Response of Soil – Foundation – Structure Interaction of Tall Building (Frame - Wall) Structural System under Seismic Effect

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CERTIFICATE OF AUTHORSHIP/ORIGINALITY

I certify that the work in this thesis has not previously been submitted for a degree nor has it been submitted as part of requirements for a degree except as fully acknowledged within the text.

I also certify that the thesis has been written by me. Any help that I have received in my research work and the preparation of the thesis itself has been acknowledged. In addition, I certify that all information sources and literature used are indicated in the thesis.

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ABSTRACT

The unavailability of standards or validated analysis techniques of estimating the soil -foundation-structure interaction (SFSI) lead to either simplifying or ignoring this interaction. The structural and geotechnical engineers consider the foundation effect on the multi-story building design. Where both the structural and geotechnical analysis is usually conducted individually. The geotechnical engineer may simplify a multi-degree of freedom to a single degree of freedom oscillator, and on the other hand, structural engineers may ignore the soil-foundation-structure interaction SFSI or represent the nonlinear soil-foundation-Structure interaction with simple linear springs, where the nonlinear Interaction between the superstructure and the substructure is neglected. This study was carried out using experimental and numerical approaches to analysis the Interaction of soil foundation structures under seismic effect.

Experimental work was performed through a series of shaking table test events for different parametric studies such as building height, soil density, and foundation type under the impact of shaking waves representing the soil vibration of seismic effect.

Numerical simulation was performed using two popular software packages i.e. ABAQUS and ETABS package to solve the three-dimensional problem of soil foundation structure response under seismic effect. The results obtained from the software will then be compared with those obtained by experimental work.

Based on the literature review, the following parameters (which are believed to have an influence on the Soil Structural Interaction response) were investigated in this study:

- Building characteristics such as the height and mass,
- Soil properties including the dynamic stiffness, damping ratio, shear, angle of internal friction and shear wave velocity,
- Pile group configuration and the nonlinear interaction between piles and the Soil,
- Type of the foundation such as Raft, and Raft-Pile foundations.
- Characteristics of the input motion (earthquake type).

The main purpose of the experimental tests was to investigate the effect of the parameters on the structure and compare the outcome of those tests with the predictions from the software programme to validate the numerical model for further dynamic studies.

The experimental work was divided into four stages: Firstly, the fixed base stage. Secondly, the soil container stage. Thirdly, the soil-foundation-structure interaction (raft foundation). Fourthly, the soil-foundation-structure interaction (pile foundation).

Comparing the results of the numerical model and the experimental measurements, it can be concluded that the employed numerical model is appropriate for the simulation of the soil-foundation - structure interaction under dynamic effect. The scale models demonstrate some behaviour of the prototype in economical way without examining the prototype itself. Consequently, the proposed numerical model of raft foundation and pile foundation are valid and qualified method of simulation with sufficient accuracy which can be employed for further numerical dynamic soil-structure interaction investigations.

to consider the amplification of lateral deflections of soil-foundation -structure interactions under the seismic effect of the shear wall – columns structural system, a simplified calculation method of soil-structure interactions moment has been proposed. The proposed procedure enables structural engineers to extract the response of soil structure interaction in more reliable ways to ensure the design safety and reliability.

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- Experimental and numerical simulation of scaled model of tall multi-storey wall-frame system for seismic studies (under review)

- Soil-foundation-structure interactions of a multi-storey wall-frame structural system under seismic loading conditions (under review)

- Evolution and simplification of different parmeters on the response of soil foundation structure interaction (under review).

- The effect of soil mechanical properties on the response of tall Multistorey Structure-raft on pile foundation under dynamic excitation (proposed paper)

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LIST OF NOTATIONS

A	area
A_{loop}	area of the hysteresis loop
a	foundation width
C	cohesion
c	damping coefficient of the structure
$[C]$	damping matrix
c_h	horizontal damping coefficient of the subsoil
c_θ	rocking damping coefficient of the subsoil
c_x	
C_s	seismic design coefficient of the fixed-based structure
C_u	coefficient of uniformity
Δ	Corresponding displacement
D	damping Ratio
SD	Structure displacement
e	actual void ratio
e_{min}	minimum void ratio
e_{max}	maximum void ratio
E	modulus of elasticity (Young modulus)
E_s	soil subgrade reaction
F	force
f	natural frequency of fixed base structure
$\tilde{f}$	natural frequency of soil-structure system
f_{ck}	specified compressive strength
f_m	natural frequency of the model
f_p	natural frequency of the prototype
G	shear modulus of the soil
G_0	shear modulus of the soil at small strains
G_{max}	largest value of the shear modulus
G_{sec}	secant shear modulus
G_{tan}	tangent shear modulus
Gs	specific gravity

g	gravity
h	height of the structure
h^{θ}	lateral displacement at the top of the structure due to rotation of the base
I	moment of inertia
I_c	flexural rigidity of the building columns
i	step
i+1	step increment
K	stiffness of the structure
k_s	shear spring stiffness
k_n	normal spring stiffness
k_x	stiffness in x direction
$[K]$	stiffness matrix
m	mass of the structure
$[M]$	mass matrix
ρ	soil density
p_m	scaled model density
p_p	Prototype density
$\mathrm{R}_{\mathrm{i}}^{\mathrm{ext}}$	external force
R_i^{int}	internal force
T	soil factor
T	the natural period of the fixed-base structure
τ, τ_c, τ'_{zy}	shear stress
u	lateral displacement at the top of the structure due to structural distortion
$u_{g,}$	horizontal seismic excitation
$\widetilde{u^g}$	effective input motion
ü	acceleration
$\dot{u}$	velocity
υ	Poisson’s ratio
v_s	*Shear velocity*
$\widetilde{\omega}$	effective frequency
ω_0	the natural frequency of the fixed base structure
W	structure factor

ω	dilation angle
ζ	the equivalent viscous damping ratio
$\tilde{\tilde{\zeta}}$	effective damping ratio
Γ_n	Maimum displacment
λ	geometric scaling factor
σ_y	yield stress
ϕ	friction angle
γ_c, γ_{zy}	shear strain
$\propto$	damping control
β, γ	damping parameter
r	radius
γ	density
γ_{max}	maximum density
γ_{min}	minimum density

LIST OF ABBREVITIONS

SFSI	soil foundation structure interaction
MDOF	multi degree of freedom
SDOF	single degree of freedom
3D	three dimensions
FDM	finite difference method
FEM	finite element method
RC	reinforcement concrete
SSI	Soil structure interaction
FIM	Foundation input motion
SSPSI	soil pile structure system interaction

CHAPTER ONE - INTRODUCTION

1.1 General

Earthquake excitations may affect many multistory structures. Damages may occur due to structural faults and resonance effects or soil conditions. The particular failure can vary depending on the structural system and support, including foundations types and the soil conditions. Under the seismic effects, structures and the soil underneath are subject to seismic ground motion. This motion transfers as a motion acceleration to each part of the construction system. Construction mass and motion acceleration cause vibration within the structure and lead to partial damage or collapse of the whole structure. Numerous multistory buildings have been built in earthquake zone areas with different types of foundations. In the foundation design of the multi-storey buildings, several options are available such as shallow foundation and pile foundation. The design engineers select the appropriate foundation type to carry both gravity and earthquake loads. However, various foundations behave differently during an earthquake (Yegian et al., 2001). The response of structures under seismic effects has therefore been a major concern for design engineers around the world.

The seismic response of structures is usually determined by assuming fixed support at the structure base. This approach is acceptable when the structure is constructed on solid rock, whereas two considerations are needed when determining the seismic effects on the structures based on soft soils. Firstly, the forces on the structure originating from the free-field motion, which are generated by the response of the structure's body and the base system. Secondly, further deformations accrued within the structure as a result of the dynamic behaviour of the soil underneath. The assumption of soil influences on the structure movement and reaction of the structure influences to the soil response is referred to as the soil-structure-interaction (Kramer, 1996).

Earthquakes cause shaking of the ground and structures resting on the ground. Buildings resting on the ground experience motion at the structure base and inertial forces generated horizontally at the floor levels of structure. There are a variety of parameters that have an impact on the structure response to seismic effects such as, structure geometry, foundation types, characteristics of soil etc. When the ground motion shakes the building base, the building will swing back and forth causing differential displacements and resulting in loads transferring to the underneath and surrounding soil through the foundation which is typically a raft or pile (Bowles, 1997).

1.2 Soil-foundation-structural interaction statement & contribution to knowledge

The unavailability of standards or validated analysis techniques for estimating the soil-foundation-structure interaction (SFSI) leads to either ignoring or simplifying this interaction. The structural and geotechnical analysis is usually conducted individually. The geotechnical engineer may simplify a multi-degree of freedom to a single degree of freedom oscillator. Moreover, the structural engineers may ignore the SFSI or use simple linear springs to represent the nonlinear SFSI and neglect the nonlinear interaction between the superstructure and the substructure (Hokmabadi et al., 2013).

During earthquake excitations, the building or structure interacts with the surrounding soil. The dynamic behaviour of structure and soil should be studied at the same time when dynamic loads in a particular time act on superstructure and surrounding soil. It has been established that structures can be designed carefully and constructed safely against several seismic performance criteria to prevent collapse during earthquakes. The nature of foundation, structural system and the ground motion duration and characteristics are the primary functions of structure response (Deepa and Nandakumar, 2008).

Some simple theoretical assumptions are considered in any operation assessing the reaction of several types of foundations under seismic action. Consequently, a simplified method represents the subsoil by proposing a series of linear springs, while the superstructure is simulated as a single degree of freedom. The oscillator is adopted in the codes regulation regardless of the foundation type. Furthermore, the linear equivalent behaviour for the subsoil is selected without considering the nonlinear behaviour of soil (such as soil damping, shear modulus). The soil responses are represented directly with a constant value of stiffness and damping during the design procedure. Therefore, in the seismic design of the buildings more research on soil is required considering the influence of SFSI with a rigorous accounting of the higher modes of response and different foundation types (Yegian, Mullen and Mylonakis, 2001).

The main advantage of an experimental simulation model in geotechnical engineering under controlled conditions is to provide the opportunity for better understanding of SFSI. Moreover, it is used as a reference for numerical and empirical analysis. Shaking table tests for the multistory structures are highly in demand, where the dynamic properties of the prototype structures such as natural frequency and the number of simulated stories are required. Moreover, conducting a complete set of experimental tests in this study with different foundation types namely, raft foundation, and raft on pile foundation leads to experimentally comparable results. These results are used to determine the influence of SFSI on the superstructure supported by different types of foundations under the seismic load (Tabatabaiefar and Massumi, 2010).

Pile foundations or deep foundations are the most common foundation in civil engineering mainly supporting constructions with large loads. These types of foundations are employed to transmit the structure load into the soil layers by piles elements. Piles are mainly either bearing piles or friction piles. Bearing piles are commonly used to transfer the foundation loads from the low bearing capacity strata through the soil to the deeper soil strata with a high bearing capacity such as rock or very dense soil, where the end of bearing piles are terminated. While skin friction provides greater ability for friction piles, these piles are mostly used in cases where the high bearing capacity soil is extremely deep (Bowles, 1997). Considering the nonlinear response of the soil under earthquake motions, the foundation-structure interaction under the seismic action can be determined in a process involving inertial interactions between foundation and structure, and dynamic interactions between the foundation and the soil underneath (Tabatabaiefar and Mansoury, 2016). However, in engineering practice linear springs are used to model soil-pile interaction in simple methods such as Winkler model.

Due to the limitations of Winkler methods, the researchers utilised advanced analytical tools to perform fully-nonlinear mathematical models to study the seismic effects on the pile foundations. However, the adopted numerical models need to be verified against the experimental measurements before utilising them as a tool for nonlinear time-history of soil-foundation-structure interaction analysis. Therefore, efforts are required to develop a verified numerical modelling procedure to be capable of considering the significant aspects of SFSI analysis. Thus, this model can be used for further investigation of the influence of SFSI on the seismic response of buildings.

1.3 Aims and Objectives

This study aims to look into the influence of the foundation type and soil on the response of the regular multistory dual structural systems (frame-wall structural system) under seismic effects and to examine the structure analysis for safe, and reliable design.

The study deals with the evaluation and quantification of the effects of foundation type (raft and raft on pile foundation) on the response of structures considering SFSI, which is significantly important in the design of structures based on performance. Different types of foundation can alter the dynamic system properties such as stiffness, damping, and natural frequency. These are investigated by conducting both experimental and numerical modelling. ABAQUS and ETABS, a three-dimensional finite element program, was used for numerical modelling and examination of the influence of SFSI under seismic conditions on the response of multi-story shear wall- columns systems. Adopting the verified numerical models, a set of experimental shaking table tests were conducted to verify and validate the proposed numerical soil-structure model at the Salford University, Manchester, United Kingdom. To achieve the aims of the study, the following objectives were set:

1.3.1 Literature Review

Following the introduction, a comprehensive survey of the literature associated with the seismic soil-foundation-structure interaction (SFSI) is presented in Chapter 2. The dynamic behaviour of soils, the modelling techniques to simulate the impact of soil-foundation-structure interaction (SFSI) on the behaviour of structure, and the available building codes for seismic soil-structure interaction are presented. Furthermore, previous experimental investigations of (SFSI) are reviewed and discussed.

1.3.2 Experimental work

This part of the study comprises the following activities:

- Simulating the complex of a 3D non-linear scale structural model for experimental shaking table tests.
- Verification and calibration of the soil-foundation- structure model components for shaking table tests including structural models, foundations types, soil mix, and a soil container.
- Preparing and testing the dynamic properties of soil and container.
- Treating the dynamic soil behaviour, foundation, structure, and investigating the soil-structure interaction with seismic effects as accurately as possible.
- Conducting a series of planned experimental shaking table tests.

1.3.3 Numerical work

- Development of an enhanced nonlinear three-dimensional soil-foundation-structure model.
- Direct determination of story drifts by employing a multi-degree of freedom (MDOF) under seismic effect.
- Detailed study of the response of the regular multistory dual structural system supported by different types of foundations to the seismic events.
- Examining the adequacy of conventional design procedures excluding the influence of foundation type to guarantee the structural safety.

- Acquiring a better understanding of the fundamental parameters that affect the soil- foundation–structure interaction under seismic loads of superstructure regarding shear distribution, the rocking of the superstructure, lateral deformations and foundation depths.
- Studying and comparing the effects of the foundation type on the superstructure's seismic response about shear distribution, the rocking of structure, lateral deformations, foundation depths, and height of the structure.
- Proposing a simplified design procedure to enable structural engineers to determine the soil structure interactions for regular multi-storey (wall columns) structural system building frames utilising fixed base analysis as well as other site conditions and structural characteristics.

1.4 The thesis layout

Chapter 1 outlines an introduction to the aims and objectives and the organisation of the thesis. Chapter 2 presents a literature survey on the soil foundation structure interaction under seismic effects. The dynamic behaviour of structures was investigated, the available modelling techniques for (SFSI) simulation were discussed, and the available seismic building codes related to soil- foundation structure interaction were summarised. Furthermore, previous numerical and experimental investigations of (SFSI) are reviewed and discussed.

Chapter 3 illustrates the modelling procedure, the scaling methodology and the scaling factors utilised in the simulation of the soil container and superstructure. Furthermore, instrumentation setup and the soil container test preparation and experimental structural models are described. The proposed numerical model for soil foundation structure was verified using the laboratory shaking table tests. Finally, the influence of different foundations, structure height and soil types on the response of the superstructure were investigated.

Chapter 4 presents the three-dimensional numerical simulation of soil foundation structure by ABAQUS software. The numerical model's different components such as soil elements, structural elements, pile elements, soil container, interface elements and boundary conditions, and the dynamic loading were described.

Chapter 5 illustrates the validation of all stages of experimental shaking tests and investigates the capabilities of the numerical model in simulating soil foundation structure models. The results of the shaking table tests (reported in Chapter 3) are employed to verify and calibrate the numerical model by ABAQUS software. Accordingly, the scaled model of two basements plus fifteen-storey structure is simulated for different types of foundations, and the results are compared with the experimental measurements.

Chapter 6 investigates the different characteristics of SFSI and its impact on the response of the superstructures under seismic effects. Parametric studies of different foundation types, soil type, and structure height are conducted. For this purpose, verified numerical models of Chapter 5 were adopted in the parametric study. Results are presented and compared in terms of the maximum lateral deflection of the superstructure under the effects of ground motion, soil, foundation type, and structure height. In this chapter, a simplified procedure was proposed to calculate the soil structure interaction effects. The proposed equations were used to determine the additional moments due to SFSI effects and were applied on the foundation level. The conclusions of the current research and recommendations for further work are presented in Chapter 7, followed by references and appendices.

CHAPTER TWO - LITERATURE REVIEW

2.1 General

Geotechnical Engineering is an essential part of the earthquake engineering. Soil-foundation-structure-interaction is a complicated subject required to be analysed and examined by several experimental and numerical models. For twenty years ago, the soil-foundation-structure interaction analysis has been utilised in practice as follows. Structural engineers used to design their frames considering the structure as a fixed base. On consummation of the study, they supply the moment, shear, and reaction applied at the pedestal of the structure to the foundation design engineer to do foundation design. On the other hand, soil mechanics specialists conduct a ground investigation at the construction site studying different soil parameters. Based on various lab and field investigations, the allowable bearing capacity value of the ground is calculated. This becomes the bearing capacity input value which is utilised by the foundation design engineer. The foundation engineers used to review the soil report, find out the bearing capacity of the soil underneath, and study the recommendations of the geotechnical investigation report to obtain the nature of foundation.

Each of the above activities used to be performed individually with some interface data. While structural/foundation engineers recognise the impact of soil on foundation and structure based on the soil report, the soil bearing capacity value is only of interest to the soil mechanics specialists (Hokmabadi et al.,2016; Yegian et al., 2001). Structural failure under seismic effects can result from inadequacies of either structure or foundation or a combination of both (Figure 2-1). In this type of failure, the soil supporting the foundation plays a vital role. The foundation behaviour under seismic effects is estimated by the response of the soil deformation underneath. There are two types of ground soil response: liquefaction and amplification of the soil field motion.

Figure 2-1 Building tilted by ground failure caused by soil deformation (Taiwan earthquake, 2018)

2.2 Multi-storey buildings under seismic forces

The building behaviour under seismic excitation is a vibrational effect. An increase in the structure mass mainly has two impacts on the procedure of earthquake design. These are an increase in the force which causes crushing, or walls and columns buckling. The mass pushes a member down to move or bent out by the lateral forces. This effect is the so-called p-Δ effect. In earthquakes, mostly the vertical load causes the collapse of buildings and very rarely the buildings fall over. The ground motions and motion duration are the major concern in structure design under seismic effects. In general, tall structures have a different response to ground motion compared to low-rise structures. The inertia forces depend on the ground acceleration, building mass, and the structure's dynamic characteristics (Figure 2-2). If a structure and its foundation are constructed on stiff ground, the inertia force F can be determined by Newton's law F = Ma,

Where

M is the structure mass,

and 'a' is the acceleration.

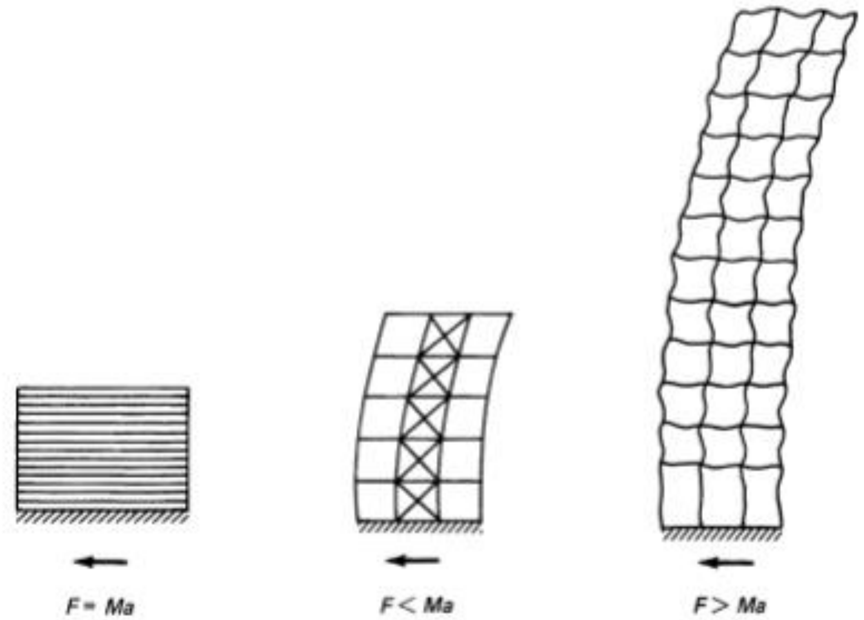

Figure 2-2 Schematic of seismic force representation(Taranath, 2009)

Structure deformation can absorb some energy. Tall buildings are more flexible than low-rise buildings. The lateral force magnitude is not only influenced by ground acceleration but also by its foundation types. In an earthquake, the building behaviour and ground motion depend on the dynamic properties of the building in the so-called response spectrum. The fundamental frequency of a tall building is dependent on its stiffness function, structure mass and damping ratio, and can depend upon the operating structural system and materials used in the construction. Within the few seconds of an earthquake starting, the ground acceleration increases up to a peak value (Stafford and Coull, 1991) (Li at el, 2002; Taranath, 2009; Clough and Penzien, 2013).

2.3 Background of Structure dynamic behaviour

For buildings with uniform stiffness and mass distribution, dynamic analyses are used to investigate the structural characteristics such as vertical distribution of lateral forces, dynamic loads resulting from torsional motions and the influence of higher modes. The available dynamic codes analysis is dependent on the static methods in which the simplified procedures proposed by single-mode response and corrections of the higher mode effects are used. This method is suitable for buildings with regular structure systems.

Dynamic analysis methods are suitable for design of buildings with irregular or unusual structural systems which have elastic response spectrum analysis and time-history analysis. The response spectrum analysis is more simple than the time-history analysis procedure. Time-history is incorporating time effects in determining the dynamic structure response. The structures response can be represented as either a simple or a complex oscillator under the ground motions excitation. The simple oscillator can be represented by a floor mass with two supporting columns and a single degree of freedom system (SDOF) (Figure 2-3), while the complex oscillator is represented by the multi-mass system with a multi degree of freedom (MDOF) system (Taranath, 2009).

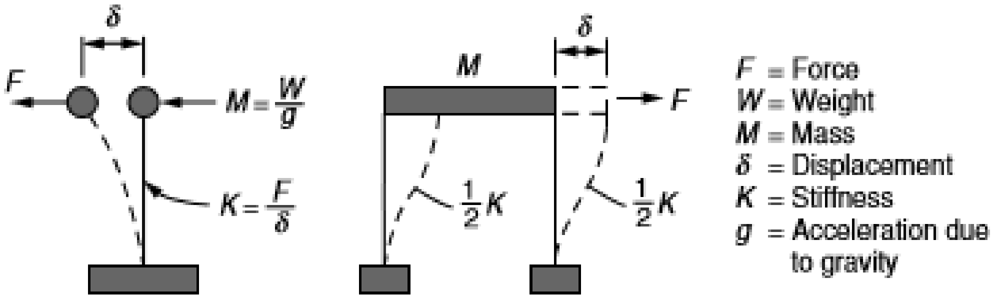

Figure 2-3 Single degree of freedom system

The floor mass M is the resultant of dividing the floor weight W of the system by the gravity acceleration g, (M = W/g). The system stiffness K is determined by dividing the applied force F by the corresponding displacement Δ. If the structure is subjected to external force and then released suddenly, the structure vibrates at a specific frequency representing the time for one complete cycle of mass movement. The relationship 2.1 gives the period T (Taranath, 2009):

$$T = 2\pi\sqrt{\frac{M}{K}} \qquad 2.1$$

The system vibrates forever in the absence of damping (Figure 2-4). In an actual structural system, the structure has a damping value depending on the structural properties. The amplitude of motion is gradually decreased until the structure stops completely as shown in (Figure 2-5).

Multi-storey buildings can be analysed by lumping masses approach at storey level intervals. During the structural vibrations, the mass of each storey deflects in one direction to another, but in high structural vibration mode, some of the storeis may move in opposite directions. Alternatively, in the fundamental mode, all floor masses deflect simultaneously in the same direction. The ideal modes number is equal to the number of structural floors. Each structure mode shape has a different natural frequency connecting the deflected masses. The multi-degree-of-freedom (MDOF) structural system can be simplified by an equivalent single mass system approach which has an equivalent value of stiffness and mass. The equivalent stiffness and mass represent the combination of storey stiffness amd mass (Figure 2-6), which is computed based on response spectra of single-storey mass systems (Taranath, 2009; Clough and Penzien, 2013) .

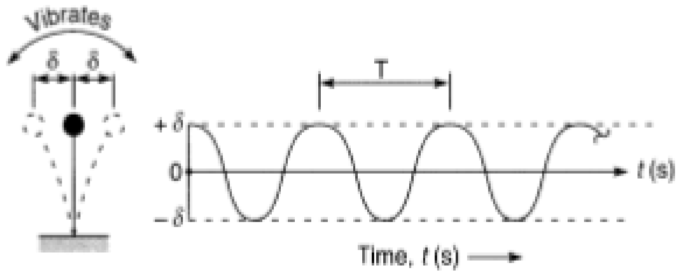

Figure 2-4 Undamped free vibrations of a single degree of freedom system (Taranath, 2009; Clough and Penzien, 2013)

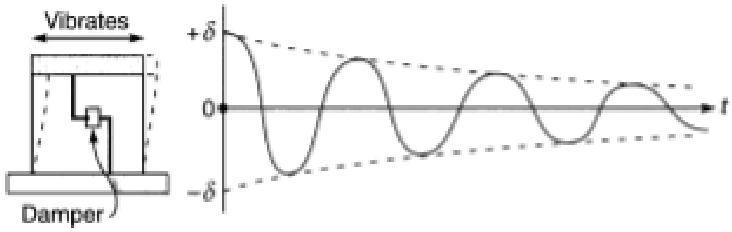

Figure 2-5 Damped free vibrations of a single degree of freedom system (Taranath, 2009; Clough and Penzien, 2013)

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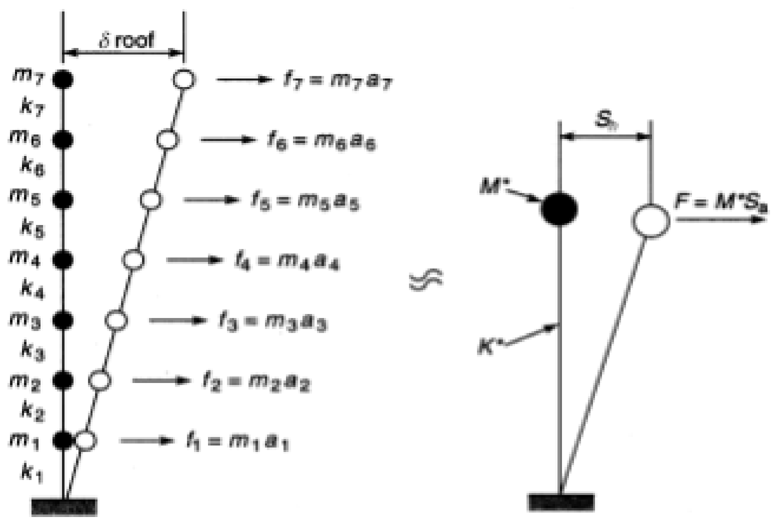


Figure 2-6 Representation of multi-mass system by a single mass system (Taranath, 2009; Clough and Penzien, 2013)

For most multi-storey structures, the nonlinear response can occur during seismic excitations, making the nonlinear analysis more suitable for building design. Despite the availability of nonlinear analysis programs often they are not used in the design practice because of complicated results which are hard to be interpreted and applied to the design criteria. Instead, based on linear elastic procedures the response spectra are used (Fan et al, 2009), (Stafford Smith at el, 1991), (Li at el, 2002), (Taranath, 2009), (Clough and Penzien, 2013).

2.4 Dynamic Behaviour of Soil

The soil response to dynamic loads is associated with the mechanical soil properties. In this section, the seismic problem of the multi-storey buildings is considered. The mechanical properties are shear wave shear modulus (G), Poisson's ratio (ν), velocity (v_s), and the damping ratio (D). The specific expression "dynamic soil properties" is used in many non-dynamic type problems. The low strain levels of soil mass are inducted under wave propagation. However, soils subjected to seismic effects may result in stability problems as considerable strain is induced. (Figure 2-7) shows the hysteresis behaviour of soil under the dynamic load. The hysteresis response of soil can be estimated by considering two important parameters of hysteresis loop shape (Kramer, 1996a). The loop inclination represents the stiffness and the tangent shear modulus varies with the dynamic force. However, the average value of the loop may be estimated by the secant shear modulus (G_{sec}).

$$G_{sec} = \frac{\tau_c}{\gamma_c} \qquad 2.2$$

Where

γ_c is the shear strain

τ_c is the shear stress.

And G_{sec}describes the general inclination of hysteresis loop.

The damping ratio is represented by the area of the hysteresis loop for the energy dissipation as follows:

$$\zeta = \frac{1}{2\pi} \frac{A_{loop}}{G_{sec}\gamma c^2} \frac{\tau_c}{\gamma_c} \qquad 2.3$$

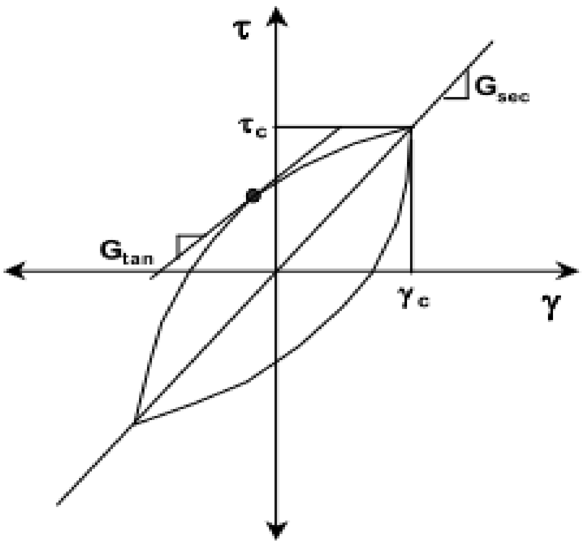


Figure 2-7 Hysteresis Loop (Tabatabaiefar and Massumi, 2010)

Due to the strain amplitude variation, different size loops are generated. The strain will increase if the secant shear decreases. Therefore, at lower strain the shear modulus (G) reaches to maximum value modulus is generated at lower shear strain, the shear on the modulus of maximum shear (G_{max}) which is called the modulus ratio. (Figure 2-8) illustrates a cyclic loop of the soil behaviour which is represented as the loss of soil element stiffness versus the strain amplitude. The damping ratio shows the material ability to dissipate the system's dynamic load. The increasing damping force causes the system energy to dissipate through the ground by friction, plastic yielding, or heat. If the resultant value is less than one, the damping ratio is defined as under damping, while for values equal to or greater than one, the damping ratio is defined as critical damping and over damped, respectively. In earthquake engineering, most problems are within the underdamped limits which are affected by the soil stiffness under the seismic effect.

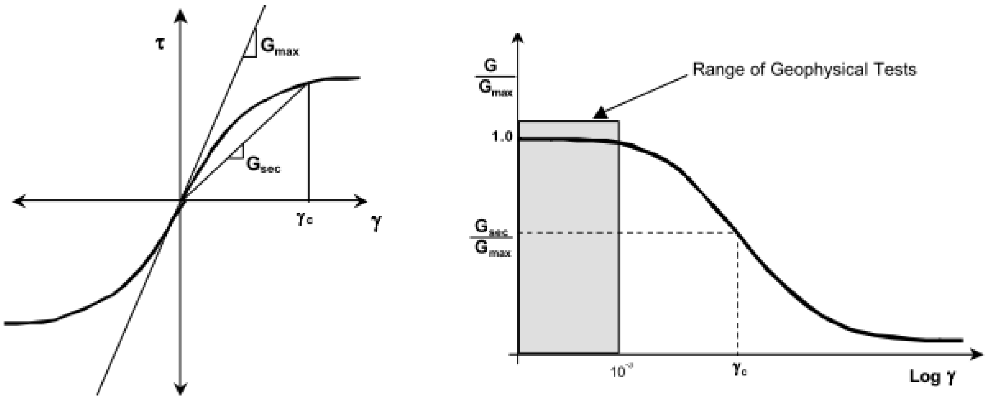


Figure 2-8 Stress-strain curve with a variation of shear modulus and modulus reduction curve (Tabatabaiefar and Massumi, 2010)

The stress strain behaviour of cyclically loaded soil is complicated, and the geotechnical engineers recognise that this behaviour is challenging to simulate accurately using simple models. The simplicity and accuracy of this behaviour depend on many factors in the proposed model. In the methods involving the soil physical model the indication of the low-strain is based on the equivalent linear model approach. This method is simple and commonly used in a dynamic model. However, representation of many soil properties under dynamic force is insufficient.

Shear wave velocity (v_s) is used as a parameter for characterisation of shallow soil geophysical models in order to determine the soil shear modulus. The importance of the shear wave velocity is that the particle in motion travels perpendicular to the direction of wave propagation. Furthermore, a shear wave is able to measure the shear properties of the soil skeleton irrespective of fluids because the shear wave flows through the solid particles only, while fluids cannot take shear. Maximum shear modulus (G_{max}) is determined by the simple elastic relationship based on the shear wave velocity

$$G_{max} = \rho \cdot vs^2 \qquad 2.4$$

Where

ρ is the soil mass

and vs is the shear velocity.

The dynamic shear modulus is estimated by advanced correlations based on the standard penetration test, Atterberg limits and grain size distributions (Vucetic and Dobry, 1991), (Luna and Jadi, 2000). The shear modulus is used to conduct advanced soil modelling to represent the dynamic response of the soil-foundation-structure system. Shear modulus at low-strain levels is measured by geophysical techniques utilised to measure the parameters of the stiffness matrices which are used as input for finite element analysis of soil foundation model under seismic effect.

2.5 Dynamic behaviour of Foundation

The stiffness of foundation elements has an impact on the response to soil-foundation-structure interactions subjected to a dynamic load. The deviation of foundation motions from free-field motions is based on the foundation stiffness. Variable ground motions within the building cause one of these deviations due to the stiffness and strength of the foundation system. Another cause of foundation motions deviation is the embedment effects, in which the level of foundation motions are reduced because of the reduction of ground motion with depth below the level of the free ground surface. For the foundations supported by piles elements, the piles interact with wave propagation at the foundation level.

2.5.1 Embedded raft foundation

In the structures with basement, the base slab is embedded under the ground level, so the foundation motions are reduced as a result of the reduction in ground motion with depth below the ground level. The available methods of analysis are based on the application of rigid cylinder footing embedded in a uniform soil with an infinite

thickness or half space thickness. Kausel et al. (1978) and Day (1978) described analytical solutions of foundation input motions at the base of the cylinder of embedded foundation as a function of ground motion. Structure rocking is also introduced as a result of differential displacements occurring over their embedded depth level.
In (Figure 2-9), D is the footing depth, L is the foundation base diameter, and (vs) is the shear wave velocity.

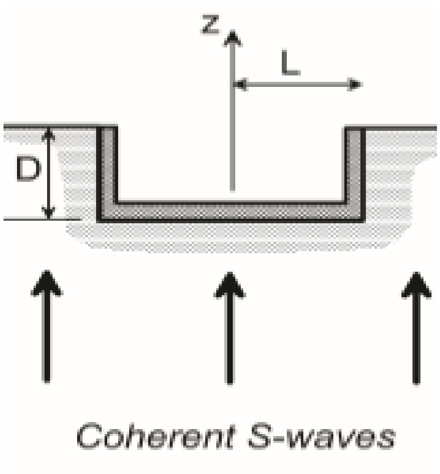


Figure 2-9 Embedded raft foundation (Stewart, 1999)

Stewart, (1999) reviewed the rigid cylinder model predictions for a structure with embedded base level. They concluded that there is a dynamic significance interaction of soil structure system as a result of embedded base effects. In general, the results illustrate the reduction of ground motions at the foundation level relative to the free field motions.

2.5.2 Pile foundation

In the building with foundations supported by piles, the soil-structure dynamic interaction is complicated. This interaction is a result of pile influence on wave propagation below the foundation level and also forming a gap by the potential of soil to settle away from the pile within the soil-structure system. It is a complex interaction problem, and well-calibrated engineering models are not available. Berones and Whitman (1982); Barghouthi (1984); Mamoon and Banerjee (1990); Fan et al. (2009), Kaynia and Novak (1992) ; Nikolaou et al. (2001), described the vertical piles and pile groups in elastic soil under dynamic loads. These studies do not incorporate the effects adequately. Kim and Stewart (2003) concluded that there are variations between motions measured at the base level and free-field ground motions. Kim and Stewart (2003) proposed solutions for the interaction problem based on varying flexural rigidity.

2.6 Concept of Soil-Foundation-Structure Interaction (SFSI)

The soil-structure interaction concept was developed in early 20th century with advances in SSI analysis methods in the mid-20th century. Kausel (2010) described SSI as the static and dynamic phenomena of a compliant soil and a super-structure. Both the structure and the soil through its foundation develop reaction to the seismic loading due to the dynamic requirements at their interface. This reaction is ultimately changing the structure and soil response, and is known as soil-structure-interaction effect of structure response in comparison with structure supported by fixed base under seismic effect (Wolf, 1985; Mylonakis et al., 2000; Shakib et al., 2004; Pitilakis et al., 2008). The fixed-base structural response is commonly used in the dynamic analysis of conventional building structures. It is recommended that SSI effects must be studied for relatively soft soils or structures with a high aspect ratio (tall building in comparison with its width). The soil structure interaction can be ignored when considering structures founded on very stiff soils or rock (BSI, 2008b).

Incorporation of foundation and structure interactions in equations governing the motion is relatively complicated. The right-hand side of the dynamic equation of motion of the soil-structure system equation 2.5 consists of a combination of different matrices corresponding to the soil, foundation and the structure. This combination makes the equation mathematically sophisticated to be solved by conventional methods in which the whole system of soil foundation structure is modelled numerically in a single step.

The soil structure system dynamic equation can be written as follows:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -[M]\{1\}\ddot{u}_g + \{F_v\} \qquad 2.5$$

where,

[M], [C] and [K] are the structure mass, damping, and stiffness matrices, respectively.

$\{\ddot{u}\}$, $\{\dot{u}\}$and $\{u\}$, are the node accelerations, velocities and displacements of the structure which are relative to the underlying soil foundation, respectively.

$\{\ddot{u}g\}$ is ground acceleration,

and {Fv} is the force vector corresponding to the viscous boundaries.

Zhang and Wolf (1998) indicated that a simple analysis is adequate to demonstrate the significant effects of soil-structure interaction considering the structure as a simple SDOF (single degree of freedom) system characterised by mass (M), stiffness (K), and damping coefficient (c). Furthermore, the soil is assumed rigid at the base considering the soil as a hard deposit. Therefore, the natural frequency by this assumption is a fixed base system, and it only depends on the structure stiffness and mass and can be determined as:

$$\boldsymbol{\omega_o} = \sqrt{\frac{\boldsymbol{k}}{\boldsymbol{m}}} \qquad 2.6$$

The equivalent viscous damping ratio (ζ) can be calculated using:

$$\zeta = \frac{\boldsymbol{c\omega_o}}{\boldsymbol{2k}} \qquad 2.7$$

As per Zhang and Wolf (1998) this indicates that the soil structure system is represented by a simple dynamic model. In this system, the foundation can translate and rotate. This system consists of the rigid bar with the horizontal and rocking springs

(Figure 2-10)

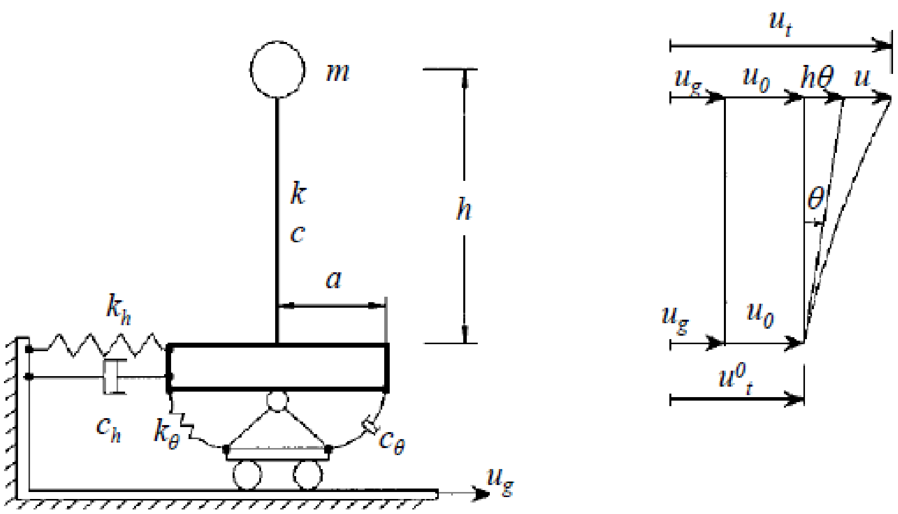


Figure 2-10 Coupled dynamic model of structure and soil for horizontal and rocking motions proposed by Zhang and Wolf (1998)

In (Figure 2-10), (h) is the height, (k) is the spring stiffness coefficient, (m) is the structure mass, (r_o) is the radius of the base.

This system indicates that the main effects of soil-structure interaction for a horizontal excitation are lateral displacement (u) at the top of the structure and the lateral displacement (h^θ) due to the foundation rotation.

C is the damping coefficient. Zhang and Wolf (1998) explained that the coupled system can be replaced by an equivalent one degree of freedom system (Figure 2-11).

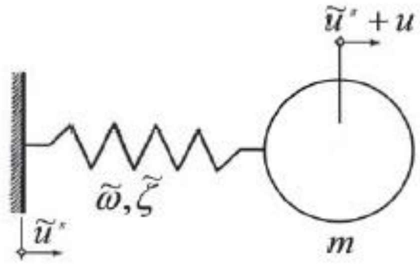


Figure 2-11 Equivalent one degree of freedom system presented by Zhang and Wolf (1998)

Where

$\tilde{\omega}$, $\tilde{\zeta}$ and $\widetilde{u^{g}}$ are the effective frequency , effective damping ratio and effective input motion, recpectivly.

The effect of soil-foundation-structure interaction on the total response of the structure is simplified as an SDOF model which is subjected to an arbitrary input motion. For simplicity, foundation stiffness and damping coefficients are assumed to be frequency independent and calculated based on equations 2.8, 2.9, 2.10, 2.11 as suggested by Vucetic and Dobry (1991).

$$k_x = \frac{8Gr}{2-v} \quad 2.8$$

$$c_x = \frac{4.6}{2-v} . \rho v_s r^2 \quad 2.9$$

$$k_\theta = \frac{8Gr^8}{3(1-v)} \;; \quad 2.10$$

$$c_\theta = \frac{0.4}{1-v} \; \rho v_s r^4 \quad 2.11$$

(Figure 2-12) presents the maximum response value of the structure considering the dynamic behaviour of soil-structure system .

Referring to (Figure 2-12) (a), it is evident that the soil-structure interaction tends to reduce the demand (base shear) of the structure. However, as shown in (Figure 2-12) (b), the soil-structure interaction increases the overall displacement of the structure due to translation and the foundation rotation (Han and Cathro, 1997). Accordingly, considering the soil-structure interaction effect can be necessary for tall, slender structures that may be affected when relative displacements become large (Kramer, 1996). Moreover, any increase in the total deformation of the structure influences the total stability of the structure.

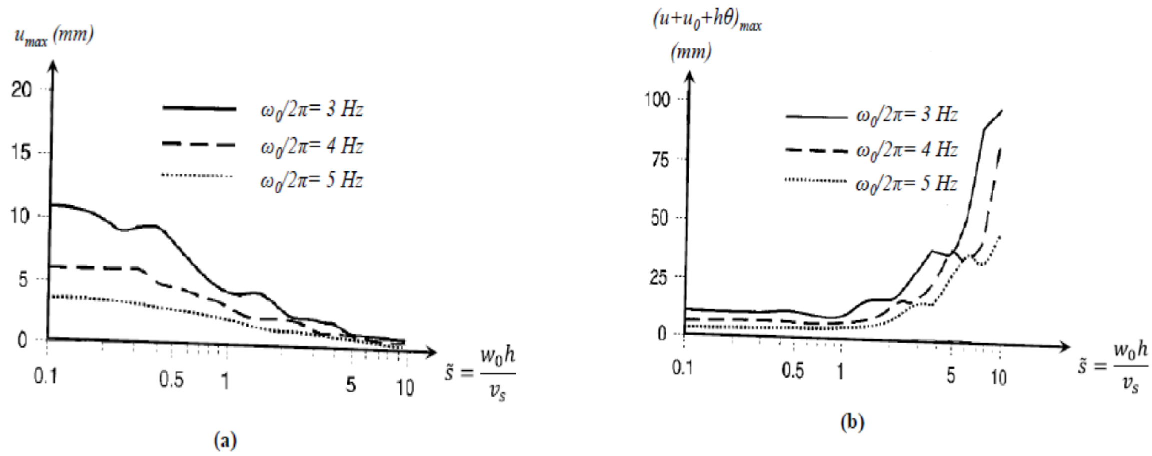


Figure 2-12 Response of the equivalent soil-structure system: (a) maximum structure demand, (b) maximum total displacement of the structure relative to the free field ground motion (Wolf and Obernhuber, 1985)

2.7 Available modelling methods for soil-foundation-structure Interaction

The proper modelling of soil medium is the essential stage in SFSI analysis.

Soil medium is commonly modelled and represented by using three main methods:

- Winkler model (spring model)

According to Bowles Joseph (1996), the Winkler's theory assumes that each layer of soil responds independently to the adjacent layers where the soil behaviour is represented by dashpots and springs. The subsoil is simulated by linear spring (Figure 2-13). The pressure-deflection relation is given by:

$p = K\Delta$ 2.12

where p is the applied pressure, K is the coefficient of subgrade reaction, and Δ is the deflection.

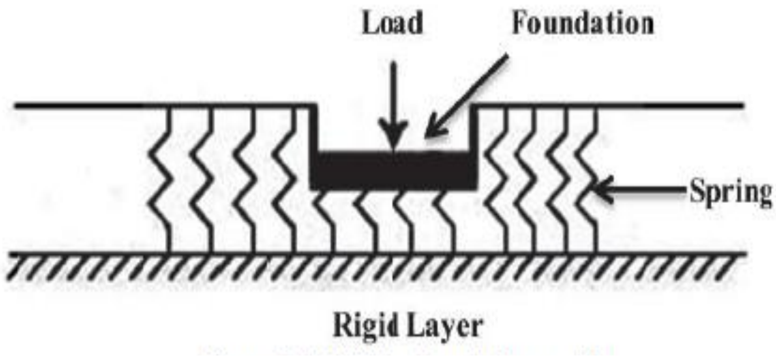


Figure 2-13 Winkler foundation model (Bowles, 1996)

- Lumped parameter on elastic half-space

In this method, three translational springs and three rotational springs are attached to three perpendicular axes for each base of the same structure (Figure 2-14). In this method, the spring's stiffness is dependent on the structure frequency, especially when the foundation is extended and resting on the soft soil. The damping coefficients are proportional to soil shear wave velocity and foundation areas (Zhang and Wolf, 1998), and is given as in the following equations:

$c = \rho . vs . Ao$ 2.13

where

c is the damping coefficient,

ρ is the soil mass,

vs is soil shear wave velocity

and A_o is the foundation area.

Tabatabaiefar, (2012) concluded that this method cannot deal accurately with geometric and material nonlinearity, hence nonlinear response modelling of both soil and structure becomes complex and more advanced modelling approaches would be required.

Also, they mentioned that with the increasing availability of powerful computers and the wider application of numerical methods compared to analytical approaches, the use of numerical methods has become a common means of modelling such complex interactive behaviour.

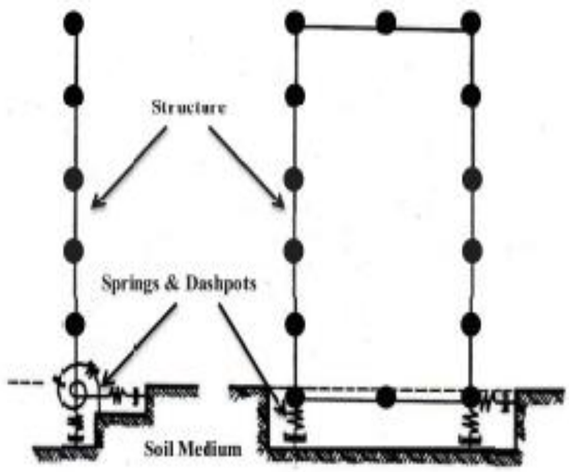


Figure 2-14 Soil model in lumped parameter methods (Bowles Joseph, 1996)

- Numerical methods.

The advantages of powerful computers have significantly changed computational aspects. The finite element analysis method (FEM) or finite difference analysis method (FDM) has become more popular for studying complex behaviours. Both methods are adopted by numerical models to produce a set of mathematical equations which are identical for the two solution methods. According to Bowles, (1996), numerical techniques can incorporate the effects of material nonlinearity, material condition, radiation in damping and the structure geometry in dynamic soil-foundation-structure interaction analysis.

2.8 Free Field Ground Motion

In the practice of earthquake engineering, one of the most critical problems is the methodology of ground motion determination. This determination is based on the equivalent linear approximation, which is performed through 1g site response analysis to the dynamic soil response (Schnabel et al., 1972). Although the available method is simplifying the nonlinear soil behaviour, it does not consider many characteristics of ground motion such as the soil deformation. In most of the large earthquakes worldwide, the non-linear soil response is recorded and the site-specific ground motion is affected significantly. Therefore, it is necassary to utilise a proper method to describe the soil realistically. Several research numerical codes are capable of performing non-linear soil response analysis.

The analysis is commonly carried out in the time domain with the non-linear soil response. It is possible to simulate a non-linear constitutive model ranging from a simple elastic-perfectly plastic model like Mohr-Coulomb model to a more complicated model that accounts for large strains and liquifaction (Karatzetzou et al., 2014)

During an earthquake, different types of seismic waves are propagated (Figure 2-15). The free field surface motions are acquired when the seismic waves reach the ground surface in the absence of any structure. If the seismic waves reach the construction surroundings, then the soil foundation structure interaction (SFSI) would take place. Many parameters are involved in the (SFSI) action.

The interaction between the relative rigidity of foundation and the surrounding soil changes the acceleration amplitude, the frequency content and the duration of motions recorded at foundation level.

Furthermore, the vibration of the superstructure propagates energy back into the foundation and the surrounding soil.

This energy can change the characteristics of motion recorded at the foundation level. The Free field ground motion is changing with the surrounding soil motion due to complex interactions of SFSI system.

The phenomenon described in this section called SFSI will be used in this research to highlight the importance of foundation and soil conditions in earthquake engineering.

