NUMERICAL MODELING OF LIQUEFACTION UNDER SLOPED GROUND CONDITION USING PM4SAND AND UBCSAND MODELS

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ABSTRACT

Numerical simulations of a centrifuge test, in which dissipation patterns, lateral spreading, and shear strain localization were measured and recorded, are performed for elaboration on the numerical modeling approach of dynamic effective stress analysis. In this paper, two constitutive models, PM4Sand and UBCS Sand, which are considered most suited for such an application, are calibrated based on available laboratory data and is described in detail. The models’ performance as compared to the centrifuge test results is evaluated. The analyses shows that the processes of pore pressure dissipation and void redistribution observed in the centrifuge test were successfully computed with PM4Sand. UBCSand shows good agreement only for shake 10, where void redistribution is less pronounced. On the other hand, for stronger motion, a deep-seated failure mechanism is computed which is not in agreement with the experimental evidence.

Keywords: Lateral spreading; Void-ratio redistribution; Shear-strain localization; PM4Sand; UBCSand.

1. INTRODUCTION

Since 1986, relatively small earthquakes have been recorded near producing gas fields in the provinces of Groningen, Drenthe and Noord-Holland and in northern Germany. Initially, the relationship to gas production was not clear. Also the associated consequences were expected to be limited. A multidisciplinary study was initiated by the Ministry of Economic Affairs in the early 1990’s, which was guided by a Scientific Advisory Committee (SAC) that focused on the relationship between gas production and earthquakes. It was concluded that the observed earthquakes were indeed of nontectonic origin and most likely induced by reservoir depletion (i.e. gas production). There has been a recent renewed focus on, and widespread attention for, the issue of seismicity induced by gas production in Groningen. Studies concluded that the uncertainty associated with the earthquake hazard in the Groningen field was larger than previously thought. It was recognized as well that the earthquakes are not just a nuisance but could pose a potential safety risk as well (NAM, 2015).

Among the critical infrastructure in Groningen area are levees, which are part of the Dutch water defense system. In order to better assess the liquefaction resistance of those critical infrastructure, dynamic effective stress finite element analyses are more frequently used. In this regard, the role of initial static shear stress (sloping-ground conditions) on the undrained cyclic behavior of sandy soils has been long investigated. The conclusion is that initial static shear stress has a significant effect on the liquefaction resistance of sandy soils, in conjunction with the earthquake-induced cyclic shear stress, the soil relative density, confining pressure, among other factors. Moreover, the presence of low permeable layers overlaying liquefiable soils may lead to major consequences due to the effect of void redistribution. Numerical studies of void redistribution and associated shear localization have included nonlinear one dimensional site response analysis by Yang and Elgamal (2002) and Seid-Karbasi and Byrne(2007). Kamai and Boulanger (2013) presented numerical simulation results using the PM4Sand in FLAC software (Itasca, 2009).

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The purpose of this paper is to elaborate on and evaluate the applicability of two constitutive models, PM4Sand and UBCSand, on reproducing liquefaction induced lateral-spreading and void redistribution as observed in centrifuge tests and closely measured (Marinucci et al., 2008). The centrifuge model (SSK01) consisted of two mild slopes separated by a channel in the middle and one of the slopes was treated with drains. The comparison presented here includes displacements, (excess) pore-pressures, shear strains, and post-liquefaction behavior.

2. CASE STUDY

The purpose of the centrifuge test (SSK01) was to evaluate the effectiveness of geosynthetic drains for mitigating liquefaction of sands (Marinucci et al., 2008). A flexible shear beam container, supported by 9-m radius centrifuge, was used to carry the soil specimen. The test was performed at UC Davis at a centrifugal acceleration of 15g, and the measurements are presented hereafter in equivalent prototype units.

