Seismic Design of the Te Bay Reclamation of the Lyttelton Port of Christchurch

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ABSTRACT
Lyttelton Port of Christchurch New Zealand (LPC) was damaged during the Canterbury Earthquake Sequence, mainly during the 22 February 2011 event (M_w=6.2, α_{max}=0.86g, NZS 1170.5:2004 Site Soil Class B), the epicentre of which lies less than 7km from the port facilities. The rebuild is a major program of work that requires extensive repair, restoration and reconfiguration of the port assets. A major part of the reconfiguration is the Te Bay Reclamation of approximately 34ha of new land and a new 700m wharf (400m to 600m from the existing shoreline), to be constructed in two stages. The basis of the concept seismic design of the Te Bay Reclamation Project is the rock response Contingency Level Earthquake spectrum. During the concept design phase, the 22 February 2011 event was selected as a prudent input motion.

The effect of the soil deposit (thickness of natural soil with and without the reclamation fill) on the ground motions at the surface has been analysed with site specific 1–dimensional (1D) soil amplification studies. These included the set-up of the 1D site response analysis using both equivalent linear and nonlinear methods (SHAKE2000 and Deepsoil) and the comparison with the site response predictions of PLAXIS using both the Mohr-Coulomb and the Hardening Soil Small Strain constitutive models to establish a benchmark for the time history analyses. 2D dynamic analyses have also been undertaken to evaluate the earthquake induced displacements using PLAXIS with the Hardening Soil Small Strain constitutive model on the typical N-S cross section through the wharf. These have been used to provide input to the Soil-Foundation-Structure-Interaction design to assess the wharf pile earthquake induced stresses and associated displacements.

1 INTRODUCTION

Lyttelton Port of Christchurch (LPC) is embarking on a long-term plan to develop a modern container terminal at Te Awaparahi Bay (Te Bay). The new terminal will require approximately 34ha of reclaimed land at Te Bay. The proposed reclamation is a key part of the Port's move east as outlined in the Lyttelton Recovery Plan.

10ha of the 34ha of land has already been reclaimed within Te Bay using demolition rubble generated by the 2010/11 Canterbury earthquake sequence (Figure 1).

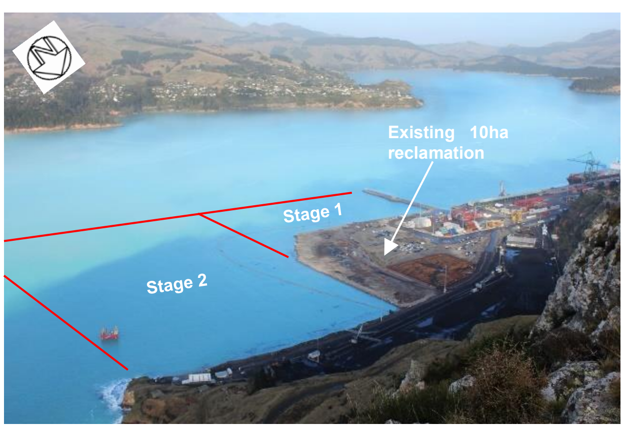

Figure 1: Existing 10ha reclamation

A two-staged wharf construction forms part of the project scope, with each stage being 350m in length. The reclamation stages and areas of significance referred to in this document are shown in Figure 2.

This paper presents the findings during the Concept Design Phase of the project completed in 2015, whilst the Detailed Design phase is due to start in 2017.

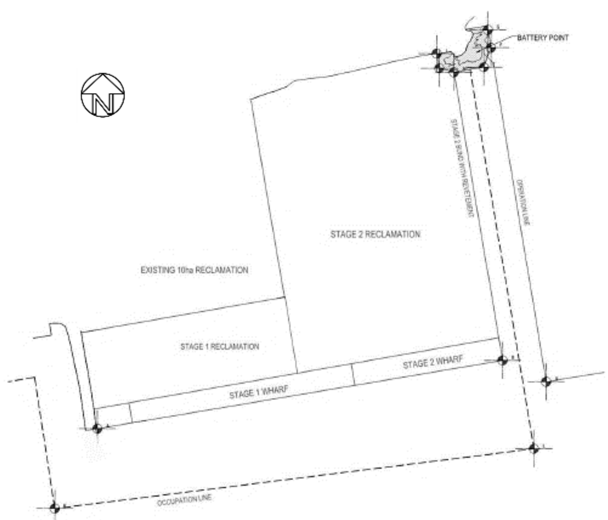

Figure 2: General layout

1.1 Stage 1 Reclamation and Wharf

Stage 1 of the project development comprises:

1. Stage 1 Reclamation (approximately 5ha of operational land) including construction of a bund at the southern edge of the reclamation (Southern Bund) with temporary wave protection (revetment);
2. Stage 1 Wharf, 350m long, with a berth pocket depth of -15.4m CD, and permanent wave protection (revetment).

Because of the anticipated level of settlement of the newly reclaimed land, 3-4 years of consolidation is required prior to commencing construction of the Stage 1 wharf. During this consolidation period, a temporary revetment would be placed to protect the bund from erosion.

This temporary revetment approach will require maintenance over the 3-4 years prior to the construction of the permanent revetment and wharf structure.

The larger rock armour required for the permanent revetment would be placed after piling and before construction of the deck.

This approach also allows for reshaping the Southern Bund, if needed, after initial settlement. The total duration for construction of Stage 1 is estimated at approximately 6 years.

