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Validation of a finite element modelling approach on soil-foundation-structure interaction of a multi-storey wall-frame structure under dynamic loadings

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1. Abstract

Validated numerical approaches are very important in dynamic studies of soil-structure interaction. Experimental outputs of physical models are required to validate the numerical approaches. Testing and analysis of an experimental scaled model is economical in comparison with investigating real size structures. However, a set of scale factors are required to model a full-scale structure accurately as a scaled model in a laboratory environment. In this paper, the scaling procedure and design of a scaled multi-storey concrete wall-frame structure with a scale factor of 1:50 are addressed. **A dry sand with round shaped particles with a specific grain size distribution was adopted in this study.** A flexible soil container was then designed and built to represent the soil boundary behaviour during time-history seismic excitations. The experimental investigations were divided into three different stages: fixed based structure without soil interaction; soil container without any structure; and, a structure with raft and pile foundations in the soil container. Then the same experimental stages were modelled numerically in 3D using finite element software. The results showed that the finite element simulations produced a good response when compared with the experimental results and these numerical models are suitable to be employed for further dynamic studies.

Keywords: seismic; dynamic; finite element; wall-frame structure; multi-storey buildings; soil-structure interaction.

2. Introduction

The motions of soil influence the structural response, which is referred to as soil-structure interaction (Kramer, 1996). This interaction is amplified as the behaviour and properties of soil

32 varies under dynamic loading. The unavailability of standards and validated analytical
33 techniques for estimating the soil-foundation-structure interaction (SFSI) lead to either
34 simplifying or ignoring the interaction. Hence, the structural and geotechnical aspects of the
35 foundations are analysed separately when it comes to seismic studies. For example,
36 geotechnical engineers may simplify a multi-degree of freedoms to an oscillator with single-
37 degree of freedom system, while the structural engineers replace the non-linear behaviour of
38 the structure with linear springs or ignore the soil-structure interaction altogether
39 (Tabatabaiefar, 2012; Massimino and Maugeri, 2013; Hokmabadi et al., 2014b).

40

41 Interaction between soil, foundation and building structure is the main concern to the designer
42 engineers which is mainly governed by the prevailing ground conditions, the type of
43 superstructure, the foundation type, the magnitude and distribution of the building loads, plus
44 seismic excitation (Sinn et al., 1995). It was shown that the foundation on flexible soil
45 significantly increases the overall displacement of the superstructure compared to a structure
46 on a rigid soil or a foundation fixed to bedrock (Hokmabadi et al., 2014a; Guin and Banerjee,
47 1998; Han, 2002). This increase in total deformation due to flexible soil may lead to structural
48 instability due to the secondary moment at the base (Ma et al., 2009). Furthermore, Hokmabadi
49 et al. (2014) showed that the soil-structural interaction of multi-storey tall buildings is
50 significant with raft foundations compared to pile foundations. Hence, in foundation and
51 superstructure design of high-rise buildings, the soil-structure interaction (SSI) should be
52 considered and not be restricted to conventional design methods, like considering the soil
53 bearing capacity approach with an applied factor of safety (Poulos et al., 2016). Consequently,
54 the high-rise buildings design methodology has been changed recently. The full three-
55 dimensional finite element modelling of buildings becomes unusual without considering the
56 effect of soil-foundation-structure interaction due to dynamic behaviour during the seismic
57 excitation (Hallebrand et al., 2016).

58

59 **3. Structural design**

60 This study involves soil container with structure. A detailed literature review of 1-g shaking
61 table tests of a soil container with and without structure and foundation are summarised in
62 Table 1. In the literature, scaled structures were physically modelled as either single-degree of
63 freedom (SDOF) systems, lumped mass multi-degree of freedom (MDOF) systems or multi-
64 degree of freedom scaled models of an actual structure. When it comes to the high-rise

65 buildings, SDOF would not be suitable. Furthermore, stability of a lumped mass MDOF system
66 is hard to achieve in an experimental investigation. Hence MDOF scaled model approach has
67 been widely implemented in recent studies.

