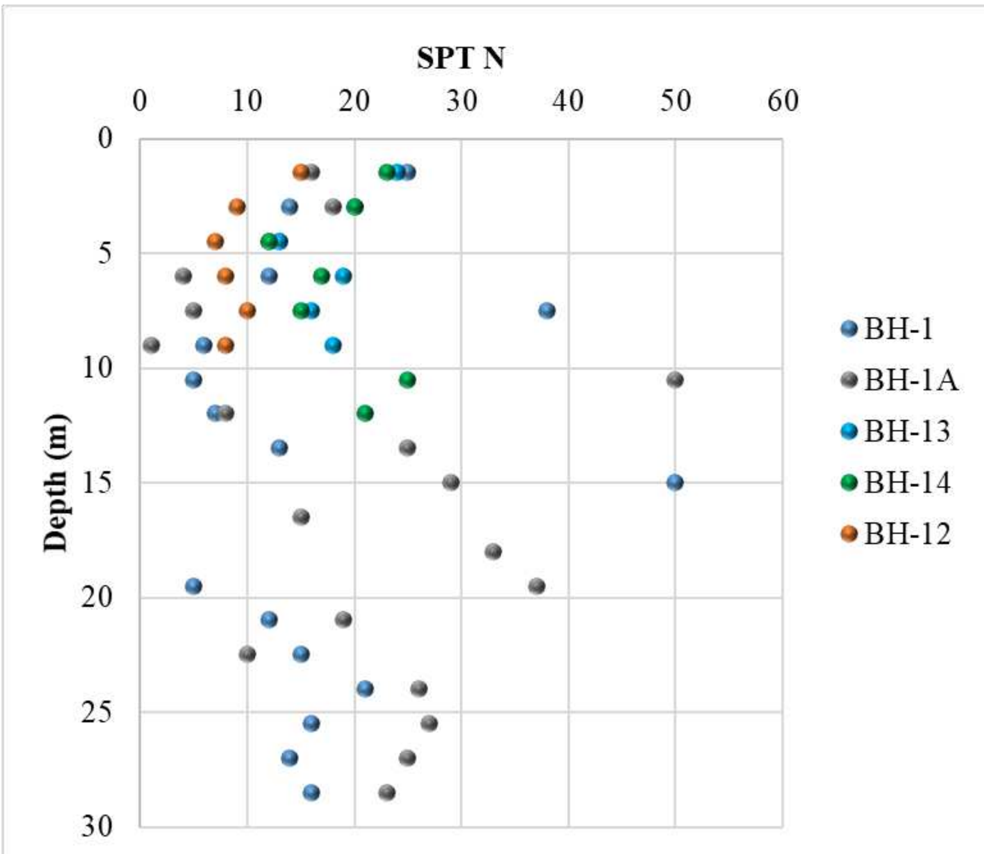


Figure 3. Variation of SPT test results with depth.

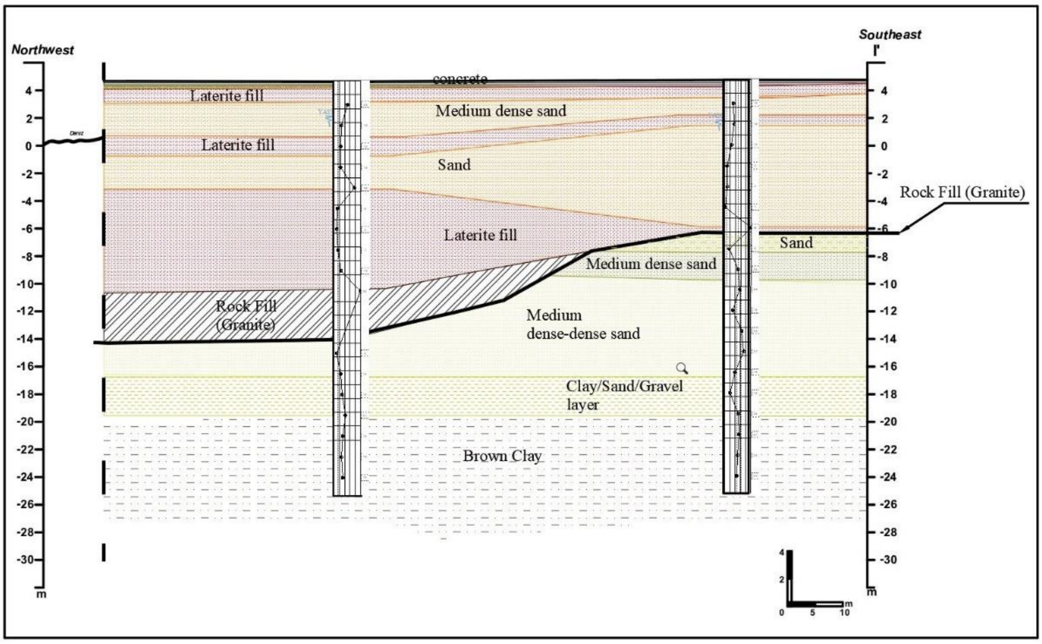
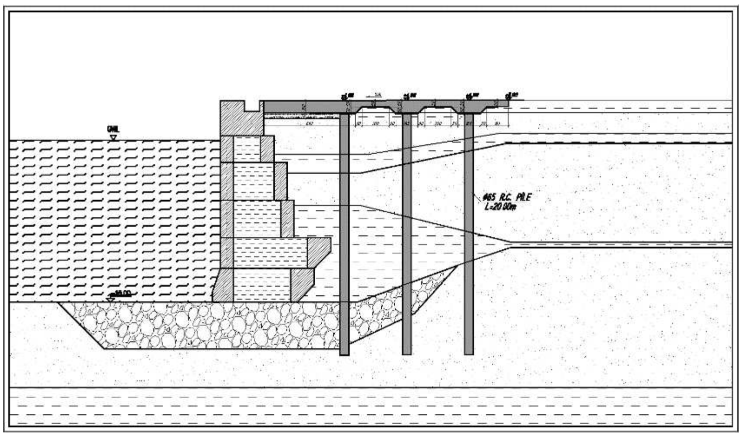


Figure 4. BH-1 and BH-1A soil profiles.

