Ground Improvement for Liquefaction Mitigation for Anchored Sheet Pile Wall System: Seismic Soil-Structure Interaction Analysis

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ABSTRACT

The seismic performance of an anchored sheet pile wall system was analyzed using UBCSAND and PM4SAND soil constitutive models for liquefiable soils. Model calibration was performed to determine input parameters for the seismic analysis. The calibration considered some important aspects that significantly affect the results of the numerical simulation, including cyclic resistance ratio, the development of the excess pore water pressure, the overburden effect, the static shear stress effect, and the modulus reduction and damping. Ground improvement methods including stone columns and earthquake drains to reduce the soil liquefaction and improve the seismic performance of the sheet pile wall system were analyzed. Grounds improved with stone columns and earthquake drains were modeled considering equivalent soil density and permeability. The analysis results indicate that soil liquefaction and seismic responses of the sheet pile wall system estimated using PM4SAND model agree reasonably with the those estimated using UBCSAND model. The ground improvement can be reasonably modeled to capture important seismic ground responses.

1 INTRODUCTION

Liquefaction of soil presents a significant seismic hazard to waterfront structures. Finite element or finite difference analyses which consider the development, distribution and dissipation of pore water pressure are becoming routine tools for modelling soil liquefaction during earthquakes. UBCSAND (Beaty and Byrne, 2011), PM4SAND (Boulanger and Ziotopoulou, 2017), Pressure Dependent Multi Yield 02 (Yang et al., 2008), and SANISAND (Dafalias and Manzari, 2004; Taiebat and Dafalias, 2008) are several constitutive models that have been developed for liquefaction simulation.

UBCSAND is a two-dimensional effective stress plasticity model that predicts the shear stress-strain behavior of the soil using an assumed hyperbolic relationship and the build-up excess pore water pressure during cyclic loading (Beaty and Byrne, 2011). PM4SAND is a critical state compatible, stress-ratio controlled, bounding surface plasticity model, developed to approximate the range of behaviors important to geotechnical earthquake engineering practice (Boulanger and Ziotopoulou, 2017).

Analyses using both models and comparison of the results could provide insight into the model responses and could suggest the approximate range of simulation results. Poor agreement between results obtained using two different models can identify issues with the simulation and lead to subsequent improvements.

In this study, seismic soil-structure interaction analyses were performed to evaluate seismic performance of an anchored sheet pile wall system. Sandy soils were modeled using UBCSAND and PM4SAND soil constitutive models to analyze the development of excess pore water pressure. Ground improvement options including stone columns and earthquake drains were analyzed to improve the seismic performance of the sheet pile wall system.

2 SUBSURFACE CONDITIONS

The soil profiles used for the study were adapted from Ohama No. 2 Wharf at Akita Port in Japan (Figure 1). The wharf included a sheet pile wall with a total height of about 22.5 m and the top of the wall is at approximately El. + 2.0 m. The sheet pile wall is connected to an anchor with high strength steel tie rods. The tie rods are installed near the top of the wall at a horizontal spacing of 2 m. The anchor is located 20 m behind the sheet pile wall and consists of about 15 m long steel pipe piles with a pile cap.

The backfill soils behind the wall typically consist of sand with thicknesses varying from approximately 12 m right behind the sheet pile wall to 8 m at the anchor wall location. The backfill sand was estimated to be loose with a fines-corrected $(N_1)_{60cs}$ value of 9.

The native soil deposits generally consist of dense sand with an estimated $(N_1)_{60cs}$ of 40 overlying a 2.5 m thick clay deposit, which is underlain by compact to dense silty sand to sands with an estimated $(N_1)_{60cs}$ in the range of 25 to 30.

The mean water level is located at approximately 2 m below top of the sheet pile wall, at an elevation of zero. The details of the Akita Port Wharf wall and ground conditions were used for a parametric study.

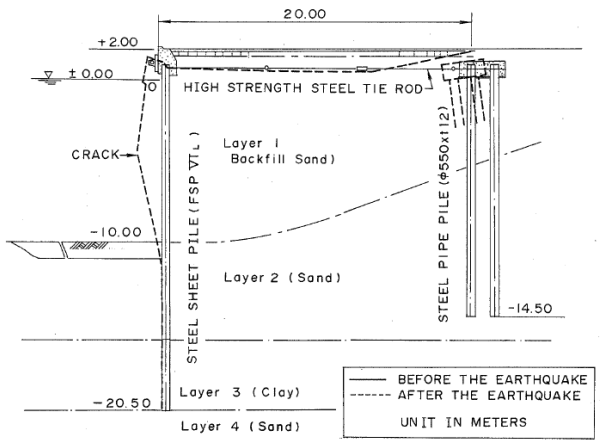

Figure 1. A cross section at Ohama No. 2 Wharf - Akita Port, Japan (Iai and Kameoka, 1993)

3 EARTHQUAKE MOTIONS

Dynamic analyses of the sheet pile wall-soil system were carried out using earthquake motions from the Chi Chi (Taiwan, 1999), Loma Prieta (Northern California, 1989), and Landers (California, 1992) earthquakes. The design motions were defined as bedrock outcrop motions. Response spectrum matching and baseline correction of the earthquake motions were performed before their use in the dynamic analyses. The Response Spectra of the earthquake motions, obtained for 5% damping are presented in Figure 2. The magnitude of the earthquakes and the duration of the modified motions are summarized in Table 1.