68

69 In general, multi-story wall-frame concrete structures without damping systems are rigid and
70 flexibility within the structure is limited due to brittleness of concrete. Therefore, the structure
71 is expected to be rigid compared to underlying soil. In this system, most of the deformation
72 occurs within the soil rather than in the structure. Furthermore, in the analysis of multi-degree
73 of freedom systems, there are different mode shapes occurring during the seismic excitation.
74 The first mode (deflection mode) shape is the most critical in regular multi-story shear wall
75 column structural systems, due to the mass participation ratio being higher than other modes.
76 Therefore, in this study, deflection and seismic excitation were considered in one direction to
77 obtain the maximum response of the structure.

78

79 **3.1 Prototype**

80 The prototype model building used in this study consisted of a concrete wall-frame structural
81 system with two basement floors and fifteen stories above ground level with a total height of
82 53 m, width of 10 m and length of 10 m. The structural form and sections were designed based
83 on Eurocodes design guidelines (Eurocode 2, 2014; Eurocode 8, 2014; Cobb, 2014). ETABS
84 (CSI, 2015) software was employed for analysis and design of this structure. The live load (2
85 KN/m²), dead load (5.5 KN/m²), wind load (wind speed of 45 m/s for the terrain category of 2
86 based on Eurocode 2 (2014)) and seismic elastic spectra (Eurocodes Soil Type C with
87 maximum acceleration intensity of 0.2 g) were considered in the design of the structural
88 geometries and materials. A compressive strength (f_{cu}) of 40 N/mm², a mass density of 2400
89 kg/m³ and Young's elastic modulus of 36000 N/mm² were used for this concrete wall-frame
90 structure. The final design of the structural frame and sections are illustrated in Figure 1.

91

92 It can be noted that the selected characteristics of the multi-story building represent the
93 common construction practices and conventional buildings in megacities. The prototype meets
94 the required level of safety according to the European Codes of practice. The necessary
95 parameters for this study such as natural frequencies, total weight and dimensions of the
96 prototype were obtained using the ETABS software package as shown in Table 2.

97

98 **3.2 Scaled model**

99 **3.2.1 Design**

100 Moncarz and Krawinkler (1981) explained the 1-g scale model test procedure, where the ratio
101 (E/ρ) of the scaled model to prototype equals the scaling factor λ known as “Cauchy condition”
102 to unity implying, where E and ρ are the Young’s modulus of elasticity and the density of mass,
103 respectively. The scaled models can be classified into three different categories based on the
104 degrees of accuracy: true, adequate, and distorted models (Moncarz and Krawinkler, 1981).
105 True models require the geometric and dynamic simulation factors on the scaled model.
106 Adequate models use the primary features which influence the behaviour of the scaled model,
107 where secondary features may not be considered. The distorted model does not comply with
108 the simulation requirements. To simulate the overall behaviour of tall buildings within the
109 means available and to focus on the soil-foundation-structure behaviour, the adequate model
110 type with primary features of mass and frequency were used for this paper.

111

112 The scale factors used in this study according to Pitilakis et al. (2008) are summarised in Table
113 3. The scale factor of 1:50 was selected to scale down the prototype model. Thus, the scaled
114 model dimensions are 1.06 m in height (H), 0.20 m in length (L), and 0.20 m in width (W) as
115 shown in Figure 1. In this adequate model, the natural frequency and the total mass of the
116 structure governed the design of the scaled model (Tabatabaiefar and Mansoury, 2016; Pitilakis
117 et al., 2008). The total mass (scale factor of 1) and frequency (scale factor of $\lambda^{-\frac{1}{2}}$) of scale
118 model were calculated as 9.33 Hz and 23.2 kg respectively, shown in Table 2.

119

120 **3.2.2 Physical construction of the scaled model**

121 In order to obtain the dimensions of steel plates and tube (representing slabs and column,
122 respectively), the expected scaled model was also analysed and designed using the ETABS
123 software, and the dimensions were selected to meet the required natural frequency and the total
124 mass of the scaled model as shown in Table 2. Grade 255 steel (255 N/mm^2) was adopted in
125 all the elements of the scaled model. This was due to the fact that structural steel is flexible and
126 constructible to the test environment, while a concrete structural model could not be
127 constructed with the required dimensions and dynamic properties. In the scaled model, each
128 floor is supported by vertical steel tubes (8 mm in external diameter and 1 mm in thickness) as
129 the column elements. Dimensions of $220 \times 220 \times 5 \text{ mm}$ and $200 \times 200 \times 2 \text{ mm}$ steel plates
130 were selected as the base and the typical floor slab of the scaled model, respectively. The