1.2 Stage 2 Reclamation and Wharf

Stage 2 of the project development comprises:

1. Construction of a bund around the perimeter of the entire Stage 2 reclamation, comprising an Eastern Bund and Southern Bund.
2. Temporary wave protection (revetment) on the face of the southern bund to allow piles to be driven during wharf construction.
3. Permanent wave protection (revetment) on the eastern bund.
4. Reclamation of approximately 16ha.
5. Wharf extensions, 290m in length to the east and 60m to the west of the Stage 1 development, with a berth pocket depth of -15.4m CD, and permanent wave protections (revetment) on the southern bund.

Construction of the perimeter bund is proposed to be commenced immediately after completion of the Stage 1 bund.

Following the construction of the bunds, land can be reclaimed within the perimeter bund using a range of hard fill (i.e. rock fill, gravels, or concrete rubble) or dredged material.

The extension of the wharf to the east of Stage 1 would occur 10-15 years post completion of construction of the Stage 2 bunds and once expected and/or acceptable levels of settlement have occurred.

2 SEISMIC PERFORMANCE CRITERIA

The Project Agreement mandated the use of NZS 1170.5:2004, ASCE/COPRI 61-14 & POLA Seismic Code 2010 as the basic earthquake design codes for 100-year design life.

Time history analyses have been undertaken to evaluate the earthquake performance and induced displacements. These have been used to provide input to the Soil-Foundation-Structure-Interaction (SFSI) design to assess pile stresses and displacements.

The Lyttelton 22 February 2011 event has been selected as Contingency Level Earthquake (CLE) from the Canterbury Earthquake Sequence events that have been recorded at the local LPCC (Site Soil Class B) strong motion recording site (Table 1) as a prudent input motion.

Table 1: LPCC (Site Soil Class B) recorded Canterbury Earthquake Sequence events

Event	Magnitude M_w	α_{max} (g)
Lyttelton 22/2/2011 (CLE)	6.2	0.86
Darfield 4/9/2010	7.1	0.24g

3 SITE CONDITIONS

The upper ground profile in the Te Bay area in general consists of fine-grained relatively young marine sediments (silt, clay and interbedded fine sand layers).

To the north of the bay, this relatively young deposit rests on volcanic derived colluvium overlying weathered volcanic rock (typically basalt). To the south, denser and stiffer materials are encountered below the young marine deposits and likely correspond to older marine deposits.

The transition between the colluvium and the old marine deposits is not well known, as only limited investigations have been undertaken to date. Per NZS 1170.5:2004 the site subsoil class is assessed as Class D – deep soil site. The preliminary stratigraphy is shown in the following Figure 3.

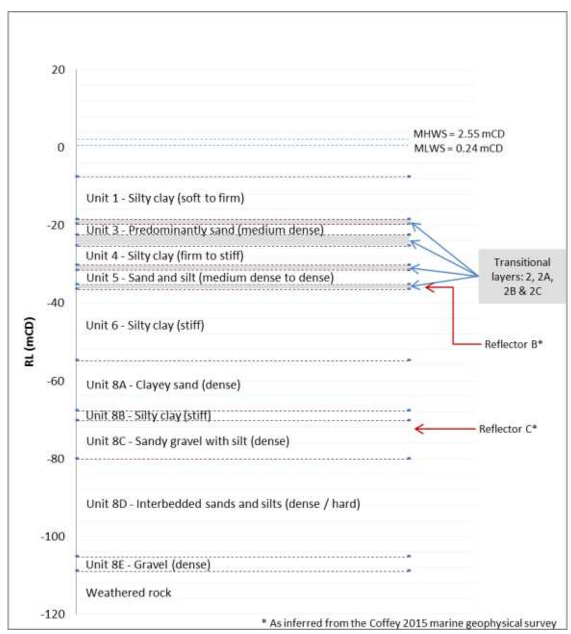

Figure 3: Preliminary in situ stratigraphy

A shear wave velocity (V_s) profile with a 2m interval has been recorded in borehole A01 located at about 50m from the southern end of the breakwater and at ~4.3m RL. The profile was matched to the existing Te Bay geotechnical units and is shown in the following Figure 4.

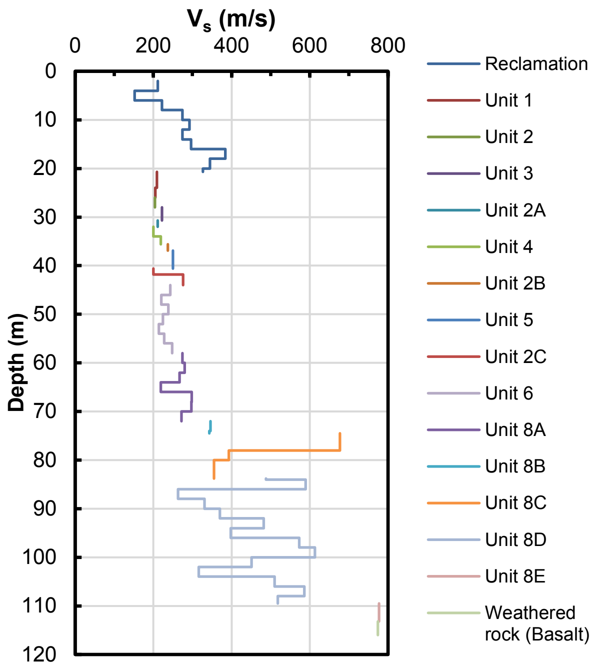

Figure 4: Shear wave velocity profile

The preliminary average material properties are presented in the following Table 2. The shear strength properties were derived after assessing existing site investigating and lab tests. The material descriptions are shown in Figure 3.

