Comparative Analyses of Quay Wall Case Study UsingPlaxis 3D

1Prof.Ehab R. Tolba, 2Ass. Prof.Sherif Abd Ellah, 3Ass. Prof. Elsayed M. Galal,
4Ass. Prof. Ezzat Ahmed Sallam, 5Eng. Mohammad Ahmad Kamal

1, 2, 3&4Faculty of Engineering, Port Said University, Egypt,
5 Marine QC Manager. TOA CORPORATION, Kuwait

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Abstract – This paper presents the displacement behavior of the quay wall system, using Plaxis 3D version 2013. The horizontal displacement calculated by Plaxis 3D for the existing quay walls case study located in Rotterdam Port in South Holland is evaluated by the field measurements. At first, the three-dimensional numerical performance of Rotterdam's existing quay wall is compared with the site horizontal displacement measurements over five (5) years which provide an interpretation of the increment causes of the measured horizontal displacement on top of the quay wall. Extensive comparisons are made to include the interaction between the soil and existing quay wall. Both Hardening Soil Model (HSM) and the Mohr-Coulomb Model (MCM) are utilized. The results highlighted the paramount importance of considering the time-dependent effects on the deformation behavior of the quay wall in addition to the undrained and drained soil properties. The long-term horizontal quay wall movement increments due to consolidation play an essential role in most quay wall deformation problems. Finally, a comparative study is performed between displacement results obtained from Plaxis 3D and 2D to validate the behavior of the quay wall case study model. This result proves that Plaxis 3D 2013 prediction is satisfactory for reaching reliable displacement values when compared to the field measured and is a powerful tool for simulating and performing such quay wall analysis if compared to Plaxis 2D results.

Keywords: Quay wall, Sheet Pile Wall, Plaxis 3D 2013, Hardening Soil Model, Mohr-Coulomb soil model, time-dependent soil behavior.

1. Introduction

Quay walls are retaining structures constructed to retain earth and any other fill material in ports construction for ships berthing and mooring. Analysis of different types of quay walls systems has been studied by many authors using the finite element technique to investigate the behavior and failure mechanisms of quay walls structures Don and Warrington, 2007; Krabbenhoft et al., (2005); Krabbenhoft and Damkilde, (2003); Lyamin and Sloan, (2002) and Briaud and Lim, (1999). More efforts have been extended with full-scaled field tests to understand the behavior of quay walls structures, Briaud et al., (2000). Barley, (1997) provided considerable results to represent failure mechanisms of sheet-piling quay walls based on field observations. The development of three-dimensional (3D) finite element analysis has greatly enhanced our ability to model complex berthing structures. According to the continues development in software and computer technology, the choice of the numerical solution becomes preferable because of the high accuracy of predicted results with low cost and time if compared to physical simulations.This environment provides assessment studies by numerical modeling simulations which helps the marine engineers to analyze different problems and prevent potential hazards. The most useful advantage of the numerical solution is the ability to change the input parameters easily and observe the consequences to find the optimized solution. Hamza, and Hamed, (2000) carried out a three-dimensional analysis for the east Port Said quay wall by FLAC 30 software to evaluate the resulting displacement and straining actions under the different load combinations. Jonkman et al., (2013) carried out analytical analysis of one existing quay wall at Rotterdam by Begemann-De Leeuw method and has been compared with finite element Plaxis method. It was concluded that the results from Plaxis method are more accurate than Begemann-De Leeuw method when compared to the measured results. PLAXIS 3D is a finite element code for soil and rock plasticity and its version 2013 is employed in this study for the analysis of practical existing quay wall problems.

The main objective of this study is to verify the numerical analysis effectiveness of PLAXIS 3D 2013 through comparing the calculated displacements with the field measured values as well as Plaxis 2D V8.6 results. For Rotterdam case study, the general description of the quay wall is given, followed by the properties of the soil and structure elements. Then, the 3D finite elements model is described. Finally, the results are summarized and discussed, in separate tables and graphs.

2. Anchored Quay Wall at Rotterdam

Rotterdam is Europe's largest port and had been previously the largest port of the world for a long time. That port is located at Rotterdam city in the Netherlands country. Rotterdam is the second-largest Dutch city after the capital Amsterdam, which located in the province of South Holland, at the mouth of the Nieuwe Maas channel leading into the Rhine–Meuse–Scheldt delta at the North Sea. Rotterdam history goes back to 1270, when a dam was constructed, after which people settled around it for safety.

A quay wall has been reconstructed at Rotterdam Port in 2003 by a logistics company called Gevelco located at the Brittanie harbor which had intentions to expand its capacity and to receive large vessels. Figure 1 shows the map location of the case of Rotterdam Port. Due to port improvement, an existing jetty has been replaced with a new 319 [m] quay wall length named Deep-Sea Quay Wall. The layout of the Rotterdam port is shown in Figure 2 including deep sea quay wall. Seven (07) points have been monitored along the deep-sea quay wall with regular field measurements on top of the quay wall since the commencement of the operational stage in 2004 till 2010 as presented in Figure 2. These Measurements indicate that horizontal displacements of the quay wall increasing in seaward direction nonlinearly over time. Figure 3 indicates the development of horizontal displacements over time for deep-sea quay wall as stated by Vrijling et al., (2011). The behavior of Rotterdam deep-sea quay wall has been presented by many authors such as De Klerk BV et al. (2003) and Anrooij van et al. (2003).

Figure1. Rotterdam Port location on map

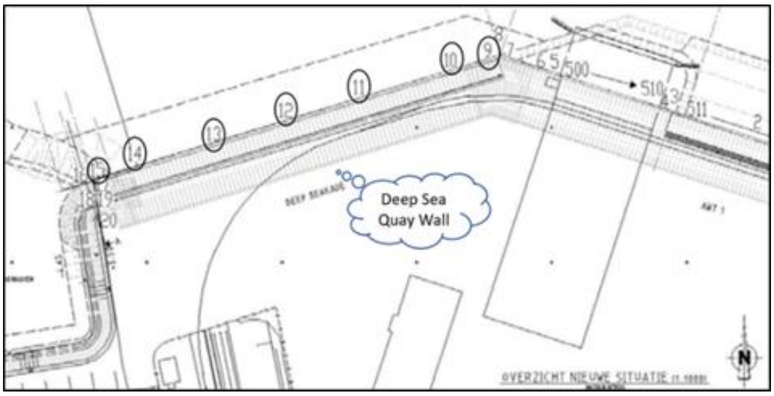


Figure2.Overview of considered measurement points

Rotterdam deep-sea quay wall consists of combined tubular sheet piles 1420 [mm] diameter with triplets sheet piles. The tubular piles for this quay wall system extends from top elevation of +5.50 [m] down to -31.50 [m] referenced to the Normal Amsterdam Level [NAP]. The inner triplet consists of three sheet piles PU20 which are driven to NAP -20.00 [m]. The quay wall concrete deck is constructed on existing hollow section concrete piles with an outer diameter of 600 [mm] and a thickness of 125 [mm] which extended toNAP -25.0 [m]. The center to center piles spacing in both directions are 4.9 [m]. Some new concrete piles with dimensions 450 [mm] x 450 [mm] have been constructed down toNAP – 26.0 [m]. The seabed level on front of the Deep-Sea quay wall is situated atNAP 12.65 [m].

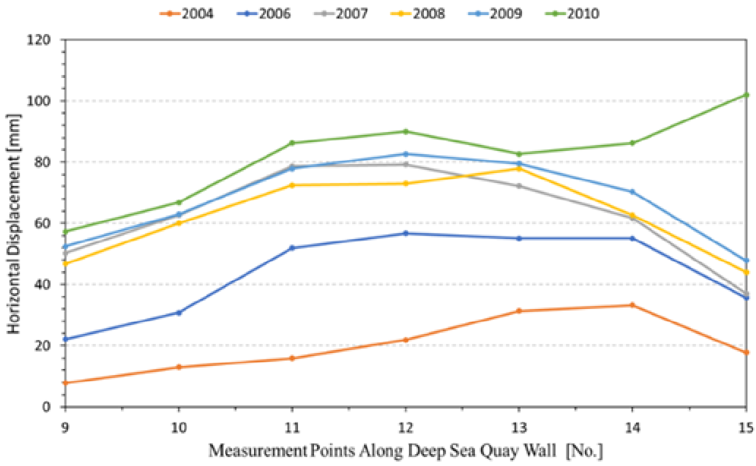


Figure3.Development of horizontal displacements over time (Vrijling et al., 2010)

The anchor wall profile has standard steel sheet piles AZ36 located at 38 [m] from the quay wall between NAP +4.00 [m] and -4.00 [m], with a concrete waling 1.7 [m] x 1.7 [m] for the anchorage. The AZ36 sheets are placed under an angle of five (5) degrees. The tie rod exists of 24 strands FeP 1860 tendon with a strand diameter of 15.7 [mm] and attached to the tubular piles at NAP + 1.75 [m] and runs to + 0.00 [m] at the anchor wall. The anchors center to center distance is 3.28 [m] and has 1320 [kN] pre-tensioned with an angle of two degree (2°). Six rows of straight piles remained after demolishing the existing jetty, with a center to center distance of 4.9 [m] in plane. Figure 4 shows the Deep-Sea Quay Wall as described by J. K. Vrijling et al. (2010).

