DYNAMIC ANALYSES OF LIQUEFACTION AT PALINURUS ROAD IN THE CANTERBURY EARTHQUAKE SEQUENCE

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ABSTRACT

Two-dimensional (2D) nonlinear dynamic analyses (NDAs) have become effective means for evaluating the dynamic response of dams, levees, and other earth-fill structures affected by liquefaction. They can account for site-specific ground motions, realistic cyclic stress-strain responses, and spatially variable subsurface profiles; all of which are neglected in common 1D simplified liquefaction evaluation methods, including liquefaction vulnerability indices (LVIs). The performance of a “free-field” case history site located along Palinurus Road in Christchurch, New Zealand is evaluated using 2D NDAs to examine why 1D LVI approaches over-estimate liquefaction effects at this site for the 2010 Darfield and 2011 Christchurch earthquakes. The NDAs are performed using the PM4Sand and PM4Silt constitutive models for the sand-like and clay-like portions of the subsurface, respectively, within the FLAC finite difference program. Cyclic strength parameters were obtained from in-situ shear wave velocity and cone penetration test (CPT) data. A parametric study is performed to assess the sensitivity of different representative property selections for each stratum and the use of a CPT inverse filtering procedure to correct for thin-layer and transition zone effects. Ground deformations and flow patterns caused by the dissipation of excess pore pressures during and after ground shaking are examined. The NDA and LVI results are evaluated for their ability to predict the observed spatial variations in liquefaction surface manifestations (e.g., sand boils) across the site during the earthquakes. The results provide insights on how stratigraphic details and other factors can affect the actual degree and extent of liquefaction surface manifestations.

INTRODUCTION

Case history studies have shown that simplified liquefaction analysis methods can systematically over-estimate the degree and extent of liquefaction surface manifestations (e.g., sand boils or ground deformations) in specific areas or deposits (e.g., Boulanger et al. 2019, Chu et al. 2007, Beyzaei et al. 2018, Maurer et al. 2014, van Ballegooy et al. 2014). Simplified liquefaction analysis methods include a number of one-dimensional (1D) Liquefaction Vulnerability Indices (LVIs) that involve depth-weighted integration

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of predicted strains or factors of safety against liquefaction triggering using data from individual borings or cone penetration test (CPT) soundings. These methods have frequently been observed to over-estimate liquefaction effects for deposits where the sedimentary stratigraphy includes interbedded or alternating beds of sands, silts, and clays. Note that a unit may be described as interbedded when it consists of alternating thin layers (i.e., less than 10 cm thick) of different lithologies (Nichols 2009).

Several factors may contribute to a tendency for over-estimating liquefaction effects in deposits with interbedded or alternating beds of sands, silts, and clays (Boulanger et al. 2016). These include limitations in: (1) site characterization tools and methods, (2) liquefaction triggering or deformation correlations, and (3) analysis approaches and neglected mechanisms. The first set of limitations is concerned with the challenges in characterizing thin layers, transition zones, graded bedding, lateral discontinuities, and partial saturation near the water table. The second set describes the uncertainties and biases associated with correlations for cyclic resistance ratio (CRR), and shear and volumetric strains, which are not well-constrained for intermediate soils (e.g., low-plasticity silty sands, clayey sands, or sandy silts) and do not directly account for the effects of age, stress-strain history, cementation, and anisotropy. The third set includes difficulties in addressing spatial variability, pore pressure diffusion, multi-dimensional deformation geometries, and the dynamic response. Two-dimensional (2D) nonlinear dynamic analyses (NDAs) can provide an improved basis, relative to LVIs, for interpreting case histories, as demonstrated by Cubrinovski et al. (2018), Hutabarat and Bray (2019), and Boulanger et al. (2019). Nonetheless, the over-estimation bias of 1D LVIs for these types of deposits is likely due to a combination of the above limitations.