131 connections between the columns and floors were provided using 4 mm diameter steel thread
132 bars screwed by nuts on both ends of the top level and the base floor. Steel plates of 200 × 160
133 × 3 mm were attached vertically to the lower levels to represent the basement retaining walls
134 as shown in Figure 1. Once all the members were assembled, the total weight of the scaled
135 model was measured as 23.7 kg. The resonance of the as-built scaled structural model (natural
136 frequency) was determined by hammer test, which was approximately 9 Hz. The total masses
137 and the natural frequencies of the as-built physical scaled model are shown in Table 2.

138

139 **4. Soil container design**

140 To simulate the soil beneath the structure, three types of containers namely, rigid container,
141 laminar container and flexible barrel were utilised for dynamic studies in the literature, as
142 illustrated in Table 1. A rigid container does not represent the actual boundary conditions of
143 the soil (Gohl and Finn, 1987; Meymand, 1998). Therefore, the focus was directed towards
144 laminar and flexible containers. The laminar soil container consisted of an aluminium frame
145 with rectangular hollow sections made. Those frames are separated by rubber layers (Biondi et
146 al., 2003). The aluminium frame's function was to provide the soil lateral confinement, while
147 the rubber layer function was to allow the soil to have the shear deformation (Meymand, 1998;
148 Prasad et al., 2004; Hokmabadi et al., 2014). The main part of the flexible container was the
149 flexible membrane with stiffening rings, which represents the response of the free field site
150 under dynamic events during shaking table test (Meymand, 1998). Meymand (1998) compared
151 various soil container types in his numerical study. The results showed that the flexible wall
152 container simulates the soil prototype more accurately, while the rigid wall container does not
153 replicate the behaviour of soil under dynamic conditions. This observation was further
154 validated using a flexible barrel container in 1-g shaking table test (Meymand, 1998).
155 Moreover, Crosariol (2010) and Moss et al. (2011) tested both flexible barrel and laminar
156 containers, and the flexible barrel container provided the best response in comparison with the
157 experimental results of a prototype. On the separate note, the laminar container is complicated
158 to design and expensive to construct. Therefore, a flexible container with stiffening rings was
159 adopted in this study. **Qaftan et al, (2018) presented a detailed explanation of the behaviour of**
160 **this flexible container.**

161

162 The main concern of the earthquake model tests is the soil boundary effects on the soil response
163 within the soil container. The main function of the soil container was to confine and hold the

164 soil part in place during the dynamic excitation. The ideal soil container should minimise the
165 boundary effect and simulate the free field soil behaviour as it exists in the prototype. The key
166 parameter in the design of the soil container was to satisfy the dynamic shear stiffness of the
167 soil container in the same manner as the adjacent soil deposit to achieve the real response
168 (Hokmabadi et al., 2014).

169

170 Moss and Crosariol (2011) concluded that the flexible barrel container with its constructional
171 details must be designed and constructed properly to minimise the boundary effect, and the
172 container diameter should be five-times or greater than the structure width. Hence, the
173 dimensions of the container were selected as 1 m in diameter and 1m in depth. The flexible
174 container was designed and constructed at the University of Salford (Figure 2). The flexible
175 container consisted of a 5 mm flexible cylindrical membrane wall supported individually by
176 stiffener strips, which were made of steel straps. The top part of the container was supported
177 by lifting hooks from an overhead crane. The arrangement was made to support the top part
178 and was made loose, so it would not interfere with the behaviour of the system. The bottom
179 base was fixed on the shaking table.

180

181 **4.1 Soil properties and placement**

182 Often the soil properties were characterised using the dynamic properties such as shear wave
183 velocities, shear modulus and damping in seismic studies (Wolf, 1985). Most of the studies on
184 soil-structure interaction were conducted on clay soil, as the change in volume of clay during
185 the seismic excitation is insignificant, which simplifies the numerical simulation of the soil
186 sample. When it comes to a typical sandy soil, seismic excitation changes the volume of the
187 soil either from loose to dense or dense to loose. This phenomenon significantly alters the
188 stiffness and the behaviour of the sand. As a first step, the soil-structure interaction of multi-
189 story wall-frame building structures on sandy soils without volumetric changes during seismic
190 excitation should be investigated before studying the behaviour of soil-structure interaction on
191 sandy soil with volumetric changes (Stromblad, 2014). This approach was considered to
192 minimise the complexity of the numerical model.