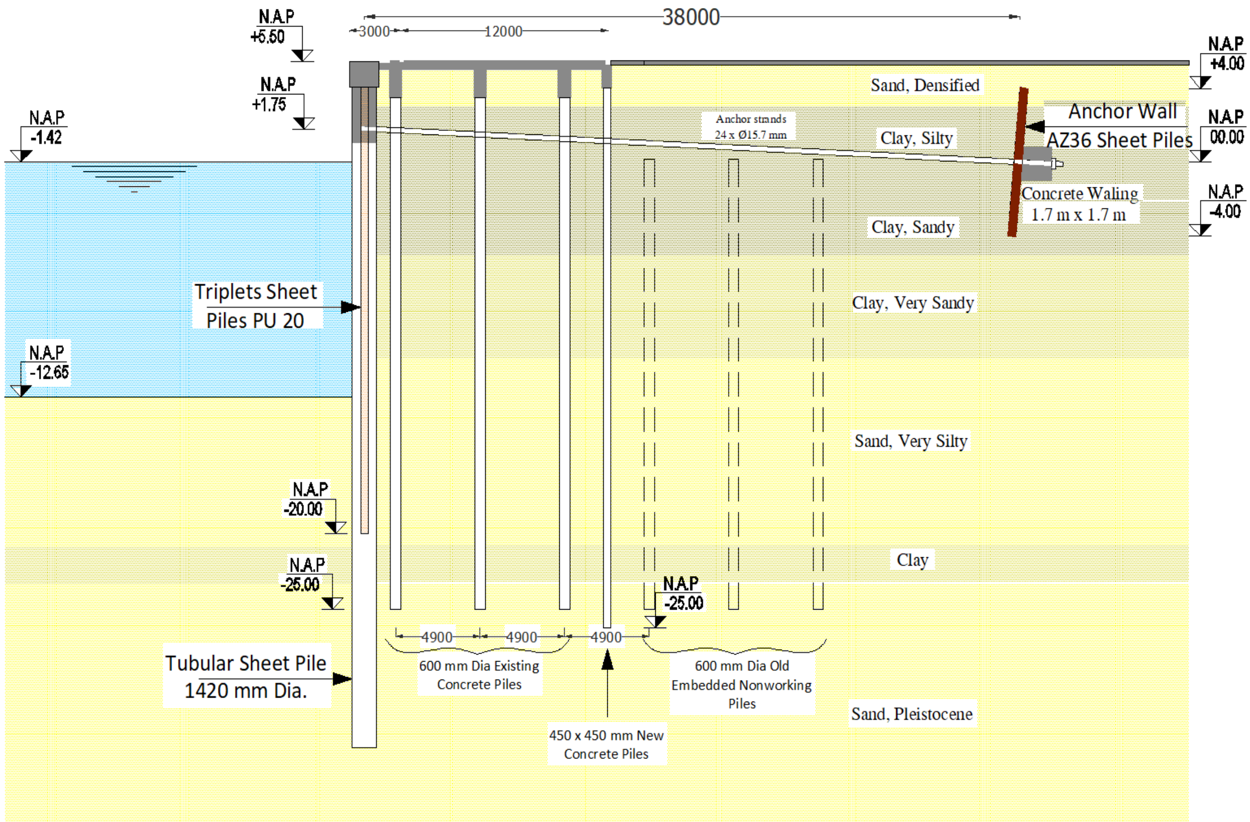


Figure 4. Section elevation for deep sea quay wall system

2.1. Analysis with the 3D finite element method

This clause presents the finite element model for the Rotterdam quay structure by finite element software, PLAXIS 3D 2013, which has been widely accepted as a powerful tool to solve boundary value problems. It can provide an interpretation of the model with the reality based on the three-dimensional assessment by numerical modelling of the soil behavior and elements properties. The quay structure elements, soil properties and external forces are defined in the software input. The basic parameters of the finite element model are established including the description of the model, the basic units and the size of the model area. Model units (Length= m; Force= kN; Time= day) are assigned.

2.1.1. Definition of soil stratigraphy

PLAXIS simulates behavior of materials with different constitutive models such as but not limited to the Linear Elastic model for concrete elements, the Mohr-Coulomb model for soil or rock, the Hardening Soil model for modeling the behavior of compressible soil, and the Soft-Soil-Creep model to model the viscous and time dependent behavior of the soil creep. Only the Mohr-Coulomb model and the Hardening Soil model are discussed in this section, as they are used in this study. The PLAXIS isotropic linear elastic model follows Hooke's Law (Malvern, 1969). Mohr-Coulomb failure criterion is concerned with stress conditions on potential rupture planes within the soil. The soil mass failure will occur if the resolved shear stress "τ" on any plane in the soil mass reaches a critical value. This can be written in equation [$\tau = \sigma' \tan(\varphi') + c'$] where, c': effective cohesion; σ': effective normal stress; and φ': effective friction angle.

As the development of plastic strains occurs, the yield surface does not admit changes of expansion and hence considered as fixed yield surface. The basic idea of the MC model is shown in Figure 5. The linear elastic perfectly-plastic Mohr-Coulomb model involves five input parameters, i.e. Young's modulus E and Poisson's ratio ν for soil elasticity; φ and c for soil plasticity and Ψ as an angle of dilatancy. This Mohr-Coulomb model represents a 'first-order' approximation of soil behavior. For each layer one estimates a constant average stiffness or a stiffness that increases linearly with depth. Due to this constant stiffness, computations tend to be relatively fast and obtain a first estimate of deformations.The Hardening Soil model HSM is an isotropic hardening model that PLAXIS uses to model nonlinear behavior of loose sands to dense sands and over-consolidated clays. The fundamental difference with the Mohr-Coulomb model is the stress-dependency of the stiffness of the soil and the hyperbolic relationship between the stress and the strain, as shown in Figure 6.

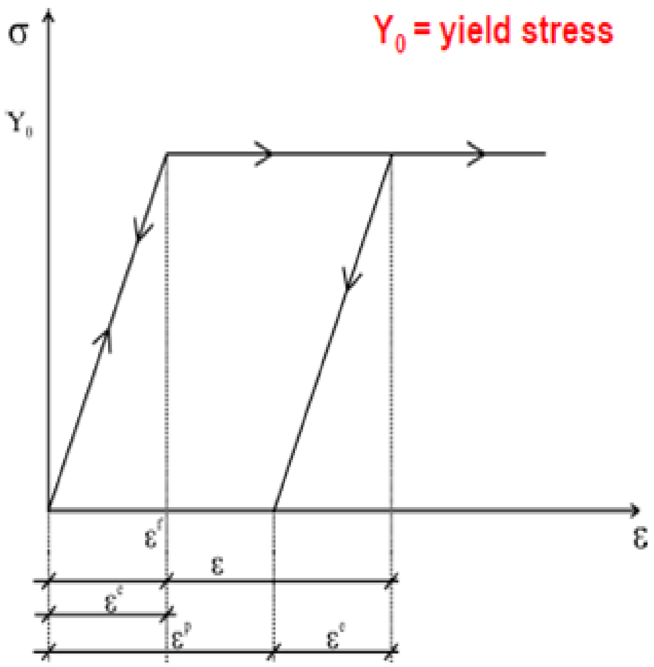


Figure 5. Mohr-Coulomb model stress-strain relation

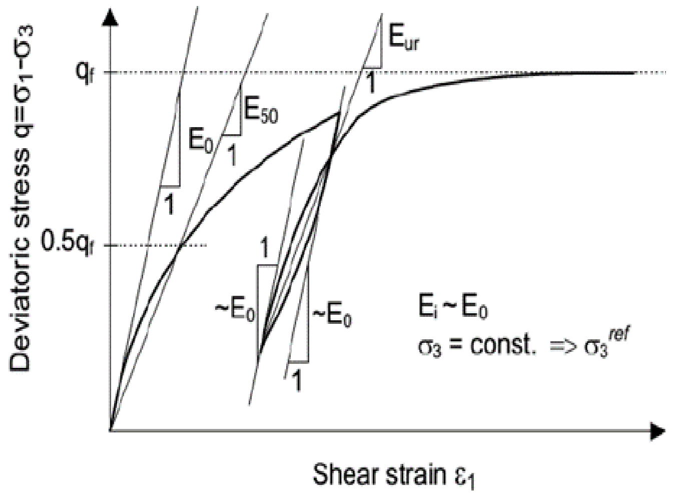


Figure 6. Hardening soil model stress-strain relation (after Obrzud, 2010)

Table 1. Stiffness and general properties for Rotterdam Hardening Soil model

Layer	Type	Level		Unit weight [KN/m³]		Kx [m/day]	Ky [m/day]	C [kPa]	ϕ° [deg]	Ψ [deg]	E_{50}^{ref} [kN/m²]	E_{oed}^{ref} [kN/m²]	E_{ur}^{ref} [kN/m²]	m [-]	R_{inter} [-]
		Start	Finish	γ_{unsat}	γ_{sat}										
Sand, densified	Drained	+5.50	-3.00	18	20	1	0.1	0.01	35	5	r	57000	171000	0.5	0.7
Clay, silty	Undrained	-3.00	-2.00	14	18	0.1	0.01			0	4500	3600	50841	0.8	0.6
Clay, sandy	Undrained	-0.50	-5.00	12	17	0.001	0.001	15	35	0	5400	2700	43390	0.8	0.6
Clay, very sandy	Undrained	-2.00	-10.50	12	17	0.01	0.01	15	35	0	5100	4080	41338	0.8	0.6
Sand, very silty	Drained	-5.00	-20.50	18	20	0.1	0.01	0.01	32.5	3	6800	5440	81600	0.5	0.7
Clay	Undrained	-10.50	-22.50	11	17	0.01	0.001	10	34	0	6200	3100	27047	1.0	0.5
Sand, dense	Drained	-20.50	-60.00	18	20	1	0.1	0.01	35	5	50000	50000	150000	0.5	0.7

HSM simulates the soil behavior by defining three different moduli, the secant modulus for a mobilization of a 50% of maximum shear strength (E_{50}), the un-loading reloading modulus (E_{ur}) and the oedometer modulus (E_{oed}). The model takes soil dilation into consideration. The nonlinear elasto-plastic hardening constitutive soil model for the soil layers in the drained and undrained condition is applied with the same properties of the existing system as stated by J. K. Vrijling et al. 2010. The parameters for the Hardening Soil model are given in Table 1. For all soil layers, the default value of (0.3) is adopted for Poisson's ratio (ν_{ur}) in the unloading reloading condition as stated by J. K. Vrijling et al. 2010.

2.1.2. Definition of structural elements

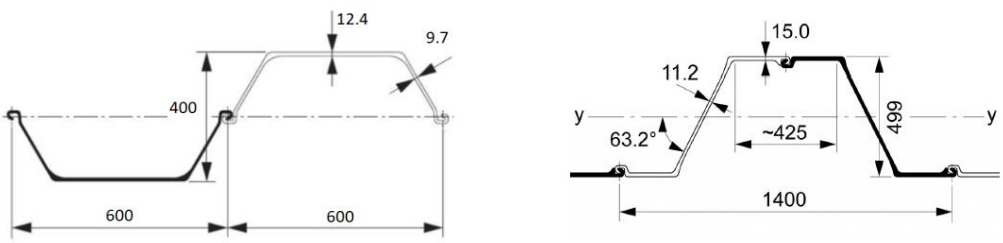
For this case study, the piles volumes are modelled as linear elastic material with plate's elements properties where dimensions are simplified to consider the capacity of the computer and running time used to solve the model. The moment of inertia for a hollow circle is set equivalent to the moment of inertia for a rectangular plate. The material properties for the different piles sets of the anchor wall, combined wall, and foundation piles are shown in Table 2. The two (2) different sheet pile walls sections are simulated in Plaxis as plate elements. The basic properties of each sheet pile wall are extracted from the material data sheets are presented in Table 3 and Figure 7.

