ENHANCING SEISMIC RESILIENCE IN URBAN ENVIRONMENTS THROUGH THE VIBRATING BARRIER DEVICE

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Abstract

Earthquakes are a well-known natural hazard to urban environments. Recent disasters in Mexico, Ecuador, Italy and Japan manifest the clear need to address the seismic resilience of existing buildings in a different and more affordable way. Construction industry has successfully introduced devices such as isolators, dampers and tuned mass dampers to mitigate dynamic vibrations induced by earthquakes in new buildings, but such devices are rarely used for the protection of existing buildings, as they generally require substantial alteration of the original structure. In the case of heritage buildings, critical facilities and urban areas, especially in developing countries, those traditional localized solutions might become impractical. Therefore, what we are witnessing nowadays is the lack of substantial actions to protect existing cities in seismic prone areas with consequent number of fatalities and loss of historic and artistic heritage. In this regard a novel device called Vibrating Barrier (ViBa) has been recently proposed. Up to now the ViBa has been only developed to protect individual structures or a small cluster of buildings. This study focuses on the enhancement of the seismic resilience in urban environments by further developing the study of the ViBa device as a non-localised vibration control strategy. To this aim a novel procedure to identify simplified discrete models of clusters of buildings in the urban environment has been proposed. This research initially focuses on the soil-structure and structure-soil-structure interaction of realistic buildings through numerical and experimental tests. One of the open research questions in this framework is the definition of the proper soil interaction mechanical parameters. This study addresses the parameter identification of simplified discrete models developing a novel two stage time-domain identification procedure. The procedure is validated against numerical models and experimental tests. The time domain identification procedure has been further extended to the urban environment using finite element models of realistic cities available in literature. A simplified discrete model has been developed to represent a cluster of buildings within the urban environment able to account for site-city interaction effects. A novel procedure to determine the optimal design parameters of the ViBa device in urban environments following a stochastic approach has been proposed. The validation of the ViBa device as a non-localised vibration control strategy has been undertaken both in a full-scale numerical model and a scaled physical model of a realistic cities. The adoption of the ViBa has been shown to be beneficial by reducing the maximum average peak displacement of every single building analysed.
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Declaration

I declare that the research contained in this thesis, unless otherwise formally indicated within the text, is the original work of the author. The thesis has not been previously submitted to this or any other university for a degree, and does not incorporate any material already submitted for a degree.

Signature                      Date
To the grain of wheat that chose to die and give life to many
1 Introduction

In the development of society there has always been a pressing need for preserving the life of the individuals that form it. Despite all the advances in technology, earthquakes remain up to this day a major cause for the loss of lives. Moreover, the destructive effect of earthquakes is often intensified in densely populated urban areas, especially in developing countries. Even though construction industry has successfully introduced devices such as isolators, dampers and tuned mass dampers to mitigate dynamic vibrations induced by earthquakes in new buildings, substantial actions to protect the existing urban environment in a global manner have not yet been taken.

The present study focuses on the development of the novel ViBa device (Cacciola and Tombari 2015) as non-localised vibration control strategy able to address the seismic protection of an urban environment in a holistic fashion. The proposed design methodology of the ViBa in an urban environment involves the development of simplified models of clusters of buildings in the urban environment. To this aim, a novel time domain identification procedure is here developed and validated with FE models and experimental tests. Furthermore, a novel design procedure to define the optimum parameters of the ViBa following a stochastic approach is presented herein. The effectiveness of the ViBa, as a viable non-localised vibration control technique, is evaluated numerically through two case studies of realistic cities and experimentally by means of scaled physical models.
1.1 Research Aim and Objectives

The scope of this research is to develop a valid methodology for the design of the ViBa as a non-localised vibration control strategy aimed to protect groups of buildings in a holistic manner within an urban environment. For this purpose, a stochastic design approach, which ensures high reliability of the outcome is explored.

In light of recent developments and new insights regarding the site-city interaction (SCI) phenomenon, the proposed study is aimed to incorporate into the ViBa’s design methodology both site and SCI effects. In the context of vibration control of groups of buildings in urban environments, dynamically independent units need to be defined. This research intends to provide insight into the appropriate definition of clusters of buildings for design purposes.

Given the high computational expenses of analysing large domains, representing urban settlements, using “high order discrete models” (HODM), this study explores the development “low order discrete models” (LODM), able to capture with sufficient detail the dynamic behaviour of the selected clusters of buildings contained within the urban environment.

Finally, considering the limited availability of experimental evidence on SCI and holistic vibration control strategies in urban environments, this study intends to provide further experimental results by undertaking scaled SCI models with the implementation of the ViBa device as a non-localised vibration control technique.

In order to achieve the aims of the present investigation, the following objectives must be attained.

- To identify in literature the main aspects dominating soil-structure interaction (SSI), structure-soil-structure interaction (SSSI) and site-city interaction (SCI) phenomena.
- To develop a LODM able to capture with sufficient detail the dynamic behaviour of both SSI and SSSI systems including the impact caused by site effects and wave propagation.
- To design the ViBa and its respective interaction parameters with the neighbouring structures following a stochastic approach by the minimisation of the second order statistical moments of the target response.
• To assess the effectiveness of the ViBa as a non-localised vibration control strategy for clusters of buildings in the urban environment through both HODM of a full-scale city model and a scaled physical model when subject to earthquake ground motion.
1.2 Original Contribution

A novel procedure has been proposed to determine the optimal design parameters of the ViBa device in an urban environment under seismic stochastic excitation. The design is undertaken using the LODM of a cluster of buildings in an urban environment, the parameters governing the behaviour of the LODM are defined through bounded variable time-domain least square identification techniques. The design principle consists in the minimisation of the so-called penalty function formed of the second order statistical moments of the response at different locations in the model. Furthermore, the effectiveness of different configurations of the ViBa device are appropriately implemented into the urban environment and their performance assessed for the 2001 Nice earthquake.

Moreover, a novel LODM able to account for the dynamic cross interaction including time passing effect on the structural response has been developed. The LODM models identified through bounded variable time-domain least square identification techniques have been validated with both numerical and scaled physical models. One of the open research questions in the framework of identification techniques, is the definition of the proper soil interaction mechanical parameters. In this study they have been identified through a novel two-stage identification procedure, aimed to overcome the problem of disproportionate sensitivities.

In the framework of the study of SCI the amount of experimental data on the subject is very limited, moreover, experiments that address the non-localised reduction of vibrations in urban environments in a holistic manner are even scarcer. To the author’s knowledge experimental data on the non-localised vibration control of structures in an urban environment has not yet been generated. This research intends to provide insight into the performance of the ViBa as a non-localised vibration control solution in an urban environment through both a full scale numerical and a scaled physical model of a realistic city proposed by Semblat et al. (2008). The ViBa device was successfully implemented in the physical model as a non-localised vibration control strategy yielding response reductions in a group of buildings within the city.
The bounded variable time-domain least square identification procedure has been further extended to the urban environment using simplified numerical models available in literature. A LODM has been developed to represent a cluster of buildings within the urban environment able to account for site-city interaction effects.
1.3 Thesis outline

Chapter 2 provides a state of the art review on the subjects of SSSI, SCI and non-localised vibration control strategies. A brief description of SSI is given. The review on the study of both SSSI and SCI is divided into the three main approaches undertaken by the research community namely, analytical, numerical and experimental. An overview on some of the non-localised vibration control techniques available up to date is provided in the final section of this Chapter.

In Chapter 3 the bounded variable time-domain least square identification procedure for the determination of the LODM parameters describing a single structure coupled with the soil resembling an SSI scenario is first outlined. Different types of discrete models are studied. The bounded variable time-domain least square identification procedure is applied to the models and results are validated against finite element (FE) models.

Chapter 4 develops on the use of the bounded variable time-domain least square identification procedure for the determination of the LODM parameters describing SSSI scenarios and the novel two-stage identification procedure, aimed to overcome the problem of disproportionate sensitivities is presented. The discrete models used in Chapter 3 are used in this Chapter. A novel LODM able to account for the dynamic cross interaction including time passing effect on the structural response is presented. Finally, the bounded variable time-domain least square identification procedure is applied to the novel model for the three superstructure alternatives and results are validated against finite element (FE) models.

Chapter 5 focuses on the development of a methodology to both made and test small scale models exhibiting SSI and SSSI effects. The models are used as a validation and verification (V&V) tool for the proposed identification procedures presented in Chapters 3 and 4. As the models construction and testing campaign was part of the NUGENIA+ pilot project LOSSVAR and has benefitted from financial support by FP7 NUGENIA+, a brief description of the project and its main goal is given. Details on the construction methodology and testing approach are presented. Finally, dynamic force excitation tests on both, SSI and SSSI systems founded on a homogenous soil stratum are presented, and validation of the procedures developed in Chapters 3 and 4 is discussed.