193

194 Dry sand with certain characteristics was used to reduce the volumetric changes during seismic
195 excitation. The grain size distribution of the sub-rounded sand particles is shown in Figure 3.
196 The densities of the sand according to BS 1377 (1990) shows that the maximum and minimum

197 dry densities were 16 kN/m^3 and 14 kN/m^3 , respectively. The difference between the
198 maximum and minimum void ratios was approximately 0.12. The as-placed density of soil in
199 the experimental investigation cannot reach maximum or minimum values. Hence, the
200 difference in densities during the excitation is smaller than the difference between maximum
201 and minimum densities. This aspect was observed in the experimental investigation. Thus, the
202 changes in densities of the sandy soil has a minor effect on the soil response. The specific
203 gravity of the selected sand was 2.68. The angle of internal friction was measured as 34° using
204 the direct shear tests and the elastic Young's modulus was derived as $80 \times 10^6 \text{ N/m}^2$ using
205 triaxial test. The dilatancy of round sand particles is defined by Ryan and Polanco (2008) as:
206 $w = \emptyset - 30$ (1)

207 where w is the dilatancy and \emptyset is the angle of friction. Other relevant properties of soil can be
208 found in Table 4. To achieve a uniform density, the sand was placed up to 600 mm in the
209 container by using the eluviation (raining) technique (Pitilakis et al., 2008; Dave and Dasaka,
210 2012). The actual relative densities were checked and measured by using small cups with
211 known volume to collect samples at different locations within the main container.

212

213 **5. Numerical modelling**

214 **5.1 Finite element modelling**

215 Numerical simulations were carried out using the ABAQUS/CAE finite element software on
216 the scaled model structural system. Hokmabadi et al. (2014) suggested that the nonlinear
217 dynamic response is required to capture the time-history output of the soil-foundation-structure
218 interaction. Hence, several studies were conducted using the direct calculation method, in
219 which nonlinear time history analyses were performed of whole soil foundation structure
220 systems using ABAQUS (Fatahi et al., 2018; Nguyen et al., 2016; Nguyen et al., 2016). Nguyen
221 et al, (2016) studied the effect of raft foundation size on behaviour of a moment-frame structure
222 modelled using ABAQUS software. Nguyen et al, (2017) considered the same moment-frame
223 supported by pile foundations, where the seismic performance of buildings and the response of
224 pile end bearing in soft soils were considered.

225

226 There are mainly two analytical methods to solve the dynamic behaviour in finite element
227 analysis namely, implicit and explicit dynamic analyses. Implicit and explicit solutions are
228 based on Euler-time integration solution, which is the quantity calculated from the previous
229 time step. The implicit analysis uses Newton-Raphson iterations to enforce equilibrium

230 between the applied external forces and internal structure forces. Therefore, the implicit
231 analysis is more accurate for the analysis of larger time history (Sun et al., 2000). Since the
232 whole-time history of the scaled model is significantly large in this study (20 seconds), the
233 implicit integration method with 0.01 sec time increment was implemented. Run times of the
234 numerical simulations were influenced by the required information at each node point. Hence,
235 the complexity of the model and the required information were adjusted to minimise the
236 excessive run time.

237

238 **5.1.1 Structure and foundation**

239 All parts of the structure and foundation were modelled to exhibit the linear elastic material
240 behaviour in order to eliminate the influence of structural plastic deformation. This was
241 confirmed by the stiff structural systems used in this study, and the applied seismic events were
242 expected to stay within the elastic limit of the structure. Therefore, shell element type was used
243 for the floor slabs, side walls and the base. A beam element was utilised for the columns (Figure
244 4). The structural base and wall were considered as the raft foundation in experimental and
245 numerical investigations. The piles were also modelled using the beam element similar to the
246 columns. All the joints within the structure and the joints between the base and pile were rigidly
247 connected and merged as one unit for this computational study. The interaction between the
248 soil and structure for both raft and pile foundations was considered as a rough surface with
249 hard contact. This was achieved by the sand coating on the retaining walls and both raft and
250 pile foundations in experimental investigations.

