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Effect of Deepening In-front Of Port-Said East Port Diaphragm Quay Wall

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ABSTRACT

During the last two decades, new container ship generations had come into service, as a result of the huge growth of container trade. New container ships with larger dimensions may lead to the need to develop many of container terminals by either just deepening in front of quay walls or by deepening and replacing existing quay cranes with ones of higher capacities. In Port Said area there are several ports that need to keep pace with the tremendous progress in ship sizes. One of these ports is the Port Said East Port container terminal located on the Mediterranean Sea to the north of Egypt. The diaphragm wall which services as a berthing structure in this port is one of the deepest diaphragm wall structures built in soft clay, 62.5m deep below lowest astronomical tide (LAT). The existing water depth in the front of the quay wall is 18 m. This paper describes a finite element approach for analyzing the behavior of the quay wall under development scenarios using static calculation only. The finite element programs PLAXIS 2D Version 8.2 and PLAXIS 3D Version 1.6 have been used to analyze the performance of the structural elements, soil and the overall stability under deepening and the increase of crane wheel loads to accommodate the expected future ship sizes. The results showed that the diaphragm quay wall can resist safely 4 m deepening in front of the quay wall considering the existing crane loads. While, the results showed that width of cracks limitation will restrict increasing quay cranes loads.

Key words: Soil structure interaction, Quay wall, Barrette, Plaxis3D, Numerical Model, deepening.

1. INTRODUCTION

Quay walls are earth retaining structures, which used for mooring of ships and separate between land and water areas. They should be designed and constructed to resist safely the vertical loads such as; cargos, trucks, cranes etc., as well as the horizontal loads resulted from ship impacts, wind, and soil pressure. To fulfill the features of quay walls, three types of structures can be considered as main types of quay walls (gravity walls, embedded walls and open berth quay). *Handbook of Quay Walls* [2].

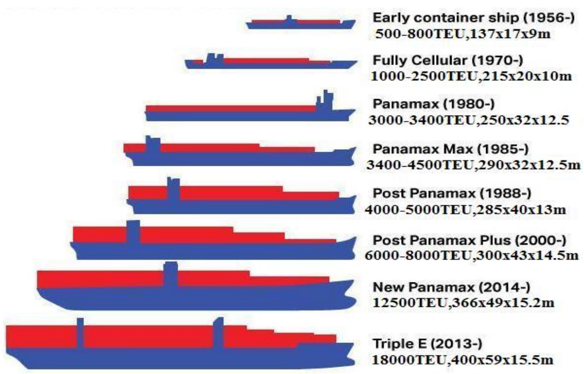

Figure 1: Evolution of container ships.

The continuously increasing dimensions of the ships play a significant role in the design of ports and lengths of quay walls. This fact require that the length of the quays to be extended and the retaining height in front of these structures to be increased by deepening. Figure 1 shows the evolution of container ships.

The diaphragm wall subjected to lateral loads induced by horizontal soil pressure and many other types of loads may be based on semi-empirical or theoretical analysis. The available data are generally limited and complicated by variations in geometry or soil conditions. Hence, there are many uncertainties in the estimation of bending moments and lateral deflections induced in diaphragm walls under these conditions. The literature on the adequacy of the finite element method (FEM) for modeling of such berthing structures to analyze their behavior during deepening is limited. If the bending moments and deflections induced in diaphragm wall due to deepening process can be accurately estimated, then the capacity of the structure elements can be checked accurately, the overall stability of quay wall can be correctly calculated and the cost of the deepening project can be lower as possible.

This paper discusses the finite element analysis for the diaphragm quay wall of the Port Said East Port container terminal, located on the Mediterranean Sea to the north of Egypt, due to development scenarios represented in deepening and increasing quay cranes loads. Figure 2 shows the location of the studied quay wall in Port Said East port.

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The first design and construction of this quay wall started in 1998 and the work ended in 2002, the design of this quay wall was discussed in *Hamza. and Hamed* [8].

Figure 2: Location of the studied quay wall.

The Finite element method can offer several approximations to predict true solutions. The accuracy of these approximations depends on the modeler's ability to portray what is happening in the field. Often, the problem being modeled is complex and has to be simplified to obtain a solution.

A number of case studies have been reported in the literature which gives the relationship between soil properties, structural properties, dredging sequence and the wall deflection. Among these *Dibiagio and Myrvoll* [5], *Davies* [4] , *Tedd et al.*[19], *Clough and O'Rourke* [3], and *Tamano et al.*[18]. The aspects of their studies included effects of wall construction on ground movements and changes in lateral earth pressure and water pressure and a numerical modeling of the effects of wall construction and ground movements.

Hamza and Hamed [8] carried out a three dimensional analysis for the east Port Said quay wall to evaluate the resulting displacement and straining actions under the different load combinations

Muthukkumaran and Sundaravadivelu [12] carried out a research on application of the analytical method to study the effect of dredging on piles and diaphragm wall-supported berthing structures.

Sincil [17] carried out a numerical analysis of anchored concrete pile walls and a comparison of field measurements and numerical values in terms of the stability of the structure and soil.

Ong et al. [13] made a comparison of finite element modeling of a deep excavation using 2-D finite element software, SAGE-CRISP version 5.1 and PLAXIS version 8.2.

Karamperidou [10] carried out a parametric analysis of seven different quay walls, for various loading combinations of given loads using an advanced computer program, PLAXIS.

Farshidfar and Nayeri, [6] Uses the shear strength reduction method to study soil slopes stability. In this method shear strength is considered to be reduced as less as failure occurs.

