Numerical 3D Modelling of a Quay Wall System in Soft Ground Conditions

Modélisation Numérique en 3D, de murs de quai en présence de sol mou

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ABSTRACT: For quay wall structures challenging geotechnical conditions can be met due to the marine environment and it is quite common that a conservative design is adopted due to soft soil deposits and strict serviceability port requirements. The aim of this study is to examine a special cofferdam type of quay wall solution for soft soil conditions taking into account different constitutive soil models, load conditions and 3D soil-structure interaction effects. The investigation was based on finite element model analysis with advanced constitutive models, like the modified cam-clay and the soft soil creep models. The effect of the soil models was examined in detail regarding the different loads and model assumptions of the soft ground conditions. It was found that the modified cam-clay model was appropriate and the most numerically stable to model the soil-structure interaction of the soft soil and the quay wall. Based on the numerical 3D model a detailed performance analysis was completed and the assessment of the adopted structural solution was also conducted. The obtained results can support quay wall projects in soft soil, either on the level of feasibility study, or at the design phase.

RÉSUMÉ: La construction de murs de quai maritimes se fait assez souvent dans des conditions difficiles. Il est dès lors courant d'utiliser une approche conservative, à cause notamment de la présence de sols mous et des réglementations portuaires en vigueurs. L'objectif principal de cette étude est d'examiner un type de quai spécifiquement conçu pour des conditions de construction en sol mou. Cette étude est basée sur des modèles constitutifs, des modèles de chargements et des modèles d'interactions sol-structure en 3D. La méthode des éléments finis a été utilisée en parallèle d'autres modèles, comme le MCC (Modified Cam-Clay) ou un modèle de sol mou. Les prévisions des modèles de sols ont été examinées avec attention au regard des différents chargements et des différentes hypothèses de sol mou. Il a été trouvé que c'est le MCC qui est le modèle le plus approprié est le plus stable pour modéliser les interactions entre le quai et le sol. En se basant sur le modèle 3D, une analyse détaillée de la solution retenue a été faite. Les résultats obtenus et la méthode utilisée permettent d' appuyer et renforcer les capacités d'analyse des projets de murs de quai en sol mou, aussi bien lors de l'étude de faisabilité, que lors de la phase de conception.

KEYWORDS: quay wall, numerical modelling, soft ground condition, creep, modified cam-clay, port structure.

1 INTRODUCTION

The demand for new ports and increasing capacity of harbours has been high in the past decades. The functionalities and quay structures have been changing until recently. The typically used quay wall types are the gravity and sheet pile wall, the relieving platform structures and the open berth quays (De Gijt and Broeken 2014).

In case of soft ground conditions (which is the usual case for marine clay deposits) and large retaining heights, it is necessary to limit the deformations of the front wall and to fulfil strict deformation criteria. In order to achieve this goal either the quay wall must be anchored (with single, or multiple rows of anchors), or a relieving platform should be used. In general, the more slender the quay structure with robust overall behaviour it means the more economical solution it is for great retaining heights in soft ground conditions (Richwien 2008). This structural concept, demands integrated design approach with the application of finite element analyses in order to consider the complex soil-structure interactions in soft soil conditions (Ovando-Shelley E. and Rangel-Nunez 2013), the time-dependent behaviour of the soil and the deformations of the quay structure.

Within this study a special cofferdam like structure was analysed, which ensures stiffness and robustness of the retaining wall together with the enclosed soil (Abouseeda 2016). This encouraged further the necessity of numerical analysis as this quay wall is not a conventional structure.

The main focus of this paper however, is not the structural solution, but the effect of the various constitutive soil models, namely the Modified Cam-Clay (MCC) the Soft Soil Creep (SSC) and the linear elastic-perfectly plastic Mohr-Coulomb (MC), on the performance of the structure.

2 CASE STUDY

The system consists of two walls with a distance of 30.5m; a front wall, which was a combi-wall and at the rear side a secant pile wall (see Figure 1). On the top of the walls stiff and strong capping beams were constructed. The walls were connected with transverse beams, supported by bored piles at the port fenders at every second primary pile with a spacing of 7.2m. In between these beams, plate anchors were attached to the walls with special interlocked mechanical connections (Abouseeda 2016) to limit the bending moment and deformations of the front wall.

