

# Impacts of fixed-end and flexible boundary conditions on seismic response of shallow foundations on saturated sand in 1g shaking table tests

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## Abstract

A key consideration in physical modeling of seismic problems in geotechnical engineering is the impact of the model container boundaries on the soil layer response. The container boundaries may alter the stress-strain behavior from free-field conditions through the possible reflection of incident shear waves and generation of P-waves within the soil layers. In this study, 1g shaking table experiments were performed to evaluate the impacts of container boundary conditions on the response of saturated, loose sand layers subjected to harmonic base motions in (1) a free-field condition, (2) with a shallow foundation, and (3) with a shallow foundation supporting a single degree of freedom super-structure. The sand layers were formed in a newly-fabricated laminar shear container that can be converted to a rigid box by adding elements to the end walls. Acceleration, excess pore water pressure, and settlement measurements demonstrate that the rigidity of the container boundaries can have a major impact on seismic behavior of the models. In particular, the observed permanent settlement of the foundations increased by 58% to 115% in the soil models with fixed-end (or rigid) boundaries compared to those in soil models with flexible conditions. This was attributed to non-uniformity of strains near the fixed-end container boundaries and a higher level of energy trapped inside the model. Furthermore, higher spectral accelerations were captured in tests with

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3 22 fixed-end boundary conditions compared to those with flexible boundary conditions. Scaling  
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5 23 issues associated with 1g shaking table testing were also discussed.

6  
7 24 **Keywords:** Laminar shear container; Boundary condition; Shallow foundation; 1g shaking  
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9 25 table; Saturated loose sand

## 10 26 **Introduction**

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12 27 Considerable economic losses and casualties were reported in past earthquakes when the  
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14 28 settlement, tilting, or bearing capacity failure of shallow foundations resulted in major damages  
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16 29 to overlying super-structures (Tokimatsu et al. 1994; Yoshida et al. 2001; Bird et al. 2004; Bray  
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18 30 and Dashti 2014; Bray and Lique 2017; Franke et al. 2019). Liquefiable soil layers may affect  
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20 31 the seismic response of shallow foundations significantly, as shear stiffness and strength  
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22 32 degradation of the soil may lead to bearing capacity failure or lateral spreading, while post-  
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24 33 liquefaction drainage may result in excessive settlements and tilting of foundations. Examples  
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26 34 of large earthquakes where shallow foundation issues were observed include the 1964 Niigata  
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28 35 (Yoshimi and Tokimatsu 1977), the 1999 Kocaeli (Acacio et al. 2001; Bray et al. 2000), the  
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30 36 2007 Peru (Taucer et al. 2009), the 2010 Chile (Bertalot et al. 2013), the 2011 Tohoku  
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32 37 (Tokimatsu et al. 2012), the 2011 Christchurch (Bray et al. 2014), and the 2016 Kumamoto  
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34 38 earthquakes (Tokimatsu et al. 2019).

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36 39 Various approaches have been employed to evaluate the seismic behavior of shallow  
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38 40 foundations, such as analytical and numerical modeling, reduced-scale or full-scale physical  
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40 41 model testing at 1g, small-scale physical model testing at an elevated acceleration field ( $N_g$ ) in  
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42 42 a geotechnical centrifuge, and the empirical models originated from the field reconnaissance  
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44 43 reports of actual earthquakes. Of these methods, reduced-scale physical modeling experiments  
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46 44 at 1g and in a geotechnical centrifuge at  $N_g$  can be beneficial to gain insight into seismic  
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48 45 problems as they permit parametric evaluation with controlled specimen preparation and  
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50 46 testing, and can be densely instrumented. These experimental approaches also permit  
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52 47 straightforward interpretation of ground amplification and soil-structure interaction (SSI)  
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54 48 problems, and in the right conditions may result in realistic variations of the co- and post-  
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56 49 seismic excess pore water pressure (EPWP) and consideration of soil nonlinearity.

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58 50 Numerical studies have been performed to evaluate the mechanisms and understand key  
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60 51 parameters controlling the shear-induced settlement of structures resting on shallow foundation  
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53 52 due to soil liquefaction (Pane et al. 2016; Macedo and Bray 2018; Karimi et al. 2018; Forcellini  
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55 53 2019). Furthermore, probabilistic methods have been used to assess liquefaction potential in

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3 54 addition to liquefaction-induced settlement of shallow foundations (e.g., Jafarian et al. 2013;  
4 55 Shahnazari et al. 2016; Bullock et al. 2018). Field observations and reconnaissance studies  
5 56 were also employed to evaluate case histories and key factors controlling the settlement and  
6 57 tilting of existing structures based on observed damages in recent earthquakes (Yoshimi and  
7 58 Tokimatsu 1977; Acacio et al. 2001; Bray et al. 2000; Bertalot et al. 2013; Bray et al. 2014;  
8 59 Tokimatsu et al. 2012; Tokimatsu et al. 2019). Moreover, numerous experimental studies have  
9 60 investigated different aspects of seismic behavior of shallow foundation on dry or saturated  
10 61 condition using physical models in both 1g shaking table and Ng centrifuge tests (Liu and  
11 62 Dobry 1997; Adalier et al. 2003; Dashti et al. 2010a; Dashti et al. 2010b; Hayden et al. 2014;  
12 63 Jafarian et al. 2017; Mehrzad et al. 2018; García-Torres and Madabhushi 2019; Jafarian et al.  
13 64 2020).

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23 65 A majority of previous physical and numerical modeling studies concentrated on understanding  
24 66 the mechanisms and key parameters governing the settlement and tilting of shallow founded-  
25 67 structures during earthquakes. However, the effects of lateral boundary condition on the  
26 68 seismic performance of the shallow foundations have not been systematically investigated in  
27 69 direct comparisons. Only a few studies are available in the literature involved investigation of  
28 70 boundary condition effects on soil responses in free-field conditions (Whitman and Lambe  
29 71 1986; Lee et al. 2012), slopes (Hung et al. 2018), or for a shallow foundation on dry sand (Pozo  
30 72 et al. 2016). No previous research is available where the effects of boundary conditions in terms  
31 73 of flexible and fixed-end on response was compared for different soil-structure interaction  
32 74 conditions (e.g., free field conditions, shallow foundation on saturated sand, shallow  
33 75 foundation beneath a single degree of freedom (SDOF) super-structure on saturated sand).

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42 76 Whitman and Lambe (1986) compared the performance of containers with different boundary  
43 77 conditions when modeling a sand layer in centrifuge base shaking experiments. They compared  
44 78 the response of a soil layer in a rigid container with the same soil condition in a container  
45 79 consist of a number of stacked-ring devices with different aspect ratios to evaluate geometrical  
46 80 limitations and model performance. They concluded that even distant walls can affect the  
47 81 results when liquefaction occurs. Fiegel et al. (1994) compared the performance of the Caltech  
48 82 LSB container, the Cambridge equivalent shear beam (ESB) container, and a rigid container  
49 83 with that of a hinged-plate container (HPC) in shaking table tests in the geotechnical centrifuge  
50 84 at UC Davis. They found that LSB containers have smaller values of acceleration amplification  
51 85 ratio and higher natural frequency compared to the other containers and this could lead to lower  
52 86 system stiffness. This is essential if the design goal for the container is for the soil movement

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3 87 to govern the system deformation.  
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5 88 Lee et al. (2012) examined boundary effects of an LSB container using dry and saturated sand  
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7 89 models. They found that the main frequencies, acceleration amplitude phase lags, and the  
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9 90 profiles of the obtained root mean square accelerations remained intact if the distance of  
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11 91 instruments from the moving end walls were more than one-twentieth of longitudinal  
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13 92 dimension of the model. Moreover, they observed that discrepancies between the measured  
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15 93 EPWP at the model center and at a distance of one-fourth of the long side of the model from  
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17 94 the lateral boundaries are negligible. Pozo et al. (2016) focused on the assessment of soft lateral  
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19 95 boundary effects on a shallow foundation in terms of pre- and post-peak load-displacement  
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21 96 responses using the particle image velocimetry (PIV) method. They observed the strain  
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23 97 development at stiff and softer lateral boundaries and identified a relationship between these  
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25 98 observations and the mechanical behavior of the shallow foundation. Hung et al. (2018)  
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27 99 investigated the effects of fixed and flexible boundary conditions on the response of slope in  
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29 100 liquefiable site and showed that the slope experienced different accelerations in the fixed and  
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31 101 flexible boundary conditions. Ghayoomi et al. (2013, 2014) evaluated the response of an acrylic  
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33 102 flexible shear beam (FSB) container that permitted visual observation of buried structures  
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35 103 during shaking table testing in the centrifuge while still maintaining plane strain conditions and  
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37 104 hydraulically-sealed conditions.

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39 105 This paper first presents the design and construction of a new light-weight, acrylic, single-axis  
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41 106 LSB for use in 1g shaking table testing. Dynamic performance of the container was then  
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43 107 assessed through a series of 1g shaking table tests on loose sand layers in both dry and saturated  
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45 108 conditions. For the main testing program, three series of 1g shaking table tests were then  
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47 109 performed to evaluate the boundary effects of the container on the distributions of acceleration,  
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49 110 displacement, and EPWPs in saturated, loose sand layers in (1) free-field condition, (2) with a  
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51 111 shallow foundation, and (3) with a shallow foundation supporting a SDOF super-structure.  
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53 112 Considering the results of the tests on a saturated, free-field sand layer as a reference, the  
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55 113 impacts of including a solitary shallow foundation resting on the sand surface, and a shallow  
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57 114 foundation supporting SDOF super-structure were investigated.

### 58 115 **Container boundary conditions**

59 116 In both 1g and  $N_g$  shaking table experiments on soil layers, a container is needed to retain both  
60 117 the soil and pore fluids. To properly model one-dimensional (1D) wave propagation in a finite  
118 118 soil stratum in a shaking table experiment, the following criteria for container designs should

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3 119 be satisfied: (1) the horizontal cross-section of the container should remain constant during  
4 120 shaking to maintain plane strain conditions (zero lateral strain); (2) the walls of the container  
5 121 in the shaking direction should not impede the movement of the soil in response to basal  
6 122 shaking; (3) the mass of the container should be as low as possible to lessen the dynamic inertial  
7 123 forces at the boundaries, and approximately zero stiffness for horizontal shearing to make sure  
8 124 the soil governs the system movement; and (4) the end walls of the container must be roughened  
9 125 so that complementary shear stresses can be developed that match existing shear stresses on  
10 126 the container base (Whitman and Lambe 1986; Zeng and Schofield 1996).

11 127 One of the critical issues in physical modeling of geotechnical problems is to reproduce, as  
12 128 close as possible, lateral boundary conditions representative of those in a field setting to  
13 129 minimize undesirable effects on test results. Lateral boundary conditions can have a significant  
14 130 impact on the model response in physical modeling experiments and may alter the dynamic  
15 131 behavior of a soil system lead to preventing correct simulation of free-field response (Lee et  
16 132 al. 2012). Hence, considering these effects in shaking table tests in both 1g and Ng is crucial  
17 133 when analyzing model behavior and interpreting the results.

18 134 In the last four decades, several efforts have been made by different researchers to design and  
19 135 develop different types of containers, including: (1) rigid or fixed-end containers (Fishman et  
20 136 al. 1995), (2) rigid containers with flexible boundaries (Pozo et al. 2016), (3) hinged-plate  
21 137 containers (Whitman and Lambe 1986), (4) flexible shear beam (FSB) containers (Ghayoomi  
22 138 et al. 2013 and 2014), (5) equivalent shear beam containers (Zeng and Schofield 1996), (6)  
23 139 laminar shear beam (LSB) containers (Hushmand et al. 1988; Turan et al. 2009), and (7) active  
24 140 boundary containers (Takahashi et al. 2001). Both 1g and Ng shaking table tests have been  
25 141 conducted using these types of containers to understand their key properties and performance  
26 142 in modeling 1D wave propagation in soil column. Among all developed containers, LSB  
27 143 containers are the most popular due to their accuracy in modeling realistic site (Li et al. 2020).  
28 144 An LSB comprises a stack of laminates (or frames) supported individually by bearings that can  
29 145 be mounted on a shaking table and deform in the direction of motion to mimic the free-field  
30 146 displacement of a soil layer during basal shaking. The bearings allow relative displacement  
31 147 between the frames with minimal friction and facilitate relatively free movement of the soil  
32 148 layers similar to the in-situ conditions. Table 1 summarized laminar shear containers employed  
33 149 in previous studies by different researchers.

34 150 Rigid or fixed-end containers were the first container type used in physical modeling  
35 151 experiments to determine the seismic bearing capacity of shallow footings (Okamoto 1956).

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3 152 Rigid containers are unable to replicate the realistic free-field seismic response of soil layer  
4 153 overlaying a rigid rock mainly due to three important issues (see Fig. 1): (1) Strain  
5 154 dissimilarities at the boundaries due to fixed end walls that restrict soil deformation (Zeng and  
6 155 Schofield 1996), (2) Lack of complementary shear stress near the smooth end walls (Lee et al.  
7 156 2012), and (3) generation of P-waves due to interaction between the soil and fixed-walls which  
8 157 find no way out to the infinite half-space, effectively trapping them inside the model owing to  
9 158 the fixed-end artificial boundaries (Lee et al. 2012).

159 One of the crucial issues in performance of the model containers is their size. Effects of  
160 container size on the soil response in laminar shear containers have not been investigated to  
161 date and no related research in the literature practically examined this issue. An intuitive  
162 understanding would be that the larger the model is, the more precise the model response will  
163 be (Zhang et al. 2008). For instance, the full-scale prototype ( $N=1$ ) could be considered as the  
164 ideal model. However, bigger container size required stronger laminates profile resulting in  
165 higher box to soil mass ratio that could affect the soil response due to augmented inertial  
166 forces. Moreover, increased container mass could have undesirable effect on bearing  
167 performance in providing freely movement of adjacent laminates.