Table 1. Earthquake motions

Earthquake	Moment Magnitude	Duration (s)
Chi Chi	7.6	60
Loma Prieta	6.9	40
Landers	7.3	45

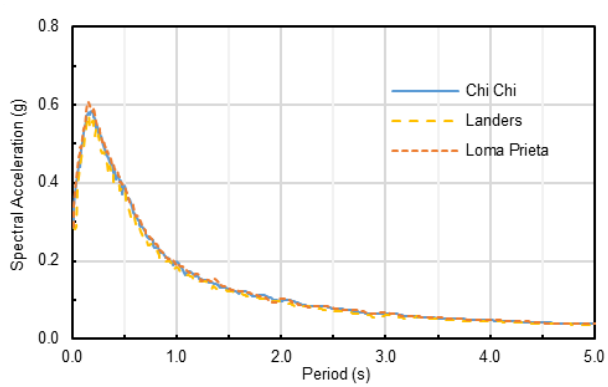

Figure 2. Response spectra of the spectrally matched earthquake motions

4 CONSTITUTIVE MODEL CALIBRATION

The calibration of the UBCSAND (version 904aR, 2011) and PM4SAND (version 3.0, 2017) models considered some important aspects that significantly affect the results of the numerical simulation. The calibration was based on results of single element simulations of cyclic direct simple shear (cDSS) test with stress-controlled loading.

The input parameters were calibrated to give reasonable agreement between simulation results and empirical correlations, including the cyclic resistance ratio (CRR) with (N1)60cs, the development of the excess pore water pressure, the overburden effect (K□), the static shear stress effect (K□), and the modulus reduction and damping.

Generic input parameters for both UBCSAND and PM4SAND include relative density, elastic shear modulus and friction angle.

The soil relative density (D_r) was determined from SPT $(N_1)_{60}$:

$$D_r = \sqrt{\frac{(N_1)_{60}}{46}} \quad [1]$$

The elastic shear modulus was determined as a function of $(N_1)_{60}$:

$$G = G_o p_a \left(\frac{p'}{p_a}\right)^{0.5} \quad [2]$$

$$G_o = 21.7 * 20 * (N_1)_{60}{}^{0.333} \quad [3]$$

Where p_a is the atmospheric pressure and p' is the mean effective stress.

The constant volume friction angle ϕ'_{cv} of 33 degrees and K_o (the ratio of horizontal effective stress to vertical effective stress at the start of loading) of 0.5 were used for both UBCSAND and PM4SAND.

Liquefaction triggering was defined as the development of 70% excess pore water pressure ratio R_u (i.e., ratio between excess pore pressure and the initial effective overburden stress) or the development of 3.75% shear strain.

4.1 UBCSAND calibration

The parameter m_hfac1 in the UBCSAND formulation is used to adjust the plastic shear modulus with confining stress. Calibration of liquefaction triggering and K_σ effect can be performed using the fitting parameter m_hfac1.

The parameter m_hfac1 was defined by Beaty and Byrne (2011) as a function of $(N_1)_{60}$ and the initial effective stress (σ'_v), shown in equation 4.

$$m\_hfac1 = a_N \left(\frac{\sigma'_v}{p_a}\right)^{b_N} \quad [4]$$

where a_N and b_N are functions of $(N_1)_{60}$.

The parameter m_urstif used to adjust the plastic shear modulus is a function of the relative change in stress ratio and the loading history. The default formulations of m_urstif were used in this study.

4.2 PM4SAND calibration

The contraction rate parameter h_{po} is a primary input parameter in the PM4SAND formulation used to adjust the plastic shear modulus to elastic modulus. This parameter was defined as a function of $(N_1)_{60}$ and σ'_v, similar to Eq. [4] to calibrate the liquefaction triggering and K_σ effect.

The PM4SAND model consists of 21 secondary parameters. C_{kaf} and h_o are the two secondary parameters that need modifications during model calibration. Default values were set for other secondary parameters.

The parameter C_{kaf} controls the effect of static shear stresses on plastic modulus. C_{kaf} was set equal to 2 in this study based on the static shear bias calibration.

The parameter h_o adjusts the plastic modulus to elastic modulus. A h_o value of 1.2 was set to provide reasonable modulus reduction and damping relationships.

4.3 Calibration results

The calibration results are presented in Figures 3 through 9. The cyclic stress ratios (CSR) causing liquefaction in 15 uniform cycles (Mw = 7.5, $\sigma'_v = 100\ kPa$) versus $(N_1)_{60cs}$ are shown in Figure 3. The CSR values estimated by both UBCSAND and PM4SAND agree closely with the liquefaction triggering curve proposed by Idriss and Boulanger (2008).

The number of uniform cycles to liquefaction at different CSR values are presented in Figure 4. The general trends estimated by both UBCSAND and PM4SAND are consistent with those shown in Idriss and Boulanger (2008).