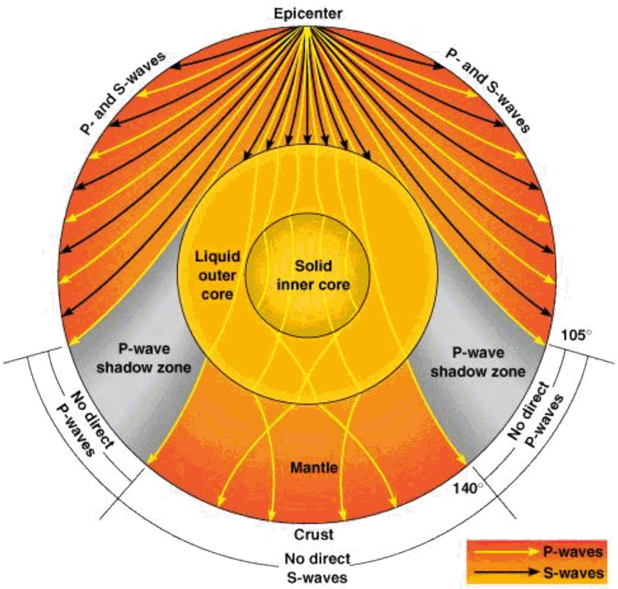


Figure 2-15 Common seismological terms used for evaluation of an earthquake for a given site b (Chowdhury and Dasgupta (2008)

Ground responses are used to indicate the free ground motion (Zhang and Wolf, 1998; Chowdhury and Dasgupta, 2008; Kramer, 1996). There are mainly four types of stress waves propagated in the soil medium ((Figure 2-16) that are of interest to the civil engineers. These waves are as follows:

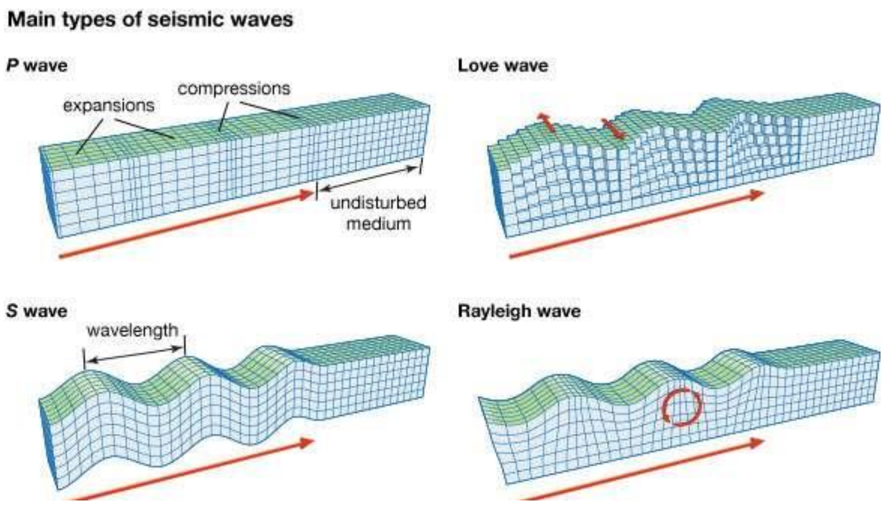


Figure 2-16 Types of seismic waves (Chowdhury and Dasgupta, 2008)

a) P- waves (Body waves)

P-waves are faster than other wave types. They can move through both soil and water. In the earthquake, the shear waves initially arrive producing longitudinal extension and compression within the soil medium. However, soils have the ability to resist the compression and dilation effects. P-waves have insignificant impact on ground distortion.

b) S-waves (Shear waves)

S-waves are slower than p-waves and move in soil medium only, while the soil resistance to shear deformation is weak. These waves result in maximum damage to the ground level during earthquakes. S-waves are known as shear waves, which cause the shear deformation within the soil medium.

c) L-waves (Love waves)

The L-waves are similar to s-waves. These waves produce transverse shear deformation through into the ground level and cause an impact on a bearing of elastic half-space overlain by finite elastic layer.

d) R-waves (Surface waves)

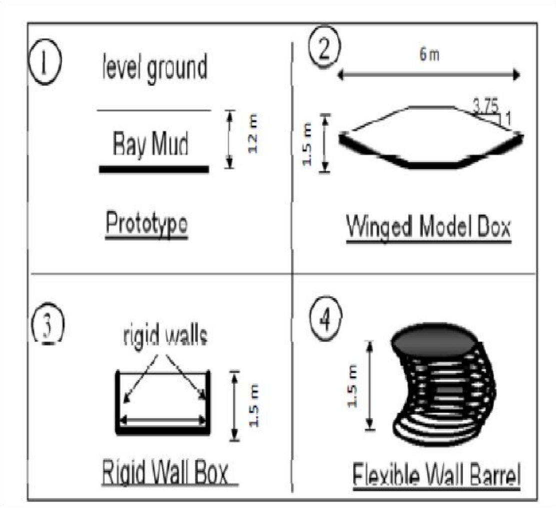
R-waves are surface waves.They create the ripple on the ground surface. These waves create vertical and horizontal movement. The waves travel far from the earthquake source and an amount of energy dissipate within the soil medium. They are an important aspect of the foundation response study, supporting the earthquake force generated and transmitted through the ground.

2.9 Simulation of Soil Boundary Condition for Soil Container

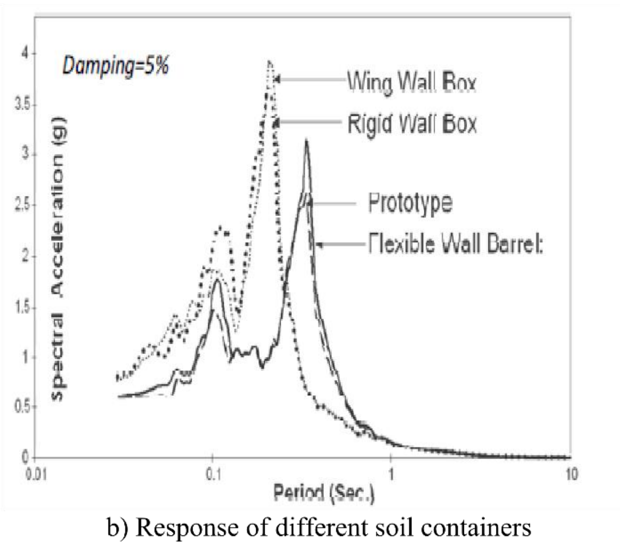
When conducting the earthquake model tests, the major concerns in experimental dynamic model tests are the artificial boundaries effect on the response of the soil structure model. The soil container is used mainly to hold and confine the soil in place during dynamic excitation. The soil container is simulating the soil behaviour of the free field soil same as it exists in the prototype.

To achieve the real prototype soil response, the critical parameter in designing the soil container is the reduction of the soil container boundary to satisfy the same response of dynamic shear stiffness for both the soil within the soil container and the adjacent soil deposit (Hokmabadi et al., 2014a).

There are mainly two types of containers, namely laminar container and plastic barrel that have been utilised for dynamic study in the literature. The laminar soil container consists of a rectangular hollow section made by aluminium frames. Rubber layers separate those frames. The function of aluminium frames is to provide lateral confinement of soil, while the function of rubber layer is to allow the soil shear deformation (Prasad et al., 2004; Meymand et al., 2000; Hokmabadi et al., 2014b).



a) Types of Soil Container



b) Response of different soil containers

Figure 2-17 Comparison of different types of soil container by Moss et al., (2010)

The main part of flexible barrel is the flexible membrane wall with stiffening rings, which represents the response of free field site under seismic effect during shaking table test (Maymand et al., 2000). Furthermore, Maymand et al., (2000) considered and compared three different types of soil containers (rigid, wing and flexible barrel containers) in his numerical study, where 12.19 m deep deposit of San Francisco Bay mud was used as a soil case study sample. The results showed that the flexible wall container precisely simulates the soil prototype while the rigid and wing wall containers do not replicate the behaviour of soil under dynamic conditions (Figure 2-17). To validate the numerical prediction, Maymand et al., (2000) tested the plastic barrel experimentally on a shaking table. Also, Crosariol (2010) and Moss et al. (2011) tested both flexible barrel and laminar containers, where the flexible barrel container provided the best response. Furthermore, the laminar container is complicated and expensive to construct. Therefore, flexible container with stiffening rings was adopted in this study. Moss et al. (2011) drew two conclusions. Firstly, the flexible barrel container and the relevant constructional details should be adequately conducted to minimise the box effect. Secondly, the container diameter should be five times the structure width. Hence, the dimensions of the container were selected as 1m diameter and 1m depth. The flexible container consists of 5 mm membrane cylinder wall supported individually by stiffener strips. The top part of the container was supported by lifting hooks from an overhead crane. The bottom base was set on the shaking.

2.10 Building code recommendation for Soil Foundation structure Interaction (SFSI)

The international seismic design codes investigated and incorporated the simplified analysis methods of soil-structure interaction. They mentioned that the site-specific studies are required for soft soils under seismic effect. Based on the design codes, the structure’s dynamic analysis founded on soft soil deposits are required, and the site conditions are needed to be considered carefully. The site effect refers to the scattering and diffraction of incident waves by the soil layers overlaying the bedrock which are reflected in the values of seismic design coefficients. Soil-foundation-structure interaction refers to the relationship between the characteristics of both the structure and the soil stratum, and one of the following methods usually presents it:

Modification of dynamic properties of the structure; or Modelling the subsoil with springs and dashpots (Lumped Parameter method)

BSSC (1997) recommendations include a procedure with details to incorporate the soil-structure interaction effects in the seismic design to determine the applied earthquake forces and estimate the lateral structure deflections. These soil-structure interactions have a defective impact on the base shear force applied on the structure, and consequently overturning moments can either increase or reduce the lateral structure deflections. The guidelines of NEHRP (2003) for new buildings to be designed are based on structure capacity and seismic demand. Seismic demand is a function of the base shear force which mainly depends on the equal mass first mode acceleration of response spectra. Inertial interaction effects are calculated by analysis of a period lengthening ratio and damping factor. These effects modify the value of base shear and lateral deflections of the structure. BSSC (1997) allows for up to 30% reduction in base shear due to soil structure interaction. The modified base shear value under soil-structure interaction influence ($\tilde{v}$), the ratio of modified base shear to the base shear of the fixed-base structure ($\tilde{v}$ / v) as well as the structural height (h), and the rocking stiffness of the subsoil foundation are employed by the code to determine the modified lateral deflections of the structure due to SSI.

International Building Code (IBC) 2012 provides a guidance to design of foundation of structure located in high-risk seismic zones, the capacity of the foundation subjected to the base shear and moments transmitted to the foundation level from the superstructure, and the superstructure to foundation adequate connections. Chapter 16 of the IBC (Structural Design) provides both time history analyses and response spectrum for earthquake design. However, there are no methods provided to calculate the soil-structure interaction in either method. The current IBC requirements call for the use of the fundamental vibration period which depends on a building’s vibration period on a fixed base and a period lengthening ratio. The period lengthening ratio depends on the lateral stiffness and height of the building as well as horizontal translational and rotational stiffness of the soil. This ratio is never less than one since flexibility always increases the period. Compared to the simple fixed base case, the modified system now takes into account a lengthened period and increased damping.

The 2010 National Building Code of Canada (Mitchell et al., 2010), presented that the effects of soil-structure interaction on the seismic response of most buildings are favourable, and thus it is considered to be conservative to ignore it. Therefore, the seismic provisions of the proposed NBCC (2010) recommend performing soil-structure interaction analysis for alternative structures only. Eurocode 8, (Code, 2005) Design of Structures for Earthquake Resistance, highlights that soil-structure interaction effects are required to be considered in the design of the structures based on the followings:

- Structures sensitive to P-Δ effects
- Massive structures
- Slender, tall structures (slender), and
- Structures supported by soft soil (Vs< 100 m/s)

For the mentioned structures, based on Lumped Parameter method, appropriate spring and dashpot coefficients are proposed for different subsoil conditions.

According to Amirsardari et al. (2014), Earthquake Actions in Australia does not include the soil-structure interaction effects in the structure design under seismic effect. Consequently, designers of the structure are not able to include those significant implications in the analysis and design procedure.

Using alternative design methods to consider the soil-foundation-structure interaction is allowed by the seismic design codes based on seismic requirements with the local authorities approval.

For the ground motions of some seismic regions such as Japan, China, New Zealand, Australia and Indonesia most probably the lateral resisting systems design of building frames is critical. Therefore, well considered and developed seismic design procedures to incorporate the SFSI effects in the seismic design of building structures are highly required.

2.11 Soil-Foundation-Structure Interaction (SFSI)

Recent improvements in seismological source modelling led to significant advances in estimation procedures for the effects of soil–foundation-structure interaction under seismic on structural design. Estimation of effects of earthquake motions load on the constructions is the most critical phase of structures engineering design. When the structure is built on the solid rock and affected by seismic actions, the high stiffness of the rock forces the motion of rock to be almost close to the free-field motion. Therefore, for the analysis purposes, structures constructed on the solid rock are assumed to be fixed base structures. On the other hand, if the same structure is founded on soft soil, it would respond differently from solid rock. It is obvious that the dynamic structure responds with an additional deformation due to the soft soil deformation. The soil reaction influences the motion of the structure including a different type of foundation and vice versa. This is referred to as the soil –foundation- structure Interaction (SFSI).

2.11.1 Soil Structure Interaction under Seismic effect (Theoretical Studies)

The unavailability of standards and validated analytical techniques for estimating the soil-foundation-structure interaction (SFSI) leads to either simplifying or ignoring the interaction (Tabatabaiefar, 2012; Massimino and Maugeri, 2013). Hence, the structural and geotechnical aspects of the foundations are analysed individually when it comes to seismic studies. The motions of soil influence the structural response, which is referred to as soil-structure interaction (Kramer, 1996). Geotechnical engineers may simplify a multi-degree of freedom to a single-degree of freedom oscillator, and on the other hand, 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).

The mutual behaviour between a foundation and building structure is highly interactive and is mainly governed by the prevailing ground conditions, the type of superstructure, the foundation type, the magnitude and distribution of the building loads, and the seismic excitations (Sinn et al., 1995). It was shown that the foundation on flexible soil may significantly increase the overall displacement of the superstructure compared to the fixed foundation (Hokamabadi et al., 2014a; Guin and Banerjee, 1998; Han, 2002). This increase in total deformation may lead to structural instability due to the secondary moment at the base (Ma et al., 2009). Hence, the foundation and superstructure design of high-rise buildings should be considered as a performance-based soil-structure interaction (SSI) issue and not limited to traditional empirically based design methods such as a bearing capacity approach with an applied factor of safety (Poulos et al., 2016). Therefore, the process of designing high-rise buildings has changed over the past years. In most recent years, it is not unusual to model full three-dimensional finite element models of the buildings without considering the effect of soil-foundation - structure interaction (Hallebrand et al., 2016).

Hoshiya and Ishii (l983) uitilised a stochastic model to estimate the dynamic behaviour of rectangular foundations embedded in soil. This type of foundation was subjected to the random vibration theory. The dynamic formula adopted in the study was based on the ground motions statistical correlation . In the stochastic model under study, the earthquake was recorded at a large scale model of foundation and ground tank. This model was used as an example to investigate the deep and shallow embedded foundations. It was observed that the foundation base slab is relatively stiff in comparison with the soil stiffness. The dynamic interaction of soil and foundation affects the slab like a low pass filter for ground motions.Veletsos and Prasad (1989) studied the soil-structure interaction of a seismically excited structure and considered the effects of inertial interaction and dynamic response. The studied structure was a linear structure supported by a circular raft foundation. The structure assumed to have single and torsional and lateral degree of freedom. The structural response of corresponding structural deformations together with the foundation input motion were measured at the peak values of the lateral deflection and torsional components. It was observed that kinematic and inertial interaction have a significant effect on the response of structural systems in high-frequency spectral regions.They also reported an increase in the corresponding response of tall structures when there is increased inertial interaction in the high-frequency values of the response spectrum, while the inertial interaction effects were insignificant for low-frequency structures.

Guin and Banerjee (1998) proposed a procedure to evolute the dynamic interaction of soil-pile foundation-structure system . A generalised formulation of finite element boundary was used to simulate the entire model.The formulation was conducted in the frequency domain. The excitation input was defined as a rock outcrop motion which was propagating S waves vertically. The linear dynamic analysis was performed on two cases. One was a multi-storey structure, while the other was a bridge. It was observed that soil-structure interaction has a significant impact on the structural system behaviour under seismic effects. Spyrakos and Xu (2003) considered the response of a large flexible strip foundation under seismic effects. The strip foundation was embedded in layered soils during the seismic excitation. A finite element formulation modelled the foundation. The modelling difficulty was the soil boundary element. The soil element was modelled as an infinitely extended boundary element formulation. The soil-structure system response was investigated, and the boundary effect was studied.

Wegner et al. (2005) proposed a numerical procedure to determine the dynamic interaction of soil-structure. Scaled boundary finite element was adopted in the modelling of the unbounded soil, while the standard finite element method was used in modelling of the superstructure. The dynamic response of tall buildings with multi-level basements under the effect of dynamic excitations was investigated. P, SV and SH waves at different angles have been included in this study.

Takewaki and Kishida (2005) proposed an analysis method for pile-group effects on the building response under the dynamic effects to study the building stiffness and strength with pile foundations. A dynamic Winkler type was used to simulate the soil element and pile within soil pile structure system. The effect of pile group was accounted for by considering the influence of coefficients defined for estimation of the pile-head bending moments and the storey drifts. It was found that the pile group effect increased the bending moments applied at the pile head and reduced the storey drift of buildings.

Carbonari et al. (2011) considered the soil-structure interaction response of wall–frame structures supported on pile foundations by the linear approach. In this approach, a linear finite element procedure for a complete dynamic analysis was developed to investigate the soil-pile interaction and radiation damping in the frequency domain. Three types of soil profiles were studied together with a real recorded earthquake as input motions. The response of parameters to the effect on structure response or deformation like deflections, inter-storey drifts, accelerations and stress resultants was evaluated. The output results were compared with those obtained from the fixed base model. It was concluded that performing complete soil-structure interaction analysis is a more reliable evaluation compared to actual response of prototype system.

Galal and Naimi (2008) conducted a comprehensive numerical study of a multi-storey structure with 20 stories for soil-structure interaction under the seismic effects resting on three categories of site classes (IBC 2009), category B, C and D which are categorised based on shear wave velocity. Based on the output results, when the supporting soil is rock or very dense soil the structure can be assumed as a fixed base. For the structures constructed on the soft soils with shear wave velocity less than 600 m/sec site classes E, D and lower limit of C (360 m/s < v_s < 600 m/s) were considered, where the structure deformation has a significant difference in comparison with fixed base structure.

The objective of Stewart et al. (1998) research was to investigate a simple procedure for considering the influence of the SSI in the fundamental frequency of buildings. Analyses were conducted by Stewart et al. (1998) for both one-storey and multi-storey buildings with different soil conditions. This study led to comprehensive charts giving the fundamental frequency of a wide range of buildings with regards to the relative soil-structure stiffness. According to Stewart et al. (1998) research, Prakash and Kumar (1998) denoted that the fundamental natural period of a soil-structure system reduces nonlinearly with the increase in the soil shear modulus. The effects of considering the nonlinear behaviour of soil on the natural period response of structures depends on the level of strains in the soil. The higher the strain in the base soil, the higher the effect of soil nonlinearity. Kumar and Prakash (1998) utilised the factors mentioned above (natural period and damping) to derive flexible base fundamental-mode parameters, which are used in response-based approaches for evaluation of the base shear forces and deformations in structures.

El Ganainy and El Naggar (2009) considered the seismic behaviour of a multi-storey structure constructed on subsoil classes C (360 m/s < v_s < 750 m/s) and E (v_s <180 m/s) by IBC2000 under the effect of soil-structure interaction response. They concluded that structural deformations of the construction resulted due to the effect of soil-structure interaction response. Lateral deformations of the buildings with flexible bases experience significant amplification ranging from 50% to about 300% in comparison to the fixed bases for buildings founded on soil class E (Vs < 180 m/s).

Kutanis and Elmas (2001) presented an idealised 2-dimensional strain finite element to evaluate the dynamic effect on soil-structure interaction (SSI). The analysis was performed based on a substructure method using developed software to estimate the impact of soil-structure interaction. The linear SSI analysis and non-linear SSI analysis were conducted. The same structure was analysed with and without soil-structure interaction. These computations were studied varying the effect of accelerations and different soil condition as well as different shear wave velocity.

Available theoretical modelling methods for SFSI analysis are as follows. Two primary methods include Substructure method and Direct method.

Substructure method: In this method, the soil-pile-structure system is divided into near-field and far-field cases. According to Kramer (1996). This assumption is based on linear relations between soil behaviour and structure behaviour.

Direct method: In this method, there are three main steps as follows:

First step: Estimation of structure base motion as a foundation input motion (FIM).

Second step: Strength function determination. The strength function is related to damping and stiffness characteristics of the soil foundation system.

Third step: A dynamic analysis of the structure supported by a soft soil at the base is represented by the impedance functions and subjected to a foundation input motion.

The limitation of previous research was adoption of substructure method in assessing the seismic response of structural systems. According to Zhang and Wolf (1998), as the method is based on the superposition principle, which is valid only for the linear behaviour of structure and soil, approximations of the soil nonlinearity using different soil properties may allow the superposition to be applied for nonlinear systems. Therefore, taking into account the exact nonlinearity of the subsoil in the dynamic analysis may not be easily achievable using this technique.

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2.12 Soil-structure interaction under sismic effects (expermental studies)

To understand the soil-foundation-structure interaction in tall buildings with different types of foundation under seismic conditions, the structure should be experimentally tested with the underlying soil. However, the full-scale field test is often very expensive, time-consuming and difficult to control or to change the test parameters. Therefore, researchers have been implementing a 1-g scaled-model approach on a shaking table, which can be achieved in a relatively short time, is inexpensive and allows performing the parametric study on those scaled models (Li et al., 2006). Therefore, simplified scaled models are required to consider the prototype as a single-degree of freedom system . The main attention on soil-foundation models is to test these models in a shaking table apparatus to obtain the linear and non-linear dynamic responses under various earthquake records (Rodriguez et al., 2016; Lu et al., 2012; Cheng and Lu, 2010; Yang et al., 2013; Tabatabaiefar, 2012; Chau et al. 2009; Chen et al. 2010). In order to understand the soil-foundation-structure behaviour, Hokmabadi et al. (2014b) and Tabatabaiefar (2012) performed experimental studies of the fixed base structure and soil-structure interaction of scaled moment-frame building structural model with a scale factor of 1:30 on clay soil using periodic force excitation of structures.

Moncarz and Krawinkler (1981) explained a 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 mass density, 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 models. Adequate models use the primary features which influence the behaviour of scathe led model. Distorted models do not comply with the simulation requirements. In order to simulate the overall behaviour of tall buildings within the means available and to focus on the soil-foundation behaviour, the adequate model type with primary features of mass and frequency were used for this work.

The soil properties were often characterised using the dynamic properties such as shear wave velocities, shear modulus and damping in seismic studies (Wolf and Obernhuber, 1985). Most of the studies on soil-structure interaction were conducted on clay soil, as the change of volume of clay during the seismic excitation is insignificant, which simplifies the numerical simulation of the soil sample. When it comes to typical sandy soil, seismic excitation changes the volume of the soil. This phenomenon significantly alters the stiffness and the behaviour of the sand. Therefore, the soil-structure interaction of multi-storey buildings with sand, which does not change the volume during the seismic excitation, should be investigated before investigate the sand with volumetric changes (Stromblad, 2014).

Cha and Cho (2007) performed an experimental study to determine the shear strength of sandy soil based on soil shear wave velocity. They adopted the effective stress and void ratio to find out the soil shear strength which are also the primary factors influencing shear wave velocity. They presented shear wave velocity, void ratio and shear strength correlations through experimental tests for various sand fields deposited.

Shear wave velocities were tested for each prepared specimen with a particular void ratio. They concluded that The relationship between shear wave velocity and effective vertical stress are found at extreme values of void ratios (e_{min} and e_{max}). Experimental results showed that the internal friction angle based on a direct shear test of each sand type varied with void ratio value, rendering a unique relationship between friction angle and void ratio, (Figure 2-19). The researchers suggested a procedure to evaluate the in-situ shear strengths of a sandy soil based on soil shear wave velocities.

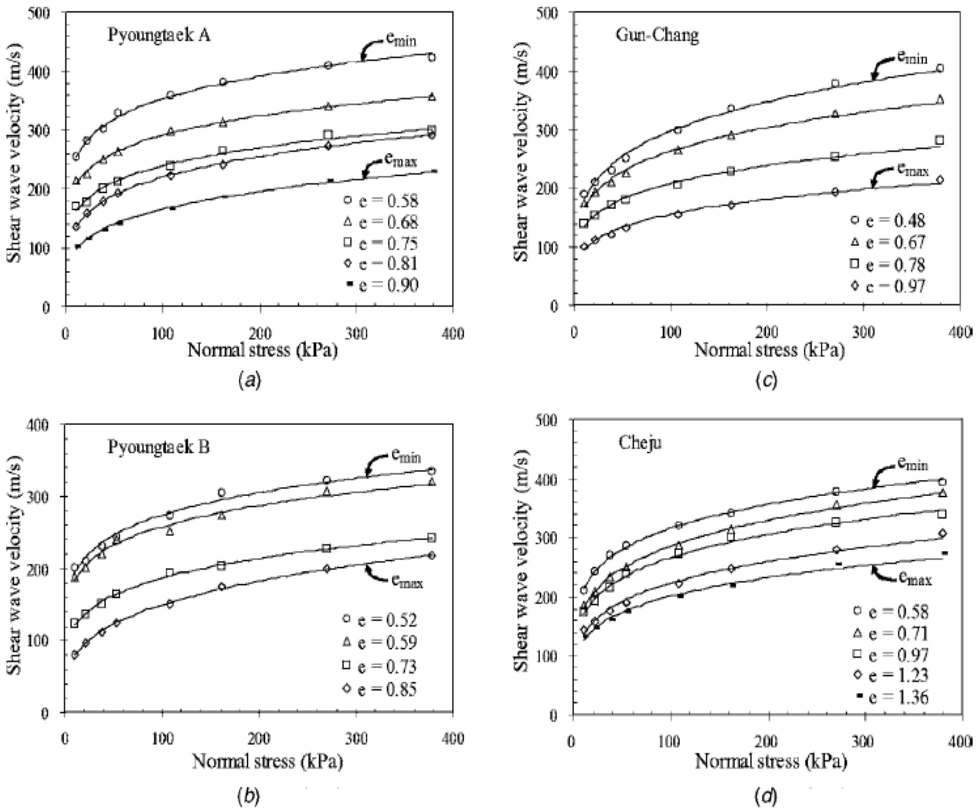


Figure 2.19 Variations of void ratio versus shear waves velocities after Cha and Cho (2007)

Hokmabadi (2014) considered the seismic interaction of soil-pile-structure system (SSPSI). A physical model with scales of 1/30 was designed for dynamic tests. Laminar soil container was selected and designed. A series of shaking table tests were conducted on the scaled model. Three model cases were invistagted: the first model was the fixed-base structure which represents the structure alone without soil-structure interaction.

The second model was the structure supported by the shallow foundation on soft clay soil. The third model was a structure supported by frictional pile foundation in soft clay soil. A three-dimensional numerical software (FLAC3D) has been employed to perform time-history analysis on the three cases mentioned. The output results were presented for the structural response parameters. Hokmabadi (2014) concluded that the most significant effects are base shear, rocking, floor deformation, and inter-storey drifts.

Massimino and Maugeri (2013) conducted two experimental models to consider a shallow foundation in the sand deposit. Shaking table tests were utilised to analyse the soil-foundation interaction. The time-histories of accelerations and displacements were recorded in the soil deposit and on the foundation. Then FEM codes were employed in the software model to analyse the results.

A comparison was made between the analytical and numerical results and also with the experimental results to validate the analytical approaches and numerical modelling. Lu et al. (2002) designed and manufactured an experimental model for shaking table tests where the similitude factors and formula of all experimental models were studied. The ratio between the container diameter and structure plan size was controlled. A flexible container was designed and constructed to minimise the boundary effects resulting from the container boundary wall.

The SSI model tests were strongly affected by the simulation design procedure and the soil boundary. Nine samples were investigated in this study including one fixed base, three box foundations and five pile foundations. A mass block with a single column with a mass fixed at its top and multistorey building with 12 R.C. frame model was used as a prototype model. Shanghai soft clay soil was employed as a soil medium. As a conclusion, there was some significant findings from the experimental tests. Maymand et al. (2000) were initially interested in the development of the 1-g scale model testing program and presented results obtained from single piles with different inertial loading conditions, illustrating the potential for both kinematic and inertial response. Finally, a comparison of the model's measured site response to the predictable free field response confirmed that the modelling system was performing as intended.

Prasad et al. (2004) focused on model test developments for dynamic experimental tests. Two testing aspects of the model were taken in to the consideration ,firstly, the manual shaking table and secondly the laminar box. Development, design, performance and calibration were described. In geotechnical earthquake engineering, model testing is the essential step that helps to understand the model behaviour and performance during an earthquake. Manual shaking table is an economic test that can be used as an alternative test for the more advanced shaking table. The laminar box is an advanced container that can enhance the accuracy in assessing the ground behaviour. Some of the fundamental calibration methods were clarified and discussed (Rayhani and El Naggar, 2008). The shaking table test is an experimental technique used in earthquake engineering to simulate ground motions. Shaking table tests have been adopted as a relatively cheap and easy tool to model complex prototypes.

In shaking table tests, a container is required to hold the soil in place. In literature, this container is called ‘soil container’ or ’soil tank’. During the past few decades, several researchers have carried out shaking table tests on soil-structure systems using various types of soil containers and structural models as summarised in Table 2.1. In geotechnical engineering, the experimental model tests offer a simulation advantage for complex structure systems providing the opportunity to understand the fundamental mechanisms of the system under controlled conditions. The experimental test can also be used as a calibration benchmark for numerical models to make quantitative predictions of the prototype response.

There are three main types of soil container, rigid, flexible, and laminar containers. Rigid containers are the simplest type consisting of the fixed wall without any moving parts. Jakrapiyanun (2002) illustrated the advantages and disadvantages of rigid wall containers. The rigid containers are suitable for the earth structure since the soil on one side of the earth retaining structure is lower than the other side. Therefore, the soil on the shallower depth is less restricted. The main disadvantage of the rigid containers is distorting the free field boundary conditions. This occurs because of the following reasons:

- The rigid walls cannot move along with the soil.
- There may be excessive energy reflections from their boundaries.
- To provide the free field conditions in this type of container, an extremely large container is required which is not feasible in most cases. Another option to reduce the reflecting energy is to attach energy absorbing layers to the container walls.
- Flexible containers allow the modelled soil inside them to move more analogous to the free-field ground motion in comparison with rigid containers. Also, a reflection of outward propagating waves back into the model from the walls could be reduced more efficiently (Maymand, 1998).
- Comprehensive literature study on 1-g shaking table test of soil container with or without structure and foundation are summarised in (Table 2-1). Main objective of this study is to understand the soil-foundation-structure interaction of tall multi-storey building. Therefore, experimental model was designed to suit the main objective of this paper.

Table 2-1 Summary of available previous shaking table experiments

Reference	Soil and soil container	Structure	Foundation	Scale factor	Objectives/Outcomes
Gohl and Finn, 1987	Rigid container and dry sand soil	Pile + mass, single degree of freedom	Pile foundation		To investigate the pile dynamic behaviour under seismic excitation
Yan and Byrne, 1989	Rigid container and dry sand soil	Reinforcement soil wall	-		To investigate the retaining wall dynamic response of soil structure interaction under seismic excitation
Richards et al. 1990	rigid container and saturated soils sand	sliding retaining walls			both initial and general fluidization of a dry sand layer are demonstrated by shaking table tests of a circular footing and a submerged buoyant box.

Valsangkar et al., 1991	**-**	**Wall steel frame**	**Fixed base**	**full scale**	**To investigate the structural dynamic behaviour**
Kanatani, 1991	**Laminar container and sand soil**				**numerical simulation of shaking table test to simulate the nonlinear characteristics of soils were induced by the elasto-plastic theory**
Zen et al.,1992	**rigid container and Trated sand by Mixing sand with amount of cement**				**Comparison of treated sand with non-treated sand under effect of seismic**
Ishimura, 1992	**Rigid container and sand soil**	**Single mass story**	**Pad footing**	**¼ scale factor**	**Sway rocking model to validate the numerical model of soil structure system**
Jafarzadeh and yanagisawa 1995	**rigid container and saturated soils sand**				**Study the effect of one- and two-dimension shaking table on the response of the soil sample**
Taylor et al., 1996	**Fixed base**	**Large scale three story model**			**To investigate the mass effect of the dynamic behaviour of structure**
Meymand, 1998	**Flexible membrane container and clay soil**	**Single pile**	**-**		**To investigate the pile dynamic behaviour under seismic excitation**
Maugeri et al., 2000	**Flexible and sand**	**Single degree of freedom**	**Foundation beam**		**To investigate response of soil structure interaction under effect of ground motions**
Lu. et al, 2001	**Flexible membrane container and clay soil**	**moment framess structure and and single degree of freedom mass system**	**Pile foundation and raft foundation**	**1/10 &1/20**	**To investigate the dynamic response of soil structure interaction and validate the numerical models**
Jakrapiyanun, 2002	**Laminar and sand soil**	**Three degrees of freedom**	**Raft foundation**		**To investigate response of soil structure interaction under effect of ground motions**
Biondi et al, 2003	**Laminar container and sand soil**	**Single story structural system**	**Beam foundation**	**1/6**	**To investigate response of soil structure interaction under effect of ground motions**

Prasad et al., 2004	**Laminar and sand soil**	**-**	**-**		**Calibrate the design of Laminar container under dynamic effect**
Menglin et al., 2004	**Laminar container and clay soil**	**Mulita story tuned mass damper system (TMD)**	**pile foundation**		**To study the response of soil structure interaction of TMD system under dynamic effect dynamic effect**
JAFARZADEH , 2004	**Laminar container and sand soil**	**-**			**Calibrate the design of Laminar container under dynamic effect**
Ueng et al., 2006	**Laminar container and sand soil**				**Calibrate the design of Laminar container under dynamic effect**
Li et al., 2006	**Laminar container and clay soil**	**Mulita story moment frame**	**Pile foundation**	**1/10**	**To investigate the dynamic response of soil structure interaction and validate the numerical models for liquefaction studies**
Bathurst et al., 2007	**Rigid container and sand soil**	**Retaining wall**	**Fixed base**		**To examine the dynamic behaviour of a linear elastic buffer of a sand backfill.**
Pitilakis et al., 2008	**Laminar container and sand soil**				**Experimental and numerical simulation of the soil container**
Abate et al., 2008	**Laminar container and sand soil**	**Single story**	**Pad foundation**		**To investigate the dynamic response of soil structure interaction and validate the numerical models**
Paolucci et al.,2008	**Laminar container and sand soil**	**Single degree of freedom structural system**	**Pier shallow foundation**		**Investigate the foundation ductile behaviour during strong seismic shaking and validate the numerical models**
Chau et al., 2009	**Laminar container and sand soil**	**Single story system**	**Pile foundation**		**To study the damage of piles during seismic excitation and validate numerical models**
Tang et al,. 2009					**The development and current situation in shaking table control system are presented from three aspects, including mode method, parameter**

					identification and control algorithm. And then, the developing trends of shaking table control system are proposed,
Turn et al., 2009	**Laminar container and sand soil**				**design, fabrication and commissioning of a single axis laminar shear box for use in seismic soil–structure interaction studies**
Moss et al., 2010	**Studied the fixable and rigid soil container with clay soil**				**Compare and investigate the dynamic behaviour of the fixable and rigid soil container under seismic effect**
Chen et al., 2010	**Laminar container and sand soil**				**Develop the laminar shear box for use in seismic soil–structure interaction studies**
Ha et al., 2011	**rigid container and Loose sand**				**illustrate that sand deposits can be liquefied again (or "reliquefied") by a subsequent earthquake after initially liquefying during seismic shaking**
Bhattacharya et al. 2012	**Laminar and rigid container with sand soil**				**six types of soil container which are summarised and critically reviewed. The specialised modelling techniques entailed by the application of these containers are also discussed.**
Tsukamoto, et al., 2012	**Laminar container and sand soil**	**rigid circular foundations**			**The settlements of model foundations and the distributions of excess pore-water pressures induced around the model foundations are observed**
Liu, 2012	**rigid container and sand soil with two container depth of 2.5 and 5 m**	**Underground structure**			**discuss the dynamic responses both in the experimental and the numerical models, show of both experimental and numerical models the interaction between soil and structure is not so significant**

Qin et al., 2013	**Laminar container and sand soil**	**Single Degree-of-Freedom Model**	**Foundation beam**	**dimensionless variable**	**The experimental response of soil structure interaction**
Dashti et al., 2013	**rigid container and sand soil**	**water reservoirs underground structure with different stiffness**			**The data from these experiments help evaluate the effects of seismic soil-structure-interaction (SSI) on the distribution of accelerations and lateral earth pressures on underground structures.**
Anastasopoulos et al., 2013	**-**	**Marable column**	**Fixed base**	**Reduced scale model**	**The marble specimens were excited by idealized Ricker wavelets and real seismic records**
Kawamata et al. 2014	**rigid container and sand soil**	**Underground structure**		**large-scale**	**The interaction of complicated 3-dimensional localized behaviours was investigated to study mechanism of soil-shaft-tunnel interaction is discussed based on the test results.**
Hokmabadi, 2014	**Laminar container and clay soil**	**Mulita story moment frame structure**	**Raft and raft on a pile**	**1/30**	**To investigate the dynamic response of soil structure interaction and validate the numerical models**
Massimino and Maugeri, 2015	**Laminar and sand soil**	**Single degree of freedom**	**Shallow foundation**		**The data from these experiments help evaluate the dynamic effects of two cases were used to evaluate the soil structure interaction**
Bojadjieva et al. 2015	**Laminar container and sand soil**	**-**	**-**	**-**	**Dynamic analysis of a sand sample for liquefaction research study**
Qi-ying et al. 2015	**rigid container and sand soil**		**Pile foundation**		**Small shaking table tests were carried out to on the liquefiable sand soil of foundation model with different pile length**
Ulgen et al, 2016	**Laminar container and sand soil**	**culverts**			**investigate the dynamic response of underground culverts by considering the soil–structure interaction**

Goktepe at el , 2017	**Laminar container and clay soil**	**Mulita story moment frame**	**Raft foundation**	**1:45**	**the dynamic parameters of the scaled model of a single layer soil, have been compared numerically to validate the numerical models for soil structure interaction studies.**
Edinçliler et al., 2017	**rigid container with sand soil**	**reinforced and unreinforced embankment models**		**1/50**	**The main focus of this study is the comparison between an unreinforced and reinforced embankment under dynamic effect**
Zhang et al., 2017	**Flexible membrane container and sand soil**	**Mulita story moment frame with damper system**	**Pile and raft foundation**		**series of SSI systems composed of different materials are tested on a shaking table to explore the damping characteristics of SSI system**

Consequently, previous researchers have emphasised the significance of soil-foundation-structure interaction in the response of superstructures and clarified some aspects of it. Some previous investigations used substructure model to represent the soil behaviour. It means that they treated the soil and structure separately. Thus the models were not able to capture the coupled behaviour of soil-foundation-structure interaction. Other groups of researchers modelled all relevant components such as soil, pile, and superstructure simultaneously, but they assumed a linear or an equivalent linear behaviour for the subsoil and linear behaviour for the superstructure without accounting for the full nonlinear coupled behaviour of both soil and structural elements.

Also, based on the literature review, the following parameters influence the structural response when soil-foundation-structure interaction is considered:

- Building characteristics such as the height and the natural frequency
- Soil properties
- Pile group configuration and the nonlinear interaction between piles and the soil
- Type of foundation such as raft, or raft on pile foundations, and
- The intensity of the input motion

2.13 Summary

Reviewing the way building codes treat the effects of the soil-structure interaction on the structural response, building codes can be categorised into three types:
Codes that provide a simplified linear method together with SDOF structure to account for SSI Codes that appreciate the importance of the soil-structure interaction in analysis and design, but do not provide any practical procedure to consider this phenomenon in the analysis.
Codes that do not highlight the importance of SSI on the seismic behaviour of the structures. In particular, the influence of pile elements and the generated soil-pile-structure interaction on the seismic behaviour of structures during the earthquake is the missing part in most of the building codes, and that is probably due to the complexity of the problem.

Consequently, previous researchers have emphasised the significance of SFSI on the response of superstructures and clarified some aspects of it. Some of the previous investigations used substructure model to represent the soil behaviour. It means that they treated the soil and structure separately. Thus, the models were not able to capture the coupled behaviour of SFSI. Other groups of researchers modelled all relevant components such as soil, pile, and superstructure simultaneously, but they assumed a linear or an equivalent linear behaviour for the subsoil and linear behaviour for the superstructure without accounting for the full nonlinear coupled behaviour of both soil and structural elements. Also, based on the literature review, the following parameters influence the Structural response when SFSI is considered:

- Building characteristics such as the height and the natural frequency
- Soil properties including the dynamic stiffness, damping ratio, and the thickness of soil
- Pile group configuration and the nonlinear interaction between piles and soil
- Subsoil effect by studying the basement wall effect
- Type of the foundation such as shallow, or raft-pile foundations
- Characteristics of the input motion (earthquake intensity)

CHAPTER THREE - EXPERIMENTAL WORK

3.1 General

In geotechnical engineering, models are tested under controlled conditions and provide an advantage of simulating complex model systems and offer the opportunity to clearly understand the mechanical behaviour of system components. Those tests are utilised as a calibration reference to produce a meaningful prediction of the prototype response. The superstructure is commonly simplified as a single-degree of freedom oscillator. The dynamic properties of the prototype structure were presented in terms of structure mass and natural frequency of the first higher modes, number of stories, subsoil density, and reaction. Flexible and dynamic soil behaviour, superstructure height level, and different input motions were carefully studied. An obvious comparison was provided between the structural responses for various types of foundations. Also, further experimental tests were performed to investigate the influence of soil-foundation-structure interaction on the dynamic response of buildings with different parameters see appendix B.

3.2 Experimental work methodology

The behaviour of the soil-foundation-structure system may not be simulated entirely in reality where the effect of higher modes would not be recognised (Moss et al., 2011; Massimino and Maugeri, 2013; Lombardi, 2014; Qin and Chouw, 2014; Al-mosawe, 2013; Yegian et al, 2001). In the current experimental tests, a multi-storey dual structural system (frame-wall) was investigated as superstructure. Moreover, the free field is simulated by selecting a flexible soil container with specific criteria to simulate free motion field and minimising the boundary effects. Soil properties were selected in the shaking table tests

All experimental tests of scaled structure model and dynamic soil models were performed utilising the shaking table apparatus at the Civil Engineering Heavy Structure laboratory at the University of Salford. The shaking table specifications are shown in (Table 3-1).

Table 3-1: Shaking Table Specifications

Table size	1m x 1.25m
Maximum Displacement	75 mm
Maximum Horizontal Force	10 KN
Maximum Acceleration	1 g

The main purpose of the experimental tests was to investigate the effect of the parameters on the structure response (such us building height, foundations type, subsoil reaction effect, and soil properties) under seismic load and compare the outcome of those tests with the predictions from the software programme validate the numerical model for further dynamic studies.

Experimental work was performed to investigate the consequences of a variety of different factors such as building height, foundations type, subsoil reaction effect, and soil properties under seismic effect.

The preparation stage of experimental work consisted of the following:

- Performing the preliminary soil tests to find out the soil mechanical properties such as grain size distribution (sieve analysis), coefficient of uniformity (Cu), specific gravity (Gs), actual soil density (ɣ), maximum density ($ɣ_{max}$), minimum density ($ɣ_{min}$), relative density, actual void ratio (e), minimum void ratio (e_{min}), maximum void ratio (e_{max}).
- Performing the shear box test for different soils to find out the stress-strain curve, angle of internal friction (ϕ), poisons ration (υ), modulus of elasticity (E) and actual shear modulus (G).
- Selecting the suitable sand for the study
- Designing and sketching up the flexible soil container, involving a membrane cylinder 1 meter deep, with 1 m diameter and 5 mm thickness. The top plate of the container was supported by four columns.

- Based on a trial model designed by ETABS software, a trial scaled model of three-levels was built up and the model was tested in terms of mass density and natural frequency (Figure 3-1). Once the output results of the laboratory tests were acceptable, the rest of the scaled model was built up.

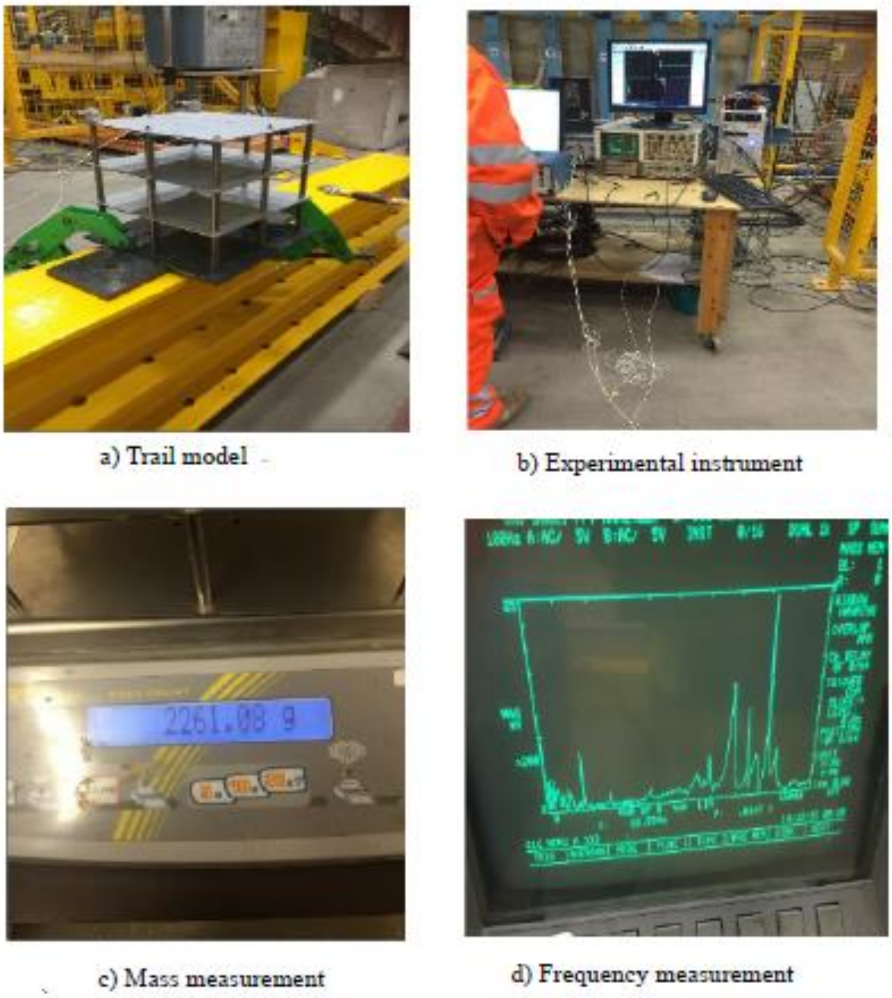


Figure 3-1 Trial scaled structural model

3.3 Prototype Characteristics

A two basement plus fifteen-storey dual concrete wall-frame structural system with a total height of 53 m and width of 10 m has been selected as a prototype for this study.

The structure consists of a regular flat slab with eight columns along the perimeter and shear core wall located in the middle of the building. The prototype model was modelled in ETABS software, and the sections of the structural elements are shown in (Figure 3-2). The structural sections were designed using the Eurocodes. The natural frequency and total mass were obtained as 1.32 Hz and 2904515.3 kg, respectively. The concrete compressive strength (f_{ck}) of 65 N/mm^2, the mass density of 2400 kg/m^3 and elastic modulus of concrete of 36000 N/mm^2 were utilised for this structural system. The necessary parameters such as natural frequencies, total weight and dimensions were obtained using ETABS software.

The structural sections were calculated after performing the conventional design procedure based on the building codes regulations. For the design purpose, ETABS (CSI, 2015) software is employed. The horizontal and vertical distance from the bedrock, depth, and lateral soil boundaries were selected to be 30 m. The first mode shape has the maximum mass participation ratio, implying that the critical mode shape is the first mode.

Taking into account the effect of foundation type on the response of structures considering SFSI is essential for the structure design performance. Four stages of shaking table test were proposed to study the dynamic behaviour of the soil structure:

Firstly, the fixed base stage; in this stage dynamic behaviour of scaled structure is considered individually without soil interaction. Secondly, the soil container stage; in this sage the soil container was considered individually to investigate the dynamic soil behaviour and the dynamic behaviour of the soil container. Thirdly, the soil-foundation-structure interaction (raft foundation); in this stage the structure supported by raft foundation was investigated to evaluate the structure responses under the effects of dynamic forces and the effects of raft foundation. Fourthly, the soil-foundation-structure interaction (pile foundation); in this stage the structure supported by raft on pile foundation was investigated to evaluate the system response under the effects of dynamic forces and the effects of the raft on pile foundation.

2 Basements + 15 typical stories
Building dimensions= 10000 mm X 10000 mm
Wall-frame concrete structural system
Columns cross-section= 600 mm X 600 mm
Slab thickness = 400 mm
Core wall thickness = 400 mm
Core wall dimensions = 2000 mm X 2000 mm
Basement wall thickness = 300 mm

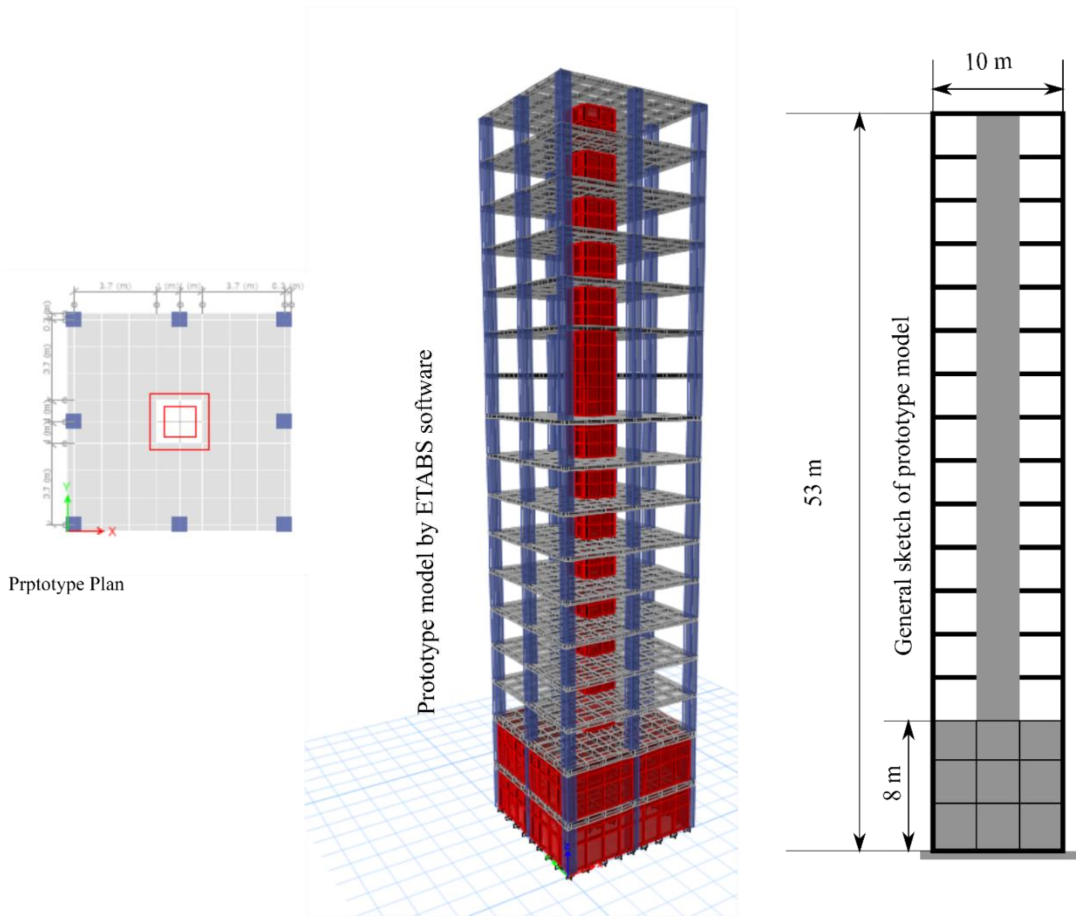


Figure 3-2 Details of the prototype

3.4 Scaling Factors for Shaking Table Tests

Earthquake force is an essential consideration in the design of multi-storey structures because of its serious damaging effect. It may be necessary to utilise the elastically scaled structural model and tests under controlled conditions to investigate the soil interaction behaviour. Li et al. (2014) concluded that the under different circumstances a set of scale relations of the dynamic soil-structure behaviour are essential factors in the simulation of the experimental models which are adapted to predict the porotype behaviour under seismic effects. The scale model test output was used to calibrate the results of the numerical model. Also, the scale models are providing an economical option to simulate the prototype model.

Geometric and dynamic similarities define the relations of the dynamically scaled model. Geometric similarity illustrates the scaled dimensions relationship with porotype dimensions, while the dynamic similarity describes the conditions of prototype and the scale model in relation to net forces and physical properties. Scaled models of many researchers (Hokmabadi, 2014; (Moss et al., 2011; Massimino and Maugeri, 2013 and Li et al., 2014) meet the geometric and dynamic simulations requirements.

Analytical models are required to be calibrated by experimental results. A set of scaling relation factors is needed to predict the prototype behaviour. Shaking table test is an experimental technique utilised to simulate the ground motions in earthquake engineering. The 1-g modelling methods of shaking table tests were adopted. Shaking table test is relatively economical to simulate the prototypes. The frequency of vibrations, number of stories and mass should be designed and examined.

The scaled models can be classified into three different categories based on the degrees of accuracy: true, adequate, and distorted models (Hokmabadi, 2014; Krawinkler and Moncarz, 1981). True models require the geometric and dynamic simulation factors on the scaled models. Adequate models use the primary features which influence the behaviour of the scaled model. Distorted models do 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 behaviour, the adequate model type with primary features of mass and frequency was adopted in this study.

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). This scaled factor is also known as “Cauchy condition”.

Where

E and ρ are the modulus of elasticity and mass density, respectively.

(a) is acceleration,

and (*g)* is gravitational acceleration.