168 In rigid or fixed-end containers, change in size could have major impacts on soil responses,  
169 leading to unrealistic/misleading results. A rigid container has to be large enough to correctly  
170 replicate the free-field site response. Fishman et al. (1995) reported that the free-field soil  
171 response may not be realized in the rigid containers for distances up to  $1.5H$  to  $2H$  from the  
172 fixed end walls, where  $H$  is the container height. It means the rigid container length should be  
173  $4H$  to reproduce free-field condition over the central portion of the model.

## 174 **Experimental Setup**

### 175 **Laminar shear container**

176 The laminar shear container at International Institute of Earthquake Engineering and  
177 Seismology (IIEES) was designed for 1g shaking table to perform seismic modeling of soil  
178 layers subjected to 1D earthquake-like input motions at the model base. It was designed to test  
179 saturated or dry soil models and allow developing of complementary stresses related to 1D  
180 wave propagation. Complementary shear stresses are mobilized at the boundaries in the  
181 direction of shear via roughened end walls. Fig. 2 shows views of the fabricated LSB on the  
182 shaking table in addition to laminate plan and sections of longitudinal and transverse I-beams.

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3 183 Based on extensive review of available LSB containers in the literature summarized in Table  
4 184 1, the authors decided to specify the container inner length to height ratio ( $L/H$ ) to be 1.4 and  
5 185 the inner length to width ratio ( $L/W$ ) to be 1.5. An optimization analysis was employed to  
6 186 calculate the optimal values of length, width, and thickness of longitudinal and transverse  
7 187 beams. As a result,  $L=500$  mm was chosen for the container outer length. By choosing the outer  
8 188 length of the container (i.e.  $L=500$  mm), longitudinal and transverse beams were then designed  
9 189 and the internal dimensions of the LSB was eventually obtained: 364.8 mm in length, 242.8  
10 190 mm in width, and 263 mm in height ( $L \times W \times H$ ). Allowable stress design (ASD) method was  
11 191 used to design all structural elements to resist maximum estimated forces acting on each  
12 192 element in seismic condition. Furthermore, additional estimated loads were considered in the  
13 193 container design calculation so that the container height could be extended to 500 mm if  
14 194 necessary.

15 195 The container consists of 18 light-weight acrylic laminates arranged in a stack to provide  
16 196 flexible lateral boundaries. Acrylic was chosen because: (1) it is lightweight so that the inertial  
17 197 effects of the container on soil layer movements could be minimum due to low box to soil mass  
18 198 ratio; (2) the whole system movement is visible due to material transparency; and (3) ease of  
19 199 machining and assembly/disassembly of acrylic material which facilitate convenient  
20 200 maintenance. The laminates are made of I-beams, with 67.6 mm in depth for the transverse  
21 201 beam and 45.8 mm for the longitudinal beam. The height of each individual beam in both  
22 202 transverse and longitudinal beam laminates is 11 mm.

23 203 The base laminate was firmly fastened to the shaking table platform via bolts. The laminates  
24 204 components consist of stoppers, rollers, and plastic holder for rollers. The  $5 \times 8$  mm cubic  
25 205 stoppers were also implemented in the transverse beams of the laminates in order to limit the  
26 206 relative displacement between the layers in longitudinal direction. Since the stoppers are  
27 207 embedded within the layer interfaces, there is no need of external mount to limit the  
28 208 deformation for possible container instability. In order to facilitate freely relative movement of  
29 209 the frames and decrease friction between them and to obtain a uniform weight distribution, 40  
30 210 acrylic rollers (with 10 mm diameter) were interlaid between each laminate. Each laminate can  
31 211 slide laterally up to 4 mm relative to the adjacent laminate in the longitudinal direction. The  
32 212 maximum cumulative lateral displacement of the LSB top layer is 68 mm with a total  
33 213 achievable shear strain of about 26%. This large displacement is provided to accommodate  
34 214 large deformation phenomena like liquefaction-induced lateral spreading. The approximate  
35 215 friction coefficient of each frame during sliding is 0.01 and the frame-to-soil mass ratio is 0.24.

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3 216 A 0.44 mm-thick latex membrane bag was fabricated to fit within the box to retain water and  
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5 217 soil grains inside the container. The total mass of the empty LSB is 15.6 kg and when filled  
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7 218 with saturated sand having a total density of 1966 kg/m<sup>3</sup> its total mass is 62.2 kg. To prevent  
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9 219 contact interference between laminates during container shaking and to minimize volumetric  
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11 220 strains due to membrane bulging, 4 mm vertical gaps are provided between each adjacent  
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13 221 laminate. The IIEES laminar shear container properties are summarized in Table 2.

### 14 222 **Shaking Table**

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16 223 The IIEES 1g shaking table used for the experiments acts in a single direction and consists of  
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18 224 a 1.4 m × 1.2 m platform that is driven by hydraulic actuators controlled by a digital control  
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20 225 unit. The table can simulate realistic earthquake motions as well as single- or multi-stage input  
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22 226 motions including harmonic spectrum motions, band-limited motions, impulse load motions,  
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24 227 and white noise spectrum motions. The shaking table has a frequency output range of 1 to 400  
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26 228 Hz, a maximum stroke of 35 mm in the excitation direction, and a 20 kN base shear capacity.

### 27 229 **Similitude laws for 1g shaking table testing**

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29 230 Because of the size of the container, only reduced-scale soil-structure interaction experiments  
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31 231 may be performed. Accordingly, similitude laws should be employed to extrapolate the  
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33 232 behavior from the shaking table tests to full-scale systems (prototype scale). Different sets of  
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35 233 similitude laws for reduced scale models at 1g have been established in the literature (Kagawa  
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37 234 1978; Kokusho and Iwatate 1979; Iai 1989; Maymand 1998). Of these, Iai (1989) derived a  
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39 235 set of similitude relations particularly for saturated sand in 1g shaking table tests in which most  
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41 236 key parameters were taken into consideration to approximately reproduce field condition. Iai  
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43 237 (1989) also examined the applicability of the derived similitude relations through available  
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45 238 tests results and reported that the similitude will give a good approximation on the behavior of  
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47 239 the prototype. Nevertheless, the low self-weight stress level at 1g gravitational field inevitably  
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49 240 affect the stress-strain and seismic response of the sand layer in physical model tests.  
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51 241 Specifically, excess pore water pressure (EPWP) generation and dilatancy behavior in sands  
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53 242 are dictated by combined effects of density and effective stress level. To overcome this  
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55 243 difficulty, Toyota et al. (2004) suggested procedures such as using a modified grain size  
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57 244 distribution, lower relative density, higher frequency of loading, and application of more  
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59 245 loading cycles in the 1g shaking table tests to retain similitude with field-scale test results.  
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246 Vargas-Monge (1998) recommended use of a lessened relative density in the model compared  
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with that of the prototype using concept of brittleness index. The relative density of the soil



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3 248 layer in the model can be determined by maintaining the brittleness index constant between the  
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5 249 prototype and the model.  
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7 250 In order to evaluate the applicability of the employed scaling laws in physical model tests,  
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9 251 numerous studies have compared shaking table tests at 1g and Ng to identify appropriate  
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11 252 scaling factors relating the corresponding results in both experiments (Gibson and Scott 1995;  
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13 253 Hayashi et al. 1997). In these studies, satisfactory agreement was observed between the results  
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15 254 of 1g shaking table tests and those of Ng centrifuge tests. Also, they all indicate that if the  
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17 255 shaking table tests are being performed carefully, application of proper scaling factors plays an  
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19 256 important role to achieve reliable results. Accordingly, 1g shaking table tests may be useful in  
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21 257 validating future centrifuge modeling studies.

22 258 In the current study, the scale factors derived by Iai (1989) were employed to relate reduced-  
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24 259 scale model and full-scale prototype parameters in the 1g shaking table tests which are  
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26 260 summarizes in Table 3 with adopted scaling factor of  $N=16.7$ . To compensate for the deficiency  
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28 261 of the lower confining stress in the current 1g model tests, the sand relative density in the model  
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30 262 test ( $D_r = 30\%$ ) was chosen to be lower than that representative of the prototype ( $D_r = 57\%$ ).  
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32 263 In other words, dilatancy of the sand is kept constant in the model and prototype by adjusting  
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34 264 the stress-density state, decreasing the relative density of the model sand to compensate for the  
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36 265 increased dilatancy produced by the smaller effective stress in the model. Further, higher  
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38 266 loading frequency (i.e. 10 Hz) and more loading cycles (i.e. 80 cycles) were applied as a base  
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40 267 input motion to retrieve for low self-weight stresses in the current experiments.

#### 41 268 **Soil properties**

42 269 Babolsar sand was employed in the shaking table tests, which classifies as poorly graded sand  
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44 270 (SP) according to USCS classification system. The sand has zero fines contents ( $FC=0\%$ ) and  
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46 271 a mean particle size of  $D_{50} = 0.15$  mm. The minimum and maximum void ratios of the  
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48 272 Babolsar sand are  $e_{\min} = 0.632$  and  $e_{\max} = 0.868$ , respectively. The specific gravity of solids  
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50 273 is  $G_s = 2.74$ . The grain size distribution is shown in Fig. 3. Babolsar sand has been used in  
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52 274 recent experimental studies that have reported other geotechnical properties of this material  
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54 275 (Jafarian et al. 2016; Jafarian et al. 2019).

#### 55 276 **Input motion**

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57 277 The base input motion for all experiments consisted of 80 sinusoidal wave cycles with a 0.3g  
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59 278 acceleration amplitude at 10 Hz frequency, lasting for eight seconds as shown in Fig. 4. The  
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279 long loading duration is to compensate for the lower confining stresses in the reduced-scale 1g

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3 280 model tests and to simulate a worst-case scenario of soil subsidence as well as tilting and/or  
4 281 settlements of structure model during shaking. The sinusoidal wave was chosen for the input  
5 282 excitation mainly to permit simple interpretation of results owing to the inherent symmetry of  
6 283 the constant magnitude sine-type seismic excitations. The acceleration amplitude of 0.3g was  
7 284 selected to ensure the triggering of liquefaction during shaking.

## 285 **Model preparation**

### 286 **Soil layer preparation**

287 Dry sand was placed by air pluviation within the LSB container to form uniform layers having  
288 a relative density of  $D_r = 30\%$  (dry density of 1966 kg/m<sup>3</sup>) in model scale until the box is  
289 filled. The drop height used in pluviation was varied using trial and error to reach consistent  
290 relative densities. During pluviation, sensors were embedded at the designated locations as  
291 shown in instrumentation layout. For the tests performed on water-saturated sand layers, after  
292 injecting CO<sub>2</sub> into the model to expel pore air, de-aired water was then flushed slowly upward  
293 through the sand layer from the base until the water table reached 5 mm above the soil surface.  
294 After approximately two hours, surplus water on the soil surface was removed using a sponge.  
295 Then, the water level was set to be at the soil surface. From measuring the amount of water  
296 flushed into the model and dividing by the required water needed for full saturation, the degree  
297 of saturation was calculated. The calculated degree of saturation was above 90% for all tests.  
298 For the second and third test series, the shallow foundation (with or without a superstructure)  
299 was then placed atop of the soil surface. Immediately after the installation of shallow  
300 foundation, vertical displacements were monitored through an image processing technique  
301 during a consolidation phase prior to the shaking onset. Initial captured settlements values were  
302 less than 0.9 mm for all tests which was deemed negligible compared to the permanent  
303 settlements (17~33 mm) measured during the shaking phase.

304 In reduced-scale 1g shaking table tests where confining stress levels are low, it is essential to  
305 characterize proper stress-strain relation in agreement with the prototype especially for  
306 considering dense sand dilatancy behavior. The loose sands dilatant behavior at low stress  
307 levels in a 1g shaking table tests generally simulate the denser sand behavior in prototype.  
308 Vargas-Monge (1998) carried out a series of element testing under constant volume condition  
309 and tried to lessen the effects of stress levels by changing the soil relative density and then  
310 developed the softening extent calculated by the brittleness index ( $I_B$ ) which previously  
311 suggested by Bishop et al. (1971). The relative density can be determined by maintaining the

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3 312 undrained brittleness index constant between the prototype and the model in the expression  
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5 313 proposed in Eq. (1):  
6

$$\frac{e_m}{e_p} = 1 + \frac{0.052}{e_p} \log_{10} N \quad (1)$$

7  
8  
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11 314 where  $e_m$  and  $e_p$  are model and prototype void ratios, respectively, while the scale factor,  $N$ , is  
12  
13 315 equal to 16.7 in the current study. To compensate for the lower confining stress in the current  
14  
15 316 1g model tests, the sand relative density of the model was chosen to be lower than that  
16  
17 317 representative of the prototype. The loosest achievable state of Babolsar dry sand in the  
18  
19 318 container was  $D_r = 30\%$  (or void ratio of  $e_m = 0.797$ ) which is equivalent to  $D_r = 57\%$  (or  
20  
21 319 void ratio of  $e_p = 0.734$ ) in the prototype scale based on adopted scaling factor ( $N = 16.7$ ). In  
22  
23 320 other words, a prototype made of Babolsar sand with a relative density lower than  $D_r = 57\%$   
24  
25 321 is unlikely to be modeled in the constructed LSB with dry deposition procedure. Hence, this  
26  
27 322 void ratio limitation requires first determining the model relative density prior to model any  
28  
29 323 specific prototype. By employing Eq. (1) and assuming a model void ratio of 0.797 ( $D_r = 30\%$   
30  
31 324 ), the prototype void ratio was calculated to be 0.734 ( $D_r = 30\%$ ) using the adopted scaling  
32  
33 325 factor ( $N = 16.7$ ).  
34

### 326 **Overview of testing program**

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36 327 The overall testing program conducted in this study are as follows:

- 37  
38 328 • *Preliminary tests*: performance evaluation of the LSB container; (1) free-field test on dry  
39  
40 329 sand (PFFD), (2) free-field test on saturated sand (PFFS).  
41  
42 330 • *Main tests*: investigating of boundary condition effects on model response: (1) first series:  
43  
44 331 free-field condition on saturated sand with flexible boundary (FFL) and fixed-end boundary  
45  
46 332 (FFR), (2) second series: shallow foundation on saturated sand with flexible boundary (SFL)  
47  
48 333 and fixed-end boundary (SFR), (3) SDOF super-structure on shallow foundation with  
49  
50 334 flexible boundary (SSFL) and fixed-end boundary (SSFR).

