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EFFECTS OF THE FRONTAL DEEPENING OF QUAY WALL FOR THE EAST PORT OF PORT SAID

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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 wheels 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.

Keywords: *Quay wall, barrette, Plaxis3D, numerical model, deepening.*

1.0 Introduction

Quay walls are earth retaining structures, which are 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 walls (Broeken and De Gijt, 2005). The continuously increasing dimensions of the ships play a significant role in the design of ports and lengths of quay walls. This fact requires that the length of the quay walls to be extended and the retaining heights in front of these structures to be increased by deepening.

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. Consedring a diaphragm quay wall, if the bending moments and deflections induced due to deepening process in front of it 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 can be lowered as possible.

This paper discuses 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 1: Location of the studied Quay Wall.

Figure 1 shows the location of the studied quay wall at Port Said East Port. The first design and construction of this quay wall started in 1998 and the work ended in 2002, (Hamza and Hamed, 2000).

A number of cases studies had been reported in the literature which gives the relationship between soil properties, structural properties, dredging sequence and the wall deflection. Among these studies are (Dibiagio and Myrvoll, 1972; Davies, 1982; Tedd *et al*., 1984; Clough and O'Rourke, 1990 and Tamano *et al*., 1996). The aspects of their studies included effects of wall construction on ground movements, changes in lateral earth pressure, water pressure and a numerical modeling of the effects of wall construction and ground movements. Hamza and Hamed 2000, carried out a three dimension analysis for the East Port Said quay wall to evaluate the resulting displacement and straining actions under the different load combinations. Muthukkumaran and Sundaravadivelu 2007, 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 2006, 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*. 2016, 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 2008, carried out a parametric analysis of seven different quay walls, for various loading combinations of given loads using advanced computer programs PLAXIS. Farshidfar and Nayeri 2015, used 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 2015, analyzed 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 was 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 considered the actual soil behaviour during calculation. Gumucio 2013, performed a parametric study in the port of Rotterdam to assess the importance of relieving structures in quay walls using finite element computer programs PLAXIS. Premalatha *et al*. 2011, developed a 2D Finite Element Model using the geotechnical software PLAXIS and carried outa numerical study on pile group supporting the berthing structures subjected to berthing/mooring forces and the forces arised due to dredging operations.

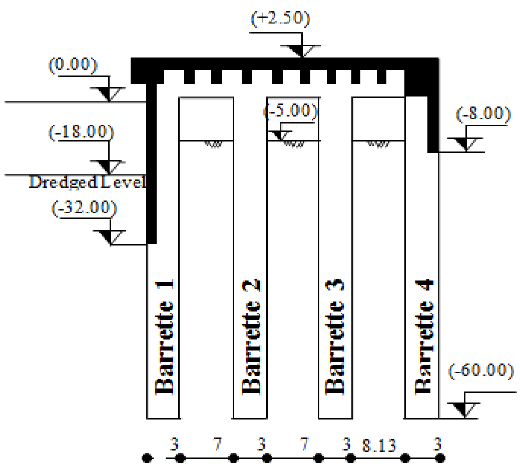


Figure 2: Quay Wall cross section (all dimensions in m.)