The effect of confining stress on liquefaction resistance is presented through the K_σ factor ($K_\sigma = CRR_{\sigma'v}/CRR_{\sigma'v = 1atm}$) and shown in Figure 5. The K_σ versus σ'_v/p_a relationships estimated by both UBCSAND and PM4SAND agree closely with the correlations proposed by Idriss and Boulanger (2008).

The effect of initial static shear bias α is presented through the K_α factor ($K_\alpha = CRR_\alpha/CRR_{\alpha=0}$) and shown in Figure 6. The general trends estimated by both UBCSAND and PM4SAND are consistent with the relationship proposed by Idriss and Boulanger (2008).

The development of excess pore water pressure during shaking is shown in Figure 7. The excess pore water pressure ratio R_u was estimated for an $(N_1)_{60cs}$ value of 10 and an CSR of 0.12 without static bias. Both UBCSAND and PM4SAND estimated soil liquefaction in 15 uniform cycles.

The calibrated modulus reduction (G/Gmax) and damping curves are shown in Figures 8 and 9. The reduction in shear stiffness with strain estimated by UBCSAND is close to the upper bound of the modulus reduction curve reported by Seed and Idriss (1970). The shear modulus reduction curve estimated by PM4SAND is close to the average modulus reduction curve reported by Seed and Idriss (1970).