251

252 **5.1.2 Soil container and materials**

253 The soil medium in the numerical modelling process was represented by nonlinear solid
254 elements. To simulate the nonlinear soil behaviour and possible shear failure, a nonlinear
255 Mohr-Coulomb model with tension cut-off (tension yield function) was adopted in the soil
256 elements (Conniff and Kioussis, 2007; Rayhani and El Nagger, 2008).

257

258 Kouroussis et al. (2009) modelled the boundary as infinite elements as shown in Figure 5(a).
259 This absorbs a part of the seismic energy to minimise lateral reflection of the seismic waves.
260 Further studies showed that this boundary condition would be suitable for the prototype model
261 rather than a scaled model tested under the laboratory conditions. Dashpot elements were used
262 in the literature to model the boundary by Hokmabadi et al. (2014). The propagating waves are
263 absorbed. The dashpot elements and any incident waves reflected back with zero energy into

264 the domain. The dashpot coefficients were determined based on the material properties of the
265 semi-infinite domain, as shown in Figure 5(b).

266

267 To ensure the boundary of the soil container was constructed with a flexible membrane in the
268 experimental investigations. The flexural container stiffened by steel rings was used to simulate
269 the adjacent soil conditions. The properties of the flexible membrane and the stiffening rings
270 were smeared, and thus a flexible plate element was used to simulate the container wall in this
271 numerical study as shown in Figure 5(c). However, the bottom surface of the container was
272 modelled as a rigid plate (this was fixed to the shaking table).

273

274 The interaction between the soil and the container is defined as a tie connection as shown in
275 Figure 5(c). The flexible wall was proposed to represent the viscous behaviour of the soil
276 container, where this wall has a tie connection with soil to ensure the flexible boundary of the
277 soil container. This boundary is tied to the soil in a manner that reduces the effects of reflection
278 by absorbing energy. The computer model of the soil container with soil is shown in Figure 6.

279

280 **5.2 Time-history analysis**

281 The dynamic problem usually can be analysed in two different ways: the response-spectrum
282 and time-history analysis methods. For large finite element analysis, it is too slow to employ
283 time-history integration of the equation of motion. Alternatively, an approximative approach
284 in the frequency domain can be used, which is called the response-spectrum analysis. However,
285 only a single value (maximum value) can be obtained from the response-spectrum analysis.
286 Furthermore, time-history analysis is the only way to incorporate nonlinear properties (soil)
287 into the analysis. In this study, the main focus was to understand the behaviour of overall
288 structural behaviour and soil-foundation-structure interaction. Therefore, the time-history
289 analysis method was adopted (Bathe, 2006; Ben, 2013).

290

291 **6. Test program, results and discussion**

292 Four time-history accelerograms generated artificially were adopted in this study. Various peak
293 ground accelerations (0.05 g, 0.1 g, 0.15 g, 0.2 g) were generated using Seismoartif software
294 for elastic spectra Type 2 with soil Type C (Eurocode 8 (2014)). The time-history events
295 (Figure 7) were applied to the shaking table in a horizontal direction. Boundary conditions are
296 symmetrical (in the z-direction) which are applied to correspond to the soil medium boundaries

297 (Figure 6). The model base is fixed in all directions except for the horizontal axis where the
298 dynamic force is applied. The experimental test series were carried out in four different stages.
299

300 **6.1 Fixed base scaled model**

301 Firstly, a scaled structural model was directly fixed on the shaking table to determine the
302 dynamic response without soil (Figure 4). Seismic responses of the fixed-base model were
303 examined subjected to four selected time-history events (Figure 7). Accelerometers and
304 displacement transducers were installed on the shaking table and the structure at levels 7, 12
305 and 17 to evaluate the model dynamic behaviour and measure the structural lateral
306 displacements and acceleration in the time domain.

307

308 As shown in Figure 4, the scaled model was numerically modelled using ABAQUS/CAE finite
309 element software. The boundary of the base slab (foundation) in the numerical model was fixed
310 in all directions except for the direction where the time-history amplifications were applied.
311 This numerical model was subjected to the same time-history events. The primary purpose of
312 this fixed based condition was to validate the scaled physical model and then to quantify the
313 behaviour structure. Moreover, these models were used to validate the structure during the
314 seismic excitations.