In Chapter 6 the stochastic design procedure for the ViBa in an urban environment is outlined. The FE software is validated for its use in the assessment of SCI against
experimental and numerical data available in literature. This is followed by the numerical validation of the urban environment model used in this research as a full-scale model. In the context of non-localised vibration control of clusters of buildings in an urban environment a criteria for the identification of dynamically independent units is developed and discussed. Bounded variable time-domain least square identification procedure are suitably employed for the definition of LODM representing groups of buildings in the urban environment. Furthermore, a novel procedure for the design of the ViBa dynamic characteristics to protect a cluster of buildings under stochastic excitation in the urban environment is presented. The implementation of the designed devices in the full-scale FE model as well as its evaluation under earthquake ground motion is conducted. Finally, the use of the ViBa as a holistic technique for the control of vibrations in a urban environment is presented as a case study.

Chapter 7 focuses on the development and testing of the scaled physical model of the urban environment. The experimental design approach of the ViBa in the urban environment is explained. The model is further validated against results available in the literature, and numerical results illustrating the site-city effects are presented. Finally, the performance of the ViBa as a non-localised vibration control device is assessed experimentally in a scaled model of an urban environment and further extended into the full scale numerical model of the realistic city.

Chapter 8 provides the concluding remarks of the present research, and recommendations for future work are given.
2 Literature Review

In this Chapter, a very brief outline on Soil-Structure Interaction (SSI) is presented to lay the foundation for deeper insight into Structure-Soil-Structure Interaction (SSSI) and Site-City Interaction (SCI). This is followed by a revision of the current non-localised vibration control strategies, particularly the ViBa device. The latter has the potential of being implemented in an urban environment to mitigate the effects of earthquake induced ground motion, which indeed represents the main focus of the present study. For both SSSI and SCI the literature has been organised according to the approach adopted by the researchers namely, analytical and low order discrete models, numerical models and experimental models.

2.1 Soil-Structure Interaction (SSI)

It is widely known that the dynamic response of a structure under fixed base conditions differs from the response of a structure coupled with the soil in which it is founded, this is due to the inability of the buildings’ foundation to conform to the deformations in the soil caused by the ground motion, and the deformations induced in the soil by the dynamic response of the building (Kramer 1996). This synergy is called Soil-Structure Interaction (SSI) and has been extensively studied since the late 19th century progressing rapidly in the second half of the 20th century due to the expansion of the nuclear power industry and the appearance and fast development of computers and simulation tools (Lou et al. 2011). Generally, the main effects of SSI are a reduction in the natural frequency of the coupled soil-structure system compared to the fixed base scenario, an increase in the damping of the system, a reduction in the structures demands and an increase in the overall displacement of the system (Kramer 1996). It is important to mention, that the increase in the damping of the system is caused by its coupling with the soil and the ways in which this dissipates energy. The amplitude of stress waves traveling through real soils attenuate with distance. This amplitude reduction originates from two main sources, namely, material damping and geometric damping, often referred in literature as radiation damping. The former is linked to the conversion of the elastic energy into heat within the material that the waves are traveling through. Real soils dissipate energy every load cycle, by the slippage of grains with respect to each other. This process is characterised by a
The hysteresis loop and the energy dissipation process associated with this mechanism is called hysteretic damping (Kramer 1996). On the other hand, radiation damping is related to the reduction of specific energy, defined as the elastic energy per unit volume, as the stress waves spread through a greater volume. The amplitude reduction of stress waves in this case is not related to the transformation of the elastic energy into a different type of energy but instead to a purely geometric effect. A more detailed treatment of this topic is beyond the scope of this review. More information can be found in Kramer (1996).

The SSI has been studied using analytical, numerical and experimental approaches. A state-of-the-art review can be found in Kausel (2010) and Lou et al. (2011). However, it is important to highlight the impact of the experimental findings made by Jennings (1970) when submitting the Millikan Library of the Caltech institute to forced excitation by roof actuators. Data corresponding to the building's fundamental frequency were recorded at distances of up to a few kilometres. These results have set an important precedent in the more advanced study of site-city interaction (Bard et al. 2006). Even though rigorous analytical formulations and numerical studies have an invaluable contribution towards the knowledge of a given phenomenon, experimental tests are of paramount importance, especially in engineering, when studying a complex problem as SSI.

### 2.2 Structure–Soil–Structure Interaction (SSSI)

The so-called structure-soil-structure interaction (SSSI) or dynamic cross interaction phenomenon has arisen as a consequence of the study of SSI, and the premise is as follows: if a given structure can interact with the soil by radiating energy back into the soil, it is just intuitive to wonder if this radiated waves will have an impact on nearby structures; and, if so, what the consequences would be? According to Randolph and House (2001) the trend in research methods in structural engineering is polarising towards computational modelling given its increasing power and decreasing cost. Nevertheless, when it comes to such a complex phenomenon like SSSI, the need for a synergistic interaction between robust analytical formulations, numerical models and physical models is crucial. The following section reports the contributions of different researchers according to the approach they followed to seek insight into the SSSI phenomenon.
2.2.1 Analytical and low order models approach

Following the developments made in Soil-Structure Interaction (SSI), namely, the analytical formulation of a single mass perfectly attached to an elastic half space (Luco and Westmann 1971; Veletsos and Wei 1971), Warburton et al. (1971) developed an analytical solution for the response of two cylindrical masses with circular base attached to the surface of an elastic half space when subject to periodic forces and moments. The main point of interest of this study was the assessment of the effect that the second mass causes on the excited mass and the conditions under which significant amplifications occur on the second mass. For simplicity no internal damping of the materials was considered, just radiation damping was taken into account since, as stated by the authors, the only expected effect, caused by the inclusion of the internal damping, is a reduction in the maxima of the response curves. The general theory for the two masses system was developed, and numerical results considering two geometrically identical cylindrical masses, for the case in which one of the masses is excited by a harmonic vertical force along its line of symmetry, were presented. The first effect noted is that, when a single mass is excited by a harmonic vertical force, the response would be mainly in the vertical axis; however, when a second mass is present, the excited mass possess three motion components, namely, vertical displacement, horizontal displacement and rocking. Nonetheless, the vertical displacement component is not greatly affected and continuous to display a significant amplitude. Additionally, the displacement amplitude of the second mass is dependent on the frequency of excitation of the first mass, that is to say, the maximum response of the second mass corresponds to frequencies of excitation that match the respective modal frequencies (vertical, horizontal and rocking mode) of the second mass. Furthermore, the response amplitude of the second mass is inversely proportional to the distance between masses. Finally, as a general trend, the rocking component of the second mass was found to be important and greater than the horizontal component.

At the early stages of the study of the dynamic cross interaction phenomenon between adjacent buildings, also called SSSI, Luco and Contesse (1973) addressed this problem by means of a simple model consisting of two parallel infinite shear walls, fixed to a rigid foundation of semi-circular cross section. They considered a perfect bond between the foundation and the soil, represented by an elastic, homogenous and isotropic half space. It is important to highlight that in order to address the SSSI problem, it is necessary to
compute simultaneously the soil response under both the seismic excitation and the stress waves generated by the foundations. In their concluding remarks, they commented on the significance of the SSSI effect for the frequency range commonly used in earthquake engineering practice. Moreover, they pointed out that the interaction effects are important, particularly for the case of a small shear wall located in the vicinities of a large structure, since for this case the base motion and shear forces will be significantly different if the small structure is analysed in isolation.

With the aim of gaining insight in the effect of SSSI on the response of a multi-structure system on a visco-elastic media, Kobori et al. (1973) developed a formulation that allows the evaluation of such systems in terms of dimensionless transfer functions. For the numerical applications, different structures-soil arrangements were considered, namely, a system with one, two and seven masses on the soil media (see Figure 2.1) and a two masses-spring system (see Figure 2.2) considering one and two buildings. It is worth mentioning that all the buildings (mass and two mass-spring) models had a square foundation. The cases of an individual isolated mass and mass-spring systems were studied for the sake of comparison. Two types of excitations were employed, namely, force excitation acting on one of the masses or basement mass (for the two masses-spring model), and uniform displacement applied on the free surface of the soil stratum. As argued by the authors, the horizontal component of an earthquake is regarded as the most significant measure for seismic design, hence just the horizontal component of both the excitation and the response were considered in their analyses.

![Figure 2.1](image)

**Figure 2.1** a) Soil-mass system with two masses $m_1$ and $m_2$ where $h$ is the stratum depth and $x$ is the horizontal distance between masses, and b) seven masses $m_1$ to $m_7$ on the soil media system (Kobori et al. 1973).
Kobori et al. (1973) concluded that the effect of SSSI in the response of the system depends greatly on whether the fundamental frequency of the first structure coupled with the soil is lower or higher than the soils’ fundamental frequency. For the case in which the fundamental frequency of the structure coupled with the soil is lower than the soils’ fundamental frequency, the coupled structure’s fundamental frequency becomes the natural frequency of the whole coupled system, and the effects of SSSI will appear in the vicinities of said frequency. On the other hand, if the fundamental frequency of the structure coupled with the soil is higher than the soils’ fundamental frequency, then the whole coupled system frequency will be close to the soils’ fundamental frequency. Furthermore, differences in shape of the response curves, namely, sharp and high for the lower frequency range case and broad and flat for the higher frequency range case, were attributed to the variation of the damping with the frequency (viscous damping). According to Kramer (1996), real soils dissipate elastic energy hysteretically, hence the dissipation characteristics are independent of frequency. This seems to be a limitation of the results provided by Kobori et al. (1973). Nonetheless, given the convenience of the viscoelastic formulation is advised to use an equivalent viscosity to guarantee the frequency independence of the damping.