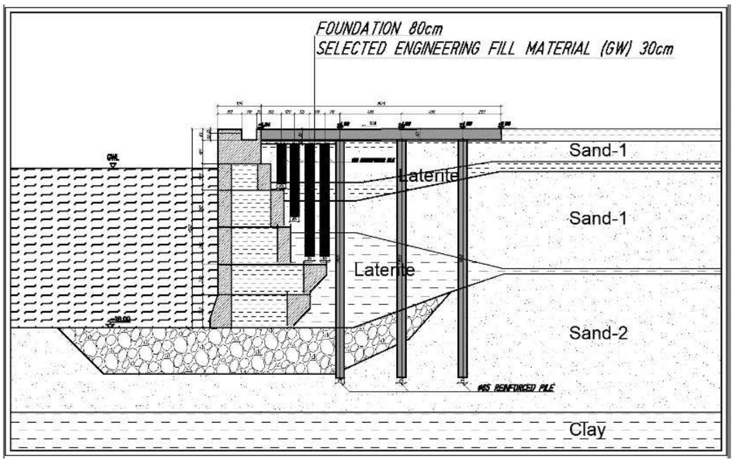
3. Superstructure and Geotechnical Design

The existing conditions of the quay wall area were insufficient to support the new crane loads in terms of bearing capacity and stability problems. For that reason, a new platform was planned for the new crane loads. Due to the new platform, it should not bring additional loads or deformations to the existing quay wall. For this purpose, three different alternatives shown in Figure 5 were evaluated in terms of structural and geotechnical design. The first alternative, as seen in Figure 5a, is planned to place the front part of the foundation, which is close to the shore and on the quay wall, and to construct a pile foundation at the back part of the foundation. The existing quay wall will support the structural stability of the system, and this will contribute to the structural design. However, when evaluated in terms of geotechnical design, in this case, there is a risk of deformation of the quay wall due to the loads on the quay structure near the shore, and this alternative was not considered appropriate. In other alternatives, as seen in Figure 5b, reinforcement alternatives with jet grout or unreinforced piles are considered in the front part of the foundation near the shore, and pile foundation alternatives are evaluated in the back part of the foundation. Although it is structurally appropriate in both cases, jet grouting may damage the quay wall due to high pressure during application in terms of geotechnical design.



(**a**) First alternatives for reinforcement of quay wall.

 doi:10.20944/preprints202309.1218.v1



(**b**) Second and third alternatives for reinforcement of quay wall.

Figure 5. Alternatives for reinforcement of quay wall.

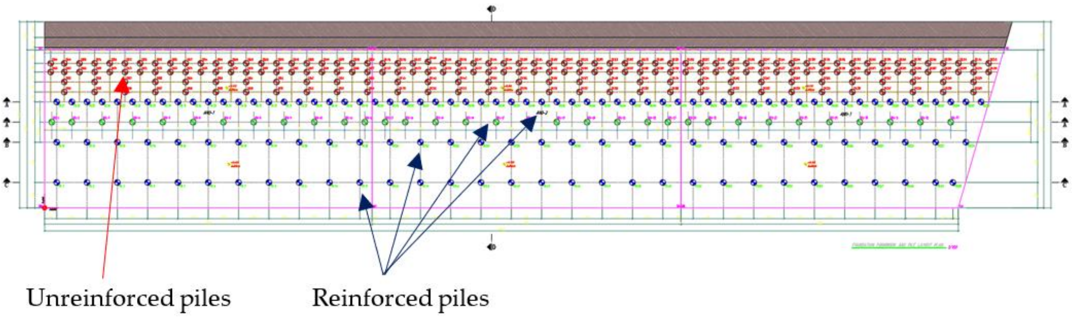
As a result of the structural and geotechnical evaluations, it was decided to use plastic piles in the front part of the foundation, which is the third alternative, and to apply the pile foundation system in the back part. However, in this chosen alternative, the plastic piles should not transfer load to the existing quay wall.

4. Numerical Modelling

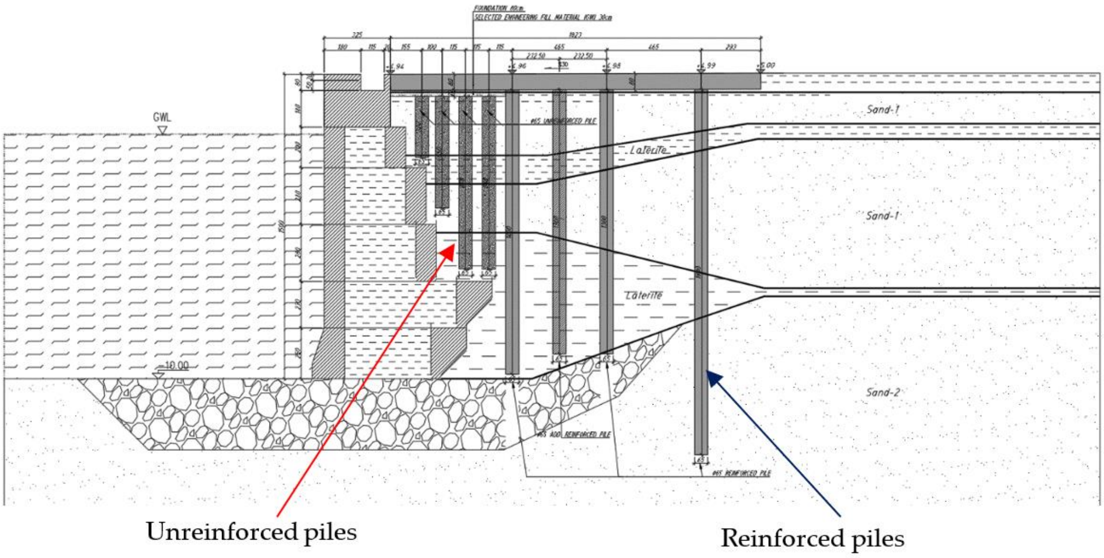
The existing conditions of the quay wall were insufficient to support the new crane loads due to insufficient bearing capacity and inadequate stability. For that reason, a piled foundation platform was planned for the new crane loads within the scope of the project. The reinforced piles have been planned in the project site as bored piles 65cm in diameter, 13 meters, 14 meters, and 18 meters in length with varying spacings and designed as 4 rows. Along with the foundation piles, it is planned to apply plastic piles for soil improvement in the front part of the foundation near the shore. By evaluating the pile foundation system previously designed for the project site, it has been understood that the part of the system designed as cantilever flooring is unsafe. In this case, in addition to the support provided by 4 rows of the piled foundation system, 4 rows (3.00, 5.50, 8.50, and 8.50 meters in length) of 65cm in diameter unreinforced bored piles are planned under the cantilever slab part. Selected engineering fill material (USCS classification; well-graded gravel, GW) was placed in 30cm thickness above the unreinforced piles. The thickness of the foundation has been designed as 80cm. The layout plan showing the piled foundation system and typical section is given in Figure 6a. The unreinforced piles are planned parallel to the sea as shown in Figure 6b.

