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Soil-Pile-Structure Interaction at Lyttelton Oil Berth

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ABSTRACT

The Oil Berth at the Lyttelton Port is a critical infrastructure link for the South Island which suffered extensive damage due to the Canterbury Earthquake Sequence of 2010/2011. A replacement Oil Berth is proposed. Design of a new wharf presents three key challenges: (i) keeping the existing wharf operational during construction to maintain a steady supply of oil and petroleum products to the upper South Island; (ii) meeting the seismic design criteria for critical post-disaster infrastructure, and (iii) significant dredging of the berthing pocket to future proof the Oil Berth for larger ships.

A finite element slope stability model was created to assess the stability of the slope and to estimate the bending moments, shear forces and displacements of the wharf piles. A back analysis of the slope was undertaken using surveyed slope movements recorded during the Canterbury Earthquake Sequence to establish baseline seismic soil parameters and validate the model. These parameters were used to complete a seismic slope stability analysis for the proposed breakwater. Soil-pile-structure interaction analysis was undertaken to determine the pinning effects of the wharf piles on the slope. The wharf piles were then analysed to determine the demands imposed on the piles by the expected slope movements. Incorporating the pinning effects of the wharf piles in our analysis increased the calculated seismic factor of safety for the existing breakwater to a level acceptable to the client. This work has resulted in the new berthing line being closer to shore without the need for expensive ground improvement works.

1 INTRODUCTION

The Oil Berth at the Lyttelton Port provides petroleum products to a significant portion of the upper South Island and is a critical infrastructure link. The existing Oil Berth is located on the southern side of Lyttelton harbour close to the harbour entrance.

The site is situated on reclaimed land that overlies harbour deposits consisting of soft to stiff silts with medium dense to very dense sands from approximately 40 m depth and volcanic boulders and rock at approximately 50 m depth. The reclamation process involved construction of a breakwater by end dumping quarried basalt gravel on top of the natural sedimentary deposits. Progressive slope failures were reported during construction. Reclamation of land behind the breakwater was then carried out with hydraulic fill dredged from the harbour bed. The reclaimed land overlies harbour deposits consisting of soft silt, which overlies volcanic rock at depth. The volcanic rock consists of basaltic to trachytic lava flows of the Lyttelton Volcanic Group interbedded with breccias and tuff.

The existing Oil Berth suffered significant damage during the Canterbury Earthquake Sequence. Damage was primarily caused by slope movement of the existing breakwater affecting both shore based infrastructure such as oil pipes and the existing wharf structure. The Oil Berth has been deemed uneconomical to repair and a replacement Oil Berth has been proposed. The existing Oil Berth must be retained in an operational manner until the replacement Oil Berth is able to receive oil shipments to maintain a continuous supply of petroleum products to the upper South Island.

At the time of this analysis the proposed location for the replacement Oil Berth was 100 m west of the existing Oil Berth location shown in Figure 1. The water in this area is shallower than in the existing Oil Berth location. Additionally, the replacement Oil Berth is proposed to have a final depth greater than the existing Oil Berth to allow for larger ships with greater drafts to dock. To deepen the berthing pocket adjacent to the proposed replacement Oil Berth location significant dredging must be undertaken.