This paper describes a 2D NDA study of a site located along Palinurus Road in Christchurch, New Zealand, where: (1) the soil profile includes laterally continuous and laterally discontinuous layers of sands and clayey silts, and (2) 1D LVI analyses were shown by Yost et al. (2019) to over-estimate liquefaction effects during the 2010 Darfield and 2011 Christchurch earthquakes. The site performance during these earthquakes, subsurface conditions, and results of the prior LVI analyses are first described. The numerical modeling approach using FLAC (Itasca 2016) and calibration of the constitutive models are described. Detailed results for a baseline set of parameters are presented first, followed by results of parametric analyses examining sensitivity to different representative property selections for each strata and the use of a CPT inverse filtering procedure to correct for thin-layer and transition zone effects. The NDA results are used to evaluate how the dynamic response and patterns of pore pressure diffusion are influenced by the details of the subsurface stratigraphy, and how they may relate to the observed patterns of liquefaction manifestation at this site during these two earthquakes. Implications of these results for the use of NDAs and LVIs in practice and future research efforts are discussed.
2010 and December 2011. The 2010-2011 CES resulted in well-documented and widespread liquefaction damage throughout the city of Christchurch. The Palinurus Road site, a grass field located in the Woolston suburb of Christchurch, did not exhibit any evidence of liquefaction during the initial 4 September 2010 Mw 7.1 Darfield earthquake, but showed several moderate sand boils during the 22 February 2011 Mw 6.2 Christchurch earthquake (these events are hereafter labeled as Sep2010 and Feb2011). As can be seen in the aerial photograph of Figure 1, these sand boils were only observed along the northeast (NE) half of the site. Palinurus Road was one of the 55 level-ground free field case history sites in Christchurch selected for a region-wide study on the system response of liquefiable deposits (Cubrinovski et al. 2018). A detailed description of available site investigation data and the sensitivity results of an in-depth 1D LVI study at the Palinurus Road site are provided by Yost et al. (2019).

The Palinurus Road site plan shown in Figure 1 depicts the aerial locations of available site investigation data obtained from the New Zealand Geotechnical Database (NZGD). The site plan includes eight CPTs and two sonic boreholes that were conducted in April 2012 as part of a geotechnical investigation considering potential sites for a proposed sewer pump station. Between 2015 and 2016, three additional CPTs, a seismic CPT (SCPT), an additional sonic borehole, and a direct-push crosshole test (DPCH) were completed as part of the regional liquefaction study previously described.

The subsurface profile presented in the top of Figure 2 shows the borehole and measured CPT data along cross-section A-A’ on Figure 1. The cone tip resistances normalized by atmospheric pressure (\(q_{\text{TN}}\)) are presented along the horizontal axis of each CPT. The CPT data are color-coded based on the Soil Behavior Type Index (Ic) to approximately identify layers of silt mixtures and clays (Ic > 2.6; blue), sand mixtures (2.05 ≤ Ic ≤ 2.6; green), and sands (Ic < 2.05; beige) (Robertson 2009). The bottom of Figure 2 depicts the same subsurface profile, but has corrected the CPT data for thin-layers and transition zones.
using the inverse filtering procedure of Boulanger and DeJong (2018) with baseline input parameters. Inverse filtering tends to increase the $q_N$ for thin sand and gravel layers, but decrease the $q_N$ for thin layers of silt mixtures and clays. The effect of this thin-layer and transition zone correction process is examined as part of the NDA sensitivity study.

Figure 2. CPT measurements before (top figure) and after inverse filtering (bottom figure) along the Palinurus Road subsurface profile (A-A' in Figure 1).

The subsurface at Palinurus Road is interpreted to have eight distinct strata within the upper 25 m as observed from the subsurface investigation data and depicted in Figure 3. The approximately 3 m thick surface stratum, denoted by stratum A, is composed of topsoil and non-engineered fill, with non-plastic silts sitting atop loose silty sands. This is underlain by stratum B1 which extends to a depth of 6 m and is primarily made up of loose to medium dense clean sands. The borings and CPTs along the southwest (SW) half of the site indicate the presence of soft to firm silt with moderate plasticity between a
depth of about 6 to 7.5 m (stratum C1), overlying a loose sand interbedded with thin (less than 50 mm thick) clayey silt layers (stratum C2) to a depth of about 9 m. This sequence of strata (C1 and C2) was not observed in the CPTs on the NE half of the site (i.e., CPTs 62759, 62760, 62761). The C1-C2 and B1 strata are underlain by the clean sand stratum B2 to a depth of about 17 m with occasional thin silt interbeds. This is followed by a 1-m thick layer of soft clay and silt with moderate plasticity to a depth of 18 m (stratum D1), overlying a medium dense silty sand to an average overall depth of 20.5 m (stratum D2). Finally, stratum E represents the upper surface of the very dense Riccarton Gravels.

