Efficient Finite-Element Model for Seismic Response Estimation of Piles and Soils in Liquefied and Laterally Spreading Ground Considering Shear Localization

Xiaowei Wang, S.M.ASCE¹; Fuyuan Luo²; Zhenyu Su³; and Aijun Ye⁴

Abstract: A key challenge for estimating seismic response of piles and soils in liquefied and laterally spreading ground is the efficient prediction of local loosening phenomenon (shear localization) at the interlayer between the liquefied loose sand and the overburden crust. To this end, a simplified finite-element (FE) model was developed. In the soil model, a soft interlayer element with a thickness and corresponding low reference shear modulus was developed to represent the shear localization phenomenon. The proposed FE models were then used to simulate centrifuge tests of a single pile, a two-pile group, and a six-pile group in sloping liquefied profiles that lead to lateral spreading. Predicted results of the shear-localization-induced soil lateral spreading displacement, as well as the structural response, agree reasonably well with the test records, indicating that the proposed FE model is capable of approximating the seismic responses of soil and piles in liquefied and laterally spreading ground. In addition, a parametric analysis was performed to build a relationship between the thickness and the low reference shear modulus of the soft interlayer element. DOI: 10.1061/(ASCE)GM.1943-5622.0000835. © 2016 American Society of Civil Engineers.

Author keywords: Piles; Soil liquefaction; Lateral spreading; Shear localization; Finite-element model.

Introduction

Shaking-induced liquefaction and associated lateral spreading has caused extensive damage to pile structures (Hamada 1992a, b; Ishihara 1997; Tokimatsu et al. 1998). The lateral spreading phenomenon commonly occurs at sloping sites where nonliquefiable crust (with deep slope) overlies liquefied loose sand. In a physical mechanism, as illustrated in Fig. 1, the earthquake-induced excess pore-water pressure in the underlying loose sand causes upward seepage of pore water, which is then impeded by the lower-permeability crust and possibly accumulates to form a water film (Elgamal et al. 1989; Kokusho and Kojima 2002; Malvick et al. 2008). As a result, a loosening soil layer (i.e., shear localization) forms beneath the crust, which causes drastic variation of shear strain (Martino et al. 2015) and displacement discontinuity at the interface between the liquefied sand layer and the overburden crust (Fiegel and Kutter 1999; Kokusho 1999; Brandenberg et al. 2005, 2007; Kulasingam et al. 2004; El Shamy et al. 2010; Chang et al. 2013; Boulanger et al. 2014).

The finite-element (FE) model for estimating lateral spreading and shear localization is a challenge to the engineering society. Previous studies focused on the seismic response of piles. Static or dynamic Winkler models without soil mesh are preferred by many researchers due to computational efficiency (e.g., Liyanapathirana and Poulos 2005, 2010; Juimarongrit and Ashford 2006; Phanikanth et al. 2013; Brandenberg et al. 2013). In their studies, lateral spreading displacements and effective stress that were (1) measured from a field or laboratory test or (2) predicted from a preliminary numerical site response analysis were used as inputs to the free end of the soil springs. The former inputs isolate the predictive uncertainties in numerical site response analysis, whereas the latter inputs do have these uncertainties; in addition, the shear localization was not considered in their numerical site response analyses. Although this Winkler method was able to approximate maximum bending moment and displacement of piles, it was difficult to combine the kinematic and inertia demands appropriately. Chang et al. (2013) developed a two-dimensional (2D) soil-spring-pile coupled FE model with interface springs between the liquefied loose sand and overburden clay crust to simulate the displacement discontinuity. Seismic loadings were input to the bottom of the soil column, and soil responses were predicted and simultaneously transmitted to the piles through pile-soil springs, which could account for the variation of strength capacity and stiffness due to changeable effective stress of the soil during the liquefaction process. However, the interface springs produced a weaker oscillation and severely underestimated the lateral spreading displacement of the clay crust, which is a critical damage index for postearthquake restoration evaluation. Other numerical studies used three-dimensional (3D) soil continua to obtain accurate results, such as Elgamal et al. (2008), Maheshwari and Sarkar (2011), and Lu et al. (2011). However, the soil shear localization is not well represented, and a fine mesh of 3D soil elements inevitably renders a time-consuming analysis. Overall, an efficient FE model for predicting lateral spreading displacement and shear localization of the soil column as well as the seismic demand of piles still needs critical investigation.

This study aims to predict, to a general accurate extent, both the shear-localization-associated soil responses and structural demand...
of pile foundations in liquefied and laterally spreading ground based on the open-source FE model platform OpenSees (McKenna et al. 2010). The proposed soft interlayer soil element characterized by a specific thickness and a low reference shear modulus is first introduced, followed by a description of centrifuge tests and model parameters. Validation of the proposed FE model is then presented. Finally, a parametric analysis is presented to shed light on selecting the thickness and the corresponding low reference shear modulus of the soft interlayer element.

