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

Omar S. Qaftan, Tahsin Toma-Sabbagh, Laurence Weekes, Levingshan Augusthus-Nelson

School of Computing, Science & Engineering, University of Salford, Manchester, UK

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

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

3. Structural design

This study involves soil container with structure. A detailed literature review of 1-g shaking table tests of a soil container with and without structure and foundation are summarised in Table 1. In the literature, scaled structures were physically modelled as either single-degree of freedom (SDOF) systems, lumped mass multi-degree of freedom (MDOF) systems or multi-degree of freedom scaled models of an actual structure. When it comes to the high-rise buildings, SDOF would not be suitable. Furthermore, stability of a lumped mass MDOF system is hard to achieve in an experimental investigation. Hence MDOF scaled model approach has been widely implemented in recent studies.

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

3.1 Prototype

The prototype model building used in this study consisted of a concrete wall-frame structural system with two basement floors and fifteen stories above ground level with a total height of 53 m, width of 10 m and length of 10 m. The structural form and sections were designed based on Eurocodes design guidelines (Eurocode 2, 2014; Eurocode 8, 2014; Cobb, 2014). ETABS (CSI, 2015) software was employed for analysis and design of this structure. The live load (2 KN/m^2), dead load (5.5 KN/m^2), wind load (wind speed of 45 m/s for the terrain category of 2 based on Eurocode 2 (2014)) and seismic elastic spectra (Eurocodes Soil Type C with maximum acceleration intensity of 0.2 g) were considered in the design of the structural geometries and materials. A compressive strength (f_{cu}) of 40 N/mm^2, a mass density of 2400 kg/m^3 and Young's elastic modulus of 36000 N/mm^2 were used for this concrete wall-frame structure. The final design of the structural frame and sections are illustrated in Figure 1.

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

3.2 Scaled model

3.2.1 Design

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

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

3.2.2 Physical construction of the scaled model

In order to obtain the dimensions of steel plates and tube (representing slabs and column, respectively), the expected scaled model was also analysed and designed using the ETABS software, and the dimensions were selected to meet the required natural frequency and the total mass of the scaled model as shown in Table 2. Grade 255 steel (255 N/mm^2) was adopted in all the elements of the scaled model. This was due to the fact that structural steel is flexible and constructible to the test environment, while a concrete structural model could not be constructed with the required dimensions and dynamic properties. In the scaled model, each floor is supported by vertical steel tubes (8 mm in external diameter and 1 mm in thickness) as the column elements. Dimensions of 220 × 220 × 5 mm and 200 × 200 × 2 mm steel plates were selected as the base and the typical floor slab of the scaled model, respectively. The connections between the columns and floors were provided using 4 mm diameter steel thread bars screwed by nuts on both ends of the top level and the base floor. Steel plates of 200 × 160 × 3 mm were attached vertically to the lower levels to represent the basement retaining walls as shown in Figure 1. Once all the members were assembled, the total weight of the scaled model was measured as 23.7 kg. The resonance of the as-built scaled structural model (natural frequency) was determined by hammer test, which was approximately 9 Hz. The total masses and the natural frequencies of the as-built physical scaled model are shown in Table 2.

4. Soil container design

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

The main concern of the earthquake model tests is the soil boundary effects on the soil response within the soil container. The main function of the soil container was to confine and hold the soil part in place during the dynamic excitation. The ideal soil container should minimise the boundary effect and simulate the free field soil behaviour as it exists in the prototype. The key parameter in the design of the soil container was to satisfy the dynamic shear stiffness of the soil container in the same manner as the adjacent soil deposit to achieve the real response (Hokmabadi et al., 2014).

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

4.1 Soil properties and placement

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

Dry sand with certain characteristics was used to reduce the volumetric changes during seismic excitation. The grain size distribution of the sub-rounded sand particles is shown in Figure 3. The densities of the sand according to BS 1377 (1990) shows that the maximum and minimum dry densities were 16 kN/m^3 and 14 kN/m^3, respectively. The difference between the maximum and minimum void ratios was approximately 0.12. The as-placed density of soil in the experimental investigation cannot reach maximum or minimum values. Hence, the difference in densities during the excitation is smaller than the difference between maximum and minimum densities. This aspect was observed in the experimental investigation. Thus, the changes in densities of the sandy soil has a minor effect on the soil response. The specific gravity of the selected sand was 2.68. The angle of internal friction was measured as 34° using the direct shear tests and the elastic Young's modulus was derived as 80 x 10^6 N/m^2 using triaxial test. The dilatancy of round sand particles is defined by Ryan and Polanco (2008) as:

$$w = \varnothing - 30 \tag{1}$$

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

5. Numerical modelling

5.1 Finite element modelling

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

There are mainly two analytical methods to solve the dynamic behaviour in finite element analysis namely, implicit and explicit dynamic analyses. Implicit and explicit solutions are based on Euler-time integration solution, which is the quantity calculated from the previous time step. The implicit analysis uses Newton-Raphson iterations to enforce equilibrium between the applied external forces and internal structure forces. Therefore, the implicit analysis is more accurate for the analysis of larger time history (Sun et al., 2000). Since the whole-time history of the scaled model is significantly large in this study (20 seconds), the implicit integration method with 0.01 sec time increment was implemented. Run times of the numerical simulations were influenced by the required information at each node point. Hence, the complexity of the model and the required information were adjusted to minimise the excessive run time.

5.1.1 Structure and foundation

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

5.1.2 Soil container and materials

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

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

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

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

5.2 Time-history analysis

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

6. Test program, results and discussion

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

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

6.1 Fixed base scaled model

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

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

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

6.2 Soil container

6.2.1 Boundary effects

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

6.2.2 Dynamic properties of soil

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

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

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

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

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

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

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

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

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

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

6.2.3 Numerical validation of the soil container

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

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

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

The effect of soil-raft foundation-structure interactions was investigated in the third stage of the study. In this series of experiments, the same instrumentation setup was used for the structure and the soil container. In order to simulate all contact surfaces of the structure and the soil, sand was coated on the bottom surface of the base plate and the side walls using hard glue. After the soil container had been secured on the shaking table, the scaled model with raft foundation was embedded within soil medium 160 mm vertically from the surface of the soil, as shown in Figure 17(a).

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

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

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

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

7. Conclusions

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

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

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

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

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