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Ground displacement at Lyttelton Port; A comparison between Plaxis 2D models and monitored movements

Déplacement de sol au port de Lyttelton; Une comparaison entre des modèles Plaxis 2D et les mouvements mesurés

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ABSTRACT: Following the damage caused to Lyttelton Port's infrastructure by the 2010 – 2011 Canterbury Earthquake Sequence (CES) a new Cruise Berth has been constructed adjacent to the Port's Eastern Mole. The Eastern Mole, which was reclaimed at the beginning of last century with deposited quarried fill into the bay's Recent Marine Sediment (RMS), suffered over 1m of movement during the CES. The RMS, where most of the movement was inferred to occur, comprises 45m of poorly consolidated loess derived silt, interbedded with impersistent layers of sand and gravel. Modelling to predict ground movement during the filling, dredging, slope reprofiling and piling for the new Cruise Berth was undertaken during design prior to construction. Results of this modelling informed and facilitated a program of movement monitoring during construction. This paper presents a summary of the 2D Plaxis analyses which informed the enabling works design and provided the allowable levels of movement during construction. Stiffness properties for the RMS were based on oedometer and consolidated undrained triaxial testing undertaken by Bjerrum in the 1950s. Both Mohr-Coulomb (MC) and Hardening Soil with Small Strain (HSSS) were employed to model the RMS. The movement of a series of pins, prisms and inclinometers monitored during construction were compared with the movements predicted by the MC and HSSS models. For the specific conditions at Lyttelton Port it is shown that, in comparison to the MC model, the HSSS model gives a better correlation with observed ground displacement. Additionally, as a Class A prediction, the HSSS model compared well with measured displacements.

RÉSUMÉ : Suite aux dommages que la série de tremblement de terre de Canterbury (2010-2011) a causés à l'infrastructure du port de Lyttelton, un nouveau quai a été construit adjacent au môle-est du port. Le môle-est, construit à la fin du 20ème siècle à l'aide de remblais déposés sur le fond marin de la baie, a souffert de plus d'un mètre de mouvement lors de la série de tremblement de terre en 2010-2011. Une grande partie de ce mouvement est présumé être survenu dans le Récent Dépôt Sédimentaire Marin (RDSM), qui se compose de 45m de loess faiblement consolidé issu de limon, interstratifié par des couches de sable et de graviers non-continues. Un model a été créé afin de prévoir les mouvements de sol lors du remplissage, du dragage, du reprofilage de la pente et du placement des pieux de fondation. En réponse aux résultats du model, une structure qui permet les travaux sur pieux a été installée et un programme de surveillance des mouvements durant la construction a été mis en place. Cet article présente un résumé de l'analyse 2D Plaxis qui a informé le design des travaux et qui a fourni la quantité de mouvement admissible pendant la construction. La dureté du RDSM a été basée sur des essais œdométriques et des essais triaxiaux de sol consolidé non drainé entrepris par Bjerrum dans les années 50. Les deux méthodes, Mohr-Coulomb (MC) et Hardening Soil Small Stiffness (HSSS) ont été utilisés pour modéliser le RDSM. Les mouvements observés durant les travaux par une série de broches, prismes et inclinomètres ont été comparés aux mouvements prédits par le modèle utilisant MC et le modèle utilisant HSSS . Pour le port de Lyttelton spécifiquement, il s'est avéré que le modèle utilisant la méthode HSSS a produit un meilleur résultat que celui utilisant MC. De plus, en tant que modèle avec des prédictions de Class A, le modèle utilisant HSSS comparait bien avec les déplacements mesurés.

KEYWORDS: Lyttelton Port, Plaxis, excavation, HSSS Model, Mohr-Coulomb.

1 INTRODUCTION

High levels of ground movement during construction of the new Lyttelton cruise berth wharf and its associated dredging work was considered a major risk during the concept phase of the project. The new Cruise Berth has been constructed adjacent to the Port's Eastern Mole. The Eastern Mole, which was reclaimed at the beginning of last century with deposited quarried fill into the bay's Recent Marine Sediment (RMS), suffered over 1m of movement during the CES, demonstrating a poor performance during a seismic event and an indication of low strength soils under loading.