Mourillon [11] analyzes the influence of the deformed combined wall on the stability of the quay structure. Apart from the deformation of the combined wall, the designed penetrated depth was not reached. The difference between the designed penetration depth and the actual penetrated depth is around 2 meters. The research based on finite element program, Plaxis 3D, which takes into account the 3-dimensional effects of the quay structure and considers the actual soil behavior during calculations.

Gumucio, [7] performs a parametric study in the port of Rotterdam to assess the importance of relieving structures in quay walls. using finite element computer program PLAXIS.

Premalatha. and Muthukkumaran and Jayabalan [15] a numerical study on pile group supporting the berthing structures subjected to berthing/mooring forces and the forces arises due to dredging operations. A 2D Finite Element Model is developed using the geotechnical software Plaxis and is validated using the theoretical solution.

Paparis et. al., [14] studied the effect of berth deepening and strengthening to accommodate larger vessels for Port Elizabeth Container Terminal.

2. EXISTING BERTHING STRUCTURE

2.1 Structural Elements

Typical cross section of the studied quay wall structure is shown in Figure 3.

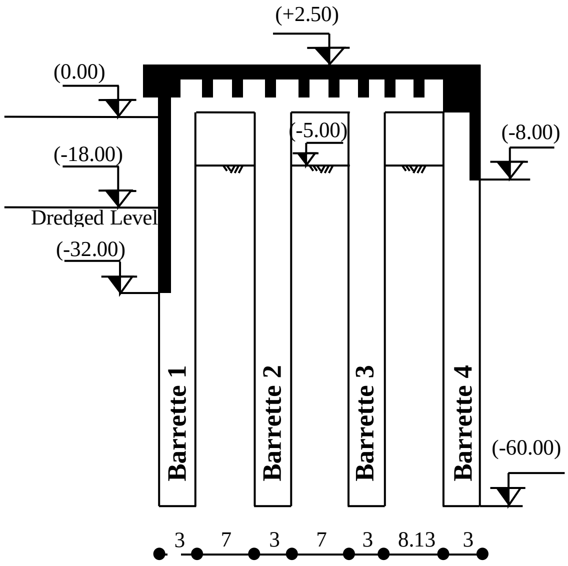


Figure 3: Quay wall cross section.

The quay wall deck of 1200 m length and 35 m width is supported on four barrettes each having 3x1 m cross-section, and extended to an average elevation of -60.0m. Between each two seaside and landside barrettes (1 and 4), there are two walls extend to -32.0 m in the seaside and to -8.0m in the landside respectively. The four barrettes are connected in the transverse direction by 3x0.8 m top beam. In the longitudinal direction the spacing between supporting structure formed from the four barrettes and the top beam is 7m. In the same direction there are front beam and rear beam which are used to support the crane, while the bollard loads accommodate by the front beam. The beam alignments of the quay wall are shown in Figure 4.

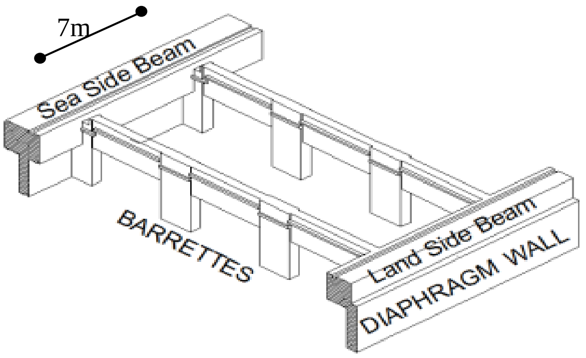


Figure 4: Quay wall beams alignment.

2.2 Geotechnical Data

The available geotechnical data for the studied area obtained from several soil samples taken from the project site by the Norwegian Geotechnical institute which conducts a specific testing program for the soil samples *Hamza. and Hamed [8]*. The resulting elastic and plastic soil parameters for the different soil layers are listed in Table 1. The sea water level is taken at elevation 0.0.

Table 1: Geotechnical parameters.

Type	Thick (m)	γ_b KN/m^3	C' Kpa	Φ' Deg	C_u Kpa	G Mpa
Clay(A)	5	17	0	24	-	1
Sand(B)	8.5	18.5	0	35	-	12
Clay(D)	15	15.5	0	24	1*	2*
Clay(E)	30	15	0	20	1*	2*
Clay(G)	34	17.5	20	20	150	25
Sand(F)	Inf.	20	0	35	-	60

1* Soil strength varies linearly with depth Cu =20 + 1.24 z (kPa), from -11.0 to -56.0.
2* the shear modulus varies linearly with depth G= 5.6 + 0.14 z (MPa), from -11.0 to -56.

2.3 Existing Loads

For the first design of the quay wall, the following types of loads were taken into consideration. These types of existing loads and its values are listed in Table 2.

Table 2: Loads considered for the first design of the quay wall.

Type of load	Value
Berthing loads	200 ton
Mooring loads	200 ton
Crane load	Vertical crane load = 80 ton/m' Horizontal crane load = 8 ton/m'
Surcharge loads	deck of the quay wall = 6 ton/m^2 road behind quay = 2 ton/m^2 stacking area behind the road = 6ton/m^2

3. DESCRIPTION OF APPROACH

In this paper, a finite element approach was used for analyzing the studied diaphragm wall supporting a berthing structure influenced by lateral soil movements generated by development scenarios presented in deepening and increasing crane loads. Considering the existing dredged level of -18.0 m and the crane load of 80 ton/m' as initial case, two more cases will be considered for the future port development, which are (1) deepening in front of the quay from -18.0 m down till -22.0 m without changing crane load (2) deepening in front of the quay from -18.0 m down till -22.0 m and increasing the crane load up from 80.0 to 120.0 ton/m'. Table 3 shows the dredged levels and crane loads used in analyzing the berthing structure performance under the different development scenarios.

