Effect of field drainage on seismic pore pressure buildup and K_σ under high overburden pressure

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Abstract

The paper studies the effect of a high effective overburden pressure (σ'_{v0} = 6 atm) under two drainage conditions, on the field liquefaction behavior of saturated Ottawa sand. A series of eight centrifuge experiments with relative density, D_r = 45% and 80% and base shaking are considered that include a 5-m saturated sand layer under a pressure of either σ'_{v0} = 1 or 6 atm (~ 100 and 600 kPa). Four of the tests had single drainage at the top of the layer (SD), while the other four tests had double drainage (DD) at top and bottom. The four SD test results had been reported before, while the four DD tests are new. A novel centrifuge technique was developed to achieve the double drainage boundary condition of two pervious boundaries at the top and bottom of the sand layer, using geocomposite at the bottom. Measured responses are compared at the same σ'_{v0} between SD and DD tests having the same input acceleration, and also between SD and DD tests where the shaking induced a similar maximum excess pore pressure ratio, $(r_u)_{max} \approx 0.8$. These comparisons include acceleration time histories, excess pore pressure time histories and profiles during and after shaking, and stress ratio and shear strain time histories. Comparisons between corresponding tests

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at 1 and 6 atm revealed significantly more partial drainage at 6 atm than at 1 atm, with even more significant variation in excess pore pressures in the DD than in the SD tests. Best estimates of field overburden pressure correction factors at 6 atm, $K_\sigma = (CRR)_6 / (CRR)_1$, were obtained from the centrifuge results with two independent methods for a failure criterion of $(r_u)_{max} = 0.8$. Those $K_\sigma = 1.2$ to $1.3 > 1.0$ for both SD and DD drainage conditions, due to the significantly lower compressibility of the sand at 6 atm. These results further emphasize the important role partial drainage may play in the field during shaking at high σ'_{v0} on the excess pore pressures and values of K_σ.

Introduction

Liquefaction of saturated sand continues to be a main research topic in geotechnical engineering, as liquefaction and related soil failure cause enormous damage during earthquakes. The main procedure used in practice to evaluate liquefaction potential is the Simplified Method, originally proposed by Seed and Idriss (1971). In the current state of practice (SOP), liquefaction triggering is evaluated with field liquefaction charts based on the Simplified Method. These charts estimate the soil liquefaction resistance from the field penetration resistance, using either the Standard Penetration Test (SPT), or cone penetration test (CPT), or, alternatively, from field shear wave velocity measurements (V_s). The charts have been typically calibrated by earthquake case histories with or without liquefaction. A number of these charts have been proposed by Robertson and Wride (1998); Andrus and Stokoe (2000); Youd et al. (2001); Cetin et al. (2004); Idriss and Boulanger (2006, 2008, 2010); Boulanger and Idriss (2012); Dobry and Abdoun (2017); and Zimmaro et al. (2019).

An example is the liquefaction chart of Fig. 1 for clean sands, based on the CPT and normalized to an effective overburden pressure, $\sigma'_{v0} = 1$ atm (~100 kPa), developed by Idriss and

Boulanger (2008). Figure 1 and similar charts have been calibrated by earthquake case histories where the liquefiable sand layer was under an effective overburden pressure less than or around 2 atm (~200 kPa) (Dobry and Abdoun 2015). On the other hand, there are field projects like tall embankment dams and other impoundments, where the effective overburden pressure may be significantly larger than 2 atm, with $\sigma'_{v0} \approx 6$ or 8 atm, or even 10 atm (Gillette 2013).

Cyclic undrained laboratory tests show that the liquefaction resistance needs to be corrected to account for the effect of overburden pressure (Seed and Idriss 1981). Seed (1983) defined the overburden pressure factor (K_σ) as the ratio between cyclic resistance ratio (CRR) at a high confining pressure, σ'_{v0}, to the CRR at $\sigma'_{v0} = 1$ atm, and proposed using this factor to correct the CRR at 1 atm. That is:

$$K_\sigma = \frac{(CRR)_{\sigma'_{v0}}}{(CRR)_1} \tag{1}$$

where $(CRR)_{\sigma'_{v0}}$ and $(CRR)_1$ are the cyclic resistance ratios in the critical liquefiable layer under $\sigma'_{v0} > 1$ atm and $\sigma'_{v0} = 1$ atm, respectively. Seed (1983) also showed that in cyclic undrained laboratory tests, K_σ decreases when the consolidation pressure increases.

After 1983, a number of researchers proposed K_σ curves of K_σ versus σ'_{v0} based on undrained cyclic results (Harder 1988; Seed and Harder 1990; Vaid and Thomas 1995; Vaid and Sivathayalan 1996; Hynes et al. 1999; Youd et al. 2001; Boulanger 2003; Boulanger and Idriss 2004; Idriss and Boulanger 2008; Montgomery et al. 2012; Dobry and Abdoun 2015). Currently, the two most popular State-of-Practice (SoP) methods for K_σ estimation are those of Youd et al. (2001) and Boulanger and Idriss (2008). Invariably, all undrained results as well as the Youd et al. (2001) and Boulanger and Idriss (2008) curves give values of $K_\sigma < 1$ for $\sigma'_{v0} > 1$ atm and $K_\sigma > 1$ for $\sigma'_{v0} < 1$ atm, with K_σ decreasing as σ'_{v0} increases. The K_σ curves from Youd et al. (2001) and Boulanger and Idriss (2008) are quite different at high confining pressures (Abdoun et al. 2020).

The National Research Council of the National Academies (NRC) recently stated that *"Some adjustment factors are not well constrained over the entire range of engineering interest by the empirical data (e.g., the stress magnitude adjustment factor, K_σ)… and these adjustment factors should be developed using experimental data (including centrifuge and shaking table experiments) and engineering mechanics principles…Additional data and research are needed to allow better understanding of these effects."* (National Academies 2017). The National Academy is not restricting their interpretation of K_σ to purely undrained conditions but defining it as a correction factor to extend the use of the field liquefaction charts to high overburden pressures, including the possibility of partial drainage in the field during shaking.

Considering the wide variation between the K_σ curves used in the SoP, as well as the NRC recommendation of additional research, Ni et al. (2020) conducted a series of four centrifuge models of idealized field conditions at Rensselaer Polytechnic Institute (RPI), under both low and high overburden pressures (1 and 6 atm). In these centrifuge experiments, a 5 m sand prototype layer with a free drainage boundary at the top, having two different relative densities (Dr = 45% and 80%), was subjected to base shaking (Table 1). Ni et al. (2020) analyzed the pore pressure responses of the sand layer under different conditions (effective overburden pressure and relative density) and also calculated the field overburden pressure factors, $K_\sigma = 1.28$ for loose sand and $K_\sigma > 1.15$ for dense sand. These values are in conflict with the SoP K_σ less than 1.0. Abdoun et al. (2020) analyzed the reasons for these higher K_σ values in the centrifuge tests, and found that much more significant and faster drainage had occurred in the 6 atm tests compared with 1 atm, both during and after shaking. The coefficient of consolidation, c_v, during the dissipation phase after shaking, was further evaluated with three different methods that used the pore pressure and settlement records. Both the c_v and drained constrained volumetric stiffness of the sand, M', values at 6 atm were found to be 2~4 times greater than at 1 atm. Moreover, these results together with other information from the literature, suggested that c_v and M' may increase proportionally to $\sqrt{\sigma'_{v0}}$. To validate further the conclusions by Ni et al. (2020) and Abdoun et al. (2020), a second set of four centrifuge models were conducted at RPI at 1 and 6 atm to simulate the same 5m prototype sand layer of Ottawa sand, having a relative density of 45%, but changing the drainage conditions. The previous four centrifuge models corresponded to single free drainage at the top of the sand and an impervious base (SD). The new four centrifuge models were performed under double drainage conditions (DD), with free drainage at both the top and bottom of the sand layer (Table 2).