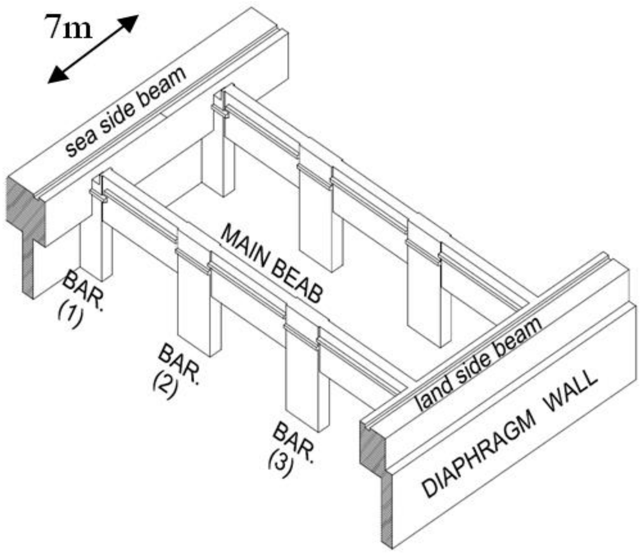


Figure 3: Quay Wall beams alignment

Paparis *et al*. 2004, studied the effect of berth deepening and strengthening to accommodate larger vessels for Port Elizabeth Container Terminal. De Gijt and Toorn 2008, discussed the factors which may play important roles in the future trend of quay wall design such as ship and cargo handling development. Douairi 2013, studied many options for creating extra depths in front of quay walls of which not all have been used in practice. Finally, Oung and Brassinga 2015, discussed widely the risks of upgrading existing quay walls such as deepenging in front of quay walls and increasing the loads on the quay surface.

2.0 Details of Existing Berthing Structure

2.1 Structural Elements

Typical cross section of the studied quay wall structure is shown in Figure 2. 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 -60.0 m. There are a front and rear walls extends to -32.0 m and -8.0 m 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 7 m. In the same direction there are front beam and rear beam which are used to support the quay crane, while the bollard loads are accommodated by the front beam. The beam alignments of the quay wall are shown in Figure 3.

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, 2000). 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: Existing Geotechnical Parameters.

Type	*Thick. (m)*	*ɣb KN/m³ unit weight*	*C' Kpa Cohesion*	*Φ' Deg Angle of friction*	*Cu Kpa Soil strength*	*G Mpa Shear modulus*
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 Cu =20 + 1.24 z (kpa), from -11.0 to -56.0.
2* the shear modulus varies linearly 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 had been taken into consideration. These types of existing loads and its values are listed in Table 2.

Table 2: Types and values of existing loads.

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/m² road behind quay = 2 ton/m² stacking area behind the road = 6ton/m²

3.0 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 represented 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 (scenarios) will be considered for the future port development, which are (1) deepening in front of the quay up till -22.0 m and without changing crane load (2) deepening in front of the quay up till -22.0 m and increasing the crane load up to 120 ton/m'. Table 3 shows the dredged levels and crane loads used to analyze the berthing structure performance under the 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

4.0 Numerical Modeling

Finite element method has become more popular as a soil response prediction tool. Prevost and Popescu, 1996 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 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 which provides the most basic soil behavior, without consideration of nonlinear stress-strain behavior or plasticity at failure. The basic principle of elasto plasticity is that stress and strain rates are divided into an elastic part and a plastic part. Mohr Coulomb model requires five input parameters which is Young’s modulus of elasticity, Poisson’s ratio, Cohesion strength, Internal angle of friction and Delatancy Angle. 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 pore pressure distribution. 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. Vertical model boundaries with their normal in x-direction (i.e. parallel to the yz-plane) are fixed in x-direction (ux = 0) and free in y- and z-direction. Vertical model boundaries with their normal in z-direction (i.e. parallel to the xy-plane) are fixed in z-direction (uz = 0) and free in x- and y-direction. The model bottom boundary is fixed in all directions (ux = uy = uz = 0). The finite element program PLAXIS 3D version 1.6 was used to model the quay wall and check the displacements and straining actions, while PLAXIS 2D version 8.2 was used to find the quay wall factor of safety. 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 4 shows the geometrical dimensions of the analyzed model and Figure 5 shows the typical finite element mesh of the quay wall. For the medium mesh representation, the total number of elements are 13496 and the total number of nodes are 43101. 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.

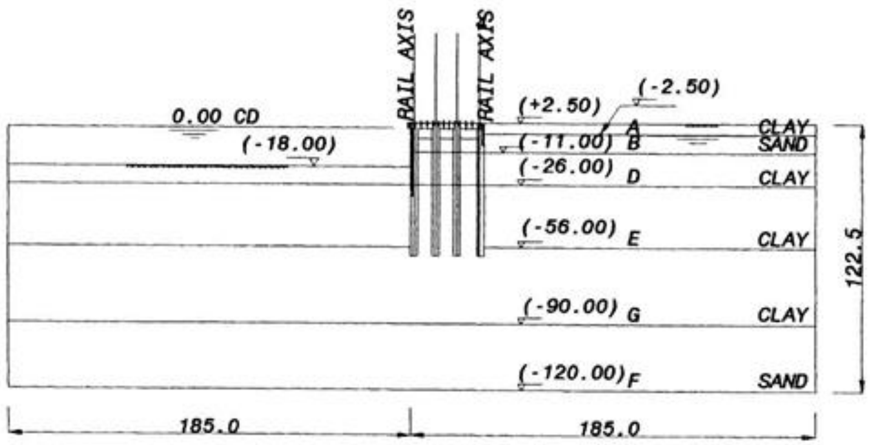


Figure 4: Geometry of the analyzed model.