51 335 Details of the testing program and loading characteristics are summarized in Table 4.

52 336 In preliminary tests, two 1g shaking table tests (PFFD and PFFS) were carried out on loose  
53  
54 337 sand layers ( $D_r = 30\%$ ) in both dry and saturated conditions to investigate the performance of  
55  
56 338 the LSB in modeling 1D vertical wave propagation in finite soil column. Each model was  
57  
58 339 shaken longitudinally under a sinusoidal input with an acceleration amplitude of 0.3g and  
59  
60 340 frequency of 10Hz for 8 seconds as shown in Fig. 4.

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3 341 For the main testing program, three series of 1g shaking table tests were carried out to explore  
4 342 the effects of container boundary conditions on saturated loose sand layers ( $D_r = 30\%$ ).  
5  
6 343 Specifically, the testing program was designed to study effects on EPWP generation and  
7  
8 344 liquefaction-induced settlement of shallow foundations and free-field ground. Each series  
9  
10 345 consists of two similar tests that differ only in the lateral displacement boundary condition. The  
11  
12 346 first test of each series was conducted with a flexible boundary where the container layers can  
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14 347 move freely with soil and the second test was conducted with a rigid, fixed-end condition where  
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16 348 the container layers are restrained from moving.  
17  
18 349 In the first testing series (FFL and FFR), the free-field responses of saturated sand layers were  
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20 350 evaluated for both fixed-end and flexible boundary conditions. In the second testing series (SFL  
21  
22 351 and SFR), a rigid steel block was positioned on the sand layer surface as an equivalent shallow  
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24 352 foundation. In these tests, the main objective was to assess the container boundary effects on  
25  
26 353 the soil response in terms of EPWP generation and surface settlement. For the third testing  
27  
28 354 series (SSFL and SSFR), an SDOF was placed atop a thinner shallow foundation to examine  
29  
30 355 the effects of the container on SSI mechanisms under the same contact stress as in the tests in  
31  
32 356 the second testing series.

### 31 357 **Shallow foundation and SDOF model**

33  
34 358 In practice, shallow foundations are generally made of reinforced concrete. However, in  
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36 359 physical modeling experiments, the shallow foundation models are usually made of material  
37  
38 360 with higher density such as steel (with  $\rho = 7800 \text{ kg/m}^3$ ) or brass (with  $\rho = 8700 \text{ kg/m}^3$ ) to  
39  
40 361 provide required contact pressure representing an actual building in prototype scale according  
41  
42 362 to similitude laws. Due to the dimension limitation in reduced-scale physical modeling tests,  
43  
44 363 concrete with lower density (i.e.  $\rho = 2500 \text{ kg/m}^3$ ) was not an appropriate option for shallow  
45  
46 364 foundation models. Target contact pressure of shallow foundation and/or super-structure mass  
47  
48 365 cannot be achieved by using low-density material such as concrete, so steel was used for the  
49  
50 366 footings.

51  
52 367 In the second testing series, the shallow foundation was a rigid block of ST37 steel having a  
53  
54 368 length of 240 mm (4 m in prototype scale) and a width of 60 mm (1 m in prototype scale)  
55  
56 369 shown in Fig. 5 (a). The shallow foundation stretches across the width of the box; thus, plane  
57  
58 370 strain condition can be assumed in these tests because out-of-plane deformation of underlying  
59  
60 371 soil is restricted by the container walls. The shallow foundation has a total contact pressure of  
372 39.9 kPa which represents the prototype-scale surcharge stress representative of a three-story  
373 building assuming that the foundation mass corresponds to a single-story mass.

1  
2  
3 374 In the third testing series, an SDOF super-structure was created by attaching a rigid steel  
4 375 cylinder to a thinner shallow foundation so that the contact stress would be approximately the  
5 376 same as the shallow foundation in the second testing series (see Fig. 5 (b)). The stiffness and  
6 377 mass of the SDOF super-structure were calculated to represent an equivalent three-story  
7 378 building. The structure models of the shallow foundation and SDOF on shallow foundation are  
8 379 shown in Fig 5. In both second and third testing series, the bottom face of the shallow  
9 380 foundation which was in direct contact with soil was roughened to provide friction between the  
10 381 soil and the shallow foundation model. The mechanical and geometrical properties of the  
11 382 shallow foundation and the SDOF resting on a shallow foundation model are summarized in  
12 383 Table 5.

13 384 The shallow foundation structural models were positioned on the soil surface without any  
14 385 embedment in both second and third test series. By embedding the shallow foundation into the  
15 386 soil layer, ground disturbance will occur inevitably which could seriously affect the test results  
16 387 in small-scale 1g shaking table tests. Thus, the current model preparation procedure was  
17 388 configured to minimize soil disturbance by placing the shallow foundation atop of soil surface  
18 389 in order to achieve uniform homogeneous sand layer with relative density of  $D_r = 30\%$  in  
19 390 model scale. Undesirable effects of sample disturbance on results could be more pronounced  
20 391 when the model is made of loose sand that could be easily disturbed (Kumar et al. 2020).  
21 392 Moreover, the scaled embedment depth in the model tests is relatively small and it might be  
22 393 impractical in model test preparation.

### 394 **Instrumentation**

395 Detailed instrumentation layouts for the tests in the experimental program are shown in Fig. 6.  
396 The sensors include five uniaxial accelerometers, two pore pressure transducers, and a linear  
397 potentiometer placed on the soil or foundation surface to quantify the settlements during and  
398 after shaking. Two of the accelerometers (A3 and A2) were installed in a vertical array in the  
399 center of the model at various depths and the other one was placed on the base layer of the  
400 container for measuring the actual input motion (A1). The last accelerometer (A4) was installed  
401 on the soil surface at the right-end side of the container for all tests. The right-end accelerometer  
402 was installed 30 mm away from the end boundary (approximately one-tenth of the model  
403 length) to capture the near-boundary acceleration response.

404 In the second testing series, an accelerometer was mounted on the shallow foundation while in  
405 the third testing series an additional accelerometer was mounted on top of the SDOF super-

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3 406 structure. In all testing series, two pore pressure sensors were embedded in a vertical array at  
4 407 the mid-height and base of the sand layer. In the first testing series, the extension rod of the  
5 408 potentiometer was glued to a thin acrylic plate placed on the soil surface horizontally. The  
6 409 acrylic material for the plate was selected because its density is approximately equal to the  
7 410 density of the liquefied soil whereas the plate settles with the soil surface. Image processing  
8 411 was also used to capture horizontal displacements of the container layers and to track the  
9 412 foundation and SDOF super-structure displacements. Pore pressure at various depths,  
10 413 acceleration time histories, surface settlement, and layers displacement were recorded  
11 414 simultaneously during each test. The measured data are presented in model scale units unless  
12 415 otherwise noted.

## 416 **Results and discussion**

### 417 **Assessment of the LSB performance**

418 To investigate the performance of the laminar shear container in modeling 1D vertical wave  
419 propagation in the finite soil column, preliminary shaking table tests were performed on loose  
420 sand layers in both dry and saturated conditions. In these tests (i.e., PFFD and PFFS), the soil  
421 layers were subjected to harmonic motion shown in Fig. 4. The sand response at the surface  
422 right-end and surface center of the container in both dry and saturated conditions are compared  
423 in Fig. 7 in terms of acceleration time history and spectral acceleration. The results show that  
424 the maximum difference between the right-end and center acceleration time histories does not  
425 exceed 7% which indicates that the boundary effect on recorded right-end acceleration is  
426 negligible and the container is flexible enough to properly model 1D soil column. Similar trend  
427 is especially visible in the spectral acceleration response of the surface center and surface right-  
428 end in both dry and saturated conditions (Fig. 7 (e) and (f)). It can be inferred that the container  
429 provides low boundary constraints during shaking of the model, while still having the sufficient  
430 rigidity to keep nearly at-rest horizontal stresses and plane-strain boundary conditions during  
431 shaking.

### 432 **Boundary effect on the settlement and EPWP**

433 The effects of container boundary conditions on surface settlements and EPWP time histories  
434 during and after shaking at various depths for the three testing series are demonstrated in Fig.  
435 8. The highlighted sections in the graphs reflect the base input shaking time span and the dashed  
436 lines in the EPWP graphs reflect the onset of liquefaction ( $r_u = 1$ ). The ratio of pore water

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3 437 pressure,  $r_u$  was calculated as the attained generated pore water pressure,  $\Delta u$ , divided by the  
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5 438 overburden effective vertical stress at the corresponding elevation,  $\sigma'_{v_0}$  as follows:

$$r_u = \frac{\Delta u}{\sigma'_{v_0}} \quad (2)$$

11 439 It is noting that the surface surcharge for the second and third test series was taken into account  
12 440 in the calculation of the effective overburden stress in Eq. (2). As shown in Fig. 8, the surface  
13 441 settlements in the tests with fixed-end boundary condition (FFR, SFR, and SSFR) are more  
14 442 pronounced in comparison to the flexible ones (FFL, SFL, and SSFL) as expected due to higher  
15 443 energy level trapped inside the model (see Fig. 1). The maximum amount of permanent  
16 444 settlement is observed in shallow foundation case with fixed-end boundary condition with 33  
17 445 mm in model scale which has increased by 112% compared to the corresponding flexible  
18 446 condition (Fig. 8 (b)), while the increase of settlement for free-field and SDOF super-structure  
19 447 conditions are 115% and 58%, respectively (Fig. 8 (a) and (c)). Settlement began to occur  
20 448 immediately from the first couple of cycles and continued with higher rate for fixed-end  
21 449 conditions. Settlement of the models continued to take place with very low rate, even after the  
22 450 base shaking finished especially when super-structure exists, until the termination of  
23 451 reconsolidation in the superior layers, but for better resolution only the first thirty seconds are  
24 452 shown in the graphs. However, the cumulative amount of the post-seismic settlement is  
25 453 negligible compared to the co-seismic settlements.

26 454 In the free-field tests (FFL and FFR), the transient EPWP rapidly increased from zero to a  
27 455 constant level for the flexible case (FFL) and the model was thoroughly liquefied during the  
28 456 first cycles of shaking at mid-height and base of the soil layer (see Fig. 8 (a)). Similar trend  
29 457 was observed for the FFR test; however, the maximum EPWP never reached the liquefaction  
30 458 limit in both elevations (mid-height and model base). Once shaking ceased, pore water pressure  
31 459 dissipation began in both fixed-end and flexible models with a little delay in flexible condition.  
32 460 This delay in EPWP dissipation is attributed to upward migration of pore water and  
33 461 solidification front after the end of shaking which begins at the base layer followed by delayed  
34 462 pore pressure dissipation in the above layers. In post-shaking time, the dissipation rate was  
35 463 noticeably higher in the flexible condition compared to the fixed-end condition especially in  
36 464 the model base. Further, higher fluctuations in EPWPs were observed in the fixed-end  
37 465 condition due to the existence of unwanted superfluous waves.

38 466 In the second series of tests (SFR and SFL), fully liquefaction was not observed beneath the  
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3 467 foundation. The amount of EPWP in the soil layer beneath the shallow foundation was slightly  
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5 468 higher than those in free-field test but the peak of EPWP was considerably far from the  
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7 469 liquefaction limit ( $r_u = 1$ ). Similar to the second test series, for the shallow foundation with an  
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9 470 overlying SDOF (i.e. SSFL and SSFR tests), the amount of EPWP in the soil layer beneath the  
10  
11 471 structure was higher than those in free-field. In fact, only partial liquefaction occurred in the  
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13 472 soil immediately beneath the foundation ( $r_u < 1$ ). Dissipation rate in the whole soil column  
14  
15 473 was higher for flexible condition.

#### 16 474 **Boundary effect on the acceleration response**

17  
18 475 The input acceleration introduced at the model base propagates in vertical direction along the  
19  
20 476 soil stratum and may be attenuated or amplified depending on the characteristics of the input  
21  
22 477 excitation and the dynamic properties of the soil. Acceleration time histories for all test series  
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24 478 are shown in Fig. 9. The dashed lines in the acceleration graphs reflect the input motion  
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26 479 acceleration amplitude (0.3g). Comparison of the time histories indicates that there are  
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28 480 discrepancies between the acceleration time histories recorded at various locations inside the  
29  
30 481 model in both flexible and fixed-end boundary conditions in free-field, shallow foundation,  
31  
32 482 and SDOF super-structure on a shallow foundation. During the initial cycles of shaking,  
33  
34 483 acceleration responses were fairly identical at the various depths of both flexible and fixed-end  
35  
36 484 conditions in all tests, since soil stiffness degradation is not high enough during the initial  
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38 485 cycles of shaking.

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40 486 In the free-field tests (FFL and FFR), after the first stages of loading, the acceleration response  
41  
42 487 was attenuated for the flexible case at the mid-height and then slightly amplified in higher level.  
43  
44 488 The former phenomenon indicates on a shear localization within the sand somewhere between  
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46 489 mid-height and bottom of the model. However, this trend in fixed-end condition was not  
47  
48 490 analogous after soil degradation commenced; the acceleration time history at mid-height is  
49  
50 491 almost identical to the base input and from the mid-height to the surface, a decay in acceleration  
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52 492 response is visible (see Fig. 9 (a)). This difference is likely due to P-waves generated by  
53  
54 493 reflections from fixed-end walls during shaking. However, the values of peak acceleration  
55  
56 494 increased while propagation of the wave from the base to the surface.