Experimental Program

A new series of four centrifuge tests were conducted at Rensselaer Polytechnic Institute (RPI), under low and high effective overburden pressures of 1 and 6 atm, and the same relative density of 45%. The centrifuge test configurations were similar in every respect to the original four tests in Table 1, except for the drainage boundaries. In these new centrifuge experiments – listed in Table 2 - all models had free drainage boundaries at both the top and bottom of the sand layer (DD), as compared to bottom impervious boundary and a top free boundary (SD) in the four centrifuge tests of Table 1. The detailed model configurations of the new DD models are shown in Fig. 2.

Model layout

The new centrifuge models had four distinct layers. From bottom to top: geocomposite layer at the base, saturated liquefiable sand layer, saturated transition coarse sand thin layer, and top dry lead shot layer. The only difference from the previous centrifuge models reported by Ni et al. (2020) was the additional geocomposite layer underneath, connected to vertical geonet strips placed around the sand layer to achieve double drainage. All other three horizontal layers on top of the geocomposite (sand, transition and lead shot), were exactly the same as before, with the same materials, functionalities and building methodologies described by Ni et al. (2020). Therefore, it is only necessary to discuss here the details of the new geocomposite layer and vertical geonet strips, as detailed under the next heading.

The building of the centrifuge models consisted of the following five general steps: (i) assemblage of the circular laminar box; (ii) placement of the circular rubber membrane inside the laminar box; (iii) placement and saturation of the bottom circular geotextile layer and attached vertical rectangular geonet strips; (iv) placement and saturation of the sand and transition layers; and (v) placement of dry lead shot layer.

Geocomposite Layer

The geocomposite layer was built with GSE DuraFlow 330 Geocomposite from Solmax (Houston, TX). This layer functioned as a free drainage boundary at the bottom of the saturated sand layer during earthquake shaking. The chosen geocomposite is composed of 8.4 mm-thick DuraFlow geonet and nonwowen needle-punched geotextile on one side (Fig. 3a).

Design logic of Geocomposite

i) Retention criteria

Retention criteria ensures that the geotextile voids are small enough to prevent the migration of the sand into the geocomposite by retaining the sand particles. The Retention ability of geotextile is checked by a representative size of soil particles and the apparent opening size of geotextile, given by Eq. 2 (Reddi 2003):

$$AOS \leq BD_{85(soil)} \tag{2}$$

where B is a function of the filtered soil properties (soil type, density, uniformity etc.), geotextile properties and flow conditions. B is a dimensionless factor in the range between 0.5 and 2. Based on the recommended values for B in different situations from Reddi (2003), B = 1.0 was chosen for the project; AOS is the Apparent Opening Size of the geotextile, and AOS = 0.212 for the selected geocomposite product (GSE Environmental 2015). D_{85} is particle size of the filtered soil at which 85% of the particles are finer, about 0.3 mm for the Ottawa F65 sand (EI-Ghoraiby et al. 2017). Based on the above, the retention criterion was achieved for the selected GSE DuraFlow 330.

ii) Permeability Criteria

To enable the pore fluid to drain vertically down from the saturated sand to the geocomposite without significant buildup of excess pore pressures at the boundary and in the geocomposite layer, the geocomposite should have a significantly higher permeability than the filtered soil, as specified by Eq. 3 (Reddi 2003).

$$k_{geo} \geq Ck_{soil} \tag{3}$$

where k_{geo} is the permeability of the geocomposite, which is 1.26 cm/sec for the GSE DuraFlow 330 (GSE Environmental 2015); k_{soil} is the permeability of the sand layer, which is 0.012 cm/sec for a relative density of 45% (EI-Ghoraiby et al. 2017); and *C* is a dimensionless coefficient in the range from 1 to 10 based on the importance and severity of the problem. For the selected geocomposite product, the permeability criterion was met even when considering the upper limit of C =10, as the permeability of the geocomposite is much higher than that of the Ottawa sand (1.26 > 10 * 0. 012 = 0.12).

iii) Transmissivity requirements

Based on our centrifuge model configuration, the excess pore fluid first drains down vertically from the sand to the geocomposite layer, then flows horizontally inside the geocomposite to the vertical geonet strips, and then flows up vertically inside these vertical strips. The horizontal fluid flowing velocity in the geocomposite layer is controlled by transmissivity. The higher the transmissivity value of the geocomposite is, the better the bottom drainage will be.

In some of the centrifuge tests, the geocomposite layer had to play this role under a high overburden pressure somewhat in excess of 6 atm. Given that the transmissivity generally decreases with increase in overburden pressure, the geocomposite selected had to have a high transmissivity under high normal load. The selected GSE DuraFlow 330 has a transmissivity of 50 cm^2/sec under 720 kPa (≈ 7 atm), as per GSE Environmental (2015).

Bottom drainage construction

The previous section described the design concepts related to the geocomposite selection. This section presents additional details on how a freely draining horizontal geocomposite layer and attached vertical geonet strips were built at the bottom of, and around the sand layer.

Figure 3 presents the detailed design of the bottom drainage using geocomposite and geonet strips. Figure 3a shows the piece of geocomposite with circular shape that constituted the bottom drainage layer. Figure 3b shows an example rectangular geonet strip, placed vertically and evenly all around the laminar container body to provide the vertical drainage path up from the bottom geocomposite layer. These rectangular geonet strips were covered with tapes to prevent any horizontal drainage directly between the sand and the surrounding strips. Figure 3b shows rectangular geonet pieces before and after covering with tapes. Figure 3c presents the assembling of the bottom circular geocomposite pieces and the vertical geonet pieces. The bottom circular pieces of geocomposite were thoroughly saturated with the viscous fluid of 20cp or 45cp, depending on the test, with this fluid viscosity being consistent with that of the viscous pore fluid used later for sand saturation after the sand pluviation. Figure 3d shows the assembled bottom gecomposite drainage layer after saturation, and after connecting it to the vertical geonet strips.

After placing and assembling the geocomposite at the bottom and all around the container, the other three distinct layers (sand layer, transition layer and the leadshot layer), were built in the same way described by Ni et al. (2020) for the single drainage experiments. Also, the sand layer and the transition layer were saturated following the same procedure described by Ni et al. (2020). Finally, the completed centrifuge models were subjected to 1-D shaking sinusoidal base shakings.

Experimental results for D_r = 45% and comparisons between single drainage and double drainage tests at 1 and 6 atm

The eight centrifuge tests listed in Tables 1 and 2 include six experiments of relative density, D_r = 45%, with σ'_{v0} = 1 and 6 atm, having either SD or DD conditions. Those six tests are the focus of this section, and they are grouped together in Table 3, where the input motions listed correspond to the input peak base acceleration measured inside the container. The maximum excess pore pressure ratio, $(r_u)_{max}$, in all DD tests were obtained at the mid-depth of sand layer, while the $(r_u)_{max}$ in the SD tests were measured at the bottom depth, as expected based on their different drainage conditions. The six experiments of Table 3 provide the opportunity to conduct additional comparisons and discussions on the combined effect of σ'_{v0} and drainage conditions on the results. This is done systematically throughout this section. In all tests of Table 3, the original intent was to reach in each test a target maximum pore pressure ratio, $(r_u)_{max} \approx 0.8$, so that values of K_σ associated with this failure criterion, $(r_u)_{max} = 0.8$, could be obtained directly from comparable 1 and 6 atm tests. However, when the same input motion previously used in the SD test was applied to the corresponding DD model, the measured value of $(r_u)_{max}$ in the DD test was much less than 0.8 due to the increased partial drainage during shaking. This was the case for Tests 45-1 (DD) – 0.045g and 45-6 (DD) – 0.3g in Table 3, which measured maximum pore pressure ratios of 0.48 and 0.18, significantly less than the 0.8 target. Therefore, these two DD experiments were repeated with larger input accelerations of 0.065g and 0.5g, respectively, reaching values of $(r_u)_{max}$ of 0.68 and 0.85, much closer to the 0.8 target.