The soil profile was composed of two symmetrical slopes of 3 degrees towards a central channel. A 1m thick layer of clayey silt overlays 5m thick layer of liquefiable sand (Nevada Sand with Dr=40%). Plastic drains were installed on one side of the centrifuge container prior the dry pluviation of the sand. The model was then subjected to five successive shaking events consisted of 20 uniform sinusoidal cycles at 2 Hz with single amplitude of 0.01, 0.03, 0.07, 0.11 and 0.3g, named Shake 8, 9, 10, 11 and 12. Two vertical instrumentation arrays of accelerometers and pore pressure transducers were installed on each side to track wave propagation and soil-water consolidation. Displacement transducers were also installed into the clay crust to measure both horizontal and vertical displacements. All experimental tests results are described in Kamai et al. 2008.

It is worth noticing that, prior to shaking, the model was spun up to a centrifugal acceleration of 22.5g to produce an over-consolidation ratio (OCR) of 1.5 on the clay crust. This OCR increased also the sand’s cyclic strength and coefficient of earth pressure.

Numerical simulation of the centrifuge test was performed by Kamai and Boulanger (2013) using FLAC, where the constitutive model PM4Sand (version 2 - Boulanger and Zioutopoulou, 2012) describes the behavior of liquefiable sand. Individual and in-sequence simulations were performed and compared to experimental results. On average, results from the individually run simulations and the in-sequence simulations gave comparable results in terms of the dynamic response and final deformations.

Differences in results between the two approaches were observed mostly in the cumulative effects of void redistribution, with the in-sequence simulation better illustrating the progressive loosening of
sand beneath the clay crust. One of the outcomes of that work was that the use of pore pressure dissipation together with updating the mesh (large deformation effects) during the dynamic phase was essential for obtaining reasonable simulation results. Conversely, the simulation without pore fluid flow did not predict the density redistribution observed in the tests and thus, underestimates the lateral displacements.

3. NUMERICAL MODEL

The finite difference model and the model geometry are shown in Figure 2. The computational mesh is the same as the one already used by Kamai and Boulanger (2013). The properties of the container, made of a sequence of aluminum and rubber rings, were already calibrated to have the correct ratio container/soil masses and the correct fundamental frequency of the container. The parameters of the elastic moduli were determined by Armstrong (2010). The Mohr-Coulomb material model is assigned to the clayey-silt crust with sig-4 hysteretic damping (Itasca, 2016), and the hysteretic parameters chosen to approximate the modulus reduction and damping curve for clays with PI=15 according to Vucetic and Dobry (1991).

![Figure 2: FLAC mesh, showing material models, location of drains and water table height.](image)

The prefabricated vertical drains on the drain-treated side are simulated by fixing the pore pressure to the hydrostatic value at all nodes of the drainage column. This approximation will produce an overestimation of the effect of drainage, as it doesn’t take into account on the hydraulic resistance of the drain itself and it neglects the three-dimensional spacing. However, this modelling approach is still sufficient for the purpose of the simulation, as it is focusing mainly on the non-treated side.

The sequence of construction is simulated in the model. The soil is progressively constructed in layers under gravity at the value of 1/15g. During this phase, a Mohr-Coulomb constitutive model is used for the sand with appropriate stress-dependent stiffness. A very soft beam is also placed between the soil and the container to prevent water flow into the container. A frictional interface is applied to the elements at the boundary between the container and the soil, allowing for relative displacements: friction angle of 10 degrees during construction and 23 degrees during shaking. Water pressure is applied within the central channel to all nodes below the water table. During shaking, this boundary condition is updated periodically, assuming that the water table elevation remains constant. The use of the elastic constitutive model for the static condition produces unrealistic stress concentrations near the toe of the slope; for this reason, the coefficient of earth pressure at rest ($K_0$) is constrained to be between 0.3 and 0.8. The bottom nodes of the mesh are restricted for vertical movement. During construction, the container is fixed horizontally and vertical displacements are allowed; during the dynamic phase, the container nodes are free horizontally but they are slaved to each other.