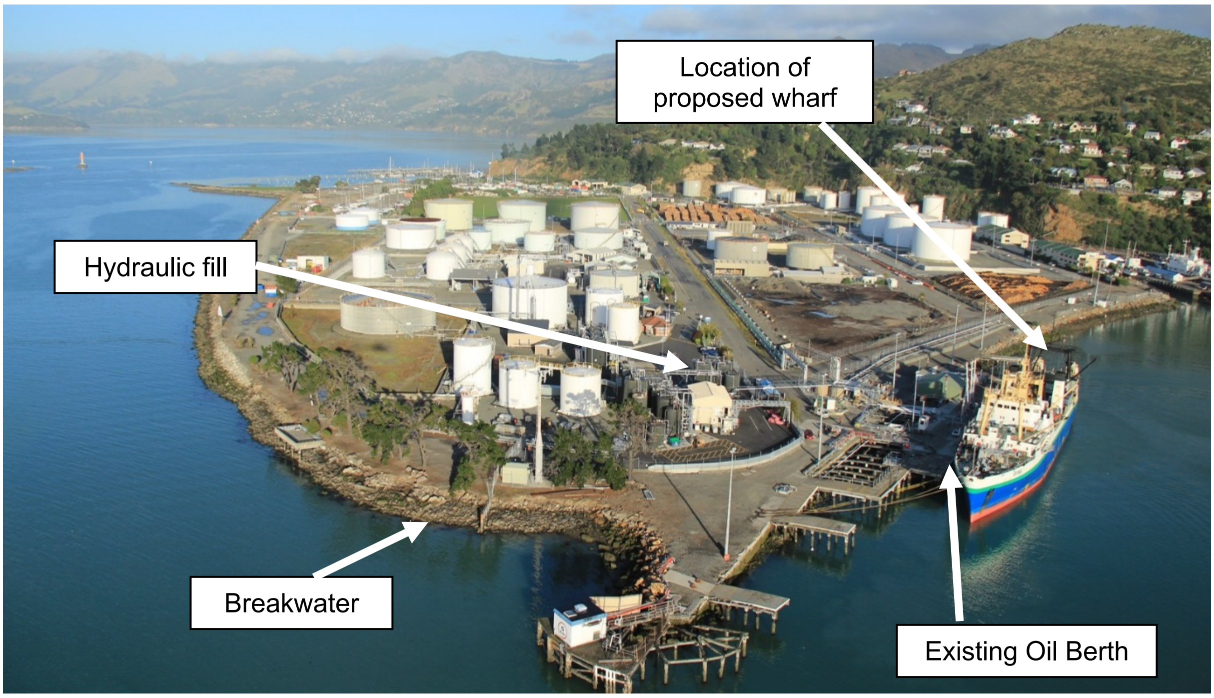

Figure 1. Proposed Oil Berth Location

The replacement Oil Berth and associated breakwater slope must be designed to provide acceptable performance under both static and seismic conditions. Difficulties in achieving acceptable performance include weak natural soils, potentially variable fill material, poor existing slope performance and an increase in slope height through dredging. The berthing line must be kept as close as practical to the shore to reduce congestion within the harbour. Slope stability analysis showed that the static stability and seismic stability of the breakwater are significantly below the level required for critical infrastructure with post- disaster functions.

To tackle this problem we incorporated soil-pile-structure interaction to determine the pinning effects of the wharf piles on the slope and the loads imposed on the piles due to slope movements. This paper presents a summary of the soil-pile-structure analysis.

2 DESIGN METHODOLOGY

2.1 Analysis methods

The analysis was undertaken using the finite element analysis program Plaxis 2D. Factor of safety is calculated in Plaxis using a strength reduction method and seismic yield acceleration was modelled with a pseudo-static acceleration. A Mohr-Coulomb soil model was used within Plaxis.

Seismic slope displacements were estimated from the yield accelerations calculated within the finite element analysis using the method described in Jibson 2007.

Pile demands were calculated based on the soil displacement profiles from the finite element model using the analysis program LPile 2012.

2.2 Design inputs

2.2.1 Performance requirement and design earthquake

Replacement wharf design was undertaken to meet ASCE 61.14. This comprised three design cases Operating level earthquake (OLE), Contingency level earthquake (CLE) and Design earthquake (DE). The seismic performance level considered in this paper is the CLE which requires a similar seismic performance to a Serviceability Limit State (SLS2) earthquake under NZS1170.5 for an Importance Level 4 structure. Following a CLE event the structure must have repairable damage which results in a loss of serviceability for no more than several months.

NZS1170 was used to calculate the design earthquake shaking for a CLE event which results in a horizontal peak ground acceleration (PGA_H) of 0.35g and a moment magnitude (M_w) of 7.5. The target static factor of safety for the slope is 1.5 and the maximum slope displacement is limited by the allowable curvature within the wharf pile elements.

2.2.2 Back analysis of existing slope

The existing breakwater profile was estimated based on the available Lyttelton Port records and the breakwater profile used in the analysis is presented in Figure 2. All levels presented are in metres Chart Datum (mCD).

The yield acceleration of the existing slope was estimated as ky = 0.053g using the slope displacements and strong ground motions recorded during the Canterbury Earthquake Sequence (CES). Soil properties were varied within the finite element analysis until the model matched the estimated yield accelerations. Figure 3 shows a comparison between the measured and modelled displacements during the 22 February 2011 earthquake.