Therefore, the experiments of Table 3 allow evaluating the effects of drainage conditions on the results using two different types of comparison: 1) comparison between pairs of SD and DD tests having the same input motion; and 2) comparison between pairs of SD and DD tests having different input motions but a similar $(r_u)_{max} \approx 0.8$. These comparison are presented under the next two headings for centrifuge experiments performed at both 1 atm and 6 atm.

The comparisons under the next two headings include acceleration time histories, excess pore pressure buildup, and pore pressure dissipation. Comparisons of excess pore pressure profiles for all six tests are presented later herein, together with other results. All presented data are in prototype units unless stated otherwise.

Comparison of SD and DD tests having the same input motion

This section presents comparisons of acceleration and excess pore pressure time histories for SD and DD tests having the same input motion.

Acceleration time histories

Figure 4 shows the comparison of measured acceleration time histories at different depths inside the saturated sand layer. Figure 4a includes measurements from the 1 atm SD and DD tests with a common input motion of 0.045g. Similarly, Fig. 4b includes measurements from the 6 atm SD and DD tests with a common input motion of 0.3g. The black curves correspond to the SD experiments, first introduced by Ni et al. (2020), while the red curves correspond to the new DD tests. The labels in Fig. 4 indicate the relative depth of the accelerometer buried in the soil within the layer (top, middle and bottom of the sand layer).

Figure 4 indicates that for these tests having the same input motion, the acceleration responses of the layer at different depths were quite similar, and more or less independent of the difference in drainage condition at the bottom of the sand layer, with some deviations observed at shallow elevations. This finding is valid for both 1 atm and 6 atm tests. Test 45 – 1 (SD) – 0.045g and Test 45 – 6 (SD) – 0.3g experienced degradation of acceleration at shallow elevations near the top of the sand layer, which was not observed in the double drainage tests (Test 45 – 1 (DD) –

0.045g and Test 45 – 6 (DD) – 0.3g), probably due to the lower pore pressure buildup in the DD experiments (Table 3).

Figure 4 shows that some of the findings originally reported for the single drainage tests by Ni et al. (2020), still hold in the double drainage tests at both low and high overburden pressure. For example, when contrasting the acceleration time histories at the bottom and top of the sand layer in Fig. 4, the amplification and de-amplification phenomena were similar in the SD and DD models. Specifically, amplification of acceleration with increasing height above the bottom within the sand layer happens in the two 1 atm tests corresponding to SD and DD conditions, while de-amplification with height occurs in the two SD and DD 6 atm tests. However, there is one slight difference between the SD and DD 1 atm tests: amplification of accelerations held at all times during shaking in the DD test ($(r_u)_{max}$ = 0.48), while amplification occurred only in the first several cycles in the SD test, followed by de-amplification afterwards ($(r_u)_{max}$ = 0.8). The reason for the de-amplification afterwards in the SD 1 atm test is most probably related to the high $(r_u)_{max}$ = 0.8, that caused stress-strain degradation in the sand, compared with the much lower $(r_u)_{max}$ = 0.48 in the DD 1 atm tests, with much less stress-strain degradation.

Excess pore pressure buildup

Figure 5 presents the comparison of excess pore pressure ratio time histories, r_u, at different depths within the saturated sand layer. Figure 5a includes measurements from the 1 atm SD and DD tests with a common input motion of 0.045g. Figure 5b includes measurements from the 6 atm SD and DD tests with a common input motion of 0.3g. Pore pressure ratio, r_u, is defined as the ratio between excess pore pressure, u, and initial effective overburden pressure, σ'_{v0}. The color codes for the curves are consistent with Fig. 4.

The comparisons in Fig. 5 show that: in the SD tests the excess pore pressure ratio, r_u, increased with depth within the sand layer, with the maximum excess pore pressure ratio, $(r_u)_{max}$, happening at the bottom of the layer. On the other hand, in the DD tests $(r_u)_{max}$ occurred at mid-depth of the layer. That is, r_u decreased in the double drainage tests when the location was farther from the mid-depth and closer to the free drainage boundaries at the top and bottom. These trends are as expected, and are confirmed later herein when discussing the excess pore pressure profiles measured in the SD and DD tests.

Figure 5 confirms what was already clear from the values of $(r_u)_{max}$ in Table 3: DD tests subjected to a comparable input motion built up much less r_u than SD tests. This reduction of $(r_u)_{max}$ due to the added bottom drainage boundary is much more significant at 6 atm than at 1 atm. (The $(r_u)_{max} = 0.8$ is reduced to $(r_u)_{max} = 0.48$ at 1 atm; while the $(r_u)_{max} = 0.76$ is much more radically reduced to $(r_u)_{max} = 0.18$ at 6 atm.) This indicates that the significant effect on pore pressure buildup of the partial drainage due to a high overburden pressure – already noticed by Ni et al. (2020) for the SD tests - is even more pronounced for DD conditions.

Comparison of SD and DD tests having a similar $(r_u)_{max} \approx 0.8$

This section presents comparisons of acceleration time and excess pore pressure time histories for SD and DD tests having a similar $(r_u)_{max} \approx 0.8$, rather than the same input motion.

Acceleration time histories

Figure 6 has the same format of Fig. 4, providing comparisons of measured acceleration time histories at three different elevations within the sand layer: bottom, middle and top. The labels in Fig. 6b have been omitted as the curves correspond to the same elevations as Fig. 6a. The black curves correspond to the same SD tests shown before in Figure 4: they are Test 45 – 1 (SD) –

0.045g in Fig. 6a, and Test 45 – 6 (SD) – 0.3g in Fig. 6b. The magenta curves in Fig. 6 constitute the only difference with Fig. 4, as these magenta curves correspond now to DD tests under a stronger input motion than their SD counterparts. As shown in Table 3 and discussed earlier, all tests in Fig. 6 have consistent maximum excess pore pressure ratios of $(r_u)_{max} \approx 0.8$, but with a stronger input acceleration required for this in the DD tests, at both 1 atm and 6 atm. The percentage increase in input acceleration to achieve this target of $(r_u)_{max} \approx 0.8$, was 45% to 67%.

Since the input acceleration was 45% to 67% stronger in the DD tests, the measured acceleration records at the bottom of the soil in Fig. 6 are also consistently higher for the DD than for the SD tests. However, at the middle and top elevations, the acceleration measurements at 1 atm are similar for SD and DD tests, while at 6 atm, the acceleration amplitudes are always greater in the DD than in the SD test at all depths.

Excess pore pressure buildup

Figure 7 has the same format of Fig. 5. The magenta curves in Fig. 7 are the only difference with Fig. 5, as they correspond now to DD tests under stronger input acceleration than the SD tests. Specifically, Fig. 7 displays the excess pore pressure ratio, r_u, over time for four tests with a similar recorded $(r_u)_{max} \approx 0.8$, see Table 3. Thus, Fig. 7 shows the effect of drainage conditions and overburden pressure for cases of similar $(r_u)_{max} \approx 0.8$.

Ni et al. (2020) stated that in the SD tests, the maximum excess pore pressure, $(r_u)_{max}$, was measured at the bottom of the layer, with r_u decreasing much faster when going from deep to shallow elevations in the 6 atm test (Test 45 – 6 (SD) – 0.045g), compared to the 1 atm test (Test 45 – 1 (SD) – 0.3g). Figure 7 indicates that this finding remains valid in the DD experiments. The difference is that in the DD experiments, this more significant decrease in excess pore pressure ratio occurs from the middle to the bottom and from the middle to the top drainage boundaries, instead of only from deep to shallow elevations toward the single top drainage boundary. Specifically, in the two DD experiments of Fig. 7 (Test 45 – 1(DD) – 0.065g and Test 45 – 6 (DD) – 0.5g), the r_u at mid-depth happened to be higher for 6 atm than for 1 atm (0.85 versus 0.68). Despite this, the r_u values at the bottom and top elevations for 6 atm were both smaller than the corresponding r_u values at the same elevations for 1 atm (0.38 versus 0.5 near the top, and 0.2 versus 0.3 near the bottom of the layer), as shown by the comparison of magenta curves in Figs. 7a and 7b.