315

316 Experimental and numerical relative lateral displacements at different levels with time for
317 various excitations are shown in Figure 8. The relative lateral displacements were determined
318 from the relative movement of the shaking table, where the total measured deformation was
319 deduced from the base (shaking table) movement. The experimental displacements were also
320 validated using the measured accelerations, achieved by double integration of the measured
321 accelerations. The numerical simulation of the scaled model of the multi-story building
322 provides a very good correlation to the experimental results regardless of any acceleration
323 inputs. Figure 9 compares the maximum relative displacements between the experimental and
324 numerical results at different story levels (7, 12 and 17) for various seismic events. The values
325 and trend of the 3D numerical predictions are in good agreement with experimental results
326 (Caicedo, 2011). The difference in natural frequencies between experimental (hammer test)
327 and numerical data is less than 1 Hz. Moreover, there is a constant difference of 2 mm between
328 the experimental and the numerical displacements in many cases. These results validate the

329 approach and modelling techniques used in the numerical scaled model. Hence, this approach
330 can be used for the next stage of this research study.

331 **6.2 Soil container**

332 **6.2.1 Boundary effects**

333 The soil container without the structure was subjected to the same time-history events.
334 Accelerometers (ACC) at the top of the soil surface would be difficult to ensure having full
335 interaction with the soil particles due to the mobility of accelerometer mass during the dynamic
336 excitation. Therefore, three accelerometers (ACC3, ACC4 and ACC5) were inserted 100 mm
337 below the soil as shown in Figure 10. ACC2 was located at the centre of the soil mass. ACC5
338 and ACC6 were attached to the soil container boundary. ACC1 was set up on the shaking table.
339 To investigate the soil container boundary effects and ensure all the accelerometers were
340 working effectively, an amplitude (0.1 g) harmonic excitation with the frequency of 4 Hz was
341 applied to the flexible container. The results in Figure 11 show that the differences in responses
342 of ACC1-ACC6 were insignificant. The response of ACC5 and ACC6 showed a scattered
343 shape. This can be attributed to the local effect on an area close to the boundary of the soil
344 container. However, the peak amplitude remained almost the same as other accelerometers.
345 The output results showed that the flexible boundary of the soil container is functionally
346 suitable.

347

348 **6.2.2 Dynamic properties of soil**

349

350 Behaviour of sandy soil is very complex under dynamic loadings and the stiffness of soil is a
351 function of confining pressure. Therefore, those limitations need to be considered and
352 incorporated within numerical models, to minimise their effects. The limitations are:

- 353 • Difficulty in obtaining undisturbed samples of sandy soil. In practice, the soil
354 parameters are determined based on the conventional soil tests. The errors are possible
355 due to the uncertainty of the soil nonlinear behaviour.
- 356 • The stiffness of soil cannot be constant when strain increases. The degradation of shear
357 modulus with strain should be, therefore, incorporated within deformation analyses.

358

359 The stiffness of soil is associated with the density of soil. So to minimize the effect due to
360 changes in density on the sandy soil during seismic excitation, a specific sandy soil, which has
361 minimum volumetric changes (and thus the density), was selected for this study.

362

363 Dynamic properties of soil can be represented with the damping ratio and a hysteretic stress-
364 strain loop, which are essential to study the nonlinear dynamic behaviour of sand. Zeghal
365 (1995) and Turan et al. (2009) proposed that the hysteretic stress-strain loops can be derived
366 using the accelerometers' response at various depths. If the soil is idealised the shear strains,
367 shear stresses and a one-dimensional shear beam at a particular depth can be calculated using
368 the acceleration outputs at these levels.