The RMS, where most of the movement was inferred to occur, comprises 45m of poorly consolidated loess derived silt, interbedded with impersistent layers of sand and gravel. Modelling which predicted large ground movement during the filling, dredging, slope reprofiling and piling for the Cruise Berth resulted in the installation of piled enabling works structures and a program of movement monitoring during construction.

The deformation behaviour of the revetment as a result of construction activities, in particular dredging activities for the berth pocket, was investigated using both the Mohr-Coulomb (MC) and the Hardening Soil with Small Strain (HSSS) constitutive models simulating the RMS material in Plaxis 2D.

The objective of this study is to investigate the differences in computed deformation of the ground when using the MC and the HSSS models compared with the monitoring data to assess their applicability in the actual design process. As a Class A prediction, the HSSS model compared well with the measured displacements.

Stiffness properties for the RMS were based on oedometer and consolidated undrained triaxial testing undertaken by Bjerrum in the 1950s.

A brief description of the project, general ground conditions, details of the enabling works, and Plaxis modelling of the construction activities are outlined in the following sections. Design predictions are then compared with the monitored performance.

2 BACKGROUND

Since the 2010-2011 Canterbury earthquake sequence of events, large cruise ships have been unable to berth at Lyttelton Port of Christchurch New Zealand. Christchurch City Council (CCC) had a strong desire for large cruise vessels to return to Lyttelton Port due to the wider tourism benefits to the City (Beca 2019).

In 2017, LPC planned to construct a new facility for cruise ships to the west of Cashin Quay 4 (CQ4) with the ability to accommodate the latest generation of cruise ships up to 360m long, carrying in excess of 6,000 passengers whilst not impeding on normal port operations. This section provides a brief background on the project and some details around geotechnical conditions of the site.

2.1 *Project Description*

The proposed Cruise Berth is located in the outer harbour on the southern side of the Eastern Mole at the western end of Cashin Quay 4 (CQ4) of Lyttelton Port of Christchurch, New Zealand. The Eastern Mole comprises an area of reclamation aligned with CQ4, deviating west north west from an elbow at the western end of CQ4 and extending to Z Berth. Z Berth comprises a timber piled marine structure on the inner harbour (north) side of the Eastern Mole.

The project consists of a wharf structure to berth the latest generation of cruise ships. Figure 1 shows an outline of the wharf structure with Z berth being at the existing inner harbour side. The new facility includes mooring lines tied back to two landside mooring structures which have been omitted from Figure 1 for clarity.

The development required filling the Eastern Mole to raise the area to a general level of +4m above the chart datum (CD), dredging to form a berth pocket and scour protection in the form of rock bags placed on the revetment slope.

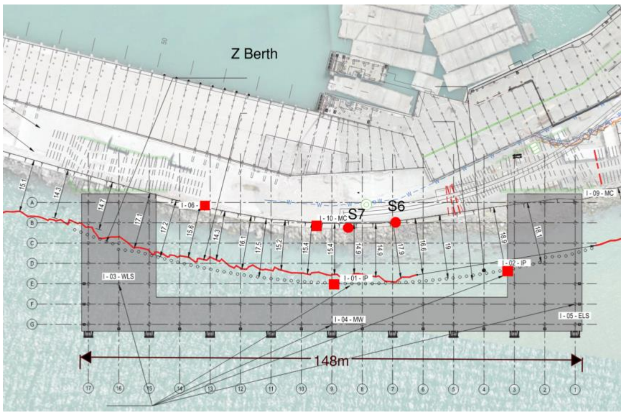

Figure 1. Cruise Berth (shaded in grey) Site Plan.

2.2 *Construction Methodology*

The main wharf structure consists of a cast in situ suspended deck slab supported on 914mm diameter closed-ended steel CHS piles, driven to a founding level between -62mCD and -65mCD in Fan deposits. The alignment required dredging around the wharf structure and along the shore with scour protection in the form of rock bags.