Table 3: Dredged levels and crane loads used in development scenarios.

Development scenarios	Case name	Dredged level (m)	Crane load (ton/m')
Case 1 (Existing)	Case (-18,80)	-18.00	80
Case 2	Case (-22,80)	-22.00	80
Case 3	Case (-22,120)	-22.00	120

4. NUMERICAL MODELING

Finite element method has become more popular as a soil response modeling and prediction tool. This has led to increase pressure on researchers to develop more comprehensive descriptions for soil behavior, which in turn leads to more complex constitutive relationship. *Prevost and Popescu* [16] stated that for a constitutive model to be satisfactory it must be able to: (1) define the material behavior for all stress and strain paths; (2) identify model parameters by means of standard material tests; and (3) physically represent the material response to changes in applied stress or strain.

For this study, the model was analyzed using a finite element approach, which allows good representation of the diaphragm wall configuration and geometry, without being unduly complicated. The diaphragm walls are modeled with beam-column elements connected to the finite element mesh, and the soil strata are represented by 15 nodded elements of elastic-plastic Mohr-Coulomb model. Soil-structure interaction is modeled by means of a bilinear Mohr- Coulomb model. The model is defined by vertical "boreholes" and horizontal "work planes". The boreholes are used to define the soil's cross section, the ground surface level, and the ground water level. While, the work planes are used to define geometry points, geometry lines, clusters, loads, boundary conditions and structures. The work planes could be used to simulate construction phases and excavations. The geometry of the volume piles is defined vertically by specifying two work planes, between which, the piles should be drawn. The piles are then defined horizontally by choosing a cross section. The finite element programs PLAXIS 3D version 1.6 and PLAXIS 2D version 8.2 are used for this study. In the model study, the same dimensions of the field quay wall are adopted. The boundary of the model is taken about two times greater than the structural area so that the boundaries do not influence the results of the problem to be studied. Figure 5 shows the geometrical dimensions of the analyzed model and Figure 6 shows the typical finite element mesh of the quay wall. The development scenarios were modeled and in each case the following results are checked; displacements for certain points, deflection and moment for structure elements and the overall stability for the quay wall.

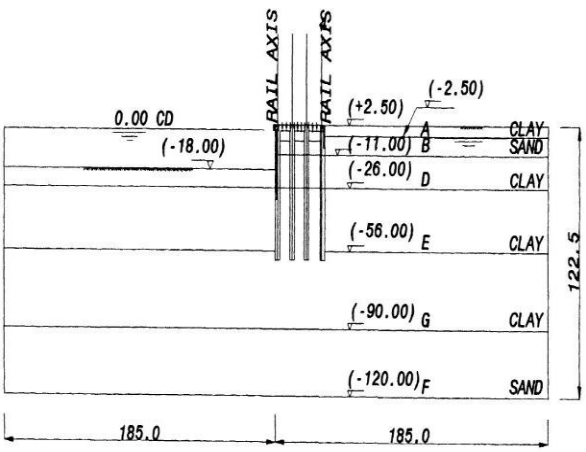


Figure 5: Geometry of the analyzed model.

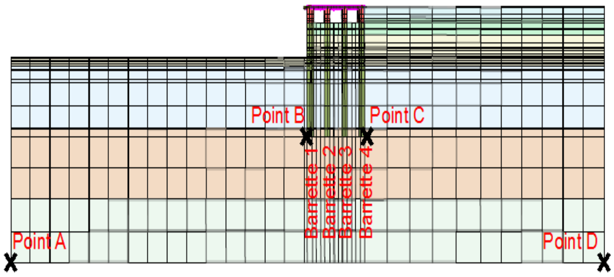


Figure 6: Finite element mesh of the quay wall.

5. RESULTS AND DISCUSSION

Results of the 3D model of the quay wall were analyzed for the previous three static cases that mentioned before. The resulting displacements and straining actions are used for checking the structural elements, the structure serviceability and the overall stability of the quay wall.

Figure 7 shows the deformed mesh for case (-18, 80) as an example. It is clear that the predominant movement of the quay for the existing case (-18, 80) is a horizontal movement and soil movement is much greater in top layers of soil and decreases towards bottom. From the deformed shape of the mesh also, it can be observed that the failure zone such as the critical slip circle may pass through the top layers. These results are repeated in the other two cases with the same trend but with a higher values due to applying deepening only in case (-22, 80) and due to deepening plus crane load increase in case (-22, 120). The previous results can be considered as qualitative results.

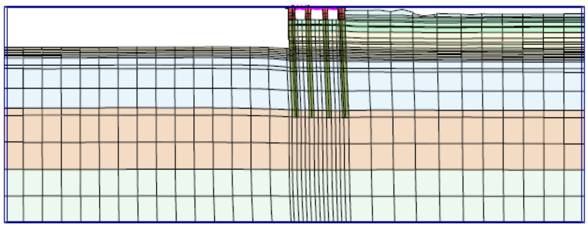


Figure 7: Deformed mesh, case (-18, 80), (scaled up to 100 times).