The area of the case study was situated at a river delta, which is generally covered by a thick and soft layer of alluvial clayey sediment. The ground water table is close to the surface with minor variation in its level. In addition, the whole area has a continuous subsiding nature over time, which means undergoing consolidation settlements and creep.

The soil profile comprises of a uniform clay layer with the depth of -55m and a 30m thick dense sand layer under it. The bottom of the excavation is at -20m (see Figure 1).

The undrained shear strength (s_u) profile follows the ratio of 0.25 against the effective earth pressure (σ'_v), which is a usual profile for soft normally consolidated clays (NC clays) (Puzrin et al. 2010).

In Table 1 the basic properties for the soft clay material are summarized. It is clear that there is no significant load bearing capacity, due to the low undrained shear strength. Based on the initial void ratio (e_{in}), compression (C_c), and recompression (C_r) indices compelling deformations can occur. Moreover, because of the very high creep rate (C_α), the effect of secondary deformations is significant over time.

Table 1. Basic properties for the NC clay model parameters

Symbol	γ_{sat}	φ'	s_u	C_c	C_r	C_α	e_{in}
Unit	[kN/m^3]	[°]	[kPa]	[-]	[-]	[-]	[-]
Value	18	22	$0.25\sigma'_v$	0.63	0.044	0.032	1.15

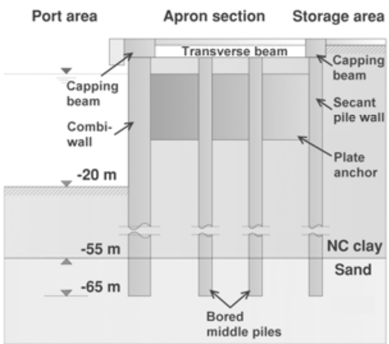

Figure 1. Cross section and the applied soil profile of the model

3 METHODOLOGY

In the current study, in order to consider the 3D effects of this special quay wall system and to be able to model out of plane load cases 3D finite element model was developed in Plaxis 3D AE (Brinkgreve et al. 2015).

In Figure 2 the geometry is shown with the mesh. A coarser mesh was generated at the boundaries, while a finer mesh was used around the structural elements to ensure proper soil-structure interaction assessment. The combi wall and secant pile wall (see Figure 1.) were modelled as a combination of plate and embedded beam elements (Brinkgreve et al. 2015) in order to simplify the original structure (Penzes 2016). The boundary conditions were defined as normally fixed deformation constrains perpendicular to each lateral planes and fully fixed at the bottom of the soil layers.

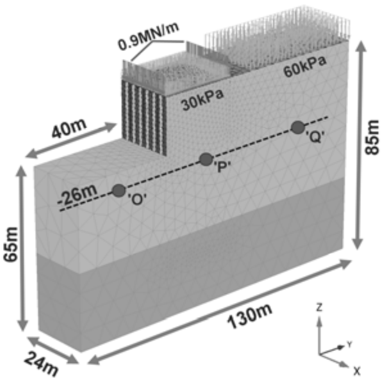

Figure 2. Geometry of the 3D numerical model with the main vertical loads and pre-defined measurement nodes

After the calculation of the initial stresses, the modelling procedure can be divided into three main parts; as construction, in which the predefined geometry was activated separately, excavation in four steps with consolidation and finally loading phases, where vertical service loads and berthing loads were considered. Every loading case was followed by a suitable consolidation phase to analyse its effect on the deformations. Here, only the most important operational load case is detailed, where combined container-traffic surface loads were applied on the apron and storage areas and crane line load on the capping beams (Comb1 load case, see Figure 2).

3.1 *Applied constitutive soil models*

As the main focus is on the various constitutive models with regards to the NC clay, the considered soil models are presented hereafter.

The MCC is an elastic plastic hardening model based on the critical state soil mechanics (Schofield and Wroth 1968), where the non-linearity is captured by means of hardening plasticity (Roscoe and Burland 1968). The state of soil is expressed by the effective mean stress (p'), the deviatoric shear stress (q) and the specific volume (v). The MCC explains fairly well the stress dependent soil strength and the hardening-softening behaviour of clayey soils during shearing. In this way, the coupled effect of shearing and pore pressure generation and dissipation is captured with the model (Wood 1990). Besides its limitations the MCC is numerically stable and it is good for the analysis of very soft soils, especially under compression (Lees 2016).