In the case of two identical structures being studied, the effect of SSSI appear to be dependent on the type of excitation and relative location of the fundamental frequency of the structure coupled with the soil with respect to the soils’ natural frequency. If the fundamental frequency of the structure coupled with the soil is lower than the
fundamental frequency of the soil and one of the structures is under force excitation, the effect is a reduction on the displacement amplitude of the excited structure. On the other hand, if the excitation is a uniform displacement on the surface of the stratum, the SSSI effects are small. If the fundamental frequency of the structure coupled with the soil is greater than the soils’ fundamental frequency, the SSSI effects occur in the frequency range between the soils’ fundamental frequency and the fundamental frequency of the structure coupled with the soil. For this case the response of the coupled system does not seem to depend on the type of excitation.

When analysing SSI and SSSI problems one of the main disadvantages are the cumbersome nature of the analytical formulations and the high computational expenses of both Finite Element Method (FEM) and Boundary Element Method (BEM) models. Mulliken and Karabalis (1998) proposed a discrete model for the dynamic analysis of three-dimensional foundation-soil-foundation interaction problem based on results obtained during more precise numerical analyses. Their model was based on frequency independent coupling functions consisting of discrete springs and dashpots to compute the response of surface foundations on an elastic half-space. Said model takes into account the wave passage effect by modifying the Wilson-θ time domain integration method. Moreover, the fact that the equation of motion is directly integrated in the time domain allows the possibility to consider non-linear soil and structures by modifications of the model parameters. Their formulation was developed by first considering the independent SSI case of a single foundation and then coupling the different neighbouring foundations using frequency independent coupling functions resulting in springs and dashpots, Figure 2.3 shows both the single discrete foundation model and the coupled system.
Figure 2.3 Single discrete model (a), where $M$, $K$ and $C$ represent the mass, stiffness and damping of the model, and coupled system (b), where $M_i$, $K_i$, $C_i$ with $i=1; 2; 3$, represent the mass, stiffness and damping of each structure and $K_{j,k}$ and $C_{j,k}$ with $j=1; 2$ and $k=2; 3$, are the cross coupling stiffness and damping terms, after Mulliken and Karabalis (1998).

It is worth highlighting that one of the main assumptions made by Mulliken and Karabalis (1998) was that the coupling between structures is independent of other degrees of freedom, i.e. vertical motion of one structure will affect only the vertical component of the surrounding foundations, horizontal motion produces only horizontal motion, etc. This is a limitation of the model since, as demonstrated by Kobori et al. (1973) and stated by Niwa et al. (1988), the vertical motion of one of the structures (in a two structures setup) induces both horizontal and rocking motion in the adjacent structure, with the latter component being predominant. Furthermore, in the model of Mulliken and Karabalis (1998), the SSI stiffness of each individual foundation is not influenced by the presence of adjacent foundations. Lastly, the coupling forces occur at a given time after the displacement of the adjoining structures (time lag effect). The model was implemented and compared with the studies of Huang (1993) and Novak (1994) for the case of two and three foundations systems, the latter having been performed with the specialized FE software SASSI. As can be seen from Figure 2.4 the agreement between the discrete model and the FEM model is shown.
Finally, the model formulated by Mulliken and Karabalis (1998) was modified to accommodate a single degree of freedom system bonded to the superficial foundations system, finding that the structural response and resonance peaks are significantly altered by the presence of other structures. One of the most important remarks of their study was that a well calibrated simplified discrete model can provide satisfactory and accurate results for the evaluation of SSI and SSSI problems when using linear elastic materials with the possibility (by means of simple modifications of the models) of considering non-linear materials.

Following a similar approach, Naserkhaki and Pourmohammad (2012) studied the SSI and SSSI phenomena by defining a discrete model of both, a single building coupled with the soil and twin buildings coupled with both the soil and with each other. For the case of SSI they found that the softer the soil layer, the longer the period of the coupled system, and as the soil gets stiffer the period of the structure converges to the fixed base period, which is in agreement with Kramer (1996). Moreover, the effect of SSSI was noticed as a reduction of the building response (top motion) of approximately 10% with respect to the coupled single building case (SSI), for separation distances up to three meters. Beyond this separation, the response of their arrangement converges to that of the single coupled building. Additionally, they studied the influence of the buildings mass on the magnitude of the interaction of adjacent buildings, concluding that heavier buildings influence the surrounding lighter buildings, while not being significantly affected by adjacent lighter
buildings. Finally, they stated that SSSI can be beneficial and should be considered in the design of structures that are close together.

More recently, Alexander et al. (2012) proposed an analytical formulation based on discrete models for soil and buildings to study the adjacent building effect (SSSI). Figure 2.5 display the model arrangement and parameters. They considered the coupling through the soil via a rotational spring. The stiffness of the coupling spring was found by equating energies between the discrete model and a finite element (FE) model. Their final formulation depended on four parameters, namely, aspect ratio of building one \((s = h_1/b, \text{ where } h_1 \text{ is the height of building 1 and } b \text{ is the foundation width})\), height ratio \((\varepsilon = h_2/h_1, \text{ where } h_2 \text{ is the height of building 2})\), normalised inter-building distance ratio \((z = \text{distance between buildings to building width, } b)\) and soil type (loose, medium, dense sand). Analyses under ground motion excitation were performed in the frequency domain, using the Kanai-Tajimi spectra, and evaluated in terms of power spectral densities (PSD). Figure 2.6 depicts the PSD of the response for both, building one and two for a given set of dimensional parameters \((s = 2, \varepsilon = 1.1, z = 0.1)\), founded on loose sand. As stated by Alexander et al. (2012), it can be seen from Figure 2.7 that the total response power increases for building one, while for building two the response power decreases. In other words, constructing a slightly taller building next to an existing building can be detrimental for the old structure. Moreover, results of parametric analysis are summarised in the contour plot presented in Figure 2.7 for the case of \(z = 0.1\) and loose sand soil. It can be observed, that there is always an amplified response (around 50\%) for height ratios ranging from 1.05 to 1.2 for different aspect ratios, while the amplification becomes greater when building one is short and fat (low aspect ratio). Finally, they showed that despite its simplicity, the discrete model was able to capture the main features of the SSSI phenomenon, when compared with FE solutions.
Figure 2.5 Two buildings discrete model with parameters after Alexander et al. (2012).

Figure 2.6 Power spectral density function of the response for building one (top) and building two (bottom) for a given set of parameters after Alexander et al. (2012).
2.2.2 Numerical Approach

Given the versatility and capability of numerical models to reproduce more complex scenarios and geometries than those described in the analytical formulations, computer models created using FEM, BEM and coupled FEM-BEM models are a popular technique in the study of both SSI and SSSI. Qian and Beskos (1995) described an enhanced approach to evaluate the interaction between adjacent three-dimensional massless rigid foundations of arbitrary shape perfectly bonded to an elastic half-space under harmonic forces. The study consisted of a combination of the half-space Green’s function formulation and frequency domain BEM using quadrilateral boundary elements. One of their main conclusions was that SSSI influences different components of the structures’ response depending on the number of structures involved, distance between them and frequency of the excitation. For the vertical and horizontal components SSSI effects increase with the number of structures, short distances and low frequencies while for the torsional and rocking components, the effect is clear for short distances and high frequencies, but does not seem to be highly affected by the number of structures.

Another reason why numerical approaches are constantly undertaken is that in the endeavour to attain greater understanding of the SSSI phenomenon, performing field and

Figure 2.7 Change in power vs. aspect ratio and height ratio, z=0.1 and loose sand soil after Alexander et al. (2012).
laboratory tests becomes expensive and time consuming. One example of this is the study of Behnamfar and Sugimura (2000) in which the experimental twin towers of the Tohoku University in Sendai, Japan were modelled numerically using 2D boundary elements. They analysed the buildings under vertically propagating SV-waves and horizontally propagating Rayleigh waves. Moreover, spatially variable ground motion was also taken into account. The input motion of their study was given by the free field ground motion recorded by the monitoring array of accelerometers of the experimental twin buildings. Two events were modelled and, according to their conclusions, there was good agreement between the 2D boundary element model results and the recorded data. Figure 2.8 display a plot comparing the response (in the frequency domain), at the top of one of the twin buildings, between the recorded data (thick line) and the numerical results of the simulation (thin lines), under vertically propagating SV-waves with and without spatial variability. It is evident from Figure 2.8 that the agreement seems to be very poor after a frequency of 3 Hz, further, the effect of spatial variability does not appear to be important. According to Behnamfar and Sugimura (2000) the usage of rough soil and structures models can provide satisfactory results from an engineering point of view. However, in Figure 2.8, is evident that, for frequencies beyond 3 Hz there are significant discrepancies between the recorded data and the simulation, suggesting that the previous statement does not apply for the full range of considered frequencies.