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58 495 In the tests with shallow foundation (SFL and SFR), the values of peak acceleration increased  
59  
60 496 as the input wave traveled from the mid-height to the soil surface. At shallower depths of the  
61  
62 497 fix-end container, the sand response is more affected by the softening of underlying sand and  
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64 498 caused more noticeable acceleration changes with time. In contrast, in flexible condition, a  
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66 499 relatively smooth acceleration response was captured.



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3 500 For the shallow foundation tests with overlying SDOF super-structure (SSFL and SSFR),  
4 501 analogous trend was captured indicating an increase of the peak acceleration values at the  
5 502 beginning of loading cycles from the mid-height to the surface for the rigid condition.  
6  
7 503 Nevertheless, the accelerometer installed on top of the foundation measured different responses  
8  
9 504 for the rigid case compared to the corresponding shallow foundation case probably due to SSI  
10 505 effects. The acceleration responses at the top of foundation were not amplified during the first  
11 506 cycles of shaking. For the rest of loading cycles, attenuation in acceleration responses are  
12 507 obvious which may be attributed to softened subsoil layer. For the flexible boundary condition  
13 508 case, the peak acceleration values at the beginning of the loading decreased as the input wave  
14 509 propagated along the soil layer from base to soil surface. After the initial cycles of shaking,  
15 510 increases in soil softening resulted in a higher damping ratio within the soil layer which in turn  
16 511 lead to significant phase lag between input and measured acceleration in all elevations. This  
17 512 could explain the acceleration necking after the early loading cycles in all tests with flexible  
18 513 boundary condition.

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28 514 Similar to the recorded acceleration time histories, the spectral acceleration responses derived  
29 515 from the accelerometers with 5% damping ratio are shown in Fig. 10 at various locations for  
30 516 free-field, shallow foundation on soil surface, and SDOF super-structure on shallow foundation  
31 517 during the shaking. Higher spectral acceleration amplitude for the fixed-end condition is  
32 518 observed especially near the input motion frequency ( $T=0.1s$ ) for all the cases most likely  
33 519 because of response amplification in fixed-end condition due to higher energy level. The  
34 520 evident lower acceleration amplitudes shown in Fig. 10 for the flexible conditions are mainly  
35 521 due to freely movement of the container layers. The slight shift of peak spectral acceleration to  
36 522 the right for the flexible condition curve indicates that the flexible container stiffness is lower  
37 523 compared to the fixed-end boundary condition.

#### 524 **Results of image processing analysis of layers**

525 Visualizing displacement of the container layers could be very helpful for better understanding  
526 of container performance and system response to base input while shaking. Image processing  
527 technique was used for capturing horizontal displacement of container layers during shaking  
528 for all test series. For this purpose, as shown in Fig. 11, two synchronized cameras were used  
529 to simultaneously capture the displacement of soil surface and points on structure in addition  
530 to horizontal movements of container layers. Fig. 12 shows the results of the image process  
531 analysis of odd layers from the base layer to the 17th layer for the flexible tests in comparison  
532 with the base (or rigid) displacement input.

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3 533 In the first couple of loading cycles, displacement of layers almost matches the base input, but  
4 534 immediately after, maximum layer displacements decayed to an approximately constant value.  
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6 535 Displacement of the 3rd layer was almost identical to the base input in all three testing series.  
7  
8 536 However, the peak values of displacement diminished from the highest value at the base (1st  
9 537 layer) to the lowest value near the sand surface (15th layer). The remarkable difference between  
10 538 measured displacement at layer 15 and layer 17 can be inferred as a shear localization near the  
11 539 15th layer. Nevertheless, more irregularity can be seen in displacement time history for the  
12 540 second and third test series near soil surface probably due to super-structure and SSI effects on  
13 541 responses.

14  
15 542 At shallower depths, where soil responses are affected by the softened deeper layer,  
16 543 displacement changes with time are more noticeable. Hence, deviation in maximum  
17 544 displacement response of layers is considerably larger particularly at shallower depth for  
18 545 shallow foundation and SDOF on shallow foundation.

19  
20 546 The results of the image process analysis of layers displacement from the base layer to the top  
21 547 layer (18th) are plotted in Fig. 13 between  $t=1s$  and  $t=1.6s$  of the shaking time (a, c, and e) with  
22 548 0.1 second time steps and  $t=2s$  to  $t=8s$  (b, d, and f) with the corresponding pore pressure build-  
23 549 up ratio ( $r_u$ ) for free-field condition, shallow foundation, and shallow foundation with a SDOF  
24 550 super-structure. In the free-field condition, after  $t=1.2s$  (Fig. 13 (a)), the displacement of the  
25 551 10th layer decrease significantly and relative displacement of the above layers declined to zero  
26 552 which can be inferred that shear localization has taken place somewhere near 10th layer due to  
27 553 pore pressure build-up. This was first apparent in Fig. 9 (a) for the acceleration response near  
28 554 mid-height in free-field condition. Significant reduction in acceleration responses at the mid-  
29 555 height in Fig. 9 (a) could be caused by formation of shear localization somewhere near mid-  
30 556 height preventing waves from traveling upward along the soil layer. Thus, the above layers  
31 557 (11th to 16th) move like a rigid body without any shear deformation until the last two layers  
32 558 (17th and 18th layers) that are close to the soil surface where the strain non-uniformity is  
33 559 considerable. However, for the rest of the shaking time ( $t=2s$  to  $t=8s$ ), layers displacements  
34 560 decrease significantly and barely surpassed 0.6 mm. This trend is especially seen in Fig. 12  
35 561 following the first couple of cycles when the pore pressure ratio reaches its maximum value.

36  
37 562 In the cases with shallow foundation (2nd and 3rd test series), the layers displacement for the  
38 563 time span of  $2s < t < 8s$  are larger in comparison with the free-field condition. This may refer to  
39 564 incomplete liquefaction mainly because of the existence of shallow foundation on the soil  
40 565 surface. The additional surcharge imposed by placement of the shallow foundation leads to an

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3 566 increase in effective stress and confining pressure beneath the foundation which is a beneficial  
4 567 factor in minimizing EPWP build-up during shaking. At the same time, soil beneath and around  
5 568 the foundation tends to move laterally due to largely applied shear stress. This mechanism  
6 569 causes the soil to dilate, resulting in a reduced EPWP build-up and incomplete liquefaction ( $r_u$   
7 570  $< 1$ ) in this region (Mehrzaad et al. 2018).

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12 571 In some recent experimental studies in both 1g shaking table and Ng centrifuge tests, lower or  
13 572 even negative EPWP beneath the foundations have been reported compared to free-field soil  
14 573 responses (Alam and Towhata 2008; Adalier et al. 2003). In another research, Karimi et al.  
15 574 (2015) showed that a reduction in excess pore pressure ratio ( $r_u$ ) under the foundation is  
16 575 observed compared to the free-field condition, due to elevated confining pressure in the soil  
17 576 layer. However, this increased resistance to EPWP generation and liquefaction under the  
18 577 foundation was experimentally shown to depend on building's confining pressure, relative  
19 578 density of soil, and ground motion intensity. Sand with a higher relative density is more  
20 579 resistant to seismically-induced EPWP build-up and strength loss. Hence, complete  
21 580 liquefaction ( $r_u = 1$ ) was observed in the free-field but not under the footing.

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30 581 In the shallow foundation condition (Fig. 13 (c) and (d)), the 14th layer has the smallest  
31 582 displacement before  $t=1.6s$ , but remarkable decrease in recorded displacement of the 8th layer  
32 583 for the period of  $2s < t < 8s$  indicates two different localizations probably occurred in different  
33 584 times which the last one lead to different soil behavior between above and below the 8th layer.  
34 585 For the shallow foundation with an SDOF super-structure, shear localization occurred in the  
35 586 13th layer and similar to the free-field conditions, the displacements in the uppermost layers  
36 587 were smaller. The same trend for the last two layers (17th and 18th layers) is also visible as the  
37 588 free-field condition.

## 38 39 40 41 42 43 44 45 589 **Summary and conclusion**

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47 590 As preliminary tests, two 1g shaking table tests were performed to investigate the seismic  
48 591 performance of the newly constructed LSB on saturated and dry loose sand deposit in modeling  
49 592 the free-field condition. The container was designed in a way that can provide both flexible  
50 593 and fixed-end boundary conditions by adding elements to the moving end walls to restrain  
51 594 layers horizontal displacement. The results of preliminary tests confirm that the LSB permits  
52 595 free movement of dry and saturated loose sand layers. Besides, the container provides low  
53 596 boundary constraints during shaking of the model, while still having sufficient rigidity to keep  
54 597 nearly at-rest horizontal stresses and plane-strain boundary conditions during shaking. Then,

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3 598 three series of 1g shaking table tests were conducted as a main testing program to evaluate the  
4 599 impact of boundary conditions on the seismic response of saturated sand layers (1) in free-field  
5 600 conditions, (2) with a shallow foundation, and (3) with a shallow foundation supporting a single  
6 601 degree of freedom (SDOF) super-structure on saturated loose sand layers. Specifically, the  
7 602 effects of fixed-end and flexible boundary conditions on the excess pore water pressure  
8 603 (EPWP) build-up, surface settlement, and acceleration response were evaluated in direct  
9 604 comparisons for the main testing program.

15 605 According to the results of the main experiment program, the following conclusions can be  
16 606 drawn on the issue of container boundary condition effects in 1g shaking table tests conducted  
17 607 on saturated loose sand:

21 608 1) Surface settlements for sand layers with fixed-end boundary conditions are remarkably  
22 609 greater than those observed in the flexible boundary conditions, possibly due to trapping of  
23 610 energy inside the sand layer in the fixed-end condition.

27 611 2) Complete liquefaction was only seen in the free-field condition while only partial  
28 612 liquefaction (i.e.  $r_u < 1$ ) occurred when the sand layer was overlain by a shallow foundation  
29 613 model (2nd series) or SDOF super-structure model (3rd series). This is mainly attributed to two  
30 614 main mechanisms: (1) the soil beneath and around the foundation tends to move laterally away  
31 615 due to the applied shear stress which causes the soil medium to dilate, resulting in a reduction  
32 616 of the EPWP and leads to incomplete liquefaction; and (2) increased confining pressure due to  
33 617 presence of shallow foundation model caused higher cyclic strength and resistance to EPWP  
34 618 generation increases.

42 619 3) As shaking started, the transient EPWP rapidly increased in both flexible and fixed-end  
43 620 conditions. However, the maximum amounts of EPWP in flexible cases were higher than those  
44 621 in corresponding fixed-end cases. Immediately after shaking ceased, EPWP dissipation began  
45 622 in both flexible and fixed-end models. A greater rate of EPWP dissipation was observed in the  
46 623 container with flexible boundary conditions.

52 624 4) During the initial cycles of shaking, acceleration responses were similar at various depths in  
53 625 both laminar and fixed-end conditions, since soil stiffness degradation was not high enough.  
54 626 Nevertheless, the value of acceleration peaks increased as the input wave traveled from the  
55 627 bottom layer to the top for the fixed-end condition in all testing series. At shallower depths,  
56 628 where softening of underlying soil layers was more significant, acceleration changes with time  
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3 629 were more noticeable. Therefore, variation of maximum acceleration responses was  
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5 630 considerably larger especially at shallower depth for the fixed-end condition. In contrast, in the  
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7 631 flexible condition, a relatively smooth acceleration response was captured. This difference is  
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9 632 likely due to effects P-waves generated by reflections from fixed-end walls during shaking.

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11 633 5) After the initial cycles of shaking, acceleration attenuation at the mid-height in the flexible  
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13 634 conditions for all test series could be a sign of shear localization somewhere near the mid-  
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15 635 height. Increasing soil softening caused higher damping ratio within the soil layer which in turn  
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17 636 lead to significant phase lag between input and measured acceleration in all elevations. This  
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19 637 could explain the acceleration necking after the early loading cycles in all flexible conditions.  
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21 638 This trend is similar for the free-field condition with fixed-end boundary. However, in the  
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23 639 second and third series with fixed-end boundaries, soil re-stiffening caused higher acceleration  
24  
25 640 response after the acceleration attenuation in the early cycles of shaking.

26 641 6) Higher spectral acceleration amplitude for the fixed-end conditions was observed especially  
27  
28 642 near the input motion frequency for all the cases most likely because of response amplification  
29  
30 643 in the fixed-end condition due to higher energy level. The slight shift of peak spectral  
31  
32 644 acceleration to the right for flexible condition curve indicates that the flexible container  
33  
34 645 stiffness was lower compared to the fixed-end condition. Thus, the flexible boundary condition  
35  
36 646 is closer to the actual situation and it is preferable over the fixed-end boundary in replicating  
37  
38 647 free-field seismic response.

39 648 7) The captured displacement of the container laminates tracked using image processing  
40  
41 649 technique for flexible tests showed that displacement values decreased from a maximum at the  
42  
43 650 base layer to the lowest amount near the surface. Further, visual observations indicate that some  
44  
45 651 non-uniformity in layers displacements for shallow foundation and SDOF super-structure cases  
46  
47 652 near the surface. In the first couple of loading cycles, displacement of layers almost matches  
48  
49 653 the base input, but immediately after, maximum layer displacement decayed to an  
50  
51 654 approximately constant value.