<u>Other experimental results and comparisons</u>

This section presents two other sets of comparisons between the tests of Table 3, again showing the influence of SD versus DD for the same σ'_{v0}, as well as the effect of changing σ'_{v0} from 1 to 6 atm for the same drainage conditions. Instantaneous excess pore pressure profiles (isochrones) during both shaking and dissipation, as well as shear strain and shear stress ratio time histories during shaking are compared for all six experiments of Table 3.

Excess pore pressure profiles

Figure 8 presents the instantaneous excess pore pressure profiles (isochrones) at four different times during shaking and dissipation for the three tests performed at 1 atm. The results allow examining the effect of the drainage conditions for both common input (Test 45 – 1(SD) – 0.045g and Test 45 – 1(DD) – 0.045g in Figs. 8a and b), as well as common $(r_u)_{max} \approx 0.8$ (Test 45 – 1(SD) – 0.045g and Test 45 – 1(DD) – 0.065g in Figs. 8a and c). In each plot, the curves with solid data points represent the instantaneous excess pore pressure profiles during shaking ($0 < t \leq 5$ sec), and the green curve with open data points corresponds to the profile at a time during dissipation, that is after the shaking ($t > 5$ sec). The data points are measurements of pore pressure transducers at different depths in the layer. Each plot also includes the total vertical overburden pressure line in dashed black color, and the effective vertical overburden pressure solid line, with this last line indicating r_u = 1.0 and hence liquefaction. None of the tests reached liquefaction, with $(r_u)_{max}$ = 0.80, 0.48 and 0.68 in the three tests (Table 3). In the plots, z = 0 means the very top of the sand layer, which is a free drainage surface in both SD and DD tests. For the single drainage Test 45 – 1(SD) – 0.045g, the isochrones have a shape roughly similar to a quarter sine curve, with an excess pore pressure, u = 0 for z = 0 at all times, validating the assumption that z = 0 is a free drainage boundary (Ni et al. 2020). The isochrones of the double drainage tests in Figs. 8b and c show that:

i) The maximum excess pore pressure at a given time occurred at or near mid-depth at all times during and after shaking, because the center elevation has the longest drainage path in these tests with double drainage condition. This is different from the single drainage test in Fig. 8a, where the maximum pore pressure at a given time was always at the bottom of sand layer, also having the longest drainage path in this scenario;

ii) u = 0 at both the surface and bottom depths of the sand layer close to the top and bottom drainage boundaries, during and after shaking.

iii) The shapes of all isochrones at any time during and after shaking are close to the idealized excess pore pressure profiles of 1-D consolidation model with double drainage condition, following approximately half sine distributions (Holtz et al. 2011). In fact, the shapes of the top half of the isochrones from 0 m to 2.2 m in Figs. 8b and c are very similar to the full isochrone shapes in Fig. 8a. This validates the effectiveness of the double drainage design in the centrifuge experiments that used geocomposite under the bottom of the sand layer;

iv) The isochrone slopes at mid-depth of the layer were always close to vertical, indicating about zero hydraulic gradients at that location. The maximum gradients occurred at elevations close to the top and the bottom.

Figure 9 presents the instantaneous excess pore pressure profiles (isochrones) at four different times during shaking and dissipation for the three tests done at 6 atm. That is, Fig. 9 is the exact counterpart of Fig. 8 for 6 atm. All observations discussed before for Fig. 8 are also applicable to Fig. 9, including findings i) ~ iv) above. However, one important difference is that the drop in $(r_u)_{max}$ when going from Fig. 9c to Fig. 9b (0.85 to 0.18), is much greater than the same drop when going from Fig. 8c to Fig. 8b (0.68 to 0.48), see Table 3. This is again a demonstration of the increased importance of partial drainage in depressing pore pressure buildup as σ'_{v0} increases.

In all six plots of Figs. 8-9, the blue curves correspond to the time of the $(r_u)_{max}$ of the test, which happened at the bottom in the SD experiments of Figs. 8a and 9a, and at mid-depth in the DD experiments of Figs. 8b, c and 9b, c. In the SD tests, the time of $(r_u)_{max}$ was always around 5 sec (end of shaking), for both 1 atm and 6 atm, as previously reported by Ni et al. (2020). However, in the four DD tests, the time of $(r_u)_{max}$ was always less than 5 sec. In these DD tests, it was ~ 4.5 sec, that is slightly before the end of shaking for the 1 atm tests in Figs. 8 b, c; and 1.5 to 3.5 sec or much earlier in the shaking for the 6 atm tests of Fig. 9b, c. This systematic difference between SD and DD experiments was clearly caused by the additional drainage surface at the bottom, compounded in the case of Test 45-6(DD)-0.3g in Fig. 9b by the relatively low shaking intensity and corresponding low $(r_u)_{max} = 0.18$.

The dissipation of excess pore pressures after shaking was found to be significantly faster for the DD tests at 6 atm than at 1 atm. This is consistent with the greater partial drainage during shaking at 6 atm discussed above. It is also consistent with the similar conclusions reached by Ni et al. (2020) for dissipation during the SD tests.

Shear Stress Ratio and shear strain time histories

Figure 10 shows the shear stress ratio and shear strain time histories of the loose sand models tested (Dr = 45%), for the different drainage boundary conditions and different overburden pressures, including all six tests listed in Table 3. Specifically, Figs. 10a and 10c present stress and strain data for the three 1 atm experiments, while Figs. 10b and 10d display the same information for the three 6 atm experiments. In Fig. 10, the black dash-dot curves correspond to the SD tests at 1 and 6 atm, while the blue and red solid curves present the data for the DD tests. The shear stress and shear strain time histories were obtained with the System Identification (SI) technique (Elgamal et al. (1995, 1996); Zeghal et al. 1995), that uses the acceleration records at different depths inside a centrifuge model. Shear stress ratio was defined as the ratio between shear stress τ, from System Identification (SI) and the initial effective overburden pressure, σ'_{v0}, at the same depth as the shear stress, that is, τ / σ'_{v0}. All stress ratio and shear strain curves in Fig. 10 correspond to the depth of measured $(r_u)_{max}$; bottom depth in SD tests and middle depth in DD tests.

As stated by Ni et al. (2020) for the SD experiments, the shear stress peaks of Test 45 – 1 (SD) – 0.045g degraded after the first several cycles due to the high excess pore pressure build up ($(r_u)_{max}$ = 0.8); this is shown by the black curve in Fig. 10a. Such degradation was not observed in any of the DD experiments at 1 and 6 atm, not even for the DD tests that also had a similar high excess pore pressure of $(r_u)_{max} \approx 0.8$. With respect to the six strain time histories in Fig. 10c and 10d, five of them show no clear increase or decrease with time. The sixth, Test 45 -6(DD)-0.5g, shows a possible increase in the cyclic strain during the shaking.

Table 4 lists representative values of the Cyclic Shear Strain (γ_c) for the six tests of Table 3, calculated as the median of all positive and negative peaks from the 10 cycles in the strain time histories of Figs. 10c and 10d. Table 4 also include the γ_c and $(r_u)_{max}$ for the two SD tests conducted by Ni et al. (2020) at D_r = 80%. All γ_c and $(r_u)_{max}$ for SD tests in the table were already reported by Ni et al. (2020). On the other hand, the γ_c for the DD tests in Table 4 were obtained using the same procedure by the authors for this paper. The γ_c values in Table 4 indicate that the strain level increased from SD to DD conditions in order to reach a similar $(r_u)_{max} \approx 0.8$. This is valid at both low and high overburden pressures (γ_c increases from 0.13% to 0.18% at 1 atm, and from 0.23% to 0.54% at 6 atm).