![Figure 2.8 Comparison of the measured (thick line) response of the twin buildings and the numerical results (thin lines) with and without spatial variability, after Behnamfar and Sugimura (2000).](image)

More recently, by means of a coupled Boundary element method-Finite element method (BEM-FEM) model, Padrón et al. (2009) studied the SSSI phenomenon of superstructures founded in pile groups. Figure 2.9 display the geometry of the problem, including notation and the considered degrees of freedom, as well as the different analysed arrangements.
All the buildings were modelled as shear-type frames founded on 3 x 3 pile arrays. Both steady state and earthquake excitation analysis were undertaken, considering vertically propagating S waves and Rayleigh waves. The resulting transfer functions were displayed in terms of the dimensionless frequency $\omega_d / v_s$, where $\omega$ is the excitation frequency, $d$ is the piles diameter and $v_s$ is the shear wave velocity of the stratum. Among their main contributions stands out the fact that the SSSI effects can be significant (beneficial or detrimental) when structures with similar dynamic characteristics are adjacent to each other (Figure 2.10), whereas, for the case of different structures the effect does not seem important (Kobori et al. 1973) (Figure 2.11). However, for the case of a short-period building placed in between two long-period buildings, the response of both types of buildings increases 20 and 30 percent respectively. This behaviour is in agreement with the results reported by Alexander et al. (2012). Moreover, whether the effect of adjacent structures is detrimental or beneficial seems to be related with the separation distance between structures. Figure 2.10 shows the results of the steady state analysis using vertically propagating S waves, for the case of three consecutive identical buildings, considering different separation distances. As can be seen the SSSI can be detrimental or beneficial depending on the buildings separation distance; additionally, the building in the centre of the arrangement seems to be more affected in comparison to those of the extremes. Is worth to mention that a particular feature can be appreciated in the plots of Figure 2.10, namely, that there is a given distance for which there is a considerable reduction and more important a redistribution of the energy in frequency. These results are very similar to those reported by Clouteau et al. (2012) (Figure 2.14).

Figure 2.9 Analysed geometry and arrangements after Padrón et al. (2009).
Figure 2.10 Transfer functions for three identical buildings for different separation distances after Padrón et al. (2009). $\lambda$ is the soils wave length at the systems fundamental frequency.

Figure 2.11 Transfer functions for three distinct buildings for different separation distances after Padrón et al. (2009). $\lambda$ is the soils wave length at the systems fundamental frequency.

2.2.3 Experimental Approach

Besides the robustness of analytical formulations and the versatility of numerical methods, the need of validation with reality is imperative. Hence, full scale and laboratory experiments have been undertaken by many research groups, mainly (but not only) encouraged by the nuclear energy sector. As part of the early attempts to study the SSI and SSSI effects experimentally, Niwa et al. (1988) performed small scale models made of silicon rubber (to simulate the soil material) and steel cylinders (to simulate the buildings). Their methodology was based on the comparison of the transfer functions of different experimental arrangements (see Figure 2.12) designed to assess the effect of the
structures’ weight, structures’ size, existence of embedment, distance between adjacent structures and motion direction (with respect to the buildings arrangement). The different setups were tested using both shaking table and vibration generator, for forced vibration tests. Furthermore, finite element models were created for validations purposes. Among their main findings are the impact of the embedment on the coupled frequency of the single building case, namely, the stiffness of the soil-structure system increases with embedment. Moreover, as stated by the authors, the response of a structure that is adjacent to another is different from the single structure case. Additionally, as the distance between adjacent structures decreases, the SSSI effects are clearly visible. Finally, they argued that when two structures are arranged at right angle with the excitation directions the response of the buildings is very similar to the single structure case; however, for the opposite case in which the structures are parallel to the excitation, each response is smaller than the single structure case.

<table>
<thead>
<tr>
<th>Weight of Structure</th>
<th>Light</th>
<th>Heavy</th>
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</thead>
<tbody>
<tr>
<td>Size of Structure</td>
<td>Small</td>
<td>Large</td>
</tr>
<tr>
<td>Existence of Embedment</td>
<td>Without Embedment</td>
<td>Embedded</td>
</tr>
<tr>
<td>Distance between the Structures</td>
<td>Short Distance</td>
<td>Long Distance</td>
</tr>
<tr>
<td>Exciting Direction to Adjacent Structure</td>
<td>Parallel</td>
<td>Right Angles</td>
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Figure 2.12 Different parameters considered in the analyses of Niwa et al. (1988).

In 1994 the Nuclear Power Engineering Corporation (NUPEC) performed a comprehensive study on SSI and SSSI. The study was particularly focused on the influence that adjacent buildings of Nuclear Power Plants (NPPs) have on each other, when subject to dynamic excitations. The investigation aimed to provide insight into the SSI and SSSI phenomena by means of field and laboratory tests divided in two stages,
namely, experimental setup tested with and without embedded foundations. For the case of the field test, the experimental arrangement consisted of three contiguous testing locations (see Figure 2.13), formed of a single building model, two identical buildings model and two different types of building. The models were made of reinforced concrete with a scale of 1:10 with respect to an existing boiling water reactor (BWR), the embedment of the foundations was 5 m deep. Both experiments’ stages involved forced excitation tests and earthquake observation. Figure 2.14 depicts the results of forced vibration tests for the case of the single building and the two identical ones with and without embedment. Two important effects can be noticed from the plots of Figure 2.14: firstly, it seems that the presence of the second structure results in a reduction of the amplitude for both cases (with and without embedment); secondly, it looks like, for the case of superficial foundations, there is a redistribution of the energy in frequency, leading to an amplitude reduction of the response. Despite this fact, Kitada and Iguchi (2004) stated that these effects are negligible and it is not necessary to take them into account in the seismic design of NPPs. These conclusions are supported by the earthquake observation results (in terms of response spectra) during the second stage of the field test (considering embedment) presented by Kitada and Iguchi (2004) and displayed in Figure 2.15. Said figure presents the seismic response in the frequency domain for a single building and the case of two identical buildings. Clouteau et al. (2012) argues that higher interaction is observed in the case of adjacent buildings with embedded foundations rather than for the case of superficial foundations. They reported reductions in the order of 30% (with respect to the single building case) of the response at the top of the building for the case of embedded foundations under forced vibration. Moreover, physical models tested by Aldaikh et al. (2016) showed amplifications in spectral power in the order of 58% for two adjacent structures with respect to the single structure case.
Figure 2.13 NUPEC field test setup (embedded case) after Kitada and Iguchi (2004).

Figure 2.14 NUPEC field test results of forced vibration tests for both the single building model and the two identical buildings after Clouteau et al. (2012).

Generally, due to the cost and time involved in field tests, parametric studies are rarely undertaken, hence, other strategies tend to be adopted. In the case of NUPEC, laboratory tests of small-scale models (1:260) were carried out using a shaking table and vibration generators for forced vibrations tests. The soil stratum was modelled using silicon rubber while the building models were made of aluminium. Figure 2.16 presents the experimental setup as well as the parametric analyses arrangements. The addressed
parameters are embedment depth, adjacent building distance, building type and arrangement.

Figure 2.15 NUPEC field test results of earthquake observation for both the single building model and the two identical buildings considering embedded foundation in the X (left) and Y (right) directions after Kitada and Iguchi (2004).

Figure 2.16 NUPEC laboratory tests setups and analyses after Yano et al. (2000).

For the case of building models without embedment, there seems to be no significant effect caused by the presence of adjacent buildings, whereas the effect can be important when embedment is taken into account. The experiments that considered embedment of the models foundation were divided in two subcases, namely, one in which the gap between buildings was left empty and another one in which the gap was filled. Figure 2.17 display the frequency response function for the case of embedded foundations, according to Kitada et al. (2001) for the case of empty gap there is reduction just in the Y direction (direction of buildings in series) while in the X direction the adjacent effect appears as a shift in frequency and a minor increment in amplitude. However, when three
buildings are present, since there is a building in both X and Y direction there is a reduction in the response with respect to the single building scenario.

![Diagram of building configurations](image)

Figure 2.17 NUPEC laboratory tests results for the case of models with embedment after Kitada et al. (2001).

Finally, despite the lack of agreement between researchers regarding the significance of the adjacent building effect or SSSI, the NUPEC study have provided, to the author’s knowledge, the first set of field tests specially targeted to study the SSSI phenomenon. These tests have given important insight and contributed with results for the verification and validation (V&V) of both laboratory tests and numerical models.