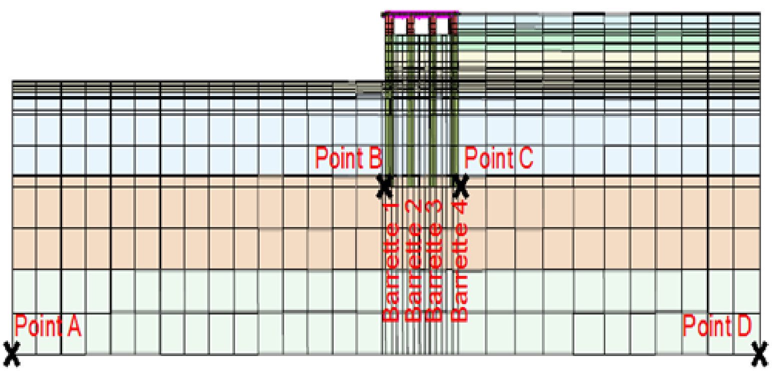


Figure 5: Finite element mesh of the quay wall.

5.0 Results and Discussion

The 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 6 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 quantitative results. Figure 7 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.

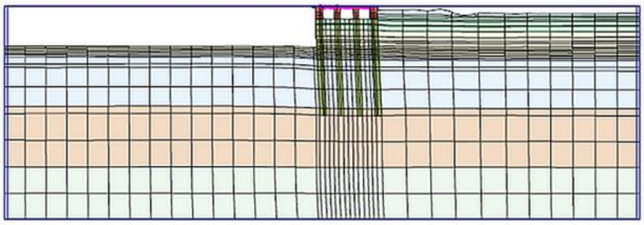


Figure 6: Deformed Mesh, case (-18, 80)

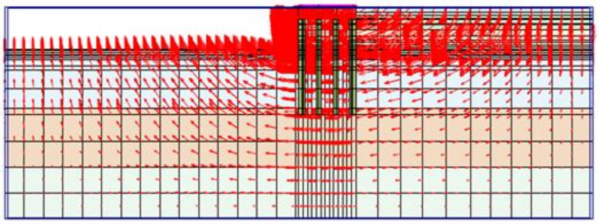


Figure 7: Displacement Vectors, (scaled up to 100 times) Case (-22, 80).

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.

5.1 Structural Elements

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

5.1.1 Barrettes

Figure 8 shows the horizontal deflection of all barrettes for the existing case of (-18, 80) and the second scenario of development, case (-22, 120) only. Barrette one was considered as an example of the results. It was found that, for barrette one in case (-18, 80) the max. value of horizontal deflection is -16.60 cm and occurs at level 2.50 m 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 is due to the deepening in case (-22, 80) and deepening plus crane loads increase in case of (-22, 120). In the same way the figure illustrates the changes for the other barrettes. Figure 9 shows the bending moment of barrettes for the existing case of (-18, 80) and the second scenario of development, case (-22, 120) only under working loads. Barrette four was considered as an example for the results. It was also found that, for barrette four the max. value of bending moment is 6772 kN.m and occurs at level 2.50 m 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. The barrettes results for moment and deflection are compared to barrettes design criteria for both strength and serviceability as will be illustrated in figures 11 and 12.

