

Influence of foundation type on seismic response of low-rise structures in liquefiable soil

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Highlights:

1. Structure-soil-structure interaction (SSSI) is investigated in liquefiable soil.
2. Empirical method for estimating peak surface motion in partially liquefied soil.
3. Raft foundations lower structural demands compared to strips during SSSI.
4. Reduced demand associated with increased post-earthquake foundation deformations.
5. Foundation effects are apparent in mainshock and aftershocks.

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Abstract

The 2010-2011 Canterbury Earthquake Sequence (CES) caused extensive damage to low-rise structures in the city of Christchurch, New Zealand, mainly due to liquefaction-induced effects including settlement and angular distortion. This paper will present the results of dynamic centrifuge tests comparing the effects of liquefaction on the seismic performance of isolated structures with different types of shallow foundations (strips or a raft), and the effect of being situated adjacent to a heavier neighbouring structure of the same foundation type (i.e. considering structure-soil-structure interaction, SSSI). Performance will be evaluated under a sequence of successive earthquakes from the 2010-2011 CES and 2011 Tohoku Earthquake, Japan, to permit study under ground motions and aftershocks generating full liquefaction either extensively or to only a limited depth below ground level. The results show firstly that lower intensity ground shaking occurs at the ground surface when liquefaction occurs and that this can be estimated as a function of the degree of liquefaction using a simple estimation method. When subjected to these ground motions, using strip foundations for isolated structures can result in a reduction in structural demand, especially when the soil is extensively liquefied. When a neighbouring structure with the same foundation type is present, the effects of SSSI within liquefied soil result in changes to natural period and damping such that raft-founded structures exhibited lower

24 structural demands. In either case (isolated or adjacent), a reduction in structural demand is
25 accompanied by an increase in post-earthquake permanent foundation deformation.

26 1 Introduction

27 Due to increasing urbanisation and population growth in recent decades, the interaction
28 between adjacent structures in urban area during earthquakes is becoming of greater concern than their
29 behaviour when they are isolated. Clear evidence of SSSI was first observed in the 1987 Whittier-
30 Narrows (California) Earthquake from field observations of the response of two adjacent seven-storey
31 buildings on moderately dense to dense granular soil [1]. Closely spaced structures may also be founded
32 on soils which are liquefiable. The reduction in soil strength and degradation in shear stiffness occurring
33 due to excess pore water pressure (EPWP) build-up is often a major cause of excessive settlement and
34 angular distortion for buildings on shallow foundations during earthquakes in urban areas, with
35 numerous examples having been observed in the 2010-2011 Canterbury Earthquake Sequence in New
36 Zealand (e.g. [2]).

37 Previous early numerical studies of SSSI (e.g. [3,4]) investigated multiple rigid block
38 interaction on a linear elastic subgrade representative of the stiffness of soil to model a highly idealised
39 urban area and studied the dynamic characteristics, (particularly associated with natural periods of
40 vibration) associated with the groups of structures. Further studies in [5] used 1-g shaking table tests of
41 more representative equivalent single-degree-of-freedom oscillators on a linear elastic subgrade,
42 considering two adjacent buildings. This work has suggested that the relative dynamic properties
43 (natural periods) of the individual structures could result in either an increase or decrease of the peak
44 acceleration and spectral power by large amount. This data was used to validate an analytical model [6]
45 that identifies not only vibrational dynamic characteristics of grouped structures, but also, the resulting
46 (elastic) structural response. To improve upon previous linear-elastic idealisations of soil, centrifuge
47 testing has also previously been conducted to investigate the behaviour of single and adjacent (paired)
48 structures on non-liquefiable (but non-linear) soil [7] which showed structural response effects
49 consistent with [5], and also introduced the effects of SSSI on foundation response in terms of

50 permanent post-earthquake settlement and rotation (tilt caused by differential settlement). Although
51 these studies have provided very useful insights into SSSI none have previously considered how the
52 interaction may change in the presence of soil liquefaction, despite extensive study of individual
53 shallow foundation behaviour on liquefiable soil (e.g. [8,9]).

54 This study aims to build on this previous work by considering: (i) the effect of liquefaction on
55 the soil-structure interaction behaviour of an isolated multi-degree-of-freedom structure having one of
56 two types of shallow foundations (individual strips or a one-piece raft) within liquefiable granular soil;
57 and (ii) how this behaviour is modified by SSSI due to the presence of a nearby (dissimilar) structure.
58 The results from four multi-event centrifuge tests are presented in this study. In the tests, a series of
59 consecutive earthquake ground motions measured at a single site from the 2010-2011 Canterbury
60 Earthquake Sequence was considered to observe performance under a strong earthquake and weaker
61 aftershock motions, followed by a long duration high intensity ‘double pulse’ motion from the 2011
62 Tohoku Earthquake which could potentially apply large accelerations (and therefore large inertial forces)
63 into the structure(s) in the second pulse of high acceleration after liquefaction had been triggered by the
64 first.

65 2 Centrifuge modelling

66 The centrifuge tests presented herein were conducted using models at 1:40 scale, tested at 40-
67 g centrifugal acceleration using the Actidyn Systèmes C67 3.5 m radius beam centrifuge facility at the
68 University of Dundee, UK. All parameters herein are presented at prototype scale unless otherwise
69 stated. Scaling laws used to determine model parameters from prototype values for centrifuge modelling
70 can be found in [10, 11].

71 2.1 Model structures

72 In the 2011 Christchurch Earthquake, 80% of the heavily damaged buildings in the Central
73 Business District (CBD) were one or two-storey buildings founded on shallow foundations [12]. This
74 type of buildings is the most common in urban areas, while being the least likely to have extensive
75 seismic detailing (compared to high value, high-rise structures in a CBD). The design of prototype

76 structures was not to replicate a specific actual building but to retain key characteristics of low-rise
77 buildings which were two-storey, single bay, moment resisting frames with concrete slab floors sitting
78 on either a square raft or separated strip concrete foundations. The storey height (3 m) and floor area
79 3.6 m × 3.6 m were representative of low-rise buildings, accounting also for the space constraints in the
80 centrifuge. The model frames consisted of four individual square columns machined from solid 6082-
81 series aluminium alloy rods interconnected by two floor slabs fabricated from aluminium plates.

82 In the case of adjacent structures, an increase in slab mass by 44% was made to one of the
83 structures (which otherwise had structural frame elements of the same stiffness) resulting in a 20%
84 lengthening of natural period T_n . This arrangement of dissimilar structures was selected as this
85 difference in natural period between adjacent buildings was observed to produce the greatest influence
86 of SSSI for linear elastic ground behaviour by [5]. It may also be thought of as representative of a case
87 where one structure from a pair of initially identical structures has had a change of use (increasing the
88 slab loading). Additional thin steel plates were bolted to the model slabs to achieve the mass difference
89 between the two structures in the adjacent cases. The foundation edge-to-edge spacing was 1.2 m at
90 prototype scale which was 1/3 of the structural bay width and 1/4 of total building width including the
91 foundations, to enable strong SSSI effects [13-15] while avoiding any building pounding and instrument
92 damage during experiments. A summary of test configurations is provided in Table 1.

93 The raft foundation was made of a single aluminium alloy plate due to the similarity in density
94 between this material and reinforced concrete (2700 kg/m³ versus 2400 kg/m³, respectively), which was
95 4.8 m × 4.8 m square in area with a high static factor of safety (*FOS*) against bearing failure on the fully
96 saturated medium dense sandy soil used (see later) due to the large area. The strip foundations were
97 made of the same material but separated on two sides of the structure (i.e. each supporting two columns)
98 being $B = 1.2$ m in width and $L = 4.8$ m in length ($B/L = 4$), providing a static *FOS* of 3 or 2.5 for the
99 'light' and 'heavy' building cases. The raft foundation had the same external footprint, but with solid
100 material infilling between the strip foundations. Both types of foundations satisfied static requirements
101 at the ultimate limiting state. The bearing capacities of the strip and raft foundations are shown in Table
102 2 following design method presented in [16]. The presence of an adjacent structure on strip foundations

103 given the foundation edge-to edge spacing of 1.2 m was not expected to affect the bearing capacity at
 104 ultimate limit state compared to a building in isolation [16, 17], however, it would have an effect on
 105 initial settlements and tilt of the structures under static conditions (both for strip and raft foundations).
 106 An adjacent building with a raft foundation was expected to increase the bearing capacity of its
 107 neighbour by a factor of 1.5 [16, 17]. The structure on the raft foundation in isolation was expected to
 108 resist a maximum seismic action $a_g = 0.44g$ in the absence of liquefaction based on conventional seismic
 109 bearing capacity approaches [18]; for the structure on strip foundations this value was 0.23g. All
 110 foundations were coated on the base and sides with a thin layer of the subsoil using an epoxy resin to
 111 approximate the rough soil-footing interface between soil and concrete cast in-situ. Figure 1 shows the
 112 instrumented model structure on strip foundations with dimensions at prototype and model scales.

113 A typical fundamental natural period (T_n) of a prototype two-storey structure was approximated
 114 using Equation 1 [19]:

$$115 \quad T_n = 0.1N \quad (1)$$

116 where N is the number of stories of the structure ($N = 2$ in this case) and T_n is in seconds.

117 The mass of each floor slab was set to be the same ($M_1 = M_2$) and determined based on a 3.6 m
 118 \times 3.6 m \times 0.5 m thick reinforced concrete slab. The equivalent single-degree-of-freedom stiffness of
 119 the structure in the fundamental mode (K_{eq}) was then determined from Equation 2:

$$120 \quad T_n = 2\pi \sqrt{\frac{M_{eq}}{K_{eq}}} \quad (2)$$

121 where:

$$122 \quad M_{eq} = M_1 \bar{y}_1^2 + M_2 \bar{y}_2^2 \quad (3)$$

$$123 \quad K_{eq} = K_1 (\bar{y}_1)^2 + K_2 (\bar{y}_2 - \bar{y}_1)^2 \quad (4)$$

124 The normalized modal coordinates associated with the fundamental mode were $\bar{y}_1 = 0.45$ and
 125 $\bar{y}_2 = 0.89$, based on an eigenvalue analysis for the two-storey structure having equal lateral stiffness
 126 and mass at each storey. By setting the four columns in each storey to have the same lateral stiffness,

127 k_{col} (i.e. $k_{col} = 0.25K_1 = 0.25K_2$), and selecting the closest available steel Universal Column size to
128 provide sufficient bending stiffness EI (UC 203×203×86), the natural period of the light structure was
129 finally 0.21 s and that of the heavier structure 0.25 s. A summary of section properties is provided in
130 Table 2.

131 2.2 Model preparation and soil properties

132 For all tests presented herein, a single set of medium dense soil properties was used. Dry HST95
133 Congleton silica sand at a relative density of $I_D = 55\%-60\%$ and 8 m depth at prototype scale was
134 initially air-pluviated into an Equivalent Shear Beam (ESB) container, then saturated using
135 hydroxypropyl methyl-cellulose (HPMC) pore fluid with a viscosity 40 times higher than water. This
136 was required to ensure the time scales for seepage and inertial effects were consistent with prototype
137 values, which is further discussed in [20]. Details of the performance of viscous pore fluids in dynamic
138 centrifuge testing can be found in [21]. Saturation was conducted by allowing the fluid to enter the
139 model under a constant head through orifices in the bottom of the ESB container at a relatively low
140 flow rate until it reached a level 2 mm above the model surface (to ensure that the soil would remain
141 fully saturated even if there was ground heave adjacent to the foundations).

142 The ESB container consisted of stacked aluminium rings separated by thin rubber layers, with
143 the aim of providing flexible boundaries that deform similarly to the fundamental mode of the soil to
144 minimise boundary effects. The designed natural frequency of the container used was 2 Hz at prototype
145 scale for a horizontal acceleration coefficient $k_h = 0.4$ at 50-g [22, 23]. Detailed discussion of all of the
146 design and performance requirements for such a container can be found in [24-26]. Physical properties
147 of the HST95 sand are listed in Table 3 after [27]. In the absence of liquefaction, the ground profile so
148 modelled represented ground type E according to Eurocode 8 [28].

149 The soil was instrumented with accelerometers and pore pressure transducers (PPT) at five
150 different depths in isolated tests and 3 depths in adjacent tests. Figure 2 shows the instrumentation
151 details of Tests SQ04 and SQ07 as examples (instrument positions are denoted by letters for SQ04).
152 ADXL-78 single-axis micro-electromechanical system (MEMS) accelerometers were used to measure

153 ground motions and infer stress-strain behaviour and both HM-91 PPTs and PDCR-81 PPTs were used
154 to measure the generation and dissipation of EPWP. Soil measurements were made close to the input
155 ‘bedrock’ (point E), at a vertical array in the free-field (Points A-E), and below each building at similar
156 depths to free-field points (Points F-G). The structures were also instrumented with the same type of
157 accelerometers (see Figures 1 and 2) to measure the vertical and horizontal dynamic motions at the
158 foundations and horizontal motions at each storey (Points H to K). The storey acceleration was derived
159 from a high pass zero-phase-shift filtering of horizontal accelerometers attached on the structures
160 (Points J and K in Figure 2), to remove any monotonic component due to permanent deformation. The
161 cyclic sway (= inter-storey drift + lateral displacement due to rotation) was derived by double
162 integration of the storey acceleration data. The dynamic inter-storey drift was then determined by
163 removing the cyclic rotational component measured from the vertical foundation accelerometers.
164 Horizontal Linear variable Differential Transformers (LVDTs) were avoided for deriving cyclic-sway
165 and inter-storey drift as the individual floors were 6 mm thick at model scale and there was initial
166 settlement during spin-up of the centrifuge which may have resulted in the horizontal LVDTs losing
167 contact with the floors. As the response of the structures was elastic, the accelerometer approach
168 outlined above was adopted instead.