**Proposed FE Model**

A fully coupled FE model [3D pile structures, 2D three-layered soil columns, and one-dimensional (1D) pile-soil springs] was developed in OpenSees and is illustrated in Fig. 2. The 2D soil is modeled by one column with fine meshes in the vertical direction, which was verified by Wang et al. (2013) through comparison with a computationally intensive 2D soil-modeling approach with fine meshes in both the horizontal and vertical directions (Kramer 2008). The motivation of modeling the pile structures in 3D space rather than 2D plane is that each pile of a pile group (e.g., 2 × 2) can be modeled separately to capture its seismic response, especially when nonlinear behavior is considered (e.g., RC piles with fiber sections). Note that the degrees of freedom of pile nodes in the Z-direction (i.e., horizontal out of plane) are fixed for computational convergency and efficiency. Each pile of the 3D structure is attached to the 2D plain-strain soil elements using nonlinear p-y, t-z, and q-z springs, which allow relative displacements between the piles and the soil column. Details about the FE model are presented as follows.

**Soil Models and Soft Interlayer Element between Nonliquefiable and Liquefied Soil**

Two soil constitutive models, Pressure Independent Multi Yield (PIMY) and Pressure Dependent Multi Yield (PDMY) materials, developed by Yang et al. (2003), are used to model clay and sand, respectively. The soil constitutive models are assigned to four-node QuadUP elements that can simulate the response of solid–fluid coupled materials under cyclic excitation (Biot 1955). The constitutive models of soils used herein have been validated by several studies (e.g., Yang et al. 2003; Chang et al. 2013; Karimi and Dashti 2015). Horizontal static forces associated with the ground slope are assigned to the clay elements to account for the lateral spreading phenomenon.

To simulate the shear localization at the interlayer between liquefied loose sand and nonliquefiable clay crust, one soft interlayer element with a thickness (h_w) and corresponding low reference shear modulus (G_s,soft) was developed. As illustrated in Fig. 2, the low reference shear modulus renders the interlayer element soft enough to generate oscillations and displacement discontinuity under seismic excitation. The thickness (h_w) accounts for the range of shear localization due to the formation of water film within or beneath the clay crust. Several physical modeling studies have illustrated the potential range of the shear localization. Kulasingam et al. (2004) conducted several dynamic centrifuge model tests of sand slopes with silt interlayers and reported that the displacement discontinuity occurred across a thickness of 0.4 m between the silt and underlying loose sand. Malvick et al. (2008) investigated the shear
localization characterization of saturated sand slope with embedded arc silt layers and revealed a loosening zone approximately 1 m thick. In addition, Kamai and Boulanger (2010) indicated that the displacement discontinuity occurred over a 0.6-m-thick interval between the clay crust and loose sand. Another 1-g shaking table test (e.g., Kokusho 1999)) observed a quite low-thickness of the loosening zone (i.e., less than 0.1 m). In general, $h_u$ falls approximately into a range of 0.1–1.0 m. It is worth noting that the thickness of the shear localization zone is a dynamic variation parameter that is difficult to capture during the shaking-induced transient liquefaction process.

As to the value of $G_{r,soil}$, there are no credible references yet to the knowledge of the authors. The physical mechanism of this soft interlayer is very complex, which is a mixture of sand and clay (i.e., silt) and may be variable during the liquefaction process. Thus, it is quite difficult to capture its dynamic constitutive model. Nevertheless, it is helpful to refer to some studies on shear modulus degradations of soil due to seismic liquefaction. Olsen (2007) used the recorded ground motions at the Port Island sites of the 1995 Kobe earthquake to quantify the shear modulus of sand as it liquefied and showed that, when completely liquefied, the shear modulus of sand for a relative density of 40–50% is approximately 1% of its initial low-strain shear modulus. Similar reduction ratios (approximately 1%) of shear modulus of liquefied soils in the 1995 Kobe earthquake by other researchers were summarized by Miwa and Ikeda (2006). Accordingly, e.g., for loose sand with an initial shear modulus of 30–40 MPa, liquefaction may cause the shear modulus to decrease to 300–400 kPa. It should be noted that, because of shear localization observed from the aforementioned centrifuge tests, the soft interlayer soil considered in this study experiences considerably larger shear deformations compared to the loose sand layers observed during the 1995 Kobe earthquake. Therefore, it is reasonable to adopt a much smaller value for $G_{r,soil}$ of the soft interlayer soil (i.e., less than 300 kPa).