Table 2. Stiffness and general properties for Hardening Soil model

Pile section	Tubular steel piles	Existing hollow circle concrete pile	New concrete piles
Outer Diameter (D) [mm] Inner Diameter (d) [mm]	1420 1380	600 350	- -
Thickness (t) [mm]	20	125	450
Pile depth (L)[m]	37	30.5	31.5
Moment of inertia (I)= $\pi\ (D^4 - d^4)/64$[m^4]	0.02156	0.005625	3.417E-3
Elastic modulus (E)[kN/m^2]	210E6	30E6	30E6
Equivalent plate section	Plaxis 3D plate properties	Plaxis 3D plate properties	Plaxis 3D Plate properties
Length (a) [mm]	1420	600	450
Width (b) [mm]	716	483	450
Pile depth (L)[m]	37	30.5	31.5
Moment of inertia (I)= bt^3/12[m^4]	0.02156	0.005625	3.417E-3
Elastic modulus(E) [kN/m^2]	210E6	30E6	30E6
Poisson'sRatio (υ_{12}) [-]	0.3	0.2	0.2
Density (γ)[kN/m^3]	78.5	25	25

Table 3. Basic properties of sheet pile walls

Sheet Pile Section	PU20	AZ36
Length [m]	319	319
Depth [m]	14	14
Width [mm]	600	1400
γ_{steel} [kN/m^3]	78	78
E_{steel} [kPa]	2.00 E8	2.00 E8
Sectional Area per meter length of the wall [m^2]	1.80E-02	2.16E-02
Moment of Inertia per meter length of the wall [m^4]	4.60E-04	8.96E-04
Section Height d [mm]	400	499



(a) PU20 (b) AZ36

Figure 7. Sheet pile section profile for quay wall system, (a) front combi wall and (b) anchor

Reference to PLAXIS 3D 2013 software manual, the model is developed by assigning the following parameters:

E_1	Young's modulus in first axial direction
E_2	Young's modulus in second axial direction
I_1	Moment of inertia against bending over the first axis
I_2	Moment of inertia against bending over the second axis
I_{12}	Moment of inertia against torsion
G_1	In-plane shear modulus
G_{13}	Out-of-plane shear modulus related to shear deformation over first direction
G_{23}	Out-of-plane shear modulus related to shear deformation over second direction
υ_{12}	Poisson's ratio ($\upsilon_{12}<\sqrt{E_1/E_2}$)
A_{23}	Effective material cross section area for shear forces Q_{23}.

In order to use the available plate elements fair geometric orthotropy, the basic material parameters are recalculated to the above similar parameters and interpreted based on equations in the software material models manual 2013. These equations are presented as follow where the Poisson's ratio's for sheet pile walls is assumed zero as per software manual recommendation.The properties of different sheet pile walls are presented in Table 4 where the tie anchor strands are inserted in the 3D model as node to node anchors, for which the normal stiffness (EA) is determined for the (24 x Ø15.7 mm) diameter pile as 7.2E+05 [kN].

$$E_1 = 12\ E_{steel}\ I_1/d_3 \quad (1)$$

$$E_2 = 12\ E_{steel}\ I_2\ /\ d_3 \approx E_1/20 \quad (2)$$

$$G_{12} = \frac{6E_{steel}I_{12}}{(1+\upsilon_{steel})d^3} \approx 6\ E_{steel}I_1/10\ d^3 \quad (3)$$

$$G_{13} = \frac{E_{steel}A_{13}}{2(1+\upsilon_{steel})d} \approx E_{steel}(A/3)\ /\ 2d \approx E_{steel}\ A/6d \quad (4)$$

$$G_{23} = \frac{E_{steel}A_{23}}{2(1+\upsilon_{steel})d} \approx E_{steel}(A/10)\ /\ 2d \approx E_{steel}\ A/20d \quad (5)$$

$$\gamma = A\ \gamma_{steel}\ /\ d \quad (6)$$

Table 4. Interpreted and assigned properties of sheet pile walls

Sheet Pile Section	PU32	GU16-400
Equivalent Height d [m]	0.40	0.499
γ [kN/m^3]	3.5325	3.396
E_1 [kN/m^2]	1.81E+07	1.82E+07
E_2 [kN/m^2]	9.06E+05	9.09E+05
υ_{12}	0.0	0.0
G_{12} [kN/m^2]	9.06E+05	9.09E+05
G_{13} [kN/m^2]	1.58E+06	1.51E+06
G_{23} [kN/m^2]	4.73E+05	4.54E+05

2.1.3. Applied loads on Rotterdam quay wall

In the terms of case reference, a surcharge load of 50 [kN/m2] is accounted in the 3D model in line to the design data of the quay wall at 12.50 [m] from the front combi wall. As published by J. K. Vrijling et al. 2010, massive steel cylinders have been stored to about 2-meter-high, which exceeded the design value with a total equavelent surcharge load of about 150 kN/m² which exceeds the design load by three times.This load variation is considered in the Plaxis 3D model by the identified phases as presented below.It should be pointed that the calculated horizontal displacements should be almost equal to the field initial measurements when the design load of 50kN/m² is applied near the anchor wall. After a higher load is applied at positions further away from the anchor wall, again additional horizontal displacements should be occurred.

2.1.4. Generated mesh for Rotterdam quay wall model

Before Plaxis 3D software can perform calculations, a mesh is created for both the soil and the quay wall structure system. The mesh properties are set to fine in order to obtain a mesh accurate enough for the finite element model situation. The PLAXIS 3D 2013 software meshes the model automatically. At the interface between the plate and the soil the mesh is more dense and at larger depths the mesh becomes coarse.

A fine mesh is generated with 25,978 elements and 40,234 nodes with an average elements size 2.00 [m]. As the tubular sheet piles systems are completely repeated every 3.28 meters, the numerical model has built by few panels only for this quay wall system. The model is developed with a plan dimensions of 140 [m] x 17.82 [m], and a depth of 40 [m]. Figure 8 shows a plan view of the fully meshed PLAXIS 3D model. The Y direction of the model is generated from zero at tubular piles center line to 17.82 [m] at tubular piles center line. For X direction, the left boundary of the model is located at 60 [m] distance from the front combi wall line where the right boundary is set at 65.80 [m] from the centerline of the new constructed piles. The perpendicular plan dimension was chosen 17.82 [m] to represent six (6) tubular piles and five (5) intermediate sheet piles along the combi wall. In addition to the combi wall system, the numerical model included the components of the anchor wall and piles infrastructure with the superstructure beams and slabs. To show the connection between the front combi sheet pile wall and the anchor sheet pile with the tie rods, the soil mass is partially hidden from the model and the configuration of 3-D finite element mesh perspective is shown in Figure 9.

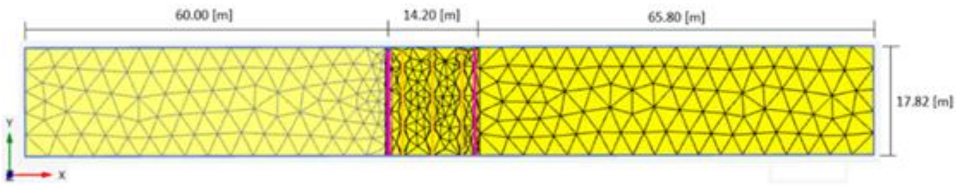


Figure 8. Geometry of the combi wall structure model - 140 [m] x 17.82 [m]

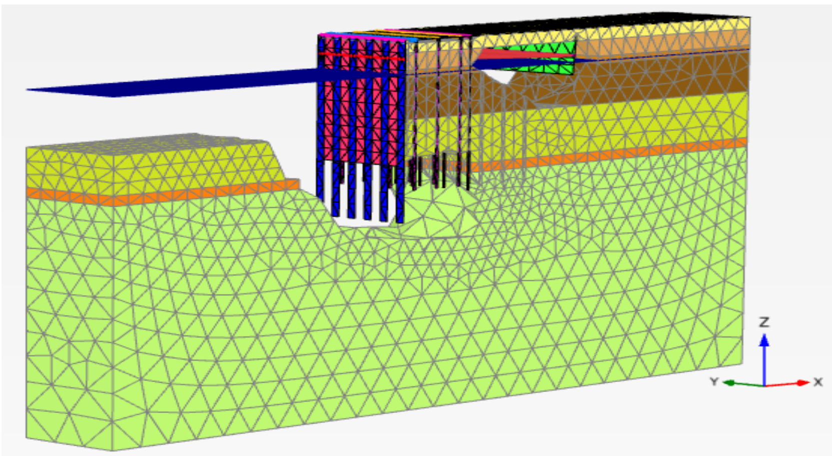


Figure 9. The generated 3-D finite element mesh of Rotterdam model

2.1.5. Modelling of the calculation stages

In order to validate the model approach in software (PLAXIS 3D 2013), the phases developed in this case are presented herein and the results are compared with the field measured values and compared with PLAXIS 2D results. The Deep-Sea quay wall model is numerically created by PLAXIS 3D 2013 with seven (7) phases in order to simulate the stage just after construction and dredging including operation stage as summarized in Table 5. The calculation type is performed by elastoplastic drained and undrained analysis however, a consolidation analysis is presented also to estimate the additional deformations contributed after operation with time intervals of 365 days. Each phase indicates objects; soil layers and forces in the stage calculation should be activated or deactivated. The model as used in this chapter consists of seven calculation phases. The first stage represents the initial phase when no excavations were performed at the Rotterdam deep-sea harbor. The ground levels on both sides of the quay structure are equal, NAP +5.50 [m]. In this phase, only the soil layers are activated where the initial effective stresses and pore pressures are calculated.

Table 5: Description of the construction phases

Construction Phase Number	Phases Description
0	Initial phase calculation based on the defined at rest pressure coefficient value (Ko) for each soil layer, where the quay wall system is not simulated yet.
1	Construction of substructure quay wall system
2	Excavation in front of the combi wall up to -0.50 [m NAP]
3	Prestressing of tie anchors by 1320 [kN]
4	Dredging up to level -12.65 [m NAP]
5	Applying of operation design loads [50 kPa Surcharge load + 80 [kN/m] Bollard loads
6	Applying of operation overdesign loads [150 kPa]
7	consolidation analysis with time intervals of 365 days after overloading

Between the initial phase and the consolidation calculation phase, the construction final stage of the quay structure is implemented in addition to the dredging and operational loading stages. Apart from the soil layers in previous phase calculation that are activated, the plates elements for the open tubular piles and intermediate sheet piles are activated till NAP -31.5 [m] and -20.0 [m]. Besides, all existing and new constructed piles are placed till NAP -25.0 [m] and -26.0 [m] respectively as well as tie anchors are activated with its anchor wall. Excavation in front of the combi wall is applied for prestressing anchoring till NAP. +1.75 [m]. In this case, the soil layers from NAP. +5.50 [m] till NAP. +1.75 [m] are deactivated. After all tie anchors have pre-tensioned with 1320 [kN] force, a dredging of the area next to the combi wall is performed till NAP. -12.65 [m]. The soil layer on the left side of the quay structure element is deactivated till construction depth. Accordingly, the construction stages of the quay structure model are fully implemented. Moreover, the operational loads are defined for this model as per the design condition. In order to simulate the actual operational applied overloads, additional vertical load is defined in the sixth phase. Finally, the time-dependent consolidation is applied to compare with the field measurements provided.

2.2. Deformation behavior of the quay model

In order to comprehend the deformation behavior of Rotterdam quay structure, specifically the combined wall, a three-dimensional numerical model is set up. This model is based on the designed quay structure and the applied loads. This model will clarify an insight into the deformations and structural forces of the quay structure which can be compared to the measurement values. When comparing the measurements with the PLAXIS 3D model, the deformation of the combi wall elements should be closely evaluated for the purpose of verification.

The model output is based on the modelling schematization described in section 2.1. Figure 10 illustrates the total displacement |u| of the soil deformed mesh as well as quay wall substructure for phase number five where maximum displacement is equal to 132.3 [mm] at the loads platform area behind the anchor wall structure. The deformed mesh of the quay structure model illustrates the displacement of the relieving platform structure and the combined wall moving towards the waterside due to the soil lateral impact and the vertical load as shown in Figure

11. The maximum horizontal displacement Ux of the soil around anchor wall reach 59 [mm] where at combi wall is 50 [mm] as shown in Figure 12 and Figure 13.