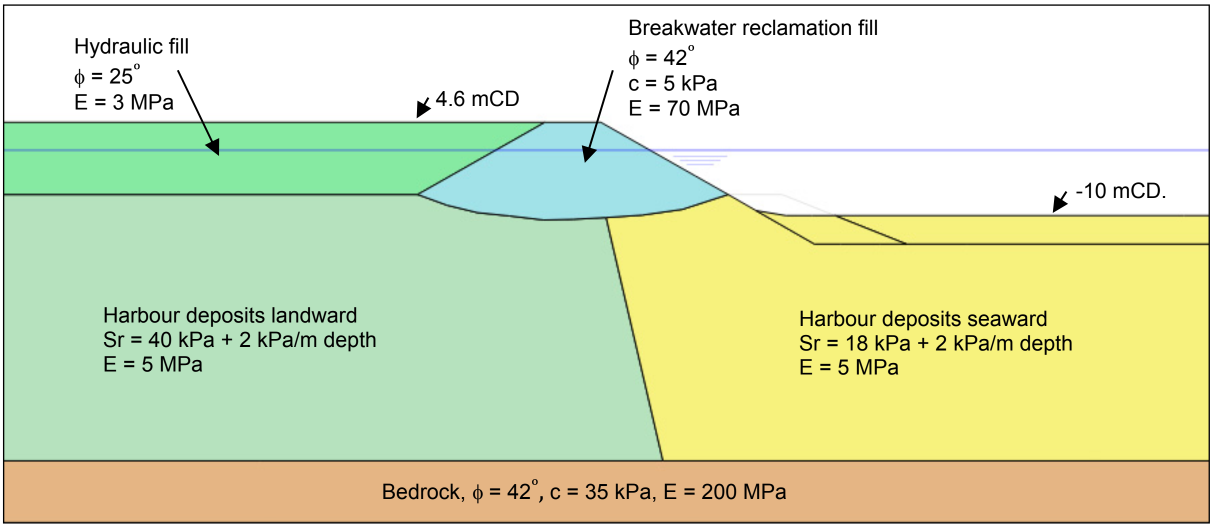

Figure 2. Existing breakwater profile

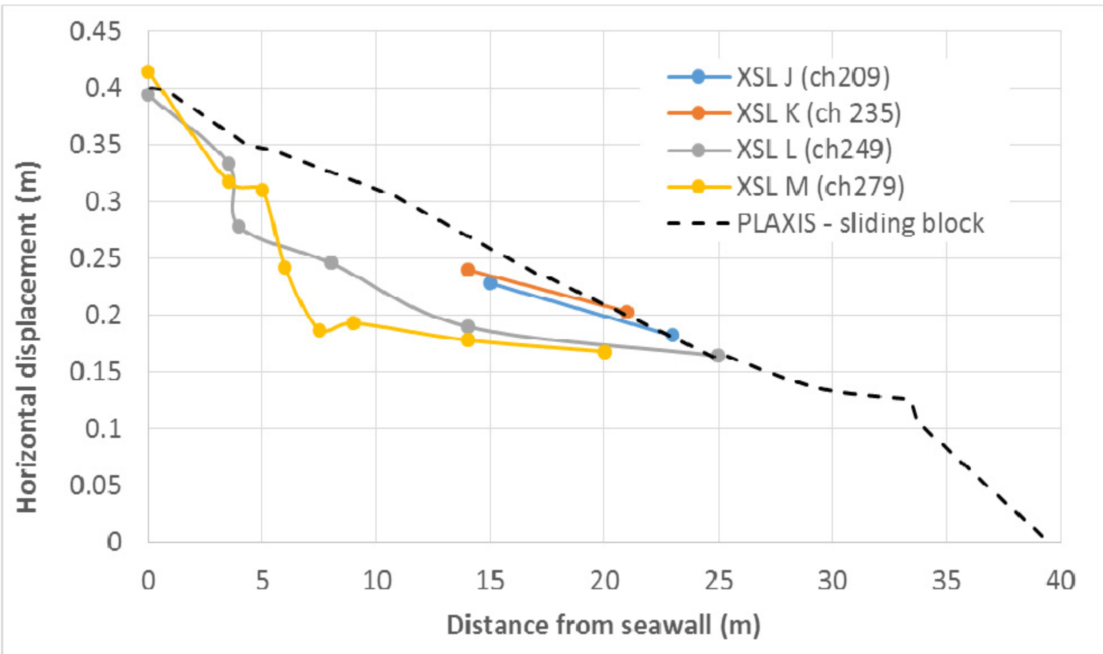

Figure 3. Existing breakwater profile

2.3 Proposed development

2.3.1 Proposed slope profile

The design requires dredging at the harbour to a level of -15.0 mCD. A number of embankment profiles were investigated to improve the slope stability following dredging while minimising the distance between the crest of the seawall and the toe of the embankment. The final embankment configuration consisted of a soil buttress at the toe of the breakwater slope comprising approximately 3 m of fill and 5 m of soil retained from the existing seabed. The soil buttress has an approximate width of 29.0 m.