For the dynamic phase, two separate time history accelerations are used, composed of 20 uniform sinusoidal cycles at 2 Hz with single amplitude of 0.07g – 0.11g, named Shake 10 and Shake 11 in the following. A velocity time history derived from the acceleration time history is then applied to the base of the model and a Raleigh damping is set to 0.5% at a center frequency of 2 Hz.

After setting all initial conditions, two constitutive models are used to describe the behavior of the sand: PM4Sand (Boulanger and Ziotopoulou, 2015) and UBCSand (Beaty and Byrne, 2011).
Description and calibration of the models are discussed later in the paper.

### 3.1 PM4Sand

The PM4Sand constitutive model is a bounding surface plasticity model (Boulanger and Ziotopoulou, 2015) that follows the basic framework proposed by Dafalias and Manzari (2004). The model has a narrow stress-ratio elastic cone and three other surfaces: the bounding, dilation and critical state. The location of the bounding and dilation surfaces depends on the state of the soil, and they move towards the critical state surface as the soil is sheared towards critical state. This model is able to take into account the effect of fabric change during the dilative phase of shearing, which is shown to be indispensable to have realistic description of soil response in reverse loading. A series of modifications to the original version of Dafalias and Manzari (2004) model were introduced (version 1, version 2 and version 3) to improve its ability to approximate the trends observed in geotechnical earthquake engineering. In particular, the updated formulation of version 3 of this model is shown to better approximate liquefaction behavior for sloping ground and irregular cyclic loading conditions.

There are three primary parameters that are the most important for the calibration and a secondary set of 17 parameters that may be modified from their default values only in special cases. The three parameters are: the shear modulus coefficient $G_0$, which is calibrated to match the in-situ shear wave velocity; an apparent relative density $D_r$, which controls the dilatancy and stress-strain response characteristics, and the contraction rate parameter $h_{p0}$, which adjusts contraction rates and can be adjusted to obtain a target cyclic resistance ratio.

The calibration of this constitutive model has been performed already in Kamai and Boulanger (2013) using data of Nevada sand. Calibration involved the critical state line, the shear wave velocity profile, and modulus degradation and damping ratio curves. Three sets of undrained cyclic tests were also used to determine the liquefaction triggering curves which were corrected to the values of OCR and $K_0$ expected in the prototype (OCR=1.5 and $K_0$=0.56). The criterion adopted for liquefaction is 3% shear strain single amplitude under constant cyclic stress ratio and no initial static shear stress bias. Two curves that bound the laboratory data were finally used for calibration of PM4Sand, named Case A (the upper) and Case B (the lower). The calibration has been repeated against the same data using the latest version of PM4Sand (version 3) and the updated set of parameters is listed in Table 1.

#### Table 1: Soil properties for PM4Sand

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Loose Sand</th>
<th>Dense Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative density $D_r$ [%]</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td>Minimum void ratio $e_{\text{min}}$</td>
<td>0.486</td>
<td></td>
</tr>
<tr>
<td>Maximum void ratio $e_{\text{max}}$</td>
<td>0.793</td>
<td></td>
</tr>
<tr>
<td>Constant volume friction angle $\varphi_{cv}$</td>
<td>32</td>
<td></td>
</tr>
<tr>
<td>Specific gravity $G_s$</td>
<td>2.65</td>
<td></td>
</tr>
<tr>
<td>Bolton’s constant R</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Bolton’s constant Q</td>
<td>9.5</td>
<td></td>
</tr>
<tr>
<td>Shear wave velocity at 1 atm $V_{s1}$ [m/s]</td>
<td>175</td>
<td>188</td>
</tr>
<tr>
<td>Contraction rate parameter $h_{p0}$</td>
<td>0.25 and 0.035*</td>
<td>0.5</td>
</tr>
<tr>
<td>Shear modulus coefficient $G_0$</td>
<td>685</td>
<td>821</td>
</tr>
<tr>
<td>Constant that adjust the ratio of plastic modulus to elastic modulus $h_0$</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Permeability [m/s]</td>
<td>$6.0 \times 10^{-5}$</td>
<td>$2.0 \times 10^{-5}$</td>
</tr>
</tbody>
</table>

* case A and case B, respectively
3.2 UBCSand

The UBCSand model is a nonlinear stress-dependent effective stress plasticity model that captures the build-up of excess pore pressure. The model describes the shear stress-strain behavior of the soil using a given hyperbolic relation, and the volumetric response is computed by a flow rule, based on Rowe’s stress dilatancy hypothesis.