The groundwater table is estimated to be at a depth of 1.2 m below the level ground surface at the time of the earthquakes, based on the site investigations and nearby piezometer readings (CGD 2014). The compression wave velocity ($V_p$) was observed to reach about 1,500 m/s just below a depth of 1.2 m, which suggests the soil is fully saturated (Yost et al. 2019). Partial saturation is therefore not expected to affect the cyclic resistance of soils below the water table.

Prior analyses of liquefaction effects at Palinurus Road by Yost et al. (2019) generally indicated an over-prediction of liquefaction effects throughout the site for all 1D LVI metrics considered and for both the Sep2010 and Feb2011 earthquakes. Of particular interest, no significant difference in LVI values were observed between the three NE CPTs (i.e., near sand boils), and the SW CPTs (i.e., far from sand boils). For example, for the Feb2011 earthquake, the mean Liquefaction Potential Index (LPI) following fines content and thin-layer corrections was calculated to be 28 for the SW CPTs and 30 for the NE CPTs, both indicating a prediction of severe liquefaction manifestation [Several manifestation thresholds have been proposed; the framework presented in McLaughlin (2017), which was informed by research conducted by R. Green (personal communications), indicates an LPI < 8 correlates to “none to marginal,” 8 ≤ LPI < 15 correlates to “moderate,” and LPI ≥ 15 correlates to “severe”]. The Feb2011 peak ground acceleration (PGA) of 0.68 g used for the LVI analyses was obtained from the ground motion model by Bradley (2014), which is conditioned on nearby seismic recording stations located above deposits that liquefied during the earthquake. The model does not remove high frequency ground motion spikes caused by cyclic mobility of the soil following the onset of liquefaction, and may ultimately produce a bias in simplified liquefaction predictions (Upadhyaya 2019). Regardless, sensitivity analyses showed that reasonable variations in the estimated PGA would not alter the general conclusions regarding over-estimation of liquefaction effects and the similarity of results for both the SW and NE halves of the site.

**NONLINEAR DYNAMIC ANALYSES METHODOLOGY**

**Numerical Model**

The Palinurus Road soil profile was modeled for 2D NDAs using the finite-difference program FLAC 8.1 (Itasca 2019) and the user-defined constitutive models PM4Sand (Version 3.1; Ziotopoulou and Boulanger 2016, Boulanger and Ziotopoulou 2017) and PM4Silt (Version 1; Boulanger and Ziotopoulou 2018, 2019). Figure 3 depicts a 100-m-
long central portion of the plane-strain mesh used in the analyses, which is based on the soil strata observed in the SW-NE trending cross section A-A′ (Figure 2). The full model mesh is 240 m long by 25 m tall, and is made up of 24,000 elements, each 1.0 m long by 0.5 m tall. Results are presented only for the central 100-m long segment of the mesh, which is far enough from the lateral boundaries to effectively reduce potential boundary effects. Stress conditions were initialized prior to dynamic loading by choosing elastic moduli that would produce a coefficient of earth pressure at rest (Ko) of 0.5 for all soil strata. The water table was initialized with a static phreatic surface at 1.2 m below the ground surface. Soil below the water table was fixed as fully saturated, while all soil above the water table was fixed at a saturation of 0.7 to allow for a realistic unsaturated unit weight. The soil properties and constitutive models assigned for dynamic analyses are listed in Table 1.

![Figure 3. FLAC mesh used for Palinurus Road NDAs.](image)

Table 1. Soil properties and constitutive models assumed for NDA models.