Through the Early Contractor Involvement (ECI), it was determined that most of the facility would need to be constructed from land, or from staging extended from the land. To mitigate the risk of instability, an Enabling Works concept was developed. This concept comprised two rows of piles installed into the reclamation fill adjacent to the inner and outer harbour with in-situ reinforced concrete capping beams connecting the piles longitudinally and tied together laterally with steel ties. The crest piles comprised closed ended CHS piles; 610mm diameter and typically 6m long on the inner harbour side and 710mm diameter and 15m long on the outer harbour side.

In order to minimize ground movements, an additional row of 'intermediate' piles installed part way down the revetment were designed to reduce the short-term risk of instability during dredging. They were driven approximately 17 – 19m from the southern edge of the outer harbour enabling works beam. They comprised open ended 810mm diameter CHS piles with their toes founded at -34mCD. They were typically placed at 2.0m centres with the soil between expected to arch between the piles effectively forming a wall. Plant comprising 250t and 280t cranes were employed to undertake pile driving. Figure 2 illustrates an indicative cross section with the dredged berth pocket in front of the intermediate piles.

To mitigate the risk of instability, it was decided to adopt an observational approach with prompt monitoring of ground movements and pile deflections for comparison against predetermined alert and action levels.

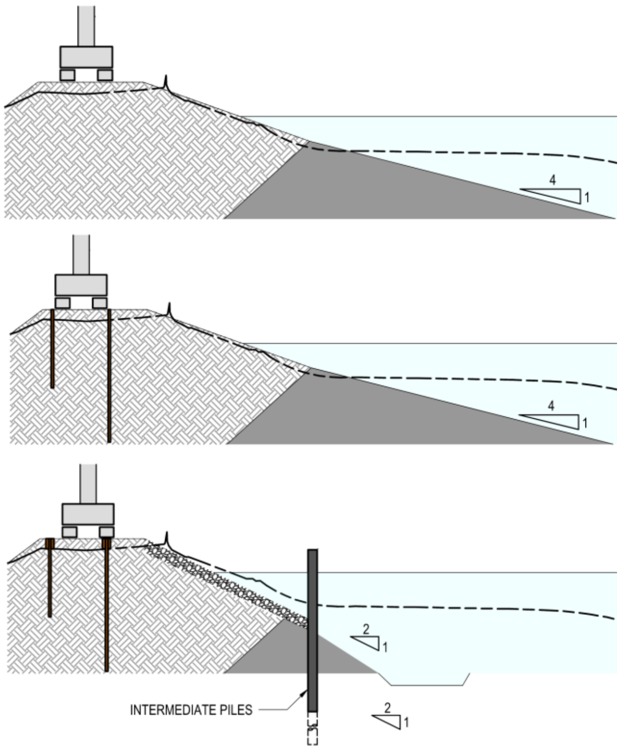

Figure 2. Schematic Construction Sequence, initial phase (top); crest pile installation (middle), intermediate pile installation and dredging (bottom).

2.3 *Ground Conditions*

The natural ground comprises a continuous layer of fine, poorly consolidated sediment that represents seabed sediments. The RMS originates from loess soils washed off the surrounding hills and is characterized by normally consolidated, low-strength and low-plasticity silt with impersistent sand/gravel layers increasing with depth. Lyttelton Volcanic Series bedrock was expected in excess of 70m below chart datum (CD).

Exploratory holes which extended to bedrock were drilled in the vicinity of the proposed facility and they indicated geological variability in the ground conditions. The exploratory holes included Standard Penetration Tests (SPTs) in granular material and hand shear vane tests carried out within the end of the core barrel in cohesive soils. In order to supplement the information gathered from the borings, Cone Penetration Tests (CPTu) were also carried out within the RMS strata under the revetment and at sea.

Some historical laboratory data from the Norwegian Geotechnical Institute were also available from 1957. This included oedometer, consolidated undrained triaxial and Atterberg limit tests undertaken on the RMS from east of the site.