The Cauchy condition is a requirement for simultaneous replication of restoring forces, inertial forces, and gravitational forces in a dynamic system (Table 3-2).

Meymand (1998) and Moss et al. (2010) claimed that the equation of motion cannot describe the entire soil-foundation-structure system. The simulation or analysis theory can be applied directly to the soil-structure system to achieve an accurate model which is a so-called true model.

Table 3-2 Scaling relations for geometric scaling factor (λ), (Moncarz and Krawinkler, 1981)

Acceleration 1	Shear Wave Velocity $\lambda^{-\frac{1}{2}}$	Stress λ	Stiffness λ^{2}
Mass Density 1	Time $\lambda^{\frac{1}{2}}$	Strain 1	Force λ^{3}
Length λ	Modulus λ	Frequency $\lambda^{-\frac{1}{2}}$	EI λ^{5}

To achieve the dynamic similarity a scale modelling procedure is required for this test program where both the scaled model and prototype are subjected to a particular condition.

By defining density and acceleration under specific scaling conditions, the length, mass and time can all be expressed in terms of the geometric scaling factor (λ), and a dimensional scaling factor describing the relationship between the prototype and scaled model can be derived for all parameter under consideration. The set of scaling factors for the contributing variables are necessary to estimate the fundamental modes of system response (Moss et al., 2010, Meymand, 1998, Tabatabaiefar, 2012, Turan et al., 2013, Turan et al., 2009).

3.5 Scaled Model Design Concept

In the literature, scaled structures of dynamic studies were physically modelled as either single-degree of freedom (SDOF) systems or lumped mass multi-degree of freedom (MDOF) systems. When it comes to the tall buildings, SDOF would not be suitable. Hence MDOF scaled model approach based on lumped mass simplification method has been widely implemented in recent studies.

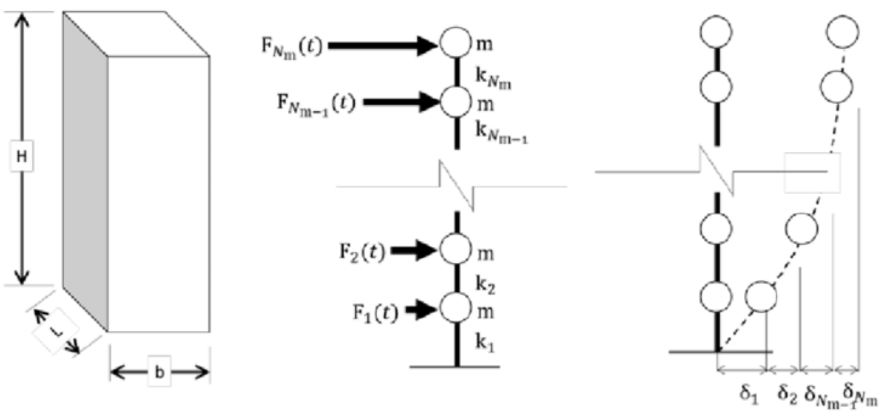


Figure 3-3 Lumped mass simplification of multi-story buildings (Serrano et al., 2017)

According to the lumped mass simplification, mass is the floor mass and K is the stiffness of floor and K is proportional to the frequency. 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 to examine the lateral deflection of the structural model. Tabatabaiefar, (2012); Hokmabadi, (2014) mentioned that the mass and frequency are playing the key role in the design of scaled model for dynamic purpose.

Figure 3-4 Scaled model of (Tabatabaiefar, 2012)

The prototype model of (Tabatabaiefar, 2012) was fifteen story concrete frame structure, flat slabs with sixteen columns in each floor. This prototype was scaled down based on the scaling methodology (see section 3-5) and the scaled model was constructed accordingly. In the scaled model of (Tabatabaiefar, 2012) Figure 3-3, the steel sections were used to simulate the slabs and frame system. based on the scaling methodology, the columns were simulated and designed by four steel plates with certain requirement to achieve the required frequency of the scaling approach. 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. The steel sections were used in the design of the scaled model element.

In the current study, the mass and frequency role were adopted in the design of the scaled model. Commonly the multi-story building with shear wall system is stiff and rigid. The physical model should achieve the frequency required of this rigid structure. To simulate this prototype experimentally, threaded connectors with bolt nut screws were used to connect the top floor and bottom base of this scaled model (Figure 3-4). This connection methodology was used in scaled model to connect floors to make the scaled model stiff and preventing any story drift during the seismic excitation. The bolt screws at the top floor help to adjust the stiffness and frequency of the scaled model experimentally.

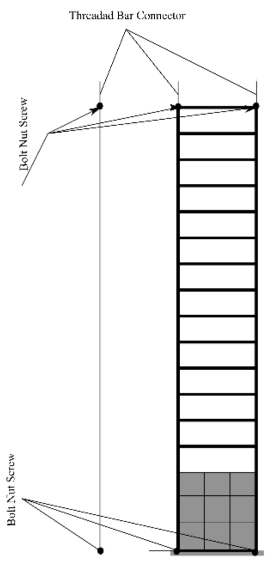


Figure 3-5 Scaled model connecter details

In general, multi-story wall-frame concrete structures are rigid and the 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.

3.6 Model Components of the shaking Table Tests

The soil-foundation-structure model elements developed for shaking table tests consist of the scaled structural model, the soil container and the shaking table events. Furthermore, the foundation details and the instrumentation properties of these components are explained in the following sections.

3.6.1 Scaled Model

The scale factor of 1:50 has been selected as illustrated in (Table 3-2). The scaled model dimensions are 1.05 m in height (H), 0.20 m in length (L), and 0.20 m in width (W). The natural frequency and the density parameters play a fundamental role in the process of model scaling. Hence, the prototype natural frequency was scaled down with suitable scaling factors, while the density of prototype and scaled model were selected as equal (Meymand, 1998).

Furthermore, structural steel model is flexible and constructible to the test environment, while concrete structural model could not be constructed with the required dimensions and dynamic properties.

Therefore, the concrete structure prototype element was scaled into a steel structure model element by scaling the natural frequency and the density of the prototype.

As per scaling factor (Table 3-2), the relationship between the scaling factors of the natural frequencies of the prototype (f_p) and the scaled model (f_m) is defined as:

$$\frac{f_m}{f_p} = \lambda^{-\frac{1}{2}} = 7.07 \qquad 3.1$$

The natural frequency of prototype is 1.32 Hz. Therefore, the required natural frequency of the structural scaled model (f_m) can be calculated as follows:

$$f_m = 7.07 \times f_p = 9.33 \text{ Hz} \qquad 3.2$$

Scaling factor of the relationship between the scaled model density (p_m) and prototype density (p_p), based on the scale factor is:

$$\frac{p_m}{p_p} = 1 \qquad 3.3$$

The prototype structure density (p_p) is calculated as:

$$p_p = \frac{m_p}{V_p} = \frac{2904515.3 \text{ kg}}{(10\ m \times 10\ m \times 53\ m)} = 548.027 \text{ kg}/m^3 \qquad 3.4$$

Where

m_p and V_p are the mass and volume of the prototype, respectively. By substituting the prototype density into the equation 3.3 the scaled model mass can be calculated as:

$$m_m = p_m \times V_m = 548.027 \times 0.2 \times 0.2 \times 1.06 = 23.2 \text{ kg} \qquad 3.5$$

Where

V_m is the volume of the scaled model.

The required characteristics of the scaled model are summarised in (Table 3-3).

Table 3-3 The required characteristics of the scaled structural model parameters

Parameter	Value	Parameter	Value
Scale factor	1:50	Natural frequency	9.33 Hz
Height	1060 mm	Typical story height	60 mm
Length	200 mm	Basement story height	80 mm
Width	200 mm	Total mass	23.2 kg

3.6.1.1 Design and construction of scaled model

It should be noted that considering the requirements of scaled dimensions and dynamic properties, the equivalent steel model is adopted in this study.

In order to obtain the requied thickness and dimensions of steel plate and tube, the scaled model was designed in the ETABS software (Figure 3-3) and the dimensions were selected to meet the required natural mass and frequency as illustrated in (Table 3-3).

Furthermore, grade 255 steel was adopted in all the elements of the scaled model. In the scaled model, each floor is supported by vertical steel tubes of 8 mm diameter and thickness of 1 mm as the column elements.

Dimensions of 220 ×220 ×5 mm and 200 ×200 ×2 mm for the steel plates were selected as the base plate and the typical floor of the scaled model, respectively. The connections between the columns and floors were designed using 4 mm diameter steel thread bars screwed by nuts on both ends of top level and base floor.

Steel plates of 200 ×160 ×3 mm were attached vertically at the lower levels to represent the basement retaining walls as shown in (Figure 3-6). The final mass and natural frequency of the scaled model were 23.7 kg and 10.1 Hz, respectively. Full detailed drawings of scaled modal are in appendix A.

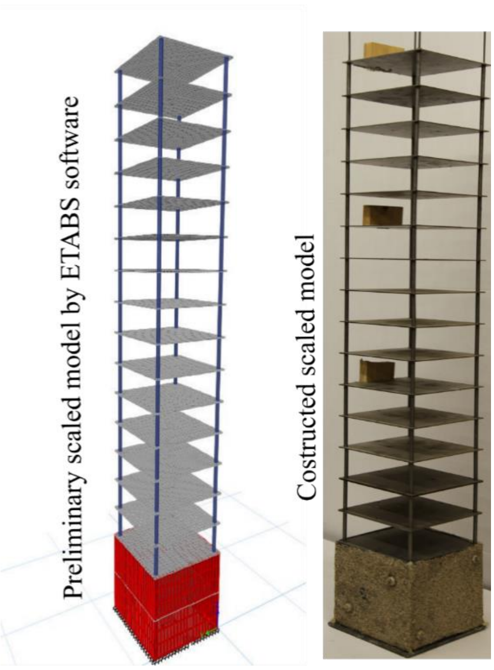


Figure 3-6 Scaled model design

3.6.2 Soil Container Design

In the soil-structure model dynamic tests, the primary concern is the simulation of boundary effects which is created by artificial boundaries of the soil container. The function of the soil container is to provide confinement and holding the soil in place during the dynamic excitation. To achieve the real response, the ideal simulation of the free field soil behaviour of the prototype is performed by minimising the soil container boundary effects. The key parameter in the soil container design is to obtain the same dynamic shear stiffness as the actual soil prototype.

Two types of containers, namely laminar container, and flexible barrel were utilised for the dynamic study. The laminar soil container is made of a rectangular hollow section of an aluminium frame with rubber layers separating the frames. The function of rubber layer is to allow the soil's shear deformation, while the aluminium frames function is to provide lateral confinement of the soil (Prasad et al., 2004; Hokmabadi, 2014; Maymand et al, 2000). The main part of the flexible barrel is the flexible membrane wall with stiffening rings, which represents the response of the free field site under seismic effects during shaking table test (Maymand et al, 2000).

Furthermore, Maymand et al, (2000) compared three different types of soil containers (rigid, wing and flexible barrel containers) in their numerical study, where 12.19 m deep deposit of San Francisco Bay Mud was used as a soil case study sample. The results showed that the flexible wall container precisely simulates the soil prototype while the rigid and wing wall containers do not replicate the behaviour of soil under dynamic conditions. To validate the numerical prediction, Maymand et al, (2000) tested the flexible barrel experimentally on a shaking table. (Moss et al, (2011) tested both flexible barrel and laminar containers, and the flexible barrel container provided the best response. Moss et al, (2011) drew two conclusions. Firstly, the flexible barrel container and the relevant constructional details should be appropriately considered to minimise the box effect.

Furthermore, the laminar container is complex and expensive to construct. Therefore, flexible container with stiffening rings was adopted in this study.

Secondly, the container diameter should be five times the structure width. Hence, the dimensions of the container were selected as a 1 m diameter and a 1 m depth. The flexible container was designed and manufactured at the University of Salford as shown in (Figure 3-7). The flexible container consists of 5 mm membrane rebar cylinder wall supported individually by stiffener strips. The top part of the container was supported by lifting hooks from an overhead crane. The bottom base was set on the shaking.

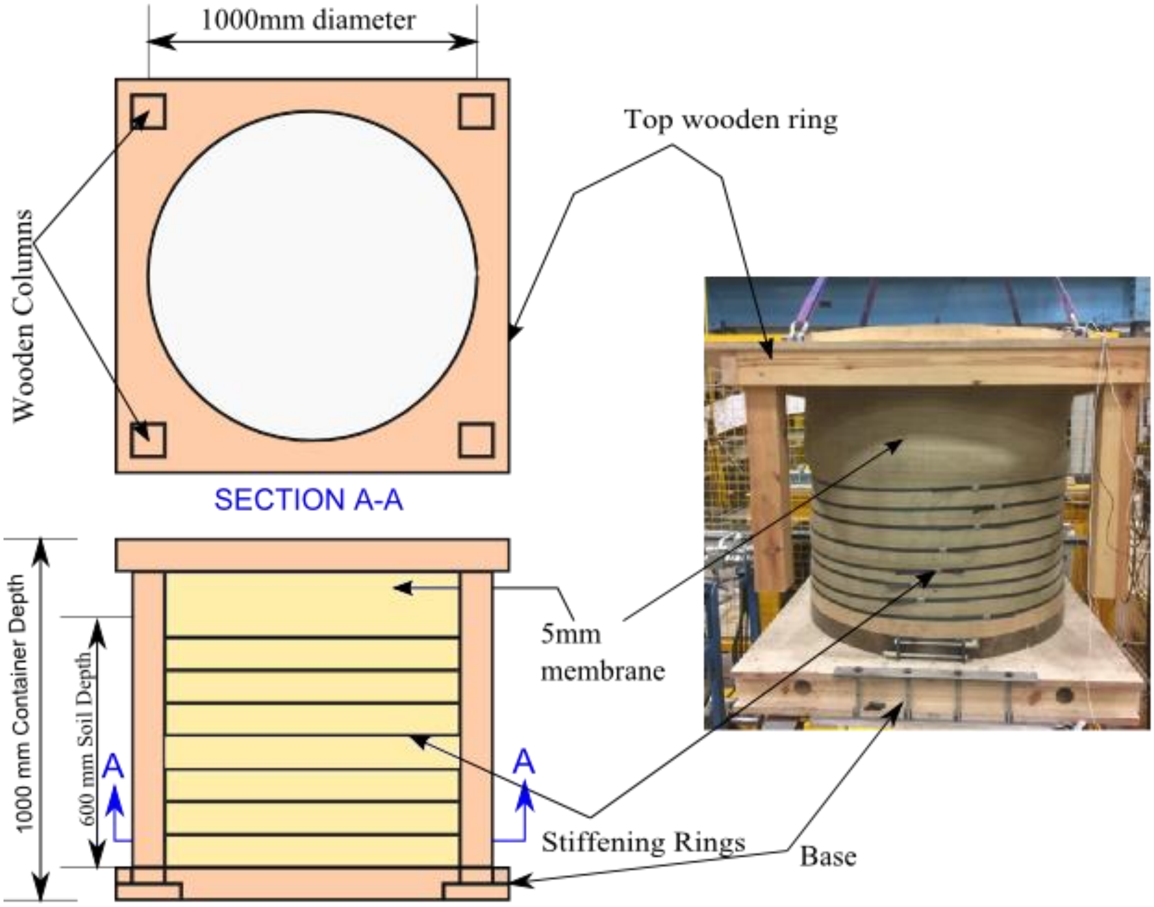


Figure 3-7 The soil container design

3.6.3 Soil Properties and placement

Dry sand with specific characteristics was used to reduce the volume changes during seismic excitation. The grain size distribution of the sub-rounded sand particles. The dry sand maximum density used in the vibration tests is 16 kN/m^3, while the minimum density is 14 kN/m^3. The specific gravity of the selected sand is 2.68. The friction angle was measured as 34° in direct shear tests. The dilatancy of round sand particles is defined by Ryan and Polanco (2008) as:

$$\omega = \phi - 30 \quad 3.6$$

Where

w is the dilatancy and ϕ is the angle of friction (Figure 3-8).

In this figure a dense sand is shown before loading (left). After applying a shear load, the volume has expanded (right) since the sand particles are not as densely packed as before, ultimately making the sand looser.

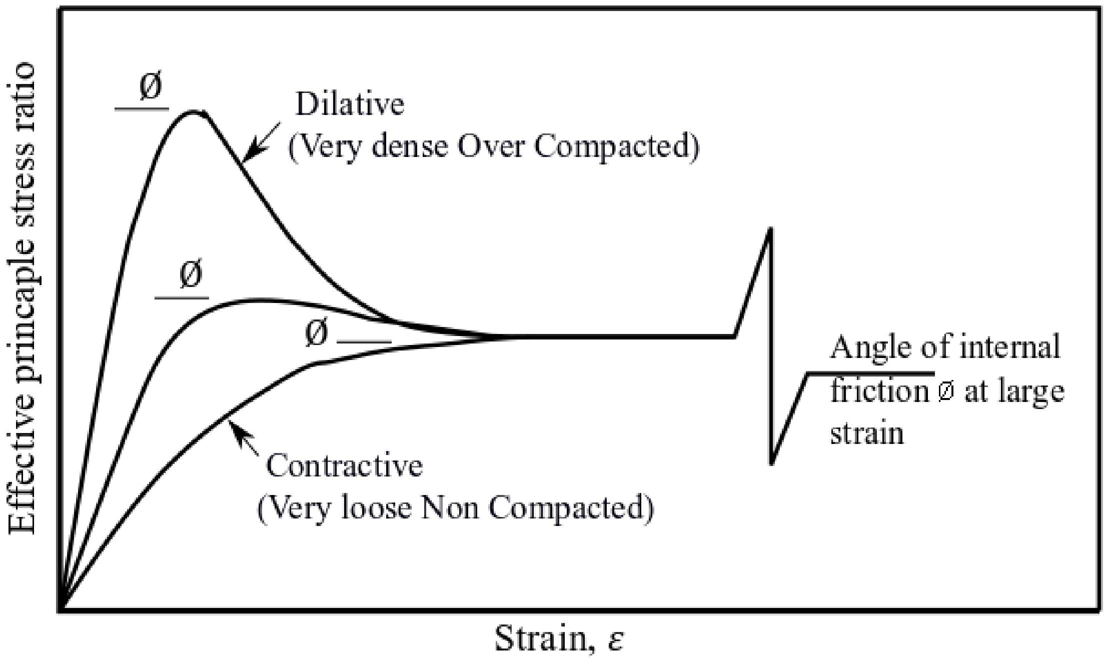


Figure 3-8 Dilatancy of dense sand (Ryan and Polanco, 2008)

Other relevant properties of soil can be found in (Figure 3-9) and (Table 3-10). The sand was placed in the container using the eluviation (raining) technique to achieve a uniform density (Dave and Dasaka, 2012; Pitilakis et al., 2008).

The actual relative densities were achieved and measured by collecting samples in small cups of known volume embedded in different locations within the main container.

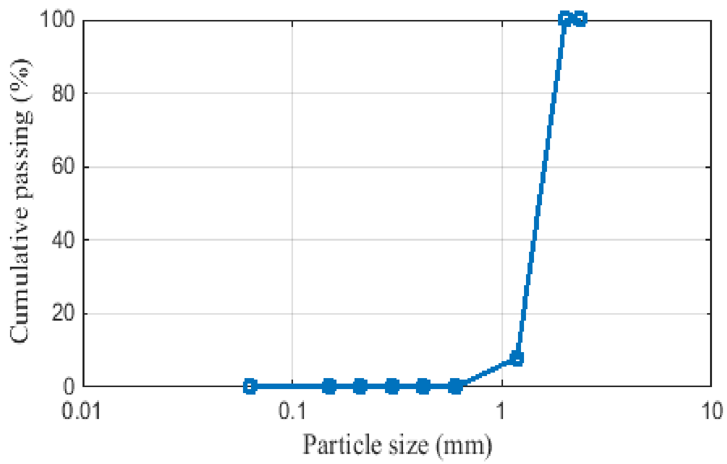


Figure 3-9 Sieve analysis of selected sand

Table 3-4 Selected sand properties

Symbol	Details	Value
D10mm	Grain size	1.3 mm
D30mm	Grain size	1.5 mm
D50 mm	Grain size	1.7 mm
D60 mm	Grain size	1.8 mm
D mm	Particle size range mm	0.6 – 1.18
Cu	Coefficient of uniformity	1.38
Cc	Coefficient of curvature	0.96
	Soil classification	SP
	Soil description	Poorly graded sand
γ_{max}	Maximum dry unit weight	16 kN/m^3
γ_{min}	Minimum dry unit weight	14 kN/m^3
e_{max}	Maximum void ratio	0.48
e_{min}	Minimum void ratio	0.6

3.6.4 Foundations Models

The two types of foundation systems introduced in this study were raft foundation, and pile-on-raft foundation. Similar to the structural scaled model, the simulation of pile model should be carried out by adopting a scale factor. The pile foundation simulation considers the flexural stiffness EI, slenderness ratio L/d relationship and relative soil/pile stiffness (Meymand, 1998).

Furthermore, the relative spacing and group interaction of the pile group are represented in the scaled model. Thus, for a scaled model with a geometric scaling factor of 1:50, the pile diameter should be 15.9 mm. In this study, the piles were considered to have a rigid and linear behaviour. The rigid pile can be obtained by scaling the flexural rigidity (EI). According to (Table 3-2), previous researchers (Tabatabaiefar et al., 2014, Hokmabadi, 2014) used different types of pile materials such as steel bars, aluminium tubes and reinforced concrete to simulate the pile element. By adopting the scale factor of required stiffness and yielding stress for scaled pile model (Table 3-2), The aluminium pile properties were selected as summation of the raft-on–a-pile foundation. The pile characteristics used in this study are summarised in (Table 3-5).

Table 3-5 Characteristics of model piles

Pile Diameter (mm)	15.9 mm
Modulus of elasticity	7X 10^7 kN/m^2
Weight	27 kN/m^3
Poisson ratio	0.33

The model piles used in this study were smooth aluminium pipe piles as shown in (Figure 3-10).

The diameter of piles is 15.9 mm, while the length of piles is kept at 300 mm at all stages of the testing programme. The model pile surface has been glued with sand particles to make rough surface and to avoid the interface problem.

Scaled structure supported by raft on pile foundation

Scaled structure supported by raft foundation

Figure 3-10 Constructed scaled model and foundation types

3.6.5 Accelerogram generation

Using the software Seismo Artif, four artificial time-history accelerograms were generated with different peak ground accelerations of 0.05 g, 0.1 g, 0.15 g and 0.2 g as shown in (Figure 3-11). These events were generated from EC8 elastic spectra soil type C, spectrum type 2, and the derived response spectra were as close a match as possible to the target response spectra. These accelerograms were adopted as dynamic load inputs for the experimental and numerical models.

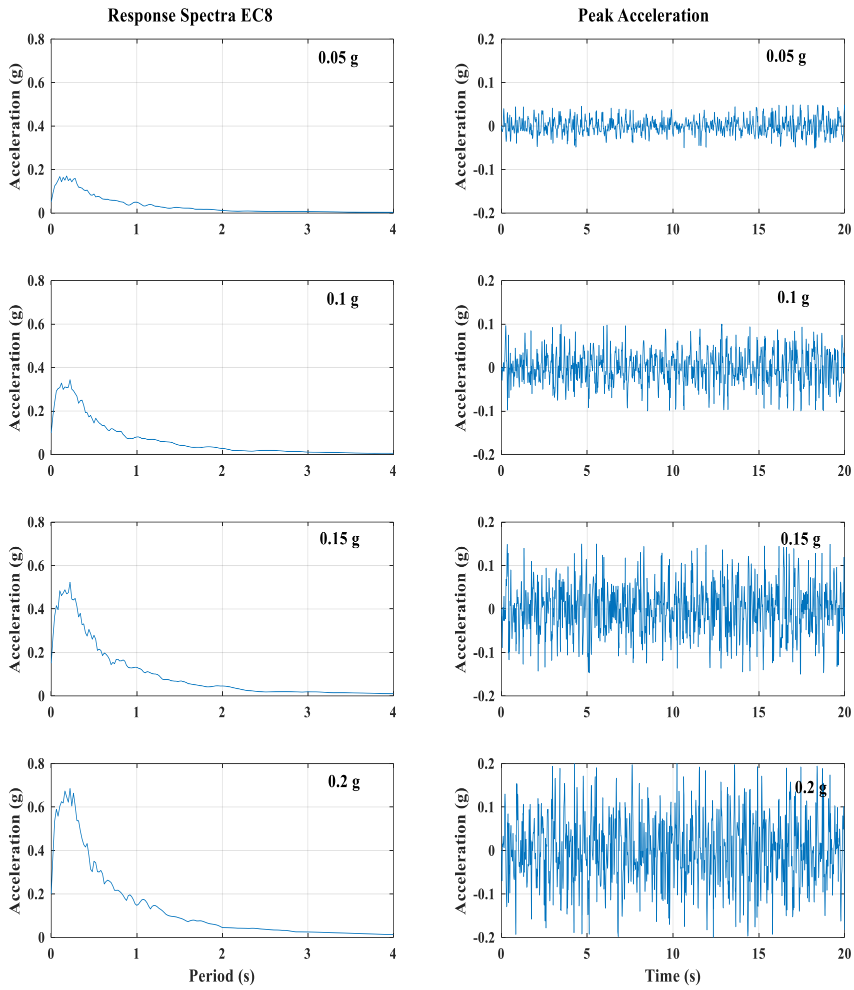


Figure 3-11 Time – acceleration inputs

3.6.6 Shaking table tests experiments

Four intensities of soil ground motions were adopted as shaking seismic acceleration events for the shaking table tests machine programs (Figure 3-12). The events included different frequency components while travelling through the ground ((Figure 3-11).

Figure 3-12 Shaking table at Heavy Structure lab, Salford University

In this study, the experimental tests were divided into four stages as follows:

- Stage 1: Fixed-base structure representing the dynamic behaviour of the structure without the soil-structure interaction
- Stage 2: Soil container, to evaluate the dynamic behaviour of soil and soil container
- Stage 3: Structure supported by the raft foundation in the sand soil to determine the dynamic behaviour of the structure supported by raft foundation
- Stage 4: Structure supported by raft-on-pile group in the sand soil to investigate the dynamic behaviour of the structure supported by raft foundation

3.6.6.1 Fixed base stage

The first stage of tests was carried out on the constructed scaled models. A fixed base model means that the structure is fixed directly on the shaking table to determine the dynamic response of structural model and verify the SFSI numerical model. Dynamic response of the fixed base model under the influence of time history analysis was examined. The constructed scaled model was fixed on the shaking table. The accelerometers and displacement transducers were then calibrated and installed on the scaled structure model at levels 2B+5, 2B+10 and B+15. The instrumentation was utilised to monitor the structure’s behaviour and to determine acceleration and structural lateral displacements in the time domain. The displacement was found by double integration of a measured acceleration in the time domain. Therefore, displacements can be determined at different levels by either double integrating the corresponding accelerations or measuring directly using displacement transducers.

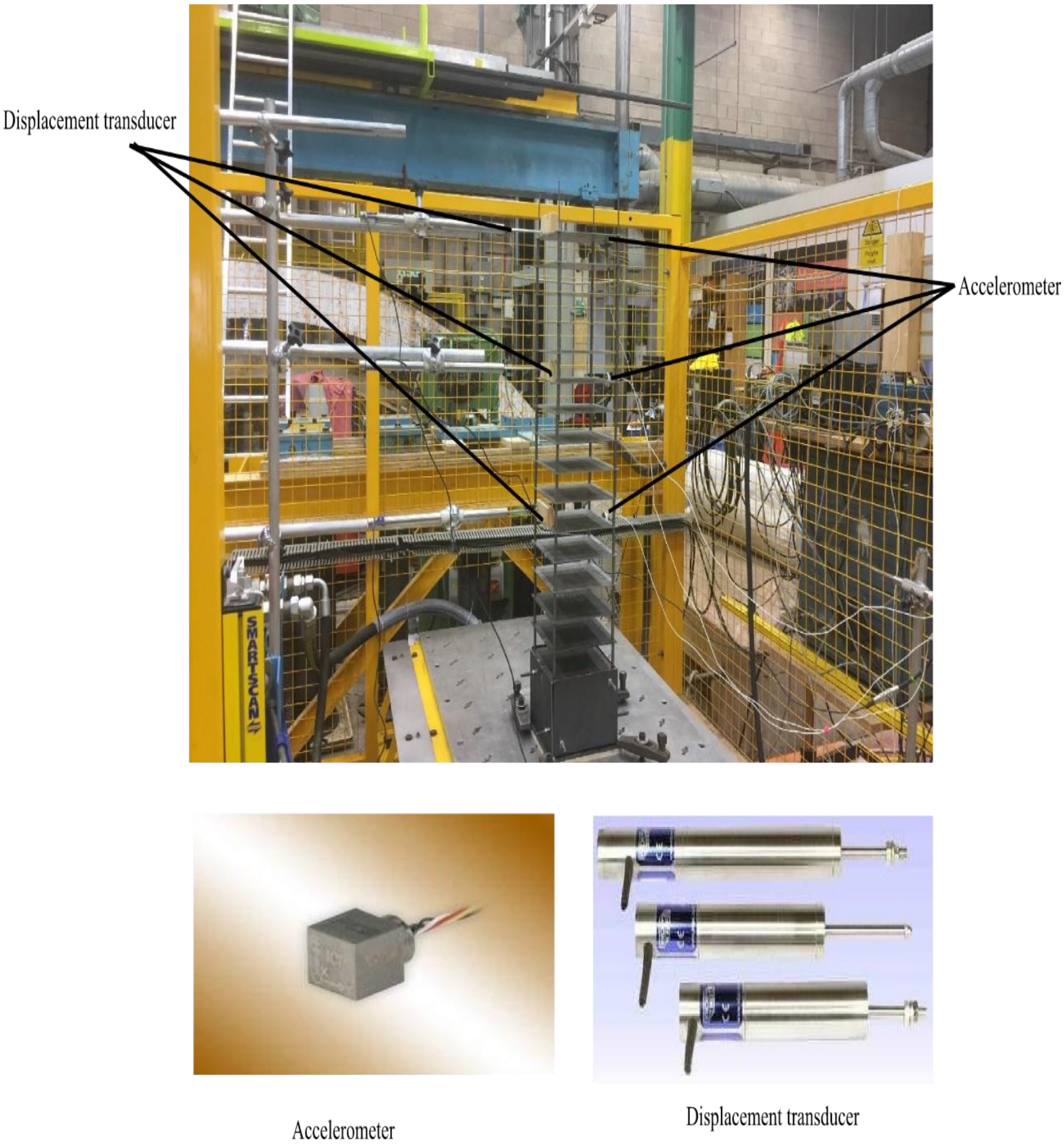


Figure 3-13 Fixed base shaking table test

(Figure 3-13) illustrates the arrangement of the accelerometers and displacement transducers at selected levels of the scaled structural model. Initially, the shocking test was performed on the structural model to determine the natural frequency of the model. The resonance of fundamental structure mode represents the natural frequency of the structure. To ensure that the measured value of the fundamental frequency is accurate and adequate the test was repeated three times. The natural frequency outputs of the constructed scaled structural model were 8.5 ≈ 9 Hz. Based on the scaling modelling methodology, the experimental frequencies test results of the scaled structural model was in a perfect agreement with the calculated frequency of 9 Hz. Therefore, the constructed scaled structural model, with a total mass of 23.7 kg and the natural frequency of 9 Hz, complies successfully with the required characteristics summarised in (Table 3-3) needed for investigation of the soil-foundation-structure interaction based on the dynamic similarity criteria.

The outputs of this stage were utilised as a reference to examine the results of different test stages ((Figure 3-14). The summery of fixed base stage experiments are shown in (Table 3-6).

Table 3-6 Fixed base shaking table test schedule

Test No.	Structure Time History	Foundation	Soil
Exp- 1	0.05 g	Fixed Base	Fixed
Exp- 2	0.1 g	Fixed Base	Fixed
Exp- 3	0.15 g	Fixed Base	Fixed
Exp- 4	0.2 g	Fixed Base	Fixed

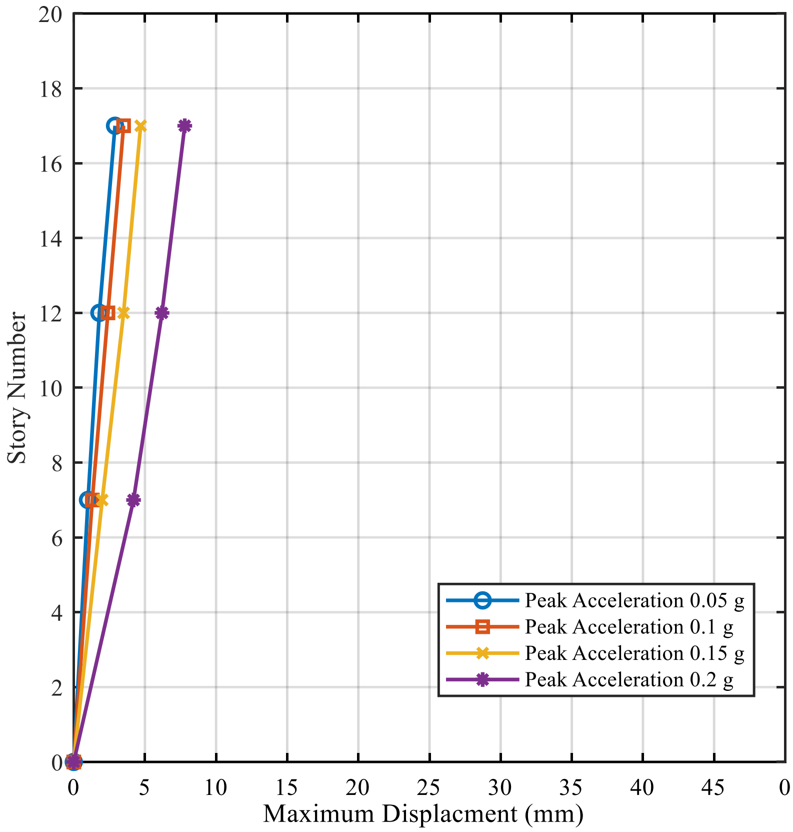


Figure 3-14 Fixed base experimental outputs

3.6.6.2 Container construction and testing procedure

Turan et al. and Chunxia et al. (2009) illustrated the 1-g seismic test procedure of single-axis flexible containers on the shaking tables. The movement of single-axis flexible containers is permitted in a single axis only which typically comprises of either rigid guide walls or laminates stacked on each other and separated by bearings in addition to single-axis containers. Meymand (1998) and Moss et al. (2010) explained in details the 1-g tests procedure of double-axis flexible containers. The movement of laminae in double-axis containers was permitted horizontally in two principal directions. The Meymand container comprised a ribbed membrane hanging from a top ring supported by a frame connected to the shaking table using universal joints. An improved flexible container was adopted in this study and the testing procedure as well as the soil and container details are clarified as follows:

- A. Experimental set-up

The flexible container was designed and manufactured at the University of Salford. (Figure 3-15) shows the flexible container, which consists of a 5 mm membrane cylinder wall supported individually by stiffener strips. The top ring is fixed by lifting hooks supported by lifting crane. The bottom base is set on the shaking table.

Figure 3-15 Soil container fixed on shaking table at Salford University

- B. Instrumentation and performed tests for soil container

The summary of soil container experimental tests is shown in (Table 3-7).

Table 3-7 Experimental Soil Container Tests

Test No.	Purpose	Type of loading	Predominant frequency (Hz)	Peak amplitude (g)
Exp- 1	To examine the container boundary wall effect	Harmonic sine wave	2	0.1 g
Exp- 2	To examine the container boundary wall effect	Harmonic sine wave	2.25	0.1 g
Exp- 3	To examine the container boundary wall effect	Harmonic sine wave	2.5	0.1 g
Exp- 4	To examine the container boundary wall effect	Harmonic sine wave	2.75	0.1 g
Exp- 5	To examine the container boundary wall effect	Harmonic sine wave	3	0.1 g
Exp- 6	To examine the container boundary wall effect	Harmonic sine wave	3.25	0.1 g
Exp- 7	To examine the container boundary wall effect	Harmonic sine wave	4	0.1 g
Exp- 8	To investigate the hysteric soil behaviour	Earthquake time history		0.05 g
Exp- 9	To investigate the hysteric soil behaviour	Earthquaketime history		0.1 g
Exp- 10	To investigate the hysteric soil behaviour	Earthquake time history		0.15 g
Exp- 11	To investigate the hysteric soil behaviour	Earthquake time history		0.2 g

(Figure 3-16) shows the layout of the instrumentation across the section along the diameter of the flexible container. Accelerometer ACC1 was connected to the shaking table. ACC2 was positioned almost at the centre of the soil mass. Locating the accelerometers on the top of the soil surface would prove problematic when mobilising the mass of the accelerometer and it would be difficult to ensure full interaction between the soil particles and accelerometer. Therefore, three accelerometers ACC3, ACC4 and ACC5 were mounted 100 mm below the surface of the soil. ACC5 and ACC6 were attached to the soil container boundary. To investigate the effects of the soil container boundaries, a small amplitude (0.1 g) harmonic excitation was applied to the flexible container via the shaking table to ensure linear soil behaviour.

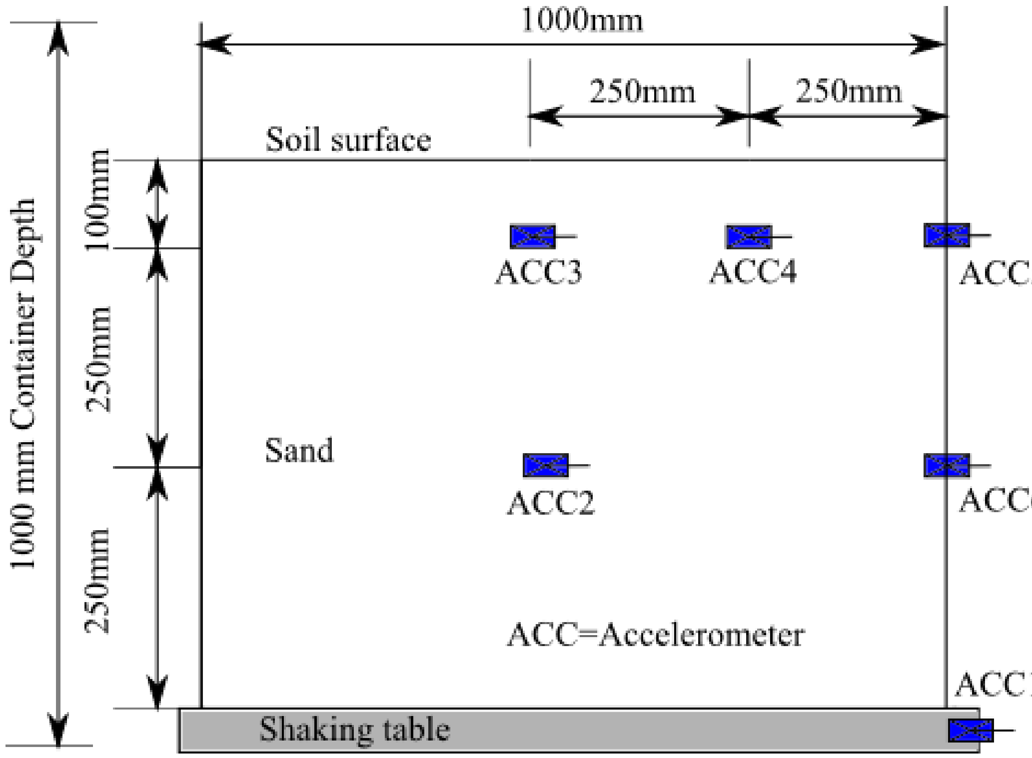


Figure 3-16 Layout of accelerometers

Since all the accelerometers are accurate at frequencies greater than 4 Hz, sinusoidal input motion was applied at 4 Hz with an amplitude of 0.1 g as shown in (Figure 3-17).

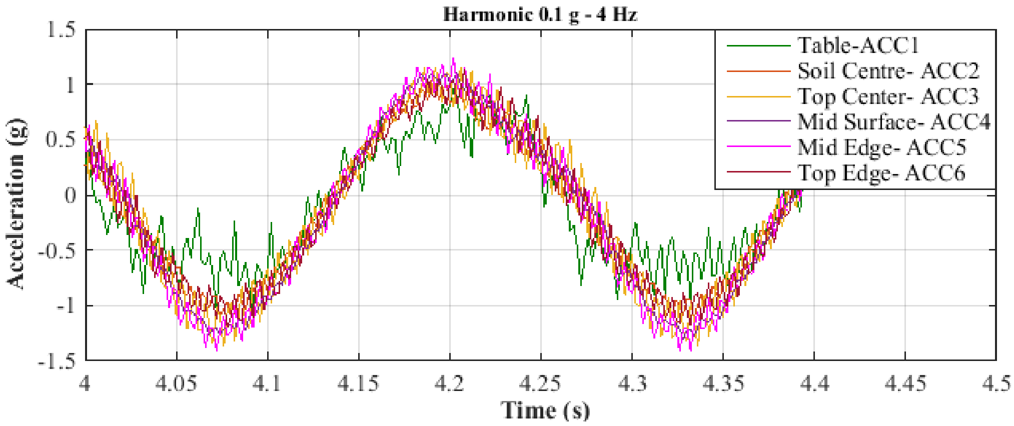


Figure 3-17 The effect of the container boundary of the soil response

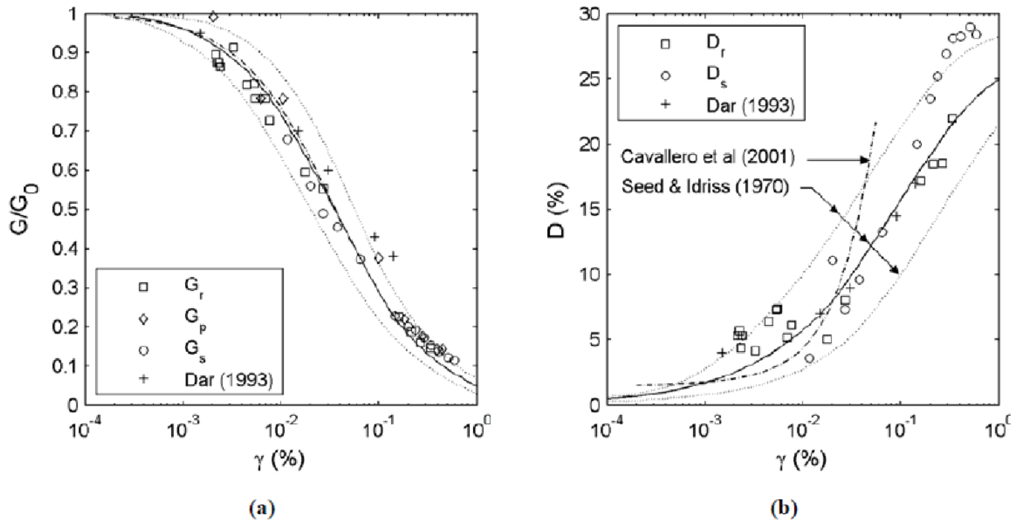


Figure 3-18 (a) The evolution of Shear modules G with shear strain γ (b) The evolution of damping Ratio D with shear strain γ (Seed et al. 1986)

Ground motion amplification of the soil container was interpreted using equation 3.7, (Seed et al. 1986)

$$\rho_{amp} = \frac{max(|\ddot{u}_{soil}(t)|)}{max(|\ddot{u}_{table}(t)|} \quad 3.7$$

Where

ρ_{amp} is the amplification factor,

and $\ddot{u}_{soil}(t)$ and $\ddot{u}_{table}(t)$ are the soil surface and table accelerations, respectively.

Hysteretic stress-strain loops are usually derived from the measurement of accelerometers response to study the nonlinear behaviour of selected sand (Zeghal, 2011) (Figure 3-18).

This procedure is summarised by Turan et al. (2009). If the soil is idealised as a one-dimensional shear beam, the shear stresses and shear strains at a particular depth can be calculated by utilising the acceleration measurements at these levels. By integrating the equation of motion using stress-free surface boundary condition, the shear stress at depth z is:

$$\tau(\mathrm{z,t}) = \int_0^z \rho \boldsymbol{\ddot{u}}\, dz \qquad 3.8$$

where

τ is shear stress,

$\boldsymbol{\ddot{u}}$ is acceleration

and ρ is the mass density.

Using linear interpolation between the acceleration measurements at different depths (e.g. from ACC1, ACC6), the discrete shear stress value at depth z is:

$$\tau_i(\mathrm{t}) = \sum_{k=1}^{i=1} \rho \frac{\boldsymbol{\ddot{u}_k + \ddot{u}_{k+1}}}{2} \Delta z_k \quad \mathrm{i}=2,3\ldots\ldots \qquad 3.9$$

where

subscript i refers to the depth z_i in (Figure 3-19),

$\tau_i = \tau(z_{i,}\mathrm{t})$,

$\ddot{u}_i = \mathrm{u}(z_i, \mathrm{t})$

and Δz_k is the soil slice thickness.

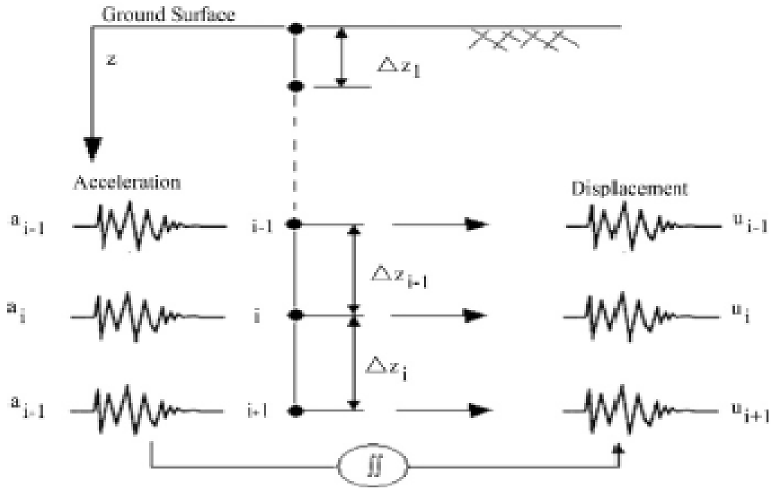


Figure 3-19 The soil response based on acceleration outputs

The corresponding shear strain value $ɣ_i$ can then be calculated in accordance with Pearson (1986) using the displacement values derived from double integration of the acceleration time-histories,viz.

$$ɣ_i = \frac{1}{\Delta z_{i-1} + \Delta z_i}\left[(u_{i+1} - u_i \frac{\Delta z_{i-1}}{\Delta z_i} + u_{i+1} - u_i \frac{\Delta z_i}{\Delta z_{i-1}}\right] \qquad 3.10$$

Where

u_i= u(z_i, t) is the absolute displacement at the level of z_i.

From the above approximations, the shear modulus and damping ratio of the soil were calculated from the shear stress–strain loops (Figure 3-18). τ'_{zy} and $ɣ_{zy}$ generally have a limiting value. Brennan et al. (2005) found the set of equations giving the best representative values of Gs and Ds in addation to the equtions of τ'_{zy} and $ɣ_{zy}$

$$D_S = \frac{1}{4\pi} * \frac{\oint \tau'_{zy} dɣ_{zy}}{(\tau'_{max} - \tau'_{min})(ɣ_{max} - ɣ_{min})/8} \qquad 3.11$$

$$G_S = \frac{(\tau'_{max} - \tau'_{min})}{(ɣ_{max} - ɣ_{min})} \qquad 3.12$$

$$\tau'_{zy} = \boldsymbol{\rho} d(\ddot{u}_d(t) + (\ddot{u}_{d=0}(t))/2 \qquad 3.13$$

$$\gamma_{zy}=(u_d(t)-u_{d=0}(t))/d \quad 3.14$$

The soil shear modulus using the secant slope and the damping ratio was calculated using the area of the corresponding shear stress-strain loop (Seed et al., 1987) as in Figures (Figure 3-20) and (Figure 3-21).

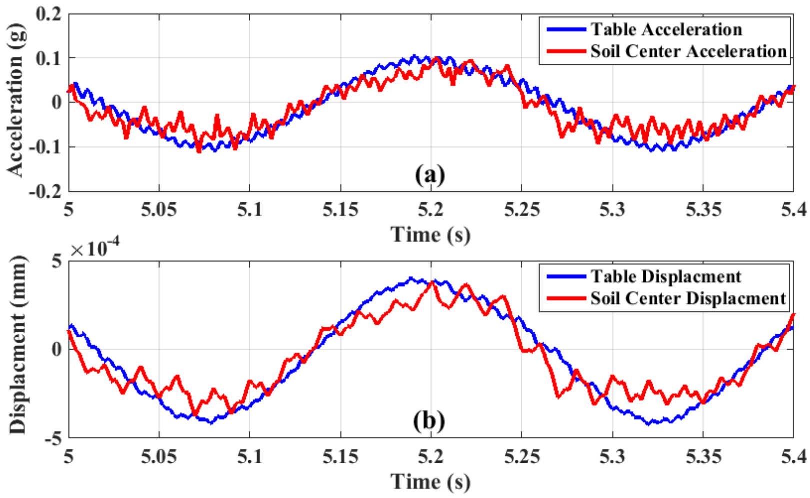


Figure 3-20 Measurement of time-acceleration and time displacement at the table and soil mass level

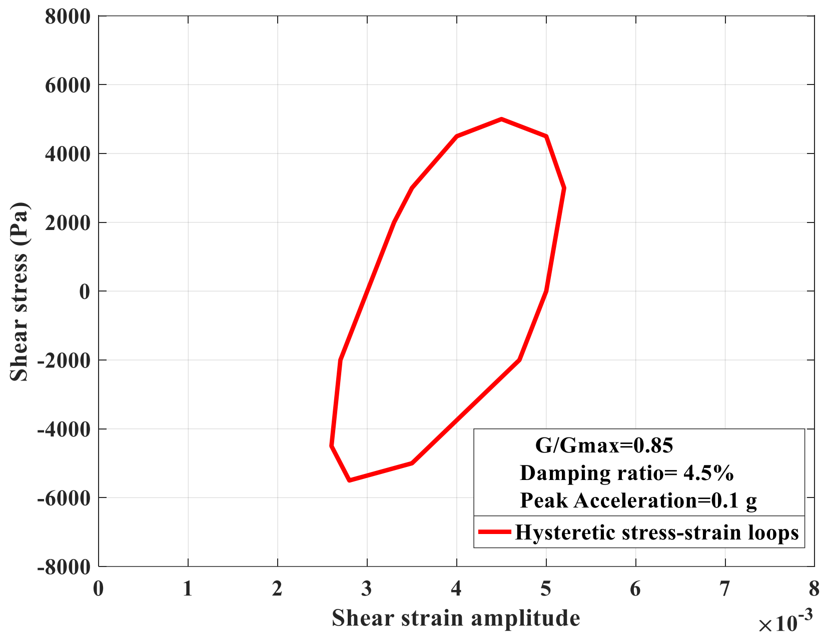


Figure 3-21 Dynamic properties of selected sand

3.6.7 Shaking Table Tests on Model Structure supported by Raft Foundation

The third stage of the shaking table tests was carried out to study the soil-structure interaction of the structure supported by raft foundation under seismic effect.

Instrumentation the structure in the soil-structure system was similar to the fixed-base structure in (Figure 3-13). The summery of stage 3 experimental tests are shown in (Table 3-8).

Table 3-8 Stage 3 structure supported by raft foundation experiments schedule

Test No.	Structure Time History	Foundation	Soil
Exp- 1	0.05 g	Raft	Sand soil
Exp- 2	0.1 g	Raft	Sand soil
Exp- 3	0.15 g	Raft	Sand soil
Exp- 4	0.2 g	Raft	Sand soil

Firstly, after the soil container was secured on the shaking table, the scaled model was embedded within soil medium 160 mm from the top of the soil surface as shown in (Figure 3-22). Then, the shaking tests were carried out as per (Table 3-8).