Following a different approach, Trombetta et al. (2012), using a geotechnical centrifuge, analysed SSSI effects on inelastic building models. They considered two types of buildings, namely, one large and heavy with two predominant modes and another lighter and smaller with one predominant mode, Figure 2.18 shows the building models as well as the different arrangements tested. The experiment was carried out by subjecting the models to scaled earthquake (in time) ground motion. A total of 23 records were used.

Their evaluation was performed in terms of intensity measure (i.e. peak ground acceleration (PGA), peak ground velocity (PGV), etc.) and demand parameters (i.e. peak roof acceleration, peak roof drift, etc.) relationships. Unfortunately, their results did not lead to a consistent link of intensity and demand parameters. For instance, the effect of in plane SSSI over the peak roof acceleration showed a detrimental trend in comparison with the single building scenario while the combined (in-plane and anti-plane) SSSI case was characterised by a reduction of the response with respect to the single building case. Given these results, one should expect a similar behaviour for the peak roof displacement, however, the findings were that, for both cases (in-plane and combined in-plane and anti-
plane arrangements) the effect was detrimental (greater displacements than those found for the single building scenario). Nevertheless, it is important to highlight, that these tests were limited to a certain number of records as well as ground conditions, hence, the only generalised conclusion that can be drawn, is that SSSI effects can significantly alter the response of structures in an urban environment.

![Building models and arrangements studied after Trombetta et al. (2012).](image)

More recently, Aldaikh et al. (2016) proposed an experimental setup aimed mainly to study the influence of structural height on the response magnitude, given a SSSI scenario. Additionally, they intended to validate the analytical formulation proposed by Alexander et al. (2012) considering an analogous arrangement. The experiment was a plain strain physical model of SSSI between two and three adjacent buildings (both cases studied). In their system, the structure models were placed on a cellular polyurethane foam base to simulate a linear elastic soil substrate and tested on a shaking table under both random white noise and scaled uniaxial ground motion. Figure 2.19 depicts the different analysed arrangements as well as the full model on the shaking table. According to Alexander et al. (2012) that the height ratio, defined as the ratio between the height of the adjacent building to the control building (SSI case), was one of the most important parameters affecting the SSSI phenomenon, this was pointed out early in the 1970’s by Luco and Contesse (1973) after their analytical studies of adjacent shear walls. This hypothesis was confirmed by the results of both the random signal and ground motion tests. Amplifications in spectral power of 16% and 21% for the two and three buildings, respectively, were reported for the white noise tests considering a height ratio of 1.1 (adjacent buildings 10% higher than the SSI single building case). Moreover, five ground motion acceleration records were scaled and applied to the model, Figure 2.20 presents the change in spectral power with the different height ratios considered for both two and three building models. Amplifications of up to 58% and 40% at height ratio of 1.1 for
both two and three buildings models respectively and reductions of up to 18% and 31% for height ratio of 0.75 for both two and three buildings models respectively, were reached. Furthermore, the authors suggest that the SSSI phenomenon is characterised by an ‘s’ shape curve (with respect to the change in spectral power vs. height ratio plot) in which the initial branch correspond to a beneficial SSSI effect while the second branch to a detrimental one. Similar trends can be seen in Alexander et al. (2012).

Figure 2.19 Experimental arrangement, (a) single building, (b) two identical buildings, (c) three identical buildings and (d) full model on shaking table, after Aldaikh et al. (2016).
2.2.4 Summary

The following key outcomes can be outlined from the SSSI literature review:

- The interaction effects are important, particularly for the case of a small shear wall located in the vicinities of a large structure; for this case the base motion and shear forces will be significantly different if the small structure is analysed in isolation (Luco and Contesse 1973).
- The effect of SSSI in the response of the system depends greatly on whether the fundamental frequency of the first structure coupled with the soil is lower or higher than the soils’ fundamental frequency.
- According to Padrón et al. (2009), whether the effect of adjacent structures is detrimental or beneficial seems to be related with the separation distance between structures; this is in agreement also with Naserkhaki and Pourmohammad (2012).
- Aldaikh et al. (2016) suggest that the SSSI phenomenon is characterised for an ‘s’ shape curve with respect to the change in spectral power vs. height ratio plot, in which the initial branch correspond to a beneficial SSSI effect while the second branch to a detrimental one.

It can be concluded that whether detrimental or beneficial, adjacent structures have an effect on the dynamic response of each other. This effect seems to be inversely proportional to the inter-structure distance and depends greatly on whether the
fundamental frequency of the first structure coupled with the soil, is lower or higher than the soils’ fundamental frequency. Furthermore, the presence of an adjacent structure can lead to a redistribution of the energy in the SSSI system.

2.3 Site City Interaction

Following a similar rationale as that established for the study of SSSI, namely, that if a structure can back radiate waves into the soil as consequence of the soil-structure coupling, then such waves could have an impact on neighbouring structures. It is intuitive to wonder if a group of buildings might have a conjunct effect on both the site response and the individual neighbouring structures. In the same way that SSSI phenomenon inherit its analysis methods and approaches from the study of SSI, this is also the case with SCI, with the peculiarity that full-scale experimental studies will be extremely complex and challenging, and even simplified experimental setups are scarce.

2.3.1 Analytical and low order models approach

Following the seismic events in the Mexico City, namely the 1985 and 1995 earthquakes, a significant amount of research has been undertaken to explain the anomalies in the records (great amplifications and long durations). Among these studies, the analytical model proposed by Gueguen et al. (2002) stands out. This study focused on the effect of buildings in Mexico City-like environments. Their model was formed of a realistic arrangement of buildings as distributed in the Roma Norte area of the city. Buildings were modelled as three degree of freedom systems (DOFS), namely, two translational degrees of freedom, at the foundation and top level, and one rotational degree of freedom for the foundation. The buildings were founded on a 40 m thick clay deposit with shear wave velocity ($V_s$) of 65 m/s underlaid by a stiffer sediment stratum of $V_s = 600$ m/s. The reference point of the model was the accelerograph site Co56 of the Mexico City accelerometric network, located in the vicinities of the Roma Norte area. The model was subject to the input motion recorded during the 14 September 1995 earthquake. The wavefield radiated by each building was computed individually and superimposed to generate the total perturbation created by the urban environment, this implies that cross interaction effects or SSSI were not considered. According to the results of Gueguen et al. (2002), the radiated wavefield from the buildings has amplitudes comparable to those of the free field motion. It is important to highlight, that these results were obtained
despite the fact that the frequency of the buildings was larger than the frequency of the soil stratum, that is to say that site-city resonance was not significant. Moreover, energy calculations showed that the intensity of the radiated wavefield by the Roma Norte district was equivalent to 70% of the intensity of the input. Additionally, parametric analyses were undertaken by Gueguen et al. (2002) to evaluate the effects of the stiffness ratio between the buildings and the soil, the urbanization density and distance up to which the site city interaction (SCI) is relevant. Figure 2.21 display the intensity variation of the radiated wavefield with both the stiffness ratio and urbanisation density. The influence of the site city resonance is evident; when both soil and structures have a similar resonance frequency (stiffness ratio equal to one) the radiated wavefield intensity is greater than for the rest of the cases; furthermore, the greater the urbanization density the greater the radiated wavefield, this effect is a direct consequence of the superposition of the buildings individual responses, the higher the number of buildings considered the higher the intensity. It might be the case, however, that this trend would change if SSSIIs are considered. Finally, according to their model there seems not to be a critical distance up to which the radiated wavefield is not important. Despite the limitations of the model presented by Gueguen et al. (2002), namely, the linear elastic model, neglecting the SSSI, not considering the 2D and 3D basin effects and the simplified model of buildings, their study depicts the importance of considering the influence of structures in urban environments, given the fact that the wavefield radiated by the city can be comparable to the input.

![Figure 2.21](image.png)

Figure 2.21 Intensity variation of the radiated wave field with the stiffness ratio (Y-axis) and urbanization density (X-axis) after Gueguen et al. (2002).

More recently Ghergu and Ionescu (2009) studied the SSSI and the so called city effect of a coupled system made of an elastic halfspace representing the stratum and 2 DOF
systems (foundation and top mass) representing the structures. The equation of motion of the coupled system is a partial differential equation (PDE). They had two main goals, namely, to corroborate the dynamic cross interaction between buildings and to show the existence of the city frequencies (coupled resonant frequencies of the system). Among their conclusions, they pointed out that the influence of the perturbations generated by each building only affect the nearby surroundings of the building. Moreover, they highlighted the collective behaviour of the buildings and the existence of a city (coupled system) frequency that does not depends on the number of structures present but seems to vary (and converge) with the ratio of the width of the buildings with respect to the width of the city.