52 655 8) Plotting the results of layers displacement obtained from image process analysis between  
53  
54 656  $t=1s$  and  $t=1.6s$  with 0.1 second time steps and  $t=2s$  to  $t=8s$  with the corresponding pore  
55  
56 657 pressure build-up ratio ( $r_u$ ) helps to gain a better understanding of how the soil layers behave  
57  
58 658 during shaking. In the free-field condition, after  $t=1.2s$ , the displacement of the 10<sup>th</sup> layer  
59  
60 659 decreased significantly and relative displacement of the above layers declined to zero which

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3 660 means the above layers move like a rigid body. It could be inferred that shear localization has  
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5 661 taken place somewhere near the 10th layer. This phenomenon occurred twice for the second  
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7 662 test series in two different times while in the third series, only one shear localization occurred  
8  
9 663 near the 13rd layer. Besides, larger layer displacements for the time span of  $2s < t < 8s$ , when the  
10  
11 664 foundation model exists on the soil surface (2nd and 3rd series) may be related to incomplete  
12  
13 665 liquefaction compared free-field condition.

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3 911 **List of Tables**  
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5 912 **Table 1** Summary of the laminar shear containers employed in previous studies  
6

7 913 **Table 2** IIEES laminar shear container properties  
8

9 914 **Table 3** Similitude factors  
10

11 915 **Table 4** Summary of the shaking table tests and input motion characteristics  
12

13 916 **Table 5** Properties of structure models (parameters are in model scale)  
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3 **917 List of Figures**  
4

5 **918 Fig. 1** Boundary conditions during 1D shaking, (a) in a semi-infinite half-space, (b) in a rigid smooth end wall  
6 container  
7

8 **920 Fig. 2** Schematic view of the designed laminar shear container (dimensions are in mm); (a) overall view of the  
9 container fastened on the shaking table, (b) top view of the container in deformed shape, (c) laminate (frame)  
10 plan view, (d) container elevation view, (e) section A-A longitudinal I-beam, (f) section B-B transverse I-beam  
11  
12

13 **923 Fig. 3** Grain size distribution for Babolsar sand  
14

15 **924 Fig. 4** Input motion for all tests  
16

17 **925 Fig. 5** Structure models; (a) shallow foundation, (b) SDOF super-structure on shallow foundation  
18

19 **926 Fig. 6** Instrumentation layout (dimensions are in mm); (a) free-field with flexible boundary, (b) free-field with  
20 fixed-end boundary, (c) shallow foundation with flexible boundary, (d) shallow foundation with fixed-end  
21 boundary, (e) SDOF super-structure on shallow foundation with flexible boundary, (f) SDOF Super-structure on  
22 foundation with fixed-end boundary  
23  
24

25 **930 Fig. 7** Comparison of acceleration responses at surface center and surface right-end for free-field sand layers in  
26 saturated and dry conditions; (a) entire shaking duration (saturated), (b) entire shaking duration (dry), (c) shaking  
27 from 0.5 to 2.5s (saturated), (d) shaking from 0.5 to 2.5s (dry), (e) spectral acceleration (saturated), (f) spectral  
28 acceleration (dry)  
29  
30

31 **934 Fig. 8** Recorded time history of EPWP at mid-height and model base and settlement at the surface of a saturated  
32 sand layer for both flexible and fixed-end boundary conditions; (a) free-field, (b) shallow foundation, (c) SDOF  
33 on shallow foundation  
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35

36 **937 Fig. 9** Recorded acceleration time history at different locations for both flexible and fixed-end boundary condition;  
37 (a) free-field, (b) shallow foundation, (c) SDOF on shallow foundation  
38

39 **939 Fig. 10** Spectral acceleration (5% damping) at different locations for both flexible and fixed-end boundary  
40 conditions; (a) free-field, (b) shallow foundation, (c) SDOF on shallow foundation  
41  
42

43 **941 Fig. 11** Image processing technique used for capturing displacements; (a) SDOF on shallow foundation, (b)  
44 shallow foundation, (c) container layers  
45

46 **943 Fig. 12** Horizontal displacement of selected container layers obtained from image processing during shaking;  
47 (a) free-field, (b) shallow foundation, (c) SDOF on shallow foundation.  
48

49 **945 Fig. 13** Horizontal displacement of container layers obtained from image processing during shaking regarding  
50 pore water pressure generation ratio ( $r_u$ ); (a) free-field from  $t=1s$  to  $t=1.6s$ , (b) free-field from  $t=2s$  to  $t=8s$ , (c)  
51 shallow foundation from  $t=1s$  to  $t=1.6s$ , (d) shallow foundation from  $t=2s$  to  $t=8s$ , (e) SDOF on shallow foundation  
52 from  $t=1s$  to  $t=1.6s$ , (f) SDOF on shallow foundation from  $t=2s$  to  $t=8s$   
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949 **TABLES**950 **Table 1** Summary of the laminar shear containers employed in previous studies

Shape	Shaking	L×W×H* [mm]	L/H or D/H*	1g/Ng	Design note	Reference
Square	uniaxial	350×350×350	1.46	Ng	Stack of silicone-oiled laminates without bearings	Suzuki et al. (1991)
Rectangular	uniaxial	457×254×254	1.8	Ng	Stack of laminates separated by bearings	Van Lak et al. (1994)
Rectangular	uniaxial	900×350×470	1.9	1g	Stack of laminates separated by bearings	Gibson (1997)
Circular	biaxial	2280×2130 (D×H)	1.1	1g	Container hanging on the top laminates supported by a frame	Meymand (1998)
Polygonal (12-sided)	biaxial	584×500 (D×H)	1.17	Ng	Stack of laminates separated by bearings	Shen et al. (1998)
Rectangular	uniaxial	1000×500×1000	1	1g	Stack of laminates separated by bearings	Prasad et al. (2004)
Square	biaxial	1000×1000×1000	1	1g	Stack of laminates separated by ball bearings	Jafarzadeh (2004)
Rectangular	uniaxial	710×355×355	2	Ng	Stack of laminates separated by bearings	Pamuk et al. (2007)
Rectangular	biaxial	1888×1888×1520	1.2	1g	Laminates supported by a frame to move independently	Ueng et al. (2006)
Polygonal (8-sided)	biaxial	5000×2750×3210	0.81	1g	Ball bearings placed between the laminates	Thevanayagam et al. (2009)
Rectangular	uniaxial	900×450×807	1.1	1g	Individually supported laminates connected to an external frame	Turan et al. (2009)
Rectangular	uniaxial	1800×600×1500	1.2	1g	Stack of aluminum laminates separated by rollers	Ecemis (2013)
Rectangular	uniaxial	2100×1300×1100	1.6	1g	Stack of laminates separated by rubber layers	Tabatabaieifar (2016)
Rectangular	uniaxial	381×241×266	1.4	Ng	Individually-mounted laminates to an external frame to slide independently	Zayed et al. (2017)

\* L, W, H, and D stand for length, width, height, and diameter, respectively

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**Table 2** IIEES laminar shear container properties

<b>Geometric/ physical properties</b>	<b>Description</b>		
Outer dimension	L=500 mm	W=334 mm	H=263 mm
Inner dimension	L=365 mm	W=243 mm	H=263 mm
No. of frames	18 (including one fixed laminate at the base)		
Box-to-soil mass ratio	0.24 (with assumption of $\rho_s = 2 \text{ ton/m}^3$ for soil media)*		
Inter-layer friction coefficient	0.01		
Approximate weight (kg)	62.2/15.6 (full/empty)		

\*  $\rho_s$  is mass density of soil media

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954

**Table 3** Similitude factors

Parameter	Scaling factor (Iai 1989)	Prototype
Length	$N$	1
Density	1	1
Strain	1	1
Mass	$N^3$	1
Acceleration	1	1
Velocity	$N^{\frac{1}{2}}$	1
Stress	$N$	1
Modulus	$N$	1
Stiffness	$N^2$	1
Force	$N^3$	1
Shaking time	$N^{\frac{1}{2}}$	1
Frequency	$N^{-\frac{1}{2}}$	1

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956

**Table 4** Summary of the shaking table tests and input motion characteristics

Test series	Test ID*	Test description	Boundary condition	Soil condition	D <sub>r</sub> (%)	Input Motion	
						Freq. (Hz)	Acc. amp. (g)
Preliminary tests	PFFD	Free-field	Laminar	Dry	30	10	0.3
	PFFS	Free-field	Laminar	Saturated	30	10	0.3
1 <sup>st</sup> series	FFL	Free field	Laminar	Saturated	30	10	0.3
	FFR	Free field	Rigid	Saturated	30	10	0.3
2 <sup>nd</sup> series	SFL	Shallow foundation	Laminar	Saturated	30	10	0.3
	SFR	Shallow foundation	Rigid	Saturated	30	10	0.3
3 <sup>rd</sup> series	SSFL	S. Foundation + SDOF	Laminar	Saturated	30	10	0.3
	SSFR	S. Foundation + SDOF	Rigid	Saturated	30	10	0.3

\* PFFD: preliminary free-field on dry sand; PFFS: preliminary free-field on saturated sand; FFL: free-field on saturated sand with flexible boundary; FFR: free-field on saturated sand with fixed-end boundary; SFL: shallow foundation on saturated sand with flexible boundary; SFR: shallow foundation on saturated sand with fixed-end boundary; SSFL: shallow foundation with SDOF super-structure on saturated sand with flexible boundary; SSFR: shallow foundation with SDOF super-structure on saturated sand with fixed-end boundary

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**Table 5** Properties of structure models (parameters are in model scale)

Parameters	Shallow foundation	Shallow foundation with SDOF super-structure	
		SDOF super-structure	Shallow foundation
Length (mm)	240	240	240
Width (mm)	60	40 (diameter)	60
Height (mm)	31	60	6
Mass (gr)	3516	3117.1	533.7
Contact pressure* (kPa)	39.9	35.4**	6.1**
Young modulus E (kPa)	$2.1 \times 10^6$	$2.1 \times 10^6$	$2.1 \times 10^6$

\* In prototype scale

\*\* Total contact pressure of shallow foundation with SDOF super-structure is 41.5 kPa (35.4+6.1)