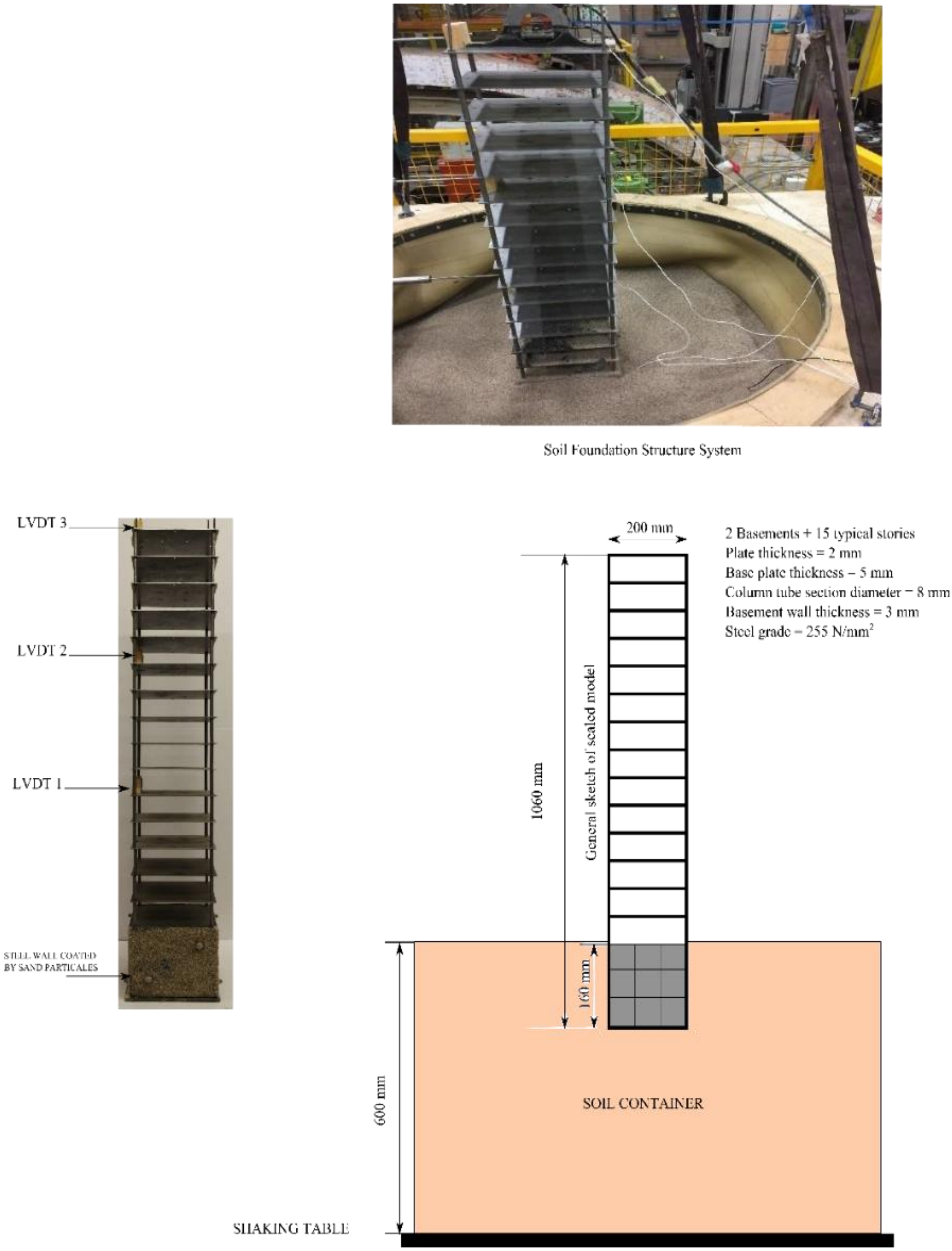


Figure 3-22 Soil foundation structure system details (raft foundation)

Initially, before starting any shaking events, the soil density was checked using small cups. The cups had a known volume to test the change in density during the shaking events. It is found that change in density was insignificant due to the soil properties.

The interaction between the soil and structure was considered as the interaction of a rough surface with a hard contact. This was archived by the sand coating on the retaining walls, and raft foundations in experimental investigations. The shaking table tests output are shown in (Figure 3-23)

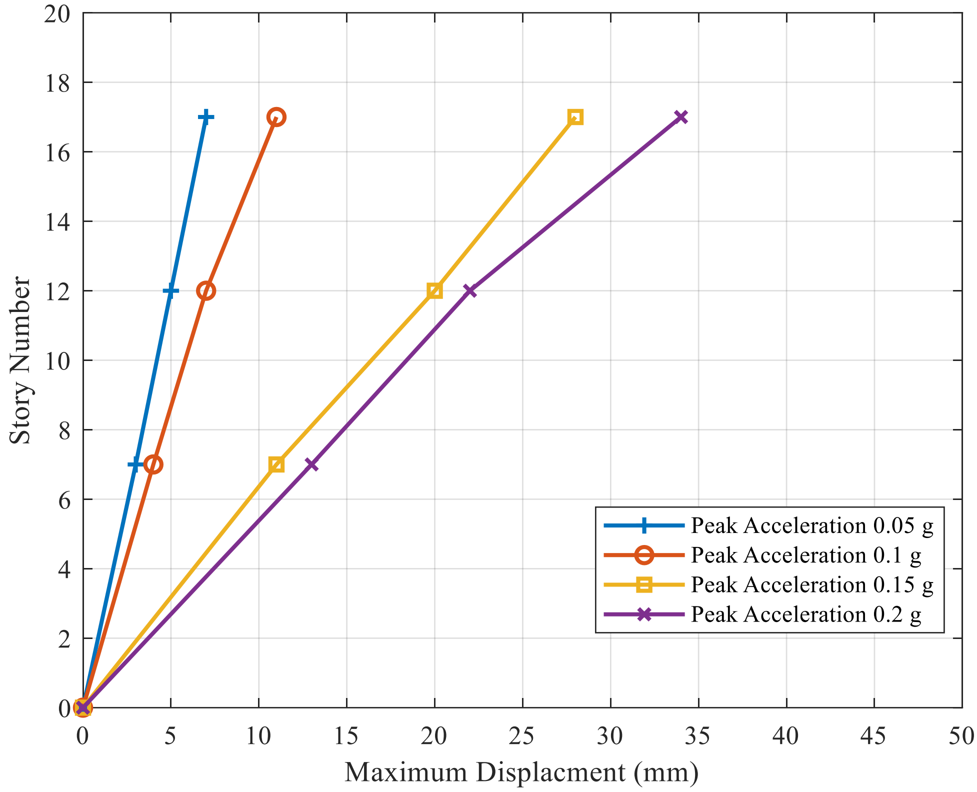


Figure 3-23 Stage 3 structure supported by raft foundation

3.6.8 Shaking Table Tests on Model Structure supported by Raft on Pile Foundation

The fourth stage of the shaking table tests was carried out to study the soil-structure interaction of structure supported by raft-on-pile foundation (4x4 group piles) under the seismic effects.

Instrumentation for the structure in the soil-structure system was similar to the fixed-base structure as in (Figure 3-13). (Table 3-9) outlines the summery of stage 4 experimental tests.

Table 3-9 Stage 4 structure supported by raft-on-pile foundation experiments schedule

Test No.	Structure Time History	Foundation	Soil
Exp- 1	0.05 g	Raft on pile	Sand soil
Exp- 2	0.1 g	Raft on pile	Sand soil
Exp- 3	0.15 g	Raft on pile	Sand soil
Exp- 4	0.2 g	Raft on pile	Sand soil

Firstly, after the soil container was secured on the shaking table, the scaled model with attached pile group (4X4 pile) was embedded within soil medium 160 mm deep from the top of the soil surface, as shown in (Figure 3-24). Then, the shaking tests were carried out as per (Table 3-9).

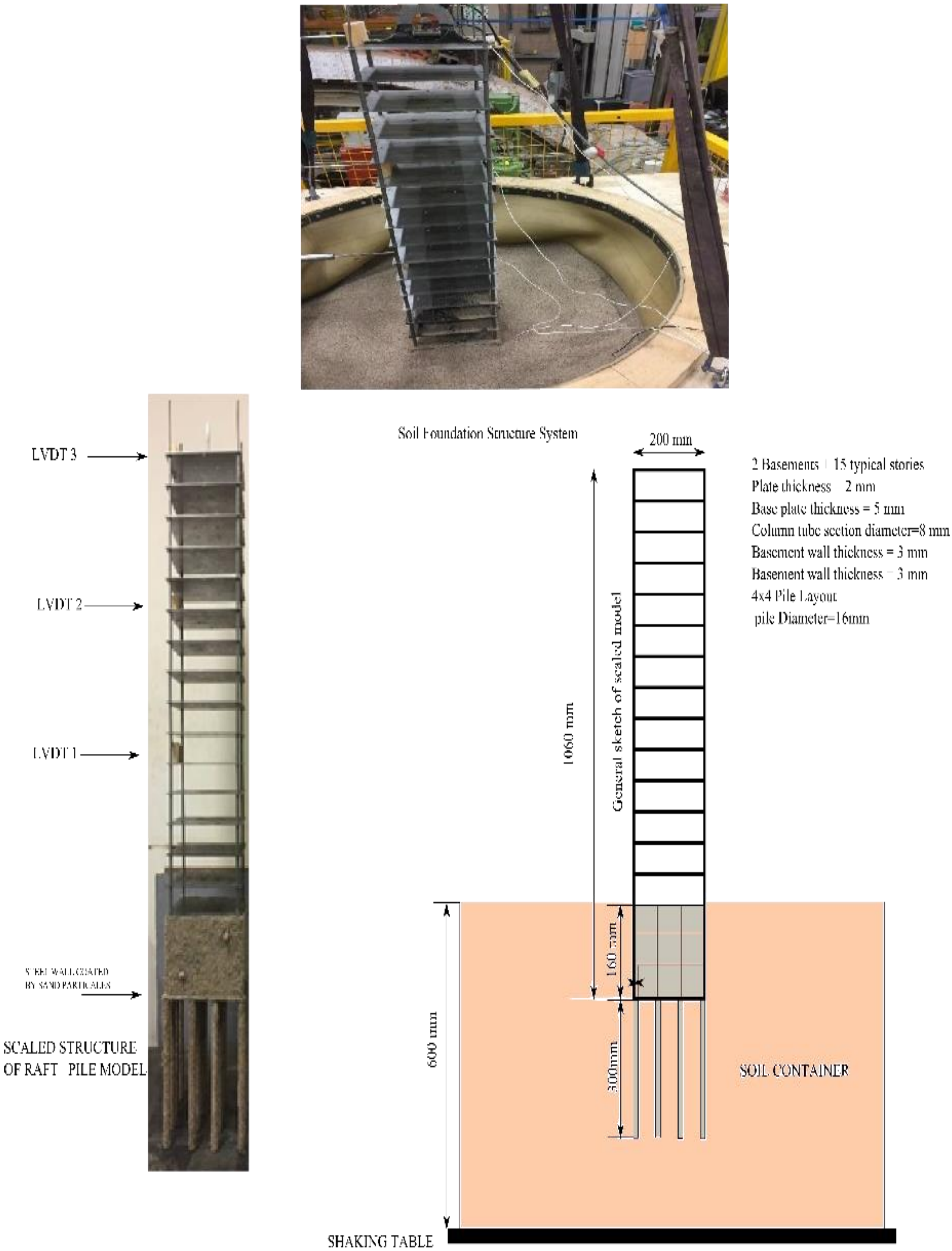


Figure 3-24 Soil foundation structure system details (raft on pile foundation)

The summary model and output results of the raft on 4x4 Pile group are shown in (Figure 3-25)

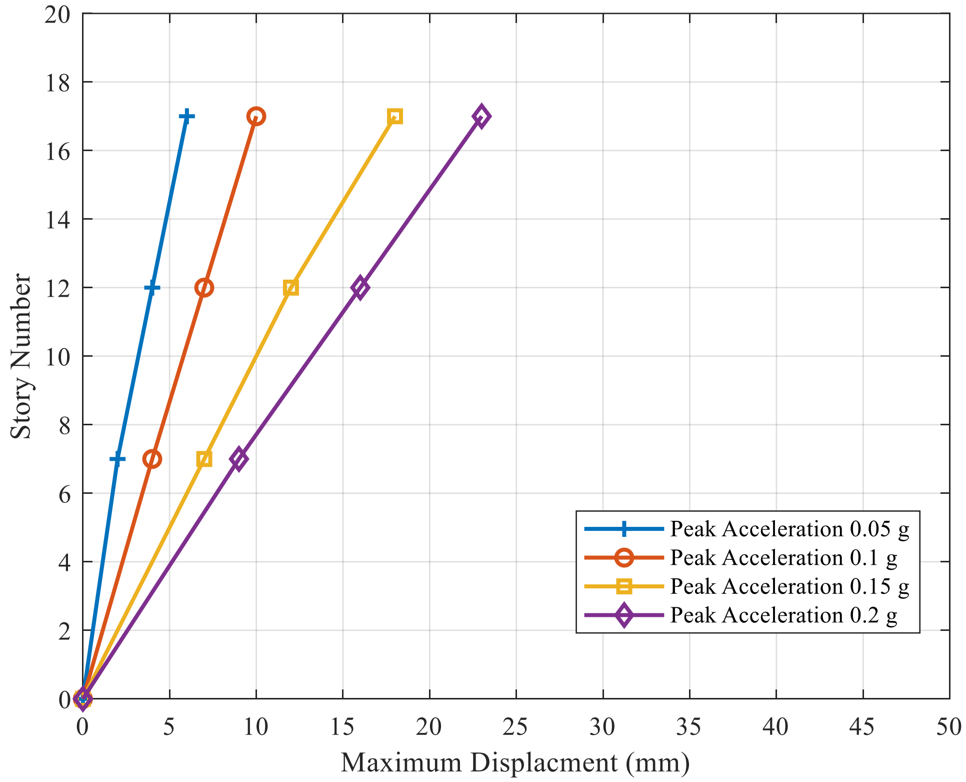


Figure 3-25 Stage 4 Structure supported by raft on pile foundation

3.7 Summary and Conclusions

The results of the shaking table tests for the maximum lateral displacements and acceleration response of the fixed-base (stage1) are presented in (Figure 3-10). These movements have been subtracted from the storey displacements with respect to the shaking table movements. Therefore, all the records are about the base movements. It should be noted that the presented data are based on the lateral deformation of each storey for the maximum lateral displacement and accelerations at the top level. This approach gives a more consistent pattern of the structural deformation in comparison with the approach where the maximum absolute storey deformation is recorded irrespective of occurrence time (Caicedo, 2012).

In this research, a comprehensive procedure for design, construction and commissioning of a scaled multi-storey building was presented for dynamic studies. Shaking table tests of four generated acceleration events were investigated on the scale model, and the results of maximum lateral deflection and acceleration were determined.

3.7.1 The shaking table tests of the soil container (stage 2)

It is well known that sand can have volumetric changes when it shears. For medium-dense soil, seismic excitations make a net contraction of the soil volume and cause the settlement of the sample surface. The tested soil void ratio of soil decreased when the soil density increased. These variations should be reflected in the calculations for stiffness and shear stress. However, for the adopted soil in question, measured contractions had a negligible effect on other parameters and the volumetric change was insignificant. That is because the selected soil for shaking table test had almost the same size particles with subrounded shape. Furthermore, the difference between the maximum and minimum soil density was small leading to minor changes in soil density during the shaking events.

Testing the experimental model is a very important issue in the research of seismic geotechnical problems because of the inadequacy of in-situ information. Therefore, a physical model is vital for simulation of semi-infinite free-field soil deposit. This thesis describes the design and performance of a flexible container, which is based on the limitation of the base shear for a small 1-g shaking table. The performance of the flexible container is evaluated using a series of model tests. The test results show that the effect of the boundary on measured accelerations is found to be insignificant.

3.7.2 The shaking table tests of the soil foundation structure interaction (stage 3 & stage 4)

Employment of raft foundation or raft-on-pile foundation is a common practice for transferring structural loads into the underlying soil layers. A series of experimental shaking table tests were conducted in this study. According to the shaking tabsle test results, the maximum lateral deflection of the structure supported by raft increases on average in comparison to the structure supported by fixed-base or raft-on-a-pile foundation. Moreover, the maximum lateral deflection of the structure supported by the shallow foundation is increased by 55% in comparison to the results obtained for the fixed base structure. Consequently, the choice of the foundation type is dominant and should be included in investigating the impact of SFSI on the response of superstructures under seismic excitations. The influence of the fioundation should be included in the conventional design procedures to achieve the structural safety and reliability.

Ignoring the actual deformability of the soil-foundation-structure system may affect the evolution of the structural damage during an earthquake. As a result, considering the effects of the soil- foundation-structure interaction can provide an alternative to the dynamic response of the superstructure.

In the past twenty years, a variety of numerical models have been developed to understand the behaviour of dynamic problems. Furthermore, more complex methods are now available to solve the complex soil-structures interaction. However, there is few experimental or prototype information to compare the results against these techniques. Prior to application of these techniques, they must be properly validated. The 1:50 scale flexible container developed in this research can offer an interesting insight into the seismic behaviour of large soil specimens. The results of the experimental investigations conducted in this chapter were employed to verify and calibrate the 3D numerical model developed in this study as explained in Chapter 5.

CHAPTER FOUR - NUMERICAL WORK

4.1 General

In order to perform an analysis of soil-foundation-structure full model, a numerical three-dimensional model was developed to investigate the soil and structure behaviour under seismic effects. The numerical nonlinear time-history dynamic analysis was performed to determine the dynamic behaviour of the soil-foundation-structure system under the effect of seismic forces. According to Liu et al. (2012), the nonlinear behaviour of soil-structure system is required for frequency domain analysis when dealing with linear and nonlinear responses. In this study, the three-dimensional software (ABAQUS) was adopted for numerical simulations. This software can be used to simulate different structure types and elemental behaviour. Those elements are possible to be fitted and adjusted to satisfy the geometrical requirements of the numerical model. The elements behave according to prescribed material properties within a constitutive model and response to boundary restraints. ABAQUS software is capable of solving complex models problems.

The soil nonlinearity follows the stress-strain law and is dependent on damping ratio. Fatahi and Tabatabei (2013) ; Beaty and byme (2001) described an overview of the soil stress-strain law method. Furthermore, Lu et al. (2012) illustrated the numerical simulation in the assessment of nonlinear soil assessment under response of dynamic loads. Accordingly, the nonlinear procedure was adopted to the model of soil-foundation-structure systems.

It should be noted that there are some other rigorous approaches to modelling soil behaviour under cyclic loads such as kinematic constitutive isotropic models (Gajo and Muir Wood, 1999), incrementally nonlinear models (Belheine et al., 2009) (Darve et al., 1995), or hypoplastic models (Chambon et al., 1994). However, the modulus reduction approach is the most common approach in modelling the soil for dynamic analysis of soil-structure systems and is employed in this study.

In this chapter, the components of the soil-foundation-structure system were numerically developed and structural elements, soil elements, pile elements, dynamic loading, and boundary conditions were explained. Due to a large number of model elements, fast computation facilities were used to create and run the developed numerical models.

The governing equations of motion were used to solve the structure – foundation-soil interactions Equation 4-1. the right-hand of this equation is about the movement of soil and structure system. Equation 4-1 consists of a combination of different vector components corresponding to the response of soil and the structure under dynamic action. This combination makes the solution by the equation mathematically complicated. Therefore, a direct numerical method is required where the soil-foundation-structure system is modelled numerically in a single step.

$$[M]\ddot{u} + [C]\dot{u} + [K]u = [-M]\ddot{u}g + F \qquad 4.1$$

Where

$[M]$, $[C]$ and $[K]$ are the structure mass, damping, and structure stiffness matrices, respectively.

u, $\dot{u}$, and $\ddot{u}$ are the model’s relative displacements, velocities and the structure accelerations with respect to the underlying ground and foundation, respectively (Abate et al., 2010), (Hou, 2012).

F is the force vector representing the viscous boundaries (Figure 4-1), and $\ddot{u}g$ is ground acceleration (Zhang and Wolf, 1998).

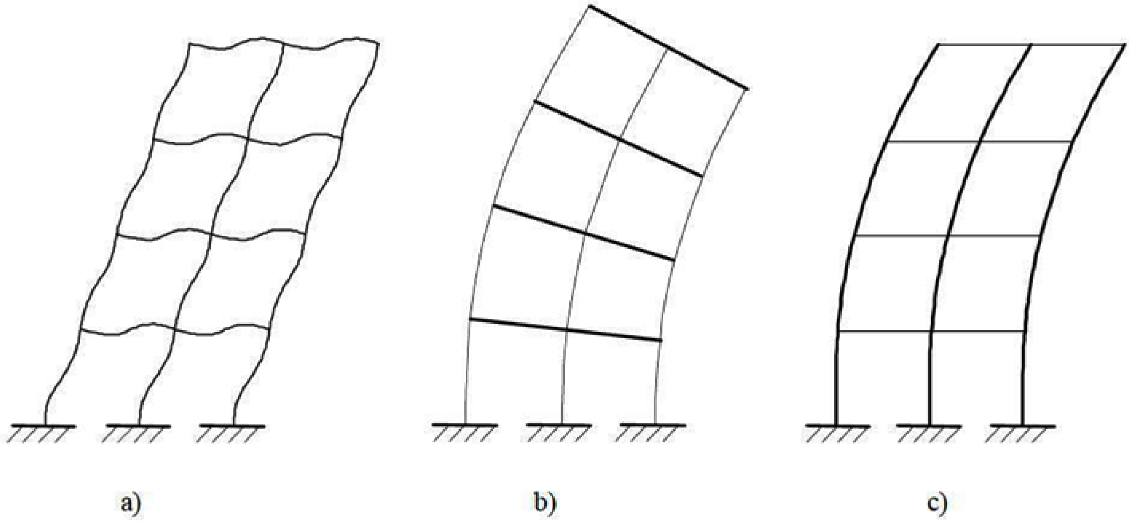


Figure 4-1 Shear deformation of multi-storey buildings under seismic force (Han and Cathro, 1997)

Numerical work was performed to validate numerical models based on the experimental measurement's tests see Appendix C. The models will be used for the parametric study of different parameters (such us building height, foundations type, subsoil reaction effect, soil properties and pile length) under seismic effect. The numerical works consisted of the following tasks:

- Designing the prototype and scaled model by ETABS software
- Building up the numerical structural model as a fixed base by ABAQUS individually (stage 1)
- Validating the output of the fixed base numerical model with the fixed base experimental tests results in stage 1 (see Chapter Five)
- Building up the soil container's numerical model (stage 2)
- Validating the output of the soil container numerical model with the soil container experimental tests
- Building up the soil-foundation-structure interaction (SFSI) model with raft foundation (stage 3)
- Validating the model for the SFSI in stage 3 with the experimental output
- Building up the soil-foundation-structure-interaction (SFSI) model with raft foundation on pile groups (stage 4)

- Validating the model for the SFSI in stage 4 with the experimental output
- Utilising the validated models for parametric studies such as structure height, basement wall-soil interaction, soil properties, and foundation types (see Chapter 6)

4.2 Finite element analysis

Solving dynamic problems of structures is a challenging subject. Alternatively, the numerical finite element method may be used. There are two commonly used finite element methods namely, the time history and the response spectrum analyses methods.

4.2.1 Time History Analysis

A time history analysis determines the equation of motion for structures subjected to an earthquake. It can simulate an earthquake's motion-time, velocity-time or displacement-time history, and they are either determined from existing recorded earthquake data or synthetically produced data.

In finite element analysis (Sun, 2010; Hughes, 1979; Amundsen, 2012) , the format of the equation of motion for time increment (i) is:

$$[\mathrm{M}](\ddot{\mathrm{u}})_i^{tot} + [\mathrm{C}]\,(\dot{u})_{\,i} + R_i^{int} = \{F_i\} \qquad 4.2$$

Where

M is the structure mass,

C is the structural damping,

K is the structure stiffness coefficient,

$\mathrm{R_i^{int}}$ is the internal force of linear numerical analysis which is calculated as:

$$\mathrm{R_i^{int}} = \left[\mathrm{K_i^k}\right]\mathrm{u_i^k} \qquad 4.3$$

F_i is the lateral force vector, and u, u̇, and ü are the relative displacement, the velocity, and the accelerations of the structure, respectively?

Furthermore, subscript i represents the iteration (time) increment. $\mathrm{\ddot{u}_i^{tot}}$ is the total acceleration of the numerical model given as:

$$\ddot{u}_i^{tot} = \ddot{u}_i + \ddot{u}_{g,i} \qquad 4.4$$

Where

$\ddot{u}_{g,i}$ is the ground acceleration.

The stiffness matrix (K) of nonlinear analysis is updated for each increment, i.

Furthermore, Newton-Raphson iterations (k) are required to modify and correct the equilibrium path of each increment as shown in (Figure 4-2).

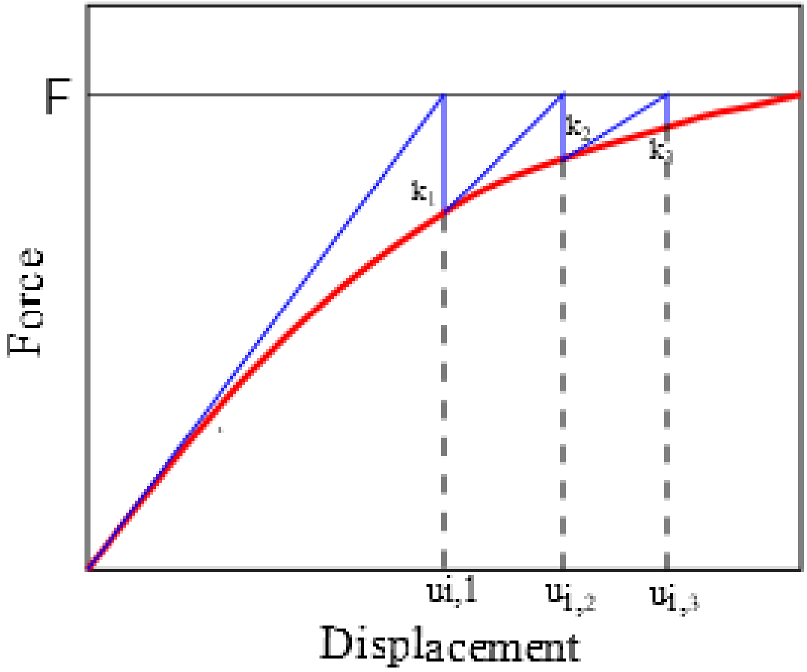


Figure 4-2 Newton-Raphson iterations used in nonlinear analysis (number of iterations, *k* =3)

Moreover, rearrangement of the ground motion equation gives:

$$\mathrm{M}\ddot{\mathrm{u}}_\mathrm{i} + \mathrm{C}\dot{\mathrm{u}}_\mathrm{i} + \mathrm{K}_\mathrm{i}^\mathrm{k}\mathrm{u}_\mathrm{i}^\mathrm{k} = \mathrm{F}_\mathrm{i} - \mathrm{M}\ddot{\mathrm{u}}_{\mathrm{g,i}} = \mathrm{R}_\mathrm{i}^{\mathrm{ext}} \qquad 4.5$$

Where

$\mathrm{R}_\mathrm{i}^{\mathrm{ext}}$ is the externally applied force.

The structural displacement is obtained by integrating the equation 4.2 using either explicit or implicit integration methods. In the explicit formula, the previous data step i, is used to determine an equation for each time increment of $\mathrm{i}+1$. In order to obtain the final and stable solutions, small time increments are required. Thus, for an earthquake which typically lasts about 10 to 35 seconds, literally millions of time steps are needed (Sun, 2010; Hughes, 1979; Amundsen, 2012). This means an explicit integration is not accurate for long times.

The implicit integration method solves the equation 4.5 for each time step interval $i+1$ by creating system's equations. This method takes a long time to solve a single increment and the outputs of the implicit solution are more stable than explicit (Sun, 2010). An implicit method called Hilbert-Hughes-Taylor's (HHT) is used in this study (Hoff and Pahl, 1988). The numerical differential equations are combined using the Newmark-Relations (Hoff and Pahl, 1988) as follows:

$$\ddot{u}_{i+1} = \frac{1}{\beta\Delta t^2}(u_{i+1} - u_i - \Delta t\dot{u} - 1) \;\; - \left(\frac{1}{2\beta}\ddot{u}_i\right) \qquad 4.6$$

$$\dot{u}_{i+1} = \frac{\gamma}{\beta\Delta t}(\,u_{i+1} - u_i) - \left(\frac{\gamma}{\beta} - 1\right)\dot{u}_i - \Delta t\left(\frac{\gamma}{2\beta} - 1\right)\ddot{u}_i \qquad 4.7$$

Combining equations (4.6) and (4.7) with equation (4.8):

$$[\mathrm{M}]\ddot{\mathrm{u}}_{\mathrm{i}+1} + (1{+}\propto)\mathrm{M}(\ddot{\mathrm{u}})_{\mathrm{i}} {-}\propto [\mathrm{M}]\ddot{\mathrm{u}}_{\mathrm{i}} + (1{+}\propto)\left[\mathrm{K}_{\mathrm{i}}^{\mathrm{k}}\right]\mathrm{u}_{\mathrm{i}+1} {-}\propto \left[\mathrm{K}_{\mathrm{i}}^{\mathrm{k}}\right]\mathrm{u}_{\mathrm{i}}$$

$$= (1{+}\propto)\mathrm{R}_{\mathrm{i}+1}^{\mathrm{ext}} {-}\propto \mathrm{R}_{\mathrm{i}}^{\mathrm{ext}} \qquad 4.8$$

where $\frac{-1}{3} \le \propto \le 0$ is a numerical damping control. The damping parameters are given as:

$$\beta = (1{-}\propto)^2 \qquad 4.9$$

$$\gamma = \frac{1}{2}\,(1 - 2\propto) \qquad 4.10$$

4.2.2 Response spectrum analysis

For large finite element, when nonlinear behaviour of soil structure response analyses is not expected, the equation of motion analysis is too slow in the application of time ingration. Alternatively, the frequency domain approximative approach can be used, which is the so-called Response Spectrum analysis. The transfer function y of the maximum displacement of a given mode n, in given direction i, can be calculated from (Bathe, 2006; Ben, 2013):

$$|y_n|_i^{max} = \frac{\Gamma_n}{{\omega_n}^2} \mathrm{SA}(\omega_n) \qquad 4.11$$

Where

$\omega_n$the frequency of a given mode, with n is separated model equations of a multi-degree of freedom system, $\Gamma_n$is the max displacement

and SA is the response acceleration.

From the peak displacement vectors of the result set N, the force, moment and stress can be determined. The finite element model of this integration is not simple. The major approximation is the modes combination of the response spectrum analysis. Several combination methods with different implications are available. There three main combination methods of the absolute sum of modal peak values as follows:

The first equation is:

$$u_i^{max} = \sum_{n=1}^{N} |u_n|_i^{max} \qquad 4.12$$

This equation is the most conservative approximation correlation of the structure response where the different structure modes all have their effect at the same time. It is obvious that such an estimate can yield a structural response that is much higher than those from an equivalent time history analysis.

The second equation is the square root of the sum of squares method (SRSS):

$$u_i^{max} = \sqrt{\sum_{n=1}^{N} \left(u_{n,i}^{max}\right)^2} \qquad 4.13$$

The assumption of this approach is that there is no correlation between modes. The output of this produces better results than equation 4.14 if the modes are well separated. Also the results of this approach are unconservative since the modes are closely packed - which is the case for three-dimensional structures, (Bathe, 2006; Ben, 2013).

$$u_i^{max} = \sqrt{\sum_{n=1}^{N} \sum_{m=1}^{N} u_{n,i}^{max} \rho_{mn} u_{m,i}^{max}} \qquad 4.14$$

Where

coefficient ρ_{mn} is derived from the random vibration theory.

To determinte the seismic action in three directions, a response spectrum has to be defined in all directions, thus introducing the problem of directional combination.

Another conservative approach is an algebraic summation of displacments, which results in the total displacement as follows:

$$u_i^{max} = \sum_{i=1}^{3} u_i^{max} \qquad 4.15$$

As a conclusion, a response spectrum analysis problem is where all modes of measurement has been performed simultaneously. In equation 4.15 the displacement is calculated by assuming that all structure modes are appiled simultaneously. Furthermore, the response spectrum uses the absolute values which lose all the information about signs. The dynamic problem can be analysed in two different ways such as the time-history and the response-spectrum analyses methods. For a multi-storey building with large number of finite element analyses, where nonlinearities are expected, it is too slow to employ time-history integration for solving the equation of motion. Alternatively, the frequency domain with an approximate approach can be used, which is the so-called response-spectrum analysis. However, only a single value (maximum value) can be obtained from the response-spectrum analysis. In this study, the main concern was to understand the behaviour of overall structural behaviour and soil-foundation-structure interaction. Therefore, the time-history analysis method was adopted in this study (Bathe, 2006; Ben, 2013).

4.3 Three-dimensional finite element software (ABAQUS)

ABAQUS is a flexible tool utilised for the finite element analysis method. This analysis method is allowing the user to model and analyse the impact of time variation on the load by defining step procedures. The simplest step in ABAQUS is the static step analysis where the time factor does not affect the load magnitude. In each “step” the user can choose an analysis procedure such as eigenvalue buckling, dynamic stress analysis etc.

The procedure can be employed to monitor the changes from step to step since the model state is updated throughout all steps of the analysis. Furthermore, the effect of the previous analysis step is reflected in each new step response. ABAQUS (Standard) software can solve both linear and nonlinear response options. Using the nonlinear step procedures in ABAQUS software, the user can control increments in size or can define the step with tolerances or error measures. Then the increments will automatically develop the response in that step. Automatic control is particularly valuable in cases where load increment varies widely within the analysis step. The main difference is the stability limit that controls the time increment.

To achieve an accurate numerical model, the models are required to be defined properly in terms of strain definition, material equations and motion laws. The resulting mathematical expressions are a set of partial differential equations which are relating the time history to output.

The numerical solutions were performed by the finite element software ABAQUS. The structural system was designed based on Eurocode and British standards. Several models were created and analysed namely, the fixed-base model, soil container model, the soil-raft foundation structure model and soil-raft-on-pile structure model. The structural element was using an elastic constitutive model while the nonlinear soil model was adopted in the simulation of the soil element considering the Rayleigh damping equation (Ryan and Polanco, 2008).

$$[C] = \alpha [M] + \beta [K] \tag{4.16}$$

where [C], [M], and [K] are the soil structural damping, structural mass, and structural stiffness matrices, respectively;
and α and β are the model coefficients. The damping ratio (ξ) can be calculated from the frequency structural mode shape then the model coefficients α and β can be determined from Rayleigh damping equation 4.2 (Chopra, 2007).

4.4 Elements in ABAQUS

There is a wide range of elements in the ABAQUS element library offering a variety of element types for modelling systems with different geometries. The elements can be evaluated by considering the following groups Family, Degrees of freedom, Number of nodes, Integration and Formulation. The elements of ABAQUS software are shown in (Figure 4-3).

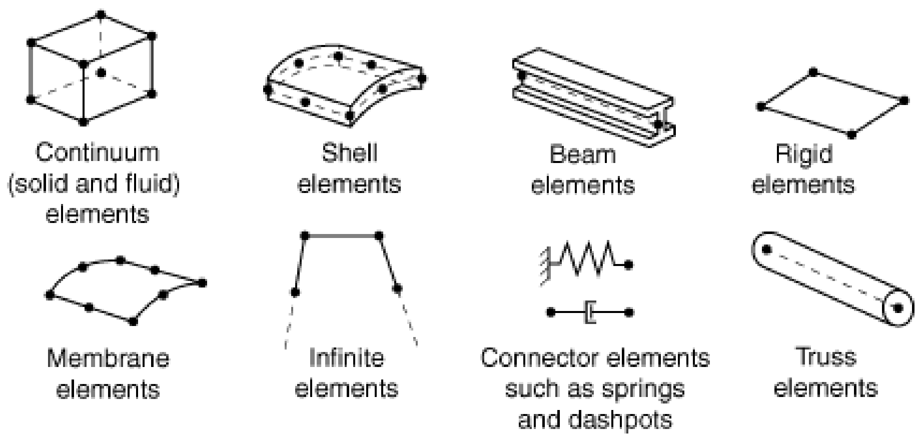


Figure 4-3 ABAQUS elements (ABAQUS documentation, 2013)

In ABAQUS, when performing nonlinear analyses of the finite-shell elements, the changes in strain for cross-sectional thickness are based on a Poisson's ratio calculated as follows.

4.4.1 Solid element:

C3D8R, is an 8-node linear hexahedral element type. The C3D8R element is a reduced integration of brick element (ABAQUS Documentation, 2013). The shape function is shown in (Figure 4-4).

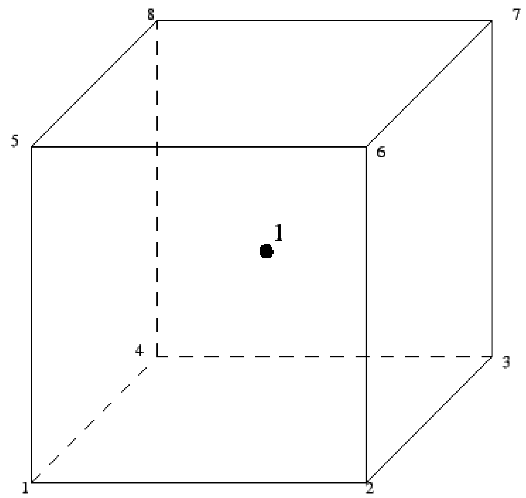


Figure 4-4 hexahedral C3D8R elements with integration point scheme (ABAQUS Documentation, 2013)

shape factor criterion is available only for triangular and tetrahedral elements. The shape factor ranges from 0 to 1, with 1 indicating the optimal element shape and 0 indicating a degenerate element.
The general criteria for this element are:

- The element is not stiff enough under action
- Stresses and strains of the integration points are more accurate than shell element.
- The C3D8R integration point is located in the middle of the elements

4.4.2 Beam element

A B31 beam element within the three-dimensional model is a one-dimensional line element. The beam stiffness is associated with line deformation such as bending and axial stretch. The main advantage of the beam elements is geometrically simplicity and having few degrees of freedom. The beam deformation can be calculated entirely by the variables located along the beam axis only. The applied forces and moments of beam elements are clarified in (Figure 4-5).

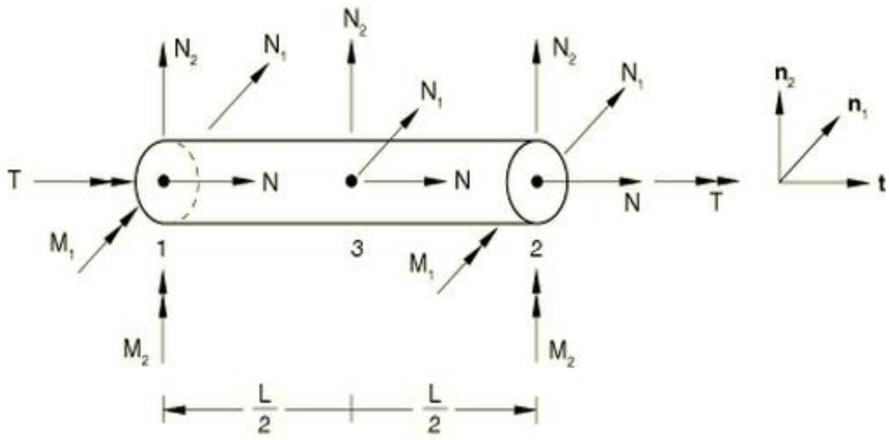


Figure 4-5 ABAQUS beam element B31 (ABAQUS Documentation, 2013)

In Figure (4-5) N is force, M is the applied moment and L is the beam length.

4.4.3 Shell element

The general purpose of shell elements S4R three-dimensional elements is to solve shell problems for all loading conditions in thin and thick shell parts. The in-plane thickness change is a function of element deformation (Figure 4-6).

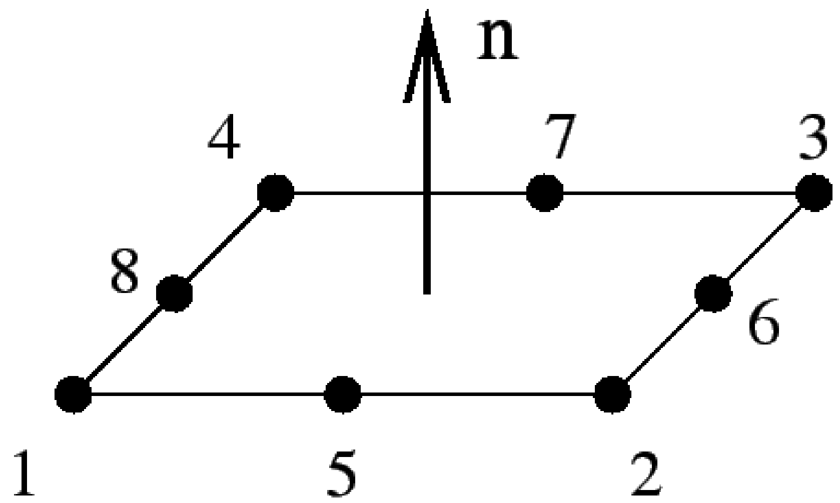


Figure 4-6 ABAQUS shell element (ABAQUS Documentation, 2013)

The ABAQUS element S4R is well suited for many impact dynamics solutions including buckling behaviour, which is involved with small-strains. These elements are used to simplify calculations of strain and hourglass control which provides a significant advantage in computational speed.

Thin shell element provides enhanced performance for the solution of large problems facilitating the reduction of the number of degrees of freedom (Kabir et al., 2016).

4.5 Soil boundary condition

Kouroussis et al. (2009) modelled the boundary as an infinite elements as shown in (Figure 4-7) (a). This element seems that they absorbed a part of the seismic excitation is to prevent lateral reflection of the seismic waves. Further studies shows that this boundary condition would be suitable for the prototype model than a scaled modelled tested in the laboratory condition. Dashpot element was used in the literature to model the boundary. The dashpot elements absorb the propagating waves in such a way that any incident wave produces zero energy being reflected back into the domain. The dashpot coefficients are determined in terms of the material properties of the semi-infinite domain, as shown in (Figure 4-7)(b).

After many trial and errors, it was found that the flexible membrane element outputs are more acceptable values in comparisons with the experimental. In the experimental investigation, the flexural container with stiffened by steel rings was used to simulate the adjacent soil condition.

The properties of the flexible membrane and the stiffening rings were smeared, and thus flexible plate element was used simulate the container wall in this numerical study. 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. The flexible wall was proposed to represent the viscous behaviour of the soil container, and this wall has a tie connection with soil to ensure the flexible boundary of the soil container (Figure 4-7)(c).

This wall helps the soil to be deformed and dispute the shaking energy to reduce the eaves reflection impact on the soil response within the soil container.

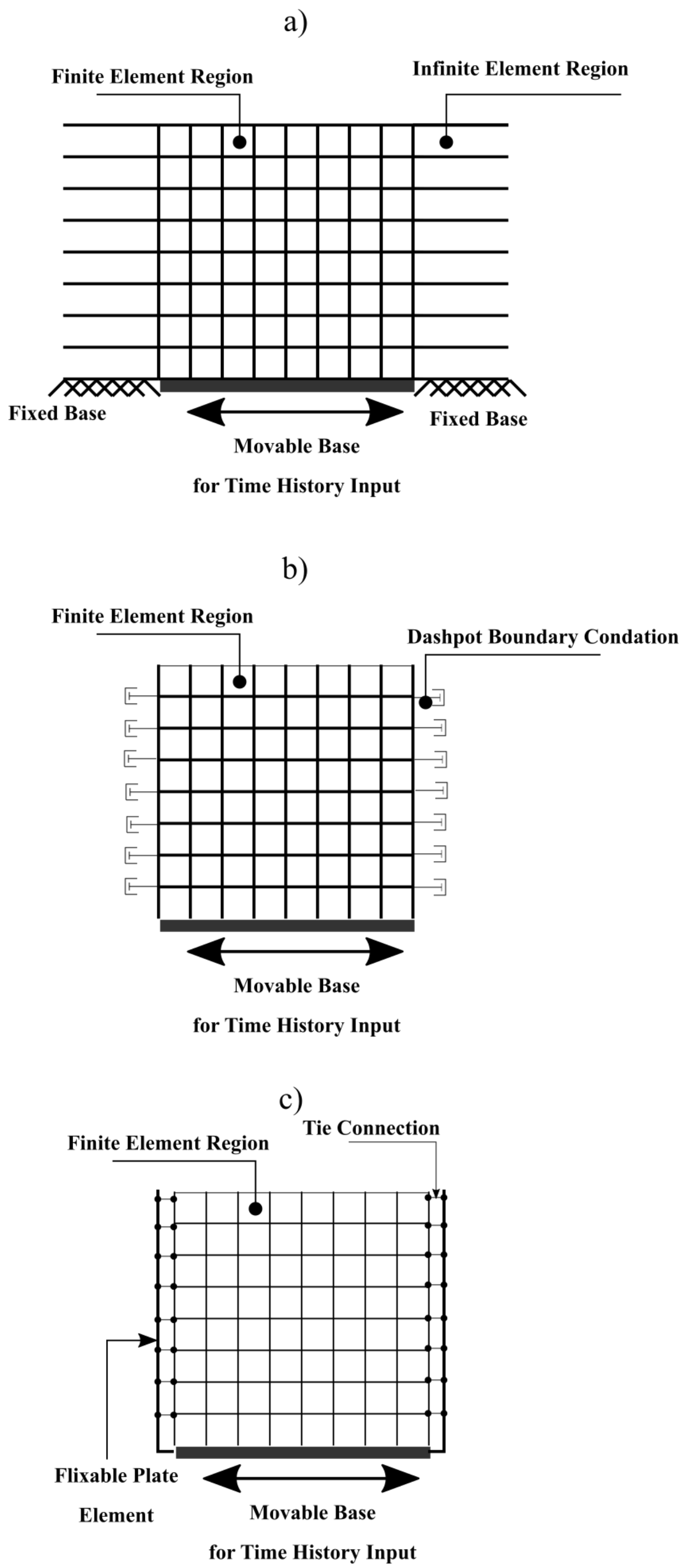


Figure 4-7 Numerical simulation of the soil boundary condition a) infinite boundary condition, b) dashpot boundary condition c) flexible plate element boundary condition

4.6 Three-dimensional models of soil-structure system

3D-models were developed to represent soil depth, Pile dimensions and soil properties from the experimental tests were adopted into the models. The model parts, such as columns, slabs, piles and soil, were defined with material properties of parts of each element. Piles were assumed to have linear elastic behaviour. In ABAQUS/CAE program a mesh of model type C3D8R (an 8-node linear hexahedral element) was used for walls, soil and soil container while S4R elements types were used for slabs, foundation and base.

Beam element types (B31) were utilised for columns and pile. The total number of elements was 12410. The interaction between surfaces is set. Rough tangential interaction means that there is no slip between surfaces in contact.

The interaction between the soil and the raft was assumed as a rough interaction to represent the adhesion between the sand and the raft surface, while the interaction between the pile and soil was assumed as an impended element within the soil medium. The connection was utilised as a connection between outer surface of soil and the soil container boundary wall (Figure 4-8).

Numerical simulations were carried out using the ABAQUS finite element software on the scaled model structural system. Running 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 running time. A numerical model was created and analysed for various seismic intensities. All parts were assumed to have linear elastic material behaviour to eliminate the influence of structural plastic deformation.

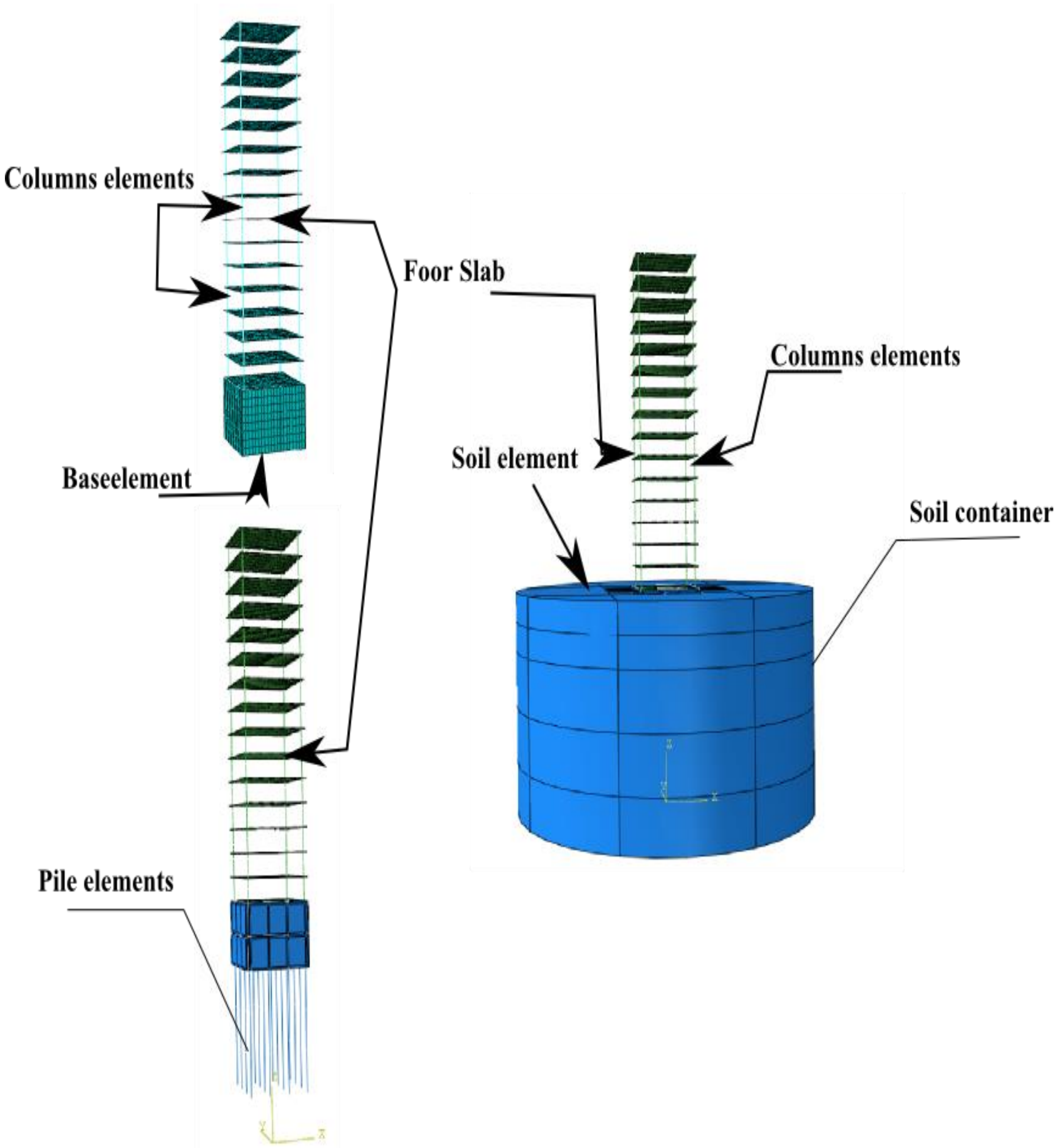


Figure 4-8 Numerical elements modelled by ABAQUS

Within ABAQUS model the soil medium was represented by nonlinear solid elements. The soil element behaves according to linear or nonlinear stress/strain law. Accordingly, a proper constitutive model representing the geomechanical behaviour of soil elements was investigated and proposed in ABAQUS to conduct an accurate SFSI analysis.

Mohr-Coulomb nonlinear model was adopted in simulation of the soil behaviour and shear failure during the shaking excitations (Figure 4-9).

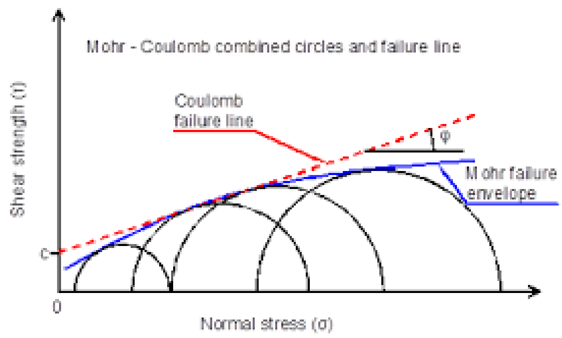


Figure 4-9 Mohr-Coulomb model

The Mohr-Coulomb soil model is a nonlinear elastic-perfectly plastic model. Many researchers (Conniff and Kiousis, 2007), (Rayhani and El Naggar, 2008) adopted Mohr-Coulomb model to simulate the soil elements under seismic effects.

The failure envelope within Mohr-Coulomb soil criterion is shear yield function and tension cut-off (tension yield function). Soil element dimensions was 1000 mm diameter and 600 mm depth. Dilatancy is a volume change that could occur when the soil is subjected to shear. An example of dilatancy for dense sand is shown in Chapter 3 (Figure 3-5). Dilatancy is measured with the angle of dilation, which is dependent on the amount of developed plastic volumetric strain during yielding as $\omega = \phi - 30$.

4-1 Soil properties defined in ABAQUS Software

Soil Properties	Denote	Unit	Value
Mass density	ρ	kg/m^3	1500
the angle of friction	ϕ	°	34
Poisson's ratio	v		0.33
Young's modulus	E	N/m^2	80E6

Symmetric boundary conditions (symmetry in the z-direction) was applied on the corresponding boundaries of the wellhead and the soil. It means no deformation in the z-direction and no rotation about the x-and y-axes (circumferential boundaries of the soil constrained in all degrees of freedom) (Figure 4-10).