In addition Boutin and Roussillon (2014) proposed two analytical methods to assess the influence of urban environments in the seismic response of cities. The city was considered to be formed of periodic arrangements of buildings, modelled as SDOF systems. The model was subjected to a vertically propagating SH waves. The first method considered the presence of the urban environment as an impedance on the interface between the soil stratum and the city. The second method was regarded as more thorough since it considered the determination of the field radiated by each building and the transition between a heterogeneous stress distribution at the soil-city interface to a deeper homogeneous one. The analyses were performed in function of a mechanic parameter to quantify the city influence, namely \( \sigma = \frac{\sqrt{K M}}{S \sqrt{v_s \rho}} \) where \( K \) and \( M \) are the stiffness and mass of the buildings, \( S \) is the length of the stratum’s surface while \( v_s \) and \( \rho \) are the shear wave velocity and mass density of the stratum respectively. It was shown that the impedance ratio between the soil and the city varies linearly with \( \sigma \) and that the presence of the city is negligible as \( \sigma \to 0 \); this implies that \( K \to 0 \) or \( M \to 0 \) and \( S \to \infty \) or \( \mu \to \infty \), which means that the effect of the city is not important for a very stiff stratum with widespread soft and light structures; this also suggests that for sites made of soft soil with heavy and relative stiff structures, the SCI effects might be important. Indeed cities like Mexico City have this particularities and have shown anomalous seismic records that could be explained by SCI effects (Groby et al. 2005). Regarding the dependence on frequency, they stated that, when the stratum frequency is below the city’s eigenfrequency, the city behaves as an additional mass to the system (affecting the original eigenfrequency of the system but somehow adopting it); while, for stratum frequencies above the city’s eigenfrequency, the city behaves as an additional spring-mass system (adding an extra degree of freedom and therefore another frequency, that
will be the new frequency of the system). These findings are in agreement with Kobori et al. (1973) and with results presented herein.

2.3.2 Numerical Approach

Early in the investigation of the site-city interaction (SCI) phenomenon, Wirgin and Bard (1996) presented some qualitative conclusions based on an idealised simplified model of the Mexico City urban site and previous research. They argued that there is an important amount of evidence indicating that noticeable changes in the ground motion intensity and duration can be caused by the presence of buildings. They studied the particular case of the Mexico City, intending to find an explanation to the anomalous long durations of the records during the 1985 Mexico earthquake. Mexico City has an important number of high rise buildings founded on a soft soil deposit underlaid by stiffer sediments. According to Wirgin and Bard (1996), this condition enhances the dynamic interactions of the buildings. Their model consisted of a periodic assemble of 2D blocks with separation distances up to 1 km, subject to antiplane motion. Besides all the limitations of the model, namely, not considering the 3D nature of the soil and buildings, no inclusion of soil’s properties variation with distance, no consideration of the inplane motion and periodic arrangement of structures (real cities are far from being periodic), they stated that the long duration of the Mexico earthquake might have been due to the effects of the buildings, especially for the late part of the records. Additionally, they argued that the construction of a tall building on soft soil might affect the distribution, amplitude and duration of the motion of the neighbouring buildings.

In the endeavour to find whether or not the presence of buildings can significantly modify the free field motion in an urban environment, Clouteau and Aubry (2001) proposed a BEM model able to account for a three dimensional building distribution attached to an elastic half space. The buildings were defined as single degree of freedom (SDOF) systems coupled with the BEM definition of the stratum. Two types of city arrangements were assessed, namely, a periodic distribution and a stochastic homogeneous distribution. According to Clouteau and Aubry (2001) there are three main factors that are needed for SCI to occur, these are; SSI has to take place for a significant number of structures, meaning that the natural frequency of the structures should be in the same range of the stratum; several buildings need to undergo SSSI; the traveling time of the distortions caused by the buildings need to be in the same range as the period of the excitation,
allowing interferences to occur. Figure 2.22 displays the transfer functions of the stratum with no buildings (dotted line) and with both, parametric distribution of buildings (solid line) and non-parametric distribution of buildings (dashed line). From Figure 2.22 it is evident the significant influence of the buildings on the response of the stratum, this can be seen as the splitting of the frequency and the reduction in the amplitude with respect to the case with no buildings present. The splitting of the frequency has been seen before (see Figure 2.14) in the NUPEC tests, indicating the coupling between two adjacent structures; for this case, the city is behaving as a whole, with a characteristic frequency, also referred as city frequency by Kham et al. (2006), coupled with the soil layer. Furthermore, Figure 2.23 presents the results for the case of the randomly distributed buildings with (solid and dashed lines) and without buildings (dotted line). It can be observed that the stratum possess two resonant frequencies at 0.25 Hz and 0.75 Hz, according to the authors, none of the considered buildings had frequencies around 0.25 Hz, this explains the good agreement between the curves (i.e. no interactions are taking place), however, this is not the case for the second resonant frequency, for which an important reduction as well as significant scattering (dashed lines represent the mean response ± the standard deviation) can be seen. Finally, Clouteau and Aubry (2001) conclude that the buildings may strongly modify the response of the stratum (site) mainly due to soil-structure interactions.

It has been shown that site effects (basin shape, topography, etc.) can play an important role in the amplification characteristics of a given area when subject to seismic motion, however, as pointed out by several researchers (Wirgin and Bard 1996; Clouteau and Aubry 2001; Gueguen et al. 2002) the presence of buildings can also alter the free field motion. Hence, it seems that the evaluation of both effects is a desirable approach to provide a more realistic scenario when analysing structures in urban environments. To this end Semblat et al. (2002) studied the influence of buildings, also called site-city interaction (SCI), through modifications of site effects. The alluvial basin of the city of Nice, France, was studied using 2D BEM models. The amplification factors along the 2 Km long basin were computed without the presence of any buildings, then five different building densities were considered, namely, 1, 3, 7, 15 and 30 buildings, were placed along the 2D basin. All the buildings were identical with a fundamental frequency of 0.6 Hz. Figure 2.24 presents the amplification factors along the length of the basin for all the building densities considered, under an excitation of 0.5 Hz. Two important conclusions
can be drawn from Figure 2.24: the free field amplification profile is modified even for the case of a single structure and as the buildings density increase there seems to be a group effect that modifies the profile along the whole basin. Finally, different frequencies of excitation were considered, finding that when the frequency of excitation matches the main stratum’s resonance frequency the amplifications range between 20% and 50% for low building densities and the deamplifications of up to 40% were reported for the two largest densities.

![Transfer function of the soil stratum with periodic buildings (solid lines, mean; maximum and minimum), non-periodic buildings (dashed lines, mean; maximum and minimum) and soil stratum with no buildings (dotted line) after Clouteau and Aubry (2001).](image)

Figure 2.22 Transfer function of the soil stratum with periodic buildings (solid lines, mean; maximum and minimum), non-periodic buildings (dashed lines, mean; maximum and minimum) and soil stratum with no buildings (dotted line) after Clouteau and Aubry (2001).
One dimensional ground response analysis has been widely used to take into account the influence of surface layers in the free surface ground motion. When the complexities of the stratum are considered, it can yield satisfactory estimations of the response that has been observed in a variety of cities (Tsogka and Wirgin 2003). However, several features of the estimated motion differ from the observed responses, namely, spatial variability, durations, and peak velocities. Tsogka and Wirgin (2003) studied the effect of a non-periodic 2D city founded on a soft soil layer underlaid by a hard half space using FEM. The buildings were homogenized into uniform blocks with a given set of properties calibrated to match the average response of the individual buildings. Both the motion and the buildings were invariant in the antiplane direction. The dynamic input was a Ricker pulse cylindrical shear horizontal displacement field. It was found that the case of 10
buildings was significantly different with respect to the single building case in terms of lengthening of the signal. While the effect of the building is noted for a time span of approximately 1 min for the single buildings case, this effect remained for up to 3 min in the case where 10 buildings were considered. Moreover, the peak amplitude of the buildings response was not systematically greater when compared to the single building case, nonetheless, due to the longer duration in the case of 10 buildings, the cumulative response at the top of the blocks made them significantly more vulnerable with respect to the single building case. Finally, Tsogka and Wirgin (2003) state that a plausible hypothesis to explain the anomalous long durations and spatial variability of events like the Mexico City earthquake (1985) could be that the small scale irregularities on the free surface and on the interface of foundations and the soil produce an important contribution to the motion of a given site. Hence, they argued that taking into account the built environment in the seismic risk analysis of a given site reduces the possibility of underestimating the seismic action.

With the aim of assessing the impact of a damping mechanism in both the stratum and buildings Groby et al. (2005) extended the study of Tsogka and Wirgin (2003) by incorporating energy dissipation in the materials. They showed that, even when the media are dissipative, the presence of buildings modifies the amplitude, duration and spatial variability of the ground motion with respect to the case of the same site without buildings. Moreover, the influence of inter-building distance was analysed. It was found that, decreasing the distance between buildings increases the cross interaction between buildings and the interaction with the soft soil layer; however, the buildings response, in terms of amplification and duration, is not significantly modified.