Based on the exploratory holes and the historical information it was inferred that the Eastern Mole was a historical reclamation in the order of 14m – 17m thick of predominantly gravel sized stone quarried from the harbour's northern hillside pushed into the Recent Marine Sediments. The revetment slope considered to be at the angle of natural repose of the quarried fill.

Based on the available data an inferred ground model was developed which is presented in Table 1 and shown in Figure 3. In this paper, the study is concentrated on the cross section along grid line 9 because this was considered the most critical area in terms of the steepest reprofiled dredge slope. An overview of the wharf structure in relation to the land is shown in Figure 1.

The highest groundwater was measured to be approximately at 2.5mCD, but was controlled by tide levels.

Table 1. Inferred Ground Model

Unit	*Approximate top of layer (mCD)*	*SPT N value**
Unit 1: Reclamation Fill	+4	7 – 28 [10]
Unit 2a: clayey silt and silt [marine deposits]	-3 to -15	No SPT available
Unit 2b: clayey silt and silt [marine deposits]	-20 to -25	5 – 17 [8]
Unit 3a: Silt, sand and occasional gravel [marine deposits]	-18 to -20	7 – 9 [8]
Unit 3b: Silt, sand and occasional gravel [marine deposits]	-25 to -28	4 – 33 [12]
Unit 3c: Silt, sand and occasional gravel [marine deposits]	-40 to -45	0 – 84 [20]
Unit 4: alluvial silt, sand, gravel and cobbles [Debris/ Fan deposits]	-60	22 – 120 [40]
Unit 5: Basalt and pyroclastic material [Lyttelton Volcanic Group]	-70	11 – 140 [50]

* Adopted values in brackets

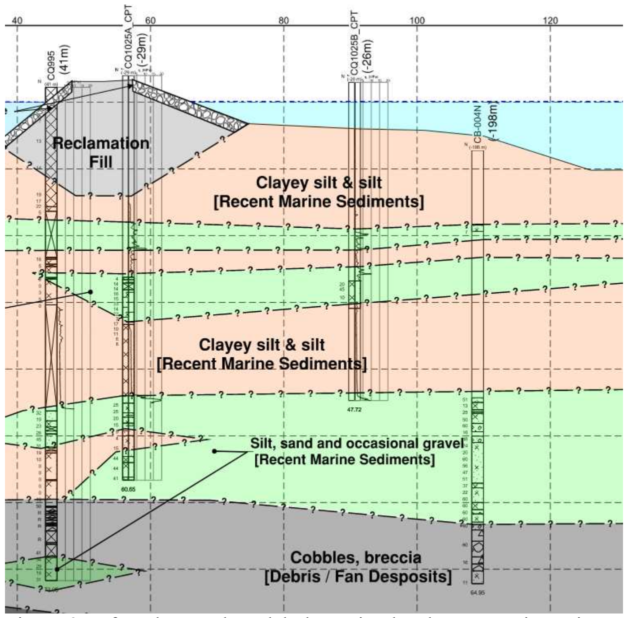

Figure 3. Inferred ground model along simulated cross section prior to berth pocket dredging

3 EVALUATION USING PLAXIS 2D

3.1 *Ground Parameters*

Considering the high risk of large ground movements during construction, it was decided that soil deformation behaviour and its impact on the enabling works structure would be assessed using the finite element code Plaxis 2D.

All soils layers were modelled with the elasto-perfectly plastic Mohr-Coulomb (MC) constitutive model, with the exception of the normally consolidated RMS below the seabed. The behaviour of this was investigated using both the MC and the Hardening Soil with Small Strain (HSSS) models.

The strength parameters were derived using the SPT values and CPTu traces. In particular, the undrained shear strength of the RMS deposits was derived based on SHANSEP model adopting a design ratio of undrained shear strength to effective vertical stress of 0.22 (Figure 4). The normally consolidated marine silts, when subjected to an increase in overburden pressure, were expected to have gained in strength. This strength gain was assumed in the design for RMS under the reclamation fill.