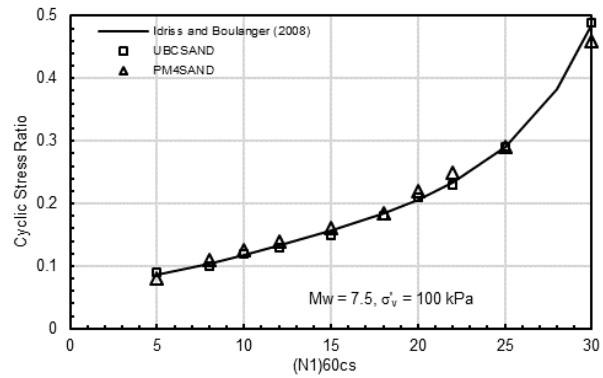

Figure 3. Liquefaction triggering curve

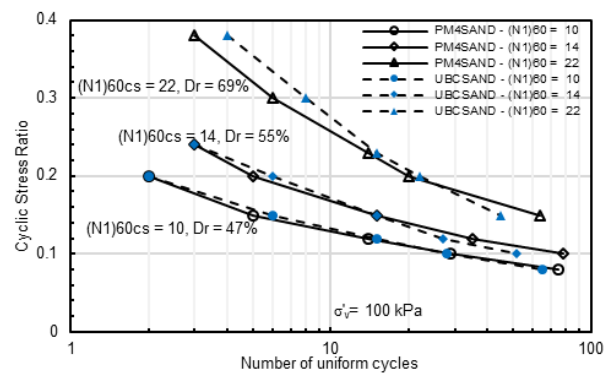

Figure 4. Number of cycles to trigger liquefaction

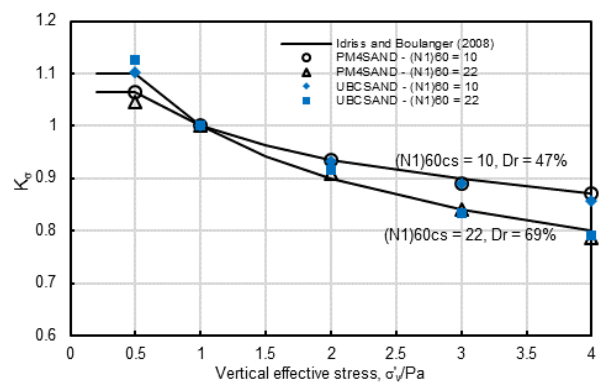

Figure 5. K_σ effect

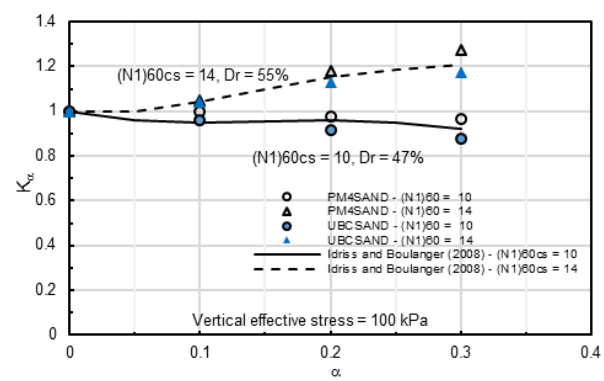

Figure 6. K_α effect

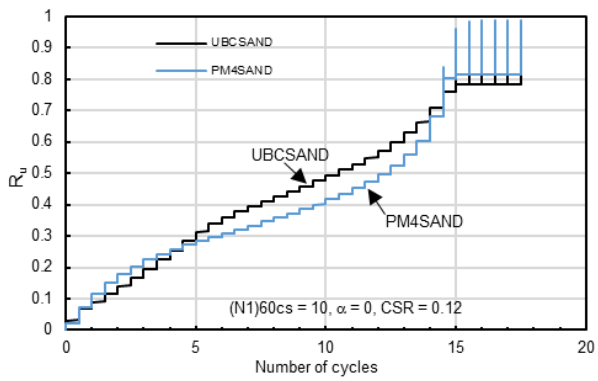

Figure 7. Development of excess pore pressure ratio R_u

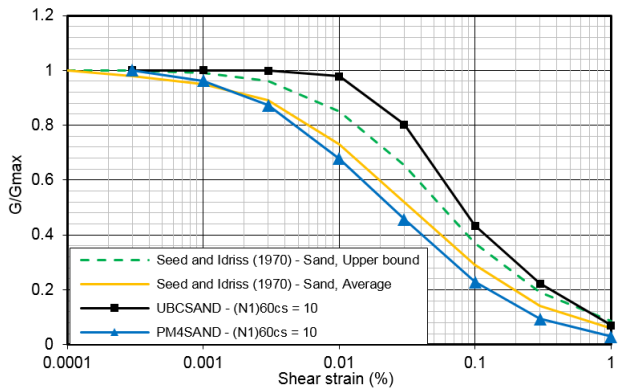

Figure 8. Modulus reduction

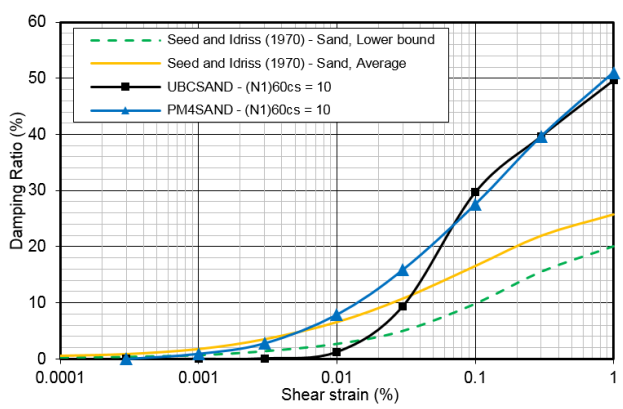

Figure 9. Damping ratio

5 SEISMIC SOIL-STRUCTURE INTERACTION ANALYSIS

5.1 Analysis stages

Coupled soil-structure interaction analyses were carried out using computer program FLAC 2D v.7 (Itasca, 2011).

Two stages were simulated in the analyses of the anchored sheet pile wall system:

- Stage 1 – Static analysis for pre-earthquake conditions;
- Stage 2 – Dynamic analysis with earthquake motions

The dynamic analysis was performed in time domain for the full duration of the input motions and then continued for an additional 5 seconds of dynamic time without any motions. This additional time was to allow for continued decay of the dynamic response. Post-earthquake analysis was not considered in this study.

The analyses were performed using UBCSAND (Case 1) and PM4SAND (Case2) to model liquefiable soils.

5.2 Boundary conditions

The lateral boundaries of the FLAC model were extended at both sides of the model to minimize boundary effects. The lower model boundary was extended to incorporate 10 m thickness of model base.

Free field and compliant base conditions were applied to the lateral boundaries and model base, respectively.

Horizontal component of the design earthquake motions was applied to the base of the model as shear stress time histories.

5.3 Soil Constitutive models and parameters

The sandy soils were modeled using Mohr-Coulomb model in Stage 1. In stage 2, UBCSAND (Case 1) and PM4SAND (Case 2) models were used for sandy soils to simulate the development of excess pore water pressure.

The clay layer was modeled using Mohr-Coulomb model in Stage 1. In stage 2 during shaking, the “Default” hysteretic model available in FLAC was used to allow for modulus reduction and damping of the clay. The input parameters of the model were calibrated with the modulus reduction and damping curves for clay proposed by Sun et al. (1988).

The FLAC model and $(N1)_{60cs}$ values used for the FLAC analyses are shown in Figures 10 and 11, respectively. The clay layer and the bottom soil layer do not require $(N1)_{60cs}$ as input for the models used. Therefore, the $(N1)_{60cs}$ for these two layers are shown a value zero in the graphical representation in Figure 11.