4.1. Calculation Methodology

The pile foundation system was evaluated using PLAXIS 2D finite element software. PLAXIS is a finite element program, developed for the analysis of deformation, stability, and groundwater flow in geotechnical engineering. It is a program for geotechnical applications in which soil models are used to simulate soil behavior. PLAXIS (plane strain model) was used to predict the total and horizontal displacements of the pile as well as stress distribution in and around the soil-pile foundation system. Then, stability analyses of the pile foundation were conducted using the SLOPE/W program. Finally, the 3D geometry of the system was modeled using CSI's SAP2000 software and reinforced concrete calculations were conducted for piles and the slab/foundation.



(**a**) Layout plan of the piles.



(**b**) Typical section.

Figure 6. Layout plan of the piles and typical section.

4.2. Soil Parameters

The soil parameters were selected from soil investigation repor (Table 1). Some of the parametersfor the anaysis were not directly available in the report, correlations available in the literature based on the SPT tests have been used. The soil stress-strain relationship has been modeled by applying the Hardening soil model. The rock material has been modeled with the Mohr-Coulomb model. For the concrete piles and raft, a linear elastic material set has been applied using the concrete weight and its stiffness. For the stability analysis conducted in SLOPE/W software, all the soil, rock, and concrete properties have been modeled with the available Mohr-Coulomb model.

The rockfill material was modeled with the Mohr-Coulomb model with unit weight as 22 kN/m^3, elasticity modulus as 80 MPa, Poisson ratio as 0.25, angle of internal friction as 40°, and cohesion value as 5 kPa.

Table 1. Soil parameters.

Parameters	Fill	S1-Laterite	S2-Sand 1	S3-Sand 2	S4-Clay	Units
γ_{unsat}, Unsaturated unit weight	19	19	19	19	19	kN/m^3
γ_{sat}, Saturated unit weight	20	20	20	20	20	kN/m^3
E_{50}^{ref}, Secant stiffness	50000	13000	16000	19500	30000	kN/m^2
E_{oed}^{ref}, Tangent stiffness	50000	13000	16000	19500	30000	kN/m^2
E_{ur}^{ref}, Unloading/reloading stiffness	150000	39000	48000	58500	90000	kN/m^2
m, Power for stress-level	0.5	0.5	0.5	0.5	0.7	-
c′, Cohesion	1	5	5	5	8	kN/m^2

ϕ′, Angle of internal friction	34	30	30	33	28	°
ψ, Angle of dilatancy	4	0	0	3	0	°

4.3. Pile Capacities and Horizontal Subgrade Modulus of Piles

The bearing capacity of the piles in soils has been calculated based on in-situ SPT test results. The piles have been designed as frictional piles; this means that only the side resistance has been taken into account while the base resistance has been ignored. By doing this, bearing capacity values have been conservatively obtained. The natural and saturated unit weights of the soil have been taken as 19 kN/m^3 and 20 kN/m^3, respectively. The saturated unit weight of the rockfill was taken as 22 kN/m^3. The unit weight of the pile material was taken as 25 kN/m^3. The groundwater level was taken as 3 meters from the surface. Since the piles are bored piles, the difference of the pile weight and the excavated soil was finally subtracted from the allowable pile bearing capacity. The buoyant weight of the pile was used since most of it is below groundwater, i.e., the unit weight of the pile material was subtracted from the unit weight of water.

Piles are generally applied o cohesionless soil layers. In cohesionless layers, the individual nominal resistance of each shaft in a group should be reduced by applying an adjustment factor η taken as shown in Table 2. For intermediate spacings, the value of η may be determined by linear interpolation.

Table 2. Design group reduction factors for bearing resistance of shafts.

Group Configuration	Shaft Centre-to- Centre Spacing	Reduction Factor for Group Effects, η
Single Row	2D	0.90
	3D or more	1.00
Multiple Row	2.5D	0.67
	3D	0.80
	4D or more	1.00

For adjustment factor η to be taken as 1, the center-to-center spacing of the piles must be 4D or more, i.e., 4 x 0.65= 2.6 meters. The group adjustment factor was applied to all the piles with center-to-center spacing of less than 2.6 meters. In A-axis the spacing is 1.75m, A′ and B axes the spacing is 2.325m, due to this the factors η for A, A′, and B piles are taken as 0.72 and 0.92, respectively. The layout of the piles on the A, A′, B, and C axes is shown in Figure 7. Pile bearing capacities were calculated as indicated in Table 3.

Table 3. Summary of the pile bearing capacity.

Pile No.	Allowable Pile Bearing Capacity Values (kN)	
	Service Condition	Extreme Condition
A-Axis; D=0.65m, L=14m	505	853
A′ and B-Axes; D=0.65m, L=13m	604	1021
C-Axis; D=0.65m, L=18m	1147	1932
Unreinforced Pile; D=0.65m, L=8.50	420	711
Unreinforced Pile; D=0.65m, L=5.50	247	418
Unreinforced Pile; D=0.65m, L=3.00	78	134

The geotechnical capacities of the piles do not exceed the allowable structural capacity of the piles (Concrete class is C30/37). Considering the soil profile and recommended parameters given in Section 4.2, the horizontal subgrade modulus of the piles was calculated using Bowles (1996) and CGS (1992) recommendations. The values have been conservatively selected as given in Table 4. These values have been used in the 3D analysis.

Preprints (www.preprints.org) | NOT PEER-REVIEWED | Posted: 19 September 2023 doi:10.20944/preprints202309.1218.v1