In addition, it is reasonable to speculate that there is a relationship between $h_u$ and $G_{r,soil}$. In this regard, specific parameters are assigned to the following centrifuge test simulation as example cases, and parametric analyses are presented later to obtain the available selection of $h_u$ and $G_{r,soil}$. Although the soft interlayer is too complex to be modeled using a specific solid–fluid fully coupled material in the OpenSees material library, the PDMY material is adopted because it is pressure-dependent and able to trigger large shear deformation under shaking. Another critical parameter for the soft interlayer element is the permeability coefficient. In this study, a relative low-permeability coefficient (i.e., $1.0 \times 10^{-6}$ cm/s) was selected due to its approximate silty feature in light of Yang and Elgamal (2002).

### Pile–Soil Interaction Springs

**p-y, t-z, and q-z Springs for Sand Layers**

The $p-y$ and $t-z$ springs in the liquefiable loose and dense sand are represented by the PyLiq1 and TzLiq1 materials (Boulanger et al. 2003; Brandenberg et al. 2013), in which the ultimate strength ($p_{ult,liq}$ for $p-y$ and $t_{ult,liq}$ for $t-z$) changes with the mobilized excess pore-water pressure ratio ($r_p$) according to Eqs. (1) and (2)

$$p_{ult,liq} = p_{ult}(1 - r_p) + p_{res}r_p$$  \(1\)

$$t_{ult,liq} = t_{ult}(1 - r_p)$$  \(2\)

where $r_p = \Delta u/\sigma_l^e$; $\Delta u$ (kPa) = measured excess pore-water pressure; $\sigma_l^e$ (kPa) = effective overburden pressure at the measured point; $p_{res} = m_p\sigma_l^p$ = residual strength of the $p-y$ spring in fully liquefied sand (i.e., $r_p = 1$); $m_p =$ reduction coefficients recommended by Brandenberg (2005); and $p_{ult}$ and $t_{ult}$ = ultimate strength of the $p-y$ and $t-z$ springs, respectively. $p_{ult}$ (kN/m) is determined from API (2005) in association with the pile diameter [$D$ (m)], depth below the soil surface [$z$ (m)], and effective unit weight of sand [$y'$ (kN/m$^3$)]. The $p-y$ relationship is developed using Eq. (3) (Parker and Reese 1970)

$$p = Ap_{ult,liq}\tanh\left(\frac{n_bz}{Ap_{ult,liq}}\right)$$  \(3\)

where $A =$ loading factor of 0.9 for cyclic excitations; and $n_b$ (kN/m$^3$) = rate of increase of the modulus of horizontal subgrade reaction normally determined from API (2005) based on the friction angle ($\phi$). In this study, a modification multiplier of $\sqrt{50kPa}/\sigma_l^e$ to the $n_b$ at large depths is used to account for overburden effective stress (Boulanger et al. 1999).

The strength capacity of the $t-z$ spring [$t_{ult}$ (kN/m)] is computed based on Mosher (1984), and is also available in API (2005). The $t-z$ curve is formulated as the following load-transfer-curve [Eq. (4)] (Vijayvergiya et al. 1969):

$$t = t_{ult} \left(2 \sqrt{\frac{z}{z_c} - \frac{z}{z_e}}\right)$$  \(4\)

where $z_c = 0.51$ cm = critical vertical displacement at which $t = t_{ult}$.

The vertical soil resistance at pile tips are represented using $q-z$ springs with the QzSimple1 material. Meyerhof (1976) gives the ultimate resistance: $q_{ult} = N_q\sigma_{y,tip}^e$ (kPa), where $N_q$ and $\sigma_{y,tip}^e$ (kPa) are the bearing capacity factor and the effective overburden stress at the pile tip, respectively. Note that $q_{ult}$ is specified as a constant, because the soil effective stress at the pile tip should not change severely. The load-deformation relationship of the $q-z$ spring follows Eq. (5)

$$q = q_{ult} \left(\frac{z}{z_{eq}}\right)^{1/3}$$  \(5\)

where $z_{eq} = 0.05D$ = critical vertical displacement at which $q = q_{ult}$.

### p-y and t-z Springs for Clay Layers

In clay layers, PySimple1 and TzSimple1 materials (Boulanger et al. 1999) are used for modeling the pile-soil horizontal and vertical interaction, respectively. Also, the ultimate resistances of $p-y$ and $t-z$ springs herein are specified as constants, because the effective stress in the clay layer is nearly constant under seismic shaking. The ultimate strength of the $p-y$ spring in the clay layer [$p_{ult,c}$ (kN/m)] is determined from API (2005). The $p-y$ relationship can be formulated using the model proposed by Matlock (1970)

$$p = \frac{1}{2}p_{ult,c}\left(\frac{y}{2.5De_{50}}\right)$$  \(6\)

where $e_{50}$ = strain corresponding to half the maximum principal stress difference, typically equal to 0.02, 0.01, and 0.005 for soft, medium, and stiff clay, respectively.