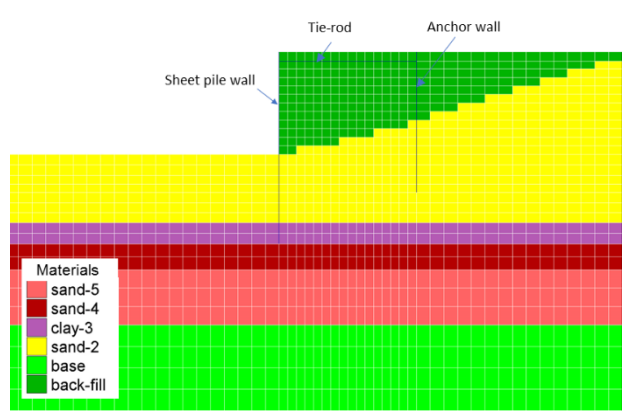

Figure 10. FLAC model

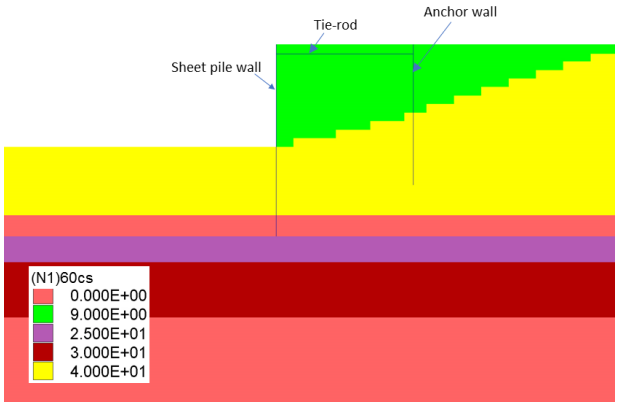

Figure 11. $(N_1)_{60cs}$ values used for the FLAC analyses

5.4 Anchored sheet pile wall

The sheet pile wall and anchor wall were modeled using beam elements. The soil grid interacts with the beam elements through interface springs attached to nodal points. An interface friction angle of 17 degrees was used to simulate the shear interaction between the beam elements and soils.

The tie-rod was modeled using cable elements, which only carry tension forces.

The sheet pile wall, anchor wall and tie-rod were modeled as elastic structures with a Young's modulus of 210 GPa. The moment of inertia of the sheet pile and anchor walls are 86,000 cm^4/m and 73,000 cm^4/m, respectively.

Hydrodynamic pressure acting on the sheet pile wall during seismic shaking was accounted for by adjusting the grid point mass on the sheet pile wall face using the formulations proposed by Westergaard (1933).

5.5 FLAC analysis results

The maximum excess pore pressure ratio R_u and soil horizontal displacement contours at the end of the analysis using UBCSAND model and Loma Prieta earthquake motion are shown in Figures 12 and 13, respectively. Results from the same earthquake motion using PM4SAND model are shown in Figures 14 and 15.

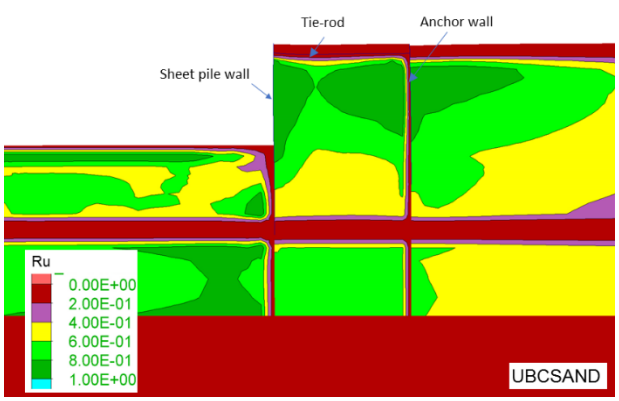

Figure 12. Excess pore pressure ratio R_u (UBCSAND)

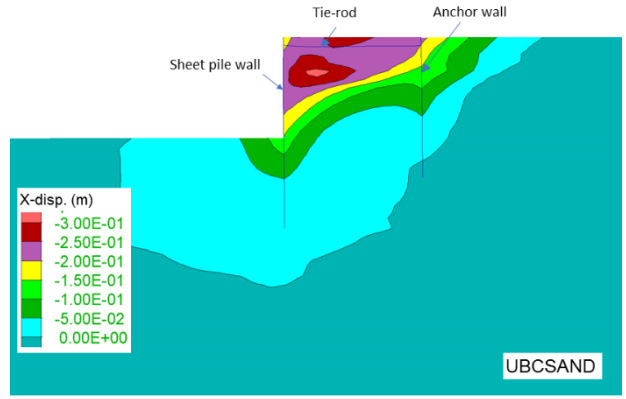

Figure 13. Horizontal displacement (UBCSAND)

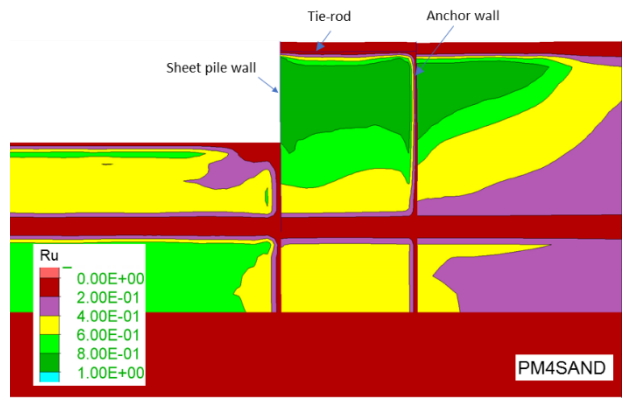

Figure 14. Excess pore pressure ratio R_u (PM4SAND)

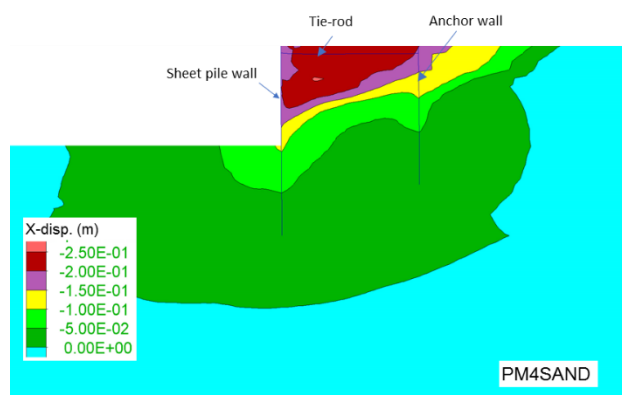

Figure 15. Horizontal displacement (PM4SAND)

The results from both cases indicate soil liquefaction (R_u > 70%) occurred in the backfill soils. Similar patterns for soil displacement were observed between UBCSAND and PM4SAND. However, the UBCSAND model predicted more liquefaction in front of the wall within the bottom silty sand layer (below the 2.5 m thick clay layer) than that from the PM4SAND model.