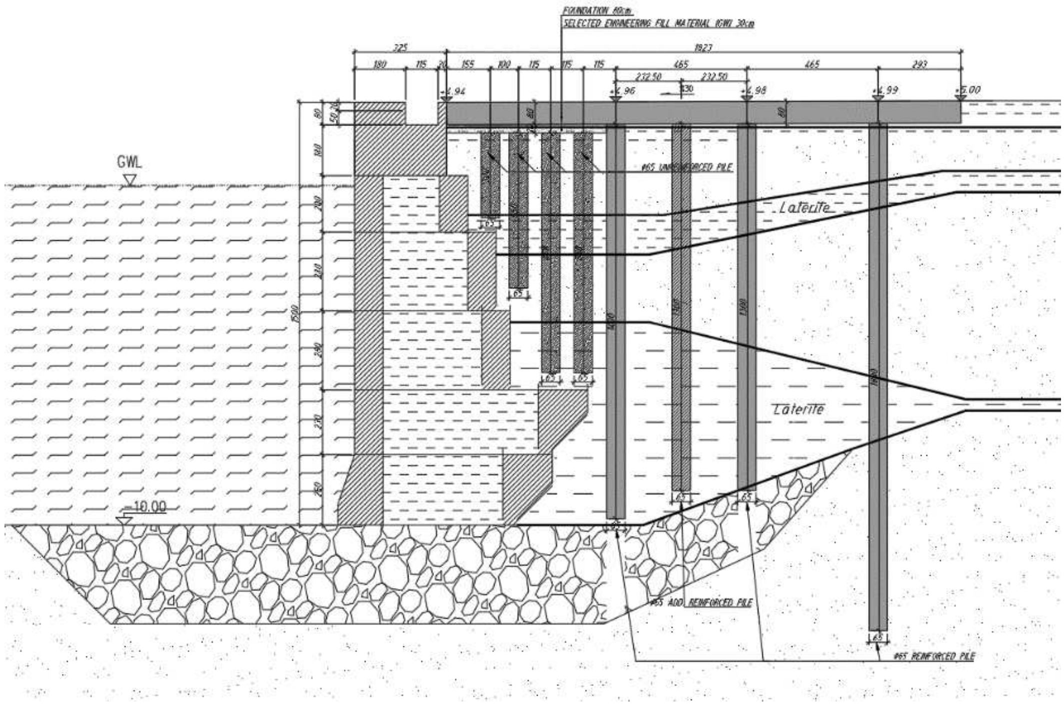


Figure 7. A, A′, B and C-axes of the pile rows in the section.

Table 4. Horizontal subgrade modulus.

Depth (m)	k_h (kN/m³)	Depth (m)	k_h (kN/m³)	Depth (m)	k_h (kN/m³)
0.00-1.00	7500	0.00-1.00	10000	0.00-1.00	10000
1.00-3.00	20000	1.00-3.00	20000	1.00-3.00	20000
3.00-14.00	17500	3.00-13.00	17500	3.00-11.00	17500

For the foundation soils under the cantilever slab, the depth where the bearing capacity will be considered is a depth range from the bottom of the footing to 1.5B below the bottom of the footing (AASHTO, 2012). To calculate the bearing capacity appropriately, the depth of influence due to the applied foundation loads was calculated. The minimum depth below the foundation where the applied stress due to foundation load decreases to 20 percent of the effective overburden pressure for coarse-grained materials (NAVFAC, 1986) is calculated as the depth of influence. The Boussinesq (1885) stress distribution solution has been selected to compute the stress distribution under the loaded areas. The depth of influence is obtained as 7.5 meters. The average SPT $N_{1,60}$ value within this depth has been calculated conservatively as 10. The bearing capacity after the application of the soil improvement by considering the area replacement ratios is calculated as follows; the average pile bearing capacity of the 4 rows (3.00, 5.50, 8.50 and 8.50 meters in length) of 65cm in diameter unreinforced bored piles is calculated as 304 kN. The calculated average unreinforced pile bearing capacity is given in Table 5.

Table 5. Average unreinforced pile bearing capacity.

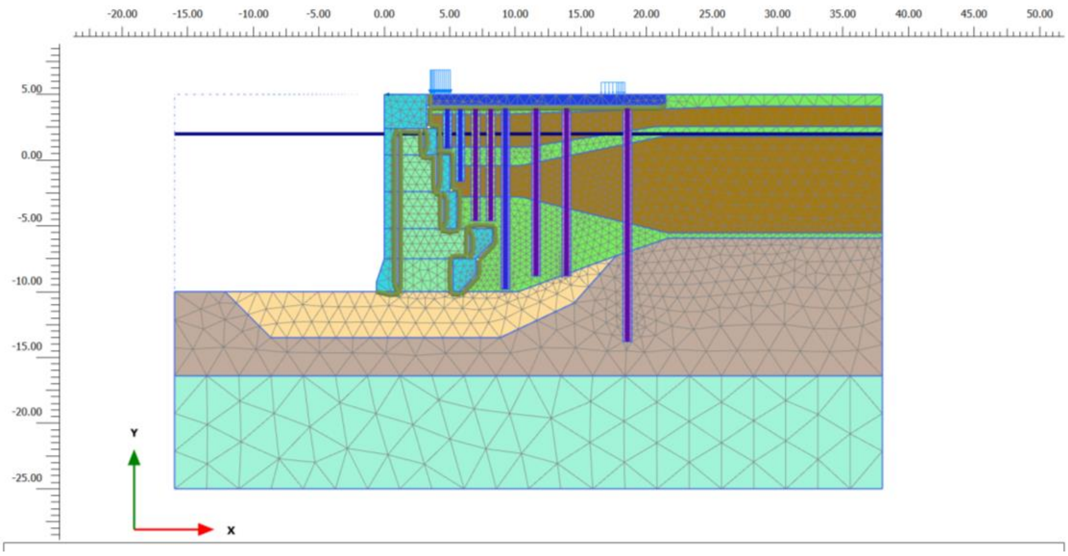
Pile No.	Pile Length (m)	Allowable Pile Bearing Capacity Values (kN)	Average Allowable Pile Bearing Capacity Values (kN)
		Service Condition	Service Condition
Unreinforced Pile; D=0.65m, L=8.50	8.5	420	
Unreinforced Pile; D=0.65m, L=5.50	5.5	247	304
Unreinforced Pile; D=0.65m, L=3.00	3.0	78	

 doi:10.20944/preprints202309.1218.v1

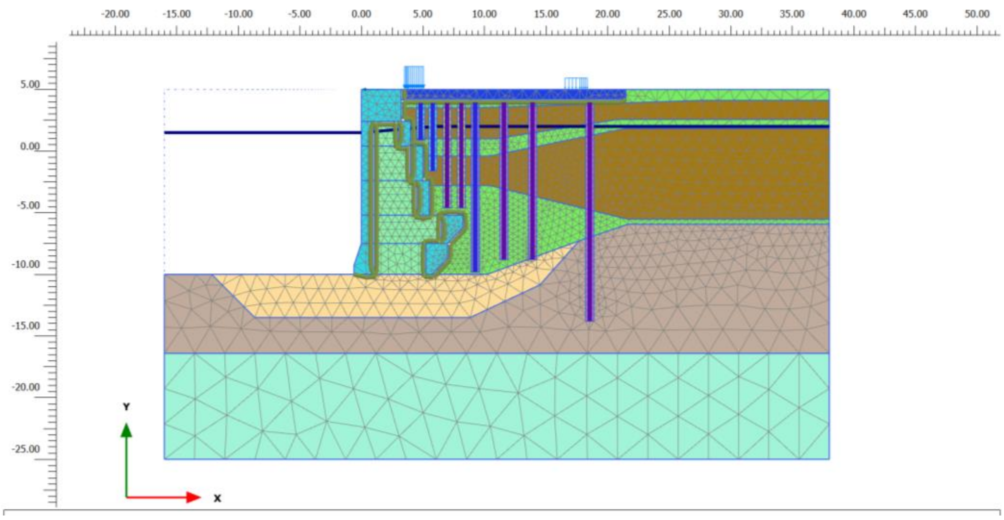
The unreinforced bored piles are planned with different center-to-center spacing. The average center-to-centre spacing is taken as 1.1m (in X) to 2.625m (in Y). The bearing capacity of the cantilever slab is sufficient under the foundation loads. Additionally, the vertical subgrade modulus to be used in the 3D analysis will be considered as 20000 kN/m^3. After improving the soil with unreinforced bored piles, a higher vertical subgrade modulus is calculated but the value was limited to 20000 kN/m^3, to be on the safe side.