The model was originally developed by Beaty and Byrne (1998) and then modified to improve the prediction for cases with significant static bias. It is important to point out that several versions of UBCSand currently exist (e.g. Shirro and Bray, 2013; Giannokou et al. 2011; Armstrong et al., 2013), and that the number of parameters and model calibration may vary depending on the version considered. The version of this model available for FLAC as a user defined soil model is UBCSand-904aR (Beaty and Byrne, 2011), and this is the version used in this paper.

The elastic component of response is assumed to be isotropic and specified by a shear modulus, $G^e$, and a bulk modulus, $B^e$, as follows

$$G^e = k^e_G p_a \left( \frac{\sigma'}{p_a} \right)^{ne}$$

$$B^e = \alpha_{BG} G^e = k^e_B p_a \left( \frac{\sigma'}{p_a} \right)^{me}$$

Where, $k^e_G$ is the elastic shear modulus number and depends on density, $p_a$ is the atmospheric pressure, $ne$ and $me$ are constants and $\alpha_{BG}$ is a factor that depends on the Poisson’s ratio as follows:

$$\alpha_{BG} = 2.0(1+\nu)/(3(1-2\nu));$$

The bulk modulus ratio $k^e_B$ can be computed as $\alpha_{BG} k^e_G$.

For plastic response, plastic strains are controlled by the yield surface and flow rule. The plastic shear strain increment $d\gamma^p$ is related to the change in shear stress ratio $d\eta$, where $\eta$ is the current shear stress ratio $\eta = \tau / \sigma'$, where $\tau$ is the shear stress and $\sigma' = 1/2(\sigma_n + \sigma_v)$. A hyperbolic relationship between the plastic modulus $G^p$ and the shear stress ratio $\eta$ is used.

$$d\gamma^p = (G^p / \sigma')^{-1} d\eta$$

$$G^p = G^p_i (1 - R_f \frac{\eta}{\eta_f})^2$$

The value of $\eta_f$ is the stress ratio at failure, which equals to the sine of the peak friction angle $\varphi_f$. The factor $R_f$ is the failure ratio, less than 1, used to truncate the hyperbolic relationship. The plastic modulus at low level of stress $G^p_i$ is computed as

$$G^p_i = k^p_G p_a \left( \frac{p}{p_a} \right)^{np} f(hfac_1, hfac_2, n_{cyc})$$

where $k^p_G$ is the plastic shear modulus number, $np$ is a constant, $hfac_1$ is a factor that controls the number of cycle to trigger liquefaction and $hfac_2$ is a factor that modifies the shape of pore pressure rise with number of loading cycles $n_{cyc}$.

The non-associated plastic flow rule is defined as follows:
\[ \text{de}_v^p = (\sin \varphi_{cv} - \eta) \text{dy}^p \]  
where \( \varphi_{cv} \) is the constant-volume friction angle. Plastic volumetric strains are contractive for \( \eta < \sin \varphi_{cv} \) and dilative for \( \eta > \sin \varphi_{cv} \). Hence, in undrained shear loading, the model can capture the increase and decrease of effective stress caused by volumetric strains resulting from contraction and dilation.