In order to provide insight into the site city interaction phenomenon, Kham et al. (2006) studied a simplified site-city interaction model using BEM. They analysed both a homogeneous, periodically spaced city and a heterogeneous, non-periodically spaced city. The stratum consisted of a trapezoidal basin of 2.4 Km at the free surface end and 2 Km at the bottom of the basin closed by 200 m ramps at the edges. The buildings were modelled as homogeneous elastic blocks characterised by their coupled fundamental frequency (SSI), two types of buildings were used, namely, 1 Hz and 2Hz (named in the paper as B1S and B2S, respectively). The input motion was vertical plane SH wave induced by a Ricker wavelet. For the case of the homogeneous, periodically spaced city just the 2 Hz buildings were used. The buildings were distributed in the central 500 m,
increasing the number of buildings was equivalent to a reduction in the inter-building spacing or an increase in city density. The response of the buildings was evaluated in terms of the buildings distortions (top minus bottom displacement) while the response of the site was assessed in terms of ground perturbations, defined as the response at a point on the surface minus the response at the same point without including the buildings (free field response). Figure 2.25 display the buildings distortions in both time and frequency domain. As can be seen, the transfer function of the single building is significantly different than those for the buildings in the case of number of buildings (N) equal to 33. This effect is still appreciable for N = 25 and is less noticeable as the buildings density decreases. It is important to highlight that the stratum resonant frequency is 2 Hz (H = 25 m, Vs = 200 m/s), this implies that the coincidence of the resonant frequency of the buildings with the one of the stratum benefit the interactions. Furthermore, by analysing different stratum thicknesses and keeping the number of buildings of N = 33 constant, it was shown by Kham et al. (2006) that the influence of the city on the free field motion becomes important when both buildings and soil resonant frequencies coincide. In addition, the ground motion modifications caused by the presence of the city contaminate the ground motion outside the city. Once again, the amplitude of said perturbations is maximum for a basin thickness of 25 m, when both buildings and soil resonant frequencies coincide.

Finally, to provide a global assessment of the city influence, kinetic energy calculations were presented, revealing that the ground energy inside the city is systematically lower than the free field motion, varying according to the extent to which both the buildings’ and basin’s fundamental frequencies coincide and the buildings density; namely, the greater the density, the greater the perturbation. This remark is very important, since it is widely known that the construction of single building with the same resonant frequency as that of the stratum is counterproductive but, according to these findings, the construction of a cluster of structures with similar resonant frequencies to that of the stratum could be beneficial. This effect could yield reductions in the order of 50% (Kham et al. 2006).
Kham et al. (2006) also presented a more realistic scenario by analysing the behaviour of a heterogeneous (two types of buildings) non-periodically spaced city. This study showed that the influence of the SCI effect is not as apparent compared to the case of the homogeneous and periodic city. This might be due to the cross interactions having been hindered by both distance and difference in frequency between both the buildings and the stratum, impeding the resonance of both buildings and stratum simultaneously. This finding reinforces that the coincidence of resonant frequencies (between structures and stratum) play a crucial role in the effects of SCI. Moreover, the radiated wavefield by the city is also affected in both amplitude and distribution, showing smaller amplitudes with respect to the homogenous and periodic city as well as an asymmetric wavefield. Additionally, the total ground motion energy can be larger than the free field energy, however, as pointed out by Kham et al. (2006) the scope of the study does not include a thorough analysis of enough scenarios to determine a trend and therefore define rules to predict areas of possible amplification in a randomly distributed city. Furthermore, the findings of Kham et al. (2006) can explain the unexpected results presented by Groby et al. (2005), who studied a city formed of 10 non-equally sized, non-equally spaced buildings. In their parametric analysis they reduced the inter building distance, expecting a greater effect on both the duration and peak amplification of the ground motion,
however they found that even though there was evidence of an increased cross interaction between buildings there was very little effect on the ground motion field, this behaviour is consistent with the fact that the coincidence of the resonance frequencies of both the soil and the structures is a crucial parameter in the SCI phenomenon. Finally, this study consolidates all the previous SCI research up to its publication date by providing the main features of the SCI effect and a methodology for its evaluation.

Following a similar methodology to the one proposed by Kham et al. (2006), Semblat et al. (2008) studied the SCI phenomenon in alluvial basins. This work can be considered as a step further with respect to the Kham et al. (2006) study, since it includes the widely known site effects, namely, consideration of the lateral depth variation (2D) of a shallow basin located in the city of Nice, France. One of the direct consequences and advantage of using 2D representations of basins is depicted in Figure 2.26, where the time histories of the February 2001 earthquake in Nice are displayed along the basin. It is evident that the amplification profile varies along the basin, and this effect would have been neglected by the common 1D ground response analysis. Moreover, Figure 2.26 also presents the spectral amplifications of the basin for experimental data, 1D ground response and 2D BEM analysis. As it can be seen, the spectral amplifications obtained assuming a 1D profile highly underestimate the reality while the 2D BEM results are within the range of the experimental data. Another important observation is that, the fundamental frequency of the basin seems to be controlled by the deepest section of the basin. As in Kham et al. (2006), the buildings were modelled as homogeneous elastic blocks characterised by their coupled fundamental frequency (SSI). Two types of buildings were used, namely, 1 Hz and 2Hz (named in the paper as B1S and B2S, respectively). For the city models they considered two uniform arrangements formed of identical and equally spaced buildings (two types of buildings considered), one irregular city, made of an arbitrary distribution of the two types of buildings along the central 500 m of the basin, and one so called realistic city, formed by both types of buildings distributed along the whole extension of the basin with a variable inter-buildings distance. The models were subject to both a Ricker wavelet, with central frequency of 0.8 Hz (equal to the basin fundamental frequency), and the February 2001 Nice earthquake. The results for the two regular cities (populated with B1S and B2S buildings) revealed that the effects are more important for the city formed of B1S buildings than to the one formed of B2S buildings, mainly because the fundamental frequency of the basin is closer to that of the B1S buildings. This
highlight, once again, the fact that SCI is highly dependent of the resonant frequencies of both the site and the city being close to each other. Moreover, the splitting of the original resonant peak (free field) of the basin for the case of uniform B1S city was evidenced. This is an indication of energy redistribution to other frequencies, leading to a reduction in the amplitude. However, this effect cannot be observed for the B2S city. Another important feature of the presence of the B1S city can be seen in Figure 2.27, namely, the radiated waves by the city seem to have a westward directivity that can be explained by the greater interactions with the deepest part of the basin, that is located towards the west of the basin. From an energy point of view, the influence of both B1S and B2S can be seen in Figure 2.28. Once again, the effect of the B1S city is prominent with respect to the B2S city. Nevertheless, an important increase in the ground motion energy outside the west side of the city is also evident. This might be cause by the radiated wavefield towards that direction.

Figure 2.26 Time histories of February 2001 quake, Nice (top), cross section of the 2D basin (Nice, France (middle) and spectral amplifications of the basin for experimental data, 1D analysis and 2D BEM model after Semblat et al. (2008).
Figure 2.27 Perturbations induced by the B1S city after Semblat et al. (2008).

![Figure 2.27](image)

Figure 2.28 Ground motion energy along the basin for the Feb. 2001 earthquake for both B1S and B2S cities, total ground motion (thick lines) and perturbations (thin lines) after Semblat et al. (2008).

In the case of the irregular city distributed in the central 500 m of the basin, Figure 2.29 display the results of both the total ground motion energy and perturbations for Ricker wavelets of central frequencies 0.8 Hz and 2 Hz, as well as for the February 2001 Nice earthquake. Comparable results to those presented in the case of the regular city are observed, with the particularity of a slight increase in the total ground motion in the eastern part of the city. Nonetheless, significant reductions are seen inside the city and an increase in the total ground energy is appreciated outside the west part of the city. The so-called realistic city displays some differences with respect to the previous two cases, particularly in the radiated wavefield by the city. Figure 2.30 shows the ground motion perturbations emitted by the realistic city, the directivity feature is no longer present, with important perturbations just in the deepest parts of the basin. Furthermore, Figure 2.31
shows both the ground motion energy as well as the perturbation energy. It seems that, in general terms, the presence of the realistic city is beneficial, with the exception of the eastern side of the city, where some amplifications take place. In addition, the increase of the ground motion energy outside the western side of the city observed for the previous cases is not present for the realistic city; this might be a consequence of the change in directivity of the radiated wavefield by the city. Finally, after the study of Semblat et al. (2008), we can conclude that the statement that the site-city resonance plays a significant role in the SCI phenomenon has been reinforced with results that show reductions of the free field motion of up to 60%. In addition, a radiated wavefield by the city was also evidenced showing a directivity feature in the case of both the regular and irregular cities.

Figure 2.29 Ground motion energy along the basin for the irregular city under both Ricker wavelet and Feb. 2001 earthquake after Semblat et al. (2008).