The strength capacity of the $t-z$ spring in the clay layer [$t_{ult,c}$ (kN/m)] is also determined from API (2005). The load-
deformation property of the \( t-z \) spring in the clay layer is represented by Eq. (4) with substitution of \( t_{\text{ult,}} \) for \( t_{\text{ult}} \).

**Pile Models**

Piles and structures can be modeled using elastic or nonlinear beam-column elements, which are chosen based on the seismic performance of piles and structures in tests. For instance, piles and structures in centrifuge tests are normally modeled by elastic beam-column elements because they are practically made of aluminum or steel due to small-scale limitations on the construction of RC specimens, which normally ensures the elastic response even under strong shaking. In contrast, large-scale RC piles or structures in 1-g shaking table tests may suffer inelastic response under strong seismic excitation, where the nonlinear beam-column element is an appropriate choice. Pile-cap connections in pile group models are modeled using rigid elements (i.e., elastic beam column with large stiffness). The masses of the superstructure and cap are assigned to their orthocentrers as lumped mass.

**Boundary Condition and Solution Scheme**

The nodes on opposite sides of the soil elements are salved together (i.e., equalDOF) to form the pure-shear condition of laminar shear boxes in centrifuge tests. The clay surface nodes are allowed to drain, whereas other soil nodes are impervious. Soil bottoms are fixed in the X- and Y-directions (Fig. 2) to impose uniform excitation. In the time-history analysis, the Krylov-Newton algorithm and \( \beta \)-Newmark integrator with good convergence (i.e., \( \beta = 0.3025 \) and \( \gamma = 0.6 \)) are used to solve the matrix equations in the FE model efficiently.

**Centrifuge Tests and FE Model Validation: Description of Centrifuge Tests**

Centrifuge tests simulated in this study are PDS01 (Singh et al. 2000) and SJB03 (Brandenberg et al. 2003), whereas the low reference pressure of 100 kPa) soft clay layer (\( S_{c} = 18 \) kPa after consolidation) to protect the clay from drying. The clay deposit overlies a 4.76-m-thick loose sand layer with a relative density of \( D_{s} = 22\% \), overlying dense sand (\( D_{s} = 90\% \)). It is worth noting that all of the soil layers were prepared to a slope of 3° relative to the container base, whereas the container base slopes 1.2° to the horizon in the reverse direction. Hence, the soil layers actually slope \( \alpha = 1.8^\circ \) to the horizon, which is a key value for estimating lateral spreading displacements.

Fig. 4 shows details about the SJB03 test. A 1.09-m-diameter aluminum \( 2 \times 3 \) pile group with a 13.7 \( \times \) 8.0 \( \times \) 2.2-m cap providing a nearly rotational-rigid pile-cap connection is embedded in multilayered sloping soils. The soil profiles are built with a 1.37-m-thick layer of Monterey sand, overlying a 2.69-m-thick deposit of heavily overconsolidated clay, overlying a 5.43-m-thick layer of loose sand (\( D_{s} = 35\% \)), overlying dense sand (\( D_{s} = 75\% \)). In this case, all of the layers are prepared with a slope of 3° to the horizon. It is worth noting that the soil profiles in two centrifuge tests are common circumstances in real sites.

**Parameters in FE Models**

The input parameters for sand and clay constitutive models are determined from their measured physical properties. Tables 1 and 2 list the calculation procedure of sand (PDMY) and clay (PIMY) constitutive parameters, respectively. These parameters almost fall in the recommended ranges developed by Yang (2000). As for the soft interlayer element (PDMY), the thickness of the shear localization zone (\( h_{w} \)) can be approximately derived from Singh et al. (2000) and Brandenberg et al. (2003), whereas the low reference shear modulus (\( G_{r,\text{soft}} \)) is selected here as an example case to clarify its capacity to match the numerical results with the test data. Specifically, \( h_{w} = 0.5 \) m and \( G_{r,\text{soft}} = 90 \) kPa for PDS01, and \( h_{w} = 0.4 \) m and \( G_{r,\text{soft}} = 80 \) kPa for SJB03. Full parametric analyses of \( h_{w} \) and \( G_{r,\text{soft}} \) are presented later in this study to build a relationship between them for parameter pick. Other parameters of the soft interlayer element are consistent with the loose sand, except for the permeability coefficient (1.0 \( \times \) 10\(^{-7}\) cm/s as mentioned before). Also, due to nonliquefied features, the top layers of the Monterey sand in both tests are modeled using the same material as the clay.
(PIMY material). Other soil profiles, except for the soft interlayer element, are meshed into 0.5 m elements in depth, which is proven to be sufficient for wave propagation (Zhang et al. 2008). The piles are discretized, consistent with the soil meshes. The lateral width and out-of-plane thickness of the plain-strain soil element is set to equal the prototype of the centrifuge container.