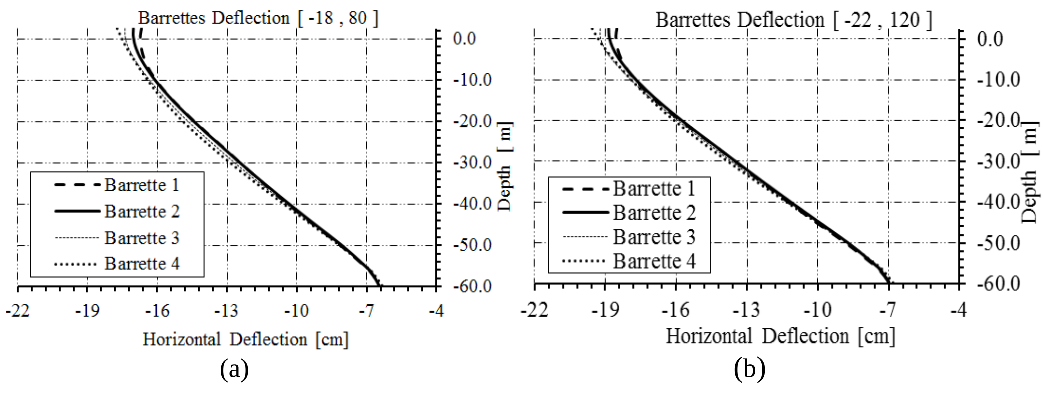
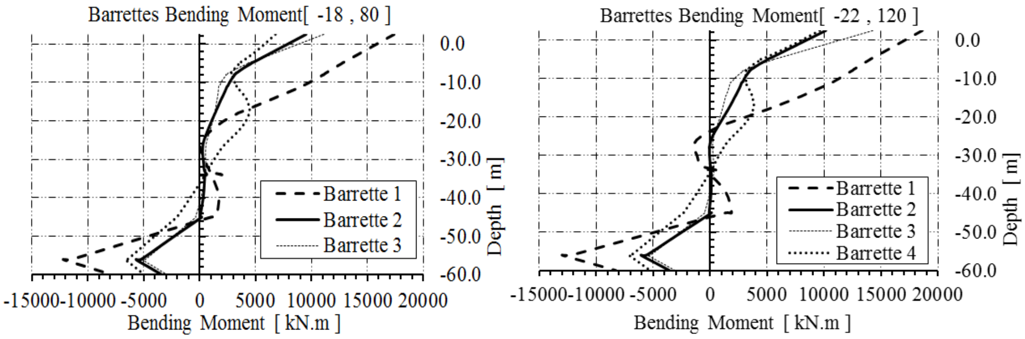
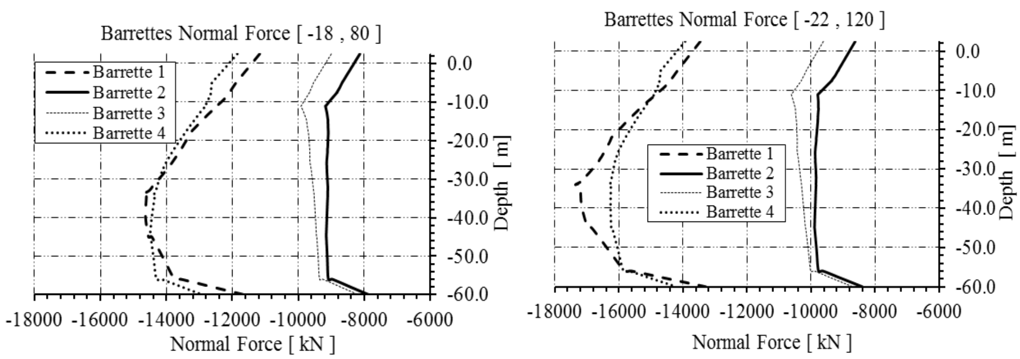


Figure 8: Horizontal deflection for barrettes (a) Existing Case (-18, 80), (b) Second scenario of development case of (-22,120).



(a) (b)

Figure 9: Bending moment for barrettes (a) Existing Case (-18, 80), (b) Second scenario of development case of (-22, 120).



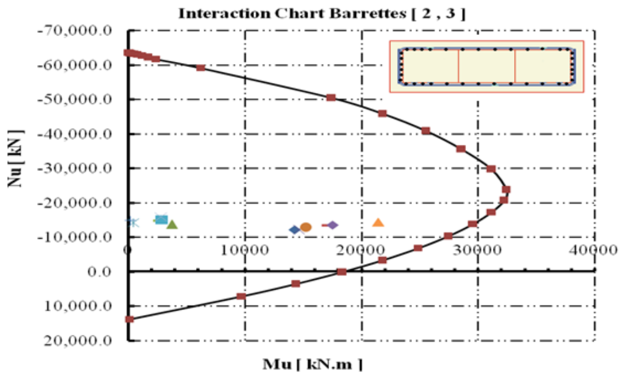
(a) (b)

Figure 10: Normal force, (Axial force) for barrettes (a) Existing Case (-18, 80), (b) Second scenario of development case of (-22, 120).