<table>
<thead>
<tr>
<th>Strata</th>
<th>Dry Density (kN/m³)</th>
<th>Porosity</th>
<th>kv (m/s)a</th>
<th>kₖ/kₜ</th>
<th>Constitutive Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>14.7</td>
<td>0.44</td>
<td>1E-05</td>
<td>1</td>
<td>PM4Sand</td>
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<tr>
<td>B1</td>
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<td>1</td>
<td>PM4Sand</td>
</tr>
<tr>
<td>B2</td>
<td>14.7</td>
<td>0.44</td>
<td>1E-04</td>
<td>1</td>
<td>PM4Sand</td>
</tr>
<tr>
<td>C1</td>
<td>14.7</td>
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<td>1</td>
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<td>PM4Sand</td>
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<td>1E-09</td>
<td>1</td>
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</tr>
<tr>
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<td>0.44</td>
<td>1E-06</td>
<td>1</td>
<td>PM4Sand</td>
</tr>
<tr>
<td>E</td>
<td>17.9</td>
<td>0.46</td>
<td>1E-02</td>
<td>1</td>
<td>Elastic</td>
</tr>
</tbody>
</table>

aHydraulic conductivity, kv, is estimated per Robertson (2010)

The boundary conditions for the dynamic analyses include a compliant (quiet) base, where the outcrop input motion is applied as a horizontal stress-time history. The left and right boundaries of the model were slaved together. Since the side boundaries are located about 70 m from either side of the considered portion of the mesh, they are not expected to adversely affect the analysis results. The pore pressure boundaries were free (i.e., impermeable) at the sides of the model and fixed (i.e., allowed to flow outside the model) at the base and top of the model. Thus, excess pore pressures generated during shaking can only drain towards the phreatic surface or through the gravel at the base of the model.
For this present work, the horizontal input motions for the Sep2010 and Feb2011 events were based on a single component of the outcropping motions recorded about 10 km away at the Riccarton High School (RHSC) strong motion station. The recordings at that station were deconvolved to the same Riccarton Gravel stratum using the 1D site response program Strata (Kottke et al. 2018), following the guidance and recommended procedure explained in Markham et al. (2015). To account for site-to-source path effects, the deconvolved outcropping motions were scaled to coincide approximately with the local ground motion model (GMM) by Bradley (2013) between spectral periods of 0.1 to 1.0 seconds. The scaled outcrop motions had peak horizontal accelerations of 0.14 g and 0.60 g for the Sep2010 (~20 km to rupture plane surface projection) and Feb2011 (within 1 km of rupture plane surface projection) events, respectively. Future work will describe the results of analyses using alternative input motions, including ground motions from physics-based simulations of these two earthquakes.

The dissipation of excess pore pressures and the flow regime was monitored for ten hours (prototype) following the earthquake motion. The simulation process was increased by scaling the permeability of each strata by a factor of 100 at the end of strong shaking, which effectively scales the post-shaking time by a factor of 1/100. Additionally, since FLAC is unable to directly model the creation of cracks as expected in the crust layer prior to the observed Feb2011 sand boil formation, the permeability of stratum A was further increased by a factor of 10 to allow a more reasonable flow response at the surface. The “PostShake” option of the PM4Sand and PM4Silt constitutive models was activated to more reasonably simulate volumetric reconsolidation strains after shaking.