![Diagram of centrifuge test setup](image)

**Fig. 4.** Centrifuge Test SJB03 layout (adapted from Brandenberg et al. 2003, with permission)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>PDS01</th>
<th>SJB03</th>
<th>SJB03</th>
<th>PDS01</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D_r (%) )</td>
<td>22</td>
<td>35</td>
<td>75</td>
<td>90</td>
</tr>
<tr>
<td>( (N_1)_{90} )</td>
<td>2.90</td>
<td>7.35</td>
<td>33.8</td>
<td>48.6</td>
</tr>
<tr>
<td>( G_s )</td>
<td>2.65</td>
<td>2.65</td>
<td>2.65</td>
<td>2.65</td>
</tr>
<tr>
<td>( \varepsilon_{\min} )</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>( \varepsilon_{\max} )</td>
<td>0.89</td>
<td>0.89</td>
<td>0.89</td>
<td>0.89</td>
</tr>
<tr>
<td>( \rho ) (t/m³)</td>
<td>1.92</td>
<td>1.94</td>
<td>2.03</td>
<td>2.07</td>
</tr>
<tr>
<td>( \varphi )</td>
<td>29.2</td>
<td>32</td>
<td>38</td>
<td>42</td>
</tr>
<tr>
<td>( f'_p ) (kPa)</td>
<td>80</td>
<td>80</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>( V_v ) (m/s)</td>
<td>130</td>
<td>151</td>
<td>209</td>
<td>227</td>
</tr>
<tr>
<td>( G_{\max} ) (MPa)</td>
<td>32.2</td>
<td>44.0</td>
<td>88.4</td>
<td>107</td>
</tr>
<tr>
<td>( G_{\max,oct} ) (MPa)</td>
<td>39.4</td>
<td>53.9</td>
<td>108</td>
<td>131</td>
</tr>
<tr>
<td>( \gamma_{\max} )</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>( \sigma )</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
</tr>
<tr>
<td>( B/G )</td>
<td>2.67</td>
<td>2.67</td>
<td>2.67</td>
<td>2.67</td>
</tr>
<tr>
<td>( B_r ) (MPa)</td>
<td>105</td>
<td>144</td>
<td>289</td>
<td>350</td>
</tr>
<tr>
<td>( d )</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>( c )</td>
<td>0.050</td>
<td>0.035</td>
<td>0.020</td>
<td>0.015</td>
</tr>
<tr>
<td>( d_1 )</td>
<td>0</td>
<td>0.2</td>
<td>0.6</td>
<td>0.8</td>
</tr>
<tr>
<td>( d_2 )</td>
<td>0</td>
<td>2</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>( Liq_1 )</td>
<td>10</td>
<td>10</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>( Liq_2 )</td>
<td>0.01</td>
<td>0.01</td>
<td>0.003</td>
<td>0</td>
</tr>
<tr>
<td>( Liq_3 )</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>( NYS )</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>( D_{10} ) (mm)</td>
<td>0.07</td>
<td>0.07</td>
<td>0.07</td>
<td>0.07</td>
</tr>
<tr>
<td>( k ) (cm/s)</td>
<td>( 1.4 \times 10^{-2} )</td>
<td>( 1.3 \times 10^{-2} )</td>
<td>( 7.9 \times 10^{-3} )</td>
<td>( 6.4 \times 10^{-3} )</td>
</tr>
</tbody>
</table>

**Table 1.** PDMY Parameters for Sand Constitutive Models of Centrifuge Tests

Not input parameters for constitutive models but, rather, for determining the constitutive parameters.

- Das (2015).
- Andersen and Schjetne (2013).
- Khosravifar (2012).
- Hardin and Drnevich (1972).
- Arulmoli et al. (1992).
- Permeability coefficient is the input parameter for the QuadUP element associated with the material property.