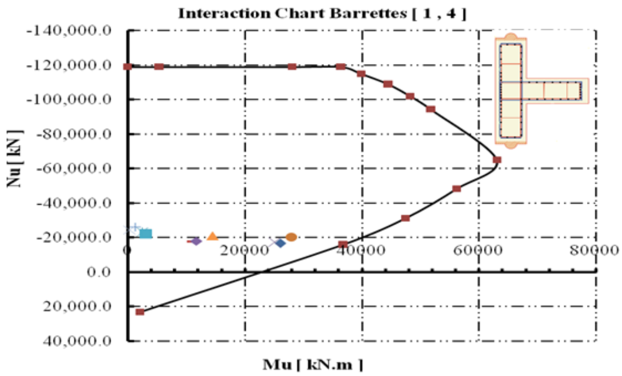
Figure 10 shows the normal force (axial force) of all barrettes for the existing case of (-18, 80) and the second scenario of development, case (-22, 120) only. Barrette three was considered as an example of the results. It is found that, in barrette three the max. value of normal force is 9900 kN and occurs at level -11.0 m 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. Figure 11 shows the interaction diagrams for all barrettes under the studied three cases.

The interaction diagrams was made 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, 1995). For this purpose a design points had been selected as follows: for each barrette in each case there were two design points with coordinates; (max. bending moment, corresponding normal force) and (max. normal force, corresponding bending moment). Figure 11 shows that all the design points are lying inside the chart which mean that the 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 had been done, but also a crack width analysis was used to check the barrettes sections. Figure 12 shows the width of crack for all barrettes under the three studied cases. The crack width analysis was made to satisfy the serviceability requirements of the (ACI 318-95, 1995) under working loads. The crack width limitation is 0.20 mm. From Figure 12, it can be observed that barrettes number one and three in case (-22, 120) break the limitation of the crack width.



(a)



(b)

Figure 11: Interaction diagrams for barrettes (a) For barrettes 1and 4, (b) For barrettes 2 and 3.

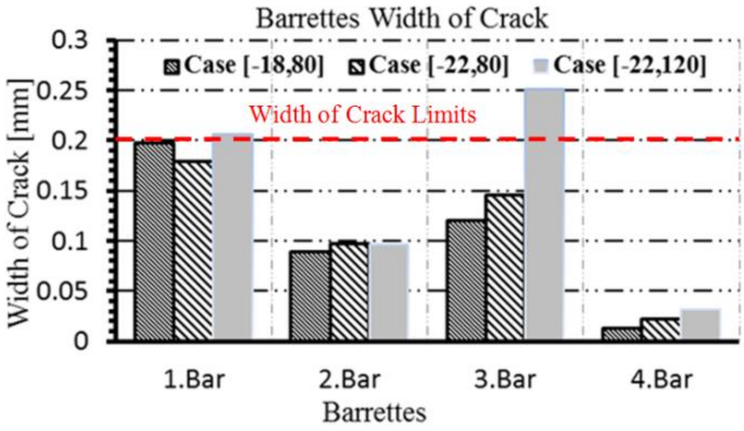
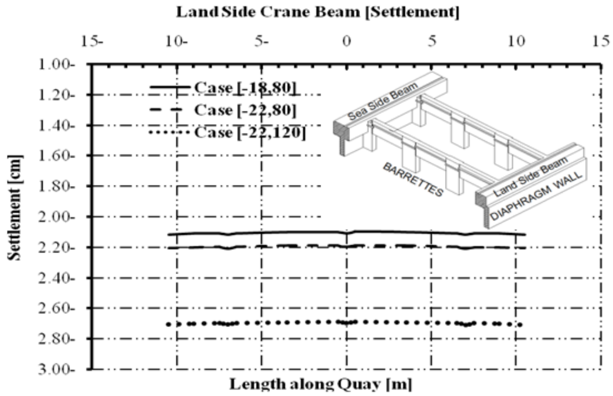


Figure 12: Width of crack for all barrettes of the diaphragm quay wall under the three studied cases.