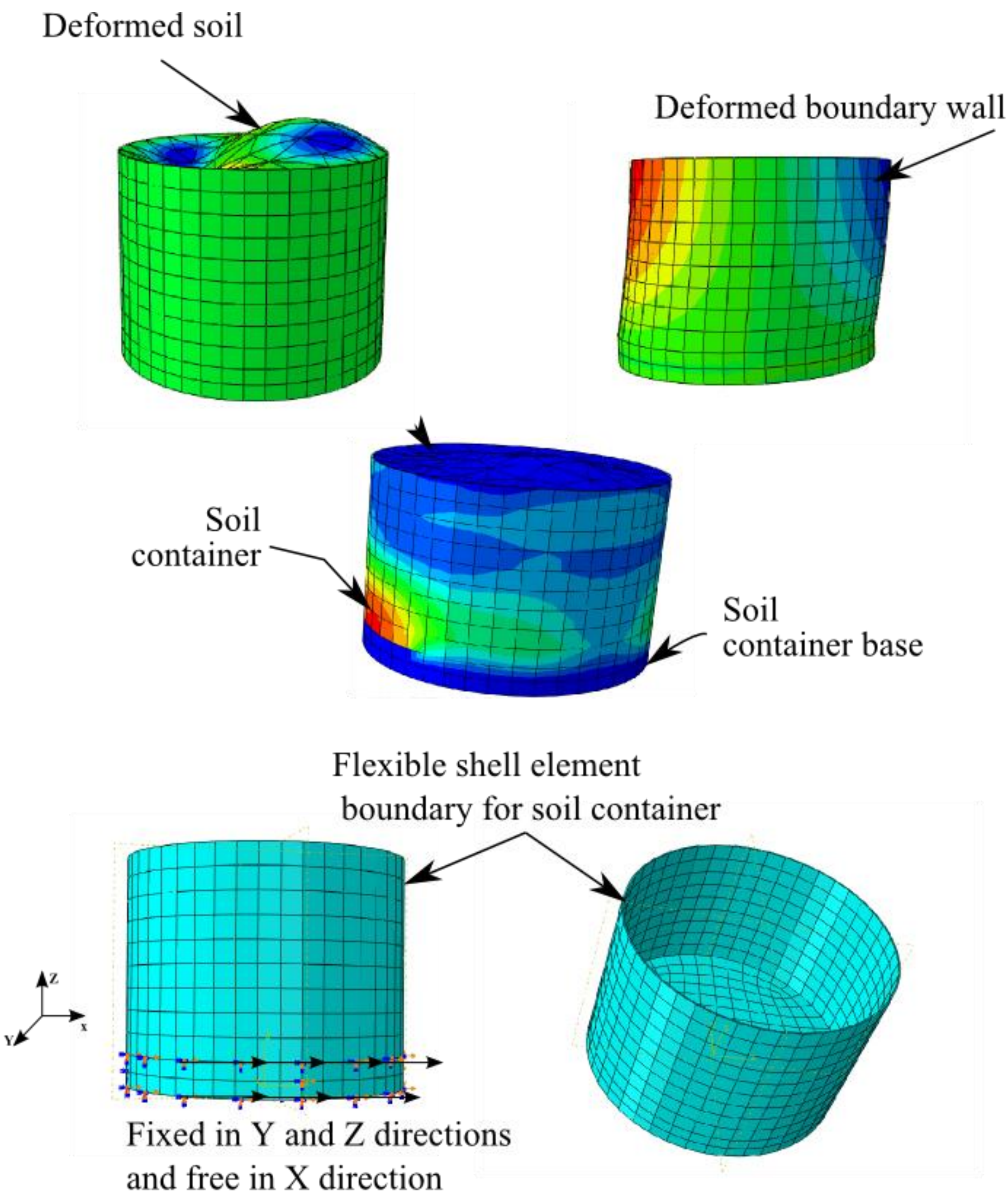


Figure 4-10 Soil element boundary model by ABAQUS software

Dynamic analysis

A dynamic (implicit) step was used to apply the seismic ground motion. The boundary condition was implemented to the soil container base by time displacement boundary condition allowing the whole system to move horizontally at given time displacement values. Four ground motions were generated representing a 0.05g, 0.1, 0.15 and 0.2g peak accelerations.

The dynamic problem of real structures is a difficult task. Therefore, finite element methods are commonly utilised for either time history or response spectrum analysis.

A finite element method was used for advanced numerical analysis and calculations. It was developed from the continuum mechanics theories, in which the equilibrium, motion and deformation studies of physical solids are taking place. ABAQUS is a powerful FEM tool for analysis and solving 3D problems and capable of running complex-harmonic analyses. In this thesis, ABAQUS/CAE version 6.13 was used. Analysis using ABAQUS involves two major procedures, viz pre-processing and post-processing.

4.7 Pre-processing

In order to create the model with ABAQUS the following sequence of steps can be followed:

- Creation of parts specifying the model geometry
- Defining the part material together with section properties
- Parts assembly
- Configuring the step analysis
- Defining the interaction properties
- Assigning the interaction of contacted parts elements
- Defining the boundary conditions and applied loads
- Designing the element mesh
- Creating a job, checking, running and monitoring a job

Procedures discussions and assumptions were made for modelling the structure, foundation, soil and pile system:

- Creating a model geometry

The first step creates the model geometry by identifying the parts of model element set up as a three-dimensional, deformable body.

- Defining the part material and section properties

The Second step defines the part materials and assigning element parts material and section of the created part.

- Model parts assembly

In this stage, the created parts were oriented in their coordinate system. One assembled model consisted of one or may parts. The assembled model was defined by creating instances of a part and then positioning the instances of parts into a global coordinate system.

The implicit step analysis steps in ABAQUS can be used to analyse linear or nonlinear response under the effect of the dynamic forces.

- Assigning the interaction properties

Interaction is defined as the contact between surfaces of two parts and consists of two components: the first one is normal to the surfaces, while the second one is tangential to the surfaces. The tangential component is related to the relative motion (sliding) of surfaces or frictional shear stresses.

In ABAQUS, the contact constraint is activated when the clearance distance between surfaces is zero. The hard contact refers to when surfaces are separated, and the constraint is removed when the contact pressure between surfaces becomes zero or negative.

The rough interaction is assumed when the system is subjected to a small force and no slip is induced. Thus, the hard and rough contacts are used for the tangential behaviour and the normal behaviour in all interactions, respectively. A modal damping ratio is introduced with a default value of 5 % (Simulia, 2013).

- Mesh design

The ABAQUS Mesh module tools allow generating meshes on the part element or assembled parts model created within ABAQUS software. In the numerical model of soil-foundation-structure system, meshing of structural parts elements was performed. Mesh module within ABAQUS software provides control over the element meshing to create particular mesh model topologies, and also optimises the size of mesh to obtain reliable results.

- Creating jobs, checking, running and monitoring a job.

After finalising the model definitions, the model is run and analysed by ABAQUS Job module. The ABAQUS Job module allows the submitted job to be monitored during the analysis progress.

- Post-processing

As the ABAQUS software performs calculations, it continuously results in output data to an output database in terms of a .odb file. This output file contains an enormous amount of output data which is required to obtain the relevant information for data analysis.

- Structural movement

The analysis of physical movements of the structure during the dynamic excitation is required to be determined . Therefore, the horizontal displacements of a selected node within the structural elements were investigated in the .odb output file. The total displacement of the structure model was obtained from the total difference in horizontal movement between the measured floor and base level.

In the soil-foundation-structure SFSI models, the structural model rotation can be calculated from ABAQUS model, at any given point in the analysis to obtain the maximum lateral deflection from subtracting the structure movement from the model base in fixed base models and the base of the soil container in soil-structure system.

4.8 Fixed-base response model

The fixed-base structural elements model consists of structural elements only. This model was used to validate the characteristics of the structural elements of numerical models under seismic effects without the effect of soil interaction. The numerical model dimensions are as same as the adapted experimental model (Figure 4-11).

Figure 4-11 Fixed base numerical model by ABAQUS

4.9 Soil-structure interaction models

In the soil foundation, structure interaction model consisted of the fixed-base model described in the previous section placed in a 600-mm thick of uniform soil deposit. The soil in this case is dense sand with properties described in section 4.5.2. As mentioned earlier, this is the basic model for determination of the soil-structure interactions (Figure 4-12).

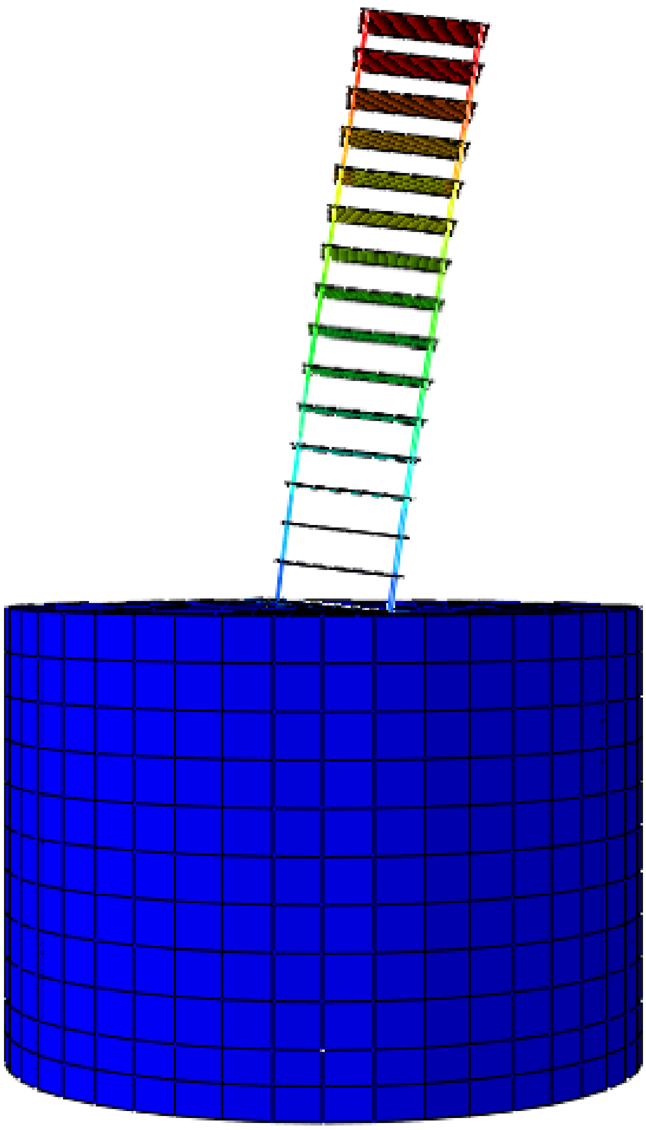


Figure 4-12 Soil foundation-structure interactions model by ABAQUS

4.10 **Summary**

In this chapter, in order to investigate the influence of SFSI on the behaviour of superstructures under seismic loads, characteristics of the 3D numerical model developed using ABAQUS was described. The numerical model performs soil-foundation- structure interaction (SFSI) analysis as an entire soil structure system, without resorting to independent calculations of soil or superstructure response. The main feature of the developed numerical model is incorporating the nonlinear behaviour of soil together with linear behaviour of structural elements simultaneously throughout the three-dimensional numerical analysis.

Nonlinear Mohr-Coulomb soil model was adopted in this study to simulate the nonlinear soil response during the dynamic excitations. Solid nonlinear elements were adopted to simulate soil elements, while linear solid elements were used for wall elements. Columns and pile were modelled as beam elements. Slabs and base were simulated as shell elements. Also, plastic adjusting of the boundary conditions were considered to represent the soil container. In fixed base model, the bottom face of this model was fixed in all directions except the direction where the dynamic force is applied. In order to avoid reflection of outward propagating waves back into the model during the dynamic time-history analysis, a flexible shell element was adopted to simulate the soil boundaries.

Due to different characteristics of soil and superstructure/piles, sliding and separation may occur at the soil-structure interfaces. Two sets of interactions were modelled in this study. For the raft foundation case, the interaction elements were placed between the foundation and the soil surface while, for the pile foundation case, the pile element was simulated as impeded within the soil medium and the interface elements. In this study, the developed 3D numerical model was used to simulate and investigate the influence of the soil-foundation-structure interactions on the seismic response of structure. The proposed soil-Foundation-structure numerical models will be verified and validated against the results of experimental shaking table test (Chapter 3).

CHAPTER FIVE - VERIFICATION OF THE DEVELOPED 3D NUMERICAL MODEL

5.1 General

In order to provide a specified analysis performance level for a structure at a reasonable cost, conducting accurate analysis to account the entire soil-foundation-structure system is required. For this purpose, efficient analytical tools amenable for use by both structural and geotechnical engineers are required.

In this chapter, to assess the capabilities of the developed numerical model in simulating of the soil structure interaction, the results of the conducted shaking table tests (Chapter 3) has been employed to verify and calibrate the developed numerical model by ABAQUS. Accordingly, the scaled three-dimensional, namely:

- fixed-base structure representing the situation individually without the soil-structure interaction,
- soil container numerical model
- the structure supported by the shallow foundation on sand soil,
- The structure supported by raft on pile group in the sand soil. Those models are simulated numerically, and the output results are compared with the experimental measurements.

The developed 3D nonlinear numerical model accounts for the various phenomena observed in SFSI experimental study, providing a further understanding of the influence of the SFSI on the seismic response of the superstructure.

5.2 Prototype & Scaled model Design

The concrete wall-frame systems are commonly used in multi-storey structures due to their structural integrity and efficiency. Multi-story buildings designed with structural wall-frame system are stiffer than moment-frame structural systems. In wall-frame structure, the shear walls can absorb a considerable amount of the base shear force or excitation by the seismic activities. Furthermore, the shear wall-frame system usually reduces the story drift and displacement Hokmabadi et al., 2014. Therefore, wall-frame structure is considered for this study.

Two basement floors with fifteen stories above ground level concrete wall-frame structural system with a total height of 53 m, width of 10 m and length of 10 m has been selected as a prototype for this study. The structural form and sections were designed based on Eurocodes (Cobb, 2014). ETABS (CSI, 2015) software was employed for the purpose of 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 force (Eurocodes Soil type C with maximum acceleration intensity of 0.2 g) were considered for the design of structural geometries and materials. The concrete compressive strength (fck) of 65 N/mm^2, mass density of 2400 kg/m^3 and elastic modulus of concrete of 36000 N/mm^2 were utilised for this wall-frame structure. The final structural sections are specified in (Figure 5-1).

It can be noted that the selected characteristics for the building represent the structural norms and construction practices of the conventional buildings in mega cities. Prototype meets the requirement of safe performance level. The necessary parameters such as natural frequencies, total weight and dimensions were obtained using ETABS software. The scale factors used in this study are explained in detail in chapter three. For this study, the scale factor of 1:50 has been 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 (5-1). The natural frequency and the total mass parameters played a fundamental role in the process of scaling detailed of the scaled model (Tabatabaiefar and Mansoury, 2016; Pitilakis et al., 2008).

In order to obtain the thickness and dimensions of steel plates and tube (represents slabs and column respectively), the scaled model was designed in the ETABS software and the dimensions were selected to meet the required natural frequency and mass of the scaled model. Furthermore, grade 255 steel (255 N/mm^2) was adopted in all the elements of the scaled model. This is due to the structural steel is flexible and constructible to the test environment, while 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 of 8 mm external diameter and thickness of 1 mm as the column elements.

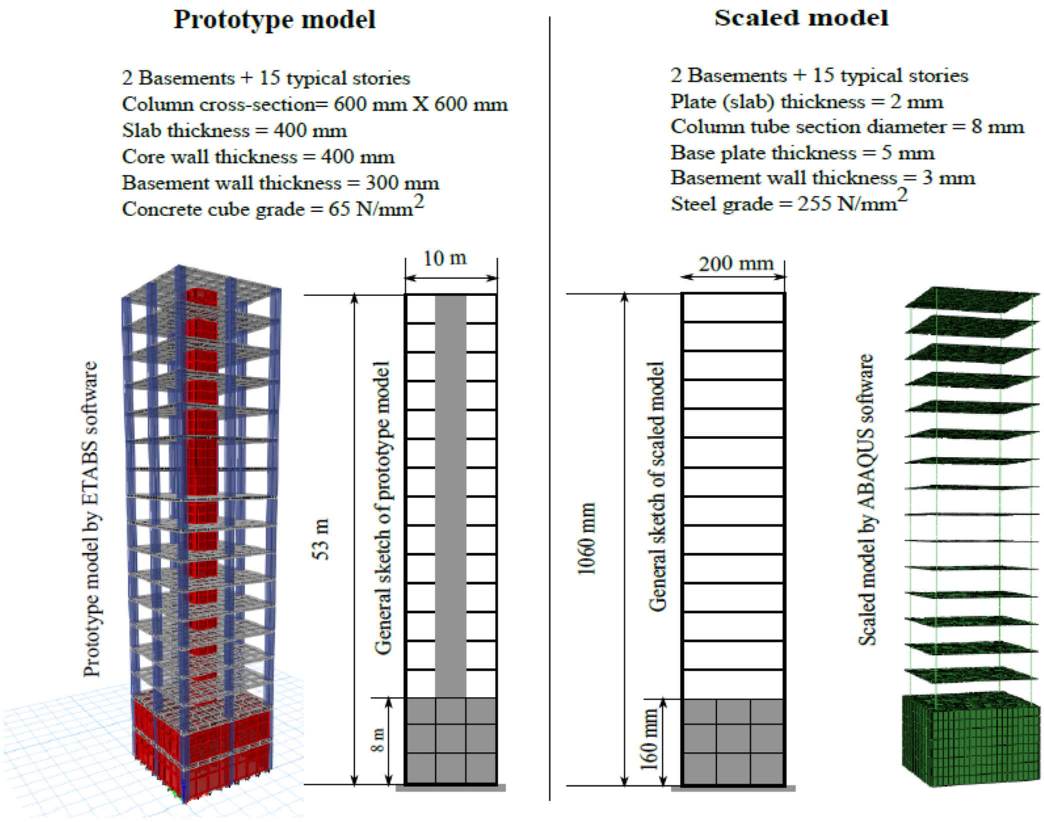


Figure 5-1 Detail of the prototype and scaled model

Dimensions of 220 × 220 × 5 mm and 200 × 200 × 2 mm steel plates were selected as the base plate and the typical floor 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 top level and base floor. Steel plates of 200 × 160 × 3 mm were attached vertically at the lower levels to represent the basement retaining walls. The total weight of the scaled model was 23.7 kg while the resonance of scaled structural model (natural frequency) determined by shocking (hummer) test was equal to 9 Hz. The expected and as built mass and frequency of the scaled model are shown in (Figure 5-1) and (Table 5-1).

Table 5-1 Summary of porotype and scaled model characteristics

Parameter	Prototype	Expected scaled model	As build scaled model
Natural frequency	1.32 Hz	9.33 Hz	9 Hz
Total mass	2904515.3 kg	23.2 kg	23.7 kg
Model height	53 m	1060 mm	1060 mm
Model length	10 m	200 mm	200 mm
Model width	10 m	200 mm	200 mm

5.3 Soil Container Design & soil properties

Moss et al. (2011) drew two conclusions. Firstly, the flexible barrel container and the relevant constructional details should be conducted properly to minimise the box effect.

Secondly, the container diameter should be five-times of the structure width. Hence, the dimensions of the container were selected as 1m diameter and a 1m depth. The flexible container was designed and manufactured at the University of Salford as shown in chapter 3 (Figure 3-4).

The flexible container consists of 5 mm membrane cylinder wall supported individually by stiffener strips. The top part of the container was supported by lifting hooks from an overhead crane. The bottom base was set on the shaking.

The dry sand with certain characteristic was used to reduce the volumetric changes during seismic excitation. The grain size distribution of the sub-rounded sand particles is shown in Chapter 3 (Figure 3-5). The maximum dry density of the sand as used in the vibration tests are 16 kN/m^3 with a minimum dry density of 14 kN/m^3. The specific gravity of the chosen sand is 2.68. The friction angle was measured as 34° in direct shear tests. Other relevant properties of soil can be found in (Table 5-2).

The sand was placed up to 600 mm in the container using the eluviation (raining) technique to achieve a uniform density (Dave and Dasaka 2012; Pitilakis et al, 2008).

The actual relative densities were achieved and measured by collecting samples in small cups with known volume extracted at different locations within the main container.

Table 5-2 Soil properties adopted in the numerical models

Parameter	Value
Mass Density, (p)	1600 kg/m^3
The angle of friction, (ϕ)	34°
Poisson's ratio, (v)	0.22
Young's modulus, (E)	80 x 10^6 N/m^2

5.4 Numerical Model Setup

Numerical simulations were carried out using the ABAQUS 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. There are mainly two integration method used to solve the dynamic behaviour in finite element analysis, which are implicit and explicit dynamic analysis.

Implicit solution is based on the quantities calculated of the previous time step. which is called Euler Time Integration solution. for large time steps, the solution remains stable. An Implicit FEM analysis is the same as explicit as the time step increment, but the implicit solution does Newton-Raphson iterations to enforce equilibrium of the internal structure forces with the externally applied loads. So, this is the primary difference between the two types of analysis is that the Implicit uses Newton-Raphson iterations to enforce equilibrium. The explicit solution is suitable for a very small-time step while the implicit analysis tends to be more accurate than for the bigger increment steps (Sun, et al 2000). Since the time step is significantly large (20 second). the adopted implicit integration method is suitable for this study.

Running 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 running time.

5.4.1 Structure and foundation

All parts of the structure and foundation are assumed to have linear elastic material behaviour in order to eliminate the influence of structural plastic deformation, due to wall-frame systems used in this study and the applied seismic events is expected to stay within the elastic limit of the structure. An 8-node solid linear hexahedral element type was used for walls, while shell element type was used for slabs and base. A beam element type was utilised for columns. The structural base and wall were considered as the raft foundation in experimental and numerical simulation. The piles were modelled using the beam element similar to the columns (Table 5-3).

Table 5-3 Characteristics of pile

Parameter	Value
Pile diameter	16 mm
Modulus of elasticity	7×10^7 kN/m^2
Density	27 kN/m^3
Poisson ratio	0.33

The interactions between walls and slabs surfaces were rigidly connected to determine the behaviour of the interfaces and all connected nodes of column and slabs were merged as one unit into the model. The same way the piles were merged with structure in the numerical modelling.

5.4.2 Soil container and soil materials

In the experimental investigation of flexural container, stiffened by steel rings were used to simulate the boundary condition. In order to smear the properties in numerical model, flexible plate element was used for the container wall. However, at the bottom surface of the container was modelled using rigid plate element (this is fixed to the shaking table). In the numerical modelling process, the soil medium is represented by nonlinear solid elements. A nonlinear Mohr-Coulomb model with tension cut-off (tension yield function) has been adopted in this study to simulate the nonlinear soil behaviour and possible shear failure in the soil elements during the excitation (Conniff and Kiousis, 2007; Rayhani and El Nagger, 2008).

5.4.3 Interactions and boundary conditions

Interaction between the soil and container is defined as tie connection. The interaction between the soil and structure for both raft and pile foundations was considered as rough surface with hard contact. This was archived by the sand coating on the retaining walls and both raft and pile foundations in experimental investigations.

The flexible wall was proposed to represent the viscous behaviour of the soil container and this wall has a tie connection with soil to ensure the flexible boundary of the soil container. This wall helps soil to be deformed and dispute the shaking energy to reduce the eaves reflection impact on the soil response within the soil container.

5.4.4 Time-History Analysis

solving dynamic problems of real structures is a challenging task. Instead, the finite element method may be used. There are two common used methods for finite element analysis, which are the time-history and the response-spectrum analyses methods. For large finite element analyses, where nonlinearities are expected, it is often too slow to employ time integration of the equation of motion.

Alternatively, there is an approximative approach in the frequency domain can be used, which is so called the response-spectrum analysis. However, only a single value can be obtained from response-spectrum analysis. Therefore, in this study the time-history analysis method was adopted to examines the nonlinear behaviour of the soil foundation structure system.

This analysis is carried out by solving the ground motion equation of the structure subjected to the seismic effect. The earthquake motion can be simulated by either time-acceleration, time-velocity or time-displacement and they are either determined from existing recorded data of earthquake or synthetically produced data see (Chapter 2 section 2.11.1).

5.5 Test program/ results and discussion

Four artificial time-history accelerograms with various peak ground accelerations (0.05 g, 0.1 g, 0.15 g, 0.2 g) were generated for elastic spectra Type 2 with soil type C using Eurocode 8 (2014). The time-history events (see Chapter 3 Figure 3-8) were applied through the shaking table in one horizontal direction.

Symmetrical boundary conditions (symmetry in the z-direction) applied on the corresponding boundaries of the wellhead and the soil. This means there is no deformation in the z-direction and no rotation about the x-and y-axes (Circumferential boundaries of the soil constrained in all degrees of freedom). The experimental test series were carried out in four different stages.

5.5.1 Fixed Base Scaled Model

Firstly, superstructure scaled model was directly fixed on the shaking table to determine the dynamic response of structural model. Seismic responses of the fixed base model under the influence of the four-selected time-history events (see Chapter 3 Figure 3-8) were examined.

Displacement transducers and accelerometers were installed on the structure at levels 2B+5, 2B+10 and B+15 to monitor the behaviour of the structure and to measure structural lateral displacements, acceleration and velocity in the time domain (Figure 5-2).

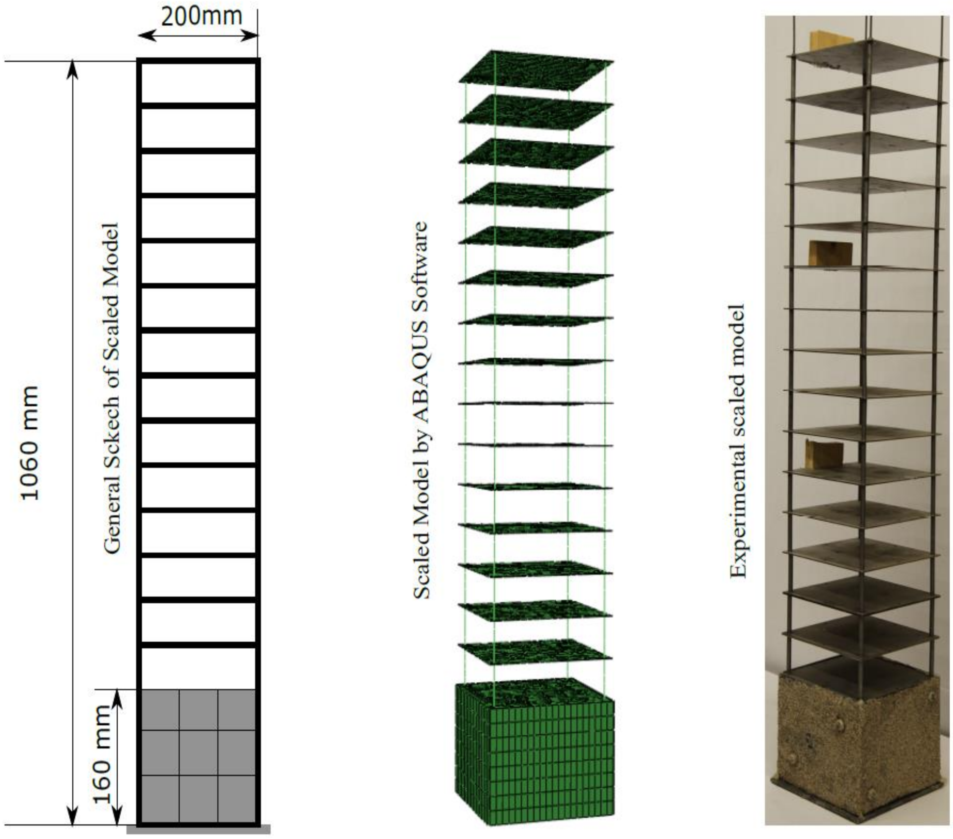


Figure 5-2 Fixed base model

The same scaled model was modelled numerically using finite element software as described earlier. 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 main purpose of this fixed based condition was to validate the numerical structural model and then to quantify the behaviour structure.

Lateral displacements at different levels with time for various accelerations using the experimental and numerical results. The 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 validated using the measured acceleration measurement. 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 5-3 (Figure 5-3) compares the maximum relative displacement to shaking table between experimental and numerical results at different story levels. This measurement method gives a reasonable deformation pattern of the structure in comparison with the absolute story deformation regardless of the occurrence times recorded (Caicedo, 2011). It can be seen that the values and trend of the 3D numerical predictions in comparison with experimental results are in good agreement. The difference in frequencies between experimental and numerical is less than 1 Hz, and there is a constant difference of 2 mm between the experimental and the numerical displacement.

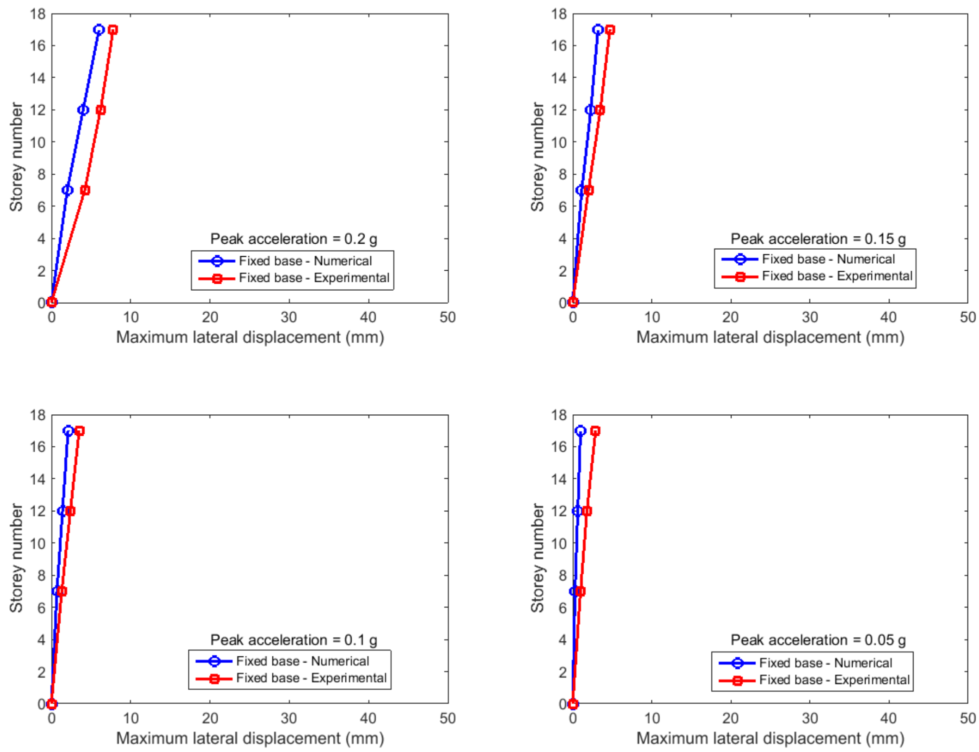


Figure 5-3 Maximum lateral displacement of experimental and numerical outputs

5.5.2 Soil Container

Nonlinear time-history dynamic analysis can be performed to simulate the realistic dynamic behaviour of the soil and the container under seismic excitations. Non-linear solid elements are employed to model the soil deposit, and flexible boundary conditions were applied. Nonlinearity of the soil medium plays a very important role on the seismic behaviour of the soil-foundation-structure system (Kim and Roesset, 2004; Maheshwari and Sarkar, 2011). The results of experimental and numerical acceleration outputs of the 0.05 g and 0.1 g peak acceleration time-history events have good agreement while the events of 0.15 g and 0.2 g are slightly are overpredicted (Figure 5-4). From plots of the power spectra (Figure 5-5), the experimental frequencies of all events have values around 7 Hz (and lower) while the numerical frequency outputs are around 5 Hz or lower. These discrepancies of both acceleration and frequency are due to experimental measurement methods. The experimental output was measured by the accelerometer, and this accelerometer has a mass of 50 grams. During the excitation, it is likely the accelerometer itself has a local effect on the experimental result in comparison with the numerical model output which is recorded from a selected node located at the centre of the soil mass. This is more noticeable for the events of 0.15 g and above. It can also be seen that when examining the spectra, there is more 'power' evident at lower frequencies for the numerical model output. This is currently being investigated but it is likely that it may well be due to how damping is currently being addressed in the ABAQUS model.

It is well known that sand can have volumetric change when it sheared (Stromblad, 2014). For the medium-dense soil, seismic excitation makes a net contraction of the deposit evidenced as settlement of the sample surface. The test soil void ratio of soil decreased when the soil density increased. These variations should be reflected in the calculations for stiffness and shear stress. For the adopted soil in question, measured contractions had a negligible effect on other parameters and the volumetric change was insignificant.

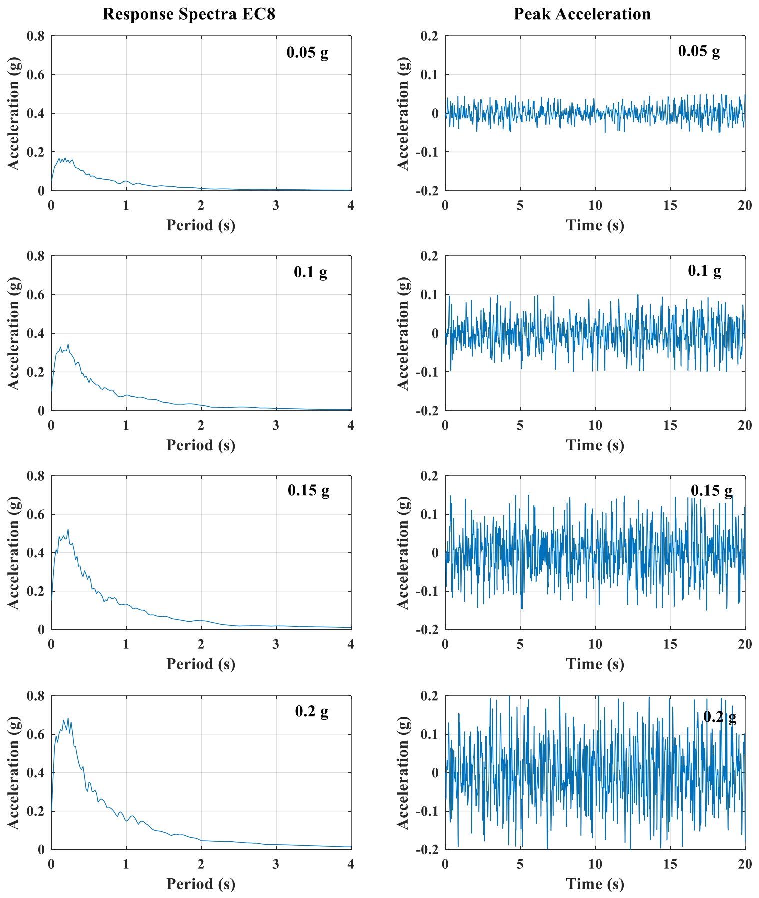


Figure 5-4 Soil container experimental and numerical acceleration outputs

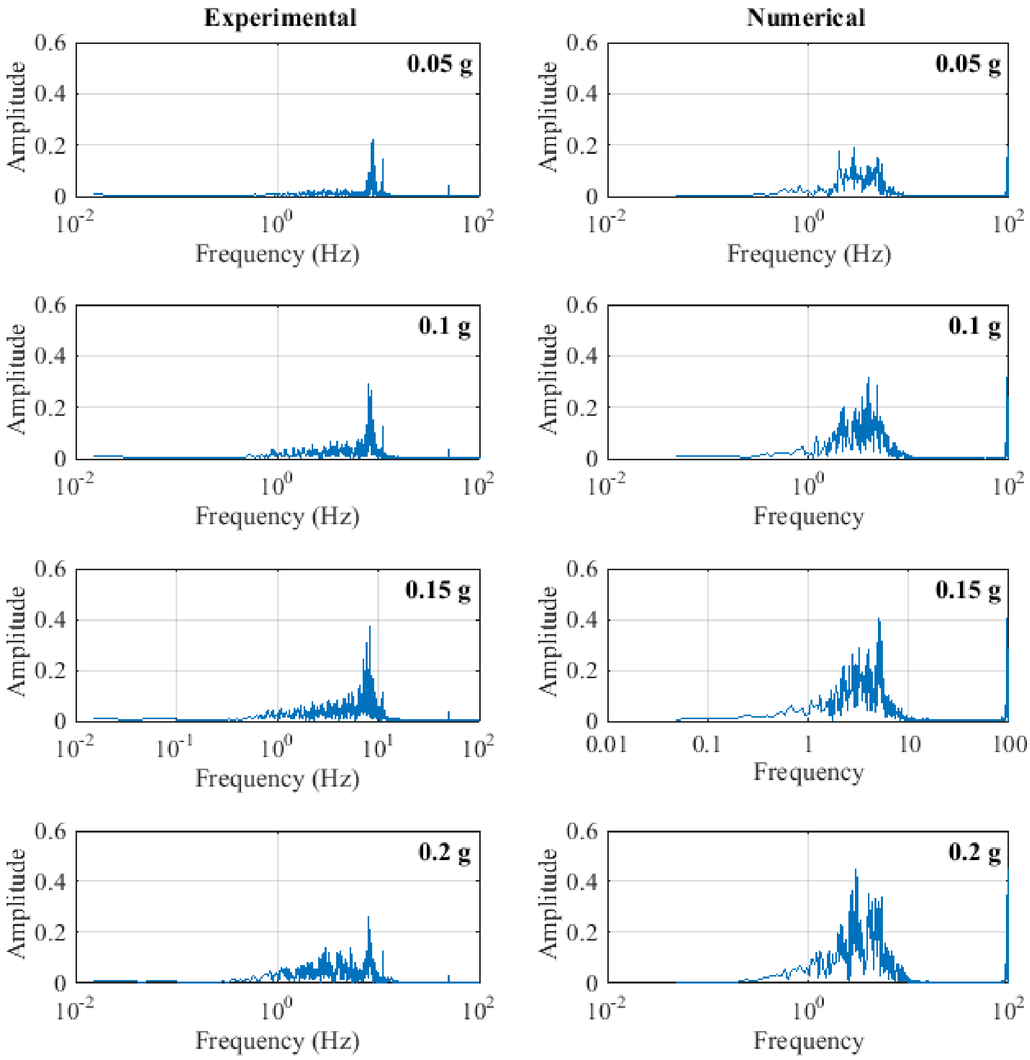


Figure 5-5 Experimental and numerical spectral outputs

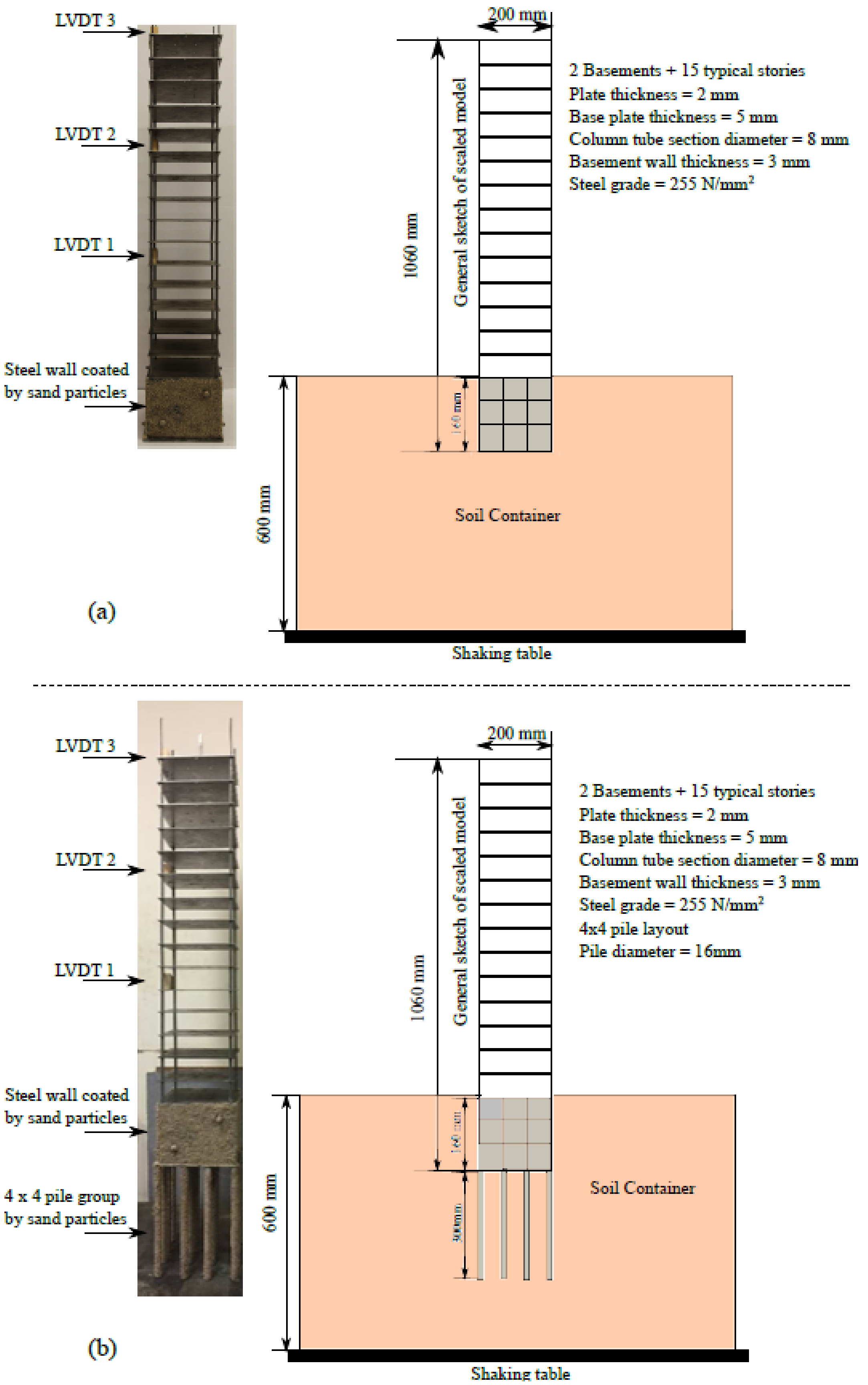


Figure 5-6 Structures supported by (a) raft foundation and (b) raft on pile foundation

5.5.3 Structure Supported by Raft Foundation

Effect of soil-raft foundation-structure interaction was investigated as the third stage. In this series, the same instrumentations setup on the structure and the soil container were used. In order to simulate all contact surface of the structure and the soil, sand was coated using hard glue on the bottom surface of base plate and the side walls. After the soil container had secured the shaking table, the scaled model was embedded within soil medium for 160 mm from the surface of the soil, as shown in (Figure 5-6) (a). The selected time-history events (see Chapter 3 Figure 3-8) were examined using shaking table. The densities of soil before and after the events were obtained using the same procedure as previously.

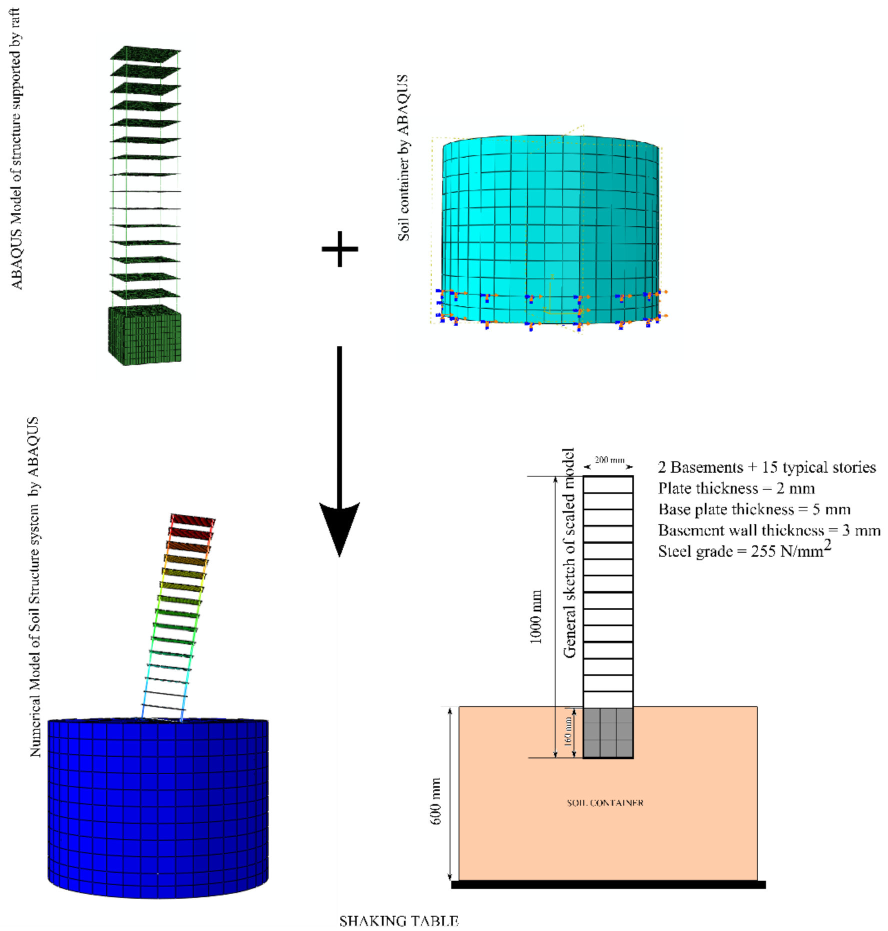


Figure 5-7 Numerical simulation of soil foundation structure interaction (raft foundation)

The experimental setup were modelled numerically as shown in (Figure 5-7), where the structure with raft foundation was placed in the middle of the top surface of the soil container. The interaction between the raft foundation and soil were modelled as described earlier.

(Figure 5-8) compares the maximum relative displacement to shaking table between experimental and numerical results at different story levels. The displacement of the numerical model has an average value of 7 mm difference between experimental and (almost 10%) more than experimental displacement in structure supported on raft foundation. This measurement method gives a reasonable deformation pattern of the structure in comparison with the absolute story deformation with regardless of occurrence time are recorded. It can be seen that the values and trend of the 3D numerical predictions compare with experimental results are in good agreement.

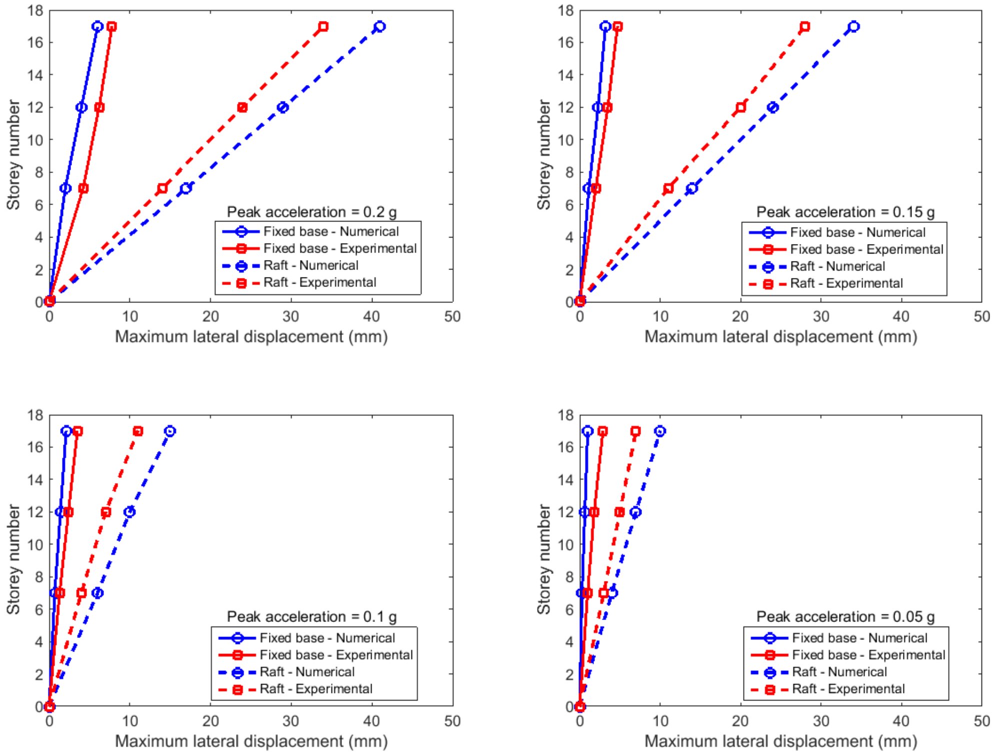
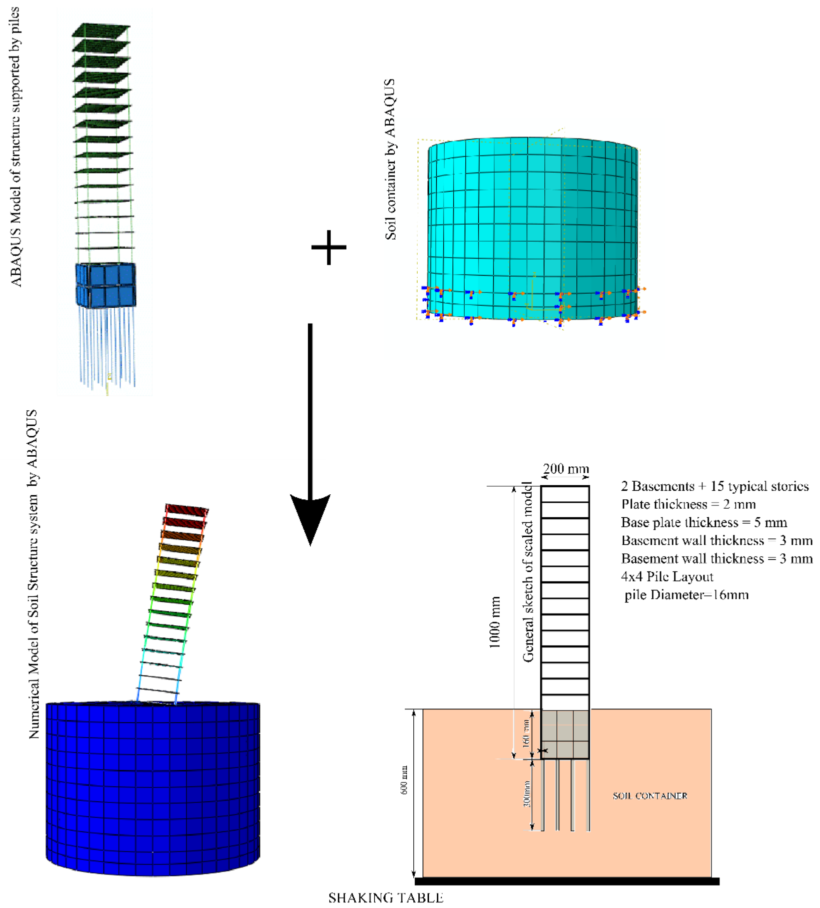


Figure 5-8 Structure on raft foundation output

5.5.4 Structure Supported by Raft on Pile Foundation

In this series, linear rigid piles were considered. These were achieved by scaling of the flexural rigidity (EI) of the piles according to Hokmabadi (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. Characteristics of the model pile used in this study are summarised in (Table 5-3). The model pile surface has been glued with sand particles to make rough surface and to avoid the interface problem. Rest of the experimental procedures were the same as the raft foundation (Figure 5-6b).



Soil Foundation Structure Interaction (Raft on Pile Foundation)

Figure 5-9 Numerical simulation of soil foundation structure interaction (raft on pile foundation)

The same pile group was numerically modelled as shown in (Figure 5-9). the maximum relative displacement experimental and numerical results at different story levels were compared and it's found that the results are in same trend and there is a reasonable discrepancy of an average of 6 mm structure supported on raft on pile foundation due to nonlinear behaviour of the soil structural system (Figure 5-10).