369

370 Interpolation the measured accelerations at specific depths (e.g. ACC1 and ACC2 which are at
371 centre of the soil and the shaking table, respectively, Figure 12), the corresponding shear
372 stresses ($\tau(t)$) and shear strain value ($\gamma(t)$) in accordance with Pearson (1986) and Brennan et
373 al. (2005) can be simplified as:

374

$$375 \tau(t) = \rho d(\ddot{u}_{ACC2}(t) + \ddot{u}_{ACC1}(t))/2 \quad (2)$$

$$376 \gamma(t) = (u_{ACC2}(t) - u_{ACC1}(t))/d \quad (3)$$

377

378 where, ρ is the soil mass density, and d is the soil slice thickness, ($\ddot{u}_{ACC2}(t)$) and
379 ($\ddot{u}_{ACC1}(t)$) are the recorded time accelerations at soil centre level and the shaking table level,
380 respectively, and $u_{ACC2}(t)$ and $u_{ACC1}(t)$ are the displacements at the centre of the soil and
381 shaking table, respectively. The displacements can be derived by double integration of the
382 measured accelerations at each level. Figures 13 (a) and (b) show the recorded accelerations
383 and the derived displacements at the shaking table and the centre of the soil.

384

385 The shear modulus degradation curves of sandy soil were used to investigate the earthquake
386 site response analysis (Seed et al., 1986). Seed et al. (1986) obtained S-shape degradation
387 curves using 30 different sand types, in which a wide range of confining pressure, void ratio
388 and relative density were investigated. From equations 2 and 3, the stress-shear strain
389 relationship was calculated. Using the maximum shear strain, the damping ratio (4.5%) and
390 shear modulus (G/G_{Max}) ratio (0.85) were obtained using the relationship provided by Seed
391 et al. (1987). Using the acceleration and the displacement, the stress-strain loop was calculated
392 for a whole cycle as illustrated in Figure 14. These parameters were used for the numerical
393 simulation of the soil container.

394 In order to make sure that there are minimum changes in the volume of soil during the shaking
395 events, which significantly influences the dynamic properties of soil, the soil was placed in the
396 same way as described earlier and the density was checked before and after each shaking event.
397 It was found out that the change in density was insignificant due to the specific properties of
398 the sand.

399

400 **6.2.3 Numerical validation of the soil container**

401 The soil and the container were subjected to the nonlinear time-history dynamic analysis to
402 simulate the actual dynamic behaviour. The soil part was represented by non-linear solid
403 elements, and flexible soil container was adopted to simulate the flexible boundary condition
404 of the soil element (Figure 15). Nonlinearity of the soil medium plays a very important role in
405 the seismic behaviour of the soil-foundation-structure system (Kim and Roesset, 2004;
406 Maheshwari and Sarkar, 2011). Comparing both experimental and numerical results, it was
407 found that the results of 0.05 g and 0.1 g peak accelerations were in a good agreement.
408 However, the numerical outputs of events 0.15 g and 0.2 g were slightly over predicted in
409 comparison with the experimental results (Figure 16). Furthermore, the power spectra figure
410 shows that the experimental frequencies of all events are almost 7 Hz or lower, while the
411 numerical frequency values were around 5 Hz or lower. In addition, all the numerical outputs
412 have higher power compared to experimental outputs (Figure 16). The difference in power
413 between numerical and experimental outputs are relatively higher in for the 0.15 g and 0.2 g
414 events, compared to the 0.05 g and 0.10 g events. Furthermore, Mohr-Coulomb soil model was
415 adopted in the numerical simulation of soil. The nonlinear behaviour of soil is hard to achieve
416 by approximate linear soil simulation.

417

418 These discrepancies of both frequency and acceleration are due to experimental measurement
419 methodology and uncertain linear soil simulation. The accelerometers have a mass of 50 grams.
420 As a result of this mass, the accelerometer itself has an impact on the experimental results in
421 comparison with the numerical model outputs, which were recorded from a selected node.

422

423 **6.3 Structures supported by raft and raft-on-pile foundation**

424 The effect of soil-raft foundation-structure interactions was investigated in the third stage of
425 the study. In this series of experiments, the same instrumentation setup was used for the
426 structure and the soil container. In order to simulate all contact surfaces of the structure and the

427 soil, sand was coated on the bottom surface of the base plate and the side walls using hard glue.
428 After the soil container had been secured on the shaking table, the scaled model with raft
429 foundation was embedded within soil medium 160 mm vertically from the surface of the soil,
430 as shown in Figure 17(a).