In the case of NC soft clays creep can have a significant effect on the deformations (Neher et al. 2001). The formulation of the SSC is based on the MCC and it is well described by Vermeer and Neher 1999. In this model the volumetric strain consists of the combination of elastic and a visco-plastic creep strains and the failure is defined by the Mohr-Coulomb failure criteria. Important to note, that in the p'-q plane the cap can increase due to the pre-consolidation stress governed by the creep strains, meanwhile for MCC the cap is defined by the pre-consolidation pressure. This implies that the stress history of the soil has a major role in the deformations for SSC. For instance, a young NC clay modelled with SSC would suffer large deformations due to the high initial creep rate. Thus, in modelling practice an initial ageing calculation procedure shall be used (Waterman and Broere 2011, Thinh et al. 2016).

As one of the most commonly used and numerically stable soil model, the elastic-perfectly plastic MC soil model was also employed in this study to ensure valuable comparison. However, the coupled shearing and excess pore pressure generation is not captured correctly by the MC model (Puzrin et al. 2010). Thus, additional caution is necessary in case of soft NC clay.

3.2 *Adjustment of the soil parameters*

The parameters of the three constitutive models were adequately adjusted in order to simulate identical strain stress response of the soft NC clay. While for the dense sand, the Hardening Soil model was used with 36° effective friction angle and 40MPa tangent oedometric stiffness.

First of all, the properties of the MCC model; the tangent of the critical state line (M) and the compression (λ), swelling (κ) indices were defined. For the critical state line (CSL) M was easily calculated from the effective friction angle (Wood 1990). For the consolidation parameters, the undrained shear strength profile ensured the adjustment criteria (Eq.1).

$$\frac{s_u}{\sigma_v'} = \frac{M}{2} \frac{OCR^{\Lambda}}{2} \frac{1}{3}(1 + 2K_0) \tag{1}$$

Where OCR is the over consolidation ratio, Λ is the ratio between elasto-plastic and plastic stiffness as a function of λ and κ (Eq. 2) and K_0 is the lateral earth pressure coefficient based on the Jáky-formula. With the assumption based on SHANSEP (Ladd et al. 1974, Steenfelt et al. 1992) Λ=0.8 was applied in Eq. 1 and an OCR of 1.46 was calculated as normally consolidated soil may show realistic OCR value around 1.5 (Brinkgreve 2002).

$$\Lambda = \frac{\lambda - \kappa}{\lambda} \tag{2}$$

Based on the compression index and Eq. 2 the MCC parameters were defined (see Table 2).

Table 2. Applied MCC and SSC parameters of the NC soft clay

Symbol	M	λ	κ	λ^*	κ^*	μ^*	ν_{ur}
Value	0.856	0.274	0.0548	0.127	0.025	0.00637	0.08

In case of the SSC model parameters (see Table 2) the above defined λ and κ were the bases with e_{in} for the modified compression (λ^*) and swelling (κ^*) indices. The modified creep index (μ^*) was calculated by the C_α creep index (Brinkgreve et al. 2015). For the unloading and reloading Poisson's ratio (ν_{ur}) a value of 0.08 was applied to align the deformations.

For the MC soil model sufficiently small 3MPa Young's modulus was used with 0.4 Poisson's ratio in order to ensure the same deformation characteristic.

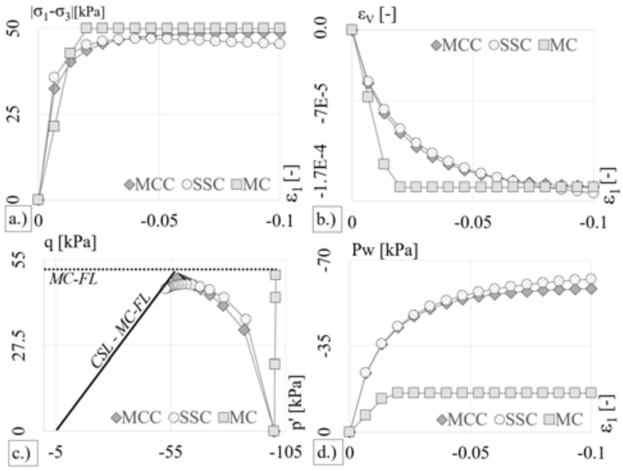

Figure 3. Stress-strain response (a.), deformations plot (b.), effective stress path (c.) and pore water pressure (d.) of the various soil models from Plaxis Soil Test module