169 After loading the saturated soil model onto the centrifuge, the isolated or adjacent structures
170 were placed on the soil surface to be nominally level, following which any initial tilt was recorded using
171 a clinometer to provide a baseline for subsequent measurements of structural rotation. An overhead
172 gantry was then placed above the structures on either side allowing the placement of linear variable
173 differential LVDTs to measure permanent settlement and rotation of the structures, and settlement of
174 the soil surface above the free-field array. Due to the small vertical cyclic displacements of the
175 foundations, the gross settlements and rotations of the foundations were derived from superposing low-
176 pass zero-phase shift eighth-order Butterworth filtered LVDT data (cut-off frequency 0.75 Hz in
177 prototype) to provide the monotonic component and high-pass zero-phase shift eighth-order Butterworth
178 filtered (cut-off frequency of 1.5 Hz in prototype) double integrated vertical accelerometer data to
179 provide the dynamic component. The gross settlement was derived by averaging the compound LVDT

180 data for the two instruments on each side of an individual structure. Rotation was derived by the
181 difference of the two compound LVDT traces divided by the width of structure, with rotation to the
182 right as shown in Figure 2 being positive.

183 2.3 Ground motions

184 Following spin-up to 40-g, a re-ordered sequence of motions from the Canterbury Series of
185 2010-2011 (Christchurch, New Zealand) recorded at the Christchurch Botanical Gardens Station was
186 applied, followed by a long duration ‘double-pulse’ motion from the 2011 Tohoku Earthquake (Japan)
187 recorded at the Ishinomaki Station. The Christchurch earthquake of February 2011 was chosen to be
188 the first motion, aiming to induce full liquefaction with the initial condition of the soil fully known (no
189 pre-shaking). Three subsequent less intense aftershocks (‘June13a’ from June 2011, the Darfield
190 earthquake of 2010 and ‘June13b’, also from 2011) were subsequently applied which were expected
191 to produce progressively lower EPWP generation, representing various partially-liquefied conditions.
192 The Tohoku motion was applied last with the aim of fully re-liquefying the soil following the previous
193 sequence of motions during the initial pulse of high peak ground acceleration (PGA) followed by a
194 second high PGA pulse while the soil remained in a liquefied state to simulate a potentially extreme
195 load case for the structure(s). Each motion was applied recording the response of all instruments at 4
196 kHz sampling frequency for 4 minutes at model scale (160 minutes at prototype scale) following the
197 end of shaking to ensure that the EPWP observed from the PPTs returned to zero and the building was
198 stationary before applying the subsequent motion.

199 The motions were applied using the Actidyn QS67-2 servo-hydraulic earthquake simulator
200 (EQS) at the University of Dundee. Details of its performance may be found in [23,29]. Motions were
201 filtered using an eighth order Butterworth filter with a pass range between 2.3-7.5 Hz (at prototype
202 scale). The nominal 5% damped response spectra of the recorded input motions are shown in Figure 3.
203 The fixed-base natural period of the light structure $T_n = 0.21\text{s}$ is also shown which falls within the
204 rising phase of the spectra. It was observed that the first and last motion have similar spectral response
205 at a fixed-base period of 0.21s indicating that these two motions were expected to result in similar peak
206 structural response in the absence of any soil-structure interaction (SSI), while being of very different

207 duration. The effects of SSI on period lengthening will be discussed from measured data later. The
 208 repeatability of input motions across the four centrifuge tests (essential for valid test-to-test comparisons)
 209 is demonstrated in Figure 4 in terms of normalised spectra, in which the Type 1 elastic design response
 210 spectrum (behaviour factor $q=1$ and Ground type A for consistency with the input motions being at
 211 ‘bedrock’ level) from Eurocode 8 for earthquakes with surface-wave magnitude greater than 5.5 is also
 212 shown for context [28].

213 3 Results

214 3.1 Free field soil response

215 Data showing peak EPWP generation in the free field are presented in Figure 5 in terms of the
 216 normalised ratio r_u (equal to EPWP divided by initial vertical effective stress). Actual pre-shaking
 217 depths of the PPTs were determined based on the static pore pressures measured during spin-up of the
 218 centrifuge and these were used in place of the nominal values shown in Figure 2 for EQ1. Thereafter,
 219 PPT positions were corrected for any floating or sinking between earthquake events based on any final
 220 (small) static offsets in measurements after EPWP had fully dissipated (i.e. $dr_v/dt = 0$).

221 Liquefaction susceptibility/trigging analyses were also conducted following the approach of
 222 [30]. The factor of safety against liquefaction (F_{SL}), determined using Equation 5, is shown in Figure
 223 5(a) (as cross markers connected with dashes lines)

$$224 \quad F_{SL} = \frac{CRR}{CSR} \quad (5)$$

225 where CRR = Cyclic Resistance Ratio and CSR = Cyclic Stress Ratio. The CRR value was determined
 226 for EQ1 using an estimated equivalent normalised SPT blowcount $(N_1)_{60}$ of 22 determined using
 227 Equation 6, which is a reasonable estimation for most aged natural deposits in terms of I_D [16,31].

$$228 \quad (N_1)_{60}/I_D^2 \approx 60 \quad (6)$$

229 where $I_D = 60\%$. The CSR value was determined based on the ground motion according to [30] and
 230 input bedrock PGA in the free-field assuming linear change with height to a value at the surface giving

231 an amplification factor of 1.4 (soil factor for ground type E in EC8) in the absence of any liquefaction
232 effect.

233 Adopting methods described in [32], the peak pore pressure ratio with depth was estimated.
234 This assumes that when $F_{sL} \leq 1$, full liquefaction existed and the peak EPWP was equal to the initial
235 effective vertical effective stress at that depth ($r_{u,predicted}=1$). Once $F_{sL} > 1$, the peak EPWP was assumed
236 to be thereafter constant with depth, providing a bi-linear approximation to an EPWP isochrone for the
237 time when maximum EPWP is reached. Dividing this constant EPWP by the increasing vertical
238 effective stress with depth gave a reducing profile of r_u with depth shown by the solid lines in Figure
239 5(a).

240 Figure 5(b) shows the measured peak $r_{u,FF}$ at each depth from all tests and events. Variations in
241 $r_{u,FF}$ between tests was thought to be caused by (i) individual local variations of density between soil
242 models from model preparation, which are unavoidable; and/or (ii) the influence of the nearby structures.
243 Although the position of the free-field instrumentation was kept at least 100 mm from the wall of the
244 ESB container, 100 mm (3.3B strip 0.8B raft) from the model structures in the adjacent cases, and 180
245 mm (6.0B strip 1.5B raft) in the isolated cases, the free-field EPWP of isolated raft and all adjacent
246 cases were all lower than in the isolated strip case, where the foundation-free-field spacing was largest
247 (6B). The free-field instruments in the isolated strip case therefore represented the best approximation
248 to true free-field conditions.

249 In EQ1 (Christchurch Earthquake) under virgin initial soil conditions the soil experienced full
250 liquefaction over all depths, consistent with $F_{sL} < 1$ everywhere. In EQ2 and EQ5 (Tohoku Earthquake)
251 extensive liquefaction was also achieved as suggested by $F_{sL} < 1$ everywhere, even with the initial
252 conditions of the soil having been altered by the previous shaking (e.g. resulting in densification,
253 particularly near-surface). In the weakest of these three motions (EQ2) full liquefaction was only
254 achieved within the upper half of the soil layer (which would likely be deep enough to lead to similar
255 structural response as in EQ1 and EQ5, given that the foundations are shallow). Figure 6 demonstrates
256 that the effects of full liquefaction to full/half depth resulted in substantial reductions in motion from
257 the bedrock input motion (isolated strip case shown), as the shear waves were not able to amplify as they

258 propagated due to the low shear strength of the soil in the fully liquefied state which limited transfer of
259 shear stress.

260 The smaller aftershocks (EQ3 and EQ4) had significantly lower PGA ($< 0.2\text{-g}$) compared to
261 the other motions (PGA $> 0.3\text{-g}$) and these also grouped together with similar behaviour, exhibiting full
262 liquefaction only at the very shallowest locations in Figure 5 (b). The estimated r_u profiles from Figure
263 5(a) can also be seen to provide a reasonable upper-bound to the measured data (i.e. conservative for
264 use in design). From Figure 6 this ‘surficial liquefaction’ resulted in approximately no reduction in
265 motion amplitude at the ground surface (i.e. reduction in ground motion attenuation due to only partial
266 liquefaction) in both cases, consistent with the similarity in r_u profiles. In these cases, the deeper soils
267 allowed motions to partially amplify as the shear waves travelled upwards, before being attenuated by
268 the low strength liquefied surface layers. As shown in Figure 7, the attenuation in PGA of EQ3 and
269 EQ4 was shallower (3.5 m below surface, consistent with the depth of full liquefaction from Figure 5).

270 Figure 8 shows the transfer function required to convert the spectra of the input motion at
271 bedrock to the corresponding free-field surface values in Figure 8 (i.e. the soil amplification factor – S
272 in EC8; BSI, 2005). The limiting values at low ($T < 0.4\text{s}$) and high ($T > 0.8\text{s}$) periods are further plotted
273 in Figure 9 together with $S = 1.4$ (for ground type E) for the case of no liquefaction ($r_u = 0$, everywhere).
274 The values are limited to only isolated cases as the free field was less affected by the presence of the
275 structures in these cases and thus better represents the true free-field condition. This is plotted against
276 a parameter representing the area beneath the r_u -depth curve, normalised by layer depth, which is a
277 measure of the cumulative amount of liquefaction within the soil. A value of 1.0 indicates that all of the
278 soil is fully liquefied (i.e. at all depths). With the exception of one datapoint, a negative trend can be
279 observed from the experimental data which is consistent with the amplification factor being 1.4 when
280 there is no liquefaction. Two trendlines can be drawn by least-squares regression for structures of period
281 $T < 0.4\text{ s}$ and $T > 0.8\text{ s}$ implying the potential reductions for a wide period range of structures and
282 interpretation can be made between the two trendlines.

283 3.2 Response of isolated structures with different foundation types

284 3.2.1 Response in fully-liquefied soil

285 This section will focus on structural demand for storey 1 only, as this storey exhibited the largest
286 deformation (inter-storey drift) and was also close to the centre of mass of the two-storey structure. The
287 time histories of structural response for strip foundations (black line) and raft foundation (grey line)
288 during EQ1 (full liquefaction, virgin soil) and EQ5 (full liquefaction, pre-shaken soil) are shown in
289 Figures 10 and 11, respectively. It is shown that strip foundations minimised transmission of
290 accelerations to the structure in each (isolated) case (Figures 10(a) and 11(a)).

291 The free field settlement recorded from the LVDT at the free-field surface in the isolated strip
292 case is shown for reference in Figure 10 (b) and Figure 11(b). This test had the most representative free-
293 field condition at the location of the transducer and only a single case is shown for clarity. The initial
294 settlement in spinning-up the centrifuge from 1g to 40g is shown as the starting settlement prior to EQ1.
295 The initial structural tilt caused by the spin-up is also shown as the starting value of EQ1 in Figure 10
296 (c). Comparing the structural and free field settlements in the isolated strip case, the initial settlement
297 during spin-up of the structure was much larger than that in the free-field due to the applied foundation
298 bearing pressure.

299 In Figure 10 (b) and (c) for EQ1, the building settlement and rotation of the isolated raft and
300 strip were largely similar. By EQ 5 (Figure 11 (b) and (c)) the post-earthquake rotation of the structure
301 with strip foundations had increased significantly, together with greater accumulated settlement prior
302 to EQ5 and larger increases in settlement during EQ5. This demonstrates that any benefit of strip
303 foundations in protecting the structure by minimising transmitted accelerations in fully-liquefied soil
304 comes at a price of greater post-earthquake foundation deformation. This is similar to the trade-off
305 between settlement and structural protection in rocking-isolated structures [e.g. 33].

306 3.2.2 ‘Double pulse’ excitation behaviour

307 The Tohoku Earthquake (EQ5) is shown divided into two regions in Figure 11, separated by a
308 dashed line. In the first part, the largest ground accelerations occurred when the soil was still liquefying,

309 while in the second part, similarly large input ground accelerations occurred when the soil was already
310 fully liquefied (Figure 11(d)). The cycles of EPWP generation of the r_u time history in Figure 10 (d)
311 and Figure 11 (d) were filtered out due to an unexpected band frequency of noise that was superimposed
312 on the signal for these instruments within the frequency range of the earthquakes. As a result, the values
313 shown indicate the monotonic component of the EPWP. The effect of the different r_u values at these
314 two different instances were reflected in the size of the storey accelerations, which were larger during
315 the first part when $r_u < 1$ and smaller in the second part when $r_u = 1$. The maximum ratios of storey
316 acceleration in the strip foundation compared to the raft foundation in EQ5 were 0.7 for $r_u < 1$ and 0.52
317 for $r_u = 1$ (Figure 11 (a)). Such large reductions of structural response due to SSI may outweigh the
318 negative effects of additional settlement (+25%), making separated strip foundations desirable over
319 rafts in liquefiable soil for isolated structures. In the second part of the motion, there was also greater
320 earthquake-induced permanent rotation of raft foundations compared to strips, but post-earthquake
321 rotations are known to heavily depend on initial conditions [7], so that such a result is not general, but
322 is dependent on the seismic history and any historical foundation deformations at a particular site.