The bending moment and lateral displacement profiles of the sheet pile wall using UBCSAND are presented in Figures 16 and 17, respectively. The wall responses using

PM4SAND are presented in Figures 18 and 19. For both cases, the maximum bending moments and lateral displacements were estimated to occur at approximately 7 m and 6 m below the top of the sheet pile wall, respectively.

The UBCSAND model estimated the sheet pile wall largest bending moment of about 1000 kNm/m and largest lateral displacement of about 0.23 m. The PM4SAND model estimated the wall largest bending moment of about 750 kNm/m and largest lateral displacement of about 0.2 m. For both cases, the largest bending moment and displacement occurred with the Loma Prieta earthquake motion. The PM4SAND model generally estimated smaller bending moments and displacements than those estimated using the UBCSAND model.

In general, both PM4SAND and UBCSAND models provide reasonable agreement regarding soil liquefaction and deformation patterns. Using both models can suggest an approximate range of sheet pile wall largest bending moments from 750 kNm/m to 1000 kNm/m and largest lateral displacements from 0.2 m to 0.23 m.

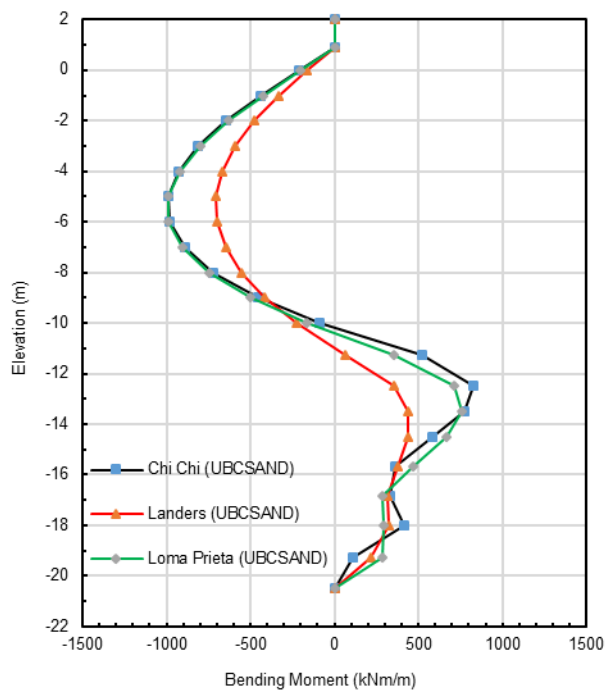

Figure 16. Bending moments of sheet pile wall (UBCSAND)

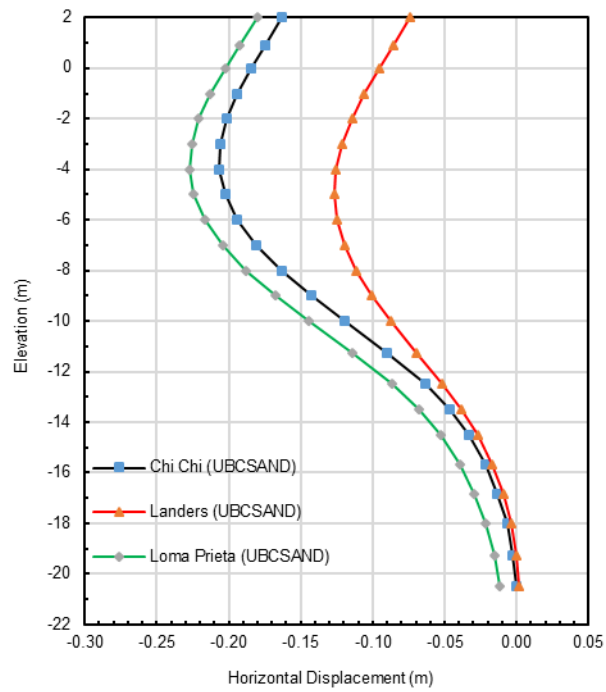

Figure 17. Lateral displacements of sheet pile wall (UBCSAND)