Figure 8 shows the displacement vectors for case (-22, 80) as an example. It is clear that the displacement mechanism of the structure is a rotational mode. The figure also shows that the soil behind the quay wall moves downward, the soil below the quay wall moves horizontally and the soil in front of the quay moves upward.

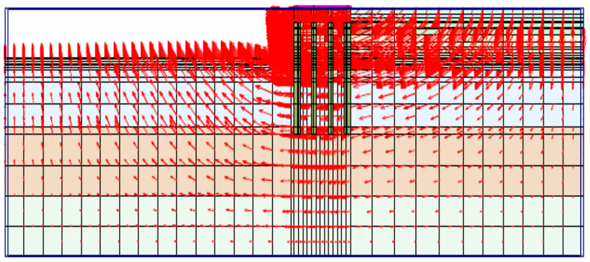
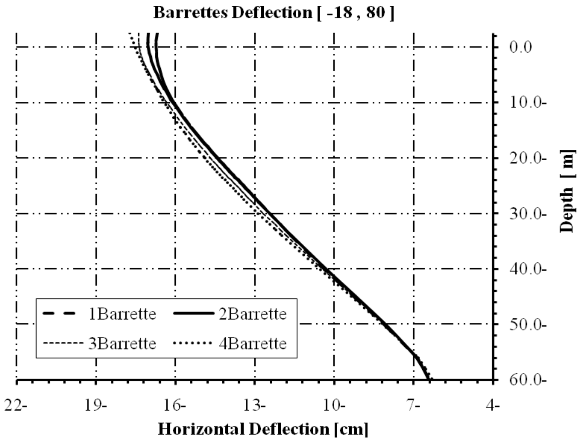


Figure 8: Displacement vectors, case (-22, 80).

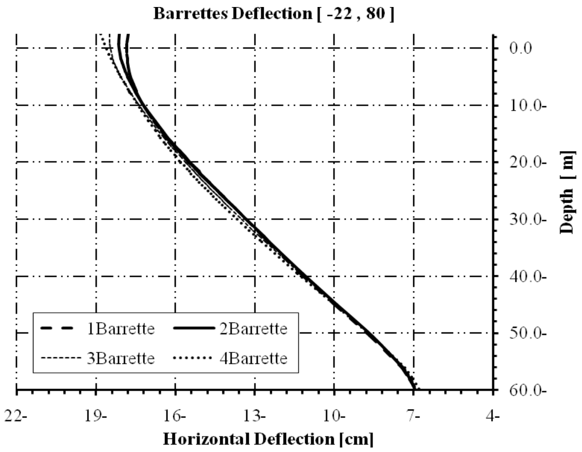
5.1. Structural Elements

In this section, the result of straining actions and the deformation of structural elements such as barrettes, crane beams and deck floor for the studied three cases will be discussed.

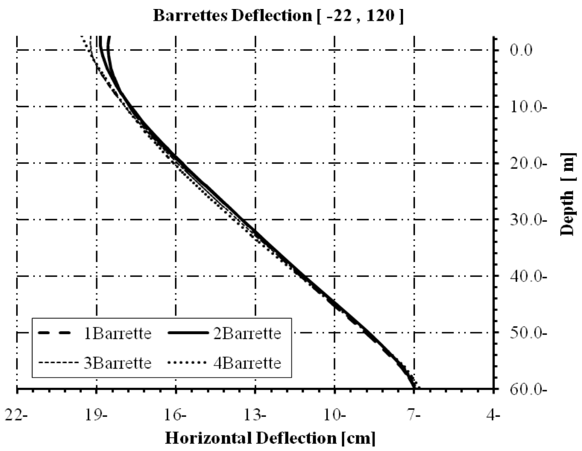
5.1.1. Barrettes



(a) Existing Case



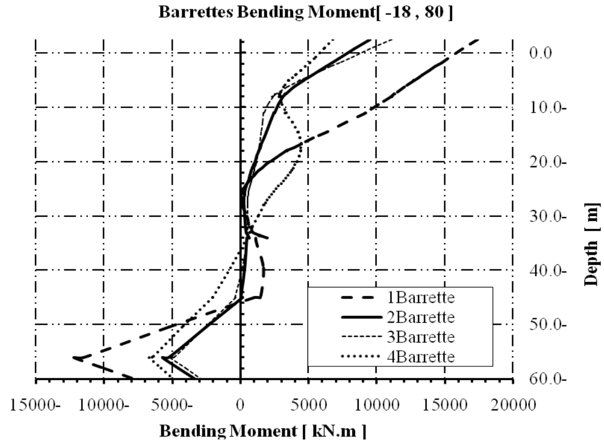
(b) Case of first scenario of development



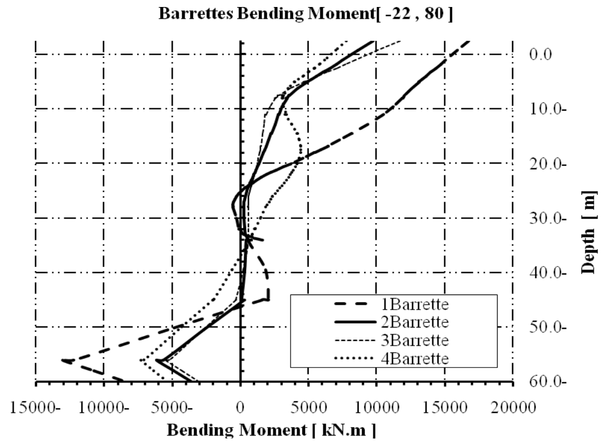
(c) Case of second scenario of development

Figure 9: Horizontal deflection for barrettes; (a) case (-18, 80), (b) case (-22, 80), (c) case (-22, 120).