959

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FIGURES

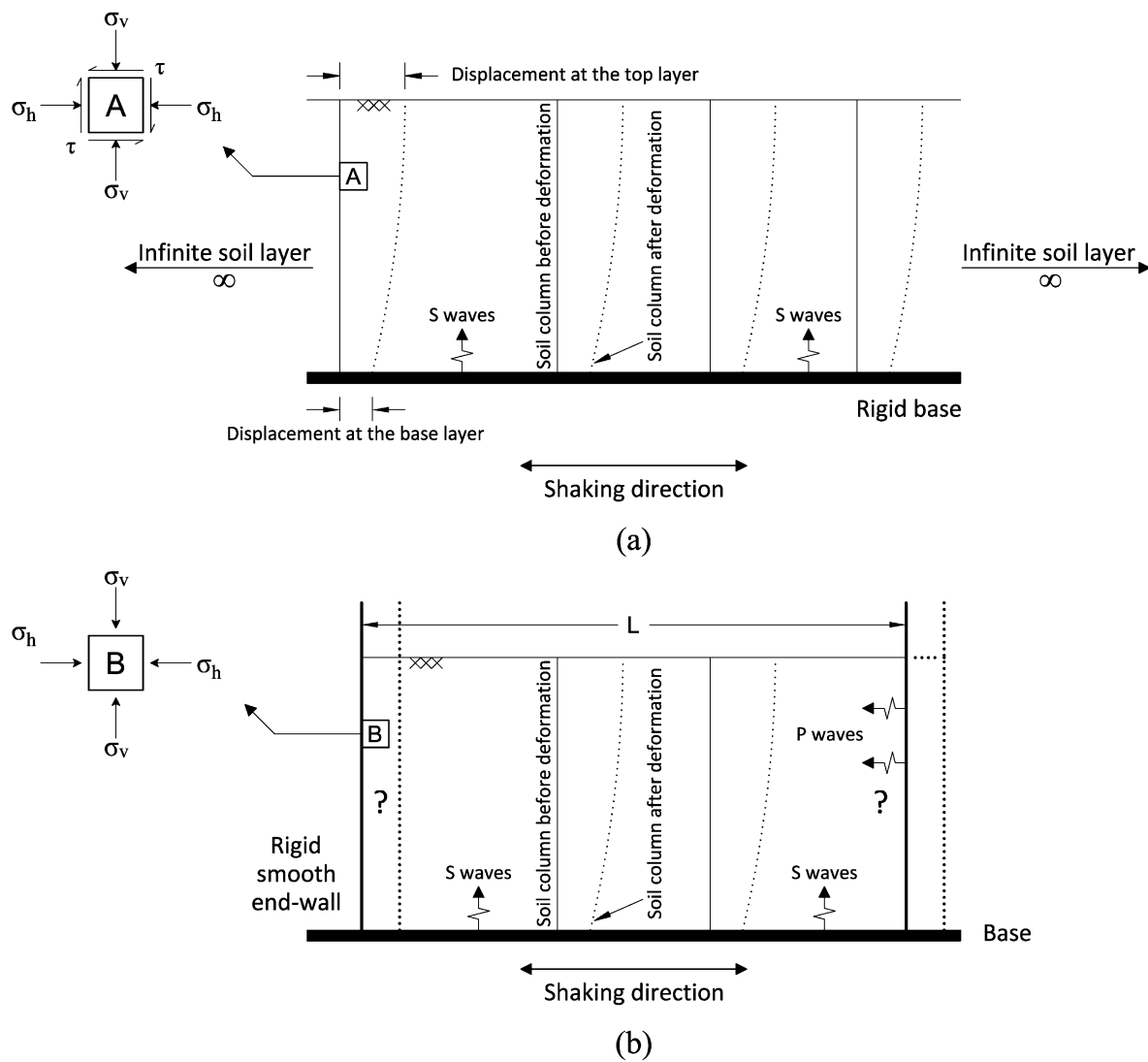
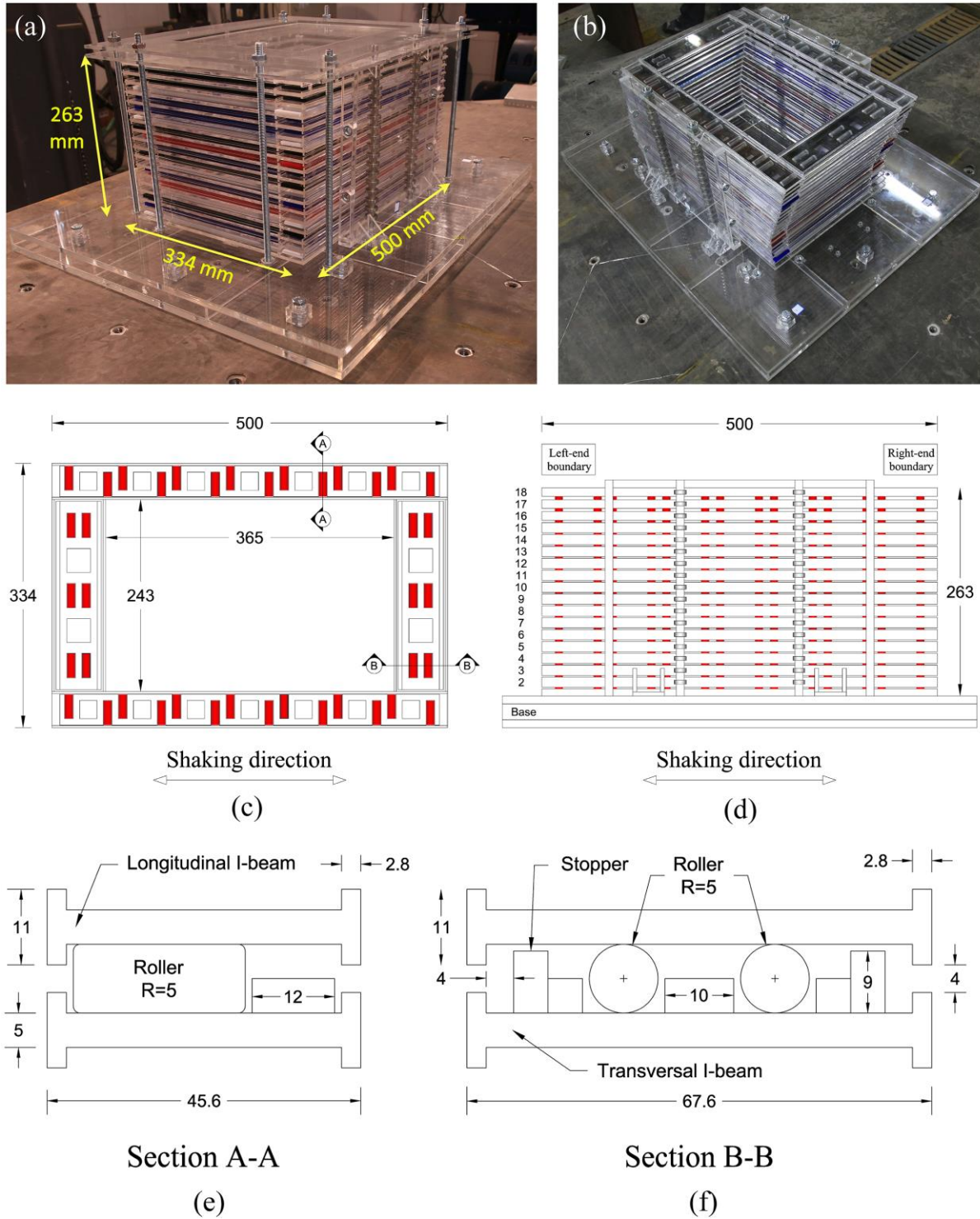
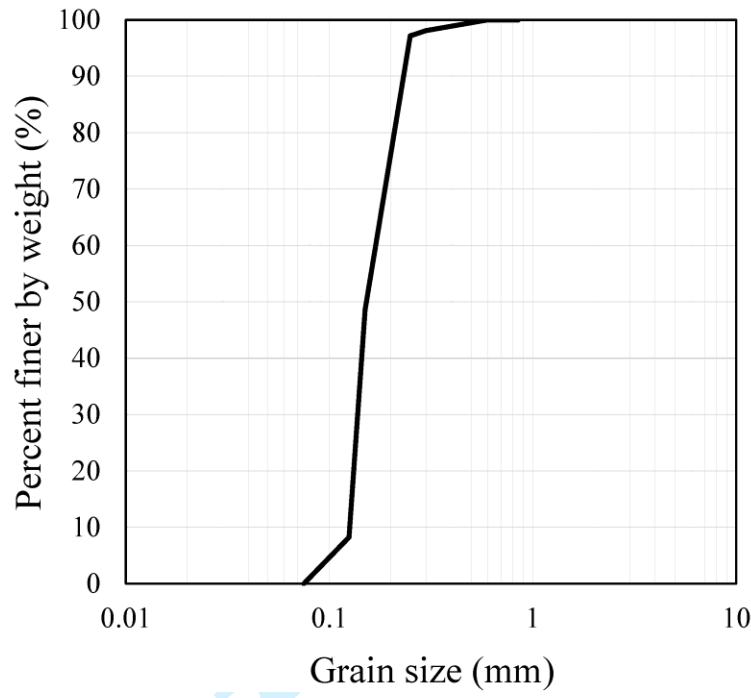


Fig. 1 Boundary conditions during 1D shaking; (a) in a semi-infinite half-space, (b) in a rigid smooth end wall container



**Fig. 2** Schematic view of the designed laminar shear container (dimensions are in mm); (a) overall view of the container fastened on the shaking table, (b) top view of the container in deformed shape, (c) laminate (frame) plan view, (d) container elevation view (e) section A-A longitudinal I-beam, (f) section B-B transverse I-beam

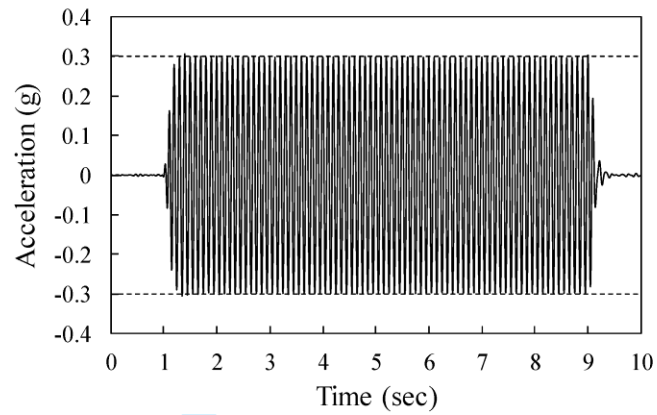
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**Fig. 3** Grain size distribution for Babolsar sand

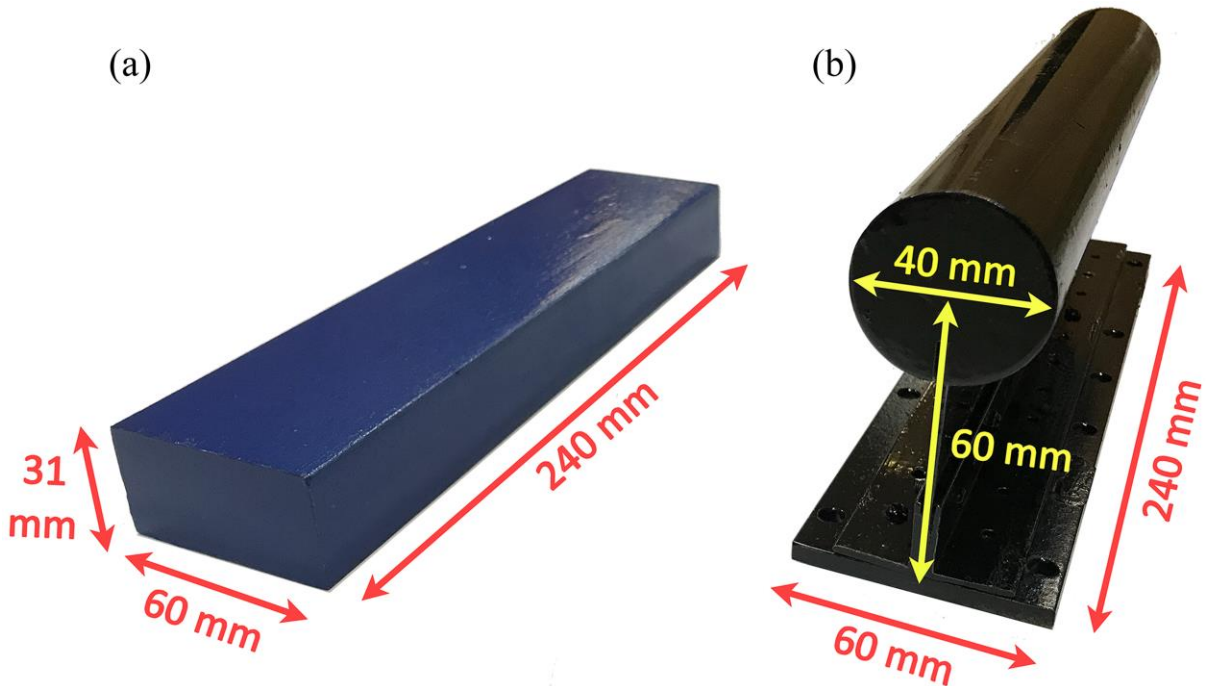


Base motion	Harmonic (sine wave)
Frequency (Hz)	10
Acceleration amplitude (g)	~ 0.3
Duration	8 seconds