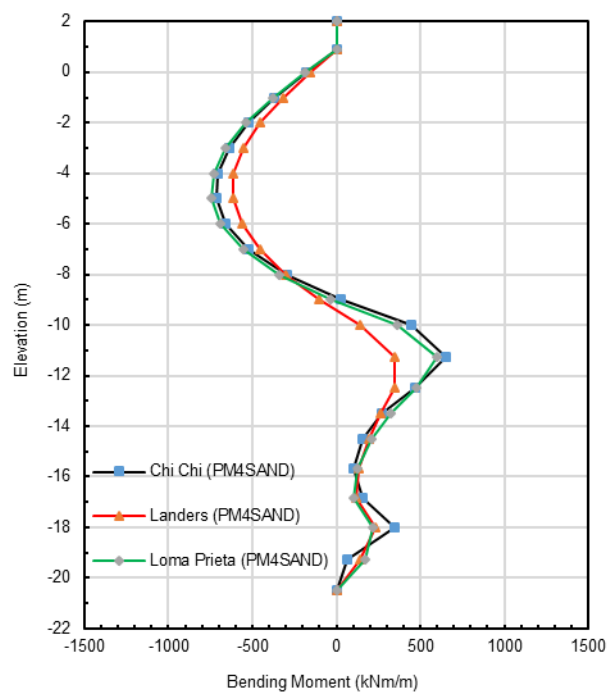

Figure 18. Bending moments of sheet pile wall (PM4SAND)

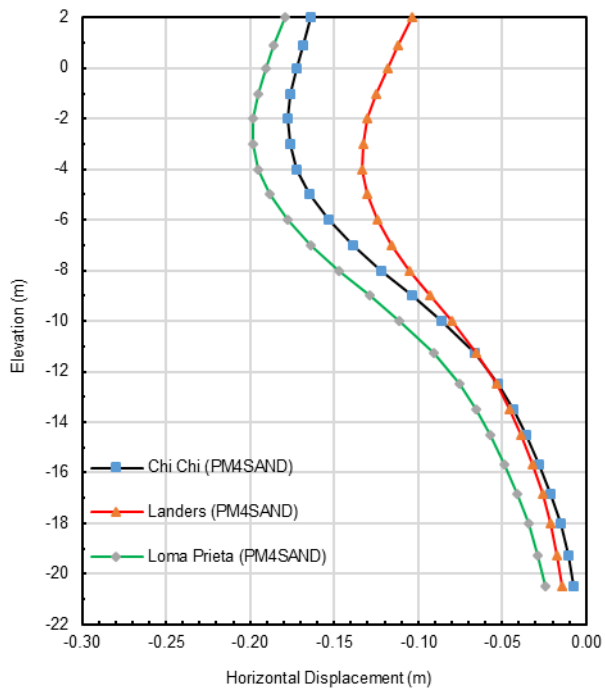

Figure 19. Lateral displacements of sheet pile wall (PM4SAND)

6 GROUND IMPROVEMENTS

6.1 Ground improvement modeling

Ground improvement options including stone columns and earthquake drains were analyzed to improve the seismic performance of the sheet pile wall system. Stone columns densify the soil and increase drainage somewhat, while earthquake drains typically increase drainage by carrying very large flow volumes sufficient to dissipate the water pressure in sands during earthquake. Post-earthquake ground settlement would still be expected if only drains were used. In this study, ground improvement was carried out for backfill soils behind the sheet pile wall and extended 5 m outside the anchor wall to the entire depth of the backfill soils.

Stone columns were not modeled explicitly in the FLAC analysis. An $(N_1)_{60cs}$ of 22 was used to account for the increase in soil density. Increase in permeability was also considered for the ground improvement zone.

The earthquake drains were first analyzed using the program FEQDrain developed by Pestana et al. (1997). The program analyzes the three-dimensional pore pressure generation and dissipation with earthquake drains. The analysis estimated the excess pore pressure ratio Ru for 1.2 m equilateral triangular grid spacing to be approximately 40%. The effect of the earthquake drains in FLAC was accounted for by using permeability values which would result in similar excess pore pressure ratios computed from FEQDrain.

The permeabilities of materials used in the FLAC analyses are shown in Table 2.

Table 2. Soil permeabilities used in the FLAC analyses

Material	Horizontal permeability (m/s)	Vertical permeability (m/s)
Backfill soils	1e-4	5e-5
Native sandy soils	1e-4	5e-5
Soils improved by earthquake drains	1e-3	1e-2
Soils improved by stone columns	1e-4	5e-3

6.2 Ground improvement – analysis results

The efficiency of ground improvement was analyzed using PM4SAND model and Loma Prieta earthquake motion. The excess pore pressure ratio R_u profiles at the end of the analysis are shown in Figures 20 and 21, for ground improved using stone columns and earthquake drains respectively. The analysis results indicated that no liquefaction occurred within the ground improvement zone. The bending moments and lateral displacements of the sheet pile wall improved by stone columns and earthquake drains are presented in Figures 22 and 23, respectively. Both ground improvement methods show decrease in the bending moment and displacement of the sheet pile wall. In this study, stone columns provided smaller bending moments and displacements of the sheet pile wall compared to those estimated using earthquake drains.