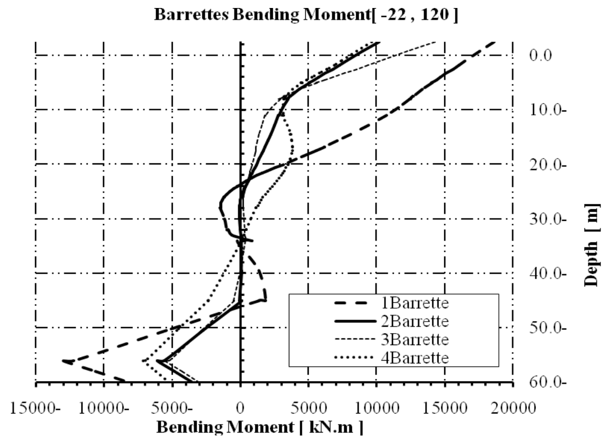
Figure 9 shows the horizontal deflection for barrettes due to case (-18, 80), case (-22, 80) and case (-22, 120). Barrette one was considered as an example of the results. It is clear that, for barrette one in case (-18, 80) the max. value of horizontal deflection is -16.60 cm and occurs at level 2.50m and this value increased in case (-22, 80) by about 6.5% and in case (-22, 120) by about 11% . This increase in the horizontal deflection can be due to the deepening in case (-22, 80) and deepening plus crane loads increase in case (-22, 120). In the same way the figure illustrates the changes for the other barrettes.



(a) Existing Case



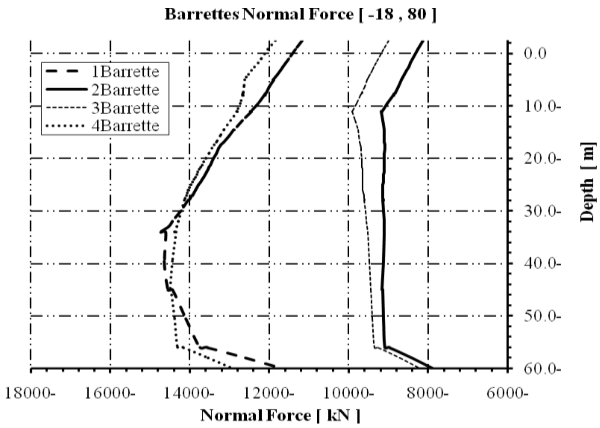
(b) Case of first scenario of development



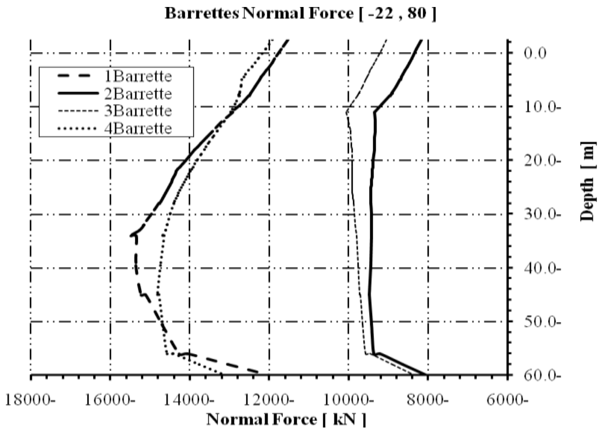
(c) Case of second scenario of development

Figure 10: Bending moment for barrettes; (a) case (-18, 80), (b) case (-22, 80), (c) case (-22, 120).

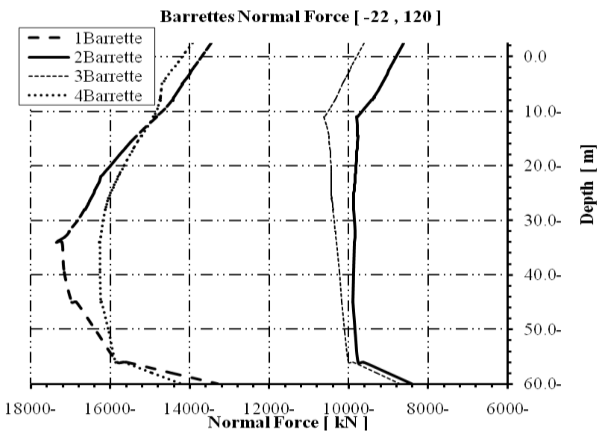
Figure 10 shows the bending moment for barrettes due to the different three cases under working loads. Barrette four was considered as an example for the results. It is clear that, for barrette four the max. value of bending moment is 6772 kN.m and occurs at level 2.50m and this value increased in case (-22, 80) by about 15% and increase in case (-22, 120) by about 42%. This increase in the bending moment is due to the deepening in case (-22, 80) and deepening plus crane loads increase in case (-22, 120). In the same way the figure illustrates the changes for the other barrettes.