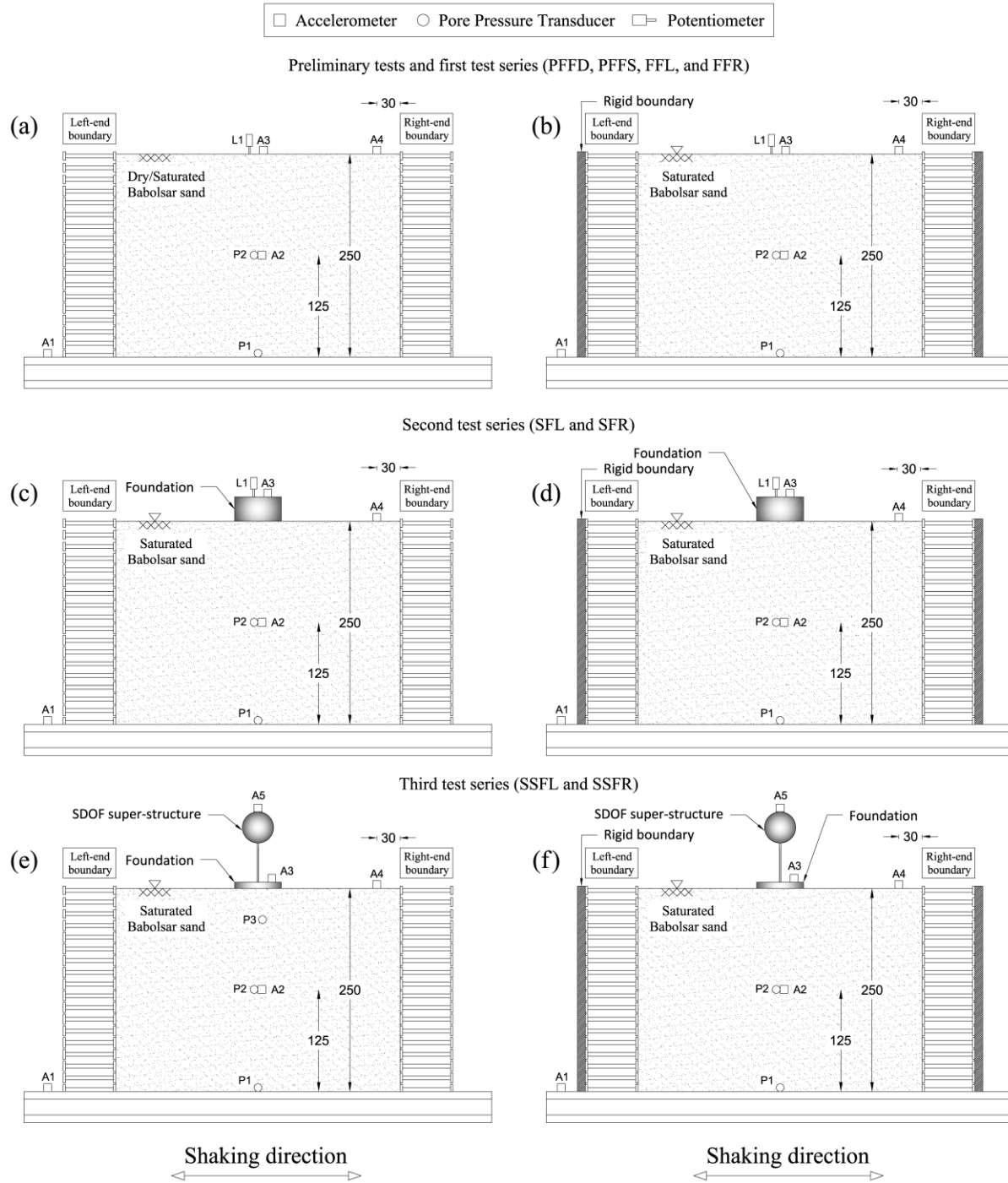


**Fig. 4** Input motion for all tests

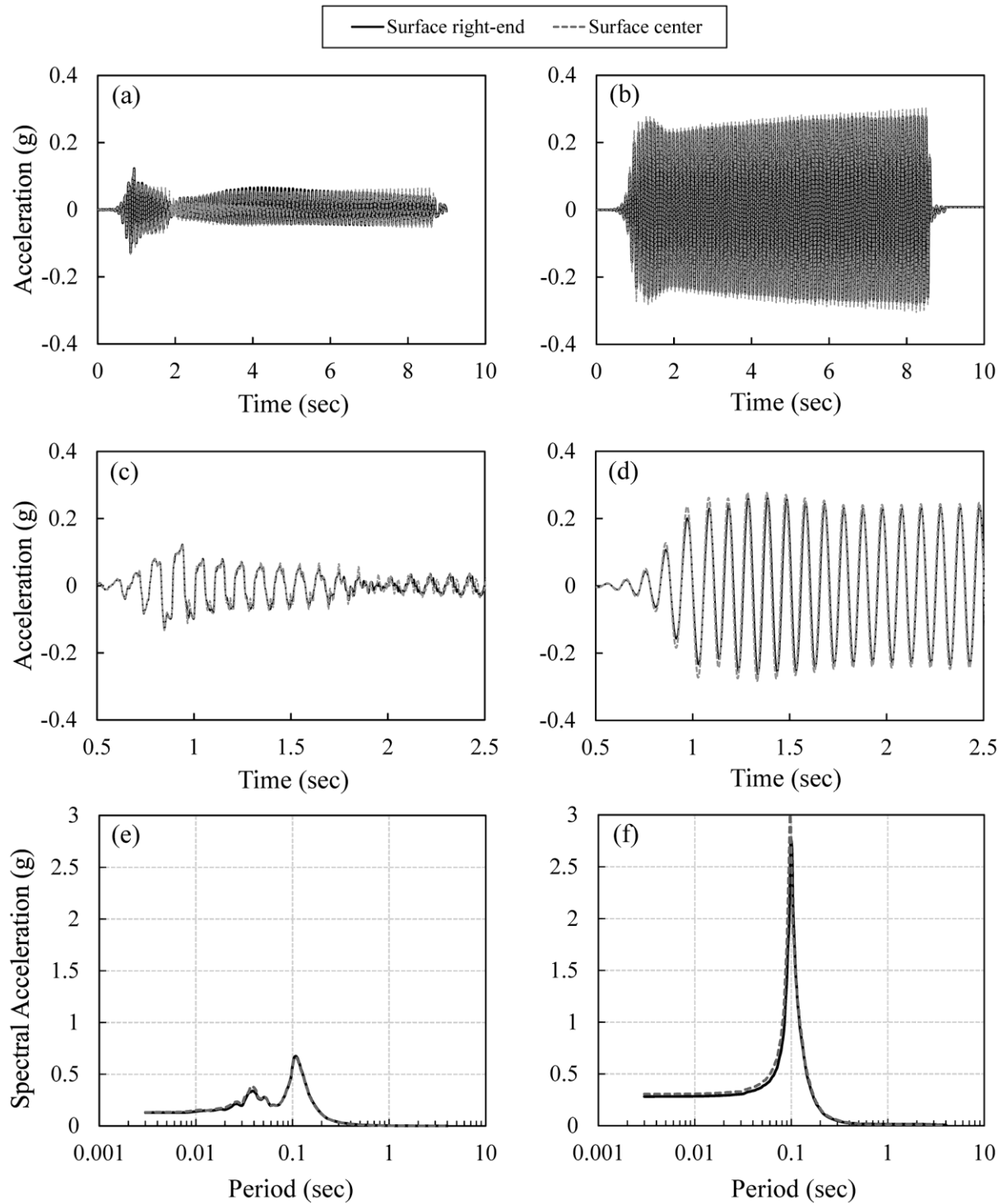
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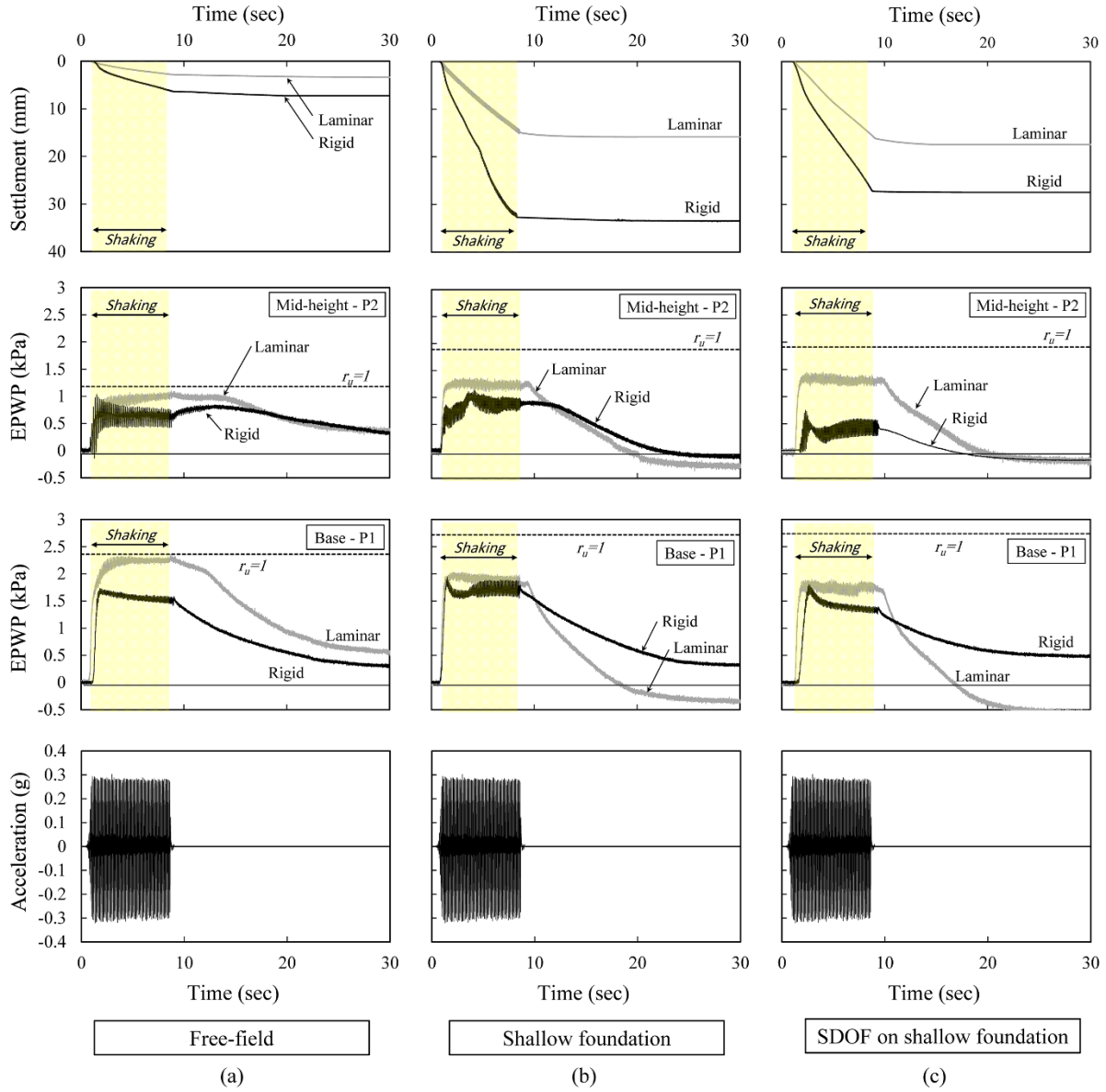
**Fig. 5** Structure models; (a) shallow foundation, (b) SDOF super-structure on shallow foundation



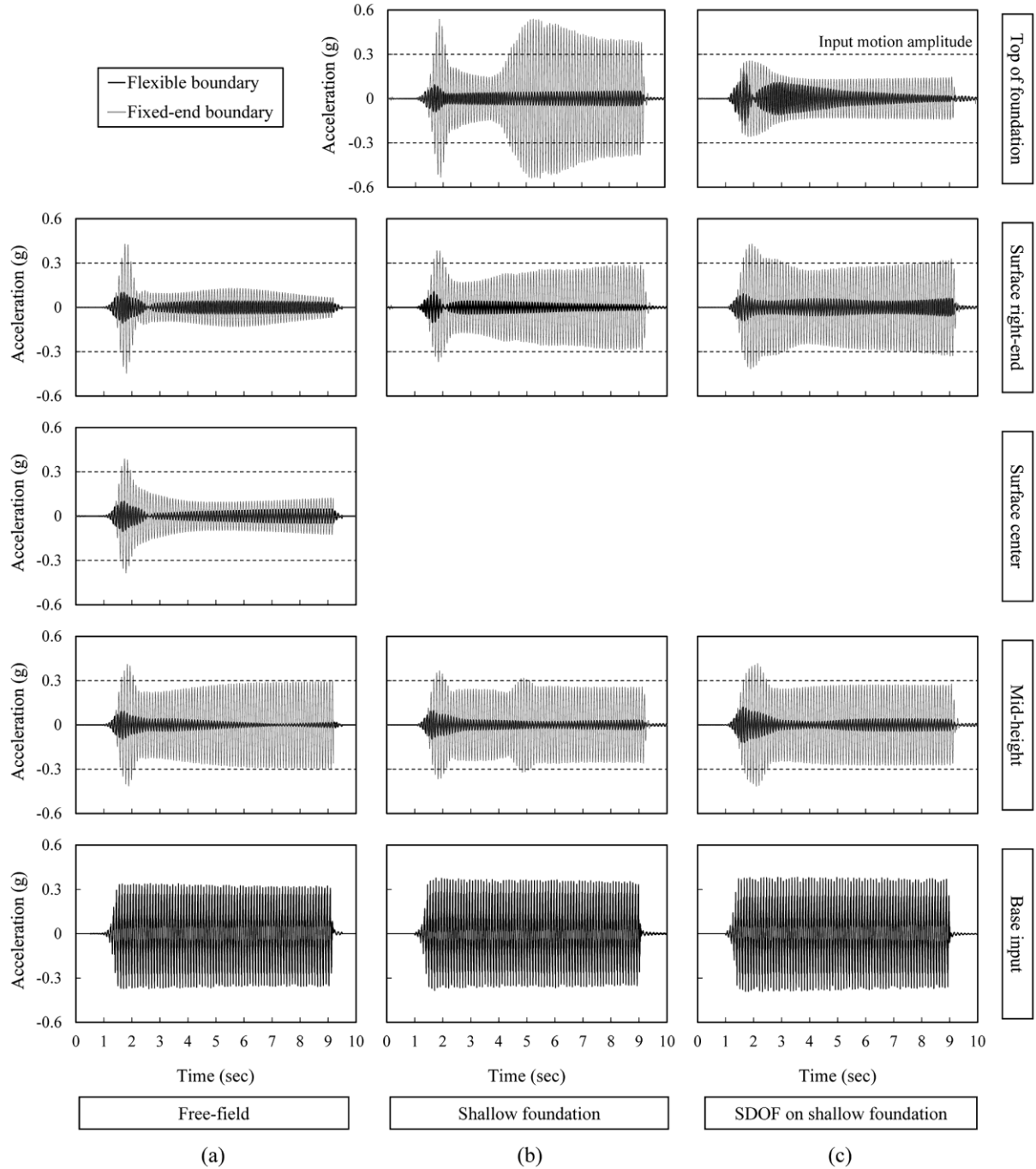
**Fig. 6** Instrumentation layout (dimensions are in mm); (a) free-field with flexible boundary, (b) free-field with fixed-end boundary, (c) shallow foundation with flexible boundary, (d) shallow foundation with fixed-end boundary, (e) SDOF super-structure on shallow foundation with flexible boundary, (f) SDOF Super-structure on foundation with fixed-end boundary



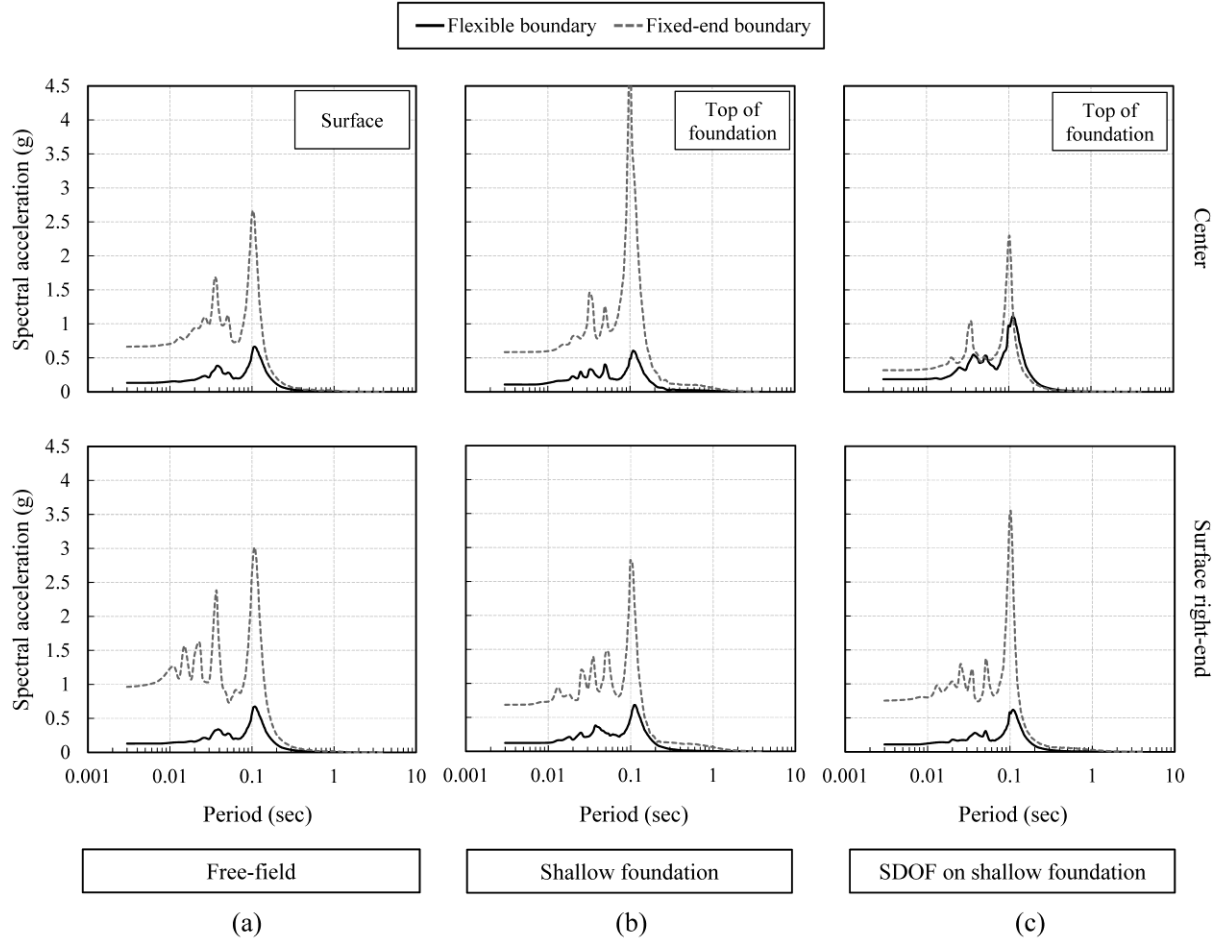
**Fig. 7** Comparison of acceleration responses at surface center and surface right-end for free-field sand layers in saturated and dry conditions; (a) entire shaking duration (saturated), (b) entire shaking duration (dry), (c) shaking from 0.5 to 2.5s (saturated), (d) shaking from 0.5 to 2.5s (dry), (e) spectral acceleration (saturated), (f) spectral acceleration (dry)