Overburden Pressure Correction Factor, K_σ

As mentioned earlier and following the original definition of Eq. 1 for σ'_{v0} = 6 atm, the overburden correction factor is defined as $K_\sigma = (CRR)_6 / (CRR)_1$, where $(CRR)_6$ and $(CRR)_1$ are obtained respectively at high and low confining pressure, from undrained stress-controlled cyclic tests (triaxial or simple shear), and for a certain number of cycles to failure, N. This original definition is entirely based on undrained cyclic testing, and was labeled by Ni et al. (2020) as the laboratory undrained K_σ.

The same general approach based on Eq. 1 was used by Ni et al. (2020) to obtain values of field K_σ from the centrifuge CSR values for the single drainage tests at D_r = 45% and 80%. This took advantage of the cyclic stresses backfigured from the experiments using System Identification (SI). This field K_σ = 1.28 from Ni et al. (2020) for D_r = 45% is reproduced here in Table 4, where it is characterized as "CRRs in Eq. 1 directly from shear stresses using SI". This field K_σ = 1.28 is associated with a failure criterion, $(r_u)_{max}$ = 0.8. A similar K_σ > 1.15 was also obtained by Ni et al. (2020) for the SD, D_r = 80% tests, and this result is also listed in Table 4. Further details on the corresponding procedure and calculations are provided by Ni et al. (2020). The same method was used here by the authors for double drainage Tests 45-6(DD)-0.5g and 45-1(DD)-0.065g, giving $K_\sigma = (CRR)_6 / (CRR)_1 < 1.30$, included in Table 4.

These two field K_σ just discussed for D_r = 45% and SD and DD conditions (1.28 and < 1.30), as well as the field K_σ > 1.15 also obtained by Ni et al. (2020) for the single drainage D_r = 80% tests, are all above 1.0, contrary to the results from undrained tests reflected in the SoP, which are invariably less than 1.0 and decrease with overburden pressure. As discussed by Ni et al. (2020) and Abdoun et al. (2020) for single drainage tests, and confirmed here for the new DD centrifuge experiments, this is due to the decreased compressibility of the sand at 6 atm, which increases the significance of partial drainage during shaking under 6 atm compared with 1 atm. On the other hand, some of these field K_σ determined directly from Stress Ratio histories such as Fig. 10a and 10b, involve an inequality sign. An example is the $K_\sigma < 1.30$ in Table 4 for DD conditions. This is due to the fact that in several of the centrifuge tests with $D_r = 45\%$ and 80%, $(r_u)_{max}$ was different from the target value 0.8, so upper or lower bounds are obtained for the CRRs and K_σ, rather than more exact values.

Therefore, the authors decided to pursue also an alternative method to evaluate K_σ from all eight SD and DD tests of $D_r = 45\%$ and 80% listed in Table 3. This alternative method still uses the ratio of CRRs in Eq. 1, but takes advantage of shear strain time histories from SI such as those in Fig. 10c and 10d, in order to remove these inequalities and provide a more precise estimate of the field K_σ associated with the centrifuge experiments. This is possible because the plots of pore pressure ratio versus cyclic shear strain in both cyclic undrained laboratory tests and centrifuge model experiments, tend to be more unique than corresponding plots based on cyclic shear stress or stress ratio, lending themselves better to interpolation and extrapolation (Dobry and Abdoun 2015, 2017). The method is described below, with the new calculated K_σ also listed in Table 4 for the SD and DD tests, and characterized there as "CRRs in Eq. 1 from shear strains using SI". Both new calculated values in Table 4 are similar ($K_\sigma \approx 1.2$), and also generally consistent with the values obtained before directly from the shear stresses. The next two sections explain how these alternative field $K_\sigma = 1.18$ and 1.20 values in Table 4 were evaluated using available information from the corresponding centrifuge tests at 1 and 6 atm.

Relationship between γ_c and $(r_u)_{max}$ in centrifuge tests

Dobry and Abdoun (2015) proposed a relation between $(r_u)_{max}$ and cyclic shear strain (γ_c) based on a series of six large-scale and centrifuge tests on 6m uniform layers of loose saturated sand, previously reported by Abdoun et al. (2013). These six large-scale and centrifuge experiments had imposed a 10-cycle uniform base input sinusoidal shaking to develop an excess pore pressure buildup below liquefaction triggering. A clean sand and a silty sand as well as two methods of sand deposition were used in these tests, where the effective overburden pressure at mid-depth of the layer was σ'_{v0} = 0.24 atm. The measured values of $(r_u)_{max}$ versus γ_c from the six tests plotted within a narrow band, which is reproduced here as the shaded band of Fig. 11.

Ni et al. (2020) tabulated the values of $(r_u)_{max}$ and γ_c for the four SD centrifuge tests listed in Table 1, corresponding to sand of D_r = 45% and 80%. These values of $(r_u)_{max}$ and γ_c are reproduced here in Table 4 and are plotted as data points in Fig. 11. Two curves – associated with σ'_{v0} = 1 and 6 atm - were passed by these four data points in Fig. 11, with both curves being parallel to the middle trend of the shaded band containing the results of Abdoun et al. (2013).

It is useful to review the way in which the representative γ_c was obtained for each SD test in Table 4 by Ni et al. (2020). The System Identification (SI) technique (Elgamal et al. 1995, 1996; Zeghal et al. 1995) was applied to determine the shear strain from the acceleration time histories measured at different elevations inside the saturated sand layer, like the shear strain time histories presented in Fig. 10 c, d herein. Each representative cyclic shear strain listed in Table 4, γ_c, was obtained by Ni et al. (2020) by taking the median value of the 10-cycle shear strain peaks, a procedure which had been proposed before by Dobry and Abdoun (2015). The $(r_u)_{max}$ in the four SD tests of Fig. 11 was measured at the bottom of the sand layer, close to the bottom undrained boundary, while $(r_u)_{max}$ from previous centrifuge and large-scale tests included in the band of Fig. 11, had been recorded at shallower elevations. Thus, cyclic shear strains for the four data points of the SD tests in Fig. 11 were evaluated at the depth where $(r_u)_{max}$ was measured.

The following observations are reached from inspection of the two SD curves in Fig. 11:

1) A greater γ_c is needed to reach the same $(r_u)_{max}$ when σ'_{v0} is higher;

2) Relative density, D_r, affects the $(r_u)_{max}$ versus γ_c relationship minimally or not at all. This is shown by the fact that a single $(r_u)_{max}$ versus γ_c curve could be passed by the data points of SD tests with different relative densities and same overburden pressure.

The authors obtained representative values of γ_c for the four new DD centrifuge tests listed in Table 2, from the shear strain time histories of Fig. 10, and utilizing the same exact procedure used before by Ni et al. (2020) for the SD tests. The corresponding four DD γ_c values of are also listed in Table 4, which now contains all values of $(r_u)_{max}$ and γ_c for the complete group of eight SD and DD centrifuge experiments. These eight pairs of $(r_u)_{max}$ and γ_c for SD and DD tests are plotted in Fig. 11.

That is, Fig. 11 contains all information available for D_r = 45% and 80% centrifuge tests done with SD and DD drainage conditions. Inspection of the graph reveals the following:

1) The four curves from the SD and DD experiments, each determined by the location of two data points, are indeed parallel to each other and parallel to the original shaded band of Fig. 11, independent of drainage condition and overburden pressure. This consistency between the shapes of the new curves with each other and with the Abdoun et al. (2013) band, serves as a confirmation of the hypothesis that curves are parallel in a plot such as Fig. 11, and reinforces the reliability of the curves;

2) Comparison of the curves of 6 atm and 1 atm in Fig. 11, confirms the conclusion, that a higher overburden pressure shifts the curve to the right for both SD and DD conditions, with a higher γ_c needed to build up the same $(r_u)_{max}$;

3) Comparison of DD and SD curves at the same overburden pressure, shows that a greater γ_c is needed to generate the same $(r_u)_{max}$ for DD compared to SD conditions. In fact, Fig. 11 reveals

that adding the bottom drainage boundary has roughly the same effect on the curve than increasing the overburden pressure from 1 atm to 6 atm.