Table 2: Summary of preliminary average material properties

Stratigraphy	γ_{sat} (kN/m^3)	c' (kPa)	φ'	s_u (kPa, virgin ground)	V_s (m/s)
Reclamation	22	0	32		266
Unit 1	17.2	3	29	1.72z + 8	200
Unit 2	17.2	3	29	1.72z + 8	200
Unit 3	17.9	0	31		222
Unit 2A	17.2	3	29	1.72z + 8	211
Unit 4	17.2	3	29	1.72z + 8	205
Unit 2B	17.2	3	29	1.72z + 8	237
Unit 5	19	0	30		250
Unit 2C	17.2	3	29	1.72z + 8	238
Unit 6	18	3	29	1.72z + 8	231
Unit 8A	18	1	28		272
Unit 8B	17.5	2	28	150	345
Unit 8C (1&2)	19	0	35		677&374
Unit 8D	19	0	33		369-538
Unit 8E	20	0	38		777
Weathered rock (Basalt)	24	5	42		>800

4 METHODOLOGY

The methodology employed comprises the following steps:

1. The set-up of a 1D soil amplification benchmark through traditional 1D soil amplification analyses, with SHAKE2000 and Deepsoil, using both equivalent linear (EL) and non-linear (NL) methods.
2. The EL and NL results are compared with each other in terms of pseudo-spectral acceleration (PSA) with period, peak horizontal ground acceleration (α_{max}), and maximum shear strain (γ_s) with depth or RL.
3. Run equivalent to 1D, ground response analyses with PLAXIS, using the Mohr-Coulomb (MC) and Hardening Soil Small Strain (HSss) constitutive models, per the methodology guideline "Ground response analysis in PLAXIS (2015)".
4. Comparison of the "1D" PLAXIS results with the 1D benchmark results.
5. The pseudo-spectral acceleration (PSA) with period, peak ground acceleration (α_{max}), and maximum shear strain (γ_s), with depth compared to the corresponding 1D benchmark results are used as criteria for selecting the preferred constitutive model.
6. Selection of the preferred constitutive model to adopt for concept design.

Once the comparisons are made, 2D time history analyses with the preferred constitutive model are run on the critical N-S section of the wharf with revetment to determine the earthquake induced ground displacements at the location of the piles.

The results are used as input to LPILE to assess the preliminary pile bending moments, curvature, shear forces, mobilised bending stiffness, and the permanent deformation. All results (PLAXIS and LPILE) were part of the close collaboration with the Structural Designer for further analysis and optimisation of the structural design.

5 1D BENCHMARK AND COMPARISONS

The 1D Soil Amplification Analysis has been carried out using the equivalent linear (EL) method with the software SHAKE2000 and both the equivalent linear and the non-linear (NL) methods using the software Deepsoil. The Class B earthquake excitation is input at the base of the model at bedrock level.

5.1 Model calibration

For both software, three G/G_0 stiffness degradation curves and damping models have been fitted; using published and widely accepted methods, to the pertinent soil layers, as follows (the pressure values have been selected as averages from the overburden stresses):

1. Set No. 1 – Reclamation and sand soil units
   - G/G_0 Model: Darendeli (2001) - M - OCR 1 - PI 0 - 1.31 atm - 1 Hz - 10 Cycles
   - Damping Model: Darendeli (2001) - M - OCR 1 - PI 0 - 1.31 atm - 1 Hz - 10 Cycles
2. Set No. 2 – Silty clay soil units
   - G/G_0 Model: Darendeli (2001) - M - OCR 1 - PI 15 - 4 atm - 1 Hz - 10 Cycles
   - Damping Model: Darendeli (2001) - M - OCR 1 - PI 15 - 4 atm - 1 Hz - 10 Cycles
3. Set No. 3 – Dense gravel soil unit 8C1
   - G/G_0 Model: Darendeli (2001) - M - OCR 1 - PI 0 - 6.6 atm - 1 Hz - 10 Cycles
   - Damping Model: Darendeli (2001) - M - OCR 1 - PI 0 - 6.6 atm - 1 Hz - 10 Cycles

5.2 Results and Comparison

The computed maximum ground acceleration α_{max} that propagates through the soil column, the maximum shear strain γ_s, and the pseudo-spectral acceleration (PSA), estimated through the 1D equivalent linear and non-linear analyses are presented in the following figures (RL truncated to -75m CD).