(a) Existing Case



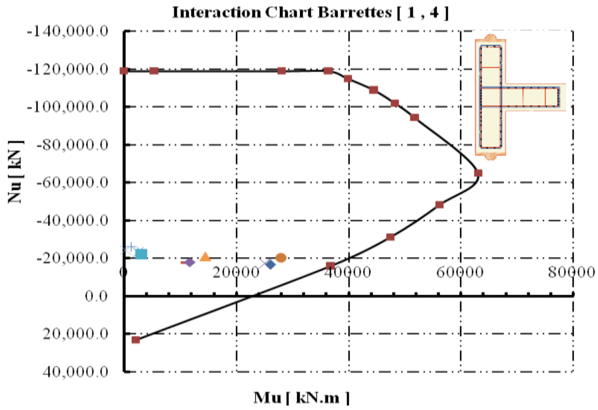
(b) Case of first scenario of development



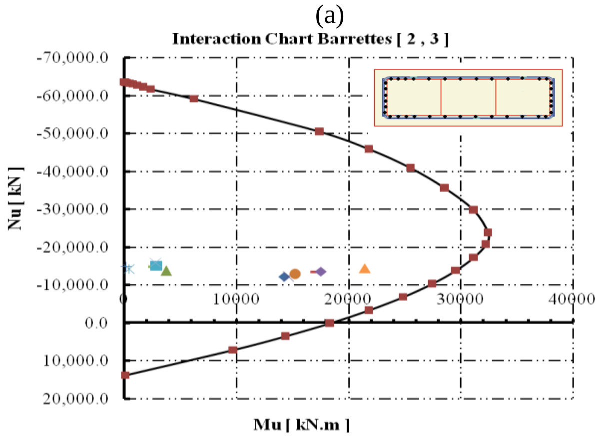
(c) Case of second scenario of development

Figure 11: Normal force for barrettes;
(a) case (-18, 80), (b) case (-22, 80), (c) case (-22, 120).

Figure 11 shows the normal force for Barrettes due to the different three cases under working loads. Barrette three was considered as an example of the results. It is clear that, for barrette three the max. value of normal force is 9900 kN and occurs at level -11.0m and this value increase in case (-22,80) by about 2% and increase in case (-22, 120) by about 7.5%. In the same way the figure illustrates the changes for the other barrettes.



(a)



(b)

Figure 12: Interaction diagrams for barrettes;
(a) Barrettes 1, 4 (b) Barrettes 2, 3.

To check that, the reinforced concrete section of barrettes (1, 4) with T. section shape and barrettes (2, 3) with Rec. section shape, satisfies the requirements of *ACI 318-95* [1], an interaction diagram was made. Figure 12 shows the interaction diagram for all barrettes due to the three cases. For the interaction diagrams a design point was selected as follows, for each barrette in each case there was two design points with coordinates; (max. bending moment, corresponding normal force) and (max. normal force, corresponding bending moment). Figure 12 shows that all the design points are lying inside the chart which mean that the concrete section is safe for all design cases for barrettes (1, 4) with T. section shape and barrettes (2, 3) with Rec. section shape.

Not only an interaction diagram check has been done but also a crack width analysis was used to check the barrettes sections. Figure 13 shows the width of crack for all barrettes due to the three cases.

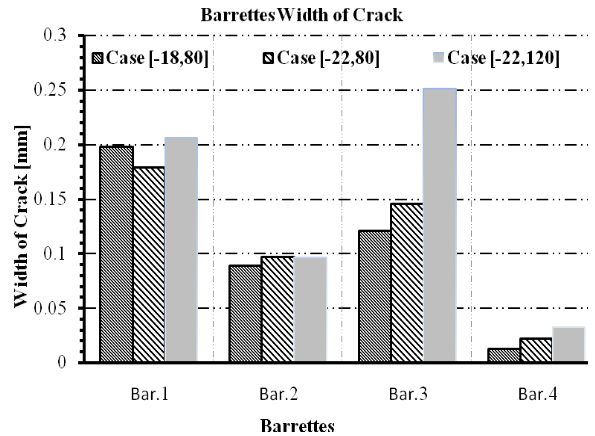
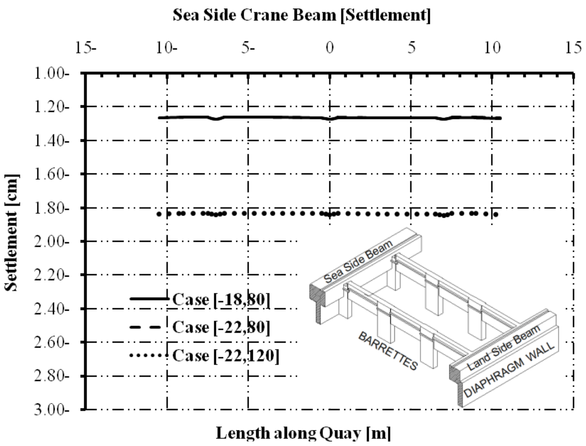


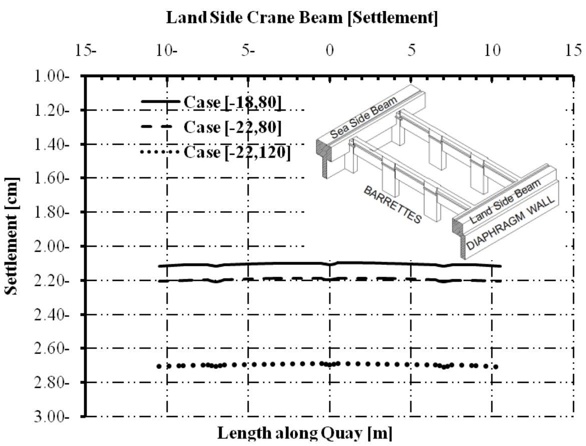
Figure 13: Width of crack for all barrettes due to the three cases.