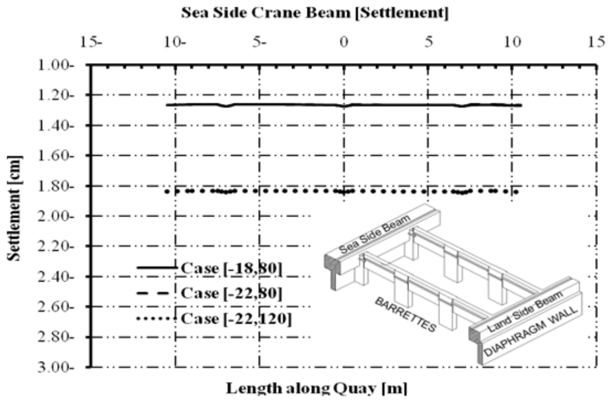
5.1.2 Crane Beams

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 13 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 that the average vertical settlement of sea side crane beam in case (-18, 80) is -1.27 cm 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 vertical settlements.

Figure 14 shows the bending moments 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 moment 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 moment 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 4009 kN.m increased by 37.5% in case (-22, 120). The land side crane beam max. value of bending moment for case (-18, 80) is 4985 kN.m increased by 32.4% in case (-22, 120).

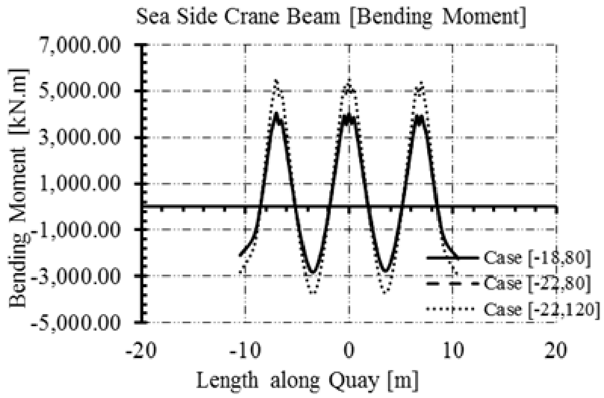


(a)

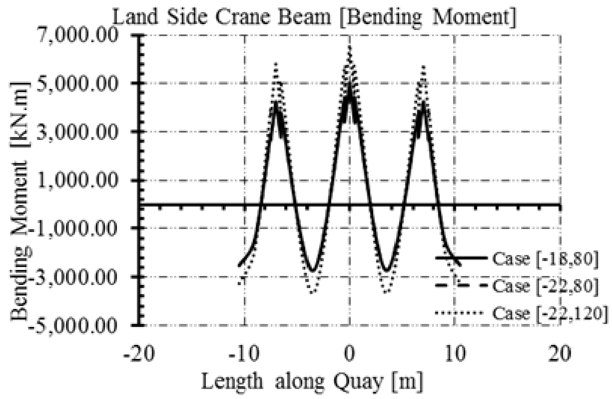


(b)

Figure 13: Vertical settlement of crane beams (a) Sea side crane beam, (b) Land side crane beam.



(a)



(b)

Figure 14: Bending moment of crane beams (a) Sea side crane beam, (b) Land side crane beam.

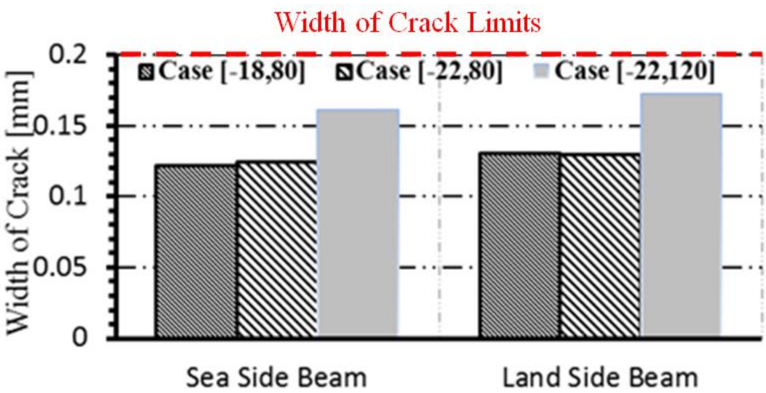


Figure 15: Width of crack for sea side and land side crane beams under the three studied cases.