4.4. PLAXIS Analysis

Plaxis 2D 2020 finite element program was used in geotechnical analysis. The soil parameters used in the model are determined in section 4.2. The geometry model for the analysis is given in Figure 8. The analysis has been conducted considering the initial groundwater level (+3.00m) and then with reference to EAU 2012 standard, the water level difference of 0.5m (+2.50m) has been taken into consideration. The crane loads were computed from the crane properties given in the crane appendix. The calculated uniform loads as 270 kPa and 135 kPa were considered in the analysis together with a Bollard Pull Load of 23 kN/m. Total horizontal displacement contours, total vertical displacement contours, and total horizontal displacements on piles obtained from analyses are presented in Figure 9 for GWL=3.00 m. The results obtained for GWL=2.5 meters and 3.00 meters are summarized in Table 6. Since there is no active earthquake movement in the project area, analyses under dynamic loads were not required.

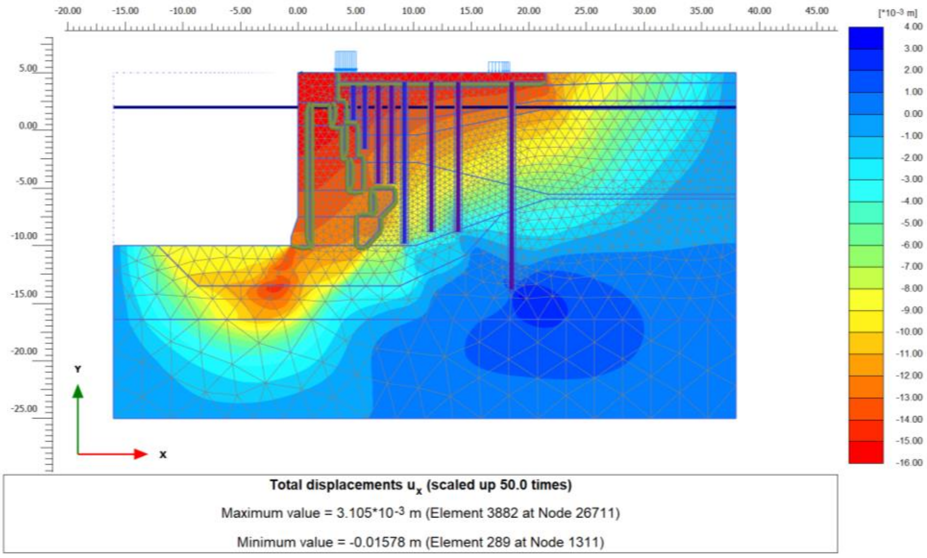


(**a**) Geometry model for initial water level at GWL=3.00m.

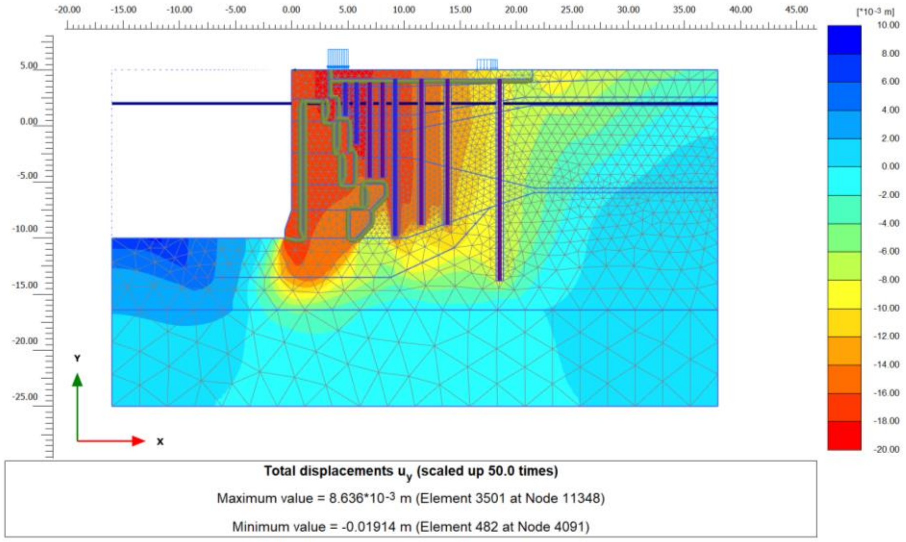


(**b**) Geometry model for water level at GWL=2.50m.

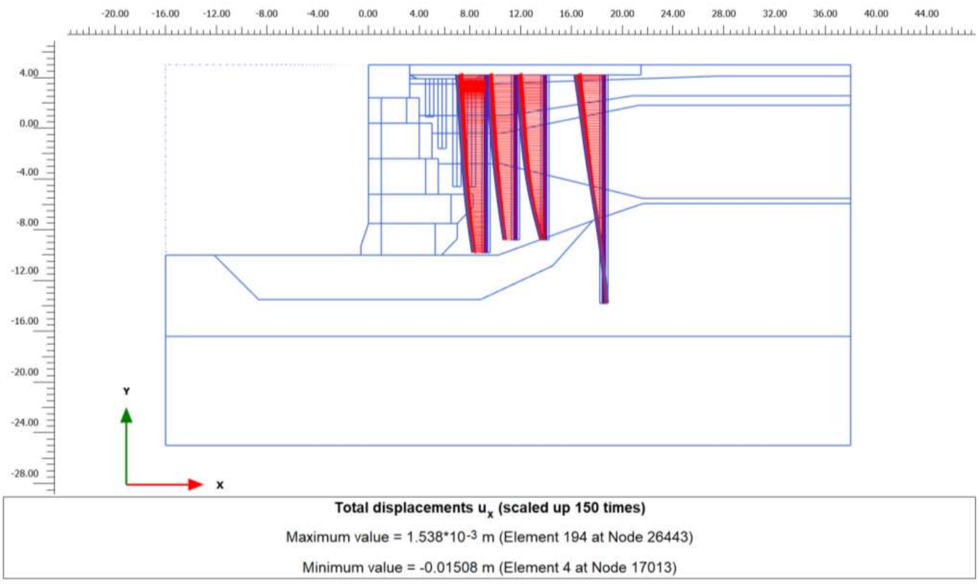
Figure 8. Geometry model.