The preferred approach for the establishment of stiffness parameters starts with laboratory testing. However, in the absence of experimental data for the determination of this parameter, approximation through correlations can be appropriate.

Oedometer and consolidated undrained triaxial testing on samples from RMS were undertaken by Bjerrum in the 1950s. The tests results were used to derive the HSSS parameters secant stiffness E_{50} and the oedometer modulus, E_{oed}, at a reference pressure of 100kPa. The unloading-reloading stiffness E_{ur} was estimated using the correlations proposed by Brinkgreve et al. (1993).

In addition, two correlations suggested by Mayne and Rix (1993) and Robertson (2012) were used to derive the small stiffness shear modulus where in-situ cone resistances from CPT testing were available. In addition, results from consolidated undrained triaxial tests were also compared against the derived moduli values from these empirical correlations.

For the calculation of the threshold shear strain $g_{0.7}$ at which the normalized small strain shear modulus G/G_0 has reduced to 70%, the Vucetic and Dobry (1991) empirical normalized modulus reduction curves were used in combination with the plasticity index of RMS samples measured in the laboratory.

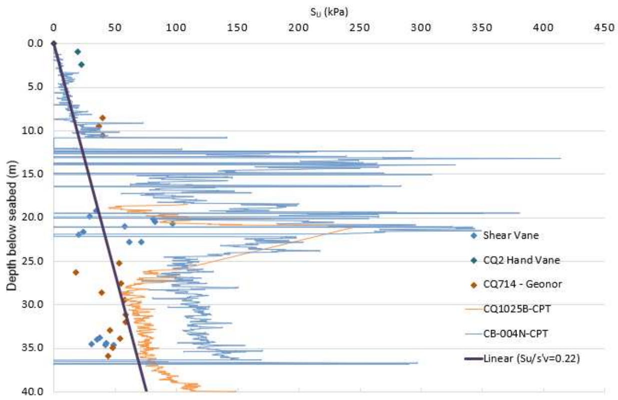

Figure 4. SHANSEP model adopting a design ratio of undrained shear strength to effective vertical stress of 0.22 based on in-situ testing

Due to space limitations, only the relevant RMS soil parameters used for the analysis are shown in Table 2.

Table 2. RMS Soil Parameters

Parameter	MC	HSSS
E' (kPa)	*1600.z**	-
E_{50}^{ref} *(kPa)*	-	2000
E_{oed}^{ref} *(kPa)*	-	1000
E_{ur}^{ref} *(kPa)*	-	8000
Power m	-	1
Strength (kPa)	1.6.z* + 5	1.6.z* + 5
G_0^{ref} *(kPa)*	-	30,000
$\gamma_{0.7}$	-	5 x 10^{-4}

* z is the depth (m) below seabed level.

3.2 *Plaxis Modelling*

To refine the design, seven 'design' areas within the project site were identified for analysis. Areas 1 and 2 were located on eastern side of the wharf structure. Areas 3 – 5 encompassed the wharf footprint and Areas 6 – 7 covered the west side of the wharf structure. The 2D finite element modelling using Plaxis was undertaken to simulate the construction sequence at Grid Line 9 or Chainage 170 (Ch. 170) within Design Area 4. This section represented the steepest slope angle along the Eastern Mole alignment.

The initial construction phases covered the modelling of reprofiling of the crest and installation of two rows of crest piles as part of the enabling works. All piles are modelled as a linearly elastic isotropic material using embedded beam elements which have built-in interface elements. Crane loads were simplified as point loads acting at the crest of the revetment. Engineered fill with geogrids were also modelled at the crest of the revetment with geogrids being elastic-plastic geogrid elements. It should be noted that at the inner harbour side of the mole (i.e. Z berth), there are 5 – 7 rows of existing timber piles which were not modelled as these piles affected the stability on the northern side of the Eastern Mole which falls outside the scope of this paper. The enabling works stage at the crest of the revetment was followed by the construction of the intermediate pile row part way down the revetment. All construction stages prior to excavation is denoted as Stage 1 or pre-dredging in this paper.