The crack width analysis was made to satisfy the serviceability requirements of the *ACI 318-95* [1] under working loads. The crack width limitation is 0.20 mm. From figure 13, it can be observed that barrettes number one and three in case (-22,120) are unsafe because they break the limitation of the crack width.

5.1.2. Crane Beams



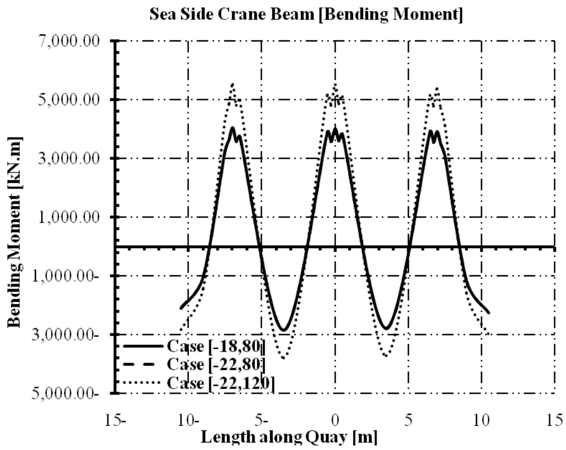
(a)



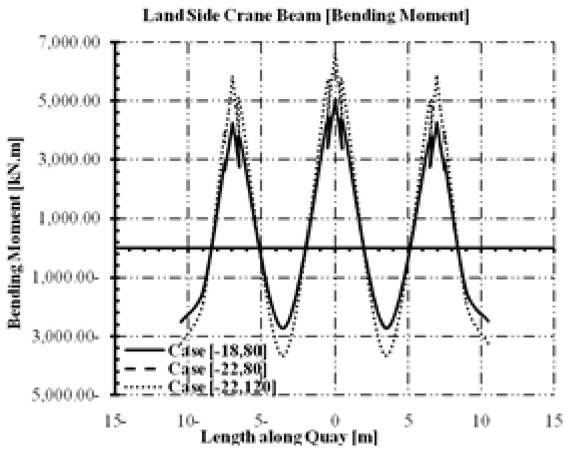
(b)

Figure 14: Vertical settlement of crane beams; (a) Sea side crane beam, (b) Land side crane beam.

As mentioned before that the displacement mechanism of the structure is a rotational mode then, it is expected that the land side crane beam will have vertical settlement greater than the sea side crane beam. Figure 14 shows vertical settlement of the sea side crane beam and the land side crane beam. From the figure it can be noticed that for the sea side crane beam the vertical settlement in case (-18, 80) almost coincides with the vertical settlement in case (-22, 80) and the average vertical settlement of sea side crane beam in case (-18, 80) is -1.27cm increased by 44.80% in case (-22, 120). The average vertical settlement of land side crane beam in case (-18, 80) is -2.11cm increased by 28% in case (-22, 120). The increase of crane load from 80 ton/m' up to 120 ton/m' may explain the increase in the settlement.



(a)



(b)

Figure 15: Bending moment of crane beams; (a) Sea side crane beam, (b) Land side crane beam.

Figure 15 shows the bending moment of crane beams under working loads. The bending moment of the land side crane beam is greater than the bending moment of the sea side crane beam for case (-22, 120) by about 20%. There is no significant changes in the values of bending moments between case (-18, 80) and case (-22, 80) for sea side crane beam or land side crane beam. There is noticeable increase in the values of bending moments between case (-18, 80) and case (-22, 120) for the sea side and land side crane beams due to the increase of crane load from 80 ton/m' to 120 ton/m'. For the sea side crane beam the max. value of bending moment for case (-18, 80) is 4009kN.m increased by 37.5% in case (-22, 120). The land side crane beam max. value of bending moment for case (-18, 80) is 4985kN.m increased by 32.4% in case (-22, 120).

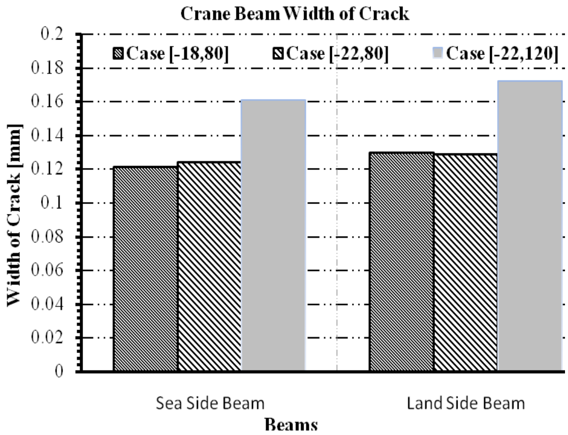


Figure 16: Width of crack for crane beams due to the three cases.

To check the sections of crane beams, a crack width analysis was carried out. Figure 16 shows the width of crack for crane beams due to the three cases. As mentioned before that the limit of crack width is 0.2 mm. From the figure it is clear that the beams are under the limitation of the crack width.