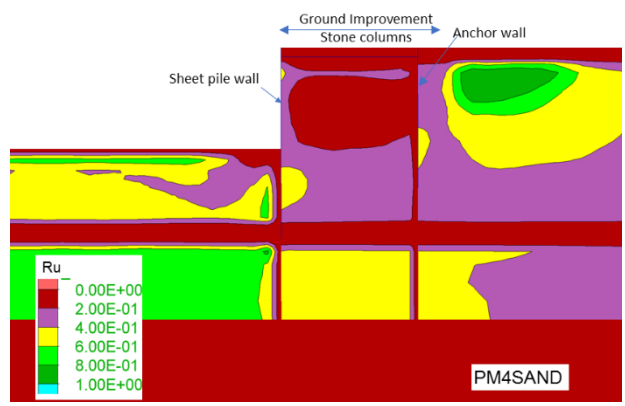

Figure 20. Excess pore pressure ratio R_u - Stone columns ground improvement

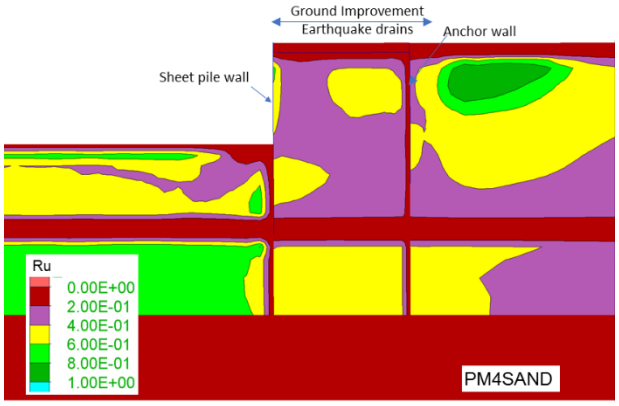

Figure 21. Excess pore pressure ratio R_u – Earthquake drains ground improvement

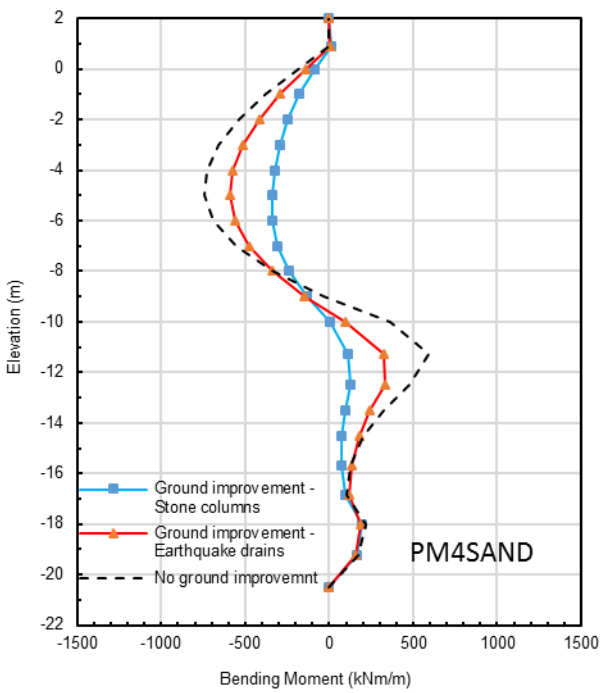

Figure 22. Bending moments of the sheet pile wall

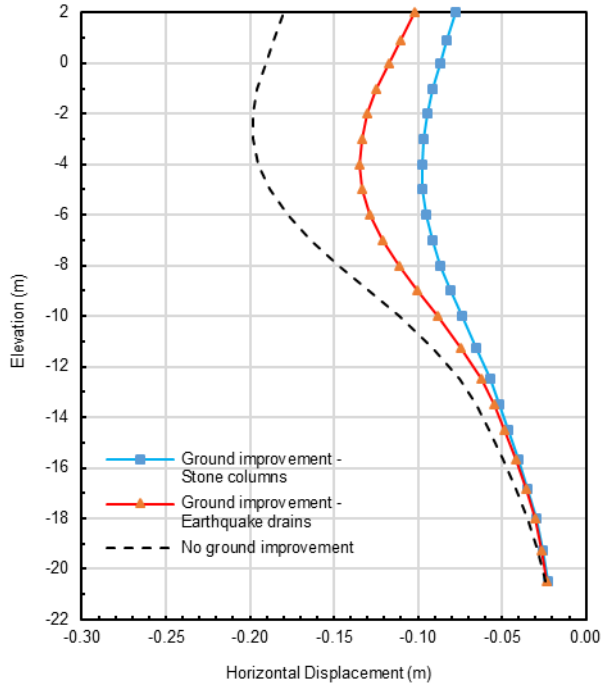

Figure 23. Lateral displacements of the sheet pile wall

7 CONCLUSIONS AND DISCUSSIONS

The importance of calibration of the UBCSAND and PM4SAND models is emphasized. This is to produce reasonable responses, including number of cycles to liquefaction, development of excess pore water pressure ratio, confining stress and static shear bias effects, and modulus reduction and damping. The calibration led to the development of model parameters that were found to capture the behavior of the anchored sheet pile wall system.

The PM4SAND model produced soil liquefaction and seismic responses of the anchored sheet pile wall system which are consistent with the those estimated using the UBCSAND model. The use of different soil models with proper model calibration can identify issues with the simulation and suggest the approximate range of simulation results.

Both stone columns and earthquake drains showed their efficiency for seismic ground improvement. Soils densified using stone column method showed better seismic performance for the analyzed sheet pile wall system.

8 ACKNOWLEDGEMENTS

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