**Calibration of Constitutive Models**

Representative properties were developed for each stratum based on available in-situ test data. Shear wave velocities (Vs) were interpreted using DPCH and SCPT measurements. The Vs of stratum E was estimated as 400 m/s based on surface wave (MASW) measurements performed at nearby sites (Wotherspoon et al. 2015). Since stratum E was modeled as an elastic material, elastic shear and bulk modulus values were estimated assuming a Poisson ratio of 0.33 and a constant shear modulus reduction factor of 0.70 to account for earthquake-induced cyclic degradation. The normalized clean sand corrected tip resistance (qc1Ncs) was calculated per the recommendations of Boulanger and Idriss (2014) using a site-specific fines content correction for all soil with Ic ≤ 2.6. The undrained shear strength ratio (su/σ'vc) was calculated based on a cone bearing factor (Nkt) of 15 for all soil with Ic > 2.6. For each applicable property (i.e., qc1Ncs, su/σ'vc), 33rd and 50th percentile values were obtained for each stratum based on the combined data from all CPTs. The 33rd to 50th percentile range is believed to encompass reasonably representative values (for an unbiased estimate of expected responses) based on the findings of Montgomery and Boulanger (2016) for NDAs involving an evaluation of post-liquefaction reconsolidation. Four sets of parametric variations were completed with the following global property assumptions: (1) 33rd percentile from measured CPT data (33Meas; the baseline case), (2) 50th percentile from measured CPT data (50Meas), (3) 33rd percentile from inverse filtered CPT data (33IF), and (4) 50th percentile from inverse filtered CPT data (50IF).
Table 2 presents the calibrated PM4Sand properties selected for the four sets of parametric studies. The unitless shear modulus coefficient ($G_o$) was determined based on the $V_s$ and effective stresses at the middle of each stratum. The apparent relative densities ($D_R$) were derived from the applicable representative $q_{c1Ncs}$ for each stratum using the relationship in Boulanger and Idriss (2014). The contraction rate parameter ($h_{po}$) was chosen based on an iterative adjustment to obtain peak shear strain of 3% with a target normalized cyclic resistance ratio ($CRR_{M7.5,1atm}$) in 15 uniform stress cycles of simulated undrained direct simple shear (DSS) loading. The $CRR_{M7.5,1atm}$ target value was obtained based on the $q_{c1Ncs}$ relationship by Boulanger and Idriss (2014). Default values were used for all secondary PM4Sand parameters.

Table 3 depicts the calibrated PM4Silt properties selected for the four sets of parametric studies. The $G_o$ was determined based on the $V_s$ and effective stresses at the middle of each stratum. The representative $su/\sigma'_{vc}$ for each stratum was increased by a 25% strain rate adjustment to obtain the undrained shear strength ratio at critical state under earthquake loading ($su,cs,eq/\sigma'_{vc}$). The $h_{po}$ parameter was chosen based on an iterative adjustment to obtain a reasonable slope of cyclic resistance against the number of uniform loading cycles to cause a 3% peak shear strain under simulated DSS loading. The simulated undrained cyclic loading response using the default shear modulus parameter ($h_o$) resulted in shear modulus reduction ($G/G_{max}$) and equivalent damping behavior that was similar to the empirical relationships of both Vucetic and Dobry (1991) and Darendeli (2001) for strata C1 and D1. Default values were used for all other secondary PM4Silt parameters.

<table>
<thead>
<tr>
<th>PM4Sand Strata</th>
<th>$V_s$ (m/s)</th>
<th>$G_o$ (-)</th>
<th>$D_R$</th>
<th>$q_{c1Ncs}$</th>
<th>$h_{po}$</th>
<th>$D_R$</th>
<th>$q_{c1Ncs}$</th>
<th>$h_{po}$</th>
<th>$D_R$</th>
<th>$q_{c1Ncs}$</th>
<th>$h_{po}$</th>
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<tbody>
<tr>
<td>A</td>
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<td>651</td>
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<td>96</td>
<td>0.32</td>
<td>0.58</td>
<td>106</td>
<td>0.28</td>
<td>0.60</td>
<td>111</td>
<td>0.28</td>
</tr>
<tr>
<td>B1</td>
<td>175</td>
<td>983</td>
<td>0.62</td>
<td>118</td>
<td>0.21</td>
<td>0.66</td>
<td>129</td>
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<td>132</td>
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<td>0.58</td>
<td>107</td>
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<td>D2</td>
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<td>114</td>
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<td>0.64</td>
<td>123</td>
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<thead>
<tr>
<th>PM4Silt Strata</th>
<th>$V_s$ (m/s)</th>
<th>$G_o$ (-)</th>
<th>$su,eq,cs/\sigma'_{vc}$</th>
<th>$h_{po}$</th>
<th>$su,eq,cs/\sigma'_{vc}$</th>
<th>$h_{po}$</th>
<th>$su,eq,cs/\sigma'_{vc}$</th>
<th>$h_{po}$</th>
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<tr>
<td>C1</td>
<td>165</td>
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<tr>
<td>D1</td>
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<td>498</td>
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<td>40</td>
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<td>0.24</td>
<td>10</td>
<td>0.36</td>
<td>30</td>
</tr>
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</table>