In Stage 2 or post-dredging, the slope and berth pocket were created by excavation in two different phases. Up to 9m of RMS was dredged at the berth pocket alignment creating a 1(V):2(H) slope angle at the steepest cross section (i.e. Chainage 170 at Area 4).

Construction stage factors of safety were evaluated first using Strength Reduction Method using Plaxis. The stability of the works during construction was designed to have a short term Factor of Safety greater than 1.2 which equates to an approximate annual probability of exceedance between 1 in 4 years to 1 in 20 years (Silva et al. 2008).

There were some assumptions and uncertainties in the modelling including the construction-imposed demands and the construction tolerances. Sensitivity analyses identified that small changes in the model result in a reduction of factor of safety below 1.2 with shallow movement indicated and some creep and movement of the ground to be expected. This was proved to be the case during construction when piling for the wharf structure commenced. For example, in one instance rapid driving of 4 adjacent piles resulted in more than 150mm of ground movement.

4 MONITORING PLAN

In order to capture the deformation behaviour of the ground, several monitoring pins, prisms and inclinometers were installed at the project site.

Inclinometers at the crest and within selected intermediate piles were installed. Prisms were also used in the inner and outer harbour sides and on selected intermediate piles. Pins were mounted on the capping beam for the southern and northern crest sides.

The frequency of monitoring were prepared in advance of construction in collaboration with the port authorities and the contractor.

In the presented cross section (i.e. Ch. 170), the development of deformations of the ground surface at land was studied at four locations i.e. four survey points at the outer harbour side of the crest comprising pins at S6, S7 and prisms on Inclinometers I06 and I10. As shown in Figure 1, the inclinometers and pins are denoted as solid squares and circles, respectively.

The ground movements at depth were also compared against model prediction at two points along the crest and another two points along the intermediate piles just above the dredged berth pocket. Two inclinometers at the crest were denoted I06 and I10 and the two inclinometers in the intermediate piles were denoted I01 and I02. Records of the rest of the monitoring points outside Area 4 are not presented in this paper.

For the purpose of validation, the numerical results of ground deformations will be compared with the measured data in the following section. Only relevant monitoring points in close proximity to the modelled cross section have been adopted for comparison.

5 COMPARISON STUDY

To assess the deformations as a result of construction loadings and dredging, several phased calculations were performed. However, for simplicity and comparison purposes, the construction phases are divided to pre and post dredging states. For these two phases, calculated deformations of the ground will be compared with measured data obtained during the construction. The objective is to investigate the performance of the MC and HSSS models employing simple laboratory and in-situ testing and empirical correlations, and to compare the numerical results with the monitoring records to assess the model performance during the design process.

As an example, Figure 5 shows the total displacement computed by Plaxis using HSSS model for the RMS material. A pin at the outer harbour side of the crest was chosen to compare against recorded values. As shown in Figure 5, notable displacement is modelled to occur around and above the intermediate pile alignment up to the crest.

Figure 6 shows the measured and computed horizontal displacements of the crest at the outer harbour side of the mole using the HSSS model. It shows that ground movements at the outer side of the crest using the HSSS model were assessed to be approximately 40 and 100mm, prior to and after dredging respectively. When compared to the measured data, and with the exception of I10, the HSSS model generally overestimated the ground movements by up to 40%. Nevertheless, the HSSS model shows a good agreement with the measured data following dredging commencement. Clearly, some discrepancy should be expected between the numerical results obtained using the HSSS model and the field measurements due to variations of the ground conditions and geometry of the slope.