2.3. Description of the field measurement

This section presents an evaluation of the displacement results on the tubular piles of the combined as per PLAXIS 3D outputs compared to the field measurements. Reference to Vrijling et al. (2010), the monitored displacement measurements for seven (7) points located along the deep-sea quay wall over five (5) years are presented in Figure 14.Taking into consideration the maximum values only per year, the displacement of the top of quay wall varies from 33 [mm] to 102 [mm] as presented in Figure 15. These measurements values are compared to PLAXIS 3D predicted results.

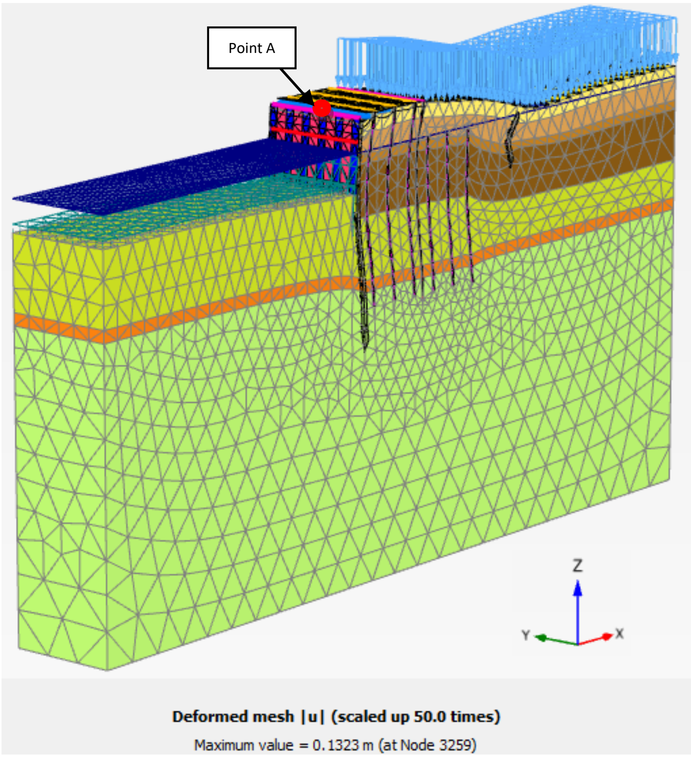


Figure 10. Deformed mesh of the Rotterdam quay wall model for Phase 5

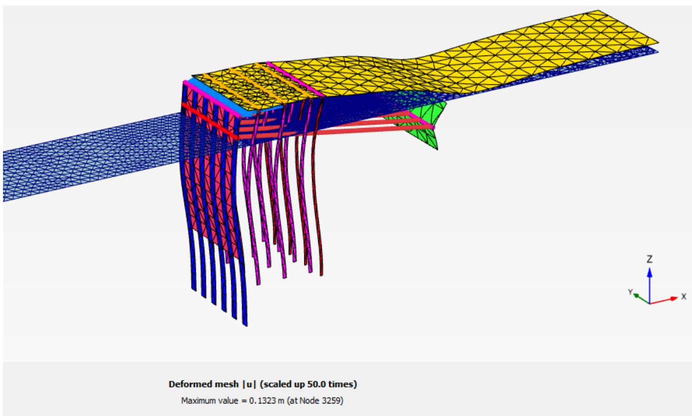


Figure 11. Substructure deformation of the Rotterdam quay wall model - Phase 5

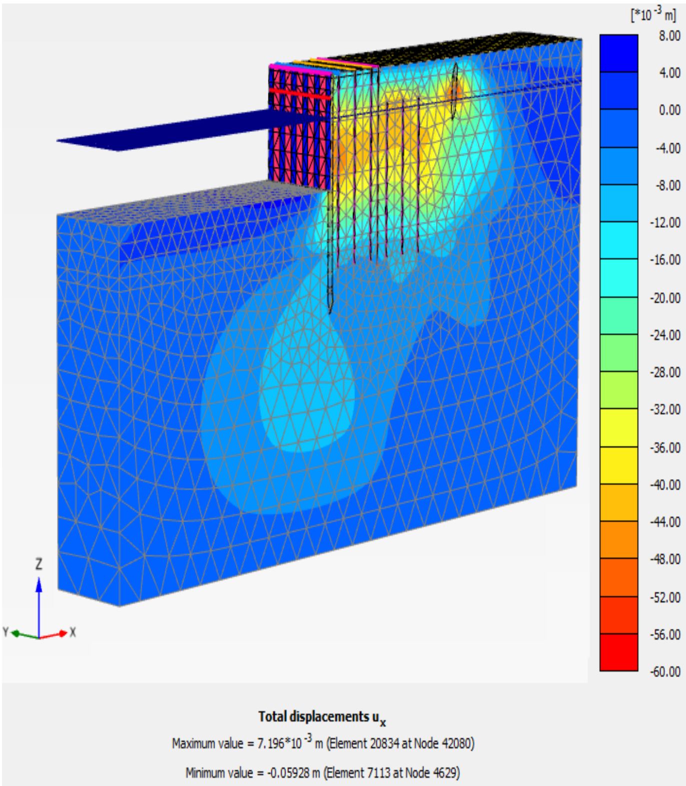


Figure 12: Soil horizontal displacement of the Rotterdam quay wall model - Phase 5

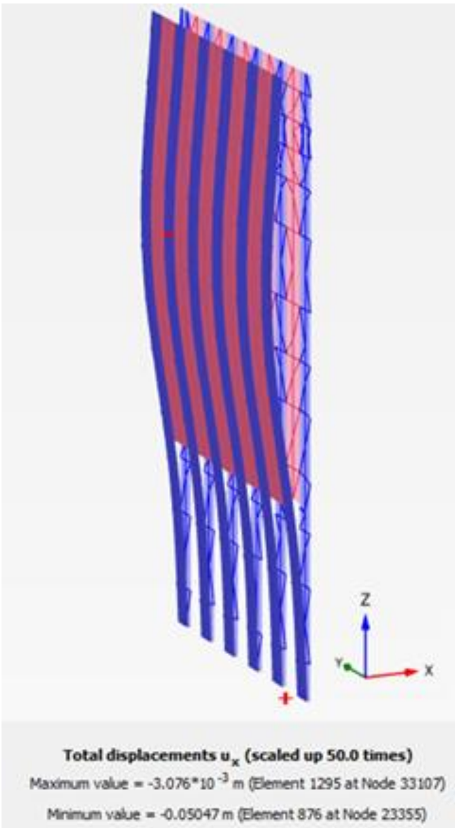


Figure.13. Combi wall horizontal displacement - Phase 5

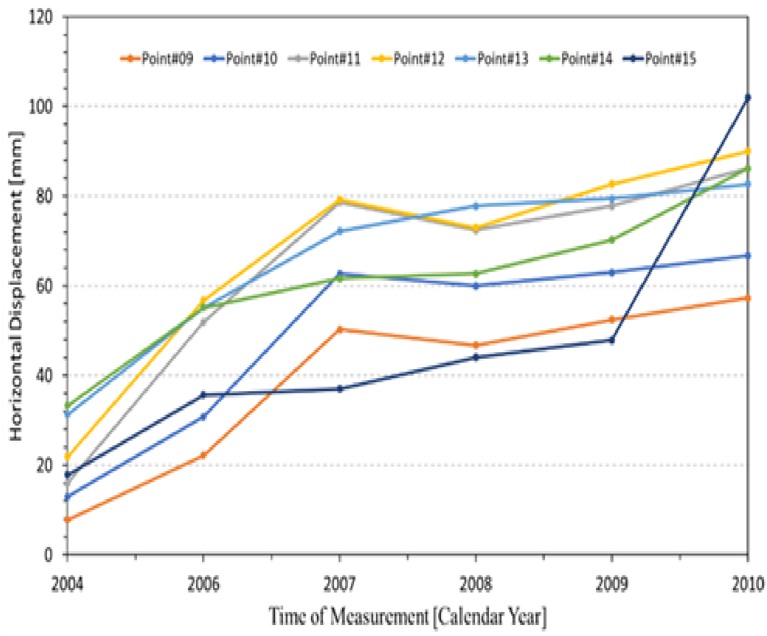


Figure 14. Nonlinear horizontal displacements of the quay wall top during five year

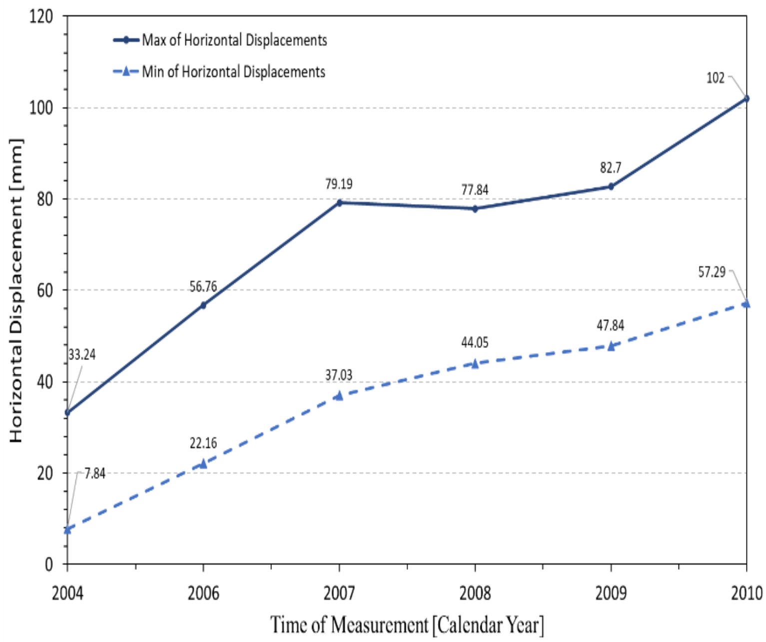


Figure 15. Maximum and minimum horizontal displacements on top of the quay wall within five years

2.4. Discussion of Plaxis results

To plot a multi-linear combi wall length-displacement relationship, results are extracted from PLAXIS 3D model for each phase to understand the behavior of the combined wall during and after construction completion. Figure 16 depicts the variations of the horizontal movements over the combi wall depth for the all seven phases based on the quay wall parameters and soil Hardening Soil Model (HS) as utilized above.