(**a**) Total horizontal displacements.



(**b**) Total vertical displacements.



(**c**) Total horizontal displacements of piles.

Figure 9. Analysis results for initial water level at GWL=+3.00m.

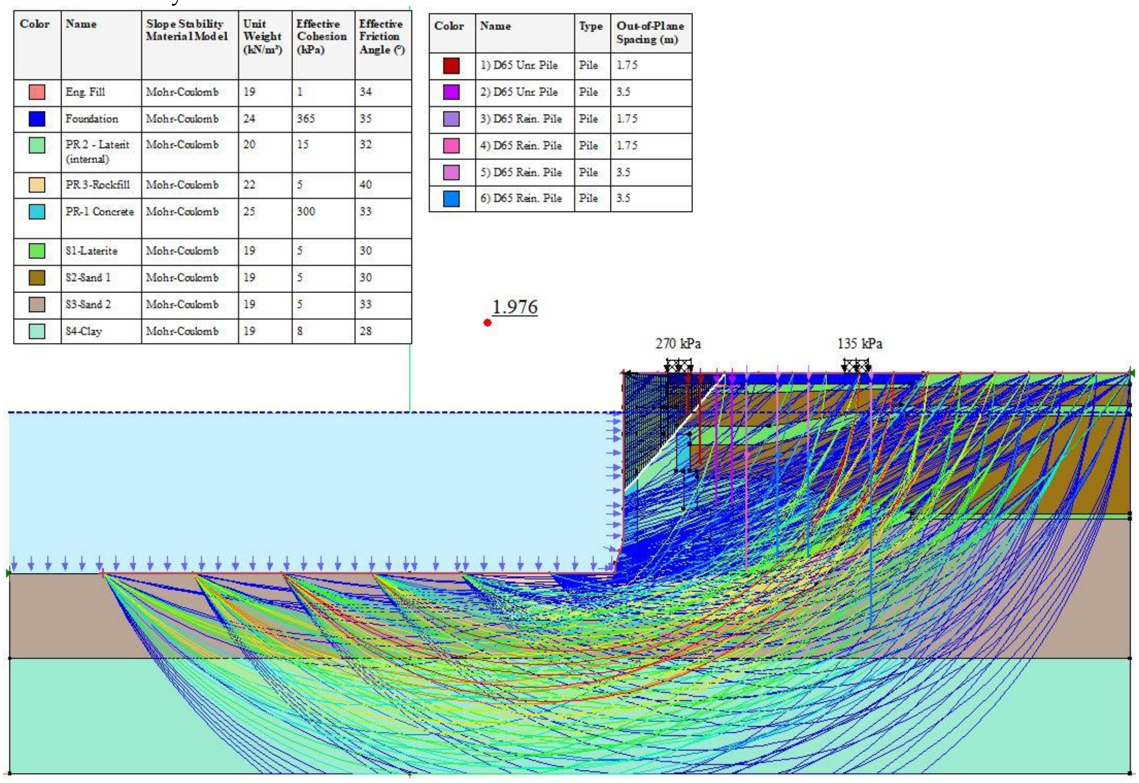
 doi:10.20944/preprints202309.1218.v1

Table 6. Analysis results for two GWL.

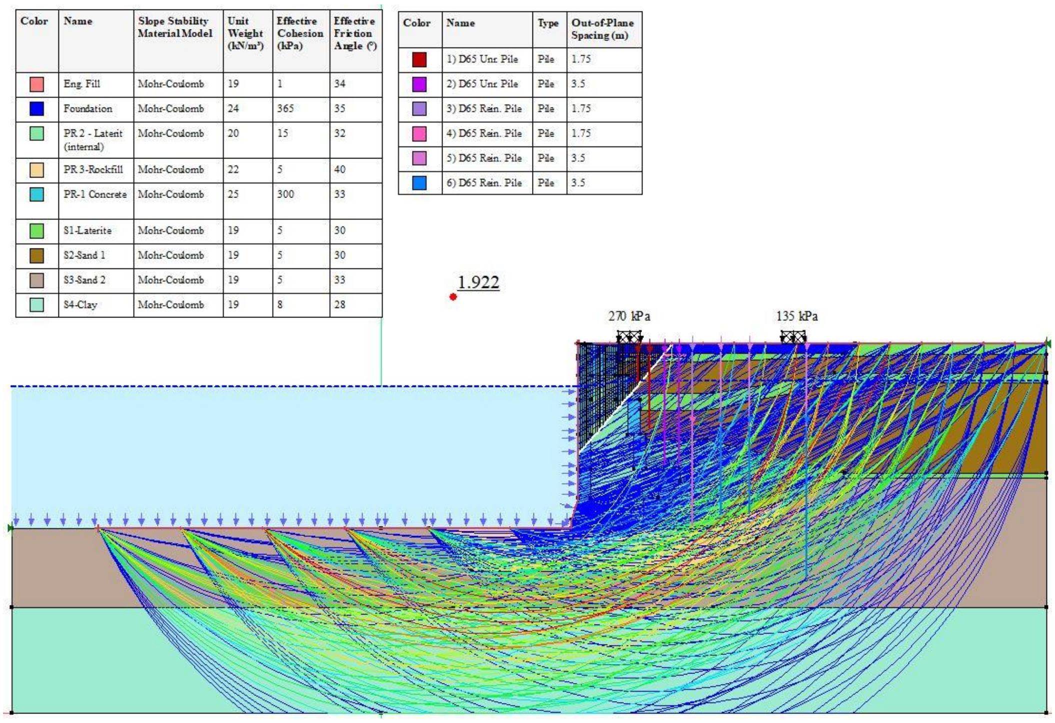
Displacement	Ground Water Level	
	+2.50 m	**+3.00 m**
Total displacement	2.985 cm	2.436 cm
Total horizontal displacement	2.109 cm	1.578 cm
Total vertical displacement	2.195 cm	1.914 cm
Maximum total horizontal displacement of piles	2.028 cm	1.508 cm

4.5. Stability Analysis with SLOPE/W

In order to determine the overall stability of the system after plastic loading, analyses were made for two different groundwater levels. The overall stability analysis results for the two groundwater levels are presented in Figure 10. For the analysis with the initial groundwater level of +3.00m, the factor of safety is obtained as 1.976 and for the analysis with the groundwater level of +2.50m, the factor of safety is obtained as 1.922. After the revision of the project, the critical factor of safety did not change, and the obtained factor of safety values are above 1.50 in all cases. So, there is no stability problem in the system.