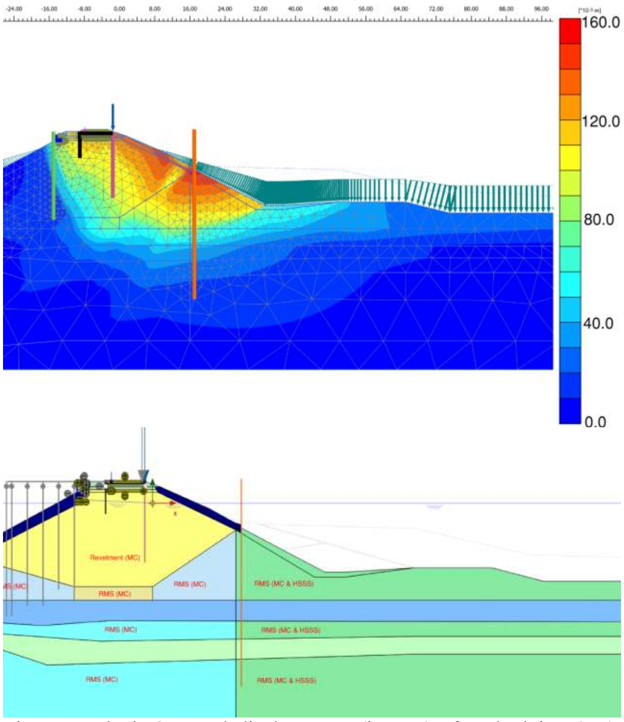

Figure 5. Plaxis 2D total displacement (in mm) after dredging (top), ground model in Plaxis (bottom) with RMS layers modelled as HSSS seaward of the intermediate piles and MC under the revetment.

Some of the large fluctuations of the monitoring points related to observed construction activities such as being hit by equipment or local instability of the crest. These were corrected in the following reports. The pins S6 and S7 which are located either side of the modelled cross section, indicate between 60 – 80% of deformations compared to the computed values. However, the pin attached to the inclinometer I10 moved 20 – 40% more than the computed estimate.

The prediction employing the MC model is omitted for clarity of the presented figure. However, in summary, the Plaxis MC simulation estimated up to 100mm and 1m of ground movement at the outer harbour side of the crest pre and post dredging stages, respectively. Comparing the MC model with the measured data, it is evident that the numerical results at the top of the mole are approximately 10 – 15 times higher than the measured values. This is hypothesized to be due to the inability of the MC model to incorporate soil hardening and small-strain stiffness behaviour of the RMS.

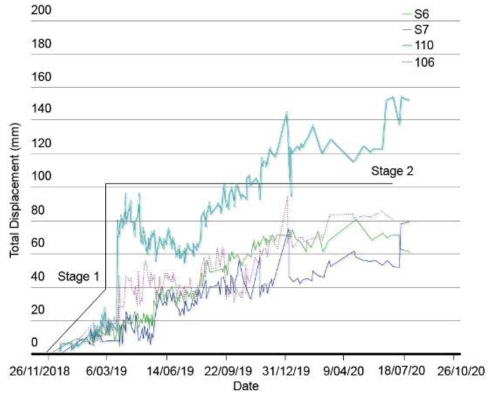

Figure 6. Comparison of Field Survey Data (outer harbour) with Plaxis Computed Results. Stage 1 prior to dredging, and Stage 2 post dredging

The ground movement at depth are further investigated by comparing the inclinometer data with the numerical results using the HSSS model as shown in Figure 7. Generally, a similar trend can be seen when the inclinometer data is compared to the numerical prediction. However, it can be seen that the numerical results obtained using the HSSS model are overestimating the ground movement at the surface more than they do at depth.

The ground movements of the crest and with depth were compared using the Plaxis model and inclinometers I06 and I10. Both of these inclinometers were installed in PVC tubes to depth of up to 3m below ground level to protect them against local instability. Hence, some shallow ground movements were measured which may not be representative for the recorded profile.

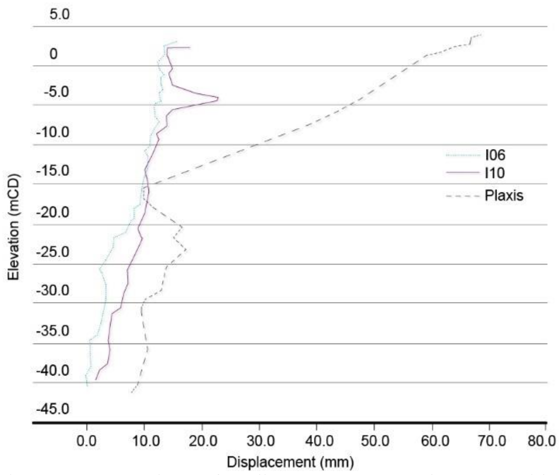

Figure 7. Comparison of Inclinometer Data at the Crest with Plaxis Computed Results.