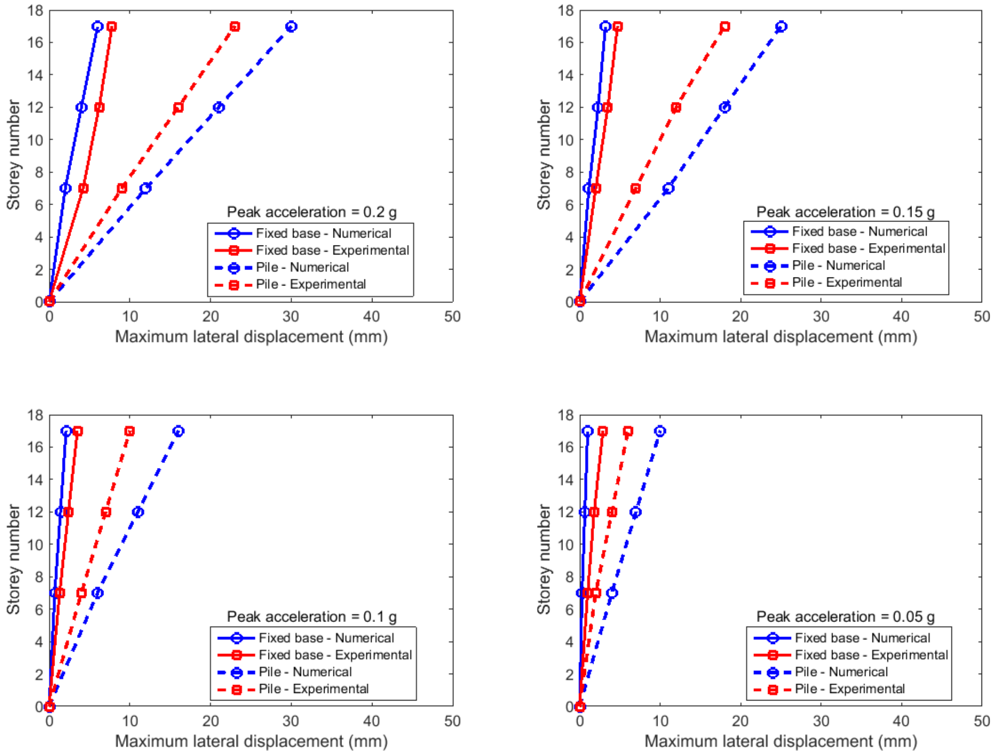


Figure 5-10 Structure on raft on pile foundation output

5.6 Summary

Comparing the results of the numerical model and the experimental measurements, it can be concluded that the employed numerical model is appropriate for the simulation of the soil-foundation - structure interaction under dynamic effect.

The scale models demonstrate some behaviour of the prototype in economical way without examining the prototype itself. These works introduce and discuss several mathematical models, based on equation of motion. These models are providing a suitable set of physical parameters characterizing the prototype properties. The results demonstrate procedure benefits from an experimental feedback and provide reliable and qualitative analytical information. Consequently, the proposed numerical model of raft foundation and pile foundation are valid and qualified method of simulation with sufficient accuracy which can be employed for further numerical dynamic soil-structure interaction investigations. Practicing engineers can adopt this verified numerical modelling procedure in the design considering the effect of soil foundation structure models with respect to the interface elements, boundary conditions, and soil properties.

Another advantage of the current numerical modelling technique is performing the SFSI model analysis in a fully coupled manner in which main components of the interaction including subsoil, pile foundation, and superstructure are modelled simultaneously without resorting to independent calculations of site or superstructure response, or application of the pile group interaction factors.

CHAPTER SIX - EVOLUTION AND SIMPLIFICATION OF DIFFERENT PARAMETERS ON THE RESPONSE OF SOIL- FOUNDATION-STRUCTURE INTERACTION

6.1 General

Several options could be adopted in the selection of the foundation type of the buildings located in high-risk seismic zones, such as shallow foundation, raft foundation, or raft-on-a-pile foundation. These options may be considered by design engineers to carry both static and dynamic loads. However, these foundations do not behave similarly when considering the soil-foundation-structure interaction (SFSI) under seismic effects. The soil-foundation-structure interaction exists where the in-situ properties of soil and foundation have a significant impact on the dynamic behaviour of the soil-foundation-structure system.

During earthquake excitations, the rocking of a structure may occur due to the inertial forces generated within the soil-foundation-structure system. The rocking of the structure causes compression stress in one side and tension stress on the other side of the foundation. As result of these stresses, settlement may occur in one side and possible uplift on the other side of the foundation. Each type of foundation undergoes a different experience incurred by the structure rocking. The rocking component amplifies the lateral structure displacement and may influence the total stability of the soil-structure system. However, a significant amount of ground motion energy dissipates due to the structure rocking which resulting in lower shear forces indirectly applied to the structure. Comparing the behaviour of different foundation types with respect to the soil-foundation-structure interaction helps the engineer to design the proper foundation to resist the impact of soil-foundation-structure under dynamic effects (Poulos et al., 2015).

6.2 Evolution and simplification work methodology

In this chapter, different characteristics of SFSI and its influence on the seismic response of superstructures are investigated. Parametric studies concerning different foundation types have been conducted. For this purpose, the previously verified three-dimensional numerical modelling procedure (Chapter 5) has been adopted. A seventeen storey full scaled structure with three types of supporting foundations was invistagted as follows. Firstly, a structure supported by the fixed base was created to represent the structure response without the effect of soil-foundation-structure interaction. Secondly, a structure supported by a raft foundation, and thirdly, a structure supported by a raft-on-a-pile foundation were examined. Finite element analyses were performed using transient analysis. Results were presented and compared in respect of the ground motion effect, soil properties amplification, and the lateral deformations.

The design engineers are required to follow the described numerical procedure to estimate the response of the multi-storey structures under the influence of soil-foundation-structure interaction subjected to a dynamic load. The entire numerical procedure could be time-consuming and sometimes complicated. On the other hand, for a design engineer, simpler and more readily available procedures are more performable than modelling complex problems which are also time-consuming. As a result, a well-developed and simplified procedure is proposed for practical purposes to consider the dynamic response of soil-foundation-structure system.

In this chapter, an empirical relationship based on the results of a reported parametric study was developed to enable design engineers to evaluate the effects of the soil-foundation-structure interactions by determining the lateral building deflections. Accordingly, the generated moment on foundation level due to the soil-foundation-structure interactions can be determined. By conventional design methods, the total moment calculated does not include soil-structure interaction effects. Therefore, the proposed method of calculating the soil-foundation-structure-interactions moment helps the foundation engineers estimate the additional moment of this interaction and add it to the total moment calculated by the conventional method to ensure that the foundation has been designed against all predicted forces. As per Hokmabadi (2014), for the structures modelled with soil, the base shear of the structure supported by soil (flexible base) is less than the base shear of structures modelled with fixed base supports. Therefore, the reduction of base shear due to SFSI could be ignored in the design procedure to contribute to safer design. However, lateral deflection amplifications due to SFSI has detrimental effects on performance and building safety and must be taken into account in any design procedure.

6.3 Nonlinear Time-History Dynamic Analysis

To evaluate the elastic and inelastic dynamic response of structural models, non-linear time-history analysis was adopted in this study. The time-history analysis is a step-by-step analysis of structure behaviour subjected to specified dynamic loading. The dynamic equilibrium equations see Chapter Two (equation 2.5).
The applied load could be in terms of time-accelerations or time-displacements or time-velocities which are relative to the ground motion. Also, it is possible to do an anaylsis for any number of time-history cases within this procedure.

In the time-history analysis, non linear load, damping and stiffness are based on values of dynamic load input. An iterative solution is required to solve the equations of motion. Furthermore, the non-linear structure properties are studied as part of a time domain analysis (Hosseini et al., 2017).

6.4 Geometric Non-linearity and P-Delta Effects in Time-History Analysis

In an earthquake, the structure has geometrically linear behaviour when its deflection is small enough. The linear load-deflection relationship of the structure is adopted by the numerical software since the equilibrium equations are utilising the undeformed geometry of the structure. However, the equilibrium equations should refer to the actual geometry of the deformed structure as a non-linear behaviour. In particular, the non-linear structure behaviour is considerable strain and rotations based on the constrictional material, the common linear stress and strain measure no longer apply in structure analysis , and the equilibrium equations are written for the Geometric nonlinearty.

The deformed configuration of structure equilibrium equations requires a large number of Newton-Raphson iterations.

The P-Δ effect refers specifically to the non-linear geometric effect of a substantial tensile or compressive direct stress upon transverse bending and shear behaviour. The structural member is considered to be more flexible under compressive stress in transverse bending and shear from ground into the structure body, while tensile stress tends to make the structural member more stiff to resist the transverse deformation (Stafford and Coull, 1991). The P-Δ anaylysis concept is applied only for fixed base structure or structure supported by siff or hard strata where the soil structure interaction has insignfacte impact on the structure response.This option is mainly used to consider the gravity load effect upon the building lateral deflection. The basic concept of the P-Delta approach is illustrated as a cantilever beam subjected to an axial load P and a transverse tip load F as shown in (Figure 6-1).

In the equilibrium situation of the un-deformed structure geometry, the moment applied to the structure support is equal to M = FL, and decreases linearly to become zero at the member end. In the equilibrium of the deformed configuration, an additional moment along the member length and variation depends on the deflected shape and the moment at the base will be M =FL-PD.

Once the non-linear analysis is performed, the final stiffness matrix is used for subsequence linear analyses. The non-linearity of the structure considered in the non-linear analysis influences the linear results (Davidson et al., 1992; Stafford and Coull, 1991).

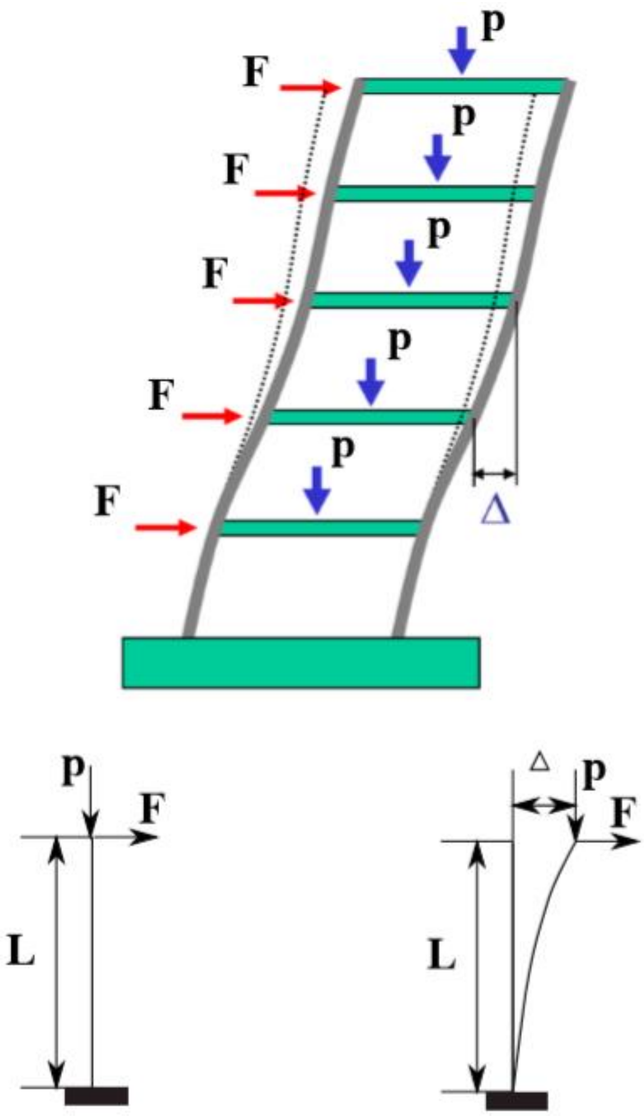


Figure 6-1 Configuration of P-∆ effect

Running the implicit step analysis in ABAQUS model helps to capture P-Delta effects and structural geometric non-linearity accurately by updating the structural matrix. Thus, to take the mentioned effects into account, all the dynamic structural analyses in this study have been performed in implicit step mode (Dassault Systèmes Simulia, Fallis and Techniques, 2013; Hügel et al, 2008).

6.5 Utilised Ground Motions in Time-History Analyses

Using the software Seismo Artif, four artificial time-history accelerograms were generated with different peak ground accelerations (0.05 g, 0.1 g, 0.15 g, 0.2 g) see (Chapter 3 Figure 3-8). These events were generated from EC8 elastic spectra of soil Type C and were adopted for dynamic load inputs to the model.

6.1 Characteristics of the Adopted Soil-Foundation-Structure Systems

A two basement plus fifteen stories dual concrete wall-frame structural system with a total height of 53 m, width of 10 m and length of 10 m was selected as a prototype for this study. The structural sections were designed and specified following the regular structural design procedure based on Eurocodes building codes (Cobb, 2014). ETABS software was employed for the design purposes following the required steps for analysis and design of structures. After creating the structure geometry, the structural sections and materials were defined. Then the live, dead, wind and seismic forces were applied based on the requirements of Eurocodes Soil type C with maximum acceleration intensity of 0.2 g. The concrete structure prototype utilised concrete compressive strength (fck) of 40 KN/m^2 and mass density of 2400 kg/m^3. The modulus of elasticity of concrete (E) was 36000 mpa. The dead load and live load were 5.5 KN/m^2 and 2 KN/m^2, respectively, and were determined as uniformly distributed loads over the floors. Wind velocity was selected as 45 m/s with terrain category of 2. The final structural sections are specified as shown in (Figure 5-1). It should be noted that the selected characteristics of the building represent the construction practices of conventional multi-storey buildings in megacities. The outputs analysis and design of the fixed base prototype is satisfying the requirements for the life safety performance level.

The scale factor of 1:50 was selected as illustrated in (Chapter 3 (Table 3-2)). Thus, the scaled model dimensions were 1.05 m in height (H), 0.20 m in length (L), and 0.20 m in width (W). The natural frequency and the density parameters play a fundamental role in the process of model scaling.

Hence, the natural frequency of the prototype was scaled down with suitable scaling factors, while the density of prototype and scaled model should be equal (Meymand, 1998). Furthermore, the steel structure model is flexible and constructible to the test environment, while the concrete structure model could not be constructed with the required dimensions and dynamic properties. Therefore, the concrete structure prototype element was scaled into a steel structure model element, by scaling the natural frequency and the density of the prototype (see Chapter 5 Figure 5-1).

6.6 Geotechnical Characteristics of Employed Subsoils

The dry sand was used as the backfill material. This type of sand has sub-rounded particles. The maximum dry density of the sand as used in the vibration tests is 16 kN/m^3 with a minimum dry density of 14 kN/m^3. The specific gravity of the chosen sand is 2.68. The friction angle was measured as 34° in direct shear tests. Sand was placed in the container using the eluviation (raining) technique to achieve a uniform density. The actual relative densities were measured by collecting samples in small cups with known volumes embedded within the soil container at different depths.

In the process of numerical modelling, the soil elements are represented by non-linear solid elements. Each element (linear or non-linear) behaves based on the prescribed stress/strain law to simulate the response of the applied forces or boundary restraints. Accordingly, a proper constitutive soil container model is required to represent the geomechanical behaviour of soil elements and also a proper soil container is required to be implemented in ABAQUS software to conduct an accurate SFSI model. In this study, a non-linear Mohr-Coulomb model was adopted to simulate the soil behaviour and possible shear failure within the soil medium during the seismic excitations (see Chapter 4 Figure 4-8). The Mohr-Coulomb model is a non-linear elastic-perfectly plastic model employed by many researchers (Conniff and Kiousis, 2007), (Rayhani and El Naggar, 2008) to simulate soil element for the soil structure system under dynamic effects(Figure 6-4). The associated failure envelope of soil corresponds to a Mohr-Coulomb criterion (shear yield function) with tension cut-off (tension yield function) for soil element dimensions were diameter 1000 mm and depth 600 mm. Dilatancy is a volume change that occurs when sand soil with round particles is subjected to shear. When shear wave velocity of the soil is less than 600 m/s, the effects of soil-structure interaction on seismic response of structural systems are particularly significant (Li et al., 2014), (Stewart, Seed and Fenves, 1999), (Galal and Naimi, 2008).

6.7 Determining the Seismic Response of Models Considering the Dynamic Soil- Foundation –Structure Interaction

In this study, a fully non-linear time history method was adopted to simulate a dynamic load applied on the soil-foundation-structure system using ABAQUS software.
The model systems were considered in conjunction with three types of sand soil: loose sand, medium dense sand and dense sand. Characteristics of the soils utilised for parametric study are shown in (Table 6-1).

Table 6-1 Sand soil adopted for parametric study

Soil Type	Dry Density KN/m^3	Modulus of Elasticity (MPa)	Angle of internal friction (ϕ)	Shear Wave Velocity m/s
Dense soil	16	80	34	360
Medium dense	15	50	30	270
Loose soil	14	30	27	180

To determine the linear and non-linear response of the studied multi-storey structure under seismic effects, the dynamic analysis was carried out for three structural height levels namely, Base, 2B+ 5, 2B+10 and 2B+15 structural systems (Table 6-1

Table 6-2 Scaled structural properties

Structure Type	Structure Mass (Kg)	Structure Hight (mm)	Fixed Base Natural frequency (Hz)
2B+5	8.5	340	29
2B+10	14.62	760	16
2B+15	23	1000	9.5

Raft and raft-on-pile foundation (Figure 6-2) have been considered as described in Chapter Three and Chapter Four.

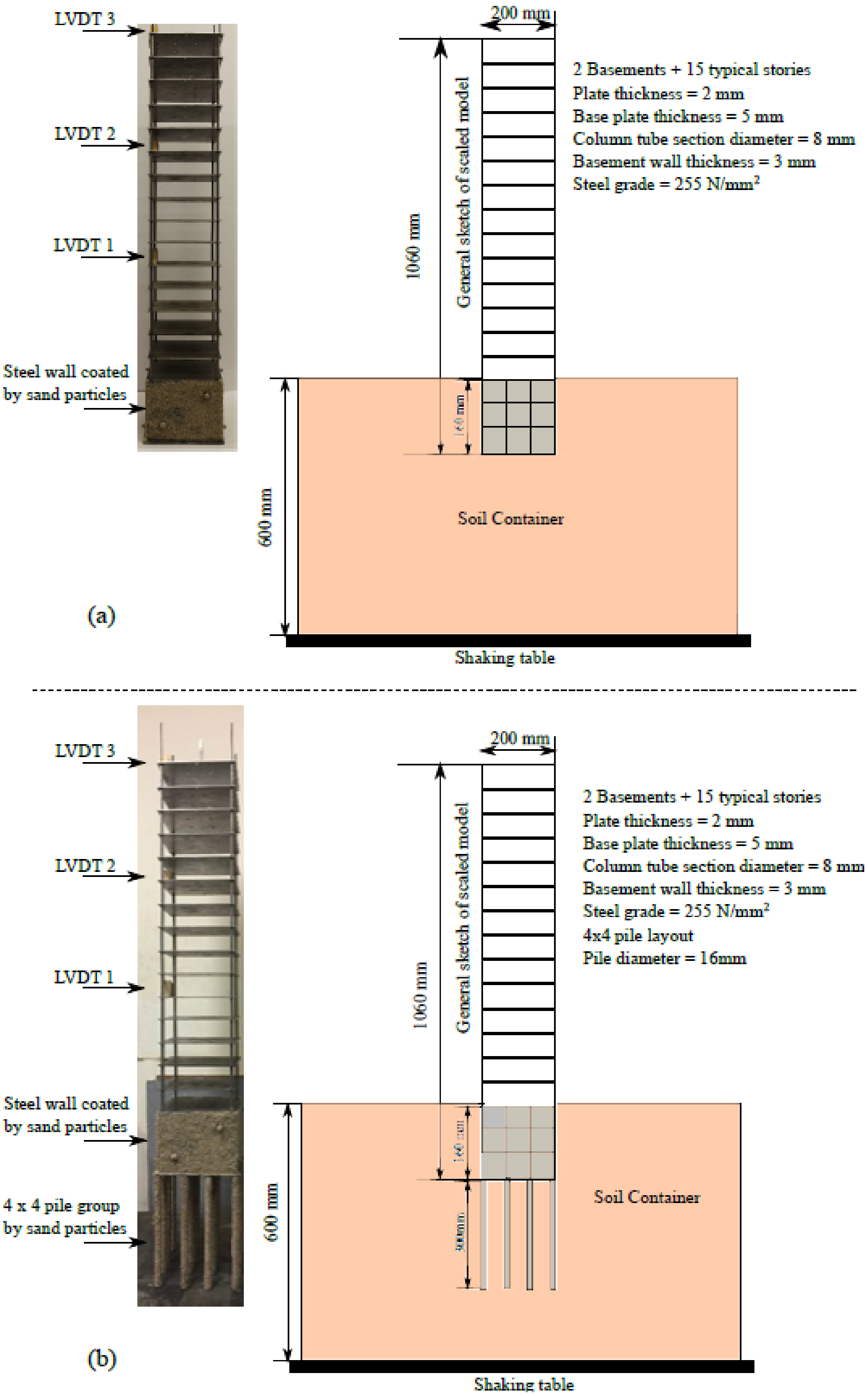


Figure 6-2 Structures supported by (a) raft foundation and (b) raft-on-pile foundation

6.2 Discussion of the Dynamic Structure Response

The results of elastic and inelastic analyses in terms of lateral deflections under the impact of four shaking events for the fixed base and flexible base models resting on three different soil types were investigated. A comprehensive comparison was carried out between the output results to illustrate a conclusion about the effects of subsoil stiffness, structural height variations, foundation types, and ground motions on the response of soil-structure system under dynamic effect. Maximum lateral rocking deflections subject to four shaking events were determined and compared. Numerical modelling of the 3D soil-structure system was utilised to predict the maximum lateral deflections of the two basements + fifteen storey structure supported by raft foundation, and raft-on-pile foundation. As discussed earlier, the adopted 3D numerical models were used to account for non-linear behaviour of the soil. To determine the lateral rocking deflections, the structure displacements were recorded at different levels of the structure. Movements of foundation within the soil medium were recorded. It should be noted that the data were based on the maximum absolute storey deformations of a selected level regardless of their occurrence time (Hokmabadi et al., 2012).

6.3 Soil-Foundation-Structure Interactions with Raft Foundation

The results of the 3D numerical predictions for the maximum lateral rocking deflections of the two basements + fifteen storey structures supported by the raft foundation is summarised and compared for different parametric studies (Table 6-3).

Table 6-3 Structure supported by raft foundation parametric study

Soil Classification	Acceleration	Structure Height	Foundation Type	Study/ purpose
Dense Soil	0.2g, 0.15g, 01g, 0.05g	2B+15	Raft	Examine the effect of ground motion
Dense, Medium and Loose Soil	0.2g	2B+15	Raft	Examine the effect of soil properties
Dense Soil	0.2g	2B+15, 2B+10 and 2B+5	Raft	Examine the effect of structure properties

The lateral deflections have been subtracted from the storey movements where there displacement of the top level measured in relative to the base level (Figure 6-3).

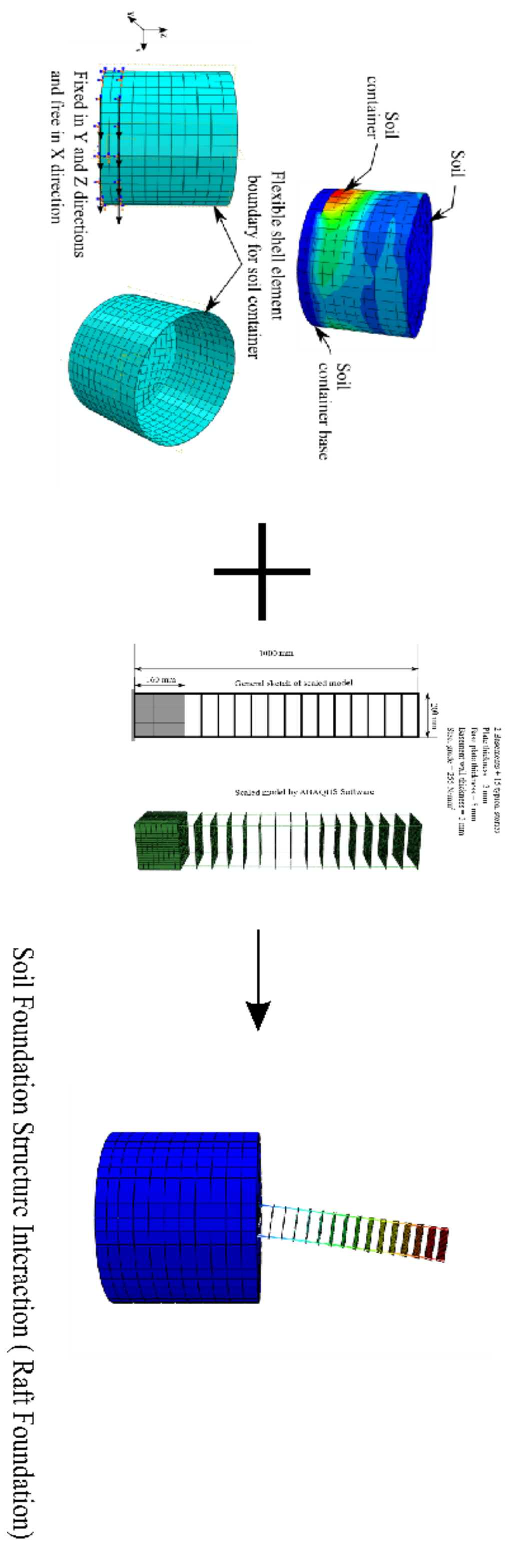


Figure 6-3 Numerical model for soil-foundation-structure interactions (raft foundation)

6.3.1 Effect of Ground Acceleration (raft foundation)

Based on Figure 6-4 and as expected, the peck acceleration of 0.2 g causes more lateral rocking deflections than 0.15 g, 0.1 g and the lowest intensity 0.05 g accelerations due to the higher inertial forces generated.

For example, the maximum lateral deflections of the soil-structure system under the influence of 0.2 g acceleration is 41 mm, while the lowest value of lateral deflections corresponding to the 0.05 g peak acceleration is 10 mm.

The increments of acceleration increase the inertia force generated within the soil mass and transferred through the structure, and as a result causes more displacements (Figure 6-4).

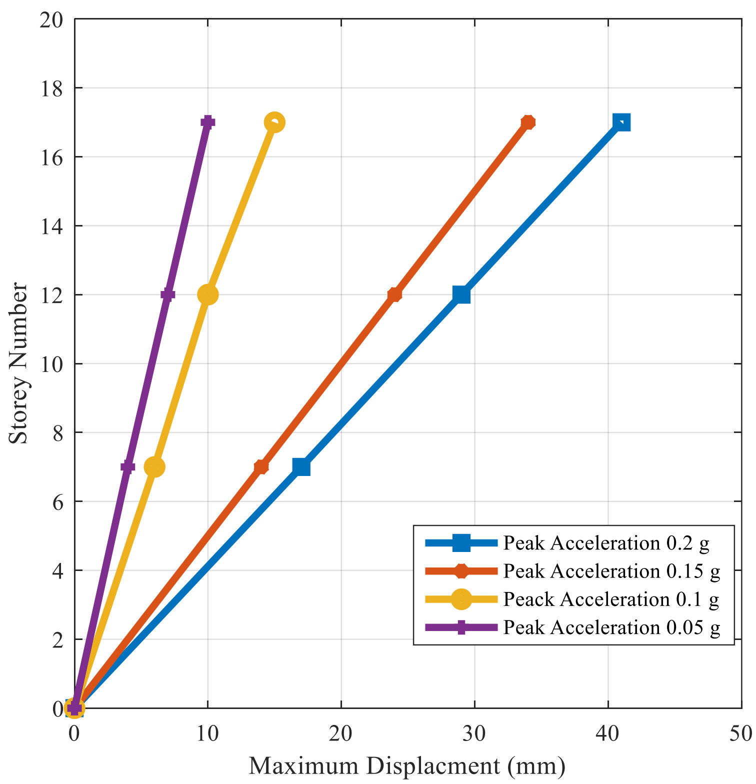


Figure 6-4 Effect of acceleration intensity on the maximum displacement (raft foundation)

6.3.2 Effect of Soil Properties (raft foundation)

In general, the soil-structure interaction tends to amplify the lateral deflection of the superstructure. Referring to (Figure 6-5), for raft foundation the higher the soil stiffness, the smaller the lateral deflections will be during the seismic events. The maximum lateral rocking deflections of the structures supported by the raft foundations increases in the loose sand more than the medium and dense sand in comparison with the fixed base model. This is because of the soil stuffiness. The lower the stiffness of the soil, the greater the structure deflections will be due to the soil deformation.

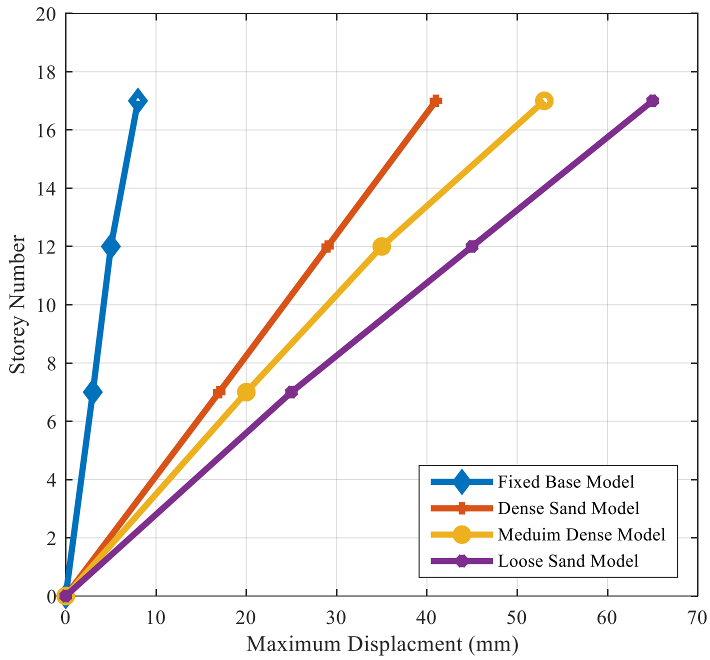


Figure 6-5 Effect of soil properties on maximum displacement (raft foundation)

6.3.3 Effect of Structural Properties (Raft Foundation)

The maximum lateral rocking deflections of a fixed base 2B+15 model supported by raft foundation on dense soil was compared for structures with various heights (2B+5, 2B+10 and 2B+15). Referring to (Figure 6-6) in the inelastic analysis case, lateral deflections of flexible base models resting on the same class of soil increased when the structure height and mass increased in comparison with fixed-base models. When the structure height increased, the reaction moment applied to the structure increased accordingly. Due to the soil's ground acceleration, the generated inertia force is applied to the structure base causing a reaction moment.

This increment in the value of moment is based on the equilibrium law whereby the structure tends to rotate around the base due to the force applied to the structure base. The reaction moment acts to resist the force effect which is dependent on the structure height structure mass. Therefore, the resultant deflections have a direct relationship with the reaction moment.

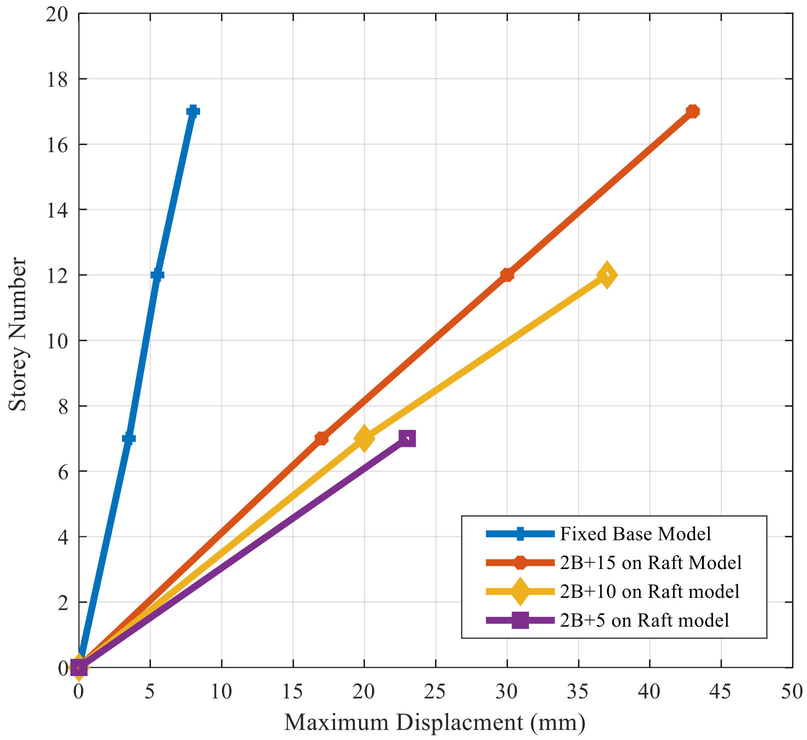


Figure 6-6 Effect of structure height on maximum displacement (raft foundation)

6.4 Soil-Foundation-Structure interactions of pile foundation

The outputs of the 3D numerical models for the maximum lateral rocking deflections of the structure supported by the raft on pile foundation is summarised and compared for different parametric studies (Figure 6-7) and Table (6-4).

Table 6-4 Structure supported by raft-on-pile foundation parametric study

Soil Classification	Peak Acceleration	Structure Height	Foundation Type	Study/ Purpose
Dense Soil	0.2g, 0.15g, 01g, 0.05g	2B+15	Raft on (4x4 Pile) Pile Length=300 mm	Examine the effect of ground motion
Dense, Medium and Loose Soil	0.2g	2B+15	Raft on (4x4 Pile) Pile Length=300 mm	Examine the effect of soil properties
Dense Soil	0.2g	2B+15,2B+10 and 2B+5	Raft on (4x4 Pile) Pile Length=300 mm	Examine the effect of structure properties
Dense Soil	0.2g	2B+15	Raft on (4x4 Pile) Pile Length=300, 400, 200 mm	Examine the effect of pile length
Dense Soil	0.2g	2B+15	Raft on (4x4 Pile), (3x3 Pile), (2x2 Pile) Pile Length=300 mm	Examine the effect of pile number

The lateral structure deflections were determined based on the relative movement of foundation subtracted from the storey movements at specific levels. Therefore, all the records are relative to the base level of the structure.

The data presented are based on the lateral deformation of each storey when the maximum deflection at the top level occurs regardless of the occurrence time.

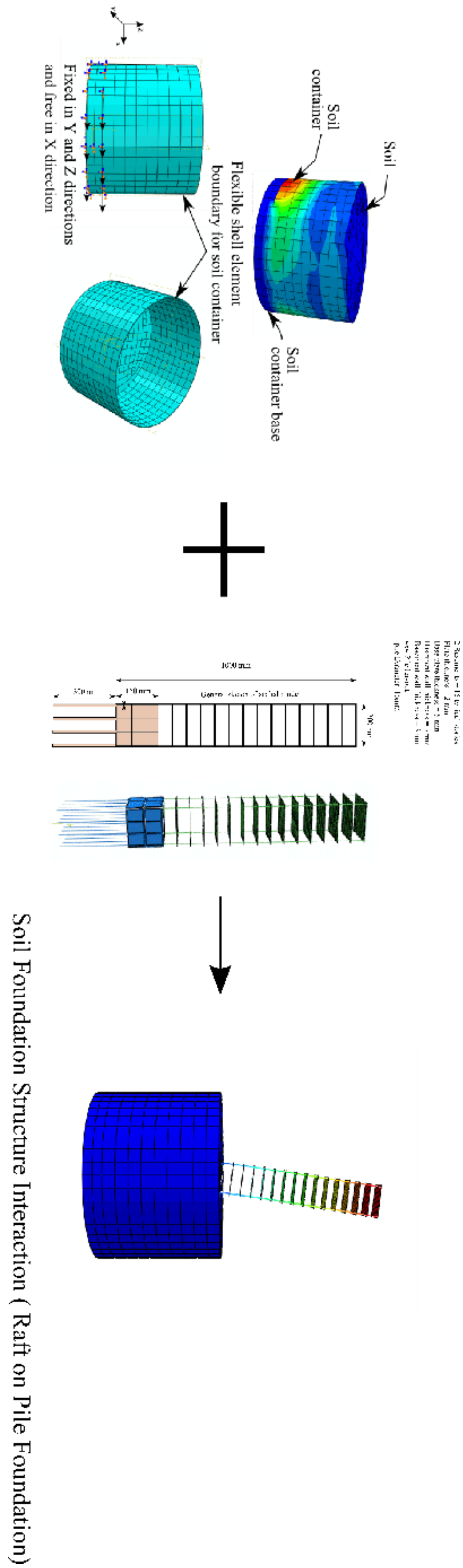


Figure 6-7 Soil-foundation-structure interaction (raft-on-pile foundation)

6.4.1 Effect of Soil Properties (raft on pile foundation)

In general, the soil-structure interaction tends to amplify the lateral rocking deflection of the superstructure. Referring to (Figure 6-8), for the raft-on-pile foundation, the greater the soil mass, the greater the lateral deflections during the seismic events due to the increment of inertia force generated within the soil mass. The behaviour of the raft-on-pile foundation is different from raft foundation. The maximum lateral deflections of the structure supported by the raft-on-floating-pile foundations increase in the dense sand more than the medium dense and loose sand in comparison with the fixed base model. Furthermore, the sand particles are non-homogeneous particles similar to clay soil. During the seismic excitations, the connection between the soil particles and pile element is lost. The sand behaves as an isolate. This behaviour subjects the structure body to higher amounts of soil inertia force compared with the pile element.

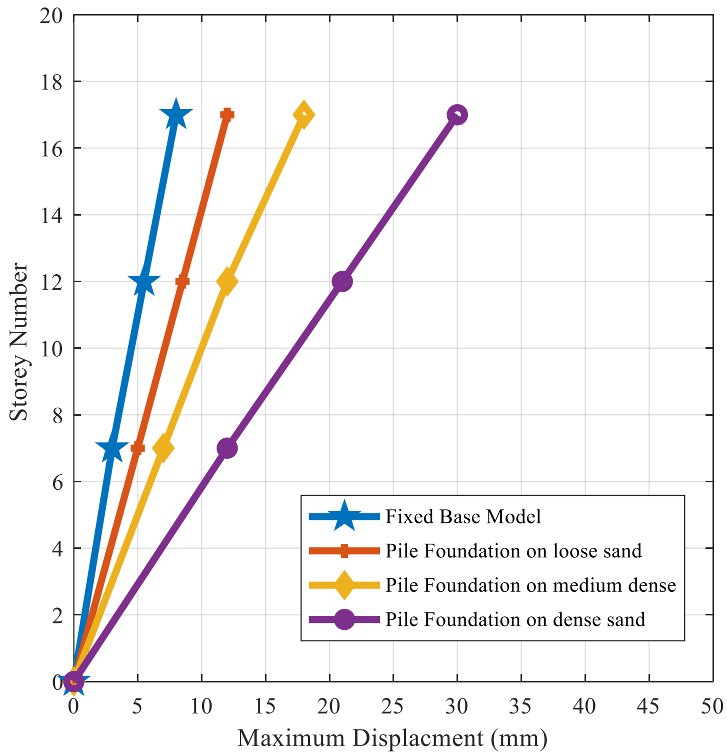


Figure 6-8 Effects of soil properties on maximum displacement (raft-on-pile foundation)

6.4.2 Effect of structural properties (raft on pile foundation)

The maximum lateral rocking deflections of a fixed base 2B+15 model supported by raft-on-(4x4) pile foundation on dense soil was compared for structures with various heights (2B+5, 2B+10 and 2B+15) as shown in (Figure 6-9). Lateral deflections of the structure resting on the soil increased when the structure height and mass increased in comparison with outputs of fixed-base models. When the structure height increased, the reaction moment applied to the structure increased along with the structure height. Due to the soil ground acceleration, the inertia force is generated and applied at the structure base and a reaction moment occurs, accordingly.

This increment in the moment value happens based on the equilibrium law which means the structure is tending to rotate around the base due to the force applied at the structure base. Accordingly a reaction moment occurs to resist the force effect depending on the structure height and structure mass. Therefore, the more reaction moment, the more deflection results.

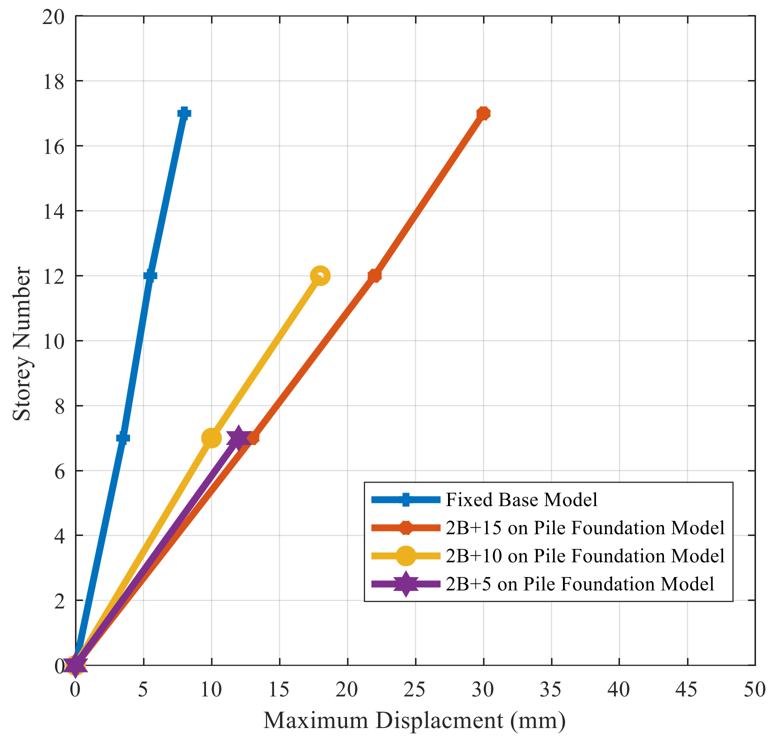


Figure 6-9 Effect of structure height on the pile foundation

6.4.3 Effect of Ground Motion (raft on pile foundation)

Based on Figure (Figure 6-10) and as expected, the pack acceleration of 0.2 g causes more lateral rocking deflections than 0.15 g, 0.1 g and the lowest intensity 0.05 g accelerations due to the higher inertial forces generated. For example, the maximum lateral deflections of the soil-structure system under the influence of 0.2 g acceleration is 30 mm, while the lowest value of lateral deflections corresponding to the 0.05 g peak acceleration is 10 mm. The increments of acceleration increase the inertia force generated within the soil mass and transferred through the structure and as a result, causes more displacements.

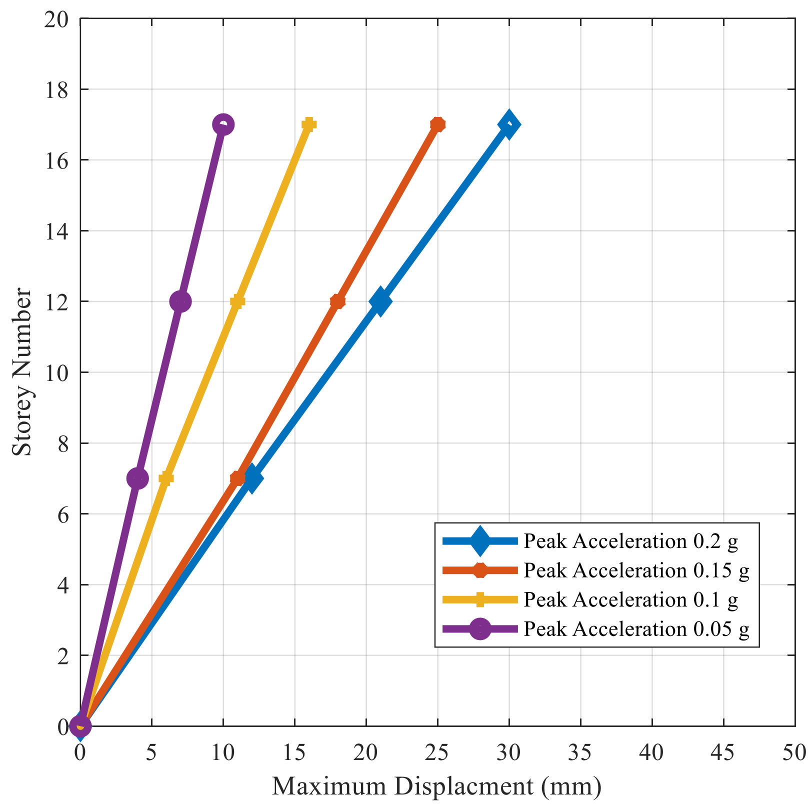


Figure 6-10 Effect of ground motion on maximum displacements (pile foundation)

6.4.4 Effect of pile length and pile number

Referring to (Figure 6-11) and (Figure 6-12) the maximum lateral rocking deflections of the structure supported by the pile foundation is greater in comparison with the fixed-base model. Increasing the pile length increases the deflection, as this increment leads to make the source of shaking being closer to the pile tip. Furthermore, when the pile length is increased, higher pile surface area is subjected to the force exerted form soil to the structure body. Increasing the number of piles reduces the deflections as expected due to the increased resistance to the friction forces applied to the pile surface.

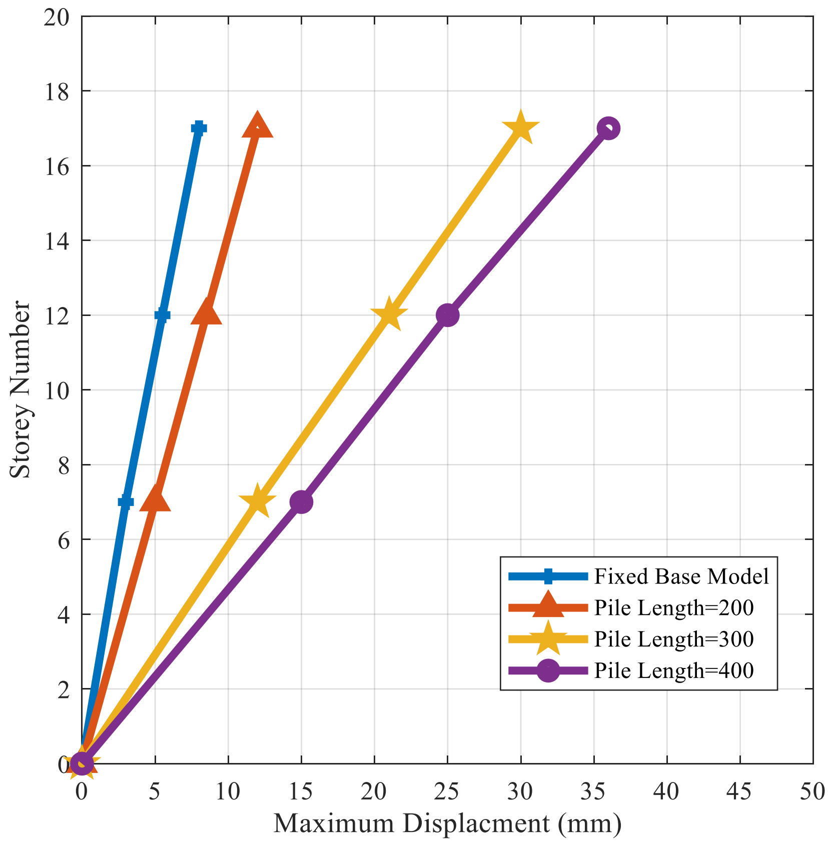


Figure 6-11 Effect of pile length on maximum displacements

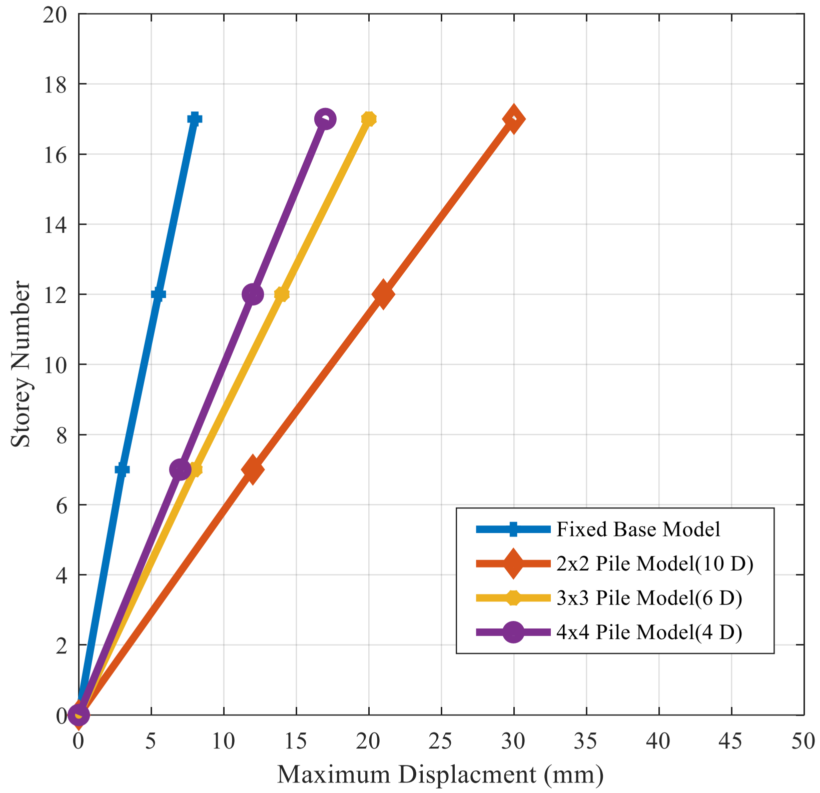


Figure 6-12 Effect of number of piles on maximum displacements

6.8 The Proposed Simplification method:

A fundamental aspect of designing an element is the analysis of applied forces and designing the structure to resist those forces. A simplification is proposed as a method of translating the soil-foundation-structure interaction into an additional moment applied to the structural system. This moment is a result of structural mass and the prediction of maximum lateral deflection of the soil-foundation-structure interaction. It is necessary to simplify a method for calculation of this deflection. The general-purpose finite element program ABAQUS was adopted in modal analysis of the soil-foundation-structure interactions with different parametric studies (Tabatabaiefar and Clifton, 2016). The proposed method is dependent on a different way of finding the maximum lateral deflection corresponding to three soil types and three structure types. Furthermore, raft foundation and raft pile were investigated in this study. The proposed method determines the correlations using factor analysis method and regression analysis method to find the best-fit equation of the output data in this study.

6.8.1 Statistical analysis tools

SPSS is the abbreviation of Statistical Package for Social Sciences and it is used by various researchers to solve complex data by statistical analysis methods. In this study, factor analysis method and regression analysis method were adopted to build correlations of the input data.

Factor analysis allow the input data by generating factors to correlated variables with one to another. it is carried out on the correlation matrix of the observed variables. A factor is a weighted average of the original variables. The factor analyst used to find a few factors from which the original correlation matrix may be generated. Usually the goal of factor analysis is to aid data interpretation. each factor identified as representing a specific theoretical factor (Comrey and Lee, 2013).

Regression analysis aims at constructing relationships between a single dependent or response variable and one or more independent or predictor variables. Regression analysis is widely used methods in data analysis. Although the computations and analysis that underlie regression analysis appear more complicated than those for other procedures, simple analyses are quite straightforward.

The general model that underlies regression analysis is based on is as the following:

$\text{Data} = \text{predictable component} + \text{unpredictable component}$

"Data" in this case is the dependent variable, the predictable component consists of the predictions generated by the regression equation, and the unpredictable component consists of the "residuals" or unpredictable parts of the data. The general idea in regression analysis is to move information into the predictable component, leaving the unpredictable component with no information or pattern (Montgomery et al., 2012).

6.8.2 Simplification Design Procedure

The proposed method determines the correlations using factor analysis method and regression analysis method to find the best-fit equation of the output data in this study. The proposed equation of maximum lateral deflections describes a correlation between structural and soil properties on raft and raft-on-pile together with the intensity of peak acceleration applied on the structural system. A set of the equations is proposed to solve the soil structure interaction based on the following assumption:

Max. lateral deflection is a relationship of the soil properties (E, ϕ, soil shear wave) see (Table 6-1) and structural properties (Table 6-2) (structure mass, structure height, structure frequency) together with foundation type under effect of earthquake peak acceleration. SPSS Statistical Software Package was utilised to Factor analysis method and regression analysis method see Appendix D. The following steps were adopted in the formation of the equations for estimation of the soil foundation structure interaction additional moment:

- Step 1: Calculation of structure factor and soil factor
- Step 2: Estimation of the scaled model Correction factor
- Step 3: Determination of Soil-Foundation-Structure Interaction.

6.8.2.1 Calculation of structure factor and soil factor

Correlations were developed using factor analysis method and regression analysis method to find the best-fit equation summarising the structural and soil properties in single factors representing the general structure and soil criteria based on the provided data.

Following the software procedure of factor analysis method, three set of soil properties were used to generate correlation factor for each of soil set (Table 6-5)

Table 6-5 Soil factor (T) analysis output

Soil Type	Dry Density KN/m^3	Modulus of Elasticity (MPa)	Angle of internal friction (ϕ)	Shear Wave Velocity m/s	**Generated Soil Factor (T)**
Loose soil	14	30	27	180	0.05130
Medium dense	15	50	30	270	0.43729
Dense soil	16	80	34	360	0.91476

Then these data were used to build a formula to estimate soil factor based on the soil properties. According to regression procedure, the Soil Factor (T) was considered as a depended variable while the soil properties as in depended variables.

Equation 6.1 presents the results of regression analysis to find the best fit to the numerical predictions of soil factor analysis (R =0.99), and the regression equation is:

$$T = -1.411 + (0.009E) + (0.064\,\gamma) + (0.002vs) \qquad 6.1$$

Where

T is Soil factor,

E= Modulus of Elasticity,

γ = Soil density,

and *v*s= soil shear velocity.