**NDA RESULTS**

**Baseline Case Response**

The dynamic response of the baseline case, corresponding to the $33Meas$ parameter set, for the Feb2011 event is illustrated in Figure 4 showing ground surface acceleration and
pore pressure responses for select points along the SW and NE sides of the model. The surface accelerations are quite different on the two sides of the model, with a higher PGA of 0.62 g at about 7.3 s on the NE side, as opposed to the PGA of 0.42 g at about 6.9 s on the SW side. The PGA contrast across the site is primarily attributed to differences in the timing of liquefaction triggering and the extent of liquefaction between the two sides. The pore pressures increase during shaking, with the triggering of liquefaction [in terms of an excess pore pressure ratio (u) of 100%] evident in the plateauing of the peak pore pressures, along with the subsequent transient reductions in pore pressure due to the cyclic mobility responses of the soil. Stratum C2 on the SW side is the first layer to trigger liquefaction at 6.1 s, followed by triggering of stratum D2 across the entire profile at about 7.3 s. By introducing a weak zone capable of attenuating the remaining cyclic demand, the early triggering of liquefaction in stratum C2 appears to be the main factor in preventing liquefaction at the center of stratum B2 (13.5 m below ground surface) along the SW side, as opposed to the NE side, which triggered at the same depth at about 8.6 s. A slow increase in the pore pressure observed after about 10 s for the NE B1 and upper B2 strata is attributed primarily to pore pressure diffusion, which may have partially contributed to liquefaction in these layers.

Figure 4. Ground surface acceleration time histories and at-depth pore pressure time histories for the Feb2011 baseline simulation.
A contour plot of the maximum excess pore pressure ratio ($r_{u,max}$) determined after shaking is shown in Figure 5. The $r_u$ is commonly defined as the ratio of the excess pore pressure to the initial vertical effective stress for the interpretation of laboratory tests. However, for the interpretation of numerical simulations, $r_u$ is better defined as *one minus the ratio of the current to initial vertical effective stress* to account for the fact that the total vertical stress can change during the simulation of 2D or 3D systems. The latter definition is used herein, with the two being equivalent if the total vertical stress does not change during loading. For practical purposes, liquefaction in the model is considered to have occurred wherever $r_{u,max}$ is greater than or equal to 95%. As expected, strata B1 and B2 have greater volumes of liquefied soil at the NE side as opposed to the SW side.

A contour plot of the maximum engineering shear strain ($\gamma_{max}$) is presented in Figure 6. The greatest strains (~ 5 to 10%) developed in strata C2 and D2. Significant strains also developed along the bottom of stratum A (~ 3%) and in the upper portion of stratum B2 on the NE side (~ 2 to 5%). The overall pattern of strains are consistent with the cyclic strengths and relative densities of each stratum.

![Figure 5. Maximum excess pore pressure ratios for the Feb2011 baseline simulation.](image)

![Figure 6. Maximum shear strains for the Feb2011 baseline simulation.](image)

**Parametric Studies**

Results for the four model cases (33Meas, 50Meas, 33IF, 50IF) with the stronger Feb2011 motion are summarized in Table 4 in terms of the thickness of soil that liquefied (defined by $r_u \geq 95\%$) in each stratum. For the Sep2010 motion, liquefaction only occurred for the 33Meas case and was limited to a 1-m thick zone of stratum C2. Thus, the results for the Sep2010 motion are generally consistent with the absence of surface manifestations following this event. For the Feb2011 motion, significant amounts of liquefaction were computed for all four cases.
An interesting observation is made when comparing the results for 33Meas and 50Meas with the Feb2011 motion (Table 4). The stronger 50Meas case developed liquefaction in an additional 4.0 m of stratum B2 on the NE side. The stronger 50Meas case developed smaller shear strains along strata C2 and D2, which appears to have reduced the dampening of the ground motion experienced at the NE side. The resulting increase in the motions at the NE side were enough to trigger liquefaction over a greater portion of stratum B2.

Table 4. Resulting liquefied layer thicknesses for the Feb2011 NDA parametric analyses.