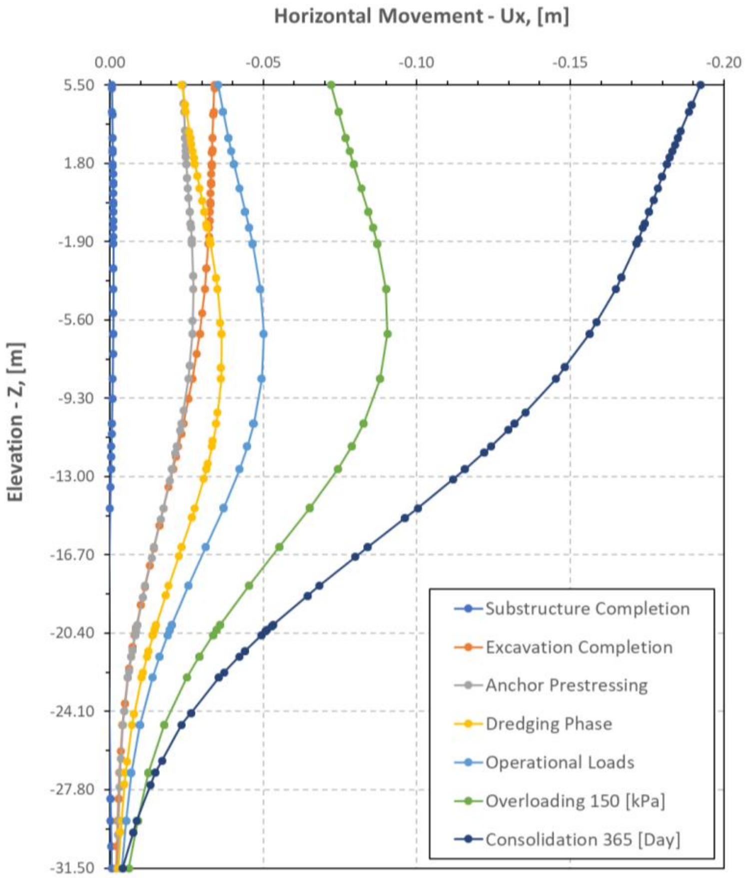


Figure 16. Predicted movements for all modeled phases over combi wall depth

As clear from Figure 17, the displacement at the top of the combi wall ranges from 0.58 [mm] to 192.38 [mm] for all seven phases analysis. It can be observed that the displacement increases 57.7 [%] after excavation in front of the combi wall while it decreases about 30 [%] due to anchors prestressing application. The result shows that the top displacement is almost not changed after the dredging where it increases more 34 [%] during the initial operational condition to reach 35.16 [mm]. When the applied load increases three times till 150 [kPa], the horizontal displacement on top of the wall incremental increase to 72.07 [mm]. Any further increase in the displacement is associated with a reduction of soil volume change in saturated cohesive soils due to exclusion of water occupied the void spaces and causes a primary consolidation. Consequently, it can be said that the predicted maximum displacement due to 365 [days] time-dependent of primary consolidation process reachs 192.38 [mm].

The horizontal-vertical displacement (H-V) relationship on top of the front quay wall is estimated by Plaxis 3D as shown in Figure 18.At the construction completion phase, the vertical displacement started from zero to 14 [mm] without horizontal displacement. During the excavation phase, the horizontal displacement reaches 34 [mm] with corresponding vertical displacement 23 [mm]. To generate a passive pressure on the dead man wall, the applied pre-tension force reduced the horizontal displacement about 40% to reach 20 [mm] without variation in the vertical displacement. After completion of dredging phase, the vertical displacement increased again to 26 [mm]. It is clear that;a linear H-V displacement relationship is occurred during the operational phase where the horizontal displacement extended to 72 [mm] with 70% corresponding vertical displacement increment to reach 45 [mm]. Finally, any further increase in the displacement is associated with a reduction in the vertical settlementduring the consilidation stage.However, the maximum horizontal displacement for the front wall is within the allowable limit (1.5% of the wall height or < 300mm) reference to Cheng-Yu Ku et al., 2017. For more understanding, the clay layer between the front and anchor wall is totally replaced with densified sand in the 3D model to compare with the orignal case. It can be observed that as much as the clay soil layer between the front wall and anchor wall is replaced, the displacement results are reduced mostly about 10%.

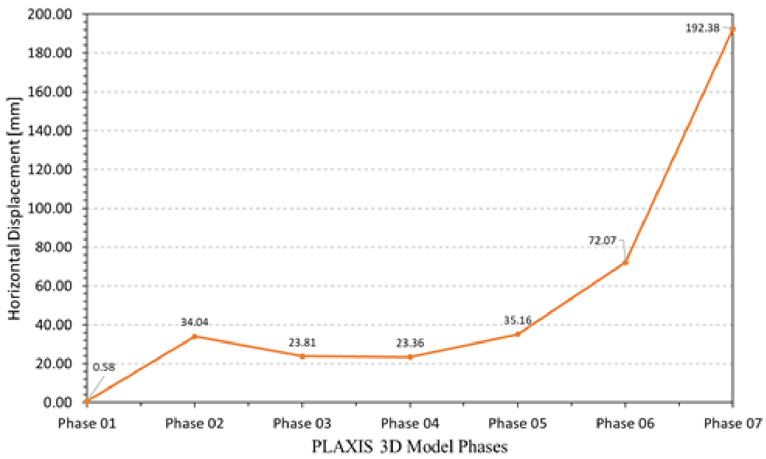


Figure 17. Horizontal calculated displacement results at the top of the combi wall for all phases

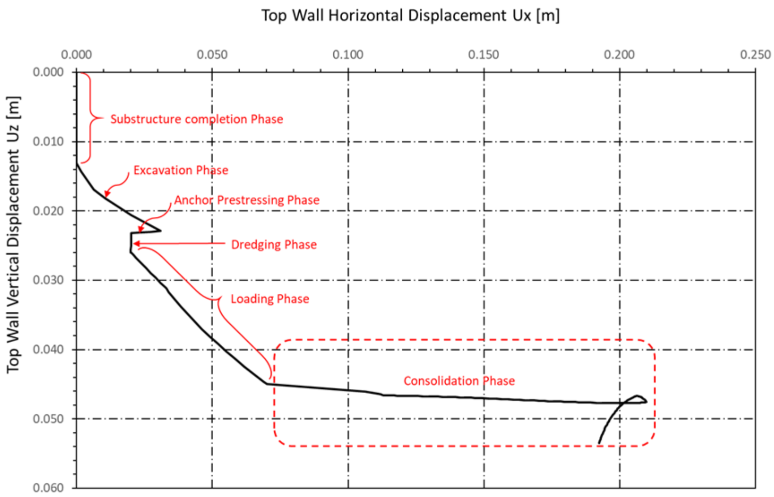


Figure 18. Horizontal vs vertical displacement relationship for front wall

2.5. Comparison between numerical predictions and field measurements

A comparison is performed between the results in Figure 15 and Figure 16 for the maximum horizontal field measurements on the top of the quay wall and the numerical results obtained from Plaxis 3D. It is clear that the measured displacement after the direct operation in 2004 was about 33.24 [mm] where its corresponding calculated value equals 35.16 [mm] as estimated by Plaxis 3D for phase number five which related to the initial operation of design load 50 [kN/m²]. The predicted horizontal displacement is about (+5.8%) difference from the field measured value for the same stage. This result proves that Plaxis 3D prediction is satisfactory for reaching reliable displacement values subject to the design load 50 [kPa]. When the applied load increases to 150 [kN/m2], the calculated displacement increases to 72.07 [mm] which is about (-8.9%) and (-7.4%) if compared with the maximum measured value in 2007 and 2008 respectively. The calculated maximum displacement result reaches 192.38 [mm] after 365 [days] consolidation which is 88% more than the measured value in 2010. Table 6 presents a summary of the comparison among the displacement predictions of Palxis 3D and field measurements as percentages.

Table 6. Comparison between predicted and measured results

Type of analysis	Horizontal displacement [mm]					
Year of measurement	2004	2006	2007	2008	2009	2010
Max measured horizontal displacement per year	33.24	56.76	79.19	77.84	82.70	102.0
PLAXIS 3D calculated phase	Phase 5	Phase 6			Phase 7	
PLAXIS 3D calculated result	35.16	72.07			192.38	
% difference	+5.8	+27	-8.9	-7.4	+133	+88

For more understanding of this case, an effort was devoted to study the comparison between the measured values of 2006 onwards and PLAXIS estimated displacements in phase 6 and phase 7. Additional two phases are performed inside the range of phase 6 and described as phase 6a and phase 6c where phase 6b will remain the same of original phase 6. These phases indicate loading and of unloading of an additional 100 [kPa] to simulate the operational overloading condition exceeding the design load of 50 [kPa].Phase 7 remains the same to simulate consolidation with time interval 365 days. The phase's list is updated in Table 7 below. The numerical results of horizontal displacements for the additional phases are plotted in Figure 16. The updated horizontal displacement results for all phases 5, 6 and 7 are presented in Figure 17.

Table 7. Description of the updated construction phases

Construction Phase Number	Phases Description
5	Applying of operation design loads 50 [kPa] Surcharge load + 80 [kN/m] Bollard loads
6a	Applying of surcharge overdesign loads [100 kPa]
6b	Applying of surcharge overdesign loads [150 kPa]
6c	Unloading of operation overdesign loads to each back [50 kPa]
7	Consolidation analysis with time intervals of 365 days after overloading

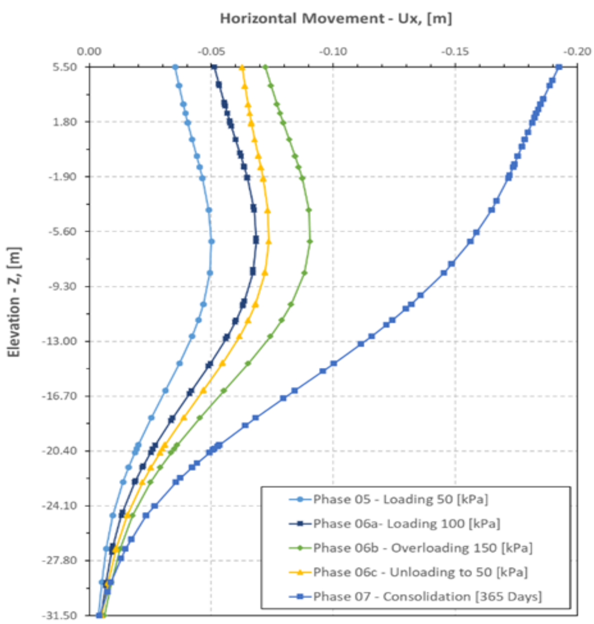


Figure 19: Updated calculated displacement results at the top of combi wall for all phases

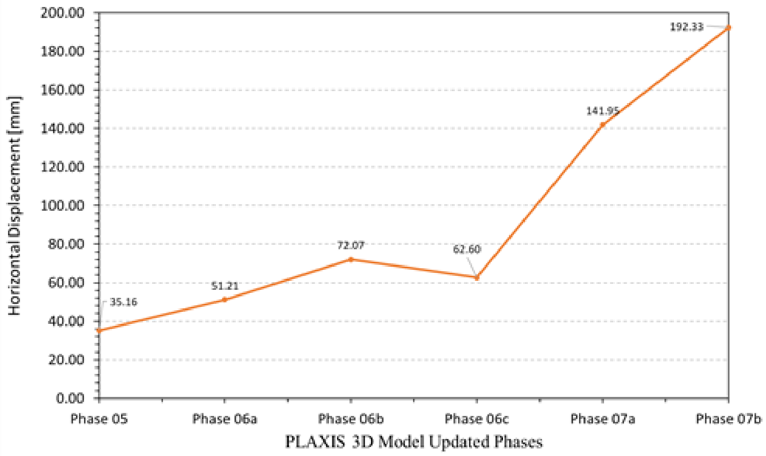


Figure 20: Horizontal displacement results at the top of the combi wall for all updated phases

Table 8, summarizes the results of updated analysis for all phases. It can be observed that, phases 6a and 6b are almost close to the maximum measured values during the years 2006 and 2007 with variation about -9.8 [%] and -8.9 [%] respectively. After the applied surcharge load decreases from 150 [kPa] to 50 [kPa], the calculated displacement value decreased from 72.07 [mm] to 62.60 [mm] which is about 13 % lower. This can interpret the reduction meaning of the field measurement from 2007 to 2008 with about 17 %. It should be pointed that the displacement result of phase 6c is much 78 % larger than the corresponding value in phase 5. Therefore, it can be concluded that when the surcharge overload reduced to its original design value, the corresponding displacement results never return to its initial condition.