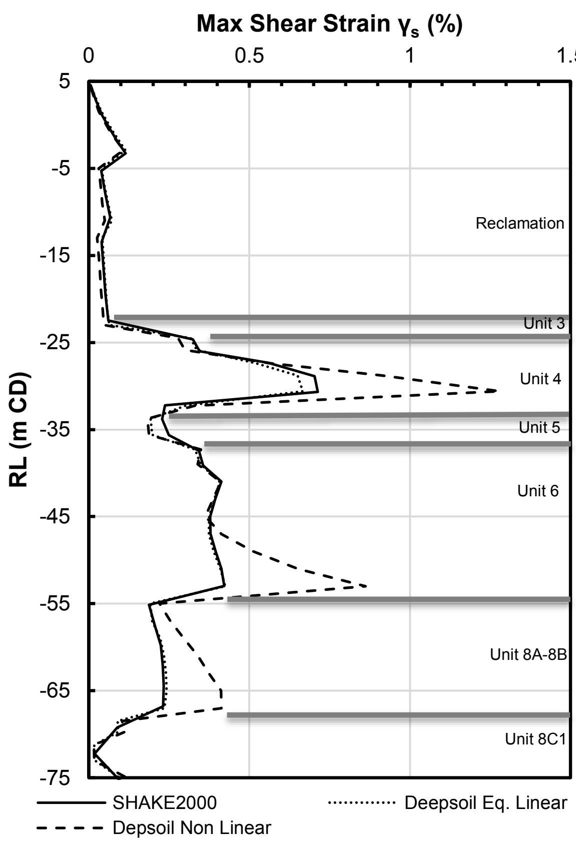

Figure 5: Maximum shear strain γ_s with RL

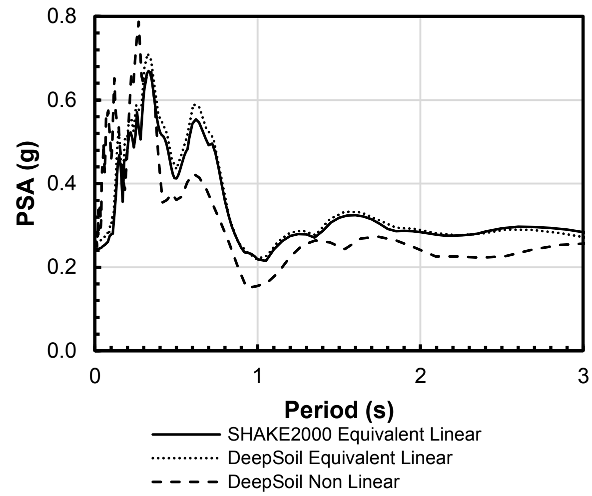

Figure 6: Pseudo-spectral acceleration PSA at ground surface

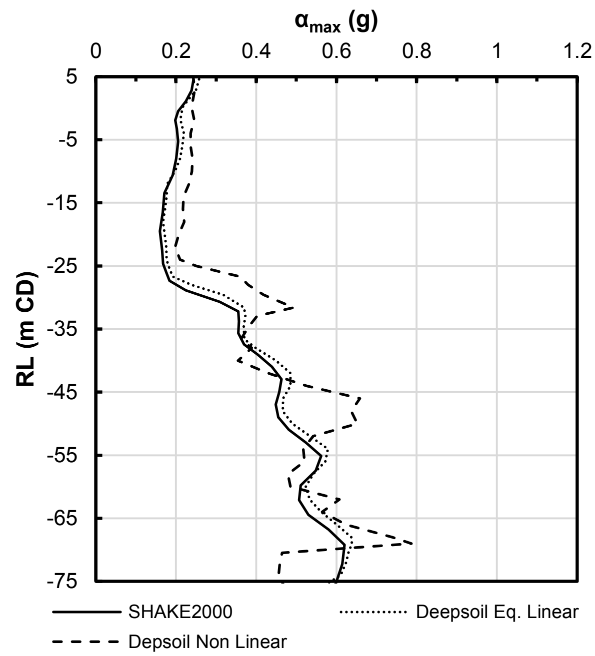

Figure 7: Maximum horizontal ground acceleration α_{max} with RL

5.3 Discussion

By comparing the 1D amplification analyses results, we understand that:

1. The SHAKE2000 and Deepsoil EL α_{max}, γ_s, and PSA results are almost identical to each other.
2. The Deepsoil NL results are close to those of SHAKE2000 and Deepsoil EL.
3. The Deepsoil NL γ_s results show larger strains at the clay layers and especially at the bottom contact of the

clay layers with cohesionless (sand or gravel) denser materials.

The α_{max} at ground surface is approximately 0.25g. It is significant to note that similar performance was recorded during the 22 February 2011 event at the LPOC (Site Soil Class D) strong motion station located at the LPC Oil Berth (0.30g), which lies to the West of the Te Bay site. The LPOC site exhibits similar characteristics of reclamation placed on top of thick soft silty clays.

Consequently, we adopted as a benchmark the SHAKE2000 and Deepsoil EL 1D soil amplification models and results, and we use the Deepsoil NL shear strain distribution as an indication of where to expect PLAXIS to show large strains, especially for the HSss model, which is non-linear.

6 PLAXIS 1D SITE RESPONSE ANALYSES

Two ground models have been set up within PLAXIS using the two most commonly used constitutive models Mohr-Coulomb (MC) and Hardening Soil Small Strain (HSss) – to see which one is a better tool in reliably predicting seismic performance at Te Bay.

For both constitutive models, the loading conditions have been set up as undrained (Undrained B) for the cohesive soil units and as drained for the cohesionless soil units (Reclamation, Unit 3, Unit 5, and Unit 8C1). The s_u profiles have been set up as a future – reclamation loaded condition (60kPa+0.6z) applicable to a certain depth.

6.1 Model calibration

The HSss model requires the following basic input properties for each soil unit:

The initial shear strain G_0, estimated via V_s and γ_{sat}, and the shear strain at 0.722 of G_0 ($\gamma_{0.722}$). The latter is selected from the 1D moduli degradation models (G/G_0 versus γ).

For the MC model, for each soil unit, an equivalent shear modulus is selected such that corresponds to the 65% of the maximum shear strain estimated from the 1D benchmark soil amplification analysis.

For reasons of “computational stability”, some viscous damping is helpful. But this cannot be more than, say, 1% to 5% (Hashash et al, 2010). If higher values are used, then the system is spuriously over-damped. Hence, for both the MC and the HSss constitutive models, additional damping is needed to model realistic damping characteristics of soils in dynamic calculations. This is done by means of Rayleigh damping (C). A reasonable damping range is, as stated previously, ξ=1% to 5%.