The width of crack verfication was also carried out for crane beams. Figure 15 shows the width of crack for crane beams under the three studied cases. From the figure, it is clear that the beams are under the limitation of the crack width.

5.2 Soil

The results of the vertical total stress for soil at points A, B, C and D shown in Figure 5 are given in Table 4. The results show that, for points A and D the vertical stress has unremarkable changes and for point A the value of total stress decreased from 1969 to 1948 kN/m^2 due to the removal of soil by deepening. While, the values of total stress at point D have no changes because it is far away from the effect of deepening and increase of crane load. For point B, the increase in the vertical stress may be due to the tilting of the structure towards sea side. For point C, the stress first decrease due to the tilting towards sea side and then increase due to the crane load increase.

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*
Existing Case(-18, 80)	-1969	-1862	-1344	-2167
Case(-22, 80)	-1948	-1920	-1331	-2167
Case(-22, 120)	-1948	-1995	-1414	-2167

For determining the factor of safety for the soil, the shear strength reduction method was used; soil shear strength is gradually decreased by program as long as the first indications of failure appear. Safety factor is defined as the ratio of real shear strength of soil to reduced shear strength. The shear strength reduction method is better than the other methods investigating slopes stability (Farshidfar and Nayeri, 2015). One of the advantages is that there is no need to the primary guess at determination of critical failure surface. Due to the high-speed computer systems, this method is used increasingly today than before. Figure 16 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%. This finding indicates that the overall stability of the quay wall is more sensitive to the frontal deepening than the increase in crane loads .

5.3 Quay Wall Operation

Regardless 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 operation will efficiently continue or not. Those factors have limits to make sure that the quay wall operation will not be affected (Susumu *et al.*, 2000). 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.

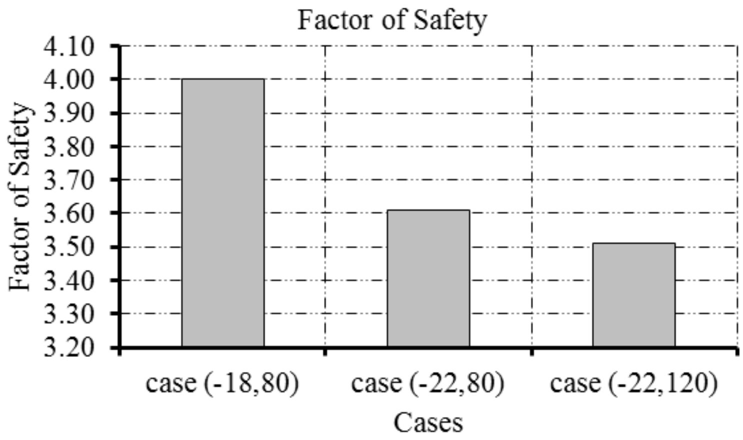


Figure 16: φ – c reduction factor of safety for the three cases.

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

Case	*S. Side beam average sett. (cm)*	*L. Side beam average sett. (cm)*	*Diff. (cm)*	*Allowable diff.*
Existing 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 anglesunder the studied three cases.

Cases	*Bar.1* ϑ_{actual}	*Bar.2* ϑ_{actual}	*Bar.3* ϑ_{actual}	*Bar.4* ϑ_{actual}	$\Theta_{all.}$
Existing 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 under the three studied cases are acceptable for the quay wall and crane operation.

6.0 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 -22 m instead of level -18 m without changing crane wheels loads, while the second scenario is to perform the same deepening and increasing the crane wheels loads from 80 up to 120 ton/m'. The analyzed results of the study including deformations, capacities of structural elements, settlements, soil stresses 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 width of crack limitations preclude the possibility of performing the second scenario when using no engineering solutions.

7.0 Recommendations

It is recommended to extend this work including the followings:

a) A dynamic analysis study for the existing diaphragm quay wall which is very important for determining whether the quay wall could be developed or not when considering the dynamic loads.

b) Searching for optimal engineering solutions for solving the problems which may be resulted due to deepening and increasing quay crane loads for both static and dynamic conditions.

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