323 3.3 Effect of adding a heavier neighbouring structure of the same foundation type

324 This section continues to focus on the behaviour of the lighter structure of the pair tested, but
325 now incorporating the SSSI from the adjacent heavier structure. Peak storey acceleration, cyclic sway
326 and inter-storey drift in each EQ are shown in Figure 12, in which the 1:1 dividing line indicates parity.
327 The maximum inter-storey drift here was around 6.5 mm (0.2% of storey height), which was under the
328 ‘no damage’ limit of 0.4% for buildings having brittle non-structural elements in EC8 indicating that
329 all of the structures performed elastically during the tests, and the use of an elastic structural physical
330 model was justified.

331 Considering the isolated structures first, Figure 12 demonstrates that in terms of inter-storey
332 drift (the part which induces bending within the columns) rafts and strips gave very similar response
333 for all earthquakes, even though the severity of liquefaction was different. The sway was lower in the
334 strip cases however, implying that the reduced storey accelerations in these cases were associated with
335 less cyclic rocking in the structures (see Figure 13(a)). This may initially appear counter-intuitive as the

336 rafts would be expected to have had a higher rotational foundation stiffness than the strips; however,
337 due to the higher bearing pressures acting on the strips (lower *FOS*, Table 2) uplift was easier in the raft
338 case than the strip case.

339 While strips and rafts saw similar structural response in terms of column deformation in the
340 isolated structure case for all earthquakes, SSSI resulted in a significant reduction in structural response
341 in the raft cases and a slight increase in response for the strip cases for all measures of structural response,
342 including inter-storey drift. To investigate this further, transfer functions for the structure of interest
343 (using the accelerometer data between the foundation and storey 1) were determined during each EQ
344 for all cases. A single-degree-of-freedom response curve for magnitude of response was used to
345 determine the best-fit fundamental natural period T_n and equivalent viscous damping ratio ξ (results
346 shown in Figure 14):

$$347 \quad \left| \frac{\ddot{x}}{\ddot{y}} \right| = \sqrt{\frac{1 + (2\xi T_n/T)^2}{(1 - (T_n/T)^2)^2 + (2\xi T_n/T)^2}} \quad (7)$$

348 where x is the absolute displacement of the first storey; y is the foundation input displacement; T is the
349 base excitation period; and T_n is effective natural period.

350 The dashed line shown in Figure 14 represents the designed fixed-base natural period of the
351 lighter structure (0.21s). The difference between the isolated data points and this line therefore indicates
352 the effect of SSI in liquefied soil in lengthening the fundamental natural period. The isolated structure
353 with strip foundations exhibited generally higher effective periods than the isolated rafts since the strip
354 foundations had lower stiffness resulting in greater lengthening. The lengthened effective natural period
355 of the structures derived from the transfer functions of all four tests fell within the area of the response
356 spectra where acceleration reduces with period (Figure 4), which explains why the isolated raft cases
357 saw greater peak acceleration (Figure 12(a)). Comparing the isolated and adjacent cases in Figure 14,
358 the structure exhibited a general reduction in period due to SSSI in strip cases (Figure 14(a)) which
359 resulted in increased amplification of storey acceleration. In contrast, for the raft foundations shown in
360 Figure 14(b), the structure had a generally increased effective period resulting in a reduction of
361 structural response. These identified changes in effective period explain the differences between rafts

362 and strips in terms of structural performance in Figure 12(a). Equivalent viscous damping results are
363 shown in Figure 14(c) and (d). In all but one case, the damping was substantially reduced by SSSI. This
364 would suggest that all measures of response should have reduced for both raft and strip cases in Figure
365 12. Figure 15 explains this combined effect graphically by applying period and damping change on the
366 EQ2 spectra accounting for surface liquefaction (i.e. at Point A) as an example. The structural response
367 of an isolated structure on a raft foundation was generally reduced by SSSI because the reduction caused
368 by the period lengthening effect outweighed the increase caused by lower damping. In the case of a
369 structure on strip foundations there were combined detrimental effects of both damping reduction and
370 period shortening resulting in an increase in structural response (at least within this descending branch
371 of the spectrum).

372 The overall (permanent) earthquake-induced post-earthquake tilt (rotation) of all structures is
373 shown in Figure 16. The structure on the raft foundation in the adjacent case was seen to lose its
374 beneficial effects relative to the isolated case due to SSSI but this was no worse than the values observed
375 for either isolated or adjacent strip foundation cases (Figure 16(a)). This greater tilt was consistent with
376 the reduced structural response in raft cases, with greater energy dissipation having occurred in plastic
377 soil deformation protecting the structure. The effect of SSSI on permanent rotations for the strip
378 foundations case was small/negligible, except for EQ5.

379 The accumulated post EQ settlement is shown in Figure 17 with the free-field settlement
380 derived from the isolated strip case as a reference. The final building settlements in adjacent cases were
381 greater than those in isolated cases, although adjacent structures also had larger initial settlements due
382 to static SSSI. In terms of the earthquake-induced settlement (shown between the dashed lines in Figure
383 17) the shallow foundations showed smaller co-seismic settlements in the adjacent cases due to SSSI
384 which is consistent with previous observations in non-liquefied soil [7]. In contrast, the raft foundations
385 showed much greater co-seismic settlement in the adjacent case, consistent with greater plastic
386 deformation within the foundation soil. However, as gross settlement is not as damaging as differential
387 movement, and the induced rotation of the raft in the adjacent case was similar to that in the strip case
388 (Figure 16), the protective effect of rafts in reducing structural demand in the adjacent case may

389 outweigh the increased foundation movement. These results suggest that raft foundations are more
390 desirable when used in urban areas in terms of reducing structural demands, though at a cost of greater
391 post-earthquake foundation deformation.

392 4 Conclusion

393 This paper has investigated the effects of liquefaction on isolated low-rise structures on strip
394 and raft foundations, and the influence of SSSI on this behaviour when a heavier neighbouring structure
395 of the same foundation type is present, in terms of soil response, structural response and foundation
396 response. It was shown that the soil factor in EC8 describing ground motion amplification (site effect)
397 could reduce significantly due to partial or full liquefaction. The depth of full liquefaction could be
398 estimated using a simple method based on the result of a standard liquefaction susceptibility analysis.
399 It was also possible to estimate an upper bound on the peak r_u -profile with depth using this method,
400 from which the soil factor could be estimated. This finding however requires further research to
401 generalise the result for more soil types/densities and building types. The results of this study suggest
402 that the selection of suitable foundation type can significantly influence the structural and foundation
403 response of buildings. In terms of SSI on liquefiable soil, using strip foundations resulted in lower
404 structural response (isolated structure) although there was a trade-off in terms of increased post-
405 earthquake foundation deformation. When SSSI occurs (i.e. in an urban area where there are closely-
406 spaced adjacent structures) raft foundations resulted in reduced structural demand caused by the
407 combined effects of SSSI-induced period lengthening (in a descending branch of the spectrum) which
408 outweighed the effects of any SSSI-induced reduction in damping. There was again a price to pay in
409 terms of increased post-earthquake foundation deformation. It is suggested that new urban areas might
410 target raft foundations as a way of reducing structural demand. The results also suggest that if adjacent
411 buildings are to be added next to existing structures, it may be beneficial for them to have raft
412 foundations, though further research would be desirable to consider the interaction between adjacent
413 dissimilar foundation types.

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416 Callon at the University of Dundee for their assistance in making the models and running the centrifuge
417 tests. The authors would also like to thank China Scholarship Council for its financial support of the
418 PhD studies of the first author.

419 6 Nomenclature

420 B =width of foundation in prototype

421 CRR = cyclic resistance ratio

422 CSR = cyclic stress ratio

423 EI =bending stiffness

424 $EPWP$ =excess pore water pressure generated in the soil

425 F_{SL} =factor of safety against liquefaction

426 FOS =static factor of safety of structure

427 I_D =relative density

428 K_1, K_2, K_{eq} =total lateral stiffness of the first storey, total lateral stiffness of the second storey,
429 equivalent lateral stiffness of the structure in the fundamental mode, respectively

430 L = length of foundation in prototype

431 M_1, M_2, M_{eq} = total mass of the first storey, total mass of the second storey, equivalent mass of the
432 structure, respectively

433 N = numbers of stories of structure

434 $(N_1)_{60}$ = normalised SPT blowcount

435 PGA = peak ground acceleration

436 $r_u, r_{u,FF}, r_{u,predicted}$ = excess pore water pressure ratio in general, in free field and predicted through
437 simple prediction method, respectively

438 $S_e, S_{e,FF \text{ surface}}, S_{e,input}$ = response spectra in general, in free field surface and input, respectively

439 SSI = Soil-structure interaction

440 SSSI = Structure soil-structure interaction

441 T = base excitation period, effective natural period, in Equation 7

442 T_n = natural period of structure, in Equation 2

443 x = absolute displacement of the first storey

444 y = foundation input displacement

445 \bar{y}_2 , = normalized modal coordinates in the fundamental mode

446 ξ = equivalent viscous damping ratio

447 7 Reference list

448 [1] Celebi M. Seismic responses of two adjacent buildings: Interaction. Journal of Structural
449 Engineering. ASCE 1993; 119 (8): 2477–92.

450 [2] Luque R, Bray J. Dynamic analyses of two buildings founded on liquefiable soils during the
451 Canterbury earthquake sequence. Journal of Geotechnical and Geoenvironmental Engineering ASCE
452 2017;143(9): 04017067

453 [3] Wirgin A, Bard P-Y. Effects of buildings on the duration and amplitude of ground motion in
454 Mexico City. Bulletin of the Seismological Society of America 1996; 86(3):914–920.

455 [4] Tsogka C, Wirgin A. Simulation of seismic response in an idealized city. Soil Dynamics and
456 Earthquake Engineering 2003; 23 (5), 391–402.

457 [5] Aldaikh H., Alexander N.A., Ibraim E, Knappett J. A. Shake table testing of the dynamic
458 interaction between two and three adjacent buildings (SSSI). Soil Dynamics and Earthquake
459 Engineering 2016; 89:219–232.

460 [6] Aldaikh H, Alexander N.A, Ibraim E, Oddbjornsson O. Two dimensional numerical and
461 experimental models for the study of structure–soil–structure interaction involving three buildings.
462 Computers & Structures 2015; 150: 79–91.

463 [7] Knappett JA, Madden P, Caucis K. Seismic structure–soil–structure interaction between pairs of
464 adjacent building structures. Géotechnique 2015; 65(5): 429–441.

- 465 [8] Liu L, Dobry R. Seismic response of shallow foundation on liquefiable sand. *Journal of*
466 *Geotechnical and Geoenvironmental Engineering*. ASCE 1997; 123(6): 557-567.
- 467 [9] Bertalot D, Brennan AJ. Foundations influence of initial stress distribution on liquefaction-
468 induced settlement of shallow foundations. *Géotechnique* 2015; 65(5): 418-428.
- 469 [10] Muir Wood D. *Geotechnical modelling*. London UK. Spon Press. Taylor and Francis. 2004.
- 470 [11] Iai S, Tobita T, Nakahara T. Generalised scaling relations for dynamic centrifuge tests.
471 *Géotechnique* 2005; 55(5) 355-362.
- 472 [12] Storie BL. Soil-foundation-structure interaction in the earthquake performance of multi- storey
473 buildings on shallow foundations. PhD thesis. University of Auckland. 2016.
- 474 [13] Jiang X, Yan Z. Earthquake response analysis of building-foundation-building interaction
475 system. *Journal of Vibration Engineering* 1998;11(1):31–37.
- 476 [14] Yahyai M, Mirtaheri M., Mahoutian M, Daryan AS. Assareh M A. Soil structure interaction
477 between two adjacent buildings under earthquake load. *American Journal of Engineering and Applied*
478 *Sciences* 2008; 1(2): 121-125.
- 479 [15] Alexander NA, Ibraim E, Aldaikh H. A simple discrete model for interaction of adjacent
480 buildings during earthquakes. *Computers and Structures* 2013; 124, 1–10.
- 481 [16] Knappett JA. and Craig RF. *Craig's soil mechanics*. 8th edn. London, UK: Spon Press. 2012.
- 482 [17] Kumar J, Kouzer KM. Bearing capacity of two interfering footings. *International Journal for*
483 *Numerical and Analytical Methods in Geomechanics* 2008; 32:251–264.
- 484 [18] Knappett JA, Haigh SK, Madabhushi SPG. Mechanisms of failure for shallow foundations under
485 earthquake loading. *Soil Dynamics and Earthquake Engineering* 2006; 26. 91-102
- 486 [19] NEHRP. Recommended provisions for the development of seismic regulations for new buildings
487 2003 Edition. Building Seismic Safety Council, Washington, DC.
- 488 [20] Schofield AN. Dynamic and earthquake geotechnical centrifuge modelling. *International*
489 *Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics* 1981;
490 2.
- 491 [21] Stewart DP, Chen YR, and Kutter BL. Experience with the use of methylcellulose as a viscous
492 pore fluid in centrifuge models. *Geotechnical Testing Journal* 1998; 21(4), 365–369.
- 493 [22] Bertalot D. Foundations on layered liquefiable soils. PhD thesis. University of Dundee. Dundee,
494 UK; 2013.
- 495 [23] Bertalot D, Brennan AJ, Knappett JA, Muir Wood D, Villalobos F. A. Use of centrifuge
496 modelling to improve lessons learned from earthquake case histories. *Proceedings of the 2nd*
497 *European conference on physical modelling in geotechnics, Eurofuge 2012*. Delft, the Netherlands.
- 498 [24] Zeng X, Schofield AN. Design and performance of an equivalent-shear-beam container for
499 earthquake centrifuge modelling. *Géotechnique* 1996; 46(1), 83–102.
- 500 [25] Madabhushi SPG, Teymur B. Experimental study of boundary effects in dynamic centrifuge
501 modelling. *Géotechnique* 2003; 53(7), 655–663.
- 502 [26] Lee S. H, Choo YW, Kim DS. Performance of an equivalent shear beam (ESB) model container
503 for dynamic geotechnical centrifuge tests. *Soil Dynamics and Earthquake Engineering*. 2013; 44,
504 102–114.