(**a**) Factor of safety for initial water level at GWL=3.00m.



(**c**) Factor of safety for initial water level at GWL=2.50m.

Figure 10. Stability analysis results.

4.6. Structural Analysis with SAP 2000

Analyses were carried out using the SAP 2000 program to determine the effects on the piles and the foundation after the geotechnical design. In the analysis, piles were defined to the model, and soil conditions were introduced to the program with springs. In addition, vertical soil springs are used in the console part of the foundation. The analysis model is shown in Figure 11. Different load combinations were used in the analyses, and the crane load moving on the crane was defined to the system as a separate combination repeatedly.

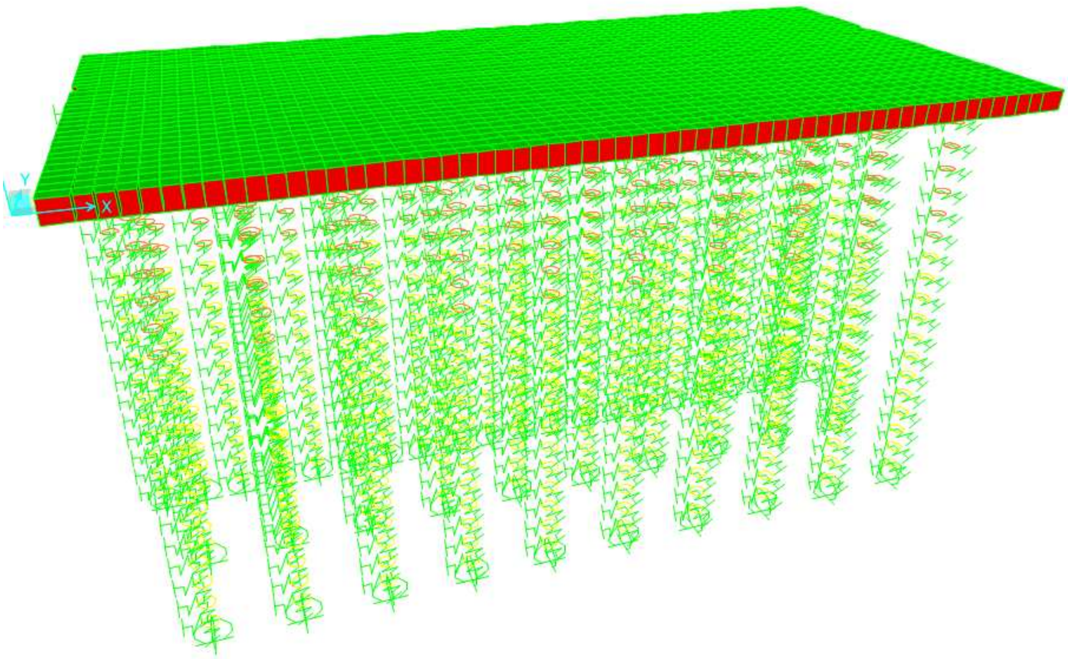


Figure 11. Structural analysis model.

4.7. Evaluation of the Bearing Capacity

The bearing capacity of the piles after conducting 3D SAP2000 analysis has been evaluated in this section. The axial forces in the piles obtained from the analyses and the calculated allowable bearing capacities of the piles are given in Table 7. As seen in Table 7, the axial forces are less than the pile-bearing capacities, so the piles can safely carry the loads imposed on them.

Table 7. Axial forces and allowable bearing capacities of the piles.

Pile No.	Service Condition		Extreme Condition	
	Maximum Axial Forces (kN)	Allowable Pile Bearing Capacity Values (kN)	Maximum Axial Forces (kN)	Allowable Pile Bearing Capacity Values (kN)
A-Axis; D=0.65m, L=14m	429	505	652	853
A′-Axis; D=0.65m, L=13m	330	604	498	1021
B-Axis; D=0.65m, L=13m	482	604	720	1021
C-Axis; D=0.65m, L=18m	1116	1147	1686	1932

4.8. Evaluation of Pile Performances by Pile Loading Tests

A pile loading test was carried out in the field to determine the vertical load capacities of the designed piles. In the experiment, reaction was taken from 4 piles and the pile loading plan is presented in Figure 12. A reaction beam of 8.5 meters was used in the experiments, and 110 tons of dead load was placed on the assembly to achieve the desired load capacity. The loading arrangement is shown in Figure 13. The fast-loading procedure was used and 250% of the pile design load (283.75 tons) was reached at loading. The pile with a diameter of 65 cm is loaded up to 283.5 tons which is 250% of the service load and thereafter pile was unloaded in 4 steps. Settlement at maximum load is measured as 7.56 mm. The average displacement after unloading is measured as 4.81 mm.

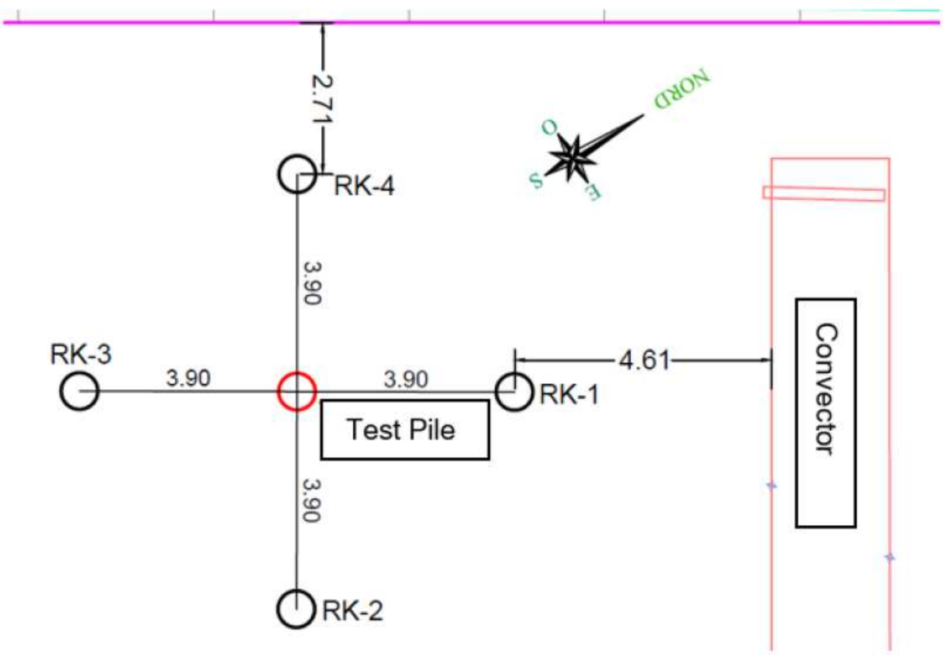


Figure 12. Pile load test plan.