The constant-volume friction angle is set to the critical state friction angle. The peak friction angle is a function of state, which can be estimated consistently to Bolton’s correlation between dilatancy and state (Bolton, 1986). For a relative density of 40% and a range of initial mean effective stress of \( p'/p_a \) between 0.1 and 0.7, the calculated \( \varphi_f \) is approximately 37 degrees. The shear wave velocity is used to calibrate \( k_G^e \) as follows:

\[ k_G^e = \frac{(\rho_{sat} V_s^2 / p_a)}{\left[ (1.0 + K_v) / 2.0 \right]^{0.5}} \]  

The factor \( R_f \) is set to 0.6 and the ratio \( k_G^e / k_G^c \) is used to capture the liquefaction triggering curve at 1 atm pressure and no static shear bias.

A summary of the soil parameters for UBCSand is listed in Table 2. The values of \( \varepsilon_{\min}, \varepsilon_{\min}, \varphi_{cv}, V_s, G_s \) and permeability are the same as those used for PM4Sand in Table 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Loose Sand</th>
<th>Dense Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>corrected SPT value</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( N_{1,60} )</td>
<td>7.36</td>
<td>29.44</td>
</tr>
<tr>
<td>Constant ( me )</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Constant ( ne )</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.15</td>
<td></td>
</tr>
<tr>
<td>Constant ( np )</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>Elastic shear modulus number ( k'_G )</td>
<td>693</td>
<td>848</td>
</tr>
<tr>
<td>Elastic bulk modulus number ( k'_B )</td>
<td>759</td>
<td>902</td>
</tr>
<tr>
<td>Ratio ( k'_G / k'_B )</td>
<td>0.5 and 0.28*</td>
<td>2.67</td>
</tr>
<tr>
<td>Peak friction angle ( \varphi_f )</td>
<td>37</td>
<td>42</td>
</tr>
<tr>
<td>Failure ratio ( R_f )</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>Fitting parameter ( hfac ) at ( \sigma' = 1 \text{atm} )</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Fitting parameter ( hfac2 )</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

* case A and case B respectively

An example of the cyclic undrained response of UBCSand model is shown in Figure 3 under constant cyclic shear stress ratio and no static shear stress bias. The model builds up excess pore pressure during cycles. As soon as the stress state starts cycling along the failure surfaces (A) developing the so-called “banana loops”, the corresponding stress-strain behavior (B) follows a repeated hysteretic loop, independently of the number of cycle. In case of no initial static shear stress and for certain sets of parameters, the threshold of 3% shear strain may not be overcome. For this reason, only in case of
non-static bias, the criterion adopted for liquefaction for UBCSand model is based on the excess pore pressure ratio \( r_u = \Delta u / \sigma_{v,0}' \) instead of shear strains, and the threshold is set to 0.95. In this case, the original experimental data were re-processed and corrected to OCR=1.5 and \( K_0=0.56 \) to determine the equivalent liquefaction triggering curves based on \( r_u \), as shown in Figure 4.

\[
\frac{\Delta u}{\sigma_{v,0}'} = \frac{\Delta u_{\text{ref}}}{\sigma_{v,0}'} = 0.95
\]

Figure 3: Example of cyclic response of UBCSand.

Figure 4: Triggering calibration for UBCSand- number of cycles required to reach 95% of excess pore pressure ratio \( r_u \) under constant cyclic stress ratio (CSR): laboratory data for loose Nevada Sand shown in open symbols; calibrated simulations results shown in full symbols; the two curves represent a power fit bounding the data.

4. RESULTS AND COMPARISON

4.1 Element test response

Figure 5 shows the overburden correction factor \( K_\sigma \) and the static shear stress correction factor \( K_\alpha \), obtained by running several undrained cyclic simple shear element tests. The factor \( K_\sigma \) is much larger than the correlation of Idriss and Boulanger (2008) for the stress range lower than 1 atm, typical of this centrifuge test. Larger values are computed for Case B. The values of \( K_\alpha \) are computed using the cyclic strength that reaches 3% strain threshold in 15 cycles under a coefficient of earth pressure at rest \( K_0=0.56 \). The results are compared against the empirical relationship by Boulanger (2003). The agreement for \( \alpha = 0 - 0.1 \) is quite satisfactory, but as the static bias increases, the response of PM4Sand starts deviating, with a general tendency of being stronger than the empirical relationship. All aforementioned single-element tests are simulated using UBCSand by changing the parameter \( hfac \) in order to match the response of PM4Sand. The relation incorporates a dependency on both vertical stress and static bias as follows:

\[
hfac_i = hfac_{i,\text{latm}} \times a \times \max \left( \frac{\sigma_i'}{\sigma_{\text{latm}}'} - 0.1 \right)^{b_\alpha} + d \times \alpha
\]

(10)
with \( a_n \), \( b_n \) and \( d_n \) are 0.9905, -0.229 and -1.1 for case A, and 0.9434, -0.387 and -1.5 for case B. It is worth noticing that, even though large values of shear static bias are expected in a limited portion of the model close to the toe of the slope, the static bias contribution on \( h f a c_i \) is found to be important to capture the co-seismic displacements.

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Figure 5: the overburden correction factor \( K_{\sigma} \) and the static shear stress correction factor \( K_{\alpha} \).

### 4.1 Co-seismic response

The comparison of simulations’ results, using both constitutive models, and measurements from centrifuge tests is presented hereafter. Two time-history events (shake 10 and shake 11) are run individually, and the results are shown in the following. The comparison focuses only on the non-treated side part of the model, which is susceptible to liquefaction.

Excess pore pressure ratio \( r_{u,\text{max}} \) contours are plotted in Figure 6 for the PM4Sand. In all cases, the largest \( r_{u,\text{max}} \) is computed in the liquefiable sand layer, beneath the clay crust in the non-treated area. As expected, as the liquefaction resistance decreases (Case B), the region of relevant excess pore pressures (e.g. \( r_{u,\text{max}} > 0.7 \)) expands towards the center and the bottom of the model. The same behavior is observed in case of shake 11.

The results of UBCSand are shown in Figure 7. For shake 10, the distribution of \( r_{u,\text{max}} \) is similar to the one computed by PM4Sand. The region with \( r_{u,\text{max}} > 0.7 \) is only a bit larger, and it extends to the bottom of the model and includes also part of the channel area. Significant differences are observed in case of shake 11, where very large values of \( r_{u,\text{max}} \) are computed also underneath the channel, changing the predicted failure mechanism. Both constitutive models compute volume change \( \Delta \varepsilon_{\nu} \) during shaking. The results are shown in Figure 7 (right) only for shake 11 where the largest \( \Delta \varepsilon_{\nu} \) are computed. Densification is observed in the treated area, due to the presence of drains, and loosening is computed in the non-treated area beneath the clay crust. The latter is the effect of void redistribution. Analyses with PM4Sand give larger (positive) \( \Delta \varepsilon_{\nu} \) with a tendency to increase with the intensity of the shaking. It is worth noticing that during the simulation the soil strength computed by PM4Sand depends on the current value of void ratio. Therefore, a significant volume increase beneath the clay crust forms a weak plane in the system, susceptible to failure. This phenomenon cannot be captured by UBCSand model, where the strength is independent of any variation of void ratio.
Shear strains are plotted in Figure 8 for both shaking events. In the graph, the iso-line $\gamma = 3\%$ is shown, with the intention to identify the region where significant strains occur. Large strains are developed beneath the clay crust in the non-treated side. As the intensity of the shaking event increases, the size of this liquefiable area becomes deeper. PM4Sand model shows a more superficial failure zone, with larger shear strains concentrated at the boundary with the clay crust; this can be a consequence of the void redistribution in combination with a void-ratio-dependent model. Conversely, UBCSand shows a deeper and subcircular failure mechanism. Unexpected large strains in the treated part of the liquefiable layer are computed also with UBCSand model in case of shake 11.