- 505 [27] Lauder K. The performance of pipeline ploughs. PhD thesis. University of Dundee. Dundee, UK;
506 2011.
- 507 [28] BSI. BS EN 1998-1:2005: Eurocode 8: Design of structures for earthquake resistance – Part 1:
508 General rules, seismic actions and rules for buildings. British Standards Institution. London, UK;
509 2005.
- 510 [29] Brennan AJ, Knappett JA, Bertalot D, Loli M, Anastasopoulos I, Brown MJ. Dynamic centrifuge
511 modelling facilities at the University of Dundee and their application to studying seismic case
512 histories. In Proceedings of the 8th international conference on physical modelling in geotechnics (eds
513 C. Gaudin and D. J. White), 227–233. London, UK: Taylor & Francis Group.
- 514 [30] Idriss IM, Boulanger RW. SPT Based Liquefaction Triggering Procedures. Report No.
515 UCD/CGM-10-02, Centre for Geotechnical Modelling, Department of Civil and Environmental
516 Engineering, University of California Davis, California; 2010.
- 517 [31] Skempton AW. Standard penetration test procedures and the effects in sands of overburden
518 pressure, relative density, particle size, ageing and overconsolidation. *Géotechnique* 1986; 36(3),
519 425–447.
- 520 [32] Madabhushi G, Knappett AJ, Haigh S. Design of pile foundations in liquefiable soils. Imperial
521 College Press, London, UK. 2010.
- 522 [33] Loli M, Knappett JA, Brown MJ, Anastasopoulos I, Gazetas G. Centrifuge modeling of rocking-
523 isolated inelastic RC bridge piers. *Earthquake Engineering and Structural Dynamics* 2014; 43(15):
524 2341-2359.

List of figure captions

Figure 1 Strip model structure: dimensions at prototype scale are shown in m; dimensions at model scale are given in mm in brackets ().

Figure 2 Test configuration and instrument positions – examples of tests SQ04 and SQ07 shown; dimensions at prototype scale are shown in m; dimensions at model scale are given in mm in brackets ().

Figure 3 Input motion spectra for nominal 5% structural damping.

Figure 4 Normalised input spectra from all tests and a total of twenty events showing ground motion repeatability.

Figure 5 (a) Liquefaction susceptibility F_{SL} and predicted excess pore water pressure ratio $r_{u,predicted}$; (b) measured $r_{u,FF}$ in the free-field along depth in all tests.

Figure 6 Acceleration response spectra of the motion at the free-field soil surface compared to the input ('bedrock') motion showing liquefaction effects (test SQ03 only shown for clarity).

Figure 7 PGA change with depth due to liquefaction effect in the isolated strip case (test SQ03).

Figure 8 Spectral reduction factor along period in the isolated strip case (test SQ03).

Figure 9 Soil factor (free-field amplification) as a function of degree of liquefaction.

Figure 10 Response history of isolated structures during Earthquake 1 (a) Storey 1 acceleration; (b) post EQ settlement; (c) overall tilt; (d) pore pressure ratio of free-field surface; (e) Input motion.

Figure 11 Response history of isolated structures during Earthquake 5 (a) Storey 1 acceleration; (b) post EQ settlement; (c) overall tilt; (d) pore pressure ratio of free-field surface; (e) Input motion.

Figure 12 Structural response of isolated and adjacent cases at storey 1: (a) storey acceleration; (b) cyclic sway and inter-storey drift.

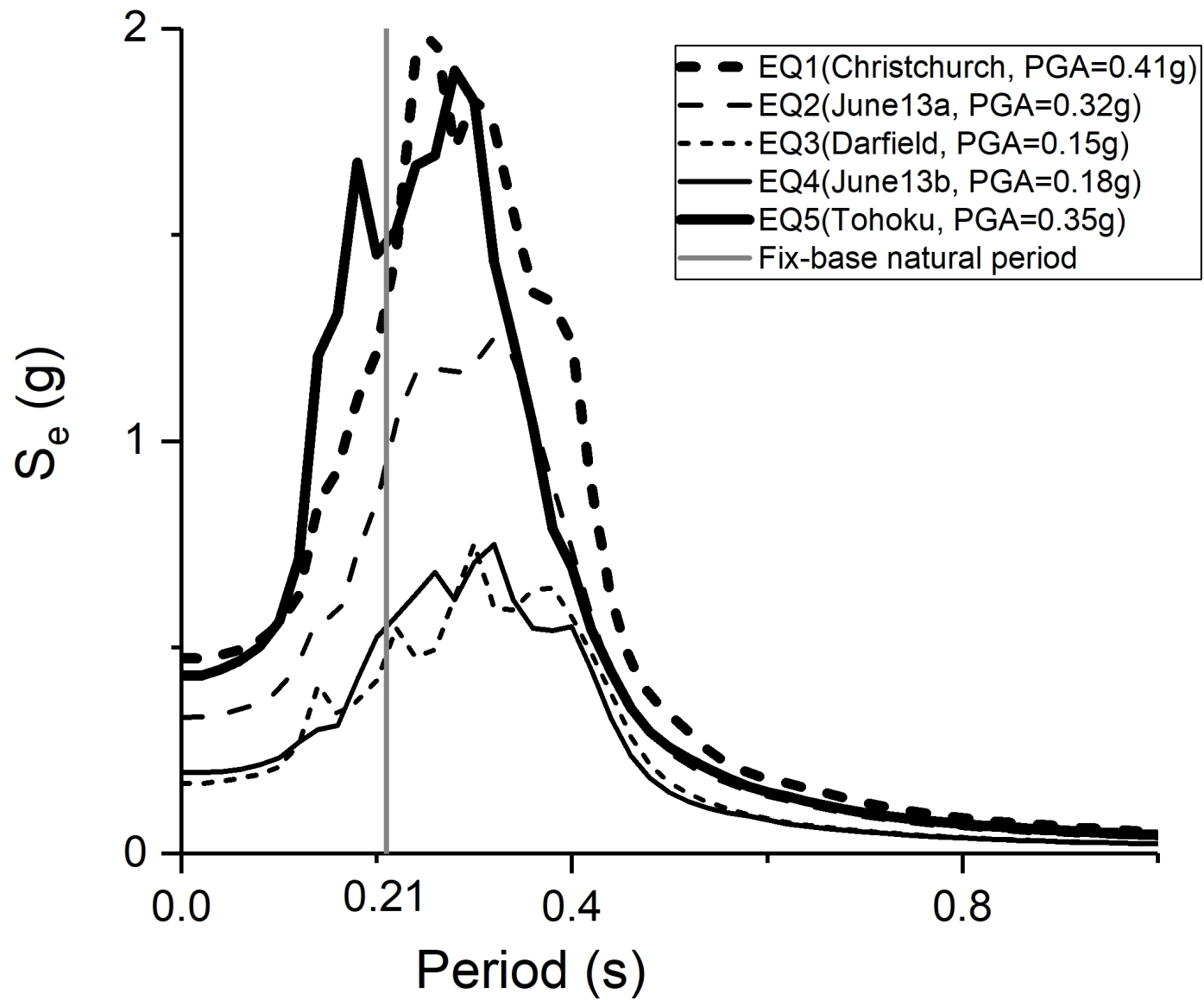
Figure 13 Vertical peak cyclic displacements of the foundations (a) isolated cases; (b) adjacent cases.

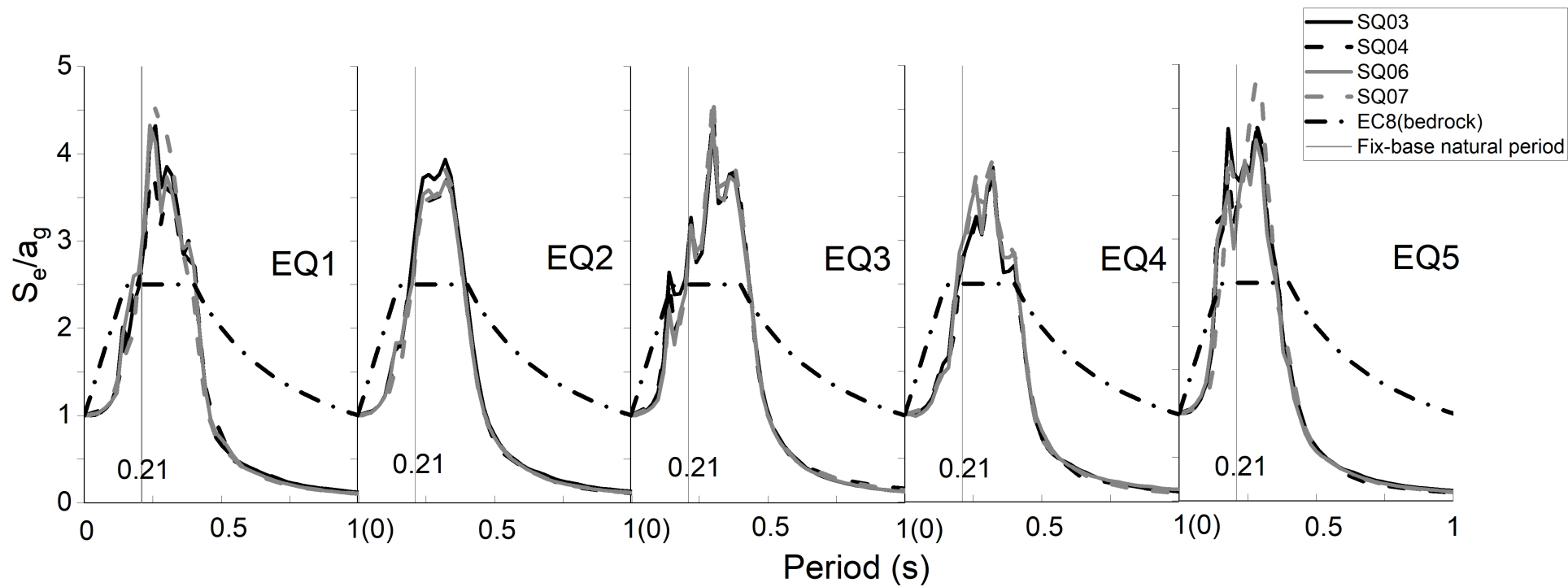
Figure 14 Transfer function results: (a) effective period for strip cases; (b) effective period for raft cases; (c) effective damping for strip cases; (d) effective damping for raft cases.