In Figure 3 the calibrated characteristic behaviour from CU triaxial tests of the different soil models is shown. In case of deformations (see Figure 3a, 3b) a fairly good agreement was found between all three soil models. Also in the p'-q plane the MCC and SSC followed a very similar path.

The main difference is found for the MC soil model. For undrained B calculation (Brinkgreve et al. 2015) MC soil model assumes isotropic, elastic behaviour during the pre-failure deformation. Thus, in case of undrained shear loading, when no volumetric change is allowed (see Figure 3b), the pore water pressure is constant (see Figure 3d). This difference in the pore water pressures, can be seen between the effective stress paths as well (see Figure 3c). Where, the mean effective stress (p') is constant and the effective stress path of the undrained MC soil model is vertical. However, in case of normally consolidated and slightly over-consolidated clays during undrained loading additional excess pore water pressure is generated and the effective stress paths deviates to the left from the vertical one, like in case of the MCC and SSC model. This difference can entail significant discrepancy in the serviceability assessment.

4 RESULTS AND DISCUSSION

It was found that in this current geotechnical situation the structure is very stiff and robust. The capping and transverse beams govern the load distribution and restrict the deformations, thus the plate anchor system has negligible influence on the overall structural behaviour (Penzes 2016). In addition, the deformations were rather small in every examined load cases.

In Figure 4 the vertical displacements are shown which were observed at Comb1 loading phase. The displacement pattern is similar for all the examined soil models; with swelling under the excavated port area and settlements under the storage area. Under the deep founded quay structure almost negligible deformations occurred.

However, at the consolidation phase three main differences can be observed. Namely, at the port area heaving is the most prominent in the SSC soil model, in the MC model, there was almost no heaving during the consolidation phase and under the storage area during consolidation, way larger settlements occurred in case of the MCC soil model, than with SSC or MC.

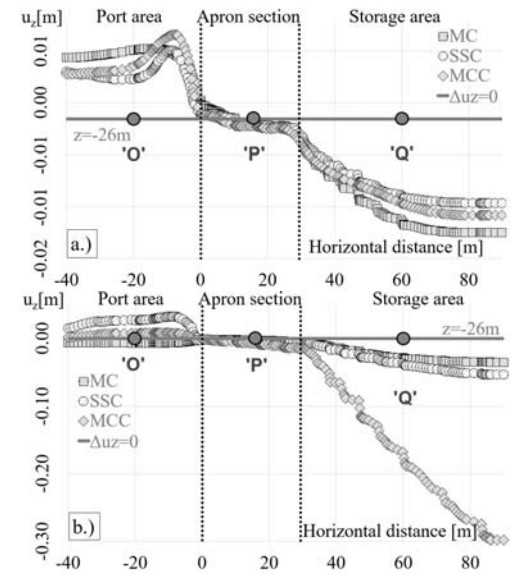

Figure 4. Vertical displacements (u_z) at -26m depth for load case Comb1 (a.), and the consolidation phase (30 days) of Comb1 (b.) (Penzes 2016)

In order to get a more detailed insight about the soil behaviour the effective stress path under the excavation (O node), the quay structure (P node) and the storage area (Q node) are also shown at the depth of -26m in case of the three different soil models (see Figure 5).

In Figure 5a the p'-q plane is shown for the MCC case, where node P was initially close to the cap in plastic state. During the excavation and loading phases no changes in the stress path was observed, as the quay structure prevented any further stress modification.

In case of node O during excavation and consolidation steps (O.1-O.4) when swelling and suction occurred the mean effective stresses decreased and the shear stresses increased in the extension zone (-q). At Comb1 load case the stress path reached the plastic yield surface (O.5) and proceeded towards the critical state line to meet with the failure criteria during the consolidation phase (O.6). As node Q was under the storage area, which was subjected to compression. It was initially close to the plastic state, which was not changed during the excavation phases (Q.1). As the loads of Comb1 were subjected to the storage area the stress path changed due to the undrained loading (Q.1 to Q.2) and caused yielding in the soil and expansion of the original yield locus due to consolidation (Q.3).