<table>
<thead>
<tr>
<th>PM4Sand Strata</th>
<th>Total Saturated Thickness (m)</th>
<th>33Meas</th>
<th>50Meas</th>
<th>33IF</th>
<th>50IF</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>SW</td>
<td>NE</td>
<td>SW</td>
<td>NE</td>
<td>SW</td>
</tr>
<tr>
<td>A</td>
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<tr>
<td>B1</td>
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</tr>
<tr>
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<td>0.5</td>
<td>7.0</td>
<td>1.0</td>
</tr>
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<td>C2</td>
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<td>2.5</td>
<td>1.5</td>
<td>2.0</td>
<td>1.0</td>
</tr>
<tr>
<td>D2</td>
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<td>2.5</td>
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<td>2.0</td>
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<td>Total</td>
<td>16.8</td>
<td>18.3</td>
<td>5.5</td>
<td>13.5</td>
<td>6.0</td>
</tr>
</tbody>
</table>

*Liquefaction is defined by 0.5 m thick elements exhibiting an excess pore pressure ratio (r_p) ≥ 95%*

Inverse filtering of the CPT data significantly reduced the thickness of soil that liquefied, as seen by comparing results for 33IF and 50IF with those for 33Meas and 50Meas, respectively. Inverse filtering resulted in reduced undrained shear strengths for the clay-like strata (i.e., C1 and D1) and increased cyclic strengths for the sand strata. For the 33IF case, a maximum shear strain of over 30% developed along the soft clay stratum D1. This reduced the dynamic response of the entire model by dampening the ground motion and reducing the overall amount of liquefied soil. For the 50IF case, the clay stratum D1 was strong enough that maximum strains of between 2 to 10% were distributed across strata A, C2, and D1, and the thickness of liquefied soils was consequently more than double that for the 33IF case. In contrast, clay stratum D1 was strong enough in the 33Meas and 50Meas cases that it developed shear strains of less than 1%, such that liquefaction was more extensive in the sand strata.

The overall pattern of excess pore pressure diffusion and ground water flow following strong shaking for all four cases with the Feb2011 motion showed that the majority of the outflow occurred on the NE side, near the right edge of the lower permeability C1 stratum (x ≈ 50 to 65 m). The total outflow volume per area (Q_VOL/A) at the ground surface is plotted versus horizontal position along the model in Figure 7. The vertical settlement (displacement) profile relative to the clay stratum D1 (Δy) during reconsolidation is also shown on this figure; relative settlements are used for this comparison because excess pore pressures below stratum D1 are expected to drain towards the stratum E gravels. The relative settlement profile Δy along the x-axis follows a similar trend as the cumulative liquefied thicknesses of Table 4. Computed settlements on the SW side range from about 3 to 4 cm for the 33Meas and 50Meas cases and from about 1 to 2 cm for the 33IF and 50IF cases. Computed settlements on the NE side range from about 7 to 9 cm for the 33Meas and 50Meas cases and from about 0 to 2 cm for the
33IF and 50IF cases. These computed settlements are reasonably consistent with the absence of ground cracking, given that settlements of less than about 10 cm would be difficult to detect visually in a grass field unless they varied sharply over short distances. More strikingly, the $Q_{VOL/A}$ tends to be less than 1 cm at the SW side for all cases ($x < 35$ m) and reaches peak values of 9 to 25 cm on the NE side just past the lateral edge of the silt stratum C1 ($x \approx 55$ m). Further towards the NE side ($x > 90$ m), the $Q_{VOL/A}$ tends to be reasonably similar to the $\Delta y$, as expected. These results illustrate that reconsolidation of the soils beneath the lower permeability C1 stratum on the SW side is accommodated by ground water flowing laterally toward the NE side, where it can more easily escape to the ground surface. Ground water fluxes of less than 1 cm on the SW side appear consistent with the absence of sand boils in this area, and ground water fluxes of up to 25 cm on the NE side appear consistent with observations of sand boils in that area.

Figure 7. Total outflow volume per unit area ($Q_{VOL/A}$) and vertical displacement relative to stratum D1 ($\Delta y$) as measured near the phreatic surface for the Feb2011 event.