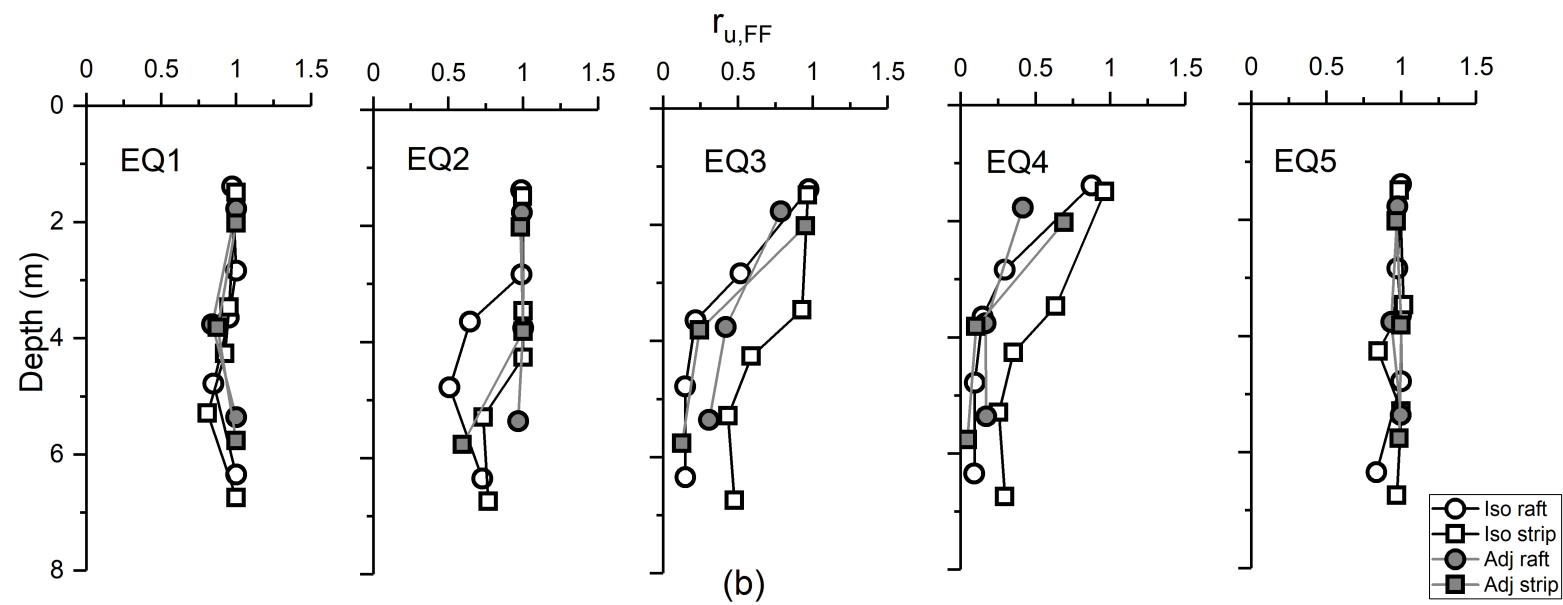
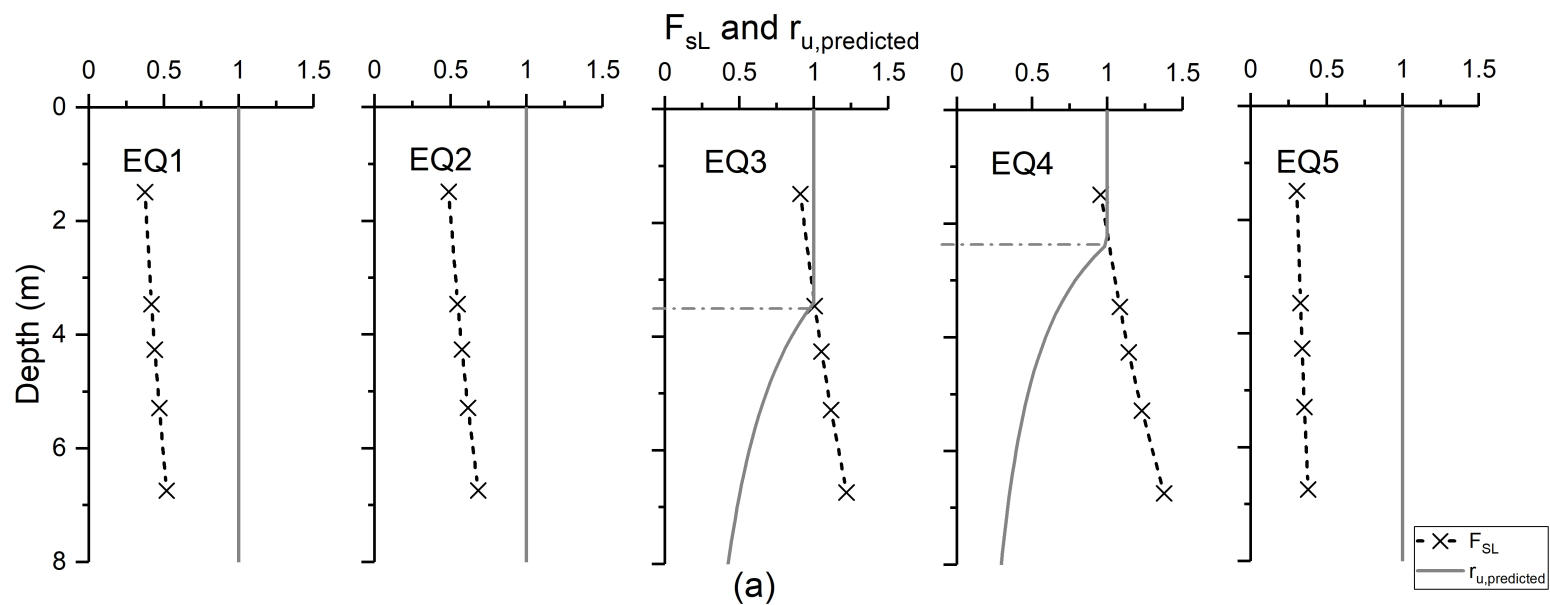
Figure 15 Effects of SSSI on damping and period of isolated relative (EQ2 spectra shown with full liquefaction).

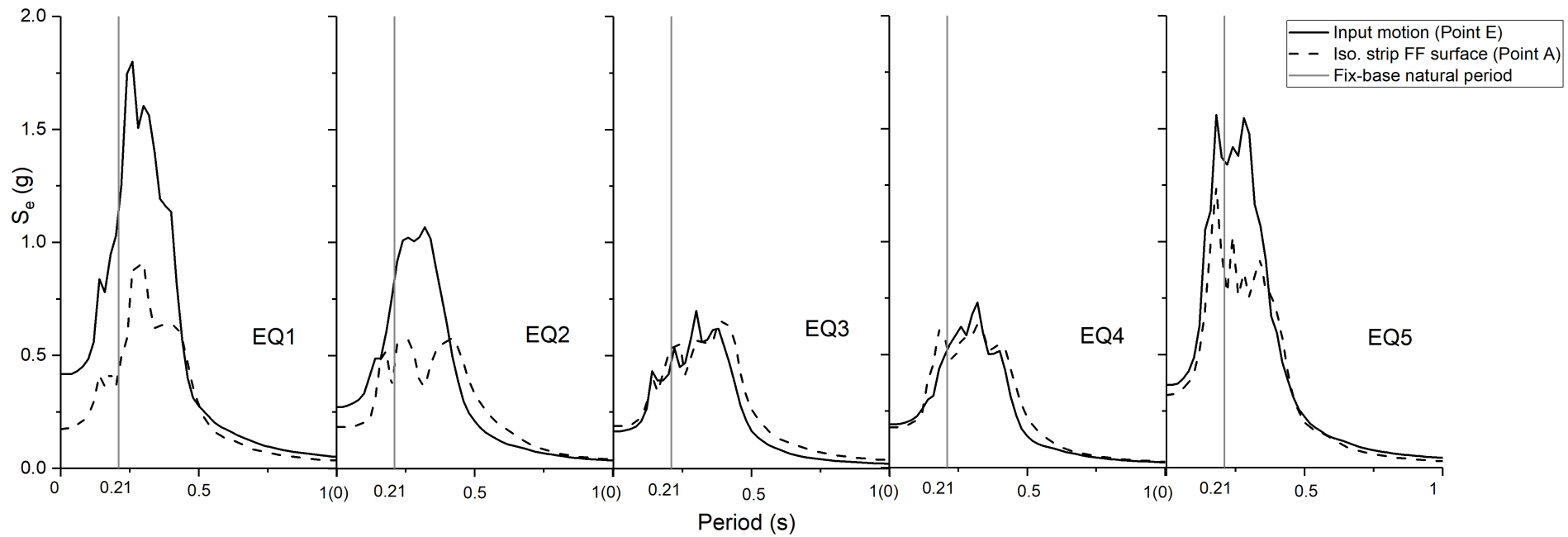
Figure 16 Earthquake induced accumulative rotation (a) for strip cases; (b) for raft cases.

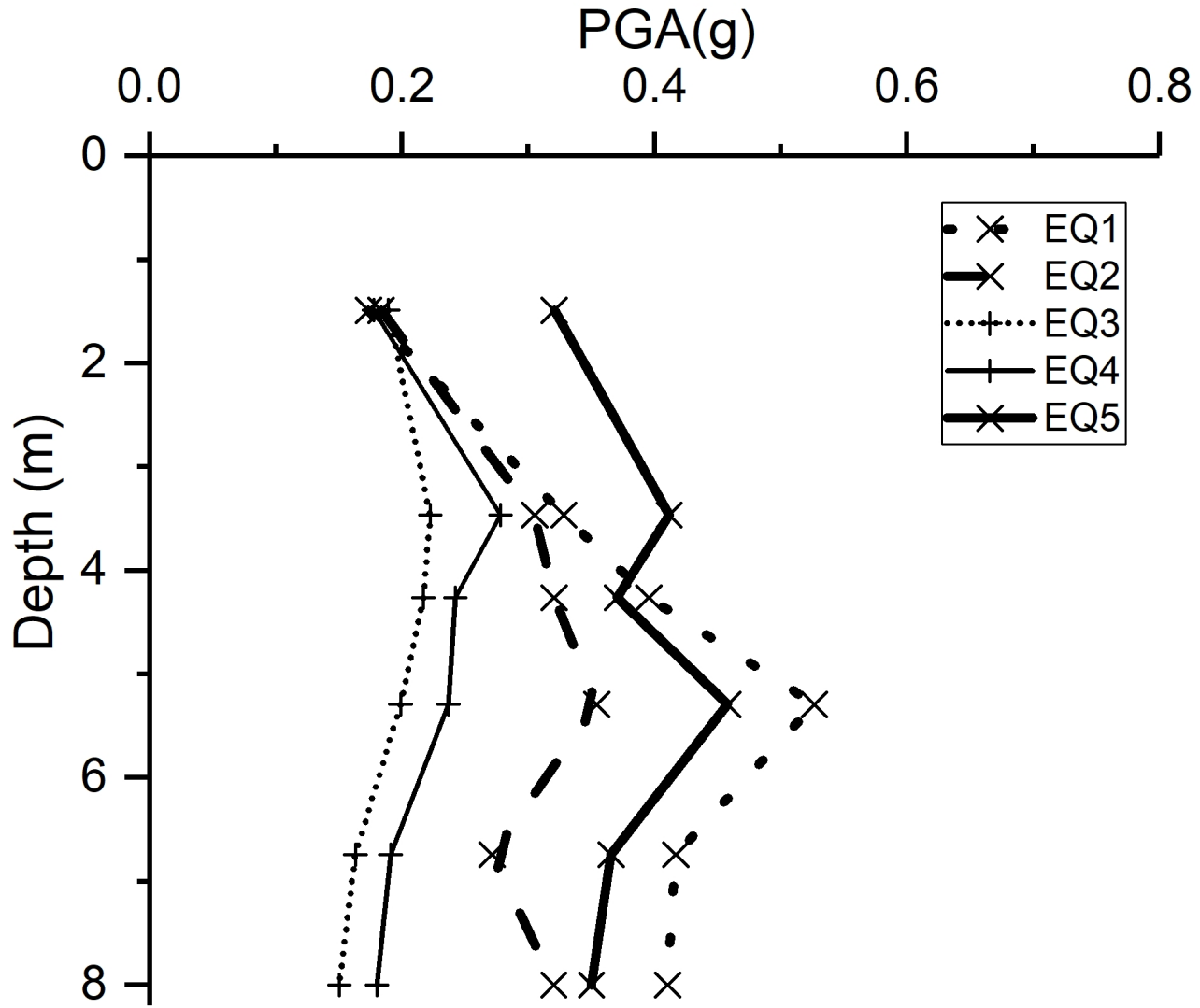
Figure 17 Earthquake induced accumulative settlement (a) for strip cases; (b) for raft cases.