5.1.3. Deck

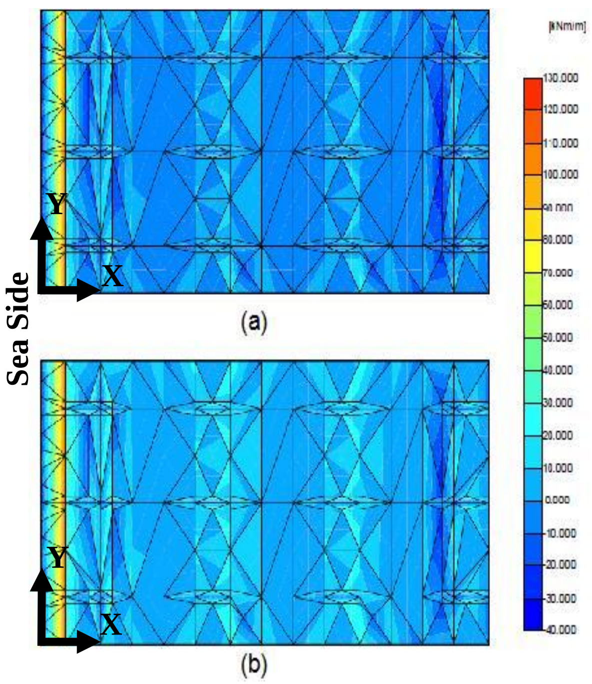


Figure 17: Bending moment in X direction for deck slab; (a) Case (-22, 80), (b) Case (-22, 120).

Figure 17 shows the bending moment for the deck slab in X direction for case (-22, 80) and case (-22, 120).The values of bending moment for the deck slab for all cases in directions X and Y have a little difference as shown in figure for X direction. The average of max. value for X direction is 123 kN.m/m and the average of min. value in X direction is -37 kN.m/m. the section of the slab can resist these values safely.

5.2. Soil

The results of the vertical total stress for soil at points A, B, C and D shown in Figure 6 are given in Table 4.

Table 4: Vertical total stress at points A, B, C and D.

Cases	Point A kN/m^2	Point B kN/m^2	Point C kN/m^2	Point D kN/m^2
Case (-18, 80)	-1969	-1862	-1344	-2167
Case (-22, 80)	-1948	-1920	-1331	-2167
Case (-22,120)	-1948	-1995	-1414	-2167

From the previous results it can be noticed that, for points A and D the vertical stress has unnoticeable changes and for point A the value of total stress decreased from 1969 to 1948 due to the removal of soil by deepening. While, the values of total stress for point D have no changes because it is far away from the effect of deepening and increasing crane loads. For point B the vertical stress increased may be due to the tilting of the structure towards sea side. For point C the stress first decreased due to the tilting towards sea side and then increased due to the crane load increase.

For determining the overall stability of the deck and the surrounding soils, the shear strength reduction method was used; soil shear strength is gradually decreased by the program till the first indications of failure appear. Safety factor is defined as the ratio of real shear strength of soil to the reduced shear strength.

The shear strength reduction method is better than the other methods for investigating slopes stability *Farshidfar and Nayeri [6]*. One of the advantages is that there is no need to the primary guess at determination of critical failure surface. Due to the availability of high-speed computer systems, this method is used increasingly today than before. Figure 18 shows the factor of safety for the three cases. It is obvious that the factor of safety of the soil decreased due to the deepening only by about 9% and decreased due to deepening plus crane load increase by about 13%.

Figure 18: φ – c reduction factor of safety for three cases.

5.3. Quay Wall Operation

Regardless of the capability of the structural elements to resist the additional straining action induced by the deepening and the crane load increase, other important factors must be taken into consideration such as the differential settlement of the crane beams and the tilting angle of the barrettes. Those factors are important to determine whether the quay wall operations will efficiently continue or not.

Those factors have limits to make sure that the quay wall operation will not be affected *Iai. et al.* [9]. The results of differential settlement between sea side crane beam and land side crane beam are shown in Table 5, while, the results of barrettes tilting angles are showed in Table 6.

Table 5: Differential settlement between sea side and land side crane beams for the three cases.

Case	S. Side beam average sett. (cm)	L. Side beam average sett. (cm)	Diff. (cm)	Allowable diff.
Case (-18, 80)	1.26	2.15	0.89	crane rail / 1000 = 3cm
Case (-22, 80)	1.27	2.22	0.95	crane rail / 1000 = 3cm
Case (-22,120)	1.84	2.7	0.86	crane rail / 1000 = 3cm

From the results shown in Table 5, it is clear that the differential settlement between sea side and land side crane beams is acceptable for the quay wall and crane operation.

Table 6: Barrettes tilting angles.

Cases	Bar.1 θ_{actual}	Bar.2 θ_{actual}	Bar.3 θ_{actual}	Bar.4 θ_{actual}	$\Theta_{all.}$
Case (-18,80)	≈ 0.1°	≈ 0.1°	≈ 0.1°	≈ 0.1°	2° - 3°
Case (-22,80)	≈ 0.1°	≈ 0.1°	≈ 0.1°	≈ 0.1°	2° - 3°
Case (-22,120)	≈ 0.1°	≈ 0.1°	≈ 0.1°	≈ 0.1°	2° - 3°

Also from the results shown in Table 6, it is clear that the tilting angles of all barrettes are acceptable for the quay wall and crane operation.

6. CONCLUSIONS

The present work demonstrates a verification study for the ability of developing the diaphragm quay wall existing at the container terminal of Port Said East Port considering two future scenarios under static condition. The first scenario is to perform deepening in front of the quay wall to the level of -22m instead of level -18m without changing crane wheels loads, while the second scenario is to perform the same deepening and increasing the crane wheel loads from 80 up to 120 ton/m'. The analyzed results of the study including deformations, capacities of structural elements, settlements, soil stress values and overall stability limitations, obtained for both scenarios had been presented. It could be concluded that, it is possible to perform deepening safely according to the first scenario, while the width of crack limitations preclude the possibility of performing the second scenario when using no engineering solutions.

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