Preprints (www.preprints.org) | NOT PEER-REVIEWED | Posted: 19 September 2023 doi:10.20944/preprints202309.1218.v1

Figure 13. Pile load test set-up.

5. Conclusions

In the study, the existing quay wall was evaluated, and geotechnical and structural alternatives of the new structure to be built for capacity increase were evaluated. Among the differ foundation ent alternative methods, a combined system was designed in the form of pile foundation and reinforced with plastic piles so as not to damage the existing dock structure. The results obtained from the study are presented below.

- The existing conditions of the port area was insufficient to support the new crane loads due to insufficient bearing capacity and inadequate stability. For that reason, a piled foundation platform was planned for the new crane loads within the scope of the project. The 4 rows reinforced piles have been planned in the project site as bored piles with 65cm in diameter, 13 meters, 14 meters and 18 meters in lengths with varying spacings. Soil improvement is also planned together with these reinforced piles.
- To this end, in addition to the support provided by 4 rows of piled foundation system, 4 rows (3.00, 5.50, 8.50 and 8.50 metres in length) of 65cm in diameter unreinforced bored piles are planned under the cantilever slab part. Selected engineering fill material (USCS classification; well-graded gravel, GW) was placed in 30cm thickness above the unreinforced piles. The thickness of the foundation has been designed as 80cm.
- The pile foundation system was evaluated using PLAXIS 2D finite element software. Stability analyses of the pile foundation were conducted using the SLOPE/W program. The 3D geometry of the system was modeled using CSI's SAP2000 software and reinforced concrete calculations were conducted.
- The results of PLAXIS analysis were concluded as; the displacement values obtained from the analyses carried out by taking the initial groundwater level as +3.00m and +2.50m, are well below the permissible limit values.
- The stability analysis was conducted with SLOPE/W and the obtained factor of safety values are above 1.50 in all cases. So, there is no stability problem in the system.
- The axial forces obtained as a result of the analysis are less than the pile-bearing capacities, so the piles can safely carry the loads imposed on them. The bearing capacity of the cantilever slab was found to be sufficient under the foundation loads.
- The pile with a diameter of 65 cm is loaded up to 283.5 tons which is 250% of the service load and thereafter unloaded 4 steps. Settlement at maximum load is measured as 7.56 mm. The average settlement after unloading is measured as 4.81 mm.

- As seen from this case study, the capacities of existing port structures can be safely increased, provided that necessary geotechnical precautions are taken.

Funding: This research received no external funding.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: Not applicable.

Conflicts of Interest: The author declare no conflict of interest.

References

1. Yang, Z.; Elgamal, A.; Abdoun, T.; Lee, C.J. A numerical study of lateral spreading behind a caisson type quay wall. International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, 26-31 March 2001, San Diego, CA, USA.
2. Alyami, M.; Rouinia, M.; Wilkinson, S.M. Numerical analysis of deformation behaviour of quay walls under earthquake loading. Soil Dynamics and Earthquake Engineering, **2009**, *29*, 525–536.
3. Zekri, A.; Ghalandarzadeh, A.; Ghasemi, P.; Aminfar, M.H. Experimental study of remediation measures of anchored sheet pile quay walls using soil compaction. Ocean Engineering, **2015**, *93*, 46–63.
4. Habets, C.J.W.; Peters, D.J.; Gijt, G.; Metrikine, A.V.; Jonkman, S.N. Model solution for performance-based seismic analysis of an anchored sheet pile quay wall. International Journal of Civil, Environmental, Structural, Construction and Architectural Engineering, **2016**, *10*, 3.
5. Hamed, O.M.; Mansour, M.F.; Abdel-Rahman, A.H.; El-Nahhas, F.M. Investigating the behavior of an existing quay wall using the characteristic parameters of port-said clay. **2017**, *35*, 483–499.
6. Tan, H.; Jiao, Z.; Chen, J. Field testing and numerical analysis on performance of anchored sheet pile quay wall with separate pile-supported platform. Marine Structures, **2018**, *58*, 382–398.
7. Skopal, V.; Clifford, F.; Widfeldt, M. The combined use of geostructures and ground improvement to the rehabilitation of existing quay walls. Proceeding of the XVII ECSMGE-2019. https://doi.org/10.32075/17ECSMGE-2019-0513.
8. Aldelfee, A.N.; Aldefae, A.H.; Zubaidi, S.L. Numerical modeling of seismic performance of gravity quay wall. IOP Conf. Series: Materials Science and Engineering, **2021**, *1058*, 012038, IOP Publishing. https://doi.org/10.1088/1757-899X/1058/1/012038.
9. Elshafey, M.; Elnaggar, M.; Abdelaziz, A.Y. Numerical investigation of quay walls reinforced with geosynthetic. Alexandria Engineering Journal, **2021**, *60*, 2303–2313.
10. Nguyen, A.D.; Kim, Y.S.; Kang, Y.S.; Kim, H.J. Numerical analysis of static behavior of caisson-type quay wall deepened by grouting rubble-mound. International Journal of Geo-Engineering, 2021, 12:1.
11. Brinkgreve, R.B.J., "PLAXIS 2D VIP User's Manual", Delft University of Technology & PLAXIS bv the Netherlands, 2020.
12. Geo-Slope İnternational Ltd. SLOPE/W Tutorial; Geo-Slope İnternational Ltd.: Calgary, AB, Canada, 2007.
13. Wilson, E.L.; Habibullah, A. SAP 2000 software, version 19. Computer and Structures, Inc. (CSI), Berkeley, 2017.
14. Bowles, J.E. Foundation Analysis and Design. 1997.
15. CGS, Canadian FoundationEngineering Manual, Canadian Geotechnical Society, Montreal, 1992.
16. AASHTO, AASHTO LRFD Bridge Design Specifications, 2012.
17. Naval Facilities Engineering Command (NAVFAC), Soil Mechanics, Design Manual 7.01, 1986.
18. Boussinesq, J., Application des potentiels a l'etude de l'equilbre et du movement des solids elastiques, Gauthier-Villars, Paris, 1885.
19. EAU, Recommendation of the Committee for Water Front Structures issued by the Society for Harbour Engineering and the German Society for Soil Mechanics and Foundation Engineering, 2012.