2.3.2 Proposed wharf pile layout

The oil wharf must be designed to resist vertical loads from the wharf structure and oil distribution infrastructure. Bored concrete piles with permanent steel casings were proposed to transfer vertical loads to rock below and resist lateral loading from docked vessels and seismic slope displacements. The proposed pile layout comprised four rows of 1.2 m diameter piles at a spacing of 6.7 m beneath the approximately 60 m long wharf with a total of 36 piles.

To maintain the berthing line as close as practical to the toe of the embankment the piles were located within the soil buttress. Locating the piles within the slope results in kinematic soil loading on the piles when the slope displaces. The soil loading on the piles is also a load resisting slope movement. The proposed pile layout within the embankment is presented in Figure 4.

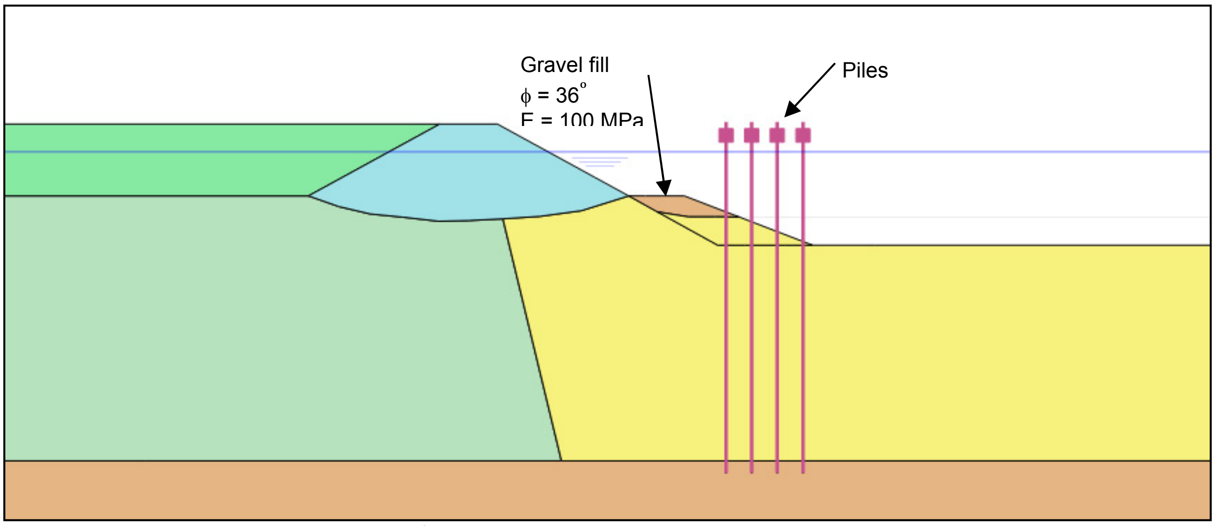

Figure 4. Proposed breakwater profile with piles

3 RESULTS

The predicted failure surfaces of the proposed breakwater profile generated from the finite element model with and without piles effects are presented in Figure 5 and Figure 6. The contours in Figure 5 and Figure 6 show the approximated failure surface only and their magnitudes are not intended to be estimates of seismic slope displacement. Red coloured contours represent larger displacements.

The results of the slope stability analysis include the static factor of safety, slope yield acceleration and predicted displacement under CLE level seismic shaking with 50% and probability of exceedance (POE). The analysis results are presented in Table 1 for the existing breakwater slope, the proposed dredged slope with a fill buttress and the proposed dredged slope including the pinning effects of the piles.