The four $(r_u)_{max}$ versus γ_c curves in Fig. 11 were used to evaluate the γ_c needed to reach the target failure criterion of $(r_u)_{max} = 0.8$, with such γ_c defined as $\gamma_{cl,0.8}$. This $\gamma_{cl,0.8}$ corresponds to an hypothetical centrifuge experiment causing an $(r_u)_{max} = 0.8$. The value of $\gamma_{cl,0.8}$ for each curve in Fig. 11 was determined from the intersection with the curve of the horizontal line shown corresponding to $(r_u)_{max} = 0.8$. Table 4 lists these values of $\gamma_{cl,0.8}$ needed to reach $(r_u)_{max} = 0.8$ in all eight centrifuge tests having both SD and DD conditions. For the SD experiments, as the curve is independent of relative density, only one value of $\gamma_{cl,0.8}$ is listed in Table 4 for each overburden pressure (1 and 6 atm). For the DD experiments, as the curve is independent of base shaking intensity, also only one value of $\gamma_{cl,0.8}$ is listed in Table 4 for each overburden pressure (1 and 6 atm). Due to the fact that all four curves contain data points close to the target $(r_u)_{max} = 0.80$, the four $\gamma_{cl,0.8}$ in Table 4 are estimates having a high degree of precision

Strain-based method for CRR and field K_σ

The values of $(CRR)_6$ and $(CRR)_1$ needed to evaluate the field K_σ from pairs of centrifuge tests at 6 atm and 1 atm, may now be calculated from the corresponding $\gamma_{cl,0.8}$ for these tests listed in Table 4. This requires consideration of the shear stiffness characteristics of the sand models in the centrifuge experiments, so these cyclic shear strains $\gamma_{cl,0.8}$ may be converted into corresponding undegraded cyclic shear stresses, $\tau_{c,0.8}$, and CRRs, with K_σ finally evaluated using Eq. 1. The corresponding stiffness information for all eight centrifuge tests is plotted in Fig. 12 in the form of G/G_{max} versus γ_c and τ_c / G_{max} versus γ_c curves, with the corresponding values listed in Table 5. That is, Fig. 12 includes the modulus reduction and normalized backbone curves common to all eight centrifuge models. The way Fig. 12 was developed and its use in evaluating K_σ is explained in the rest of this section.

The information needed for the G/G_{max} curve of Fig. 12a was obtained as follows:

- The value of G_{max} for each centrifuge test was from bender element measurements at mid-depth of the layer (Fig. 2), of the shear wave velocity, V_s, conducted in flight before the main shaking. Once V_s was known, G_{max} was evaluated with the expression $G_{max} = \rho V_s^2$ (ρ = mass density of saturated sand). Ni et al. (2020) had already done this for the SD tests in Table 4; the process was repeated by the authors for the new DD tests.
- Once G_{max} had been determined for each centrifuge test, the modulus reduction values and curves of G/G_{max} versus γ_c of Fig. 12a could be obtained. The secant shear modulus, G, at various γ_c, was determined from the cyclic stress-strain loops with the SI technique. In the process, both the cyclic shear stress, τ_c, and cyclic shear strain, γ_c, were obtained for each loop, with $G = \tau_c / \gamma_c$. Centrifuge models with the same overburden pressure, but different drainage boundary conditions and different relative densities, were expected to share the same shear modulus reduction curve (Darendeli 2001), as confirmed by Fig. 12a. Therefore, the G/G_{max} curves for 1 atm and 6 atm in Figure 12a were determined by combining information from both SD and DD tests. The information documenting the data points of Fig. 12 is listed in Table 5. Specifically, the development of the G/G_{max} reduction curve for 1 atm in Fig. 12a used four G/G_{max} versus γ_c points (black circles), corresponding to the τ_c and γ_c measured in the second cycle of shaking of the four 1 atm tests of Table 4. Similarly, the development of the G/G_{max} curve for 6 atm in the figure used four data points (red circles), also from the second cycle of motion of the four 6 atm tests of Table 4. The second cycle was consistently used for all eight data points of Table 5 and Fig. 12, because of two

reasons: (i) the cycle had a clear, symmetric non-noisy cyclic shape; and (ii) it corresponded to a relatively low excess pore pressure ratio at a time early in the shaking.

After determining the G/G_{max} curves in Fig. 12a, in a second step, the same information was converted into the normalized cyclic shear stress-strain backbone curve of Fig. 12b. This new plot of τ_c / G_{max} versus γ_c, takes advantage of the fact that the undegraded cyclic shear stress, $\tau_c = G\,\gamma_c = G_{max}\,(G/G_{max})\,\gamma_c$, so $\tau_c / G_{max} = (G/G_{max})\,\gamma_c$. All eight values of G/G_{max} and τ_c/G_{max} are listed in Table 5.

Specifically, the conversion from Fig. 12a to Fig. 12b was done as follows:

- For each data point of G/G_{max} versus γ_c in Fig. 12a, G/G_{max} was multiplied by γ_c to obtain the normalized cyclic shear stress for the corresponding stress-strain loop, $\tau_c / G_{max} = (G/G_{max})\,\gamma_c = G\,\gamma_c / G_{max}$. In a sense, this was just a recovery of the same value of τ_c previously obtained using SI and used to define the secant modulus of the loop, $G = \tau_c / \gamma_c$. The corresponding data points, tabulated in Table 5 and plotted in Fig. 12b, define the normalized backbone curves of τ_c / G_{max} versus γ_c at σ'_{v0} = 1 and 6 atm.
- The curves for 1 atm, $(\tau_c / G_{max})_1$, and 6 atm, $(\tau_c / G_{max})_6$, were fitted to the data points in Fig. 12b by using the modified hyperbolic stress-strain framework proposed by Darendeli (2001), see also Dobry and Abdoun (2015):

$$\frac{\tau_c}{G_{max}} = \left(\frac{G}{G_{max}}\right)\gamma_c = \frac{\gamma_c}{1+\left(\frac{\gamma_c}{\gamma_r}\right)^{0.919}} \quad (4)$$

and adjusting the value of the reference strain, γ_r, to provide the best fit of the curves to the data points for 1 and 6 atm in Fig. 12b. The corresponding values used for the mean representative curves of Fig. 12b in Eq. 4 are: $\gamma_r = 0.0123\%$ for $\sigma'_{v0} = 1$ atm, and $\gamma_r = 0.0385\%$ for $\sigma'_{v0} = 6$ atm.

The normalized backbone curves of Fig. 12b and Eq. 4 provide the means to evaluate the normalized cyclic shear stress for the D_r = 45% and 80% centrifuge tests, τ_c / G_{max}, associated with any undegraded cyclic strain, γ_c. It is then possible to evaluate the corresponding Cyclic Stress Ratio, CSR, of the same centrifuge tests, as CSR is the same cyclic shear stress but now normalized to the σ'_{v0} of the test:

$$\text{CSR} = \frac{\tau_c}{\sigma'_{v0}} = \frac{\tau_c}{G_{max}} \frac{G_{max}}{\sigma'_{v0}} \tag{5}$$

We are interested in a specific value of CSR for each test, the one associated with the nondegraded τ_c and γ_c causing a maximum pore pressure equal to the target value, $(r_u)_{max} = 0.8$; that is $\gamma_{cl,0.8}$ and $\tau_{cl,0.8}$. The values of $\gamma_{cl,0.8}$ were already determined and are listed in Table 4. The corresponding $\tau_{cl,0.8}$ may now be evaluated with Eq. 4 using these $\gamma_{cl,0.8}$. That is, the curves in Fig. 12b have equations of the form:

$$\left(\frac{\tau_{cl}}{G_{max}}\right)_{0.8} = \frac{\gamma_{cl,08}}{1+\left(\frac{\gamma_{cl,0.8}}{\gamma_r}\right)^{0.919}} \tag{6}$$

All values of $(\tau_{cl} / G_{max})_{0.8}$ were calculated using Eq. 6 and are listed in Table 4 for the eight centrifuge tests. Notice that the pair of DD tests at 1 atm, as well as the pair of DD tests at 6 atm, share common values of $\gamma_{cl,0.8}$ and thus also common values of $(\tau_{cl} / G_{max})_{0.8}$. Therefore, there are only four instead of eight different values of $(\tau_{cl} / G_{max})_{0.8}$ in Table 4.