Different procedures can be found in literature: Hudson, Idriss & Beikae (1994), Hashash & Park (2010), Kwok et al. (2007), Amorosi, Boldini & Elia (2010). Generally, the two target frequencies are identified through an iterative procedure. Hashash et al (2010) state that it is common practice to choose frequencies that correspond to the first mode of the soil column and a higher mode that corresponds to the predominant frequency of the input motion.

Following this practice, and using SHAKE2000 as a tool to assess the ground profile intrinsic period of vibration, we have:

- Soil column period, T_s=1.07s ➔ f_1=0.9Hz
- Predominant spectral period, T_p:0.085s ➔ f_2=11.76Hz

By rounding, we have:

- Target frequency, f_1=1Hz
- Target frequency, f_2=12Hz

6.2 Results and Comparison

The results from of PLAXIS analyses using the MC and the HSss constitutive models are each compared in turn with the 1D benchmark analyses. The α_{max} that propagates through the soil column, the γ_s, and the PSA estimated with PLAXIS are presented in the following figures (RL truncated to -75m CD) in comparison with the SHAKE2000 and Deepsoil results.

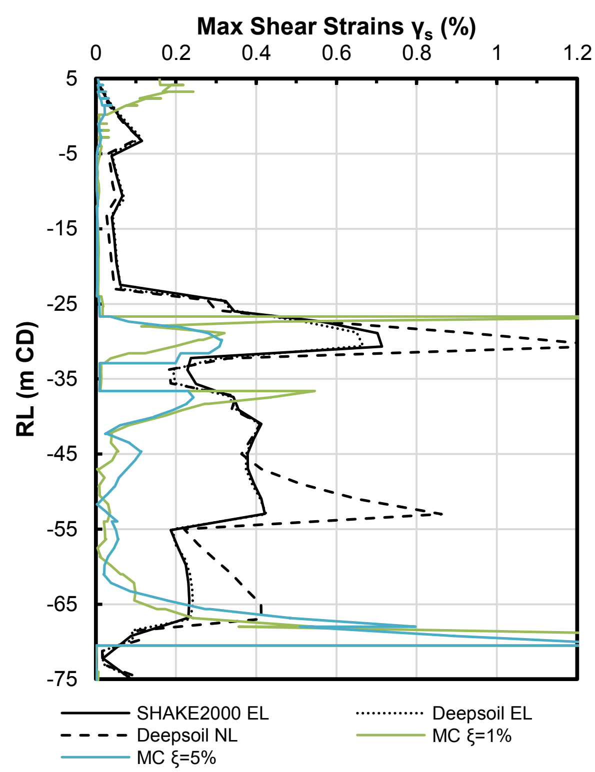

Figure 8: Maximum shear strain γ_s with RL (MC models)

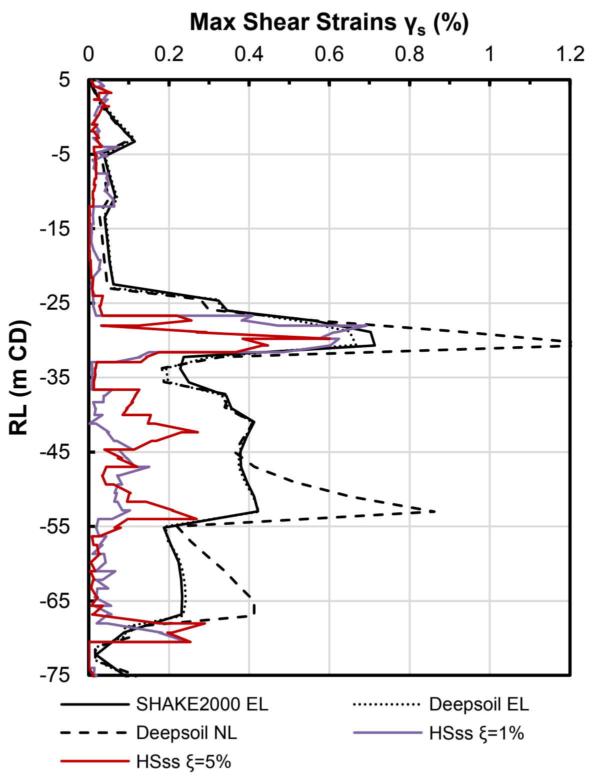

Figure 9: Maximum shear strain γ_s with RL (HSss models)

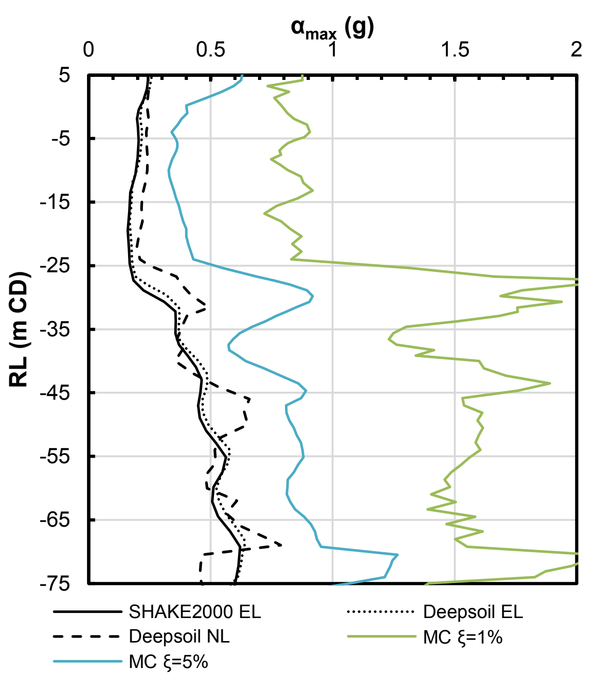

Figure 10: Maximum horizontal ground acceleration α_{max} with RL (MC models)