DISCUSSION

The 2D NDA results provide reasonable bounds on the observed patterns of liquefaction manifestation at Palinurus Road. In contrast, the 1D LVI results were practically incapable of predicting a difference in manifestations between the SW and NE sides of the site. The NDA results suggest that over-prediction of liquefaction effects by LVIs may be largely explained by three factors: (1) inadequate simplification of the dynamic response; (2) no consideration of multi-dimensional stratigraphic contributions to pore pressure diffusion; and (3) bias in CPT data due to thin-layer and transition zone effects. The largest factor appears to be the inherent limitation that LVI analysis methods do not account for the dynamic response of the soil profile as a system. Cubrinovski et al. (2018) demonstrated this limitation using 1D NDAs for representative idealized soil profiles, and concluded that the cross-interaction of dynamic effects can be critical for an accurate evaluation of liquefaction effects at sites with various sedimentary structures. The present analyses further enforce those observations. Another contributing factor observed in this analysis is that the presence of laterally discontinuous lower-permeability layers can
influence the patterns of pore pressure diffusion and consequently alter the distribution of surface manifestations (e.g., sand boils) relative to the actual locations of liquefaction triggering in the subsurface. A third contributing factor is the bias in CPT data that can arise from thin-layer and transition zone effects. Correcting CPT data for these effects significantly influenced the NDA results, although it had a relatively modest effect on LVI results for this site (e.g., Yost et al. 2019). Other factors influence both the LVI and NDA results, including uncertainties in the input motions and soil property correlations, and these will be examined further in ongoing work.

NDAs simulate more realistic behavior than LVIs, but nonetheless still have limitations. For instance, they are generally unable to directly simulate some of the physical mechanisms involved with pore pressure dissipation, including the initial generation of a water film beneath more impermeable layers (e.g., Kokusho 2000), cracking of the crust layer due to ground distortions, erosion and ground loss due to ejecta during sand boil formation, and sedimentation effects during post-liquefaction reconsolidation. Also, the modeled stratigraphy is a simplification based on a typical layout of CPTs and borings, and may not adequately capture the spatial variability of soil parameters and layer extents. As with LVIs, NDAs are subject to uncertainty from the input parameters, and good practice requires sensitivity analyses to represent a range of expected behavior. In spite of these limitations, the NDA results for Palinurus Road reasonably bound the observed liquefaction manifestations and sand boil patterns during these two earthquakes.

CONCLUSION

This paper examined the Palinurus Road case history site for the Sep2010 and Feb2011 events through a series of 2D NDAs with parametric variations of the representative properties from measured and inverse filtered CPT data. The NDA model results compared well with the observed surface manifestations for both the Sep2010 and Feb2011 events. This case history highlights the advantages of NDA methods for soil profiles with interbedded or alternating beds of sands, silts, and clays, and the importance of several mechanisms that are typically neglected by simplified methods, including 1D LVIs. The primary observations were the following:

- The existence of low permeability strata can influence excess pore pressure dissipation and ground water flow patterns, ultimately influencing the onset or progression of liquefaction and the production of sand boils. Laterally discontinuous lower-permeability layers can influence the locations where the ground water flux at the ground surface is greatest, thereby shifting the location of surface manifestations (e.g., sand boils) relative to the actual locations of liquefaction triggering in the subsurface.
- The relative timing of the onset and progression of liquefaction or cyclic softening, and the associated increase in shear strains, in individual strata within a heterogeneous deposit may significantly affect the overall dynamic system response.
- The sensitivity of predicted performance to representative property selection was highlighted, with a significant difference in response predicted when representative properties based on 33rd and 50th percentiles were used.
The use of inverse filtered CPT data reduced the estimated extent and degree of liquefaction. The influence of inverse filtering is twofold as it typically lowers the measured resistance of clay-like layers (causing a greater likelihood of high strains in these layers) and increases the resistance of sand-like layers (improving the CRR).

The results of this case study provide support for the use of these NDA methods and procedures in seismic evaluations and in examining other case histories for a broad range of civil infrastructure, including dams, levees, and other earth-fill structures. NDAs can account for site-specific ground motion responses, realistic cyclic stress-strain responses, and spatially variable subsurface profiles; all of which are neglected in common simplified liquefaction evaluation methods. Additional case studies are required to understand better the mechanisms and factors contributing to over-predictions or mispredictions of liquefaction effects in different types of soil deposits.

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