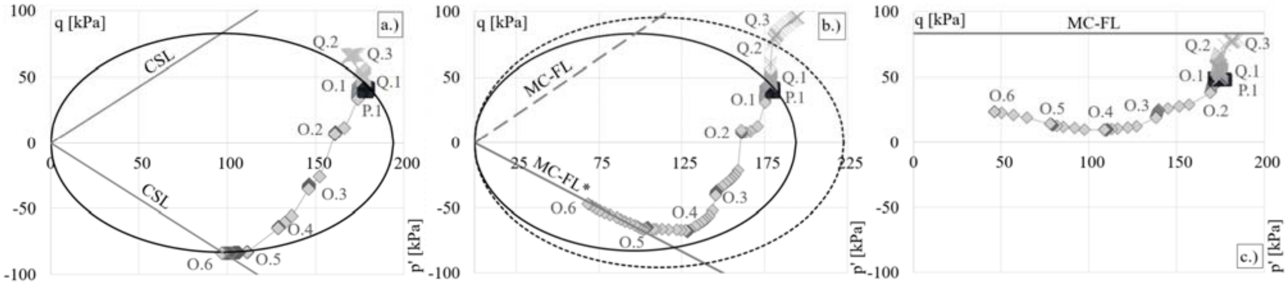

Figure 5. Effective stress path in p'-q plane in case of the MCC (a.), the SSC (b.) and the MC soil models (c.) at -26m depth from the excavation phases until the consolidation phase of Comb1 load case (Penzes 2016)

Subsequently, during the consolidation after Comb1 load case, there were no changes in the deviatoric stresses, $\Delta q=0$. Thus, the consolidation was accompanied by plastic constant volume shearing (horizontal part Q.3).

In case of the SSC model (see Figure 5b) with the dashed line the expanded yield surface is shown due to the necessary initial ageing procedure. In addition, the MC failure lines for compression (MC-FL) and extension (MC-FL*) are also represented (Wood 1990). Similar behaviour was observed like in case of the MCC model under the excavation (node O). After the four excavation phases (O.1-O.4) with Comb1 loading phase (O.5) the effective stress path reached its extension failure criteria, as this part was in swelling. Afterwards, during the consolidation phase of Comb1 the mean effective pressure decreased along the failure line. Meanwhile, with the MCC model the soil was more in the yielding state than in failure. It is also well represented, that due to the expanded yield cap, now P.1 and Q.1 were in elastic state. Moreover, in case of node Q under the storage area, there was an elastic response during the excavation and the loading phases (Q.1-Q.2) as the shear stresses were increasing without any increment in the mean effective stresses. At the consolidation phase it reached the expanded yield criteria and coupled consolidation and shearing took place (Q.2-Q.3).

In case of the MC soil model, the effective undrained stress path is shown in Figure 5c. Because the swelling was way smaller than in the MCC and SSC models the extension was less prominent (O.1-O.6). In general, elastic behaviour can be observed, as the undrained shear strength at the particular depth of the nodes ensured appropriate strength. Meaning, that there was no failure and every node was in elastic state.

5 CONCLUSION

It can be concluded, that the significant differences of the storage area's settlements came from the fact, that in case of the MCC model, the soil mostly reached its yield criteria and significant plastic deformations took place. Meanwhile, in case of the SSC and MC soil models mainly elastic deformations occurred. Consequently, there was no significant effect of creep under the storage area, as creep is a part of the plastic deformations, which was not reached.

However, the effect of creep can be found in case of the swelling at the excavated area, because with SSC the unloading characteristic is changing in time, based on the creep rate. Therefore, the larger the creep rate, the faster and more excessive the swelling becomes, which can result in failure. Similar unloading characteristic can be seen for some Palaeogene clays with high plasticity (Krogsbøll et al. 2012).

In this way, neglecting the effect of creep should be carefully considered depending on the potential stress path. The numerically stable MCC model can be an alternative modelling option, if creep is minor or negligible. On the contrary, MC model is not suggested for modelling soft NC clays.

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