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Table 2. PIMY Parameters for Clay Constitutive Models of Centrifuge Tests

<table>
<thead>
<tr>
<th>Parameter</th>
<th>PDS01</th>
<th>SJB03</th>
<th>Description and reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta_s$ (kPa)$^a$</td>
<td>18</td>
<td>40</td>
<td>Undrained shear strength from test</td>
</tr>
<tr>
<td>$\rho_s$ (t/m$^3$)</td>
<td>1.58</td>
<td>1.65</td>
<td>Saturated density from test</td>
</tr>
<tr>
<td>$c_u$ (kPa)</td>
<td>15.6</td>
<td>34.6</td>
<td>Cohesion ($C_u = \sqrt{3\Delta_s}/2$)$^b$</td>
</tr>
<tr>
<td>$f'_{ce}$ (kPa)</td>
<td>100</td>
<td>100</td>
<td>Reference pressure of clay model$^b$</td>
</tr>
<tr>
<td>$G_{max,ce}$ (MPa)$^a$</td>
<td>12.6</td>
<td>28.0</td>
<td>Maximum shear modulus ($G_{max,ce} = 700S_u$)$^c$</td>
</tr>
<tr>
<td>$G_{max,ocr}$ (MPa)</td>
<td>12.6</td>
<td>28.0</td>
<td>Octahedral ref. shear modulus ($G_{max,ocr} = G_{max,ce}$)$^b$</td>
</tr>
<tr>
<td>$\gamma_{max}$</td>
<td>0.1</td>
<td>0.1</td>
<td>Maximum shear strain of clay$^b$</td>
</tr>
<tr>
<td>$\theta_a$</td>
<td>0.41</td>
<td>0.41</td>
<td>Empirical poisson’s ratio of soft clay$^d$</td>
</tr>
</tbody>
</table>
| $(B/G)_c$                                      | 5.22  | 5.22  | Bulk modulus to shear modulus ratio $\{(B/G)_c = (2+\theta_c)/[3(1-2\theta_c)]\}$)
| $B_{cr}$ (MPa)                                 | 65.8  | 146   | Pressure-dependent coefficient, zero means independent$^b$ |
| $d_c$                                          | 0     | 0     | Number of yield surfaces$^b$ |
| NYS                                            | 20    | 20    | Very low permeability for clay$^b$ |
| $\kappa_s$ (cm/s)$^p$                          | $1.0 \times 10^{-7}$ | $1.0 \times 10^{-7}$ | Not input parameters for constitutive models, but rather for determining the constitutive parameters.

$^a$Not input parameters for constitutive models, but rather for determining the constitutive parameters.


$^c$Chew et al. (1997).


$^p$Permeability coefficient is the input parameter for the QuadUP element.

Table 3. Pile Properties in FE Models

<table>
<thead>
<tr>
<th>Centrifuge case</th>
<th>Modulus [E (GPa)]</th>
<th>Inertia moment [$I(m^4)$]</th>
<th>Section area [$A(m^2)$]</th>
<th>Unit mass [t/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single pile-shaft</td>
<td>6.89</td>
<td>$4.55 \times 10^{-3}$</td>
<td>$7.49 \times 10^{-2}$</td>
<td>0.20</td>
</tr>
<tr>
<td>1 x 2 pile group</td>
<td>6.89</td>
<td>$4.55 \times 10^{-3}$</td>
<td>$7.49 \times 10^{-2}$</td>
<td>0.20</td>
</tr>
<tr>
<td>2 x 3 pile group</td>
<td>6.89</td>
<td>$2.25 \times 10^{-2}$</td>
<td>$1.66 \times 10^{-4}$</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Piles are modeled using an elastic beam-column column in both tests because the piles practically stay elastic during shaking. The pile-cap connections are treated as a zero-rotation restraint in the models. Table 3 lists properties of piles. Parameters of pile-soil springs are obtained according to soil properties using equations mentioned earlier. Based on the relative density of the sand, the reduction coefficients ($m_p$) are taken as 0.05 and 0.3 for $p-y$ springs in loose and dense sand, respectively (Brandenberg 2005). Note that the $p-y$ and $t-z$ springs at the embedded cap in the SJB03 test consider the dimension of the cap. Rayleigh damping with a mass proportional damping of 0.0 and stiffness proportional damping of 0.006 is adopted as numerical damping in the FE models according to the estimated frequencies of soil and pile structures in the centrifuge tests.