Same procedure of factor analysis was repeated to generate the structure factors for three set of structure properties (Table 6-6).

Table 6-6 Scaled structural properties

Structure Type	Structure Mass (Kg)	Structure Hight (mm)	Fixed Base Natural frequency (Hz)	**Generated Structure Factor (W)**
2B+5	8.5	340	29	-0.13
2B+10	14.62	760	16	0.43
2B+15	23	1000	9.5	1.02

According to regression procedure, these data were used in the estimation of structure factor based on the structure properties. In regression analysis the Structure Factor (W) was considered as a depended variable while the Structure properties as in depended variables.

Equation 6.2 presents the results of regression analysis to find the best fit to the numerical predictions of soil factor analysis (R =0.97), and the regression equation is:

$$\mathrm{W} = -1.322 + (0.049\ \mathrm{M}) + (0.001\ \ \mathrm{H}) + 0.014\ \ \mathrm{f} \qquad 6.2$$

Where:
W is Structural factor,
M is the structural mass,
H is structural height,
and f is the structural fixed base frequency.

6.8.2.2 Scaled Model Correction Factor

The primary purpose of the scaled model is to represent the prototype model in a perfectly correct manner. All the dimensions were scaled based on the scaling methodology explained in Chapter Three. However, in many situations a scaled model is not simply sufficient to simulate all requirement of the prototype due to cost restrictions, lack of materials, or limitations of testing facilities.

In this study, the soil shear velocity is related to soil depth and soil mechanical properties, so a correlation is required to be determined to estimate the effect of soil depth for the scaled and the prototype models. The damping difference between the material of scaled model and the material of the prototype model needs to be considered as well. Therefore, for practical reasons concessions must be made to the simulation requirements.

Depending on the assumptions being utilised, a relevant factor is required to be calculated accordingly to calculate the relationship between the prototype model and the scaled model.
Therefore, the full prototype model for the soil-foundation-structure system was built, and the behaviour was examined accordingly (Figure 6-13).

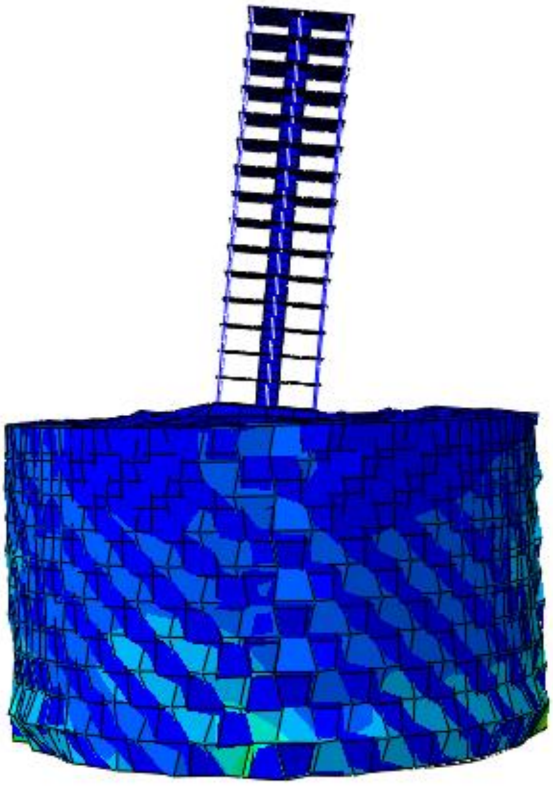


Figure 6-13 Soil-structure (prototype model) by ABAQUS

The prototype model with real dimensions of structure and soil was evaluated to examine the soil-foundation-structure interactions for three types of soil properties and then compared with the output results of the scaled model.

(Table 6-7) illustrates the ratio between the prototype and scaled model which is approximately equal to 5. In the two different cases of soil properties and foundation types, the relationship factor for the scaling factor of 1/50 is described as follows:

the prototype displacement (correction factor) = 5 X displacement of the scaled model

Table 6-7 Relationship factors for the prototype and scaled models

Soil type	Prototype Deflection (mm)	Scaled model Deflection (mm)	Foundation type	Relationship factor
Dense soil	200	41	Raft foundation	4.87
Medium dense soil	230	53	Raft foundation	4.3
Loss dense soil	250	65	Raft foundation	3.8
Dense soil	64	12	Raft on pile foundation	5.3
Medium dense soil	80	18	Raft on pile foundation	4.4
Loss dense soil	110	30	Raft on pile foundation	3.6

6.8.2.3 Determination of Soil-Foundation-Structure Interactions

The proposed concept of soil-structure interaction calculation is based on determining an additional moment applied to the foundation due to the lateral deflection and structure mass:

Additional moment = (Structure weight) x (Lateral deflection/2)

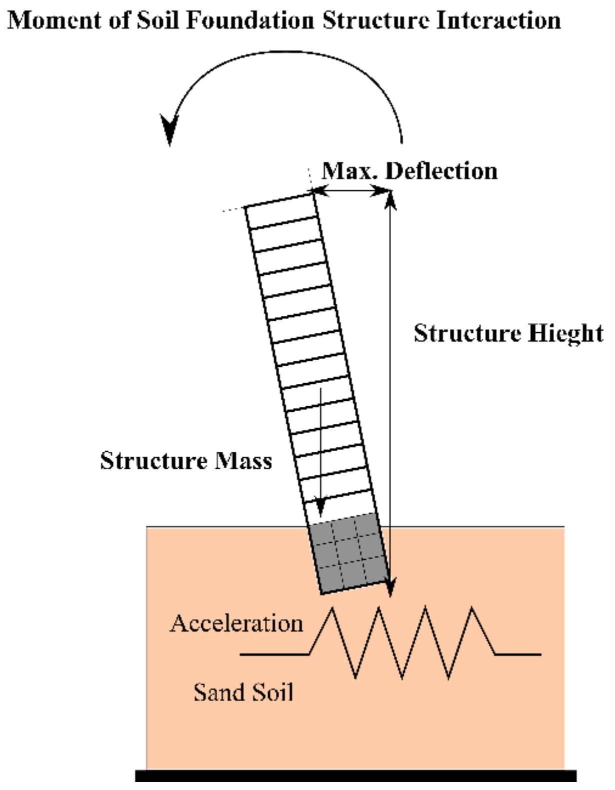


Figure 6-14 Soil-foundation-structure interaction moment of structure on raft foundation

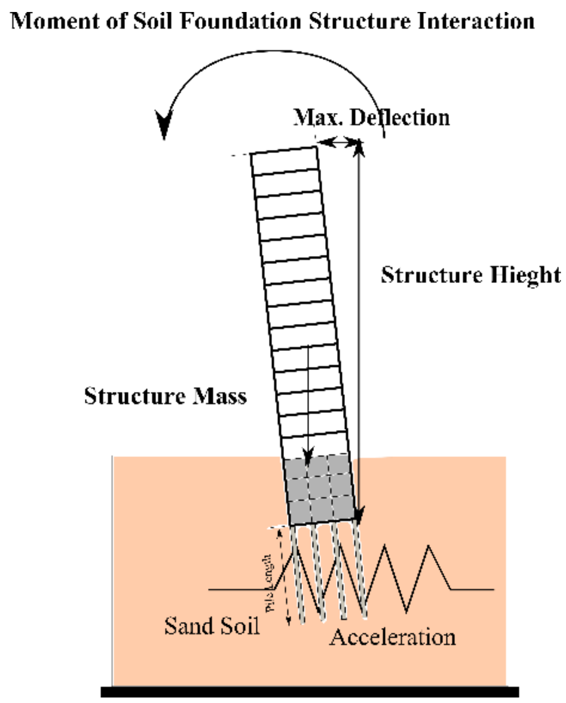


Figure 6-15 Soil-foundation-structure interaction moment of structure supported by raft-on-pile foundation

To find the maximum lateral deflection of the structure supported by raft foundation and raft on pile foundation (Figure 6-14), (Figure 6-15), the relationship was proposed between the structure factor and soil factor in terms of acceleration intensity. This correlation was built up based on the results of the parameters discussed earlier in this chapter. Summary of structure on raft parameters and structure supported by raft on the pile parameters are shown in (Table 6-8) and (Table 6-9), respectively.

Table 6-8 Summery of structure on raft parameter study

Structural factor (W)	Soil factor (T)	Ground acceleration (g)	Displacement (mm)
1.02067	0.91476	0.2	41
1.02067	0.91476	0.15	34
1.02067	0.91476	0.1	15
1.02067	0.91476	0.05	10
1.02067	0.91476	0.2	41
1.02067	0.43729	0.2	53
1.02067	0.05130	0.2	65
1.02067	0.91476	0.2	41
0.43758	0.91476	0.2	37
-0.13603	0.91476	0.2	23

Table 6-9 Summery of structure on raft-on-pile parameter study

Structural factor (W)	Soil factor (T)	Ground acceleration (g)	Pile no.	Span/Pile Diameter Ratio	Pile length	Displacement (mm)
1.02067	0.91476	0.2g	16	4	300	30
1.02067	0.91476	0.15g	16	4	300	25
1.02067	0.91476	0.1g	16	4	300	16
1.02067	0.91476	0.05g	16	4	300	10
1.02067	0.91476	0.2g	16	4	300	30
1.02067	0.43729	0.2g	16	4	300	18
1.02067	0.05130	0.2g	16	4	300	12
1.02067	0.91476	0.2g	16	4	300	30
0.43758	0.91476	0.2g	16	4	300	18
-0.13603	0.91476	0.2g	16	4	300	12
1.02067	0.91476	0.2g	16	4	300	30
1.02067	0.91476	0.2g	16	4	200	12
1.02067	0.91476	0.2g	16	4	400	36
1.02067	0.91476	0.2g	16	4	300	30
1.02067	0.91476	0.2g	9	6	300	20
1.02067	0.91476	0.2g	4	11	300	17

According to the results presented in (Table 6-8) and (Table 6-9), the best correlations were achieved in regression analyses with coefficient of determination R = 0.96.

For structures on raft foundation under the effect of soil-foundation-structure interactions:

$$SD = (6.622 + (14.531x\, W) - (26.949x\, T) + (\, 223.196\; x\; u_g))\; x\; (CF) \qquad 6.3$$

For structures supported by raft-on-pile foundation under soil-foundation-structure interactions:

$$SD = (-90.897 + (20.129\, W + 14.852\, T + 118.586\, u_g + 1.427xPn + 0.12Pl\; +\; 0.827\; Pr)\; x\; CF$$
6.4

Where

SD is the maximum lateral displacement,

W is the structural factor,

T is the soil factor,

CF is the correction factor,

u_g is the ground acceleration,

Pl is the pile length, Pr is the (pile diameter / span) ratio, Pn is the pile number

6.9 Summary

By employing the verified three-dimensional numerical model, a series of parametric studies were conducted on seventeen storey scaled model with respect to the foundation types, including a fixed base, raft foundation and raft-on-pile foundation. Different types of sand soil were considered focusing on lateral deflection behaviour. The structureal non-linearity behaviour was considered in the 3D numerical simulation.

Results of the 3D numerical simulation in this study show that the properties of the in-situ soil influence the characteristics of the peak accelerations with different soil types, However, at high acceleration levels, low stiffness and non-linearity of soil prevent the development of the peak accelerations. Moreover, earthquake records consist of greater proportions of long-period (low frequency) motion after passing through the soil deposit. The non-linear behaviour of the soil deposit influences the dynamic characteristics of ground motion by shifting the peaks in the amplification curve to the right (longer periods) and reducing the amplitudes of peak ground accelerations. In general, the ratio of the structural base shear for cases including the soil-structure interaction to that of fixed-base is less than one, demonstrating the effect of soil-structure interaction in reducing the base shear of the structure. However, the reduction ratio for the base shear is a function of the foundation type. Moreover, the amount and trend of this reduction in the structural shear forces are not the same for different levels in the superstructure. Based on the predicted maximum lateral deflections of the superstructure, the structure supported by the raft foundation experienced the most severe rocking in comparison with the raft-on-pile foundation cases, where the presence of pile elements in both cases results in considerable reduction in the maximum uplift and turn rocking experienced by the structure. Moreover, the structure supported by the pile-raft foundation experienced on average 30% less rocking in comparison to the structure supported by the floating pile foundation. In the case of floating pile foundation, this is due to the generation of compressive stresses on one side of the foundation.

The other important influence of the seismic soil-structure is its significant contribution in amplifying the lateral deflections of the structure. The amplification factor varies with the foundation type, where the presence of pile elements in raft-on-pile foundation cases reduces the amplification of the lateral deflections of the structure in comparison with the shallow foundation case. Considering the rocking dissipation, the results of this study can help the practising engineers in selecting the proper foundation type for the structures. Accordingly, the foundation types experiencing a considerable amount of rocking during an earthquake dissipate a significant amount of earthquake energy in comparison with the other types of foundations. This rocking-dissipation in turn, results in directing fewer shear forces to the superstructure and reducing the structural demand of the superstructure. However, accounting for the rocking dissipation should be adopted with extreme caution and after assessing the influence of SFSI, considering the total stability of the structure. Finally, to consider the amplification of lateral deflections of soil-foundation-structure interactions under the seismic effect of the shear wall–columns structural system, a simplified calculation method of soil-structure interactions moment has been proposed. The proposed procedure enables structural engineers to extract the maximum lateral deflections of soil-foundation structure

The following points are highly required to be studied carefully to achieve accurate results of this proposed calculation methodology and these points are:

- Improve the soil factor to include wide range of soil properties
- Improve the structure factor to include wide range of structures types and properties
- More consideration for the selected scaling factor effect by comparing the outputs of different scaling factor with existing one.
- Revised the main formula of the lateral deflection calculation to include all above.
- Furthermore; a real structure with rocking failure are required to be considered with this proposal method.

The proposed calculation procedure can be employed for practical purposes by structural engineers and engineering companies, as a reliable method of considering SFSI effect in the seismic design procedure. The proposed simplified design procedure can only be employed in the seismic design of regular shear wall column structural system resting on raft foundations and raft-on-pile foundation embedment depth and does not cover irregular and high-rise buildings. Brief MATLAB coding of the soil-foundation-structure interaction calculations is shown appendix E

CHAPTER 7 - CONCLUSIONS AND RECOMMENDATION

7.1 General

In the experimental part of this study, the simulation of soil-foundation-structure models was physically conducted with a geometric scale factor of 1:50. The detailed modelling techniques were explained including the design of the soil mix and soil container which can be used by future researchers to acquire further validation and achieve more accurate models in the method of 1g shaking table test.

A multi-storey superstructure was adopted as a prototype to study the dynamic properties including the first and higher order mode natural frequency, number of stories and density. A fixable membrane soil container was developed to simulate the soil free field response. This membrane container helps to minimise the boundary effects. The proposed experimental shaking table tests provided a valuable comprehension of the response of the structure to different conditions such as types of sandy soil, foundations and structure by simulating soil properties, superstructure, different foundation types and input motions. Four sets of shaking events were adopted in this study to obtain the response spectra type 2, EC8. These are unique shaking table tests experiments as they consider the structural model foundation (raft foundation, raft on pile foundation) and soil container in the soil-structure system with more accuracy. Sand soil with specific properties was adopted. Furthermore, to reduce the effect of container boundary on the dynamic behaviour of soil, a fixable membrane was utilised as a soil container boundary wall for shaking table tests and its lateral movements. Shaking table tests are almost identical to the normal behaviour of the free field movements of soil in reality. Four shaking events accelerations were utilised in terms of time displacement inputs in the programming of the shaking table. The experimental works were divided into four stages. The first stage is the fixed base model considering the dynamic behaviour of the structure without the effect of the soil interaction. The second stage is the soil container stage studying the dynamic properties of soil and soil container without the effect of the structure. The third stage is a soil foundation structure model for structure supported by raft foundation studying the dynamic behaviour of structure supported by raft foundation under seismic effects and finally, the stage four which is the soil foundation structure of structure supported by raft on pile foundation. Then, the maximum structural lateral displacements predicted by the numerical soil-structure model of those four stages were determined and compared with the experimental results. A comparison between the predicted and recorded lateral structural displacements showed that the predicted numerical outputs and experimental measurement results are in good agreement for all four stages. Therefore, the designed numerical model can be employed to further study the soil-structure dynamic behaviour more accurately.

The impact of SFSI under the effect of dynamic loads has been well investigated. Since the 1990s in order to solve soil foundation structure interaction problems, different design methods were proposed to substitute the classical design methods by the new design method based on the structure performance during the seismic excitations. There is a strong need to establish a design methodology to determine the response of the structure under seismic effects considering the foundation types effect and the impact of subsoil conditions. The direct incorporation analysis helps to capture the energy absorbing, hysteresis behaviour and characteristics of the real soil. The Mohr-Coulomb failure criterion is utilised to define the soil criteria.

7.2 Summary

To achieve a clear comprehension of the soil foundation, and the impact of structure on the structural response under the seismic effects, three-dimensional numerical models were developed using ABAQUS software to perform nonlinear time-history analyses on the soil-foundation-structure system. Finite element analyses were performed using real earthquake recordings taking into account both materials (soil, foundation and superstructure) and geometric nonlinearities, where hysteretic damping of the soil was implemented to represent the variations of the shear modulus and damping ratio of the soil with the cyclic shear strain capturing the energy absorbing characteristics of the soil. The outward propagating waves were prevented to reflect back into the model by considering the lateral boundaries of the soil container as a fixable membrane element. Moreover, rigid boundary conditions were applied to model the soil container base to investigate the seismic soil-structure interaction, and the earthquake input motions were adopted to the container base horizontally propagating upward throughout the model.

A numerical investigation was conducted on 2B+5, 2B+ 10, and 2B+15 storey structural models with three types of sand soil foundations: dense sand, medium dense, and loose sand soil. According to the results, it was evident that for the models resting on soil, the lateral deflections of the flexible base were not similar to the fixed base model's analysis cases.

- The lateral deflection for both types of foundation (raft, raft on the pile) increases with ground motion intensity increase due to an increase in inertia force applied to the foundation base.

- Structures resting on the raft foundation were subject to the most intense rocking in comparison with the raft on pile foundation cases, where the presence of pile elements significantly reduce the maximum uplift and in turn the rocking applied to the structure. The raft on pile foundation structures experienced 30% reduced rocking on average in comparison to the structures supported by the raft foundation. This is a consequence of compressive stress generation on one side of the foundation in the floating pile foundation case.

- Amplification of the lateral deflections is affected by the seismic soil-structure and varies with the foundation type. The presence of pile elements in raft on pile foundation cases reduces the amplification of the structure's lateral deflections in comparison with the raft foundation.

- The performance of structures supported by raft foundation resting on soil is affected by the soil properties. Lateral deflections extensively increase when the soil stiffness decreases due to the shear stress applied compared to the models with fixed base. Any increase in the structure height will lead to rising Lateral deflections. While, The response of raft on pile foundation structures resting on soil is affected by the soil properties. Lateral deflections increase with higher soil stiffness, the higher soil density leads to increasing soil mass which as a result increases the seismic force. Therefore, the force transfers from the soil to the pile element and causes more deflection compared with fixed base models. Raising the structure height increases the lateral deflections.

- The number of piles and their length has a direct relationship with the lateral deflections due to increasing soil mass-pile composition. Furthermore, stronger pile soil connection transfers more force to the structure through the pile elements.

- This study can help the practising engineers with the evaluation of the soil-foundation-structure effect on the response of the structures under seismic effect. Accordingly, the foundation types subjected to significant earthquake rocking dissipate a considerable amount of earthquake energy. This in turn, results in fewer shear forces being directed to the superstructure, thus reducing the superstructure's structural demand.

However, extreme caution should be adopted when accounting for the rocking dissipation and after assessment of the influence of SFSI, bearing in mind the following points:

- There is a reduction in the shear forces applied on the soil foundation structure interaction due to the dissipation of seismic energy as a result of structure deflection.
- The structure rocking displacement must be critically determined with regards to soil, foundation and structure criteria.
- The total structure stability should be considered carefully.

In this thesis, based on the numerical results a simplified method of calculating the soil foundation structure interaction is proposed. The simplified procedure determines the additional moment generated due to the lateral deflections corresponding to structure mass and the influence of soil foundation structure interaction on the seismic design of regular multi-storey structures.

In this simplified design procedure, equations based on different parameters were proposed to calculate the influence of SFSI. Consequently, design safety and reliability can be ensured by more precise capturing of the detrimental effects of soil-structure interaction under the seismic effects. Structural engineers and engineering companies will be able to employ the proposed simple calculation method as a reliable means of considering SFSI effect in the seismic design procedure.

7.3 Recommendations for future work

The purpose of this research project was to assess the influence of different types of foundations types and soils on the seismic response of shear wall-columns structural systems. Development of new design procedures with further numerical and experimental studies is recommended in order to consider the effect of different parameters.

Future research work may be carried out in the following areas:

- Conduct physical shaking table model and a numerical model to consider a wider range of common foundations types and different characteristics such as foundation size, basement wall interactions, active and passive pressures of the subsoil on the response of the system during the earthquake excitations, and pile group arrangement. The outcomes of this study can be applied to different foundation types.

- Extend the numerical and experimental investigations to determine the seismic response of multi-storey buildings resting on various soil types under seismic effect. Thus, the soil and structure factors introduced in the current study can be assessed over a wider range of soil foundation structure systems.

- Employ different scale factors to investigate the accuracy of the current study.

- Adopt further investigations in the cases of irregular (shear wall-columns) structural systems which are a common case in practice.

- The proposed equations in this study and similar works are to develop a new design procedure to bridge the current gap in the available design codes. The proposed design procedure should be able to address the influence of foundation type, soil layers, soil types and different structural systems. The proposed design procedure should be further improved to cover a wider range of seismic problems in the engineering practice.

- Conduct the numerical parametric study to determine the effects of structural material strength variations on the soil foundation structure interaction under seismic conditions. Many construction materials can be taken into consideration in the numerical model to consider the response of soil foundation structure interaction under seismic effects. These materials can include steel, timber.

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Appendixes

Appendix A Scaled model drawing

Appendix B Shaking table experimental outputs

Appendix C Numerical model outputs

Appendix D Factor analysis and regression outputs

Appendix E MATLAB coding

Appendix A

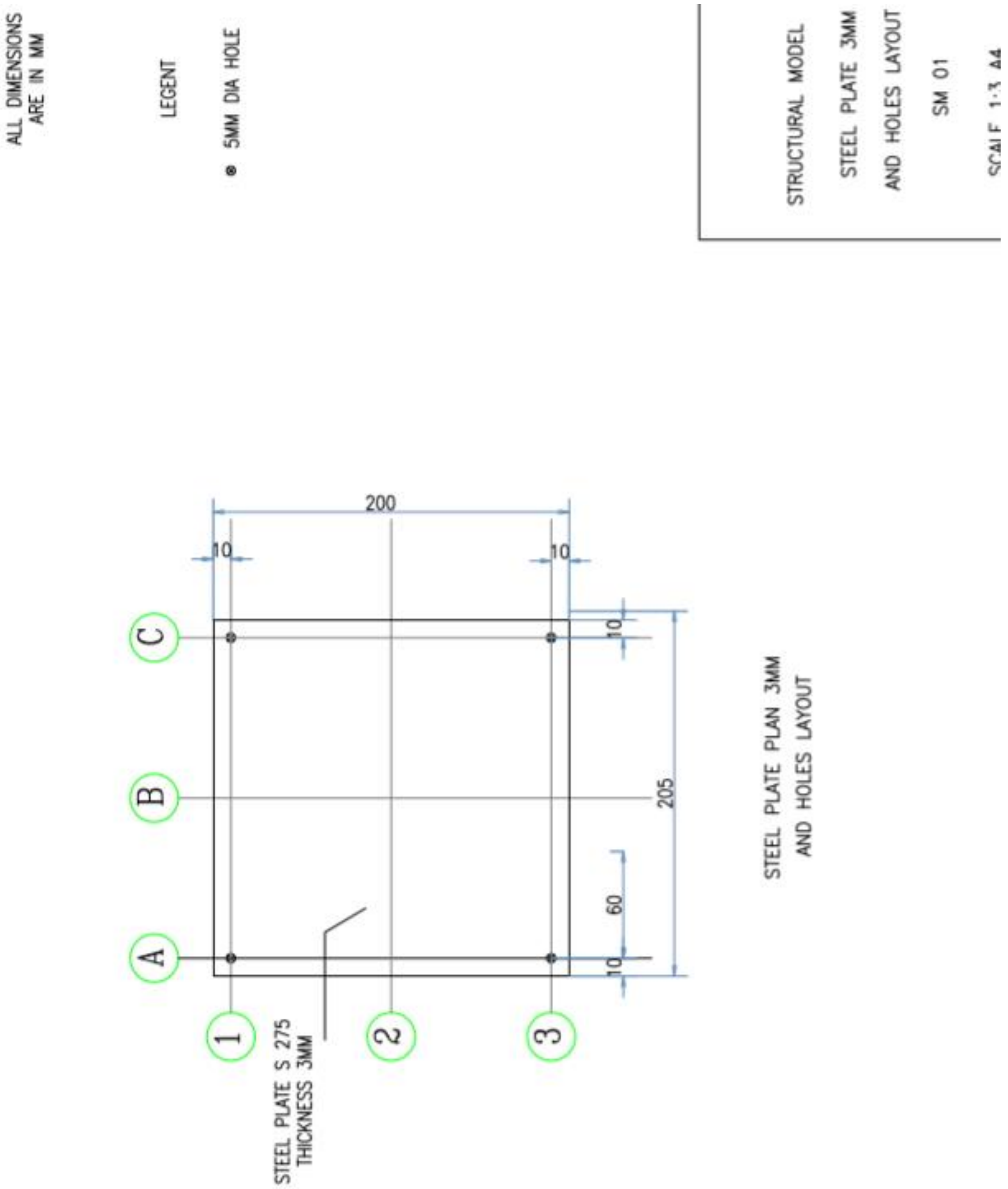
Scaled model drawing

TOTAL QUANTITY OF REQUIRED ELEMENTS

ELEMENTS SCHEDULE				
REQUESTED ITEMS	DRAWING TITLE	DRAWING NO.	QUANTITY NO.	NOTE
STEEL PLATE PLAN 3MM	STEEL PLATE 3MM AND HOLES LAYOUT	SM–01	4	
STEEL PLATE PLAN 2MM	STEEL PLATE 2MM AND HOLES LAYOUT	SM–02	13	
STEEL BAR DETAIL	STEEL BAR CONNECTOR	SM–03	4	
STEEL TUBE SECTION	STEEL BAR CONNECTOR	SM–04	156	
STEEL PLATE FOR BOUNDARY WALLS	STEEL PLATE FOR BOUNDARY WALLS	SM–03	1	
RAFT STEEL PLATE	BASE STEEL PLATE FOUNDATION	SM–06	1	

STRUCTURAL MODEL

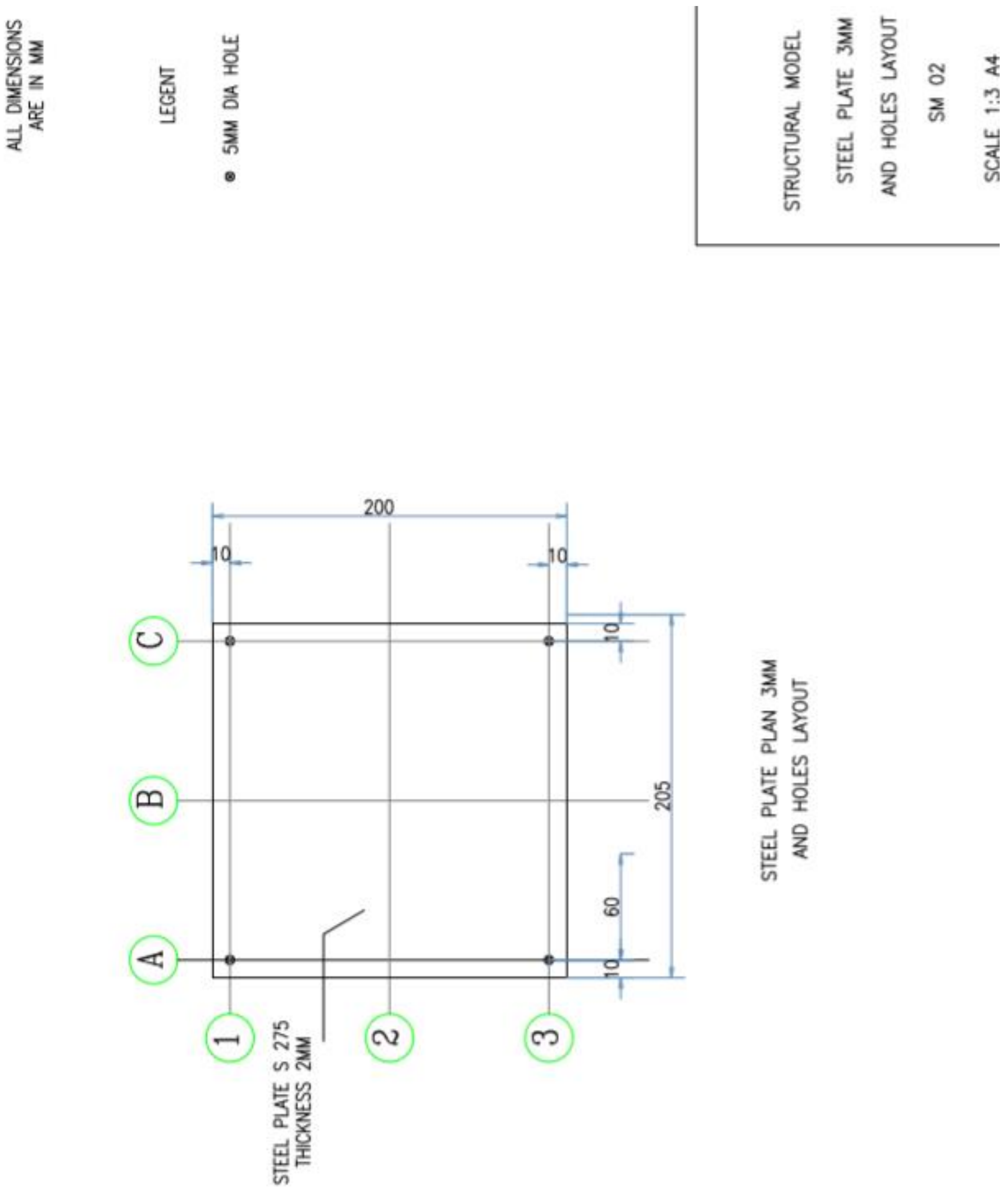
SM 00



GENERAL NOTE :

1 – ALL DIMENSION ARE IN MILLIMETER UNLESS OTHERWISE SPECIFIED

2 – STEEL PLATE GRADE S275 SHALL BE USED IN ACCORDANCE WITH BRITISH STANDARD BS 5950



GENERAL NOTE :

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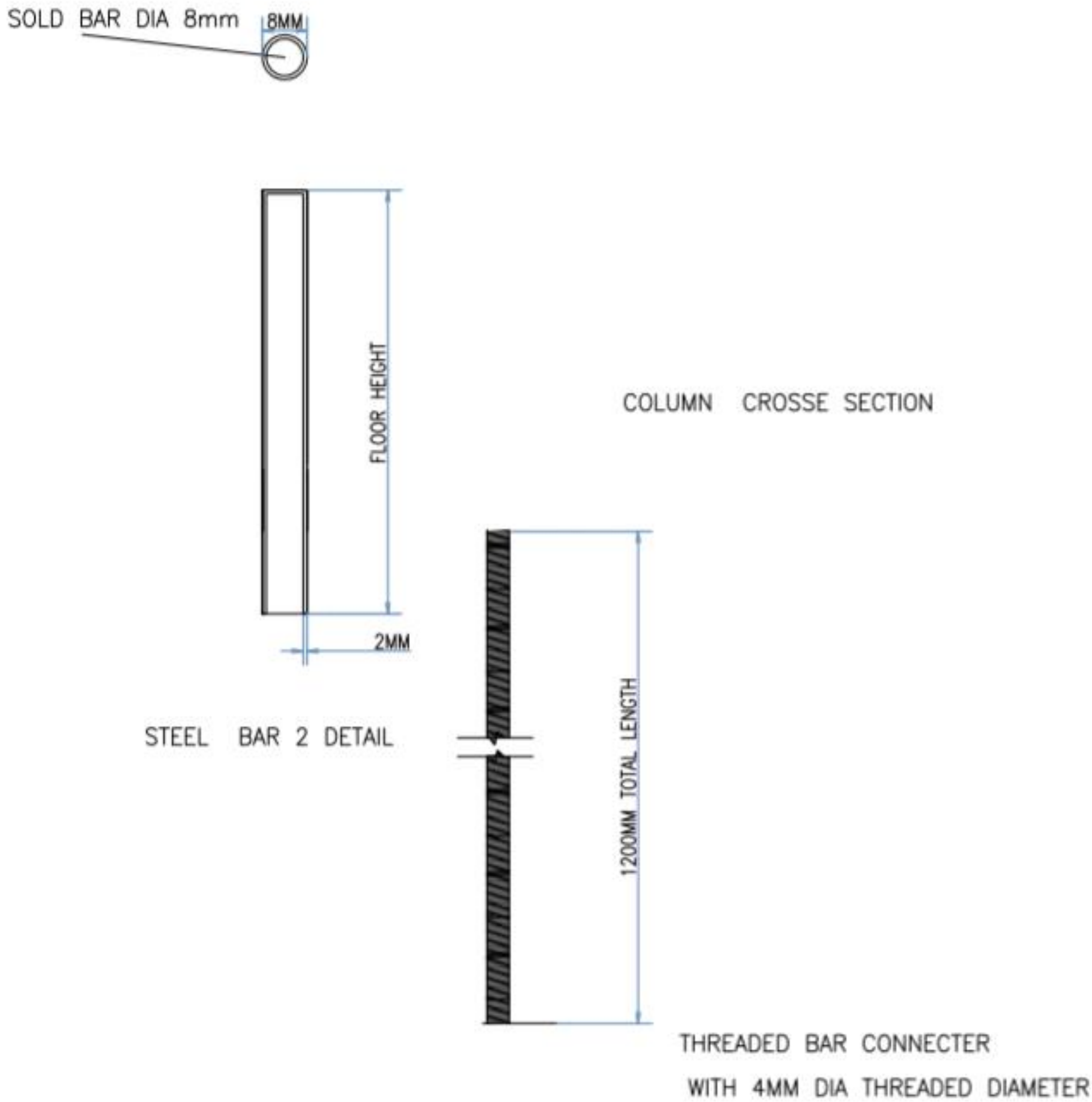


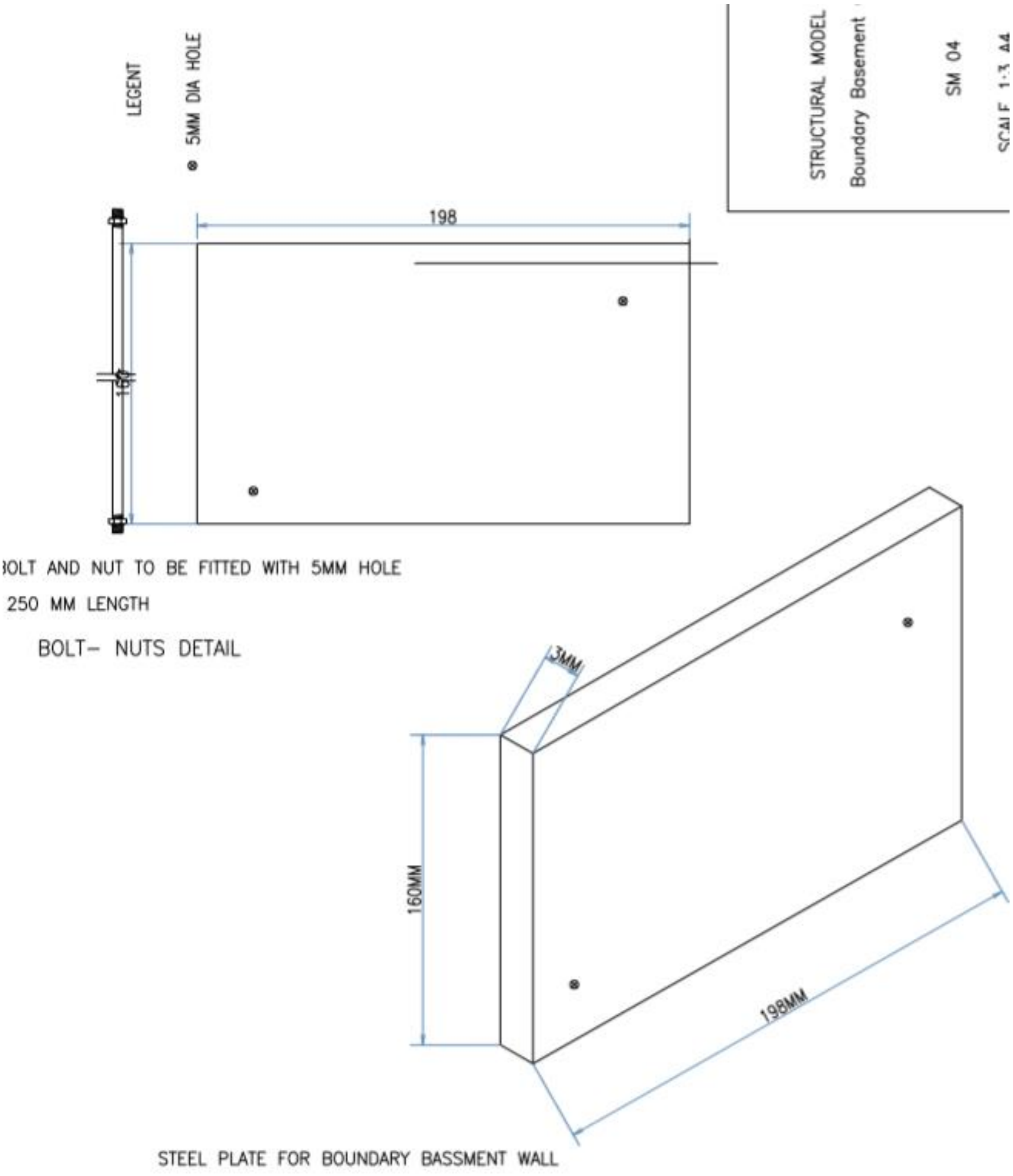
GENERAL NOTE :

1 – ALL DIMENSION ARE IN MILLIMETER UNLESS OTHERWISE SPECIFIED

2 – STEEL PLATE GRADE S275 SHALL BE USED IN ACCORDANCE WITH BRITISH STANDARD BS 5950

STRUCTURAL MODEL

STEEL BAR
DETAIL FOR TYPICAL
FLOORS
SM 03



GENERAL NOTE :

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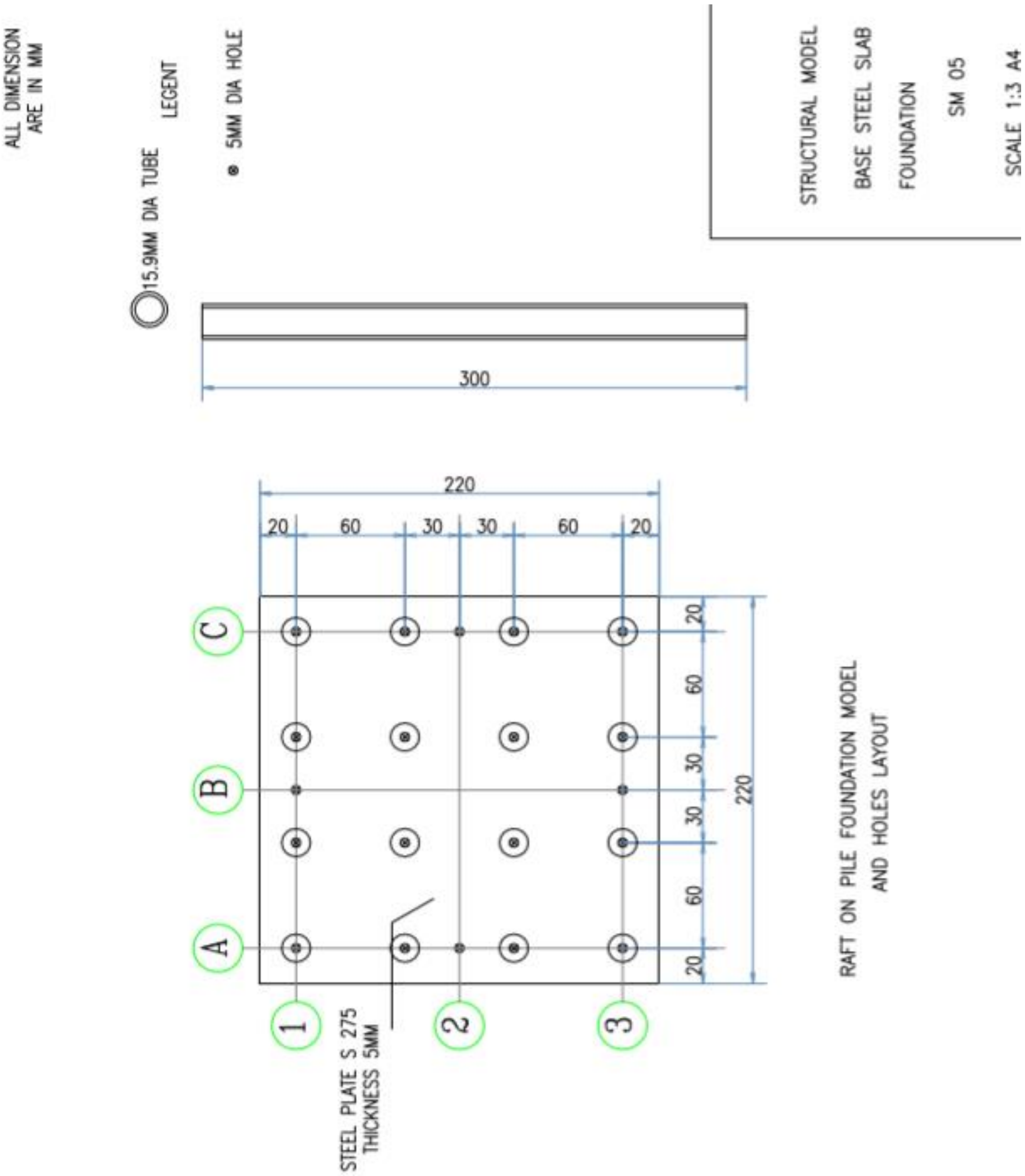
ALL DIMENSION
ARE IN MM
15.9MM DIA TUBE
LEGENT
5MM DIA HOLE
STRUCTURAL MODEL
BASE STEEL SLAB
FOUNDATION
SM 05
SCALE 1:3 A4
300
220
20 60 30 30 60 20
A
B
C
1
2
3
STEEL PLATE S 275
THICKNESS 5MM
RAFT ON PILE FOUNDATION MODEL
AND HOLES LAYOUT

Appendix B

Shaking table experimental outputs

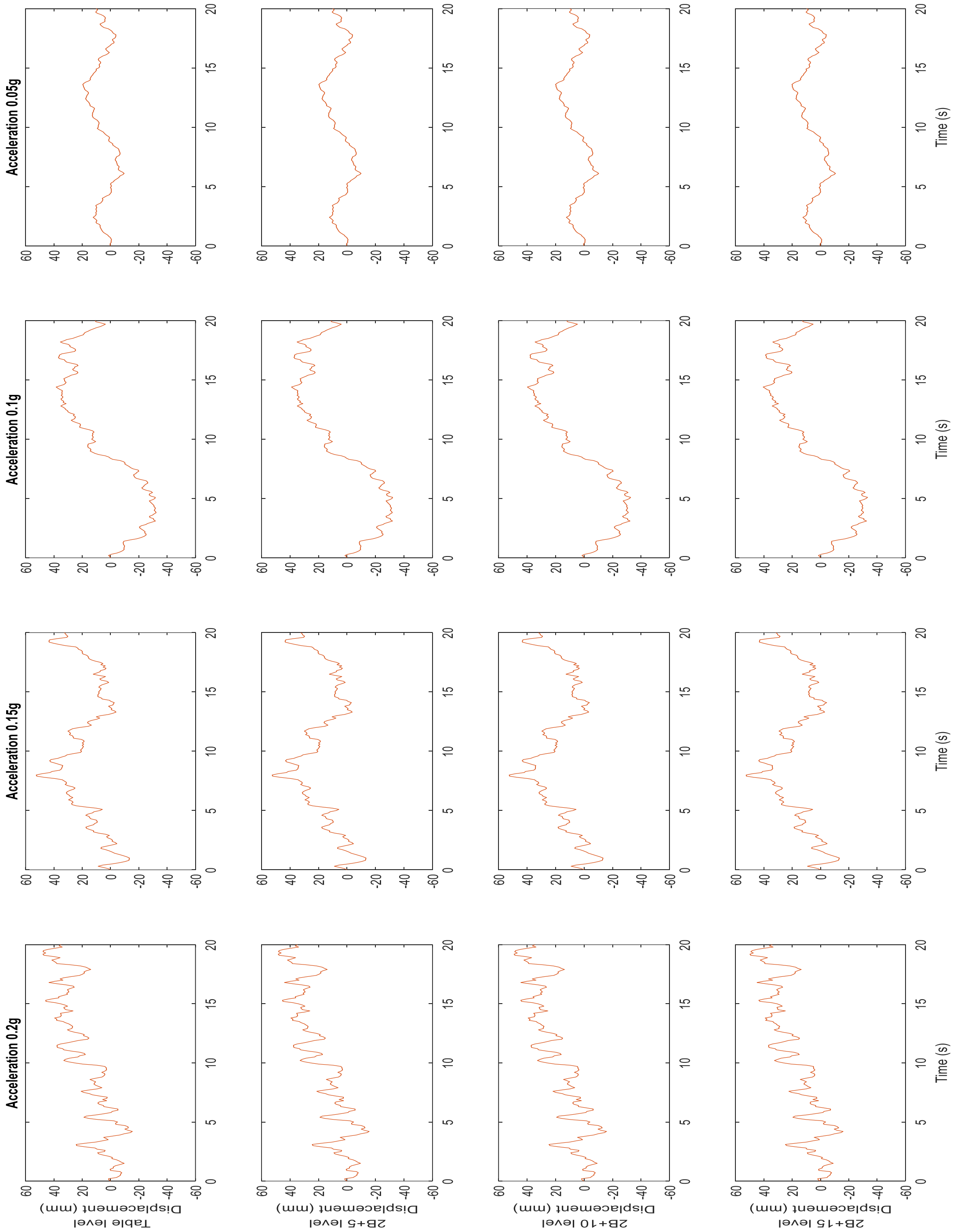

Figure A Fixed base stage experimental outputs

Soil foundation structural interaction test (Raft foundation)

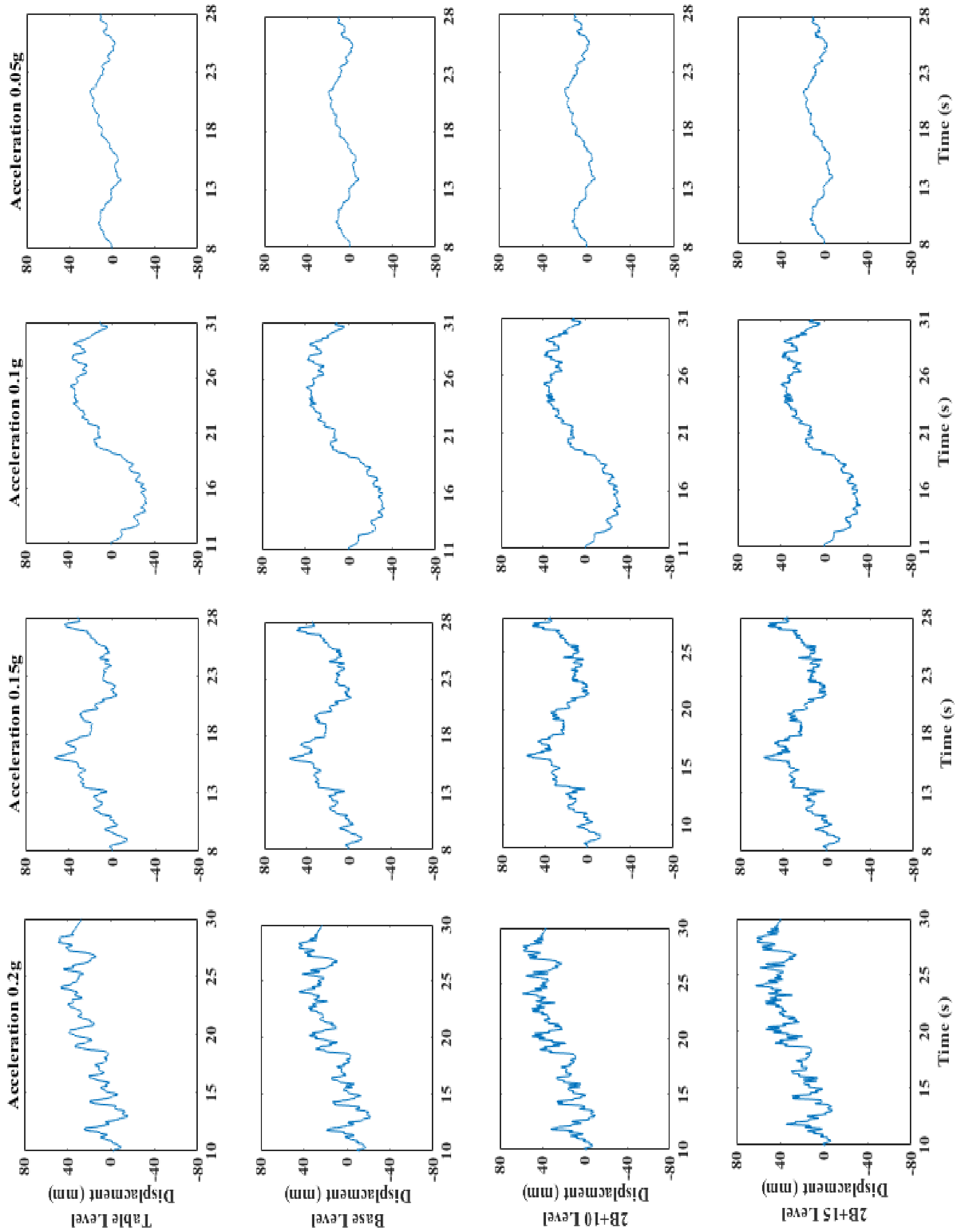

Figure B Experimental acceleration intensity effect (raft foundation)

Soil foundation structural interaction test (Pile foundation)

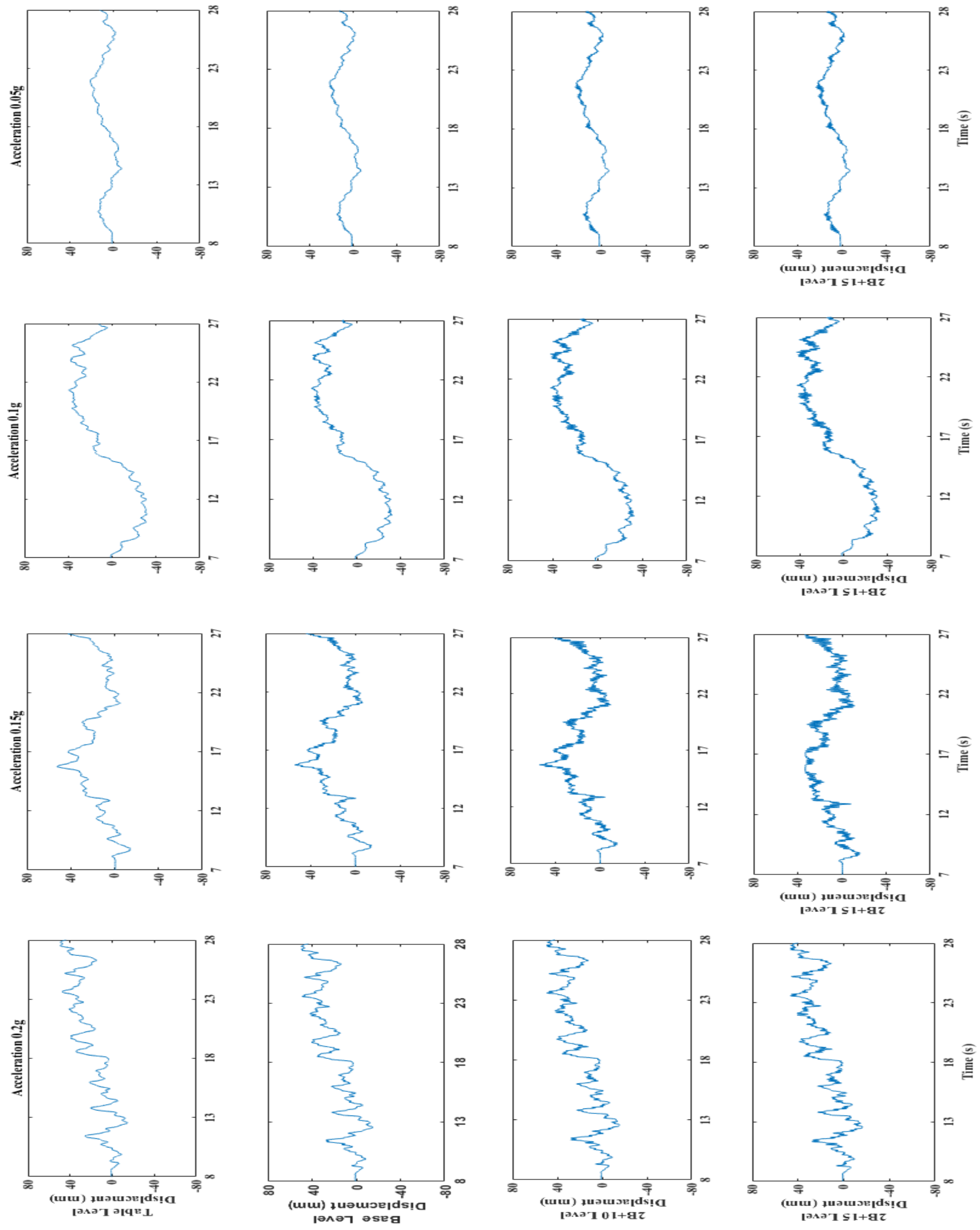

Figure C Experimental acceleration intensity effect (pile foundation)