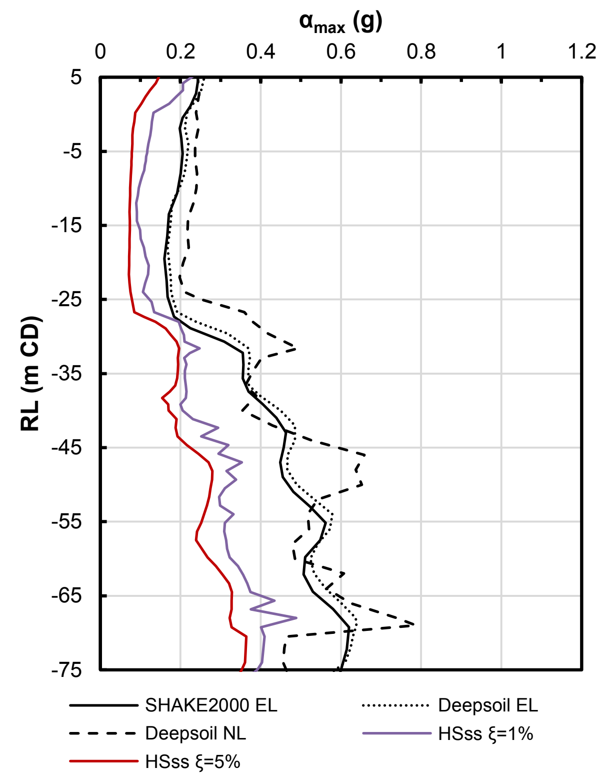

Figure 11: Maximum horizontal ground acceleration α_{max} with RL (HSss models)

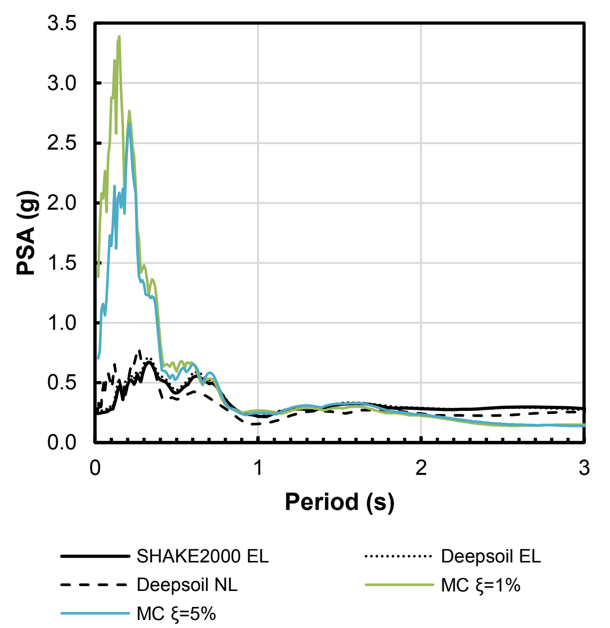

Figure 12: Pseudo-spectral acceleration PSA at ground surface (MC models)

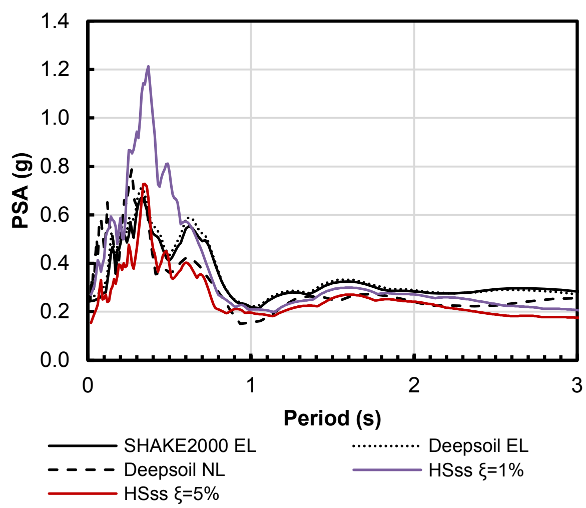

Figure 13: Pseudo-spectral acceleration PSA at ground surface (HSss models)

6.3 Discussion

From the comparison of the PLAXIS 1D results against the 1D benchmark it appears that:

1. Where acceleration predictions are important:
    - Both HSss models appear to under predict by 0.1g in the upper 20m and by 0.2g beyond 20m.
    - The MC with ξ=5% model appears to over predict by 0.25g.
    - The MC with ξ=1% model seems the worst predictor;
2. Where shear strain predictions are important:
    - HSss models appear to be the best predictors.
    - MC models appear to mismatch the ranges where shear strains develop.
3. Where PSA at ground surface predictions are important:
    - Both HSss (ξ=1% & ξ=5%) models appear to follow closely; whilst the HSss ξ=1% appears to over predict for the period range of 0.25s to 0.6s.
    - Both MC models appear to be over predicting for the period range of 0s to 0.4s.

Consequently, the constitutive model that seem to best satisfy all the comparison criteria is the Hardening Soil Small Strain model with both Rayleigh damping options (ξ=1% & ξ=5%).

7 2D TIME HISTORY ANALYSES

A 2D time history analysis has been performed on the critical N-S cross section through the wharf with revetment to assess the performance of the adopted HSss models (two Rayleigh Damping set-ups).