By replacing τ_c by $\tau_{cl,0.8}$ in Eq. 5, CSR becomes the Cyclic Resistance Ratio, CRR. associated in these centrifuge tests with the failure criterion given by $(r_u)_{max} = 0.8$:

$$\text{CRR} = \left(\frac{\tau_{cl}}{G_{max}}\right)_{0.8} \frac{G_{max}}{\sigma'_{v0}} \tag{7}$$

It is now possible to evaluate the corresponding $K_\sigma = (\text{CRR})_6 / (\text{CRR})_1$, where both CRRs are obtained from Eq. 7 for σ'_{v0} = 6 and 1 atm, respectively. Finally:

$$K_\sigma = \frac{(CRR)_6}{(CRR)_1} = \frac{\left(\frac{\tau_{cl}}{G_{max}}\right)_{0.8,6}}{\left(\frac{\tau_{cl}}{G_{max}}\right)_{0.8,1}} \frac{(G_{max})_6}{(G_{max})_1} \frac{1}{6}$$

$$= \frac{\left(\frac{\tau_{cl}}{G_{max}}\right)_{0.8,6}}{\left(\frac{\tau_{cl}}{G_{max}}\right)_{0.8,1}} \sqrt{6} \, \frac{1}{6}$$

$$K_\sigma = \frac{(CRR)_6}{(CRR)_1} = \frac{\left(\frac{\tau_{cl}}{G_{max}}\right)_{0.8,6}}{\left(\frac{\tau_{cl}}{G_{max}}\right)_{0.8,1}} \frac{1}{2.45} \qquad (8)$$

Ni et al (2020) had verified that in the SD tests of Table 4, the measured ratio between G_{max} for comparable tests at 6 and 1 atm, $(G_{max})_6$ / $(G_{max})_1 = \sqrt{6} = 2.45$ with a high degree of precision. This ratio $(G_{max})_6$ / $(G_{max})_1 = 2.45$ was again verified by the authors and found to hold for the DD tests in Table 4. The result was used to produce the final version of Eq. 8 above.

K_σ may now be obtained in Table 4 by just dividing the corresponding pairs of $(\tau_{cl} / G_{max})_{0.8}$ at 6 and 1 atm and then dividing again by $\sqrt{6} = 2.45$. This was done in the table; resulting in best estimate values of $K_\sigma = 1.18$ and 1.20 for the SD and DD conditions, respectively, independent of relative density for the range between 45% and 80%.

Table 4 also includes the uncertainties of these best estimate K_σ determined using this approach, quantified as the standard deviations of K_σ obtained with Eqs. 6 and 8. The sources of uncertainty considered were:

1. Uncertainty introduced by using the median cyclic strain as representative of each whole strain history such as those plotted in Fig. 10.
2. Uncertainty in the values of γ_r due to the scatter of the data points around the normalized backbone curves of Fig. 12b.
3. Uncertainty due to the location of the accelerometers in the centrifuge model.
4. Uncertainty in the values of $\gamma_{cl,\,0.8}$ due to the fitted γ_{cl} vs $(r_u)_{max}$ parallel lines in Fig. 11.

These sources of uncertainty were combined using the method presented by Benjamin and Cornell (1970), in order to get the combined standard deviations and ranges of K_σ around the best estimates listed in Table 4 (1.18±0.17 and 1.20±0.18 for the SD and DD tests, respectively).

Discussion

Determination of K_σ using the cyclic strains from SI followed four main steps: 1) at both 1 and 6 atm, get $\gamma_{cl,0.8}$, the cyclic shear strain needed to reach $(r_u)_{max} = 0.8$, using Fig. 11; 2) at both 1 and 6 atm, obtain $(\tau_{cl} / G_{max})_{0.8} = \tau_{cl,0.8} / G_{max}$ using the curves of Fig. 12b, fitted using Eq. 6, where $\tau_{cl,0.8}$ is the undegraded cyclic shear stress needed to reach $(r_u)_{max} = 0.8$; 3) at both 1 and 6 atm, obtain the Cyclic Stress Ratio, CRR, corresponding to this failure criterion, $(r_u)_{max} = 0.8$, using Eq. 7; and 4) evaluate $K_\sigma = (CRR)_6 / (CRR)_1$ (Eq. 8). As shown in Table 4, these K_σ obtained by going first through the cyclic strains contain no inequalities, as a result of the mild interpolations and extrapolations conducted in Fig. 11 to estimate the value of $\gamma_{cl,0.8}$ for each centrifuge test.

All K_σ listed in Table 4 are very consistent, irrespective of them corresponding to SD or DD conditions, irrespective of relative density, and also irrespective of being calculated directly with the cyclic shear stresses from SI, or by going through the strains first to get those cyclic stresses. All best estimates of K_σ in Table 4 are essentially in the range 1.2 to 1.3. That is, this work confirms and extends to DD conditions, the conclusion reached by Ni et al. (2020) for SD tests done at D_r = 45% and 80%, that the field K_σ for the field and shaking situations simulated in these model experiments, is $K_\sigma \approx 1.2$ to $1.3 > 1$. This again suggests that the current State-of-Practice where $K_\sigma < 1$ may be too conservative for some field conditions, due to the decreased sand compressibility at high overburden pressures.

Summary and Conclusions

A series of eight centrifuge tests was analyzed, four of them with single drainage conditions already reported by Ni et al. (2020) and Abdoun et al. (2020), and four new tests conducted with double drainage conditions for the same liquefiable sand layer. In the eight experiments, a 5-m thick saturated clean Ottawa sand was subjected to an input base acceleration of the same shape and duration but different acceleration amplitudes. The sand layer had a relative density of 45% or 80%, an effective overburden pressure of 1 atm or 6 atm, a drainage condition covering single drainage (SD) at the top of the layer or double drainage (DD) at top and bottom, and the models were subjected to a base input shaking aimed at achieving in the sand a targeted maximum excess pore pressure ratio, $(r_u)_{max} \approx 0.8$. Comparable models were also conducted with SD and DD conditions that had the same input shaking intensity but induced a smaller $(r_u)_{max}$ in the DD test. The overburden correction factor, K_σ, for an overburden of 6 atm and the idealized field condition of the tests was determined for a failure conditions, $(r_u)_{max} \approx 0.8$, using directly the Cyclic Resistance Ratios (CSR) measured in the tests, as well as an alternative method where the CRRs were evaluated by going first through the cyclic shear strains measured in the tests. The main conclusions are as follows:

1. A novel centrifuge technique to achieve the DD condition with geocomposite layer at the bottom of the sand layer and a dry lead shot layer at the top of the sand layer worked successfully, as proven by the negligible excess pore pressures measured at the two boundaries. This technique provides a new option to centrifuge modelers for simulating idealized single and double drainage field condition in their experiments.
2. A stronger input acceleration (1.45 ~ 1.67 times stronger) was required to achieve a similar $(r_u)_{max} \approx 0.8$ in a DD test compared to the SD test. When the same input shaking was used,

comparable SD and DD experiments had similar acceleration records at different depths in the sand layer, indicating little effect of the drainage conditions on these accelerations.