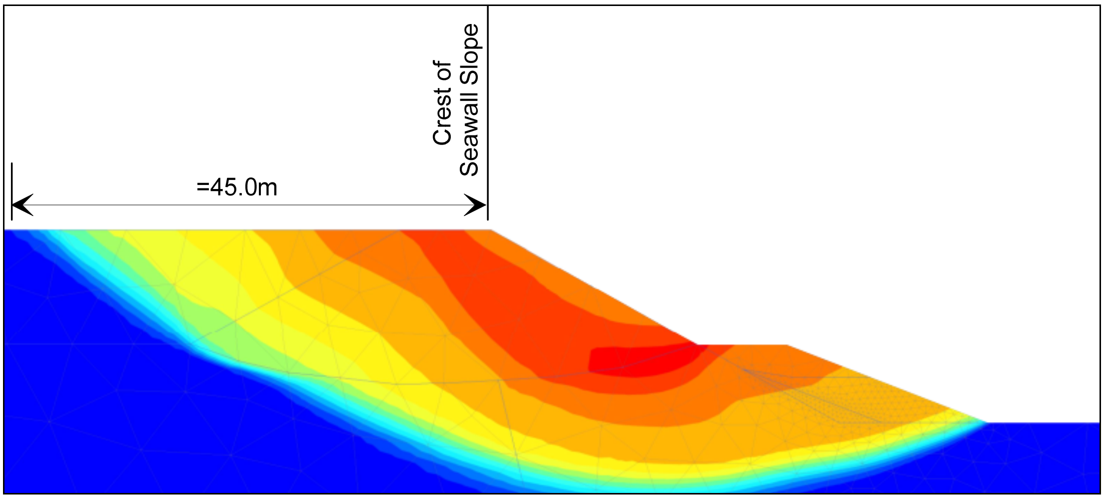

Figure 5. Predicted failure surface without pile effects

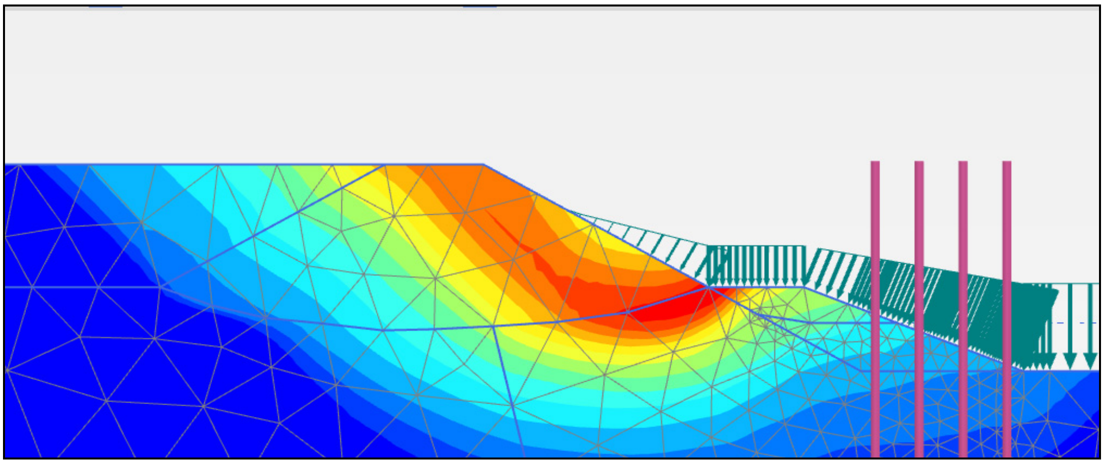

Figure 6. Predicted failure surface including pile effects

Table 1. Summary of existing breakwater analysis results

Design case	**Static factor of safety**	**Yield acceleration**	**Predicted displacement of breakwater under CLE earthquake**
			50% POE
Existing Slope of 1V:1.8H	1.21	0.053g	0.33m
Dredging with fill buttress	1.40	0.068g	0.20m
Dredging with fill buttress incorporating soil-pile-structure interaction	1.5	0.070g	0.19m

4 DISCUSSION

When pile pinning effects are included we can see that the critical failure surface becomes a smaller localised failure. This altered critical failure surface results in smaller predicted slope movements at the pile locations as well as a reduction in the shoreward effect of the slope failure.

Restraining effects of the wharf piles were successfully modelled in Plaxis which showed:

- Increase in slope FOS from 1.4 to 1.5.
- Decrease in slope displacement at the pile location from 0.16 m to 0.11 m.
- Reduction in pile bending moments and curvatures, such that they meet CLE requirements in the ASCE code.

If pile pinning effects had not been considered we would have required:

- Additional piles to meet the ASCE curvature requirements
- Ground improvement at the base of the slope to achieve the static FOS of 1.5 required by the ASCE code.

5 CONCLUSION

This case study presents the successful application of advanced analysis techniques to solve a complex geotechnical problem. Using finite element analysis we can estimate the shape of any slope failure and use this to refine our design.

Consideration of soil structure interaction has allowed us to meet the requirements of the ASCE code without resorting to ground improvement. This has resulted in significant economic benefits to the client.

6 ACKNOWLEDGEMENTS

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