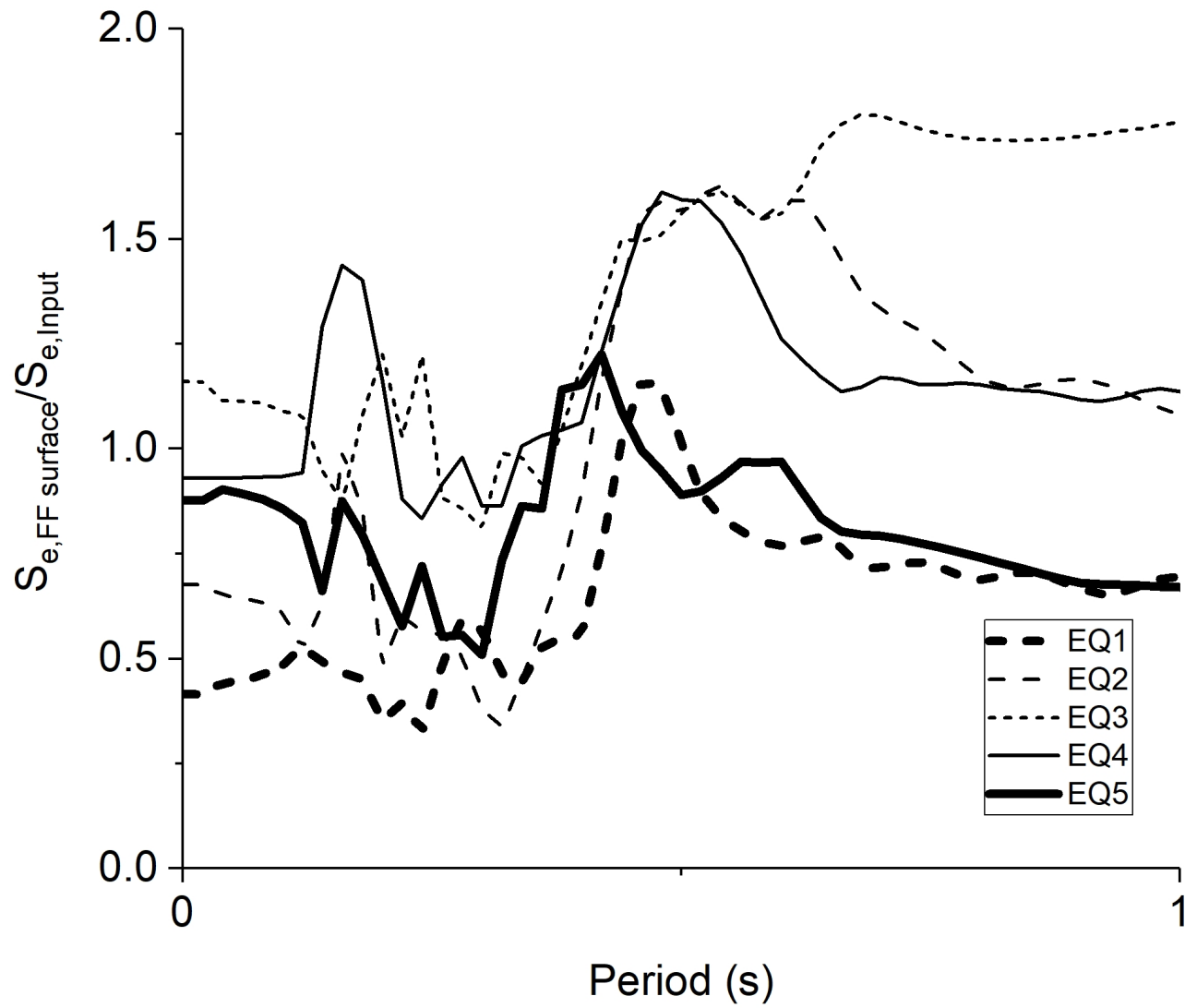


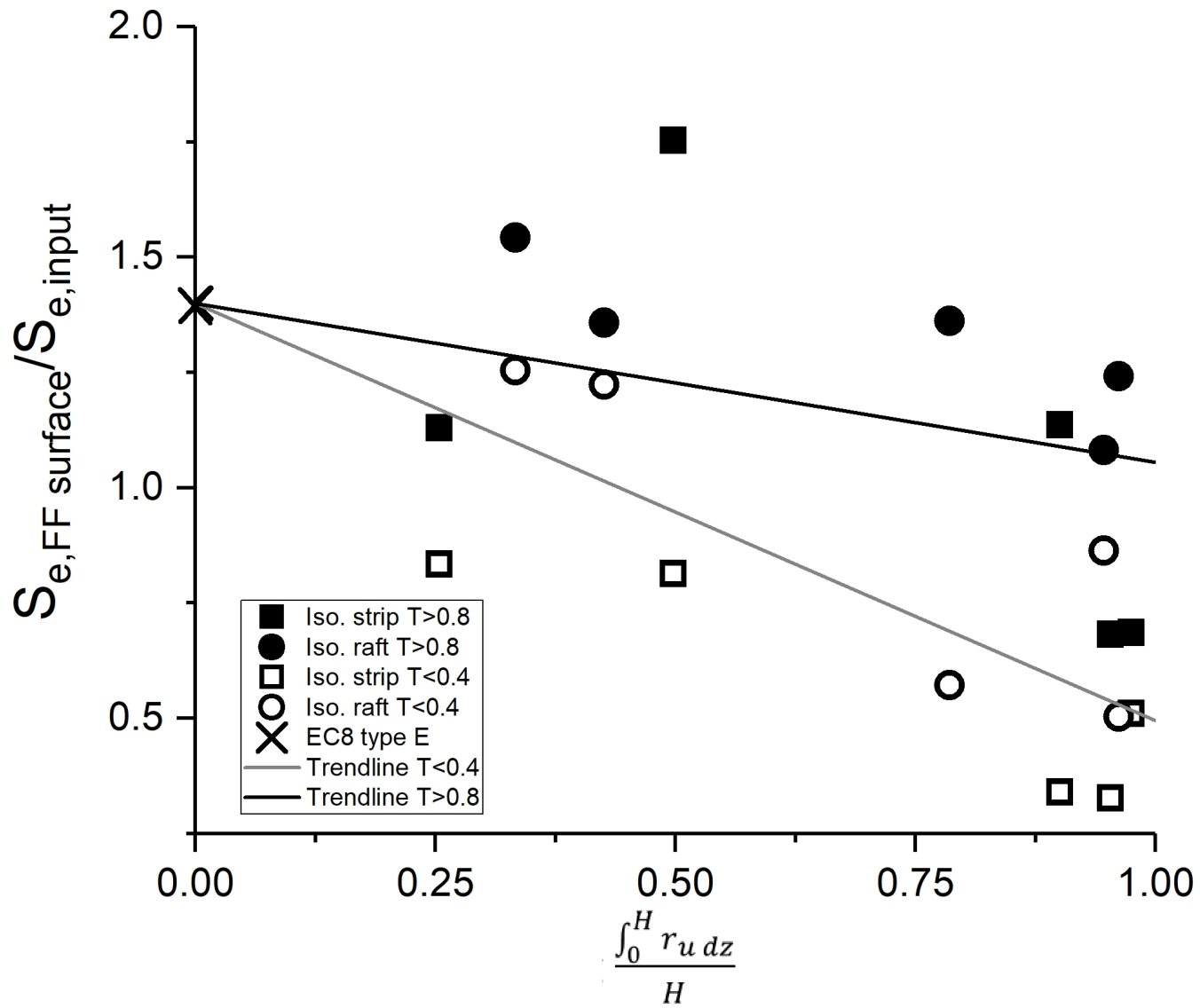


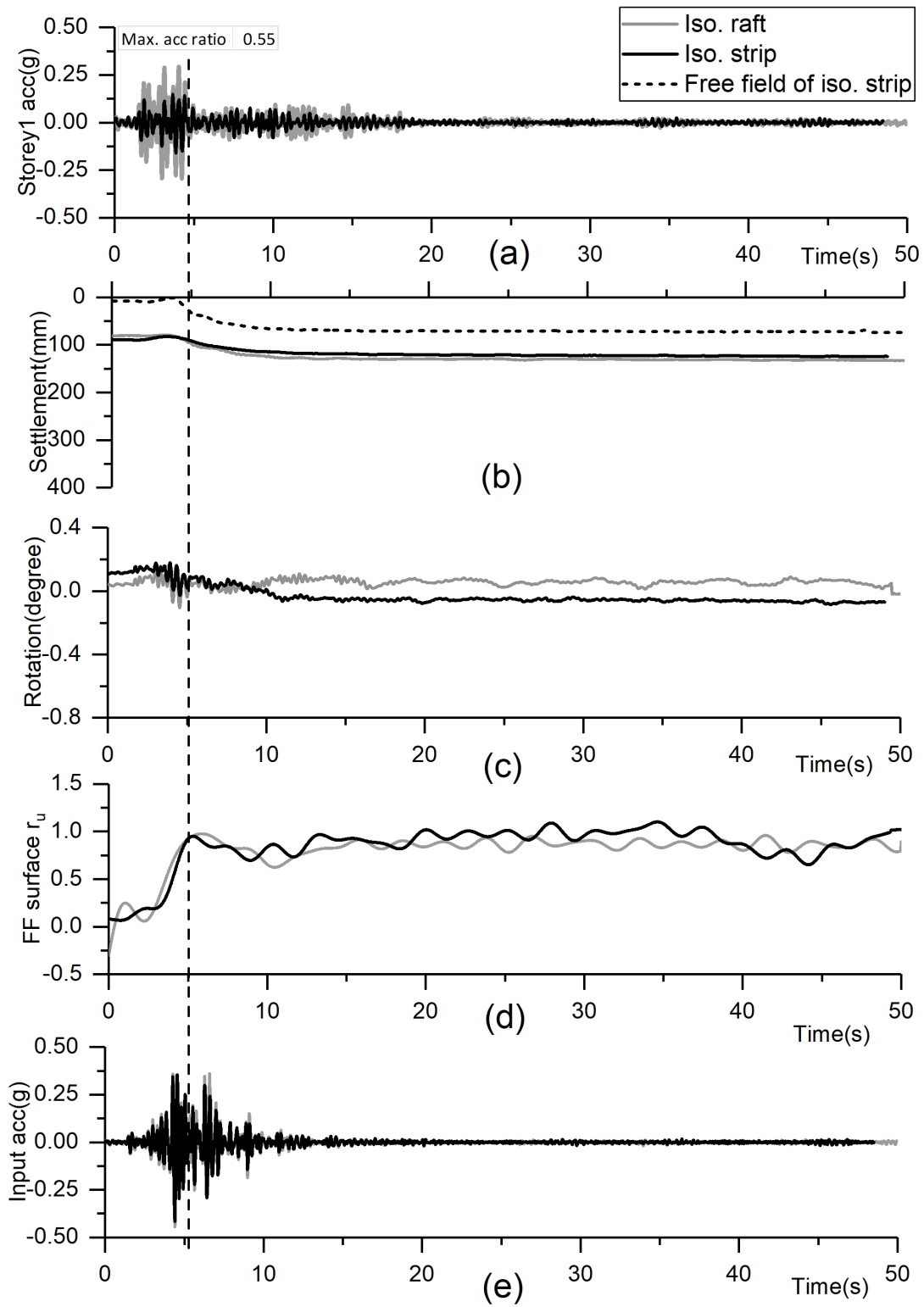


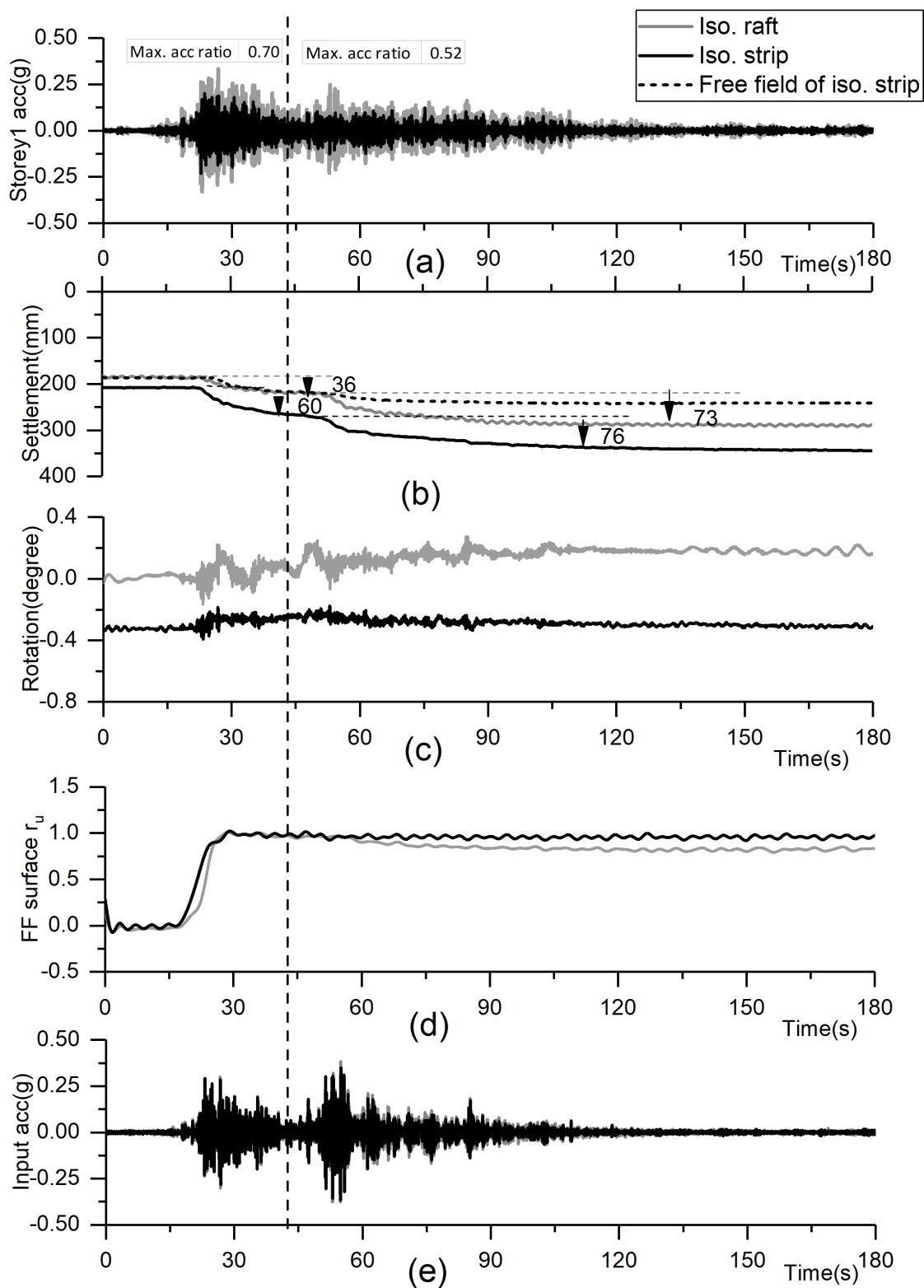


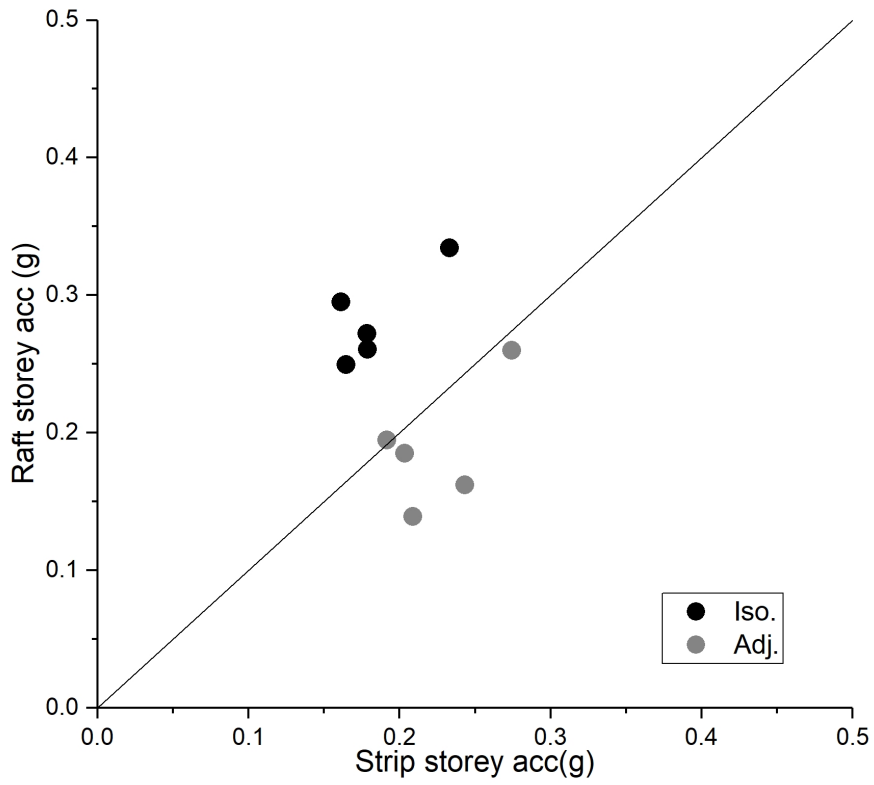




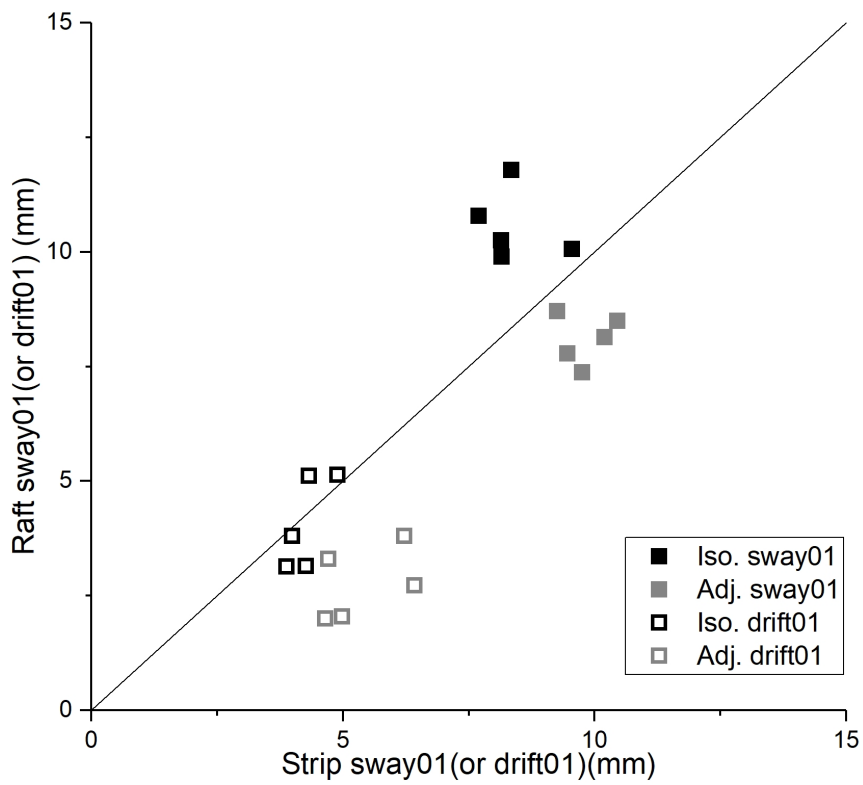