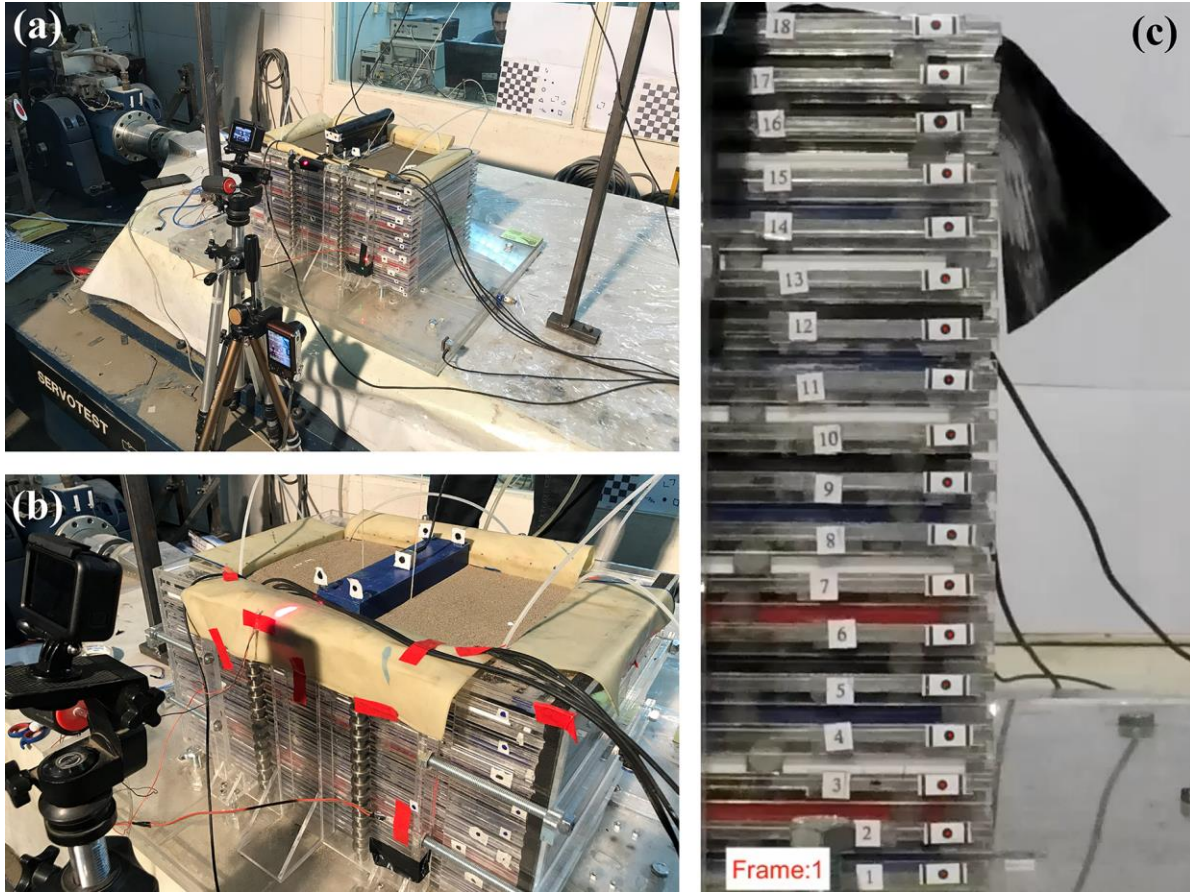
**Fig. 8** Recorded time history of EPWP at mid-height and model base and settlement at the surface of a saturated sand layer for both flexible and fixed-end boundary conditions; (a) free-field, (b) shallow foundation, (c) SDOF on shallow foundation



**Fig. 9** Recorded acceleration time history at different locations for both flexible and fixed-end boundary condition; (a) free-field, (b) shallow foundation, (c) SDOF on shallow foundation

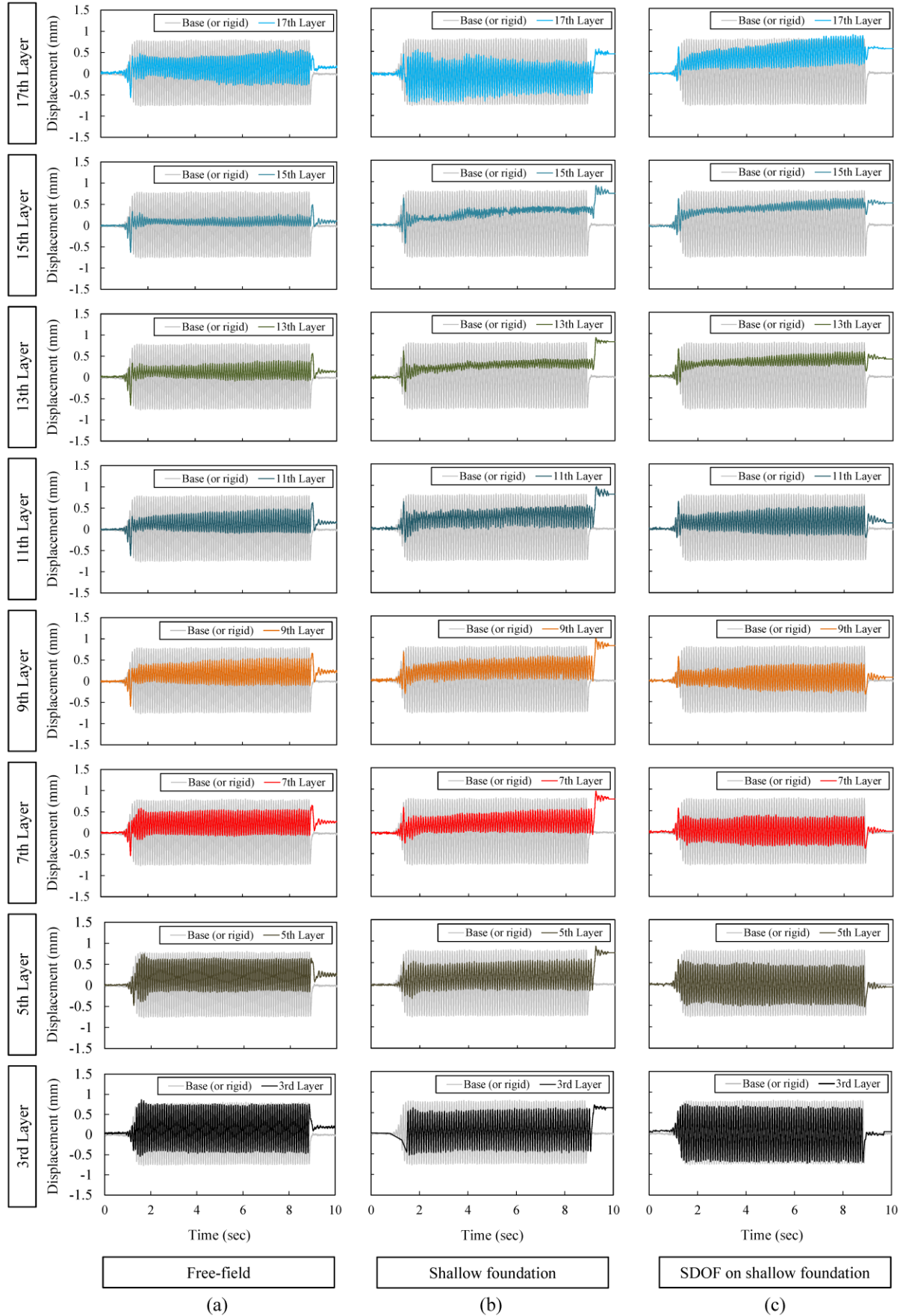


**Fig. 10** Spectral acceleration (5% damping) at different locations for both flexible and fixed-end boundary conditions; (a) free-field, (b) shallow foundation, (c) SDOF on shallow foundation



**Fig. 11** Image processing technique used for capturing displacements: (a) SDOF on shallow foundation; (b) shallow foundation; (c) container layers

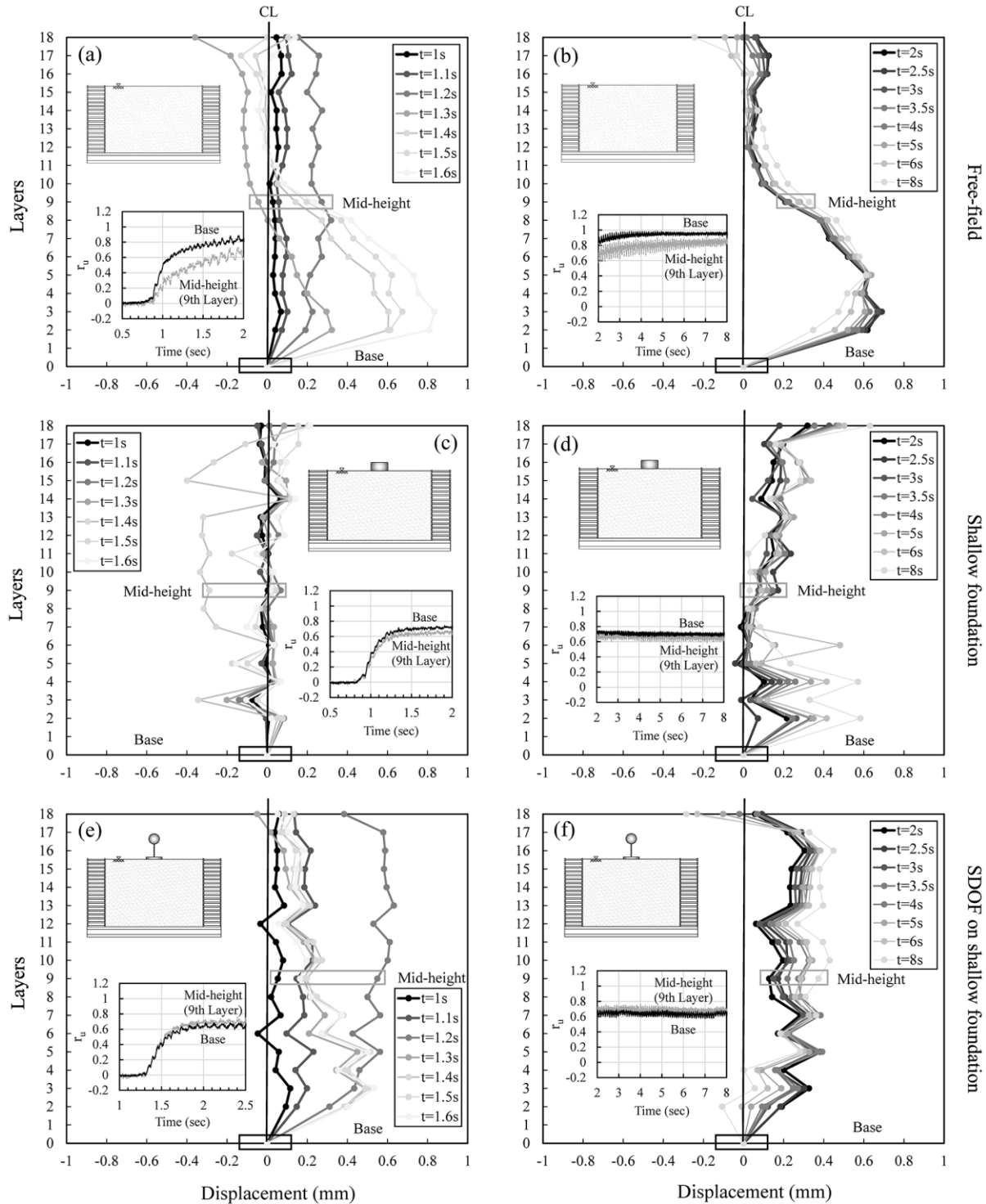




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**Fig. 12** Horizontal displacement of selected container layers obtained from image processing during shaking: (a) free-field; (b) shallow foundation; (c) SDOF on shallow foundation.

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**Fig. 13** Horizontal displacement of container layers obtained from image processing during shaking regarding pore water pressure generation ratio ( $r_u$ ): (a) free-field from  $t=1$ s to  $t=1.6$ s; (b) free-field from  $t=2$ s to  $t=8$ s; (c) shallow foundation from  $t=1$ s to  $t=1.6$ s; (d) shallow foundation from  $t=2$ s to  $t=8$ s; (e) SDOF on shallow foundation from  $t=1$ s to  $t=1.6$ s; (f) SDOF on shallow foundation from  $t=2$ s to  $t=8$ s