Table 8: Comparison between updated phases and measured results

Type of analysis	Horizontal displacement [mm]					
Year of measurement	2004	2006	2007	2008	2009	2010
Max measured horizontal displacement per year	33.24	56.76	79.19	77.84	82.70	102.0
PLAXIS 3D calculated phase	Phase 5	Phase 6a	Phase 6b	Phase 6c	Phase 7	
PLAXIS 3D calculated result	35.16	51.21	72.07	62.60	192.38	
% difference	+5.8	-9.8	-8.9	-19.6	+132.5	+88.6

Table 8 results indicate that displacement developed over time reaches about 132.5 % and 88.6 % if compared with 2009 and 2010 measurements values respectively which are much higher than the corresponding measured values.In fact, the displacement due to consolidation process was not closely monitored at site. Consequently, it can be said that maximum displacement measured value was achieved during the consolidation process period. However, this calculation phase is assigned as consolidation analysis in the loading type staged construction. It is assumed that overloading is remained about one year, so the time interval should be set on 365 days accordingly. In order to acquire a good idea of the long-term consolidation displacement mechanism, a node (A) is assigned on the top of the tubular pile as shown in Figure 7 to perform time-displacement curve and afterward, Plaxis 3D calculation is restarted. Figure 18 shows the time-displacement relationship on top of the combi wall. It can be observed that the maximum horizontal displacement reaches 210 [mm] after twenty (20) days consolidation period where it declines to 192.3 [mm] at the full consolidation analyzed period (365 days). For more clarification, Figure19 highlights the horizontal displacement behavior during the first thirty (30) days consolidation period. Thus, a horizontal displacement 102 [mm] is estimated for time interval of three (3) days which strongly matches the displacement field measurement on the top of the combined quay wall system.

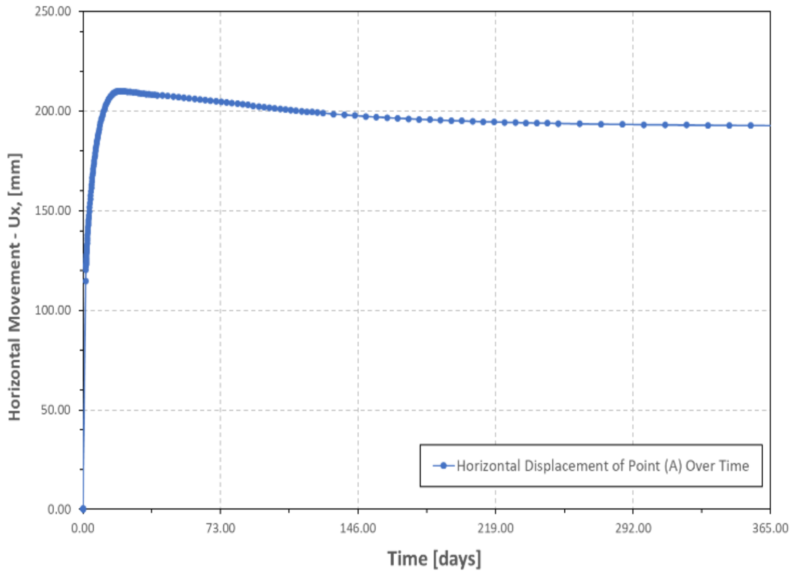


Figure 21: Top displacement prediction over 365 days with HS material model

On the other hand, it deemed necessary to mention that one of the main reasons for the seaward movement of the combi wall is the dea dman wall horizontal displacement. This displacement is also related to the compression of the soil between the combi wall and anchor wall borders. The maximum displacement of anchor wall reaches about 200 [mm] which about +4% more that corresponding displacement of the combi wall as presented in Table 9. Figure 20 indicates the anchor wall horizontal displacement for the calculated phases.A plotted horizontal displacement relation between the fronts and deadman wall at the same anchor point is presented in Figure 21. It is clear that, any horizontal displacement increase at the front wall causes about 50% increase at the anchor wall during the construction phase. Due to tie anchor prestressing, the anchor wall has moved 27mm towards the front wall to generate a passive pressure which is about 1/650 of the wall height and equals 1/300 of the anchor wall height. Again, a linear movement relation is occurred during dredging, loading and consolidation phases with displacement incremnet about 15 % for anchor wall if compared to the front combi wall displacement.

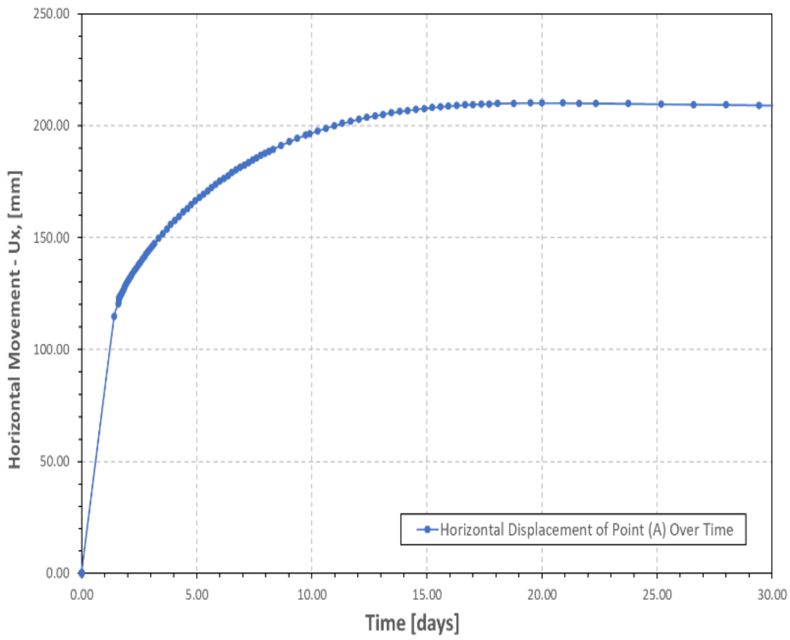


Figure 22. One-month time interval Top displacement relationship

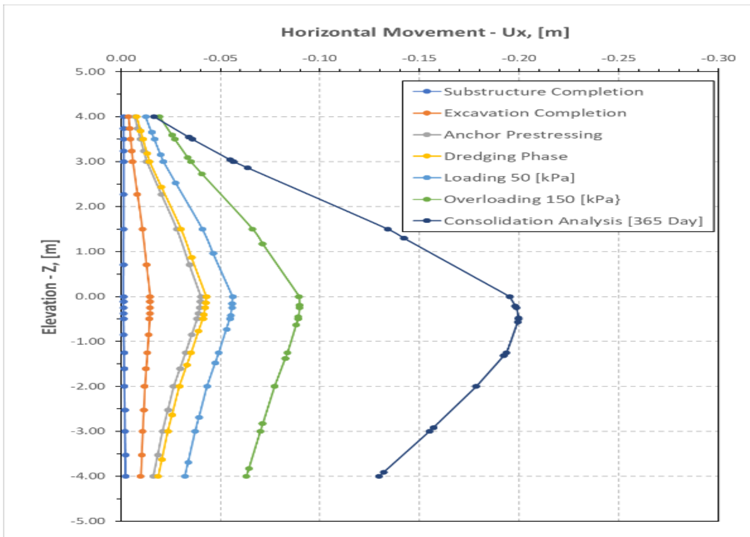


Figure 23. Predicted movements for all modeled phases over anchor wall depth

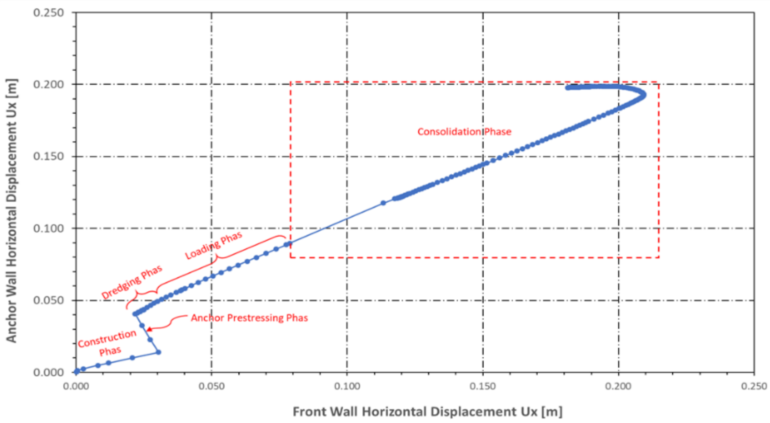


Figure 24. Horizontal displacement relation between combi wall and anchor wall

Table 9 presents a comparison among the maximum horizontal displacement results of combined front and anchor wall; for the all phases examined in this model. In both combi and anchor walls, the average displacement is about 61 to 64 [mm] respectively. However, a wide variance is observed in phase 2 where the pre-tensioning for tie anchors is not applied yet.Finally, it can be concluded that present analysis by Plaxis 3D version 2013 could satisfactorily predict the structure displacement where it is very powerful tool for modeling and performing such analysis in relatively short time. Accordingly, the estimated horizontal displacement found closes to the field measurements within (+) 5.8 %.

Table 9: Maximum horizontal displacement Comparison between combi and anchor

Construction Phase #	Max Displacement Result [mm]		%-difference
	Combi Wall	Anchor Wall	
1	1.14	2.47	+53.85%
2	34.04	14.71	-131.41%
3	27.19	40.23	+32.41%
4	36.27	43.01	+15.67%
5	50.02	56.23	+11.04%
6	90.39	89.76	-0.70%
7	192.38	199.73	+3.68%

2.6. Comparison of Mohr-Coulomb and Hardening Soil models

The linear elastic plastic model with Mohr-Coulomb (MC) constitutive soil model for the soil layers in the drained and undrained condition is applied with same model mesh. To investigate the influence of a stress dependent stiffness, the geometry is recalculated using (MC) model with $E = E_{50}$. Figure 22 depicts the variations of the horizontal movements over the combi wall depth for the all seven phases based on the quay wall structural elements parameters and soil MC Model.