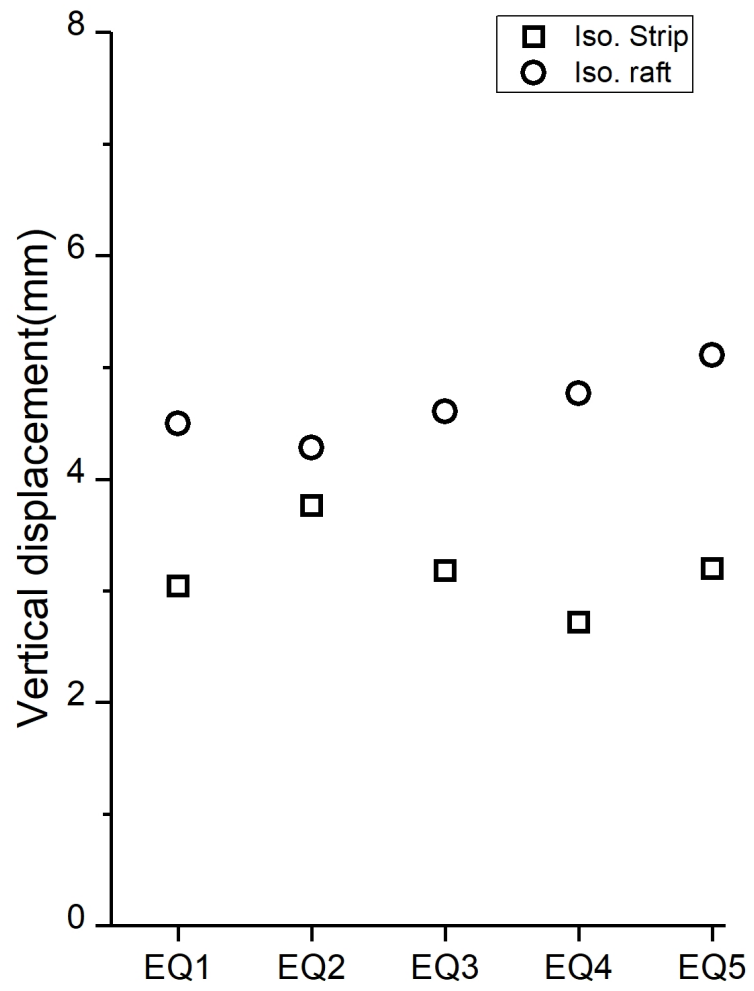




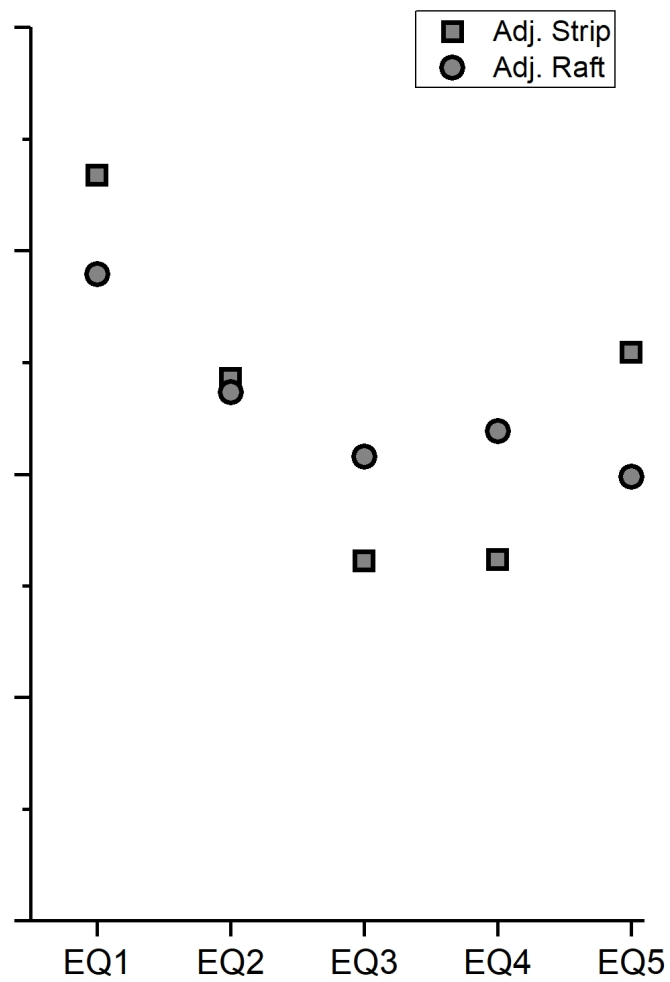
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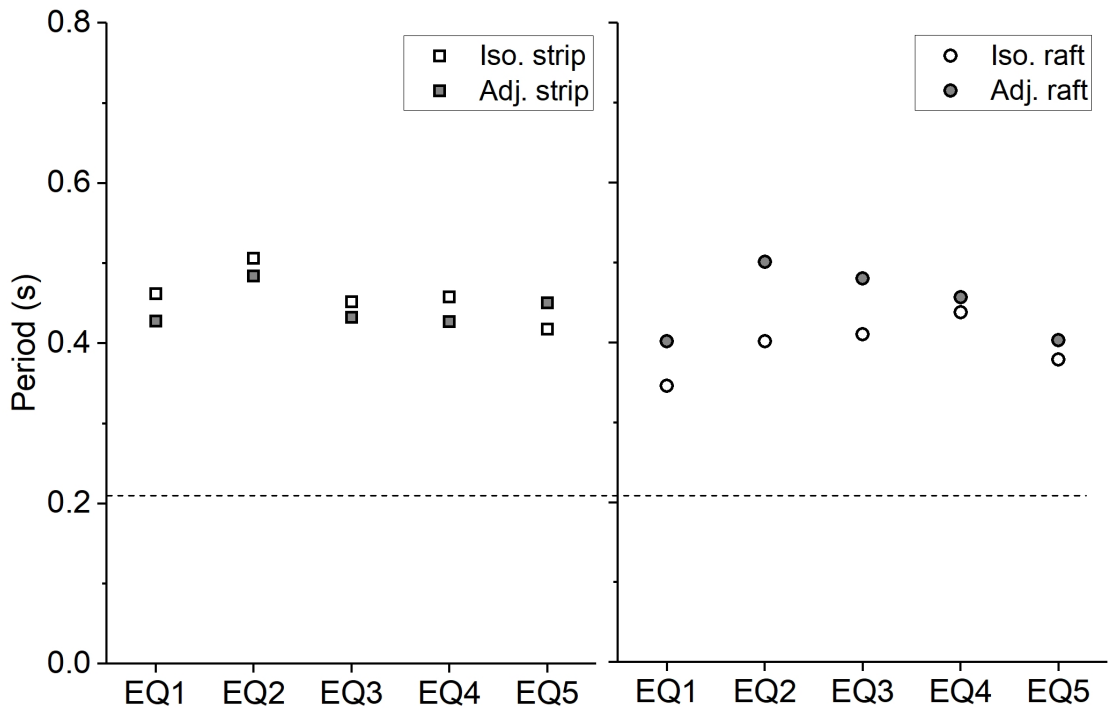
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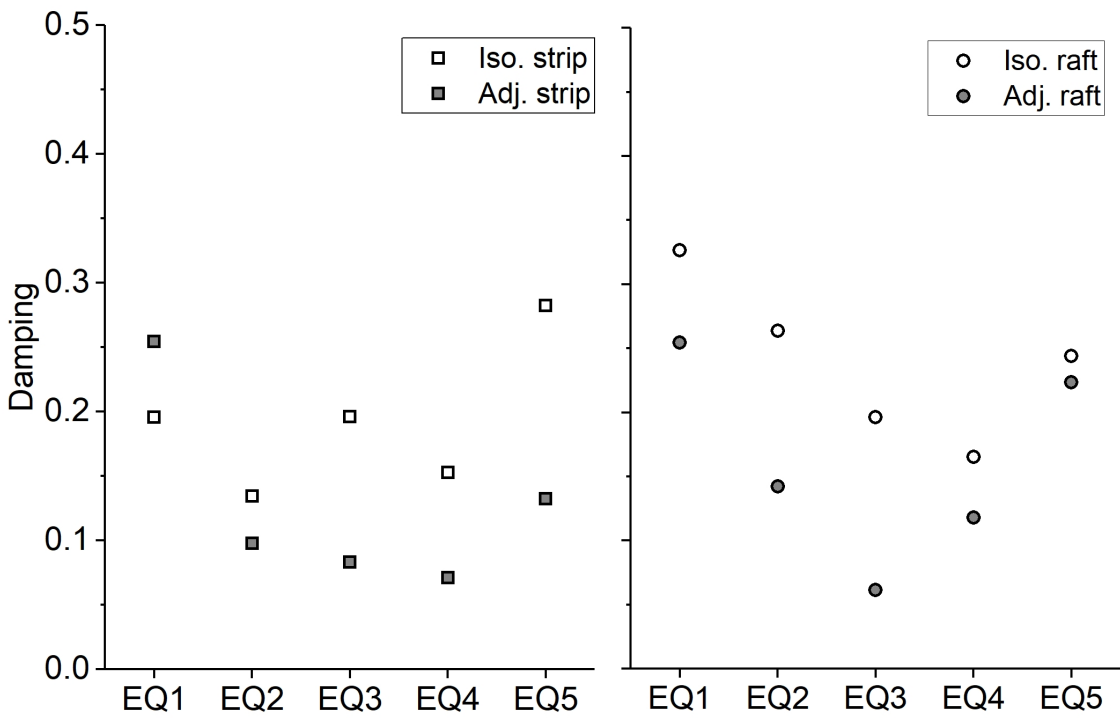