Appendix D

Numerical model outputs

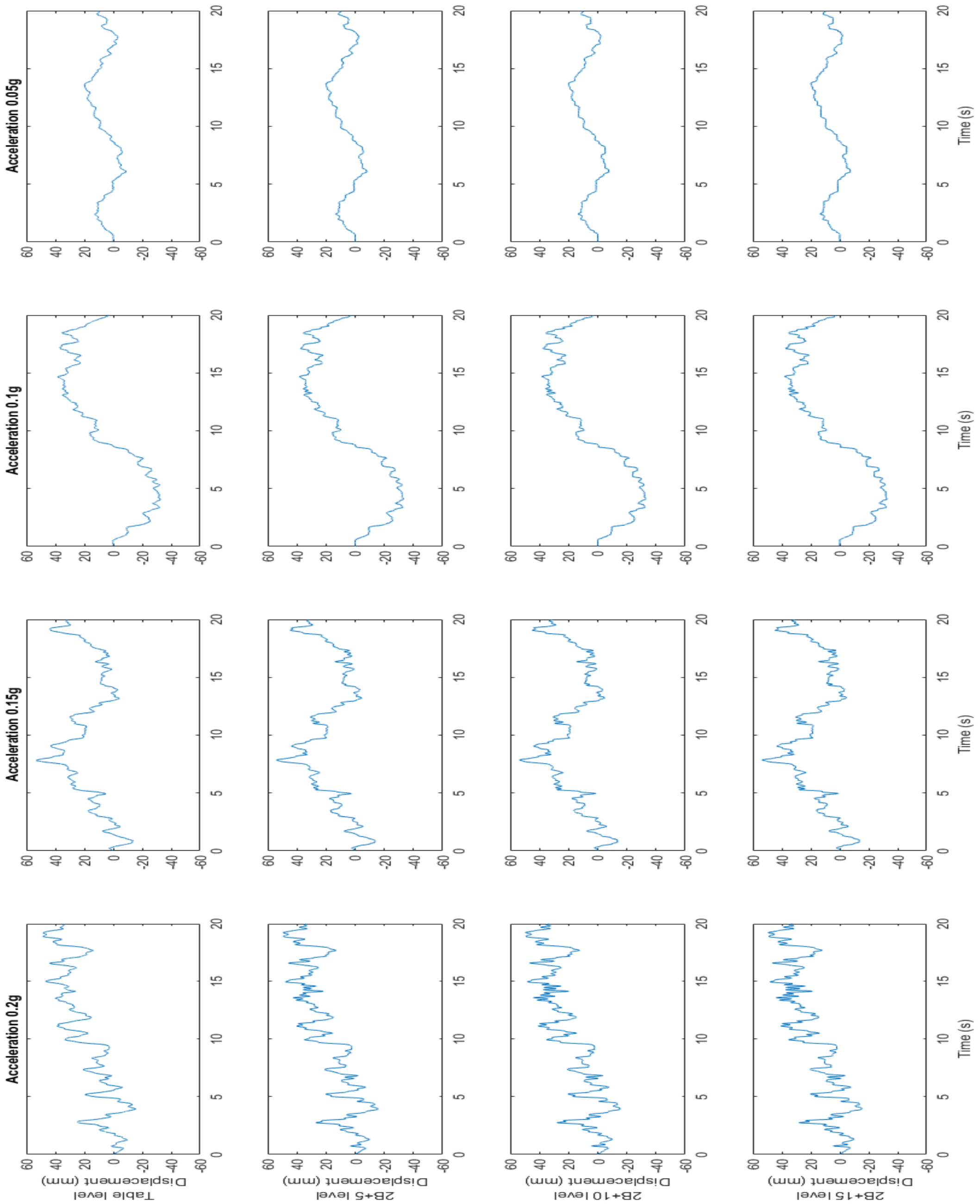

Figure D Fixed base stage numerical outputs

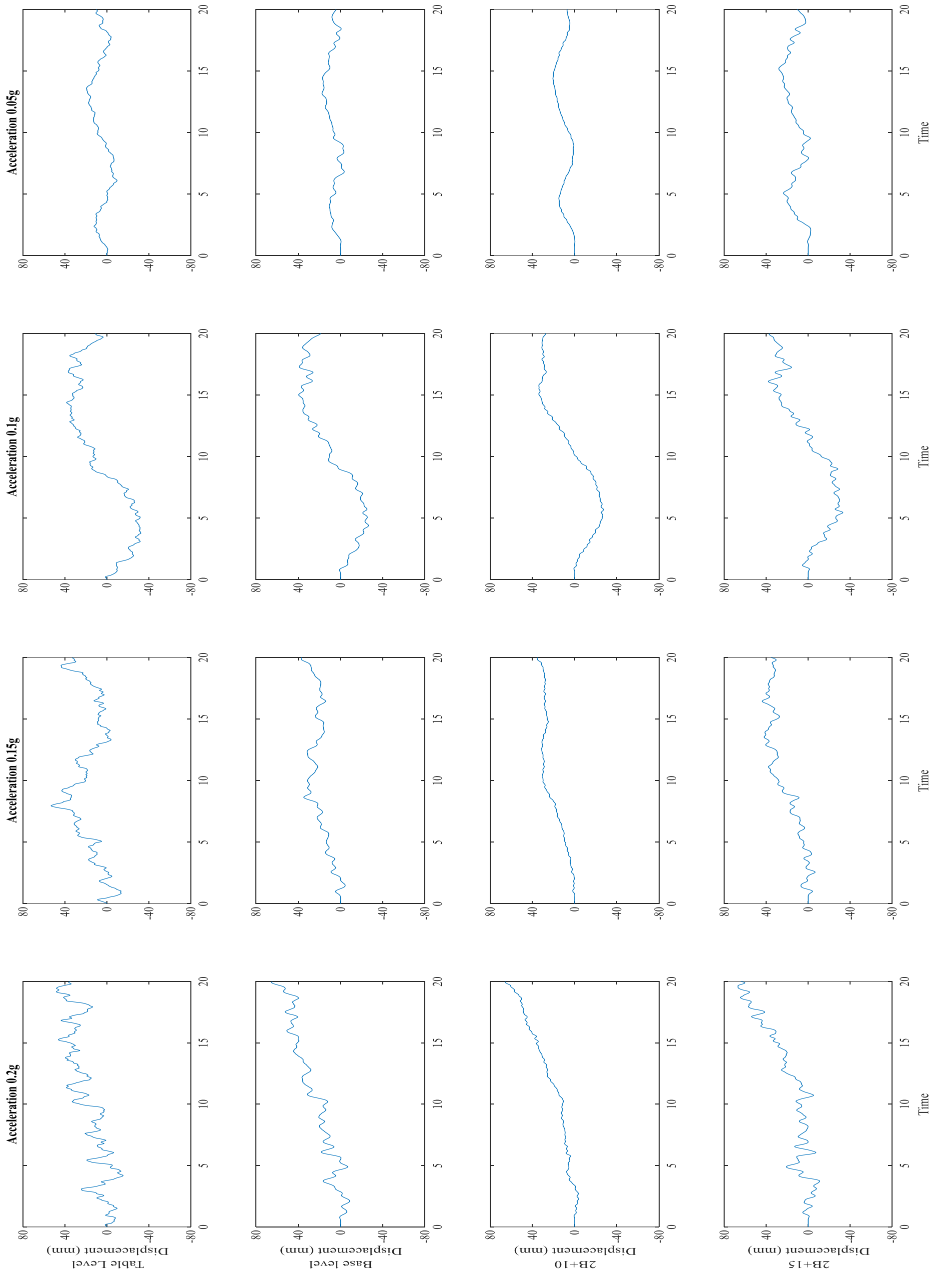

Figure E Numerical acceleration intensity effect (raft foundation)

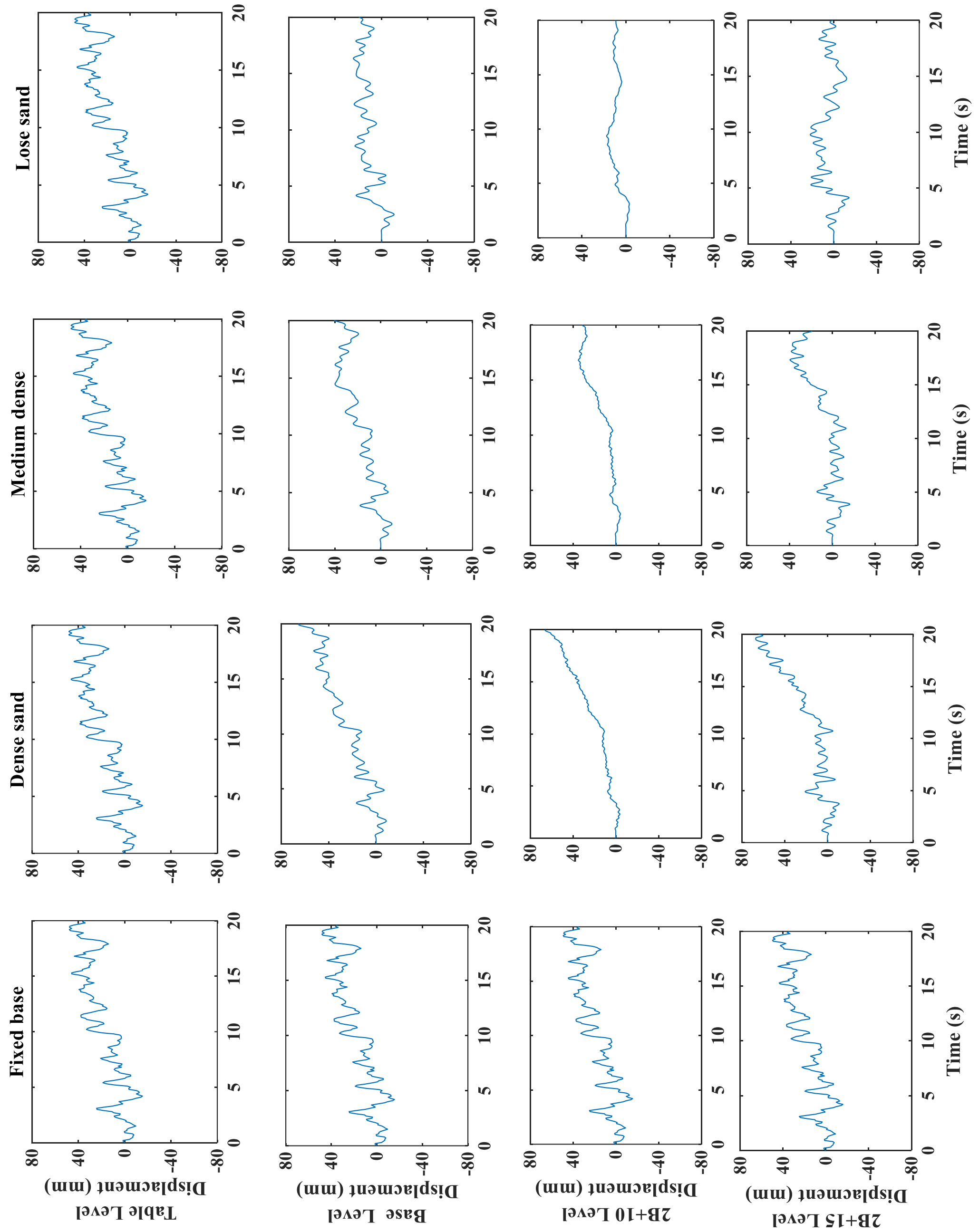

Figure F Numerical soil effect (raft foundation)

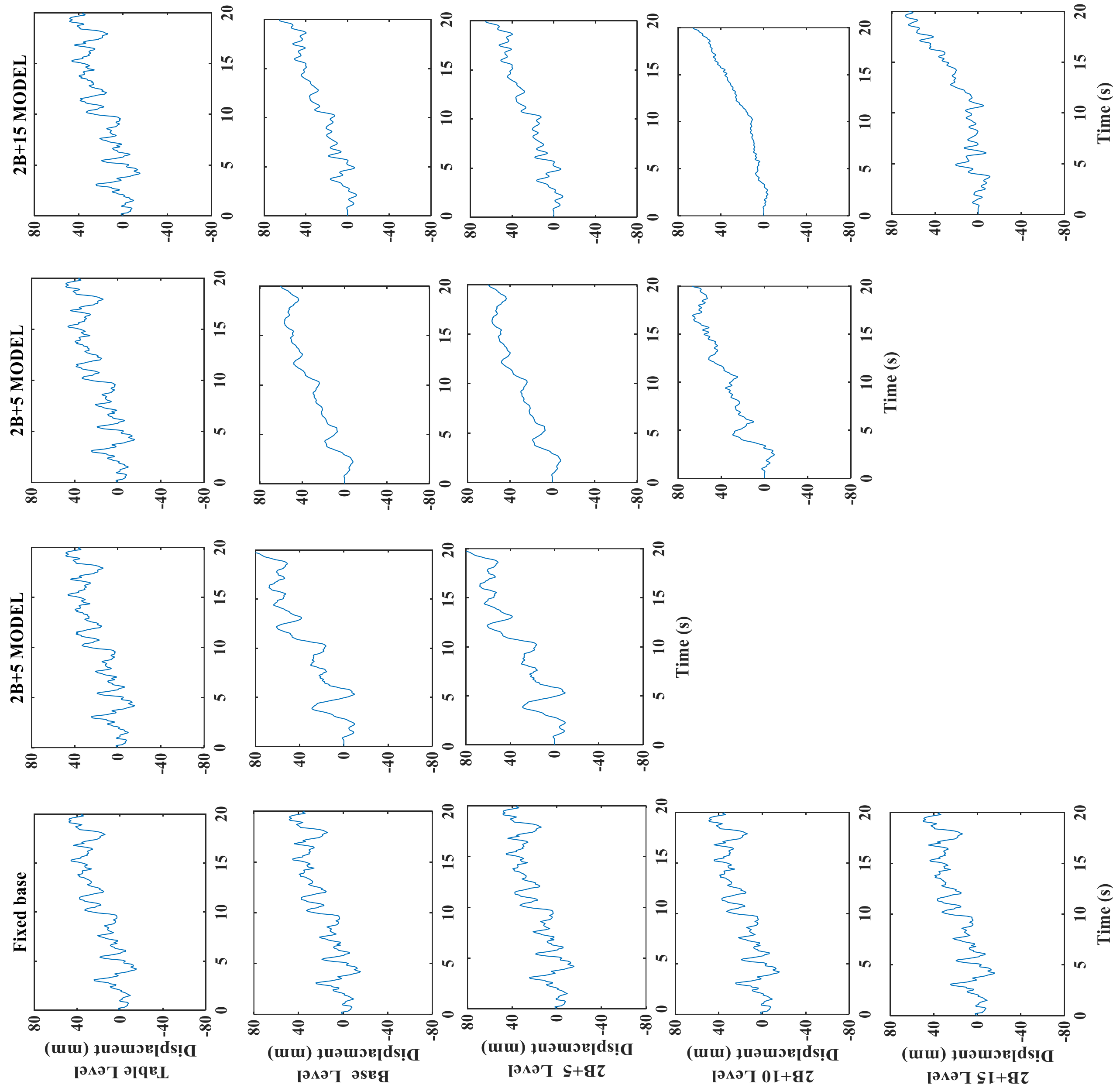

Figure G Structure properties effect (Raft foundation)

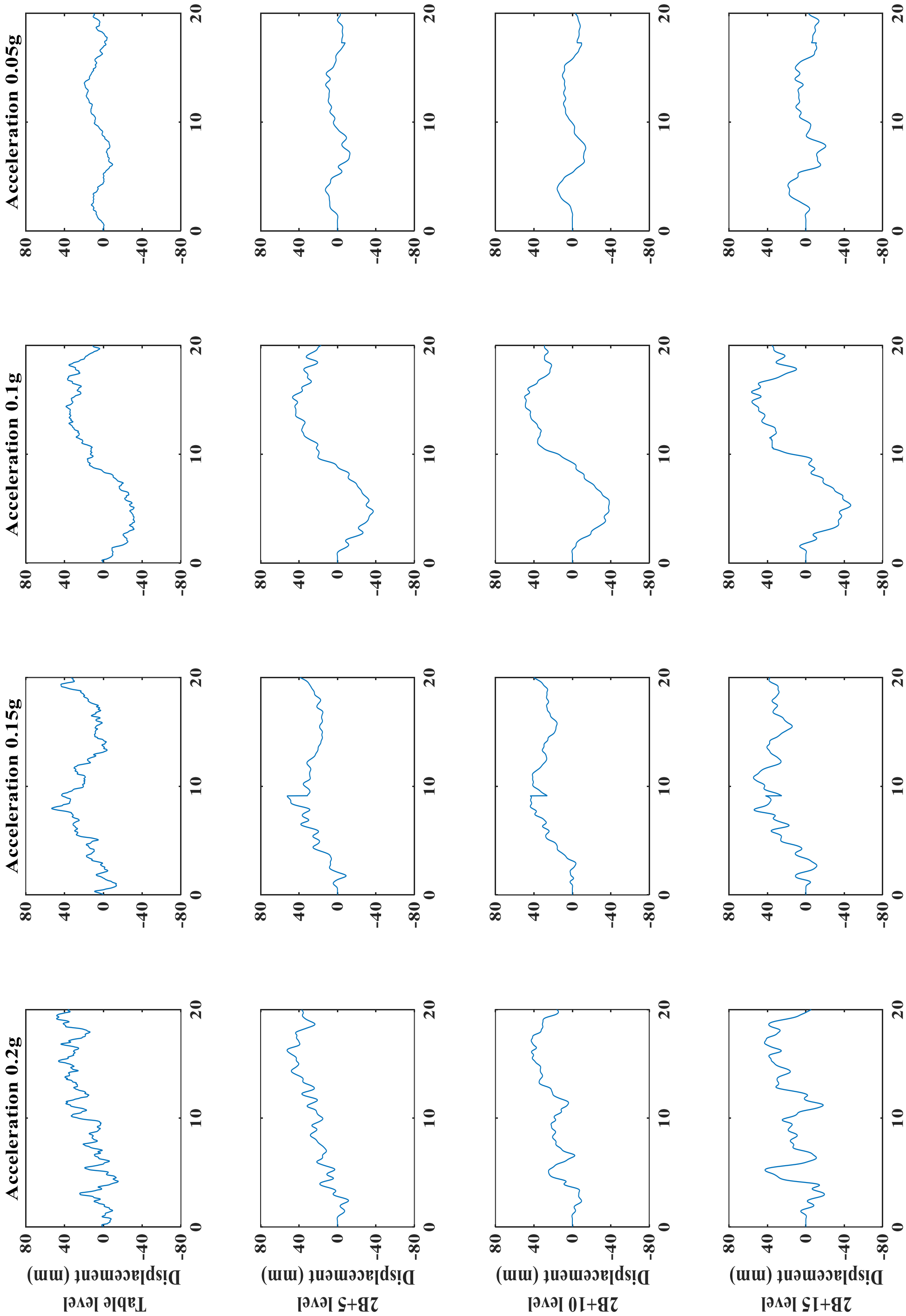

Figure H Numerical acceleration intensity effect (pile foundation)

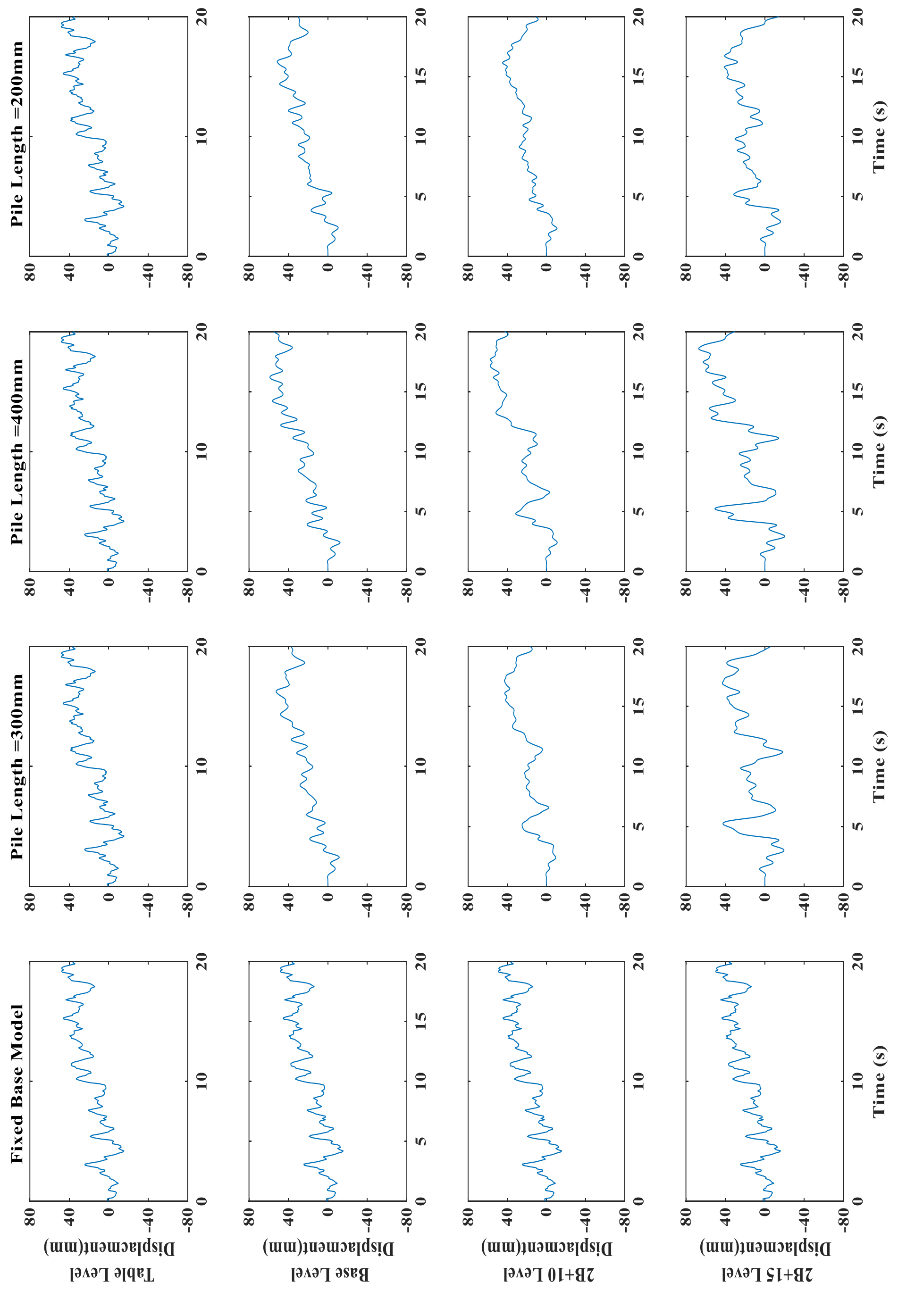

Figure I pile length effect (pile foundation)

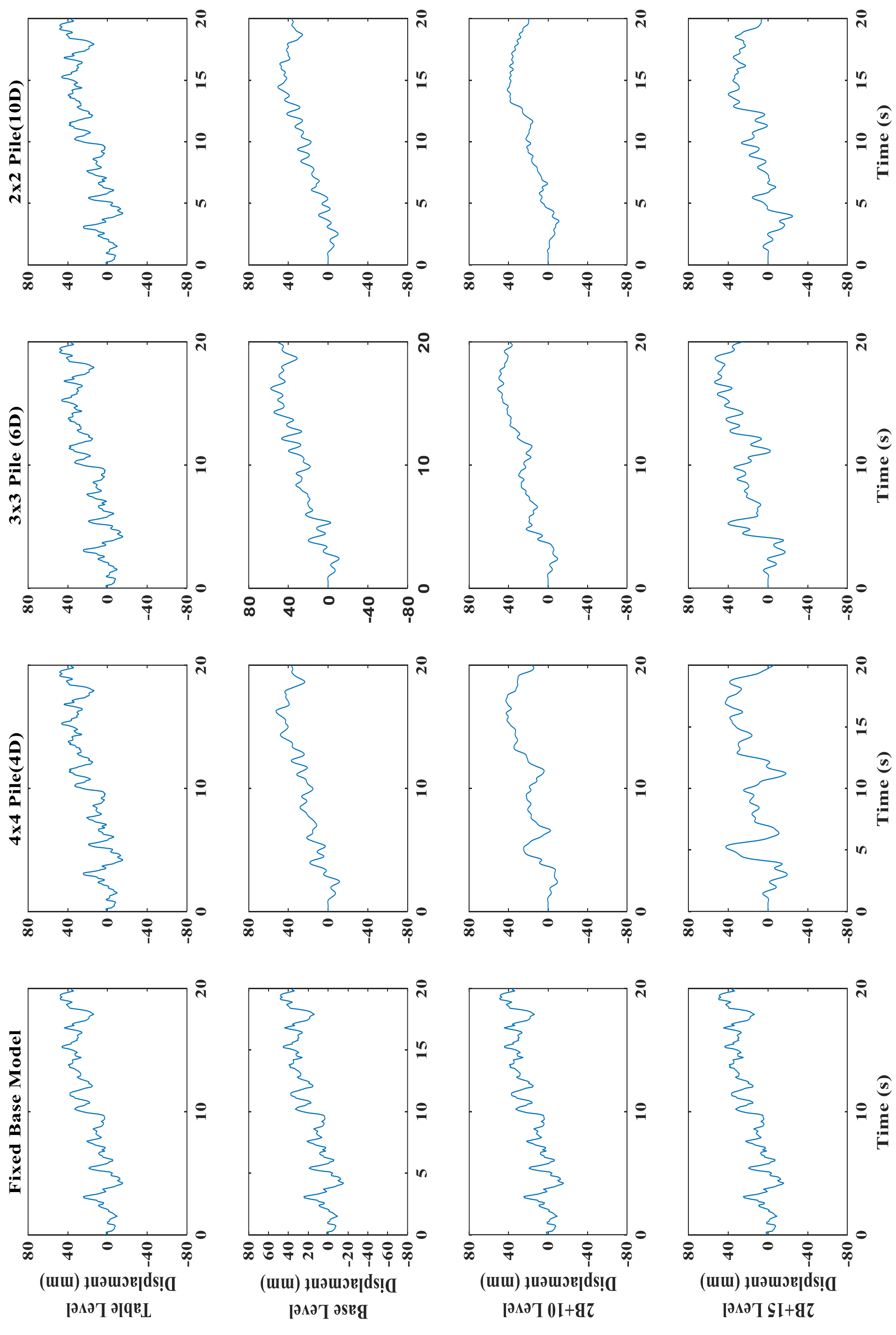

Figure O pile number effect (pile foundation)

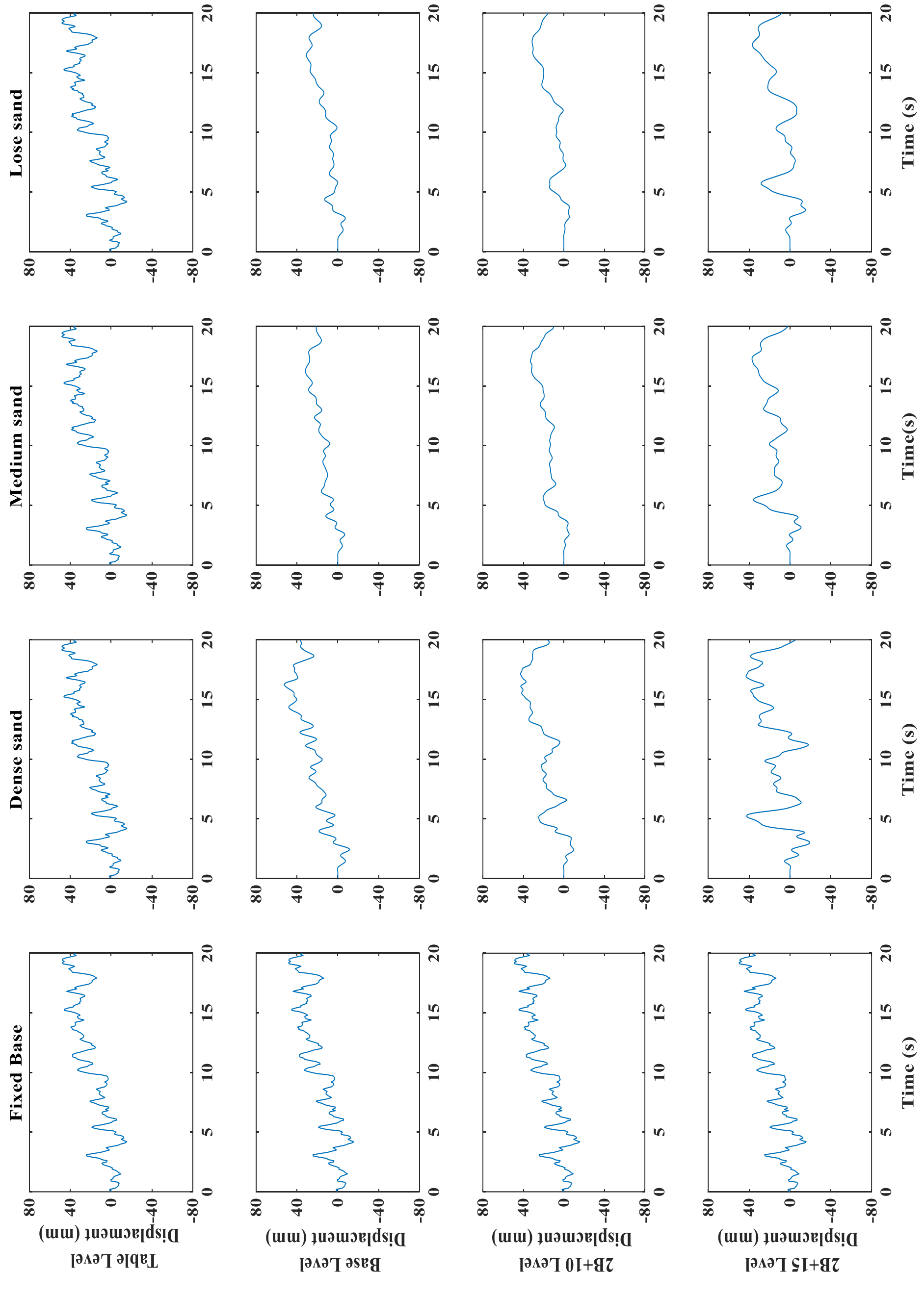

Figure P soil effect (pile foundation)

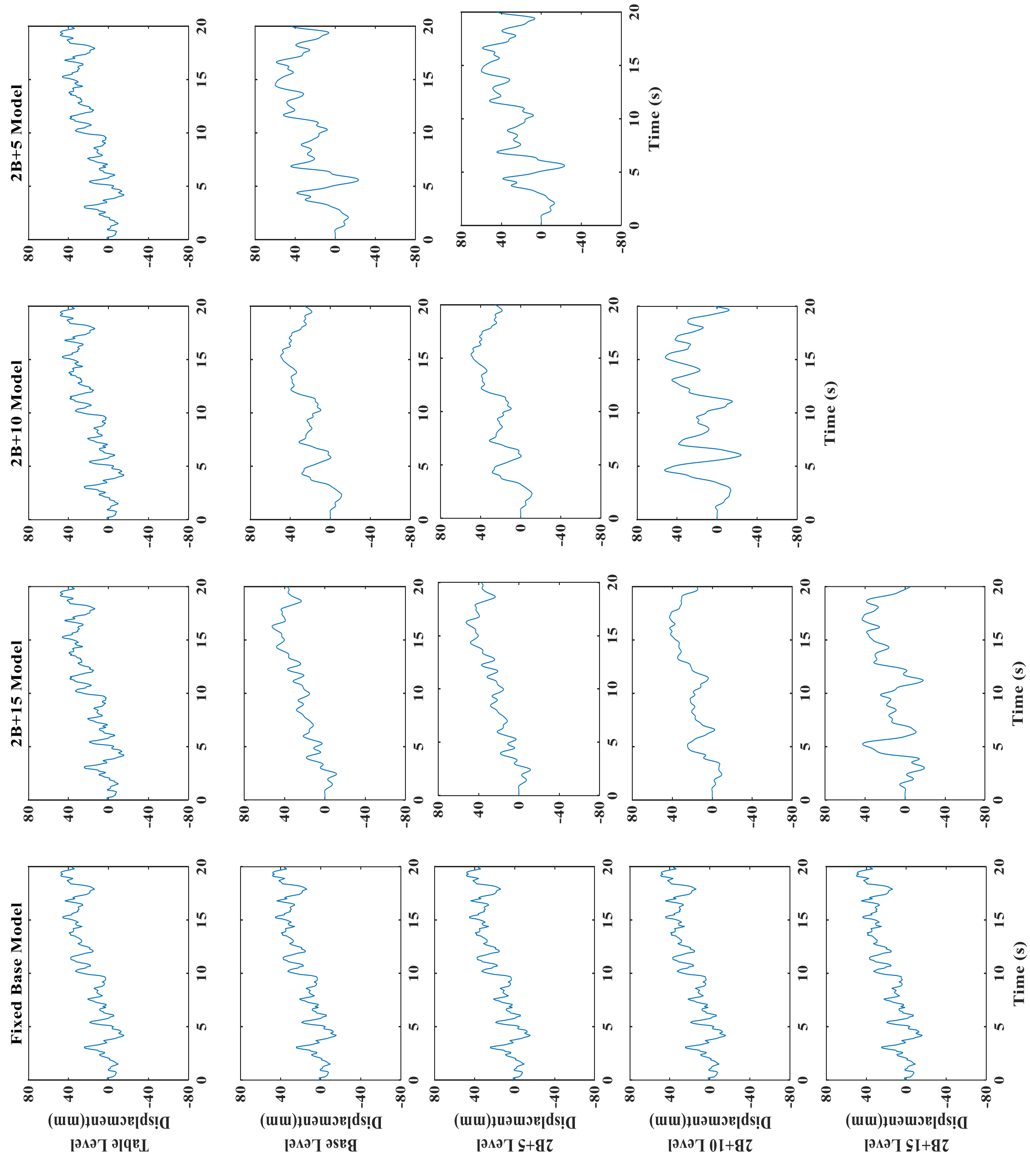

Figure Q Structure properties effect (pile foundation)

Appendix D

Factor Analysis and Regression outputs

```
FACTOR
  /VARIABLES p E vs ang
  /MISSING LISTWISE
  /ANALYSIS p E vs ang
  /PRINT INITIAL CORRELATION INV EXTRACTION FSCORE
  /CRITERIA MINEIGEN(1) ITERATE(25)
  /EXTRACTION PC
  /ROTATION NOROTATE
  /SAVE REG(ALL)
  /METHOD=CORRELATION.
```

Factor Analysis for Soil Factor

Notes

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	Split File	<none>
	N of Rows in Working Data File	4
Missing Value Handling	Definition of Missing	MISSING=EXCLUDE: User-defined missing values are treated as missing.
	Cases Used	LISTWISE: Statistics are based on cases with no missing values for any variable used.

Syntax		FACTOR /VARIABLES p E vs ang /MISSING LISTWISE /ANALYSIS p E vs ang /PRINT INITIAL CORRELATION INV EXTRACTION FSCORE /CRITERIA MINEIGEN(1) ITERATE(25) /EXTRACTION PC /ROTATION NOROTATE /SAVE REG(ALL) /METHOD=CORRELATION.
Resources	Processor Time	00:00:00.05
	Elapsed Time	00:00:00.05
	Maximum Memory Required	3264 (3.188K) bytes
Variables Created	FAC1_1	Component score 1

Correlation Matrix[a]

		p	E	vs	ang
Correlation	p	1.000	.853	.925	.997
	E	.853	1.000	.986	.892
	vs	.925	.986	1.000	.952
	ang	.997	.892	.952	1.000

a. This matrix is not positive definite.

Communalities

	Initial	Extraction
p	1.000	.937
E	1.000	.915
vs	1.000	.981
ang	1.000	.970

Extraction Method: Principal Component Analysis.

Total Variance Explained

Component	Initial Eigenvalues Total	Initial Eigenvalues % of Variance	Initial Eigenvalues Cumulative %	Extraction Sums of Squared Loadings Total	Extraction Sums of Squared Loadings % of Variance
1	3.803	95.064	95.064	3.803	95.064
2	.196	4.892	99.956		
3	.002	.044	100.000		
4	3.890E-16	9.725E-15	100.000		

Total Variance Explained

Component	Extraction Sums of Squared Loadings Cumulative %
1	95.064
2	
3	
4	

Extraction Method: Principal Component Analysis.

Component Matrix[a]

	Component 1
p	.968
E	.956
vs	.990
ang	.985

Extraction Method: Principal Component Analysis.[a]

a. 1 components extracted.

Component Score Coefficient Matrix

	Component
	1
p	.255
E	.251
vs	.260
ang	.259

Extraction Method: Principal Component Analysis.
Component Scores.

Component Score Covariance Matrix

Component	1
1	1.000

Extraction Method: Principal Component Analysis.
Component Scores.

p	E	angle	vs	Soil FAC
0	0	0	0	-1.40334
14	30	27	180	.05130
15	50	30	270	.43729
16	80	34	360	.91476

REGRESSION

```
/MISSING LISTWISE
/STATISTICS COEFF OUTS CI(95) R ANOVA CHANGE
/CRITERIA=PIN(.05) POUT(.10) CIN(95)
/NOORIGIN
/DEPENDENT FAC1_1
/METHOD=ENTER E P vs
/SAVE MCIN.
```

Regression for soil factor

Notes

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Missing Value Handling	Definition of Missing	User-defined missing values are treated as missing.
	Cases Used	Statistics are based on cases with no missing values for any variable used.

Syntax		REGRESSION /MISSING LISTWISE /STATISTICS COEFF OUTS CI(95) R ANOVA CHANGE /CRITERIA=PIN(.05) POUT(.10) CIN(95) /NOORIGIN /DEPENDENT FAC1_1 /METHOD=ENTER E P vs /SAVE MCIN.
Resources	Processor Time	00:00:00.02
	Elapsed Time	00:00:00.05
	Memory Required	3472 bytes
	Additional Memory Required for Residual Plots	0 bytes
Variables Created or Modified	LMCI_1	95% Mean Confidence Interval Lower Bound for FAC1_1
	UMCI_1	95% Mean Confidence Interval Upper Bound for FAC1_1

Warnings

For the final model with dependent variable REGR factor score 1 for analysis 1, influence statistics cannot be computed because the fit is perfect.

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	vs, P, E[b]	.	Enter

a. Dependent Variable: REGR factor score 1 for analysis 1

b. All requested variables entered.

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Change Statistics R Square Change	F Change
1	1.000[a]	1.000	.	.	1.000	.

Model Summary[b]

Model	Change Statistics df1	df2	Sig. F Change
1	3	0	.

a. Predictors: (Constant), vs, P, E

b. Dependent Variable: REGR factor score 1 for analysis 1

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3.000	3	1.000	.	.[b]
	Residual	.000	0	.		
	Total	3.000	3			

a. Dependent Variable: REGR factor score 1 for analysis 1

b. Predictors: (Constant), vs, P, E

Coefficients[a]

Model		Unstandardized Coefficients B	Std. Error	Standardized Coefficients Beta	t	Sig.
1	(Constant)	-1.367	.000		.	.
	E	.010	.000	.342	.	.
	P	.044	.000	.334	.	.
	vs	.002	.000	.351	.	.

Coefficients[a]

Model		95.0% Confidence Interval for B Lower Bound	Upper Bound
1	(Constant)	-1.367	-1.367
	E	.010	.010
	P	.044	.044

vs	.002	.002

a. Dependent Variable: REGR factor score 1 for analysis 1

Residuals Statistics[a]

	Minimum	Maximum	Mean	Std. Deviation	N
Predicted Value	-1.3668191	.9762069	.0000000	1.00000000	4
Std. Predicted Value	-1.367	.976	.000	1.000	4
Standard Error of Predicted Value	.000	.000	.000	.000	4
Adjusted Predicted Value	.	.	.	.	0
Residual	.00000000	.00000000	.00000000	.00000000	4
Std. Residual	.	.	.	.	0
Stud. Residual	.000	.000	.000	.000	3
Deleted Residual	.	.	.	.	0
Stud. Deleted Residual	.	.	.	.	0
Mahal. Distance	2.250	2.250	2.250	.000	4
Cook's Distance	.	.	.	.	0
Centered Leverage Value	.750	.750	.750	.000	4

a. Dependent Variable: REGR factor score 1 for analysis 1

```
NEW FILE.
DATASET NAME DataSet1 WINDOW=FRONT.
FACTOR
  /VARIABLES p E Angl vs
  /MISSING LISTWISE
  /ANALYSIS p E Angl vs
  /PRINT INITIAL CORRELATION EXTRACTION FSCORE
  /CRITERIA MINEIGEN(1) ITERATE(25)
  /EXTRACTION PC
  /ROTATION NOROTATE
  /SAVE REG(ALL)
  /METHOD=CORRELATION.
```

Factor Analysis for structure

Notes

Output Created		29-DEC-2018 22:12:19
Comments		
Input	Active Dataset	DataSet1
	Filter	<none>
	Weight	<none>
	Split File	<none>
	N of Rows in Working Data File	3
Missing Value Handling	Definition of Missing	MISSING=EXCLUDE: User-defined missing values are treated as missing.
	Cases Used	LISTWISE: Statistics are based on cases with no missing values for any variable used.
Syntax		FACTOR /VARIABLES p E Angl vs /MISSING LISTWISE /ANALYSIS p E Angl vs /PRINT INITIAL CORRELATION EXTRACTION FSCORE /CRITERIA MINEIGEN(1) ITERATE(25) /EXTRACTION PC /ROTATION NOROTATE /SAVE REG(ALL) /METHOD=CORRELATION.
Resources	Processor Time	00:00:00.06
	Elapsed Time	00:00:00.05
	Maximum Memory Required	3264 (3.188K) bytes

Variables Created	FAC1_1	Component score 1

[DataSet1]

Correlation Matrix[a]

		p	E	Angl	vs
Correlation	p	1.000	.993	.997	1.000
	E	.993	1.000	.999	.993
	Angl	.997	.999	1.000	.997
	vs	1.000	.993	.997	1.000

a. This matrix is not positive definite.

Communalities

	Initial	Extraction
p	1.000	.998
E	1.000	.996
Angl	1.000	.999
vs	1.000	.998

Extraction Method: Principal Component Analysis.

Total Variance Explained

	Initial Eigenvalues			Extraction Sums of Squared Loadings	
Component	Total	% of Variance	Cumulative %	Total	% of Variance
1	3.990	99.744	99.744	3.990	99.744
2	.010	.256	100.000		
3	9.657E-17	2.414E-15	100.000		
4	-4.617E-16	-1.154E-14	100.000		

Total Variance Explained

Component	Extraction Sums of Squared Loadings Cumulative %
1	99.744
2	
3	
4	

Extraction Method: Principal Component Analysis.

Component Matrix[a]

	Component 1
p	.999
E	.998
Angl	.999
vs	.999

Extraction Method: Principal Component Analysis.[a]

a. 1 components extracted.

Component Score Coefficient Matrix

	Component 1
p	.250
E	.250
Angl	.251
vs	.250

Extraction Method: Principal Component Analysis.
Component Scores.

Component Score Covariance Matrix

Component	1
1	1.000

Extraction Method: Principal Component Analysis.
Component Scores.

M	H	f	FactorW
0	0	0	-1.32
8.5	340	29	-0.13
14.62	760	16	0.43
23	1000	9.5	1.02

```
REGRESSION
  /MISSING LISTWISE
  /STATISTICS COEFF OUTS R ANOVA
  /CRITERIA=PIN(.05) POUT(.10)
  /NOORIGIN
  /DEPENDENT FAC1_1
  /METHOD=ENTER M H f.
```

Regression

Notes

Output Created		31-DEC-2018 15:41:28
Comments		
Input	Active Dataset	DataSet0
	Filter	<none>
	Weight	<none>
	Split File	<none>
	N of Rows in Working Data File	4
Missing Value Handling	Definition of Missing	User-defined missing values are treated as missing.
	Cases Used	Statistics are based on cases with no missing values for any variable used.
Syntax		REGRESSION /MISSING LISTWISE /STATISTICS COEFF OUTS R ANOVA /CRITERIA=PIN(.05) POUT(.10) /NOORIGIN /DEPENDENT FAC1_1 /METHOD=ENTER M H f.
Resources	Processor Time	00:00:00.03
	Elapsed Time	00:00:00.02
	Memory Required	3456 bytes
	Additional Memory Required for Residual Plots	0 bytes

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	f, H, M[b]	.	Enter

a. Dependent Variable: REGR factor score 1 for analysis 1

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	1.000[a]	1.000	.	.

a. Predictors: (Constant), f, H, M

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3.000	3	1.000	.	.[b]
	Residual	.000	0	.		
	Total	3.000	3			

a. Dependent Variable: REGR factor score 1 for analysis 1

b. Predictors: (Constant), f, H, M

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-1.324	.000		.	.
	M	.049	.000	.479	.	.
	H	.001	.000	.477	.	.
	f	.014	.000	.171	.	.

a. Dependent Variable: REGR factor score 1 for analysis 1

```
NEW FILE.
DATASET NAME DataSet1 WINDOW=FRONT.
REGRESSION
  /MISSING LISTWISE
  /STATISTICS COEFF OUTS R ANOVA
  /CRITERIA=PIN(.05) POUT(.10)
  /NOORIGIN
  /DEPENDENT SD
  /METHOD=ENTER W T a.
```

Regression Raft Foundation equation

Notes

Output Created		31-DEC-2018 15:45:25
Comments		
Input	Active Dataset	DataSet1
	Filter	<none>
	Weight	<none>
	Split File	<none>
	N of Rows in Working Data File	10
Missing Value Handling	Definition of Missing	User-defined missing values are treated as missing.
	Cases Used	Statistics are based on cases with no missing values for any variable used.

Syntax		REGRESSION /MISSING LISTWISE /STATISTICS COEFF OUTS R ANOVA /CRITERIA=PIN(.05) POUT(.10) /NOORIGIN /DEPENDENT SD /METHOD=ENTER W T a.
Resources	Processor Time	00:00:00.02
	Elapsed Time	00:00:00.03
	Memory Required	3456 bytes
	Additional Memory Required for Residual Plots	0 bytes

[DataSet1]

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	a, W, T[b]	.	Enter

a. Dependent Variable: SD

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.989[a]	.978	.967	3.04219

a. Predictors: (Constant), a, W, T

ANOVA[a]

Model	Sum of Squares	df	Mean Square	F	Sig.

1	Regression	2440.471	3	813.490	87.898	.000[b]
	Residual	55.529	6	9.255		
	Total	2496.000	9			

a. Dependent Variable: SD

b. Predictors: (Constant), a, W, T

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	6.165	6.842		.901	.402
	W	14.543	2.855	.341	5.094	.002
	T	-26.182	3.762	-.467	-6.959	.000
	a	223.301	21.081	.721	10.592	.000

a. Dependent Variable: SD

```
REGRESSION
  /MISSING LISTWISE
  /STATISTICS COEFF OUTS R ANOVA
  /CRITERIA=PIN(.05) POUT(.10)
  /NOORIGIN
  /DEPENDENT SD
  /METHOD=ENTER W T a pn pr pl.
```

Regression for pile equation

Notes

Output Created		31-DEC-2018 15:52:04
Comments		
Input	Active Dataset	DataSet2
	Filter	<none>
	Weight	<none>
	Split File	<none>
	N of Rows in Working Data File	16
Missing Value Handling	Definition of Missing	User-defined missing values are treated as missing.
	Cases Used	Statistics are based on cases with no missing values for any variable used.
Syntax		REGRESSION /MISSING LISTWISE /STATISTICS COEFF OUTS R ANOVA /CRITERIA=PIN(.05) POUT(.10) /NOORIGIN /DEPENDENT SD /METHOD=ENTER W T a pn pr pl.
Resources	Processor Time	00:00:00.00
	Elapsed Time	00:00:00.03
	Memory Required	5520 bytes
	Additional Memory Required for Residual Plots	0 bytes

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	pl, pr, W, T, a, pn[b]	.	Enter

a. Dependent Variable: SD

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.970[a]	.940	.900	2.65662

a. Predictors: (Constant), pl, pr, W, T, a, pn

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	1000.231	6	166.705	23.621	.000[b]
	Residual	63.519	9	7.058		
	Total	1063.750	15			

a. Dependent Variable: SD

b. Predictors: (Constant), pl, pr, W, T, a, pn

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-90.525	20.613		-4.392	.002
	W	14.845	2.276	.555	6.522	.000
	T	19.548	2.995	.556	6.526	.000
	a	118.530	16.380	.623	7.236	.000
	pn	1.426	.797	.571	1.789	.107
	pr	.826	1.489	.175	.555	.593
	pl	.120	.019	.520	6.388	.000

a. Dependent Variable: SD

Appendix E

MATLAB coding

- MATLAB CODE

```
% FT= 1 means the type of foundation is raft, FT=2 means the type of foundation is pile; so, there are two different solutions based on different equations. However, if any number rather than 1 and 2, there will not be any solution.
FT=input('Enter the value of FT: ');
if FT==1
fp = input('Enter the prototype frequency (HZ): ');
mp = input('Enter the prototype mass (kg): ');
Lp = input('Enter the prototype Length (m): ');
Wp = input('Enter the prototype width (m): ');
Hp = input('Enter the prototype height (m): ');
E = input('Enter the Modulus of Elasticity (pa): ');
Gama = input('Enter the soil density (kg/m^3): ');
Vs = input('Enter the soil shear velocity (m/s): ');
A = input('Enter the ground acceleration (m/s^2): ');
% FT is just number cannot effect the solution
fpp=fp*FT;
f = fpp * 7.7;
pp=mp/(Lp*Wp*Hp);
M= pp  *  Lp * Wp * Hp * 1/50;
W= -1.322+ (0.049*M) + (0.001*Hp*1/50) +0.014*f;
T= -1.411+ (0.009 *E) + (0.064 * Gama) +(0.002*Vs);
D= (6.622+ (14.531* W) - (26.949* T) +( 223.196*A))* 5;
AM=mp*D/2;
disp(['The Additional Moment is: ', num2str(AM), ' pa']);
else
if FT==2
fp = input('Enter the prototype frequency(HZ): ');
mp = input('Enter the prototype mass(kg): ');
Lp = input('Enter the prototype Length(m): ');
Wp = input('Enter the prototype width(m): ');
Hp = input('Enter the prototype height(m): ');
```

```
E = input('Enter the Modulus of Elasticity(pa): ');
Gama = input('Enter the soil density(kg/m^3): ');
Vs = input('Enter the soil shear velocity(m/s): ');
A = input('Enter the ground acceleration(m/s^2): ');
pn = input('Enter the pile number: ');
pl = input('Enter the pile length: ');
pr = input('Enter the pile diameter/span ratio: ');
fpp=fp*FT/2;
f = fpp * 7.7;
pp=mp/(Lp*Wp*Hp);
M= pp  *  Lp * Wp * Hp * 1/50;
W= -1.322+(0.049*M)+(0.001*Hp*1/50)+0.014*f;
T= -1.411+(0.009*E)+(0.064*Gama)+(0.002*Vs);
D=(-90.897+(20.129*W)+(14.852*T)+(118.586*A)+(1.427*pn)+(pl)+(0.827*pr))*5;
AM= mp*D/2;
disp(['The Additional Moment is: ', num2str(AM), ' pa']);
else
disp(['There is no solution for this type']);
end
end
```