Numerical Results

Example time-history results for PDS01 and SJB03 tests are shown in Fig. 5. The input ground motions in the numerical models are large motions from the Kobe earthquake recorded at the container base. Soil responses in this study were recorded far away from the piles. As can be seen in the top panels in Fig. 5, the lateral spreading displacements in both tests are predicted reasonably well, even the oscillation response in most of the cycles, which indicates that the proposed FE model with the soft interlayer element is capable of predicting the global lateral spreading displacement. It is worth noting that, in PDS01 with a slope of 1.8°, the end-shaking displacements (both the recorded and predicted ones) were less than the maxima throughout the time history, whereas the SJB03 with a higher slope (3°) exhibited an accumulated larger residual lateral spreading displacement. This phenomenon indicates that the soft interlayer element in lower sloping ground did not trigger the residual displacement as large as the maxima, whereas that in a relatively higher sloping ground did, in which a larger lateral static force was assigned on the clay crust. Further experimental studies are required to determine the physical mechanism. Also, a comparative case without consideration of the soft interlayer element (treated as loose sand) was plotted together with the predicted result from the FE model with the soft interlayer element. Obviously, the case without the soft interlayer element reproduced a weaker oscillation of displacement response during the strong excitation period (i.e., approximately 10 s). Another interesting difference was the residual lateral displacement of the clay crust; that is, the comparative cases (without soft interlayer elements) can only produce accumulated lateral spreading displacement regardless of lower or higher sloping condition, which results in an overestimation in the lower sloping ground and a severe underestimation in the higher sloping counterpart. In addition, it is worth noting that the low reference shear modulus for the interlayer soil element approximately represents a fully liquefied condition at the beginning of shaking. This may be part of the reason why stronger oscillations of displacement response are reproduced (zoom-in view in Fig. 5) at the initial moments when the pore-water pressures begin to build up.

As for the shear localization effect, Fig. 6 presents the recorded and predicted soil column deformation of SJB03 for representation. The recorded data are derived from the end-shaking deformation shape of the paper sheet embedded in the soil before shaking. The predicted result agrees quite well with the recorded counterpart, indicating the soft interlayer element can also approximately capture the local shear deformation between the loose sand and the overlying clay crust.

Again, as seen in Fig. 5, the FE model–computed excess pore pressure (Expwp) ratios coincided reasonably well with the records. However, the predicted results did not fully capture the pronounced dilation spikes (i.e., transient drops), which may be partly because a constant permeability coefficient was used in the model throughout the entire time history, and because the pressure–dependent model in OpenSees is known to underestimate the coefficient of volumetric compressibility (Howell et al. 2015). Also, the oscillations of the clay crust displacement seem to influence the dilatant features of the sand; that is, the drops in pore-water pressure occurred when

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strong dynamic oscillations took place in the displacement of the clay crust. Again, it had remarkable correlation with the acceleration response of the clay layer. Specifically, as can be seen in Fig. 5(a), an overestimated displacement oscillation of the clay crust was reproduced by the FE model of PDS01 at the first visible cycle (i.e., approximate 5–6 s), which corresponded to an overestimation of the acceleration response of the clay during this short period. After that, some acceleration spikes were also observed when stronger dynamic oscillations were predicted. In the case of SJB03 [Fig. 5(b)], the dynamic FE model reproduced a slightly weaker oscillation of displacement response at approximately 9 s, which correlated with the slight underestimation of acceleration response in clay.

Fig. 5 also presents the time histories of the maximum bending moment of the single pile, two-pile group, and six-pile group. The maximum bending moment occurred at the loose sand bottom for the single pile and the pile head for the pile groups. Predicted results for the single pile [Fig. 5(a)] agreed reasonably well with the recorded counterpart, except for an underestimation at the first negative peak epoch and the stronger oscillations after 20 s. The former might be due to the unpredicted negative $r_s$ in loose sand, and the latter was because of the slightly overestimated displacement oscillation of the clay crust during that period. As for the two-pile group, although the predicted epoch of maximum peak bending moment was not consistent with the recorded one, a quite-good prediction was reproduced for the first 11 s when main shaking occurred. The underestimation of the bending moment at the recorded epoch of the maximum peak (approximately 12 s) was due to the unpredicted dilatancy in the loose sand as well as the underestimated oscillation of the clay displacement. Again, a slight overestimation of the bending moment oscillation was reproduced during the end shaking. For the bending moment of the six-pile group in SJB03 [Fig. 5(b)], the predicted maximum peak bending moment was reasonably consistent with the recorded one, although an obvious underestimation result was observed at approximately 8 s due to the underestimation of displacement oscillation in the clay crust. To clarify the capability of different numerical methods, Fig. 5(b) also plots the numerical results from Brandenberg et al. (2013), in
which the recorded data of soil displacement and pore-water pressure were used as seismic input in the adopted Winkler model. Apparently, the predictive uncertainties of soil responses from numerical models were isolated in their study. As a result, a generally good prediction was achieved [dashed line in Fig. 5(b)], although an apparent deviation was observed during the period of 13–20 s. In addition, a representative structural displacement response, the pile cap of the six-pile group, is plotted in Fig. 5(b). In general, the FE model captured most of the displacement response of the cap, with a bit underestimation of the residual response, which might be partly correlated with the slight deviation of the end-shaking displacement of the clay crust.