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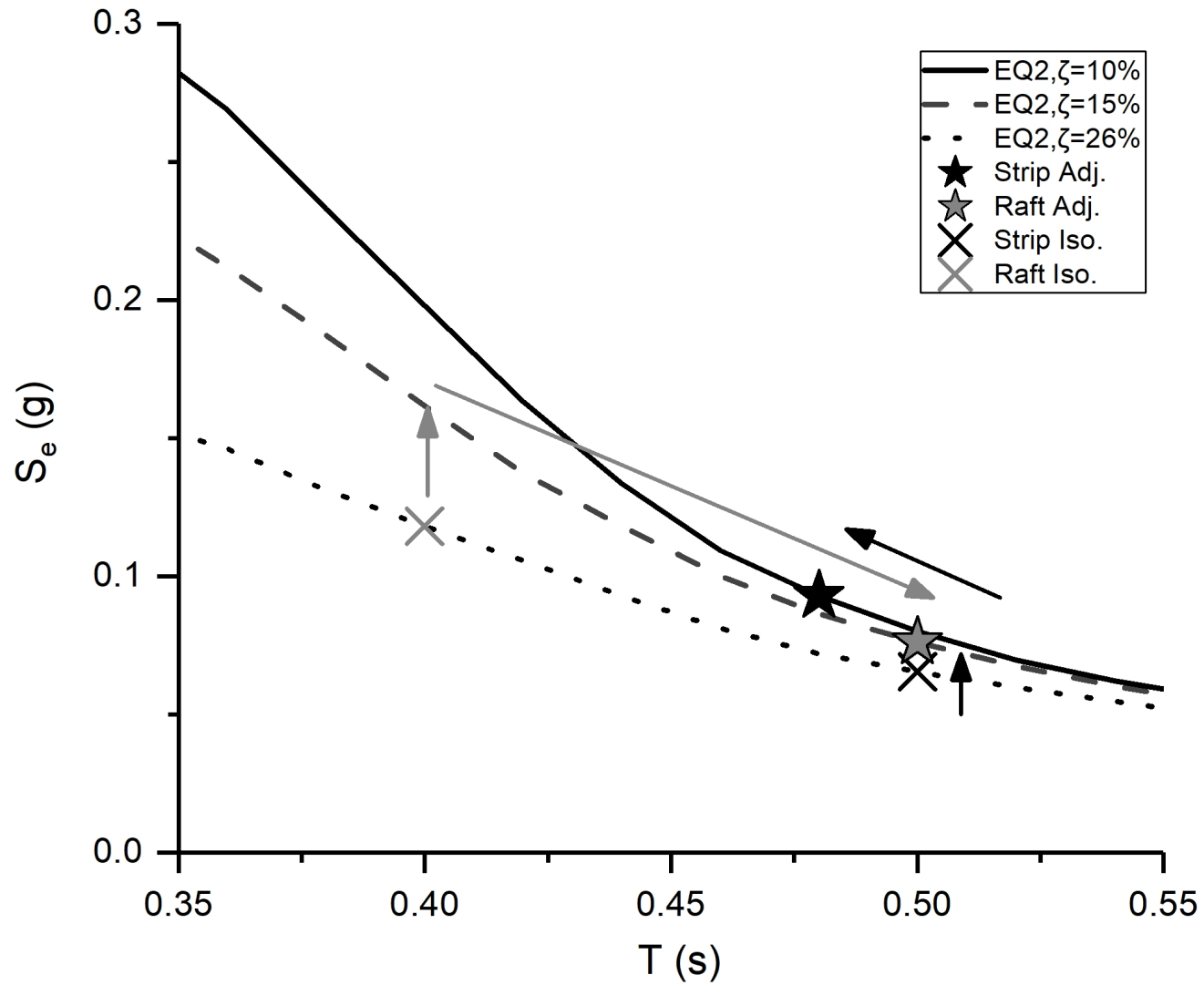
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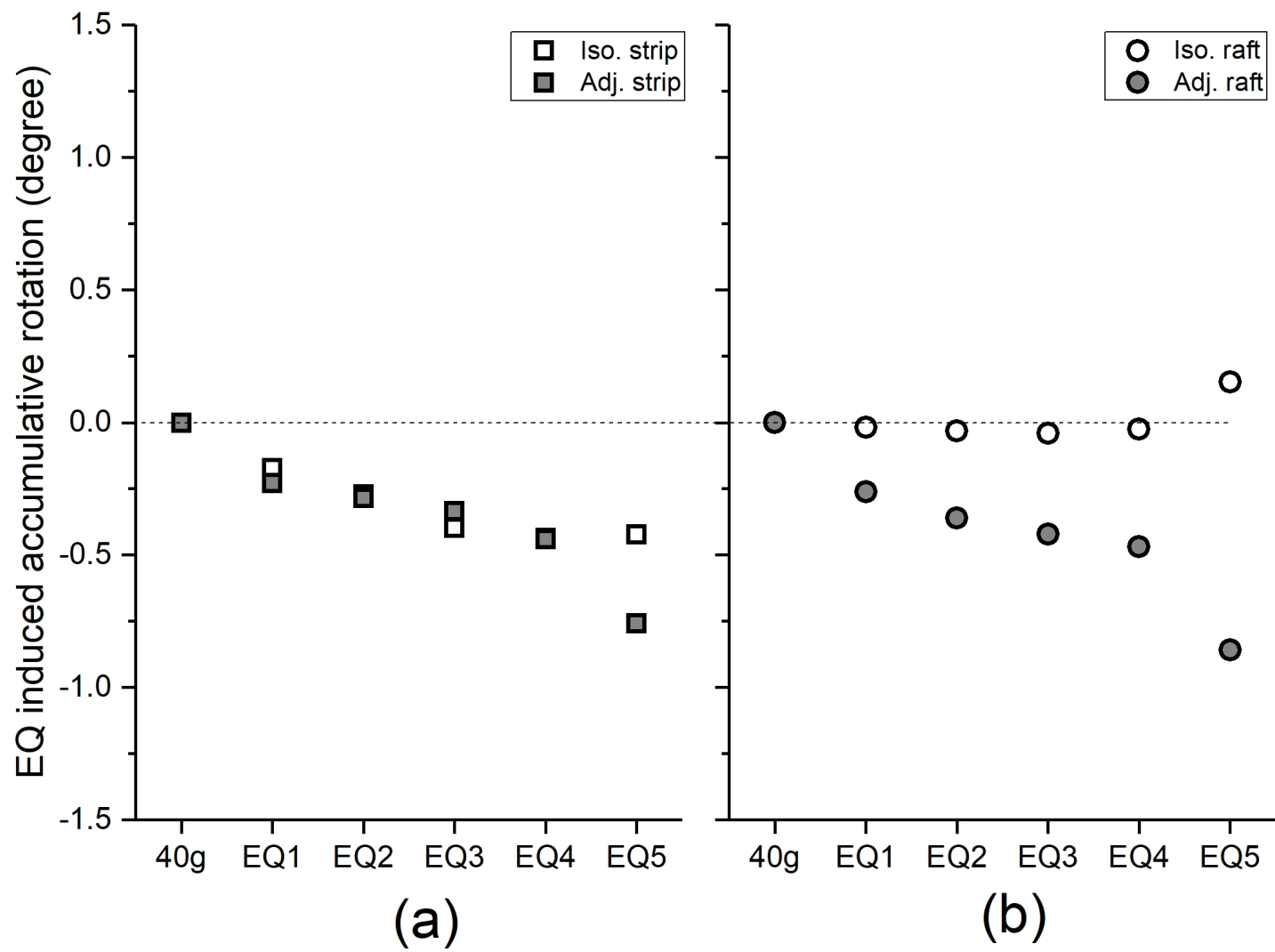
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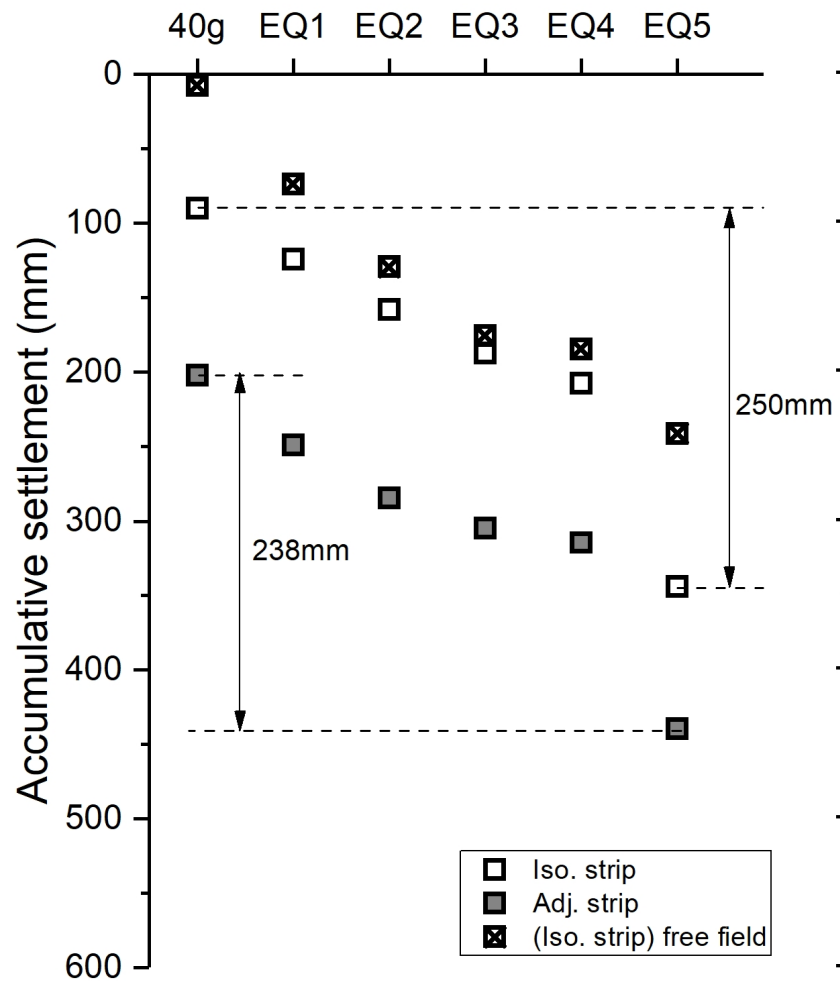


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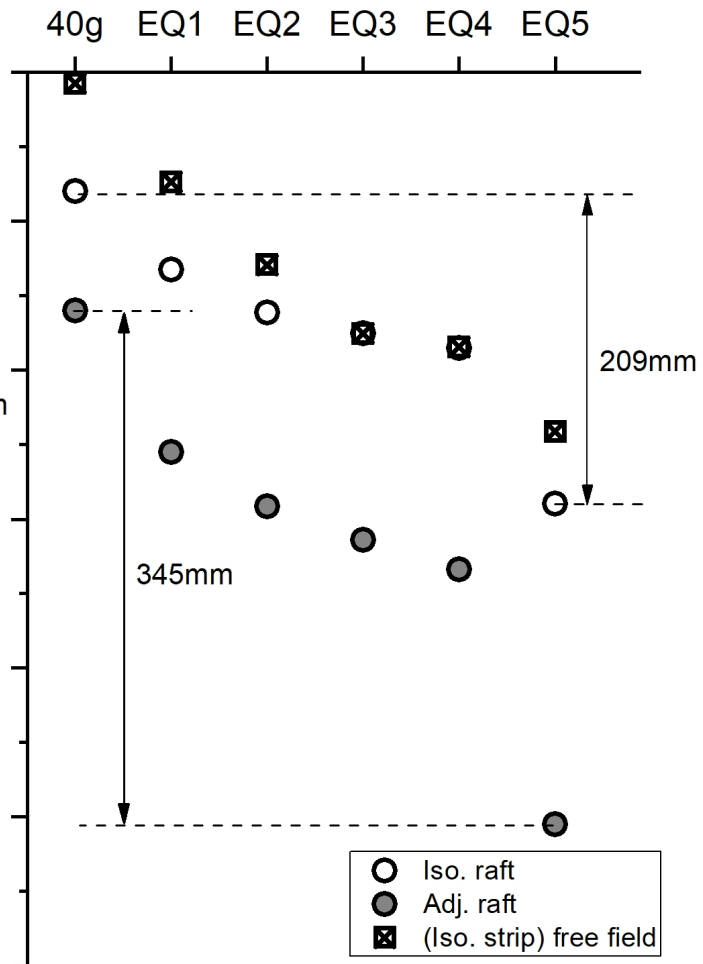
(d)







(a)



(b)

Table 1 Centrifuge test configurations.

Test No.	Configuration	Foundation type	Foundation edge-to edge spacing(m)
SQ03	Isolated, light	Strip	N/A
SQ04	Isolated, light	Raft	N/A
SQ06	Adjacent, light+heavy	Strip, Strip	1.2
SQ07	Adjacent, light+heavy	Raft, Raft	1.2

Table 2 Properties of model structures (at prototype scale).

Parameter: units	structure of interest (Light structure)	accompany structure (Heavy structure)
Storey height: m	3	
Total height: m	6	
Concrete slab dimensions: m	3.6×3.6×0.5	
M_{eq} : kg	16.5×10 ³	23.8×10 ³
K_{eq} : N/m	37.1×10 ⁶	
Stiffness of columns, EI : MNm ²	20.9	
Static FOS	3 (strip), 14.7 (raft)	2.5 (strip), 12.2 (raft)
Bearing pressure: kPa	50 (strip), 31 (raft)	62 (strip), 38 (raft)
Fixed-base natural period: s	0.21	0.25
Strip footing spacing (centre-to-centre): m	3.6	

Table 3 Properties of HST95 Congleton sand (after Lauder, 2011).

Property: units	Value
Specific gravity, G_s	2.63
D_{10} : mm	0.09
C_u (uniformity) and C_z (curvature)	1.9 and 1.06
e_{max} and e_{min}	0.769 and 0.467
ϕ'_{pk} at $I_D = 55\%$: °	38
ϕ_{crit} : °	32