The evolution of excess pore pressure in time is shown in Figure 9 at two location U-34 and U37, located respectively at the base and in the middle of the liquefiable sand layer, both on the non-treated side of the model (see Figure 2). In general, both models are able to correctly capture the trend of pore pressure build-up in time; however, PM4Sand shows better agreement against the experimental results.
Lateral displacements and surface settlements are illustrated in Figure 10 and Figure 11. The response of PM4Sand tends to accurately predict the experimental results in all simulations. UBCSand shows good agreement only for shake 10 whereas, for the stronger motion, the surface settlements tend to be over predicted. An additional simulation is performed using UBCSand model with flow disabled, where pore pressures do not dissipate (Figure 10). In this case, the surface settlement is still in agreement with the experimental results but the horizontal displacement is severely underestimated. This confirms that also for UBCSand model, at least for shake 10, pore pressure dissipation during shaking is essential to correctly simulate the key mechanisms in the tests.

Figure 9: Shake 10 - Measured and computed pore pressure time histories at two different depths: U37 corresponds to $\sigma'_{v0} \sim 27.2$ kPa and U32 to $\sigma'_{v0} \sim 53$ kPa.

Figure 10: Shake 10- Displacement time histories on the non-treated side: test results versus simulations.

Figure 11: Shake 11 - Displacement time histories on the non-treated side: comparison of the centrifuge test with simulations.

Figure 12: Total Stress Simulation. PM4Sand – Case A. Incremental displacements computed after shake 11.
### 4.3 Post-earthquake final displacement

Liquefaction-induced deformation in practice depends on the estimations of residual shear strength. A total stress analysis is performed to evaluate post-shaking final displacements and the empirically based residual strength is applied to the elements expected to liquefy. The relation proposed by Weber (2015) was used to estimate the residual strength as function of initial effective stress and the corrected equivalent clean-sand SPT value $N_{1,60,cs}$ as follows:

$$S_r (\text{lb/ ft}^2) = \exp \left( 0.1407 \cdot N_{1,60,cs} + 4.2399 \cdot \frac{\sigma_{r,0}}{\sigma_{v,0}}^{0.12} (\text{atm}) \right) - 0.43991 \left( N_{1,60,cs}^{1.45} + 0.2 \cdot N_{1,60,cs} \cdot \frac{\sigma_{v,0}}{\sigma_{r,0}}^{0.48} (\text{atm}) + 41.13 \right)$$

(10)

The shear modulus of the liquefied zone is taken as 50 times the residual strength, and the bulk modulus was assigned a value equal to 100 times the shear modulus. The criteria used to identify the liquefiable area is the one suggested by Beaty and Byrne (2011), which is based only on excess pore pressure ratio ($r_{v,max} > 0.7$). An example of post-earthquake horizontal displacements after the simulation using PM4Sand is shown in Figure 12. For all the analyses presented in this paper, the contribution of the post-earthquake displacements is significantly lower than the co-seismic one. The analyses conducted with UBCSand model produce lower post-earthquake displacements compared to PM4Sand ones, due their larger co-seismic displacements and rotations.

### 5. CONCLUSIONS

Numerical simulations of a centrifuge test, in which void redistribution and lateral spreading were measured and recorded, have been performed for comparing the performance of two constitutive models: PM4Sand and UBCSand. Both models were calibrated against the same set of experimental data. UBCSand model required additional calibration in order to give similar response to PM4Sand at different stress levels and in presence of initial static bias.

The processes of pore pressure dissipation and void redistribution observed in the centrifuge test were successfully computed with PM4Sand. UBCSand shows good agreement only for shake 10, where void redistribution is less significant. Conversely, for stronger motion, a deep-seated failure mechanism is computed which is not in agreement with the experimental evidence. Finally, pore pressure dissipation during the seismic event is found to be essential also for UBCSand model to correctly simulate the key mechanisms in the tests.

### 6. ACKNOWLEDGMENTS

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