The peak horizontal ground acceleration α_{max}, the maximum shear strains γ_s, and the pseudo-spectral acceleration (PSA) are presented in Section 7.3.

For both models, the conditions and calibration discussed in section 6 have been applied.

The performance of the traditional wharf with revetment under dynamic loading has been assessed using the following loading steps:

1. Initialization of stresses.
2. Emulation of the Stage 1 construction.
3. Application of the working loads and final berth pocket dredging.
4. Excitation using the LPCC N10W 22 February 2011 as input motion to the base of the bedrock layer.

Two models have been run to provide an envelope of anticipated deflections, with Rayleigh damping for ξ=1% and ξ=5%, per Section 6.1.

7.1 Results and Comparison

The deformations per model are presented in the following figures.

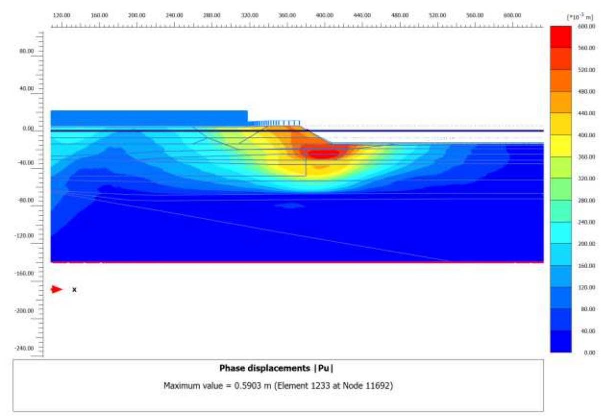

Figure 14: Total permanent displacement for Rayleigh damping with ξ=1%

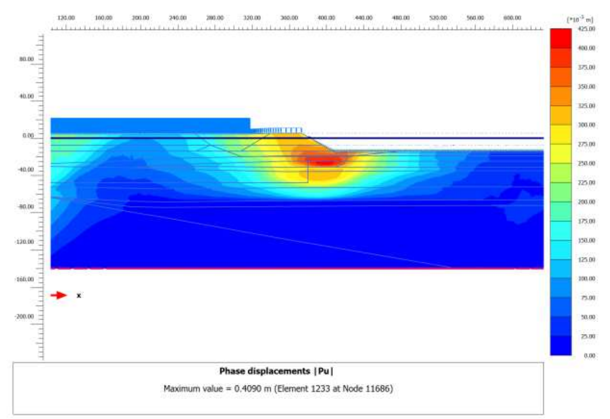

Figure 15: Total permanent displacement for Rayleigh damping with ξ=5%

Corresponding to the location of the crest of the revetment slope the α_{max} that propagates through the soil column, the γ_s, the horizontal displacements, and the PSA, estimated with the HSss models, are presented in the following figures. In addition, an indicative comparison is provided with the 1D SHAKE2000 and Deepsoil α_{max}, γ_s and PSA results.

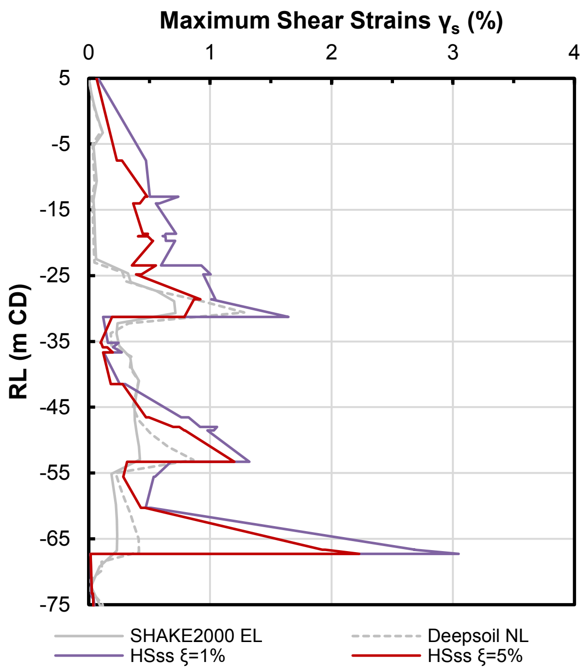

Figure 16: Maximum shear strain γ_s with RL (truncated to -75m CD)

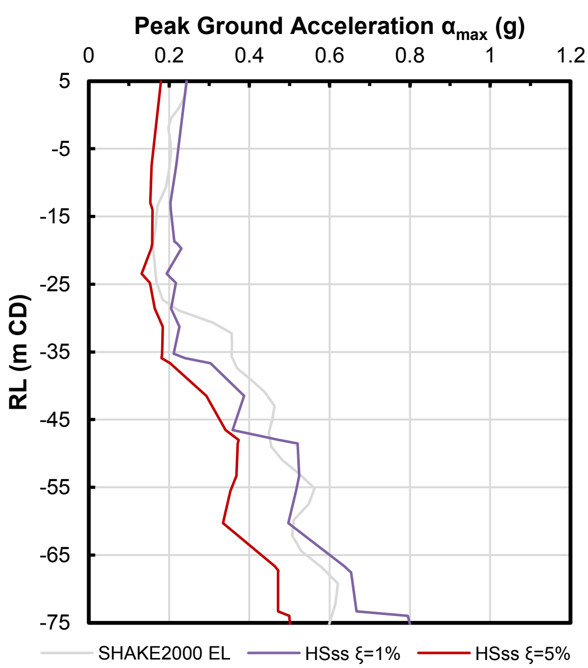

Figure 17: Maximum horizontal ground acceleration α_{max} with RL (truncated to -75m CD)