Note that the Plaxis model overestimates the ground movement above -15mCD which is the bottom of the modelled quarried fill revetment. Below this level, with some minor discrepancy, the level of assessed movement is more comparable with the recorded values. However above -15mCD, the Plaxis modelling predicted up to three times more deflection than what was measured. Note that the inclinometers at the crest are installed within the reclamation fill and underlying RMS with the toe of the inclinometers being at around -40mCD. These soil layers were all modelled using the MC model as they were considered of less significance in contributing to global ground movement. It is suggested that the large predicted movement may be better modelled if the revetment and the underlying RMS were simulated using the hardening soil model.

Further away from land, the inclinometers I01 and I02 measured the intermediate pile movements (Figure 8). They were both installed within the intermediate piles filled with concrete with toe depths at around -37mCD. The dashed line in Figure 8 indicates the computed movements at depths for the inclinometers over the water. HSSS model once again has overestimated the ground movement at depth by 30 – 80%. This overestimation range for inclinometers is comparable to the overprediction of 25 – 65% for the monitoring pins. Note that the inclinometers are measuring the intermediate pile movements with concrete infills which exhibits a stiffer response than the surrounding soil.

From the discussion above, to refine the modelling, the revetment material and the underlying RMS as shown in Figure 4 could be modelled using the hardening soil model in Plaxis.

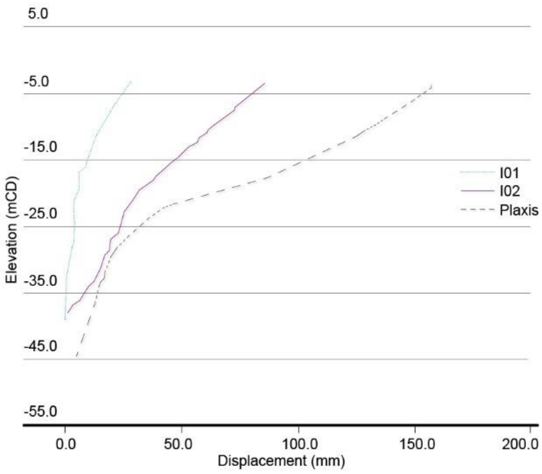

Figure 8. Comparison of Inclinometer Data over the Water with Plaxis Computed Results.

6 SUMMARY AND CONCLUSIONS

This paper has discussed the performance of two soil models available in Plaxis 2D to simulate dredging of soft marine sediments in Lyttelton Port of Christchurch, New Zealand.

It is acknowledged that in any numerical model, the assessed displacements are not only affected by the selected stiffness parameters and adopted correlations but also by other soil parameters, modelling assumptions, applied boundary conditions and phasing. The following conclusions can be drawn:

By means of the Plaxis 2D model, the deformation behaviour as a result of construction and dredging for a new wharf structure was investigated. Two different constitutive models, namely the MC and HSSS model were used in the analysis. To validate the model, the numerical results were compared with measured movements.

When compared to the measured data, this study suggests that the HSSS model, with parameters based on results from common methods of laboratory and in-situ testing is superior in capturing soil displacement behaviour when compared to the MC model.

Stiffness parameters required for a HSSS model can be readily obtained using common laboratory test methods i.e. triaxial and oedometer tests (in this case over 60 years old) when combined with in-situ testing and well-known correlations.

Further, this study suggests that a HSSS model tended to overestimate ground deformations by 25 – 80%. Improved soil sampling for laboratory testing would be expected to improve stiffness value derivations and subsequent predicted ground displacements.

Modelling all soil layers as hardening soil model could also improve the performance of deformation by Plaxis.

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