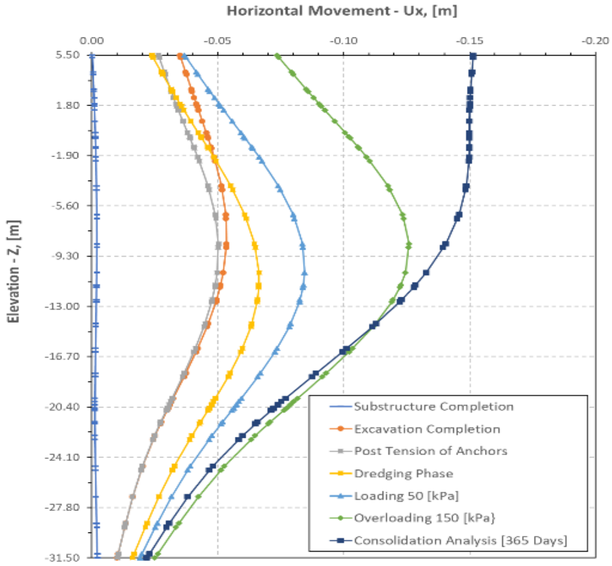


Figure 25. Calculated movements for all modeled phases over combi wall depth – MC model

Figure 23 presents combinedresults of the displacement at the top of the combi wall by both HS and MC models. It is observed that the top displacement results for phases 1 to 6 in both models are almost matching each other with an average variation of about 0.3 % as shown in Figure 22. It should be highlighted that, predictions of phase 5 horizontal displacement by the HS model is about 6 % higher than field measured result after operation in 2004, while those of MC model, is about 11 % higher. Consequently, it can be said that predictions of displacement using MC model, is mostly more conservative, if compared to HS model results. On the other hand, MC displacement result of phase 7 is about 27 %; lower than that of HS model result for consolidation analysis.

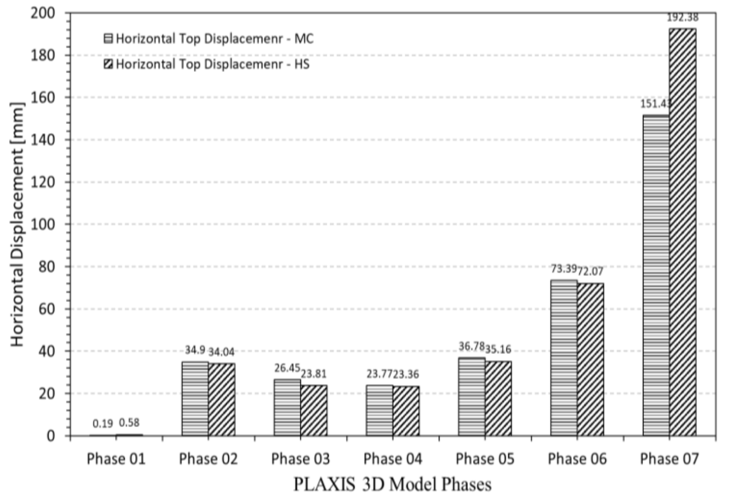


Figure 26.Top displacement results comparison between MC and HS models

Furthermore, Table 10 presents a comparison among the maximum displacement predictions of MC and HS models. It is clear that; the maximum horizontal displacement of the MC in those phases (1 to 6) is more than 70% in average as large as the prediction of HS model. However, phase 7 result for MC model is lower 21% than the corresponding result of HS model.

Table 10: Predictions of MC versus HS model

Construction Phase #	Max Displacement Result [mm]		%-difference
	MC model	HS model	
1	2.20	1.14	92.98 %
2	53.48	34.04	57.12 %
3	50.31	27.19	85.03 %
4	66.36	36.27	82.96 %
5	84.46	50.02	68.85 %
6	125.87	90.39	39.25 %
7	151.43	192.38	-21.29 %

2.7. Influence of the model width

So far, the HS model is redeveloped with a plan dimensions of 140 [m] x 9.84 [m], and a depth of 40 [m]. Figure 23 shows a plan view of the updated meshed for Plaxis 3D model (about 50% lower than original model's width) to compare its result with the first model results. The Y direction of the model is generated from middle span of the sheet pile to 9.84 [m] at middle span of PU20 sheet pile. The updated model is regenerated with 12,100 elements and 19,940 nodes with an average elements size 2.00 [m]. For X direction is remained with the same dimensions. The same procedures is performed where Figure 24 represents the horizontal movements over the combi wall depth for the all seven phases based on the quay wall structural elements parameters and soil HS Model.Reference to Figure 25, it can be said that as much as model width is smaller, the results are mostly about 7% more conservative in average excluding the initial phase due to its minor values. Table 11 presents a comparison among the top displacement predictions of both models.

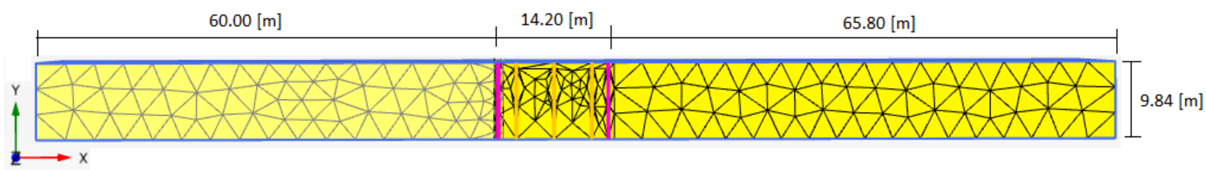


Figure 27: Geometry of the combi wall structure model - 140 [m] x 9.84 [m]

Table 11: Influence of the model width on the calculated results

Construction Phase #	HS Model Top displacement result [mm]		%-difference
	Model width 9.84 [m]	Model width 17.82 [m]	
1	0.46	0.58	-26.26 %
2	34.88	34.04	2.41 %
3	25.99	23.81	8.39 %
4	25.68	23.36	9.03 %
5	39.57	35.16	11.14 %
6	78.85	72.07	8.59 %
7	206.66	192.38	6.91 %

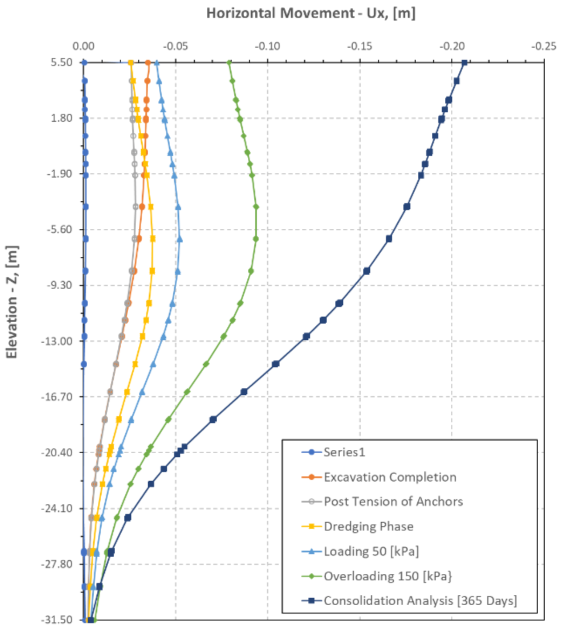


Figure 28. Calculated movements for all phases over combi wall depth –model width = 9.84 [m]

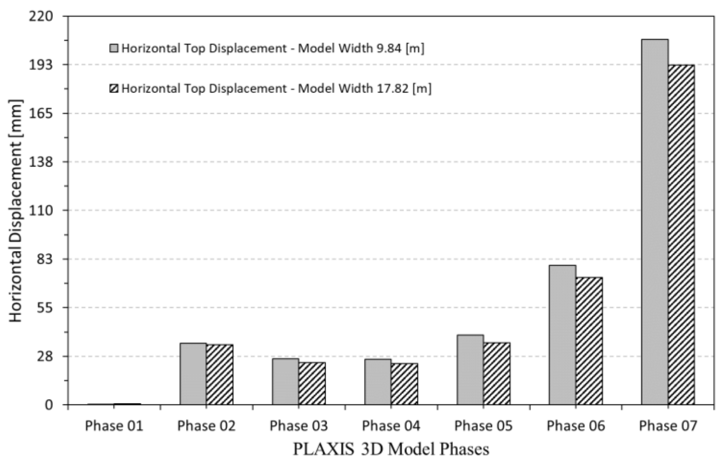


Figure 29. Over estimation of displacement when model width reduces to half

2.8. Comparison of results between Plaxis3D and Plaxis2D

This clause evaluates the displacement analyses of the Rotterdam quay wall by Plaxis 3D if compared to Plaxis 2D model.The soil stratification and constitutive models assigned in 2D model are the same as described above for the 3D analyseshowever it considers the structural elements with an effective width of one meter.In the geometry, four different material datasets have been modified. The combi wall, anchor wall,anchor strands and the concrete foundation piles. To determine the flexural rigidity for the data set of the piles, the EI is calculate and divided by the center to center distance and reduced by a "model" factor and the diameter in order to equally spread out the flexural rigidity over the plate reference to Plaxis 2D manual as follow:

$EI_{\text{plaxis}} = EI_{\text{pile}} / (S.D)$ where S = model factor [-], and D = diameter of the pile [m]

The calculated material properties for the different items of the quay wall are shown in Table 12.

Table 12.Material set input used in Plaxis 2D

Item	Materialtype	Normalstiffness EA [kN/m']	Flexuralrigidity EI [kNm2/m']	Weight w [kN/m']	Spacingout ofplane $L_{spacing}$ [m]
Tubular steel piles	Elastic	7.98E+06	1,51E+06	3,200	-
Anchor wall	Elastic	4.320E+06	1,795E+05	1,690	-
Foundation pile	Elastic	685000	35200	0.457	-
Anchor strands	Elastic	7,200E+05	-	-	3.28

After completing the geometry and dataset, the mesh is generated. A very fine mesh was generated in Plaxis 2D with 1,543 elements (6% of Plaxis 3Dmodel elements) and 12,693 nodes (30%of Plaxis 3D model nodes) with an average elements size 2.50 [m]. Figure 26 shows thegenerated 2-D finite element mesh of Rotterdam model.In order to validate the model approach), the phases developed in in software (PLAXIS 2D V8.6) are same as presented above however,the anchor pre-tensioned force is assigned 400 [kN/m] for the third phase to consider 3.28 m spacing. The total horizontal displacements Ux of the 2-D model is graphically illustrated in Figure 26.The behaviour of the horizontal movements over the combi wall depth for the all seven phases is depicted in Figure 27.