Fig. 7 exhibits the comparison of recorded and predicted bending moment distribution for the envelope circumstance throughout the time history and at the maximum peak epoch. Note that both the predicted and recorded epochs were presented for the two-pile group in the PDS01 test due to their inconsistent occurrences. As can be seen from Fig. 7, the bending moment envelopes of different pile structures agreed reasonably well with the recorded ones, although slight deviations were observed at deep depths. As for the maximum peak epochs, the FE model of the single pile reproduced an accurate result. For the two-pile group, the predicted bending moment distribution at the predicted maximum epoch [approximately 8 s, solid line and circles in Fig. 7(b)] agreed quite well with the recorded counterpart, whereas the distribution at the recorded maximum epoch (approximately 12 s, dash line and circles) exhibited some deviations near the pile head. In addition, Fig. 7(c) shows a reasonable consistency between the predicted and recorded bending moment distribution of the six-pile group. In general, the proposed FE model is capable of estimating the bending moment distribution of piles in liquefied and laterally spreading ground considering shear localization.

**Parametric Analysis**

Although the aforementioned example cases reasonably reproduced the seismic demand properties of the lateral spreading clay crust as well as the pile structures, it is better to present an available range of \( h_w \) and \( G_{s,soft} \) for the convenience of parameter selection. To this end, a parametric analysis was performed to obtain the available range for these two parameters. Specifically, \( h_w \) varies from 0.2 to 0.8 m with an interval of 0.1 m. For each \( h_w \), different values of \( G_{s,soft} \) with an interval of 10 kPa are assigned as potential cases. The maximum bending moment and soil lateral spreading displacement of the single pile in PDS01 and the six-pile group in SJB03 are selected as engineering demand parameters (EDPs). Considering the simplification of the efficient FE model in this study, a deviation ratio within 10% is selected as the criterion (i.e., the absolute differences of the recorded and predicted EDPs divided by the records fall into 0–10%).

Figs. 8 and 9 illustrate the outcomes of the parametric analysis. Circles indicate the cases satisfying the 10% deviation criteria (both the maximum bending moment and soil lateral displacement), whereas the inclined crosses represent the cases with deviation ratio(s) beyond 10%. Correspondingly, conservative available ranges are plotted as shaded area with exponential curves for boundaries in both tests. In general, available \( G_{s,soft} \) increases with the increase of \( h_w \). For further convenience, the available...
areas are gathered together to regress an exponential relationship between $G_{r,\text{soft}}$ and $h_w$, which is illustrated in Fig. 10

$$G_{r,\text{soft}} = 40 e^{1.5 h_w}$$  \hspace{1cm} (7)

It is worth noting that Eq. (7) is applicable to experimental or site conditions similar to the centrifuge tests in this study—that is, gently sloping ground with nonliquefiable crust overlying liquefiable sand that leads to lateral spreading. For circumstances with obvious differences (e.g., soil ground with a very large sloping degree), special care should be taken when using this equation. Application of the proposed FE model with the soft interlayer element begins with a preliminary evaluation of the thickness of the shear localization zone. For an experimental study, the thickness can be obtained from physical observation, whereas for engineering practice, $h_w$ can be assumed by empirical judgment in the proposed range (i.e., pick a value from 0.1 to 0.9 m), and corresponding $G_{r,\text{soft}}$ can be determined based on Eq. (7). In this manner, a reasonable prediction can be achieved with a deviation ratio below 10% for both the critical soil and structural responses. Based on the simplified FE model in this study, further fragility analysis of both soil and piles in geotechnical engineering considering soil liquefaction can be performed with computational efficiency.

**Summary and Limitations**

An efficient multiscale FE dynamic model (3D structure, 2D soil, and 1D pile soil spring) was developed in OpenSees and validated against a series of centrifuge tests involving piles in gently sloping soil profiles that result in lateral spreading. In the numerical model, a soft interlayer element with recommended thickness and low reference shear modulus was developed to approximately simulate the shear localization zone at the interface of clay and loose liquefied sand layers. It is worth indicating that the low reference shear modulus may be theoretically impacted by multiple factors (e.g., relative density of the loose sand, thickness and density of the overlying clay crust, and the ground sloping degree). In this regard, additional experimental studies are required to reveal the impact of these variables.

In general, good agreements are achieved between numerical predicted and test-recorded responses of both the piles and soil. Disparities between the predicted and recorded responses may be caused by the uncertainty of input parameters in the soil constitutive models and the simplified numerical approach (e.g., omission of the clay cracking during ground oscillation, real 3D soil-pile interactions but with 1D spring treatment). In addition, the input low reference shear modulus for the interlayer soil element actually represents a fully liquefied condition at the beginning of shaking. In this regard, an updated soil constitutive model involving soil stress–dependent reference shear modulus may require an additional study.

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