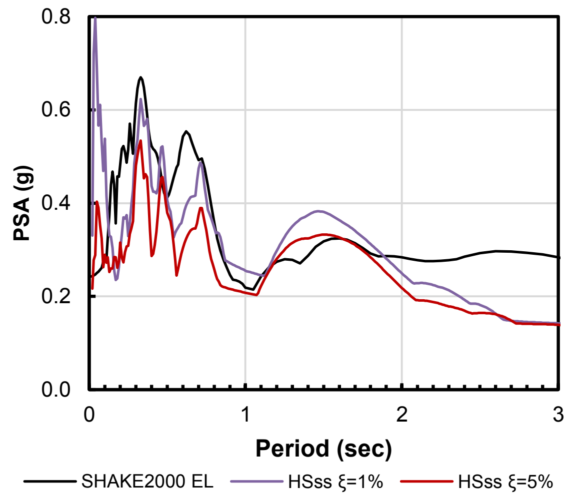

Figure 18: Pseudo-spectral acceleration PSA at ground surface

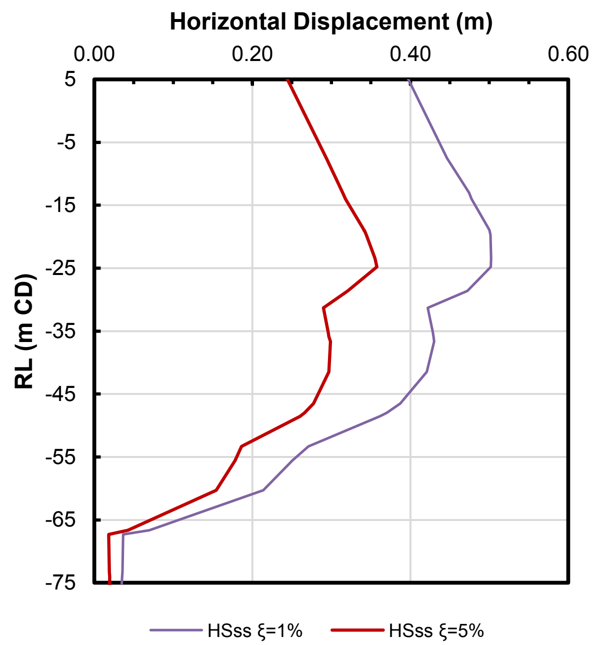

Figure 19: HSss horizontal displacements versus RL (truncated to -75m CD)

7.2 Discussion

The horizontal ground displacements estimated using the two variants of the HSss constitutive model show that a critical area of the revetment is expected to move seawards by a maximum magnitude of 0.35 to 0.50m.

The base of this movement is located at the top of the very dense gravel layer (Unit 8C1) at an RL of approximately -67m CD, where large shear strains develop. Potential shearing of the silty clays on top of the dense gravel may occur.

These horizontal ground displacements pose an additional kinematic horizontal demand on the revetment piles.

From the comparison between the two HSss models and the EL 1D benchmark it appears that:

1. Generally, both HSss models predict shear strains within the same RL range as the SHAKE2000 and Deepsoil results.
2. The HSss model with ξ=1% follows closely the α_{max} propagation results from EL. The HSss model with ξ=5% deviates by approximately 0.1g for RL between -27m CD to -47m CD.
3. A large shear strain is developed at the contact between layers 8AB (silty clays) and 8C1 (dense gravels). This level is the base of the displacements presented in figure 14.
4. Both HSss models generally follow the SHAKE2000 pseudo-spectral acceleration results.
5. The α_{max} at ground surface (~0.20g) shows a very strong de-amplification of the ground motion.

8 CONCLUSIONS

The seismic design of the Te Bay Reclamation during the Concept Design Phase of the project was successfully carried out by establishing a 1D site response analysis benchmark using both the equivalent linear and the non-linear methods to calibrate PLAXIS, which will be the basic analytical tool.

Three comparison criteria have been used to calibrate PLAXIS in both its 1D and 2D implementations. The peak horizontal ground acceleration α_{max}, the maximum shear strains γ_s, and the pseudo-spectral acceleration PSA.

The constitutive model that seems to better satisfy all the comparison criteria is the Hardening Soil Small Strain constitutive model. Therefore, it has been adopted for the seismic design.

The final ground profile, i.e. including the future reclamation, exhibits a very strong de-amplification of the CLE earthquake excitation. The selected time history (22 February 2011 of the Canterbury Earthquake Sequence - Site Soil Class D) applied at the base of the model (at bedrock) has a peak horizontal acceleration of 0.86g. The propagated through the ground profile motion is de-amplified to a peak horizontal ground acceleration of approximately 0.20g to 0.25g.

It is significant to note that similar performance has been recorded during the 22 February 2011 event at the LPOC (Site Soil Class D) strong motion station located at the LPC Oil Berth, which lies to the West of the Te Bay site. The LPOC site exhibits similar characteristics of reclamation placed on top of thick soft silty clays.

The earthquake induced ground displacements that have been estimated through the 2D analysis show a base of movement located at an RL of approximately -65m CD. This is located at the top of the very dense gravel layer (Unit 8C1), where large shear strains develop as well.

The horizontal component of the ground displacements has been used as an additional kinematic horizontal demand on the revetment piles using the LPILE software.

The concept design structural configuration of the piles, considering both the inertial and kinematic demands, is steel tube with reinforced concrete fill at the top, wall thickness 25mm, pile toe at RL -100m CD and diameter DN1500 for the landward piles and DN1200 for the seaward piles.

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