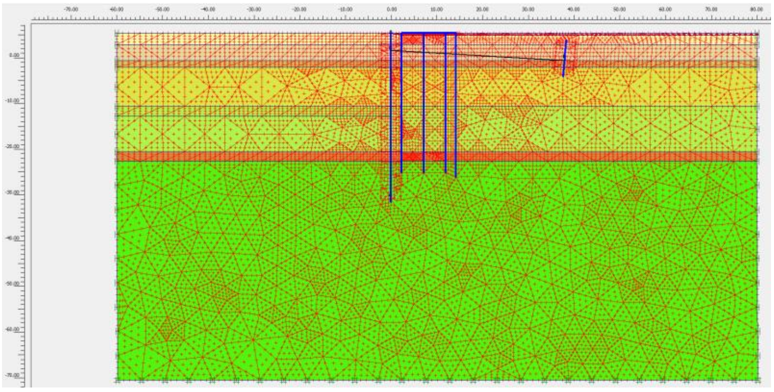


Figure 30. Generated 2-D finite element mesh of Rotterdam model

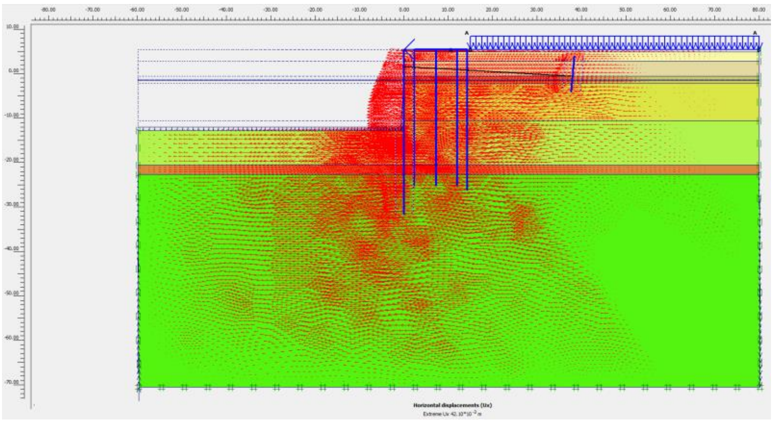


Figure 31. Horizontal calculated displacement Ux of Rotterdam quay wall 2-D model - Phase 5

Finally, the resultsof conmbi wall top displacement obtainedfrom Plaxis 2D model are compared to the 3D model results. Figure 28 shows a comparison between the calculated 2D and 3D displacement results of Phase 05.It is clear that; the2D analysis underestimates the maximum top displacement of combi sheet pile wall with approximately 45 % in average as shown in Figure 29. It can be said that the result obtained from Plaxis 3D for phase 05 is much closer to the field measurement during the operation under design loads. Accordignly, Plaxis 3D is a powerful tool for modeling irregular quay wall systems such as open cell walls which will be addressed in upcoming research

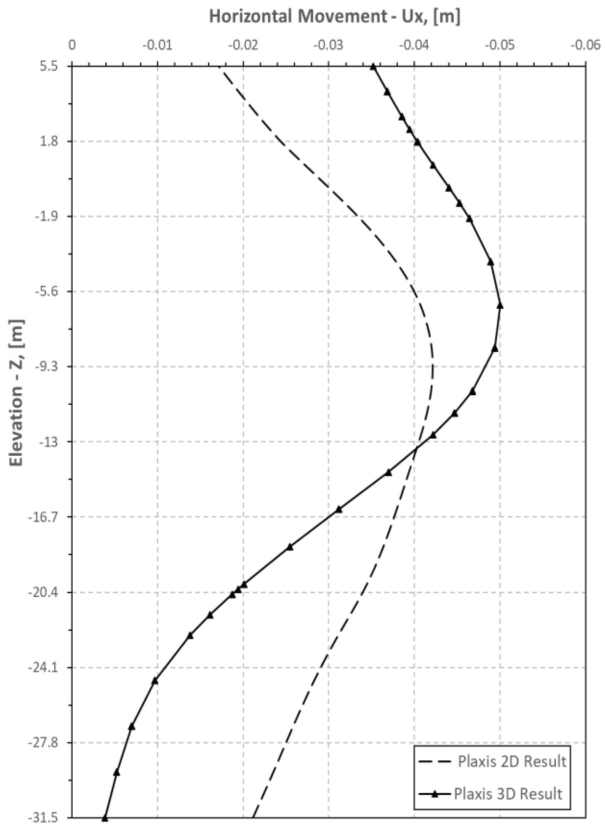


Figure 32. Comparison between the calculated 2D and 3D displacement results of Phase 05

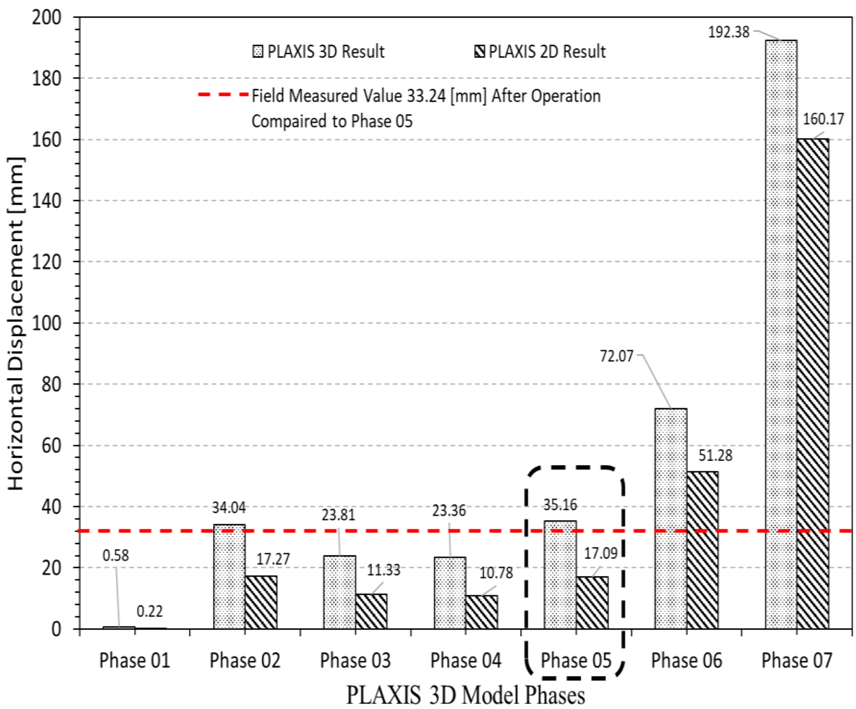


Figure 33. Top displacement results comparison between Plaxis 2D and 3D model

3. CONCLUSIONS

This paper presents a comparative study of quay wall case for the validation of PLAXIS 3D version 2013 for predicting wall displacemet. The study compares the horizontal displacements calculated by Plaxis 3D for a quay wall located at Rotterdam Port in South Holland to field measurements over five years. Afterward, the horizontal displacement results obtained from three-dimensional analysis are compared to Plaxis 2D results.Based on the study performed in this research, and keeping in mind that findings given hereafter are related to Rotterdam existing quay wall, the following conclusions could be drawn:

- The corresponding displacement values never return to its initial condition when reducing surcharge overload to its original value.
- PLAXIS 3D version 2013 could satisfactorily estimates the quay wall structure displacement within (+) 5.8 % from field measurements however; the maximum value is estimated within the consolidation behavior.
- Care should be taken to assign the soil and material parameters in order to obtain accurate results, however the long-term horizontal quay wall movement increments due to consolidation plays an essential role in most quay wall deformation problems.
- In majority, the calculated displacement obtained with HS is much closer to measured values than MC model where MC does not account for unloading in its formulation.
- For anchor quay wall type, the horizontal displacement of anchor sheets has a major impact on the front wall displacement. The relative horizontal displacement between the combi front wall and the anchored wall is represented by a linear relationshipwith about 15% displacement incremnet for anchor wall during the operational phase.
- In general, the calculated displacement of the Plaxis 3D code is mostly more conservative, if compared to Plaxis 2D results.The displacement results for the model obtained fromPlaxis-3D are higher than those obtained by two-dimensional by about 45%.
- The 3D Model dimensionseffects the output results. As much as the model width is 50% smaller, the results are mostly about 7% more conservative on average.

4. REFERENCES

1. Don C, Warrington PE (2007). "Anchored Sheet Pile Wall Analysis Using Fixed End Method without Estimation of Point of Contraflexure," Vulcanhammer info., pp. 1-27.
2. Krabbenhoft K, Lars Damkilde L, Krabbenhoft S (2005). "Ultimate limit state design of sheet pile walls by finite elements and nonlinear programming," Computers and Structures, 83: 383–393.
3. Krabbenhoft K, Damkilde L (2003). "A general nonlinear optimization algorithm for lower bound limit analysis," International J. Num. Math. Eng., 56: 165–184.
4. Lyamin AV, Sloan SW (2002). "Lower bound limit analysis using nonlinear programming," International J. Num. Math. Eng., 55: 573–611.
5. Briaud JL, Lim Y (1999). "Tieback Walls in Sand: Numerical Simulation and Design Implications," J. Geotechnical Geo-environ. Eng., 125(2):101-111.
6. Briaud JL, Nicholson P, Lee J (2000). "Behavior of a Full–Scale VERT Wall in Sand," J. Geotechnical Geo-environ. Eng., 126(9): 808–818.
7. Barley AD (1997). "The failure of a twenty-one year old anchored sheet pile quay wall on the Thames," Ground Engineering, March issue, twww.sbmasystems.com/anchorman/pdfs/GATP10.pdf. pp. 42-45
8. Hamza, M., Hamed, H. (2000). Three Dimensional Soil-Structure Analysis of Port-Said East Quay Wall. Maritime Engineering and Ports II, C.A. Brebbia& J. Olivella, ISBN 1-85312-829-5.
9. S.N. Jonkman, J.G. de Gijt, K.J. Bakker, H.J. Everts and Arjan Galema "Shielding effect of piles on quay walls", Delft University of Technology, 2013
10. Vrijling, Brinkgreve, and Grotegoed "The Gevelco quay wall - Research into the causes of the measured displacements" Delft University of Technology, 2010

11. Aannemingsbedrijf De Klerk BV, Overall planning kade Gevelco te Rotterdam, Werkendam, 2003
12. Anrooij, van L., Deelkwaliteitsplan "Foundation work" – T.b.v. demolishing and building a quay wall with associated work for the Gevelco phase zero in the Brittaniehaven in the municipality of Rotterdam, 2003
13. Malvern, L. E., Introduction to the Mechanics of a Continuum Medium, Prentice-Hall, Inc., New Jersey, 1969
14. Obrzud, R.F., " On the Use of the Hardening Soil Small Strain Model in Geotechnical Practice ", Numerics in Geotechnics and Structures, Edited by Th. Zimmermann, A. Truty, K. Podles, Elmepress International, 2010.
15. Brinkgreve RBJ, Vermeer PA (2002). "PLAXIS user's Manual," Version 2013, Balkema, Rotterdam, Netherlands.
16. Ehab R. Tolba*, Elsayed M. Galal & Mohamed H. Mourad, "effects of the frontal deepening of quay wall for the east port of port said" Malaysian Journal of Civil Engineering 29(2):176-193 (2017)
17. Cheng-Yu Ku, Jing-Jong Jang, Jui-Ying Lai, and Ming-Jr Hsieh, "Modeling Of Dynamic Behavior ForPort Structures Using ThePerformance-Based Seismic Design"Journal of Marine Science and Technology, Vol. 25, No. 6, pp. 732-741 (2017) DOI: 10.6119/JMST-017-1226-14
18. Brinkgreve, R.B.J.: Plaxis 2D version 8, Material models manual, Delft University of Technology, 2007.