3. The excess pore water pressures dissipated significantly faster in the DD than in the SD tests. Both in the SD and DD experiments, there was more drainage in the 6 atm than in the 1 atm tests, resulting in high field $K_{\sigma} > 1$ values obtained for both SD and DD conditions.

4. The drainage conditions had a significant effect on the cyclic shear strain required to reach the same value of $(r_u)_{max}$, with a higher cyclic strain for DD than for SD tests.

5. Relationships between $(r_u)_{max}$ and the cyclic shear strain (γ_c) were developed for all eight tests. It was found that both the overburden pressure, σ'_{v0}, as well as the drainage conditions had a significant effect on the location of the $(r_u)_{max}$ versus γ_c curve. Specifically, both a high overburden pressure and a DD condition shift the curve rightward. On the other hand, relative density in the range from 45% to 80% has little or no effect on the location of the $(r_u)_{max}$ versus γ_c curve.

6. The best estimates of field K_{σ} obtained for 6 atm are in the approximate range between 1.2 and 1.3, irrespective of drainage conditions (SD or DD), irrespective of relative density between D = 45% and 80%, and also irrespective of the method used to determine K_{σ}. This overall consistency validates the two methods used in this paper for K_{σ}, indicating that the current State-of-Practice where $K_{\sigma} < 1$, is too conservative for the idealized SD and DD field conditions implemented in these centrifuge tests.

Data Availability Statement

Some or all data, models, or code generated or used during the study are available from the corresponding author by request.

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Table 1 Relative density, confining pressure, g-level and drainage condition of centrifuge models with single drainage boundary at the top (Ni et al. 2020)

Test	Relative Density, D_r (%)[1]	Centrifugal g-level (g)	Effective overburden pressure, σ'_{v0} (atm)[2]	Drainage conditions
Test 45−1[3](SD)0.045g[4]	45	20	1	SD[5]
Test 45−6(SD)−0.3g	45	60	6	SD
Test 80−1(SD)−0.05g	80	20	1	SD
Test 80−6(SD)−0.3g	80	60	6	SD

(1) Relative density after spinning to target g-level
(2) Effective overburden pressure at mid depth of sand layer
(3) Test name includes "sand relative density−effective overburden pressure in atm (drainage condition)"
(4) Test name includes average rounded up prototype input peak base acceleration inside the container (g) for each test
(5) SD represents single drainage, meaning free drainage only at the top of the sand layer

Table 2 Relative density, confining pressure g-level and drainage condition of centrifuge models with double drainage at top and bottom

Test	Relative Density, D_r (%)[1]	Centrifugal g-level (g)	Effective overburden pressure, σ'_{v0} (atm)[2]	Drainage conditions
Test 45−1[3](DD)0.045g[4]	45	20	1	DD[5]
Test 45−1(DD)-0.065g	45	20	1	DD
Test 45−6(DD)-0.3g	45	45	6	DD
Test 45−6(DD)-0.5g	45	45	6	DD

(1) Relative density after spinning to target g-level
(2) Effective overburden pressure at mid depth of sand layer
(3) Test name includes "g-level–effective overburden pressure (drainage condition)"
(4) Test name includes the average prototype input peak base acceleration inside the container (g) for each test
(5) DD represents double drainage, meaning free drainage at both the top and bottom of the sand layer

Table 3 Peak base acceleration and maximum pore pressure ratios reached for different centrifuge tests under single drainage (SD) and double drainage (DD) boundary conditions with Dr = 45%

Test	Average prototype input peak base acceleration (inside container) (g)[(1)]	$(r_u)_{max}$
Test 45 – 1(SD) – 0.045g[(2)]	0.045	0.80
Test 45 – 1(DD) – 0.045g	0.045	0.48
Test 45 – 1(DD) – 0.065g	0.065	0.68
Test 45 – 6(SD) – 0.3g[(2)]	0.3	0.76
Test 45 – 6(DD) – 0.3g	0.3	0.18
Test 45 – 6(DD) – 0.5g	0.5	0.85

[(1)] All input acceleration time histories consisted of ten cycles of uniform acceleration amplitude having a 2 Hz prototype frequency

[(2)] The single drainage (SD) tests were reported in detail by Ni et al. (2020)

Table 4 Maximum excess pore pressure ratio, $(r_u)_{max}$; median cyclic shear strain, γ_c; extrapolated $\gamma_{cl,0.8}$ and undegraded $\tau_{cl,0.8}$ required to reach $(r_u)_{max} = 0.8$; and K_σ from Eq. 8 for $D_r = 45\%$ and different drainage conditions

<table>
<tr><th>Test</th><th>$(r_u)_{max}$</th><th>Median γ_c (%)</th><th>$\gamma_{cl,0.8}$ (%)</th><th>$(\tau_{cl}/G_{max})_{0.8}$ (Eq. 6)</th><th>K_σ (Eq. 8, CRRs in Eq. 1 from shear strains using SI)</th><th>K_σ (CRRs in Eq. 1 directly from shear stresses using SI)</th></tr>
<tr><td>Test 45 – 6(SD) –0.3g(1)</td><td>0.76</td><td>0.232</td><td>0.273</td><td>3.87×10^{-4}</td><td rowspan="4">1.18 ± 0.17</td><td rowspan="2">1.28</td></tr>
<tr><td>Test 45 – 1(SD) –0.045g(1)</td><td>0.80</td><td>0.133</td><td>0.127</td><td>1.33×10^{-4}</td></tr>
<tr><td>Test 80 – 6(SD) – 0.3g(1)</td><td>0.60</td><td>0.209</td><td>0.273</td><td>3.87×10^{-4}</td><td rowspan="2">>1.15</td></tr>
<tr><td>Test 80 – 1(SD) –0.05g(1)</td><td>0.92</td><td>0.153</td><td>0.127</td><td>1.33×10^{-4}</td></tr>
<tr><td>Test 45 – 6(DD) –0.3g(2)</td><td>0.18</td><td>0.131</td><td rowspan="2">0.450</td><td rowspan="2">4.25×10^{-4}</td><td rowspan="4">1.20 ± 0.18</td><td rowspan="4">< 1.30</td></tr>
<tr><td>Test 45 – 6(DD) –0.5g(2)</td><td>0.85</td><td>0.537</td></tr>
<tr><td>Test 45 – 1(DD) –0.045g(2)</td><td>0.48</td><td>0.123</td><td rowspan="2">0.214</td><td rowspan="2">1.45×10^{-4}</td></tr>
<tr><td>Test 45 – 1(DD)– 0.065g(2)</td><td>0.68</td><td>0.175</td></tr>
</table>

(1) SD centrifuge tests reported by Ni et al. (2020)
(2) New DD centrifuge tests

Table 5 Undegraded cyclic shear stress-strain parameters obtained from second cycle of shaking in centrifuge tests

Test	γ_c (%)	G/G_{max}	$\tau_c/G_{max}=(G/G_{max})\gamma_c$ [(1)]
Test 45−1(SD)-0.045g [(2)]	0.108	0.120	1.30×10^{-4}
Test 45−6(SD)−0.3g [(2)]	0.225	0.187	4.20×10^{-4}
Test 80−1(SD)−0.05g [(2)]	0.0766	0.208	1.59×10^{-4}
Test 80−6(SD)−0.3g [(2)]	0.148	0.265	3.91×10^{-4}
Test 45−1(DD)-0.045g [(3)]	0.101	0.133	1.35×10^{-4}
Test 45−1(DD)-0.065g [(3)]	0.136	0.0635	0.866×10^{-4}
Test 45−6(DD)-0.3g [(3)]	0.175	0.199	3.47×10^{-4}
Test 45−6(DD)-0.5g [(3)]	0.342	0.0936	3.20×10^{-4}

(1) γ_c in meter/meter
(2) SD centrifuge test reported by Ni et al. (2020)
(3) New DD centrifuge test