431
432 In the case of soil-pile foundation-structure interactions, linear rigid piles were considered.
433 These were achieved by scaling the flexural rigidity (EI) of the piles according to Hokmabadi
434 et al. (2014), where various materials such as aluminium tubes, steel bars, and reinforced
435 concrete were used. However, aluminium piles have been selected using the scale factor for
436 the required stiffness and yielding stress. Pile characteristics used in this study are summarised
437 in Table 5. The model pile surface has been glued with sand particles to make rough surface
438 and to avoid the interface problem. Then, the structure with pile foundation was placed inside
439 the container as shown in Figure 17(b).

440
441 The selected time-history events (Figure 7) were applied on both raft and pile foundations
442 configurations. The densities of soil before and after the events were obtained using the same
443 procedure as before.

444
445 The experimental setup was modelled numerically as shown in Figure 18, where the structures
446 with raft and pile foundations were placed in the middle of the soil container and floating in
447 the soil. The interaction between the raft and pile foundations and soil were modelled as
448 described in section 5.1.

449
450 Figure 19 compares the maximum experimental and numerical relative displacements of the
451 structure with the raft foundation attached to the shaking table at different storey levels. An
452 average of 7 mm difference between the experimental and numerical displacements was
453 observed in many cases. This measurement method gives a reasonable deformation pattern of
454 the structure in comparison with the absolute storey deformation regardless of the occurrence
455 times recorded. It can be seen that the values and trend of the 3D numerical predictions are in
456 good agreement with experimental results. Figure 20 compares the results obtained from the
457 structure in the pile foundation. Similar trends and conclusions were derived except that the
458 average difference between the displacements was 6 mm. Furthermore, the relative lateral
459 displacements of raft foundation at higher acceleration intensities were much higher than those
460 with pile foundation.

461

462 7. Conclusions

463 This paper demonstrates the experimental and numerical investigations of the scaled model of
464 a multi-story wall-frame structure with various foundations on sandy soil. From this study, the
465 following conclusions can be derived:

466

- 467 • Scaling approach to a structural prototype was introduced, followed by building and
468 testing a scaled structural model under seismic conditions. The experimental results of
469 the scaled model showed the expected behaviour. Therefore, the results from the scaled
470 model can be used to back calculate the behaviour of the prototype. Furthermore,
471 experimental investigations of the scaled physical model demonstrated that it is more
472 convenient and economical to vary the studied parameters compared to examining a
473 full-scale structure.
- 474 • A flexible soil container was designed and built to simulate the soil boundary
475 conditions. It was found out that the flexible soil container minimises the reflected
476 waves generated during the excitations and reduces the impact of these waves on the
477 soil response. **The outcomes of this study are applicable only for structures constructed
478 on dry sand soil without the presence of ground water.**
- 479 • The scaled structural model was expected to be rigid compared to the underlying soil.
480 The experimental and numerical results showed a tilting effect on the structure and most
481 of the deformations were within the soil rather than in the structure.
- 482 • To minimise the volumetric changes of sand during the seismic excitations, sand with
483 certain grain sizes was used. The sand densities before and after the seismic excitation
484 showed that the volumetric changes of sand are negligible.
- 485 • Tie connection was used to simulate the boundary between the soil and the membrane
486 of the container. The results showed that the numerical approach represents the
487 behaviour observed in the experimental investigations.
- 488 • In all stages, the comparison between experimental and numerical outputs were in good
489 agreement. As expected the structure supported by raft foundation was more sensitive
490 compared to the structure supported by pile foundation. **The structure supported by the
491 pile-raft foundation experienced on average 30% less rocking in comparison to the
492 structure supported by the raft foundation.**

493

494 The results demonstrate that the procedure benefits from experimental feedback and provides
495 a reliable and qualitative numerical model. Consequently, the proposed numerical model of
496 raft and pile foundations is a valid and competent method of simulation with sufficient
497 accuracy. This methodology is possible to be employed for further numerical study of soil-
498 structure interaction investigations under dynamic effects.

499

500 Practising engineers can adopt this verified numerical modelling procedure to study the effect
501 of foundation considering the interface elements, boundary conditions, and soil properties.
502 Another advantage is that performing a soil-foundation-structure interaction analysis with main
503 components such as subsoil, foundation types and superstructure, can be modelled
504 simultaneously without resorting to independent calculations for superstructure and
505 substructure individually.

506

507 **8. References**

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