Figure 2.30 Perturbations induced by the realistic city after Semblat et al. (2008).
In order to provide a more realistic scenario recent research has focused on studying the site-city effect at large scale (regional) using 3D models (Taborda and Bielak 2011; Guidotti et al. 2012). In particular, Taborda and Bielak (2011) analysed the effect of SSSI and SCI of a city consisting of 74 buildings placed on a realistic model of the Volvi Valley near Thessaloniki, Greece. Using a FEM approach with hexagonal elements the model represented a volume of 16 km by 29 km with a 41 km depth. They focused their analyses in a sub region of 3 km by 3 km, where the city was located. The buildings were modelled as elastic homogeneous blocks with fundamental periods varying from 0.4 s to 7.3s; the tallest building was 310 m. The considered input motion was a simulated earthquake of magnitude $M_w$ 5.2 with a hypocentre located 5 km below the centre of the valley. They argued that, from a regional point of view (when the whole model was analysed), the city has very little effect on the basins’ response. However, in the smaller sub region (3 km x 3 km) there are important effects, namely, magnitude reduction in the buildings’ motion (at surface levels) and amplification in the ground motion between buildings. In addition, these effects are appreciable just in a perimeter ranging from 300 m to 500 m (Taborda and Bielak 2011). As stated by Taborda and Bielak (2011), the ground motion amplifications might be due to the impedance contrast between the soft soil ($V_s = 200 m/s$).
and the buildings’ foundations ($V_s = 400\text{m/s}$), causing the waves to deviate the incoming waves, intensifying the wavefield among buildings. This effect is illustrated in Figure 2.32 where the difference between the free field case and that with the city are normalized and expressed in a contour plot for both displacements and accelerations; in this representation, positive values indicate a larger response in the site city case with respect to the free field, while a negative value indicates a reduction in the response with respect to the free field. Moreover, comparisons between the ground motion (time and frequency domain) at a given point for both the free field and SCI scenarios are presented in Figure 2.33, showing the significant attenuation caused by the consideration of the multiple cross interactions taking place within the urban environment. Furthermore, Figure 2.34 displays a very useful comparison between the response of the same building under fixed base condition and coupled with the rest of the model (taking into account both SSI and SSSI effects). The significant reduction that occurs when the coupled system is analysed is evident. The study concluded that SCI has important effects on the seismic hazard of urban environments (Taborda and Bielak 2011).

![Displacements](image1.png)  ![Accelerations](image2.png)

Figure 2.32 Normalized difference between the free field scenario and with the presence of the city, positive values indicate a response larger than the free field case and vice versa after Taborda and Bielak (2011).
Figure 2.33 Comparison between the free field (blue line) motion and the base of a building (red line) at ground level for the simulated earthquake in both time and frequency domain after Taborda and Bielak (2011).

Figure 2.34 Comparison between the fixed base (green line) and the coupled (red line) motion at the top of a building for the simulated earthquake in both time and frequency domain after Taborda and Bielak (2011).

An additional 3D regional scale study of SCI was carried out by Guidotti et al. (2012), following a similar approach. Using a 3D model of comparable dimensions to that of Taborda and Bielak (2011), they analysed the central business district of the Christchurch city in New Zealand. The model was subject to the 22 February 2011 Christchurch earthquake of magnitude Mw 6.2. Among their main goals was the evaluation of the seismic response of compounds of buildings as well as the effect of densely urbanised areas in the spatial variability of the ground motion. This was of particular interest as an alternative way to explain the irregular damage distribution in the city during that seismic event. Their model was 60 km by 60 km with a depth of 20.5 km; the central business area was 1 km by 1 km and the buildings were placed according to the real distribution
of the city in this area. Buildings were modelled as homogeneous blocks with a set of properties selected to match the dynamic response (fundamental periods) of the existing structures, approximatively calculated as $T = 4h/V_s$, where $h$ is the height of the building and $V_s$ is the shear wave velocity of the material set at 100 m/s for building and 400 m/s for the buildings’ foundation. Figure 2.35 shows the spatial distribution of the peak velocity, computed as the mean of the two horizontal components. It can be seen that the velocity profile is somehow amplified within the city, as also pointed out by Taborda and Bielak (2011). However, snapshots presented in Figure 2.36 reveal the radiated perturbations by the city, showing that, in addition to the physical obstacle that the city represents for the incoming seismic waves (passive modification of the wavefield) it also modifies the wavefield by generating both translational and rotational waves (Guidotti et al. 2012).

Figure 2.35 Spatial distribution of the peak velocity (computed as the mean value of the horizontal components) with and without (left) the city after Guidotti et al. (2012).

Figure 2.36 Snapshots ($t= 5.5, 6, 6.5$ and $7$ s, from left to right) of the wavefield with (bottom) and without (top) the city after Guidotti et al. (2012).
It has been widely known in the last decade that horizontal heterogeneities, also called site effects, can have a significant influence in the ground motion (Bard et al. 2006). To study the combined effect of the so called site effects and SCI, Sahar et al. (2014) analysed three different basin geometries with four different cities configurations using finite differences. Their methodology can be considered an extension of that proposed by Kham et al. (2006). Sahar et al. (2014) analysed three different basin shapes, namely, rectangular, elliptical and trapezoidal. The cities were formed of two types of buildings divided into four configurations, namely, two homogeneous and regular cities, one for each type of building, one heterogeneous and regular and one heterogeneous and irregular. They highlighted the crucial importance of site-city resonance to observe significant SCI effects and pointed out that this influence is enhanced by an input that matches that frequency. Moreover, Sahar et al. (2014) stated that SCI effects on the ground motion very much depend on the basins’ shape, showing that the reduction in maximum amplitude, kinetic energy and average spectral amplification (ASA) changes for different basins’ shape. This can be seen in Figure 2.37.

Figure 2.37 Variation of maximum amplitude reduction, kinetic energy and ASA (from left to right) for three different basins’ shape after Sahar et al. (2014).
In the attempts to understand and quantify the effects of SCI, insight has been sought through simplified models of cities (Wirgin and Bard 1996; Clouteau and Aubry 2001; Kham et al. 2006) by assuming a 2D geometry. Horizontal heterogeneities have also been accounted of in the works of Semblat et al. (2008) and Sahar et al. (2014). However, both the need for more realistic models that can accommodate the 3D nature of the SCI phenomenon, and the increasing computational power of computers has motivated some researchers to study SCI at a regional scale (Taborda and Bielak 2011; Guidotti et al. 2012), these models often include some information and modelling of the ground motion source and site conditions, providing information regarding the propagation of the waves through the studied domain (regional). Nevertheless, detailed information of the soil conditions throughout tens of kilometres is virtually unavailable, leading to some degree of uncertainty between the results estimated by the models and the reality. With the aim of taking the analysis of SCI one step forward, Isbiliroglu et al. (2015) performed an study on the coupled soil-structure interaction effect in artificial clusters of buildings located in the San Fernando valley near Los Angeles city in the U.S. Two of the main innovative features of the study are the implementation of a kinematic fault rupture model and the consideration of changes in soil damping and stiffness due to soil nonlinearities. The work by Isbiliroglu et al. (2015) was divided in two stages, namely, ground motion simulation, accounting for earthquake source, with no city (cluster of buildings) and modelling of the cities behaviour in a reduced domain using as input the data generated in the first stage. The regional domain was 81.92 km by 81.92 km and 40.96 km deep, while the reduced model was 5.12 km times 5.12 km and 1.28 km deep. Both domains were discretized and analysed using the FEM. In the case of the full model the material was modelled as viscoelastic, with elastic properties taken from the Community Velocity Model of the Southern California Earthquake Centre (CVM-S) and 5% damping. On the other hand, the reduced model was evaluated using both the viscoelastic model and a so-called softened soil conditions, to account for soil nonlinearities. The buildings were modelled as homogeneous elastic blocks formed of the same quadrilateral elements used for the stratum. Three types of buildings were used, and different materials were considered for the foundation and the superstructure. The density of the buildings material as well as the shear wave velocity of the foundations remained constant for the three types of buildings, with values of 300 kg/m³ and 750 m/s, respectively. Detail of the geometry and fundamental frequencies of the buildings can be found in Isbiliroglu et al. (2015). The input was the ground motion generated by the1994 Mw 6.7 Northridge, California
Analyses were carried out for different uniform clusters of buildings, namely, rectangular arrangements of 3 by 3, 5 by 5 and 9 by 9 of the same type of buildings; moreover, three different inter-building separations were considered for each type of building (three different set of separations per building). All the scenarios were compared with the control case, that consists of the corresponding isolated single building. In the first stage (no buildings) ground motions measured at the location of the city (reduced model) had a comparable order of magnitude with records of the Northridge earthquake in that region. Simulations carried out on the reduced model without the presence of the buildings for the two soil conditions (linear and non-linear) revealed that in the buildings area the peak ground velocity (PGV) varies from 0.55 m/s to 1.5 m/s for the linear soil model and from 0.6 m/s to 2.3 m/s for the non-linear soil model. Figure 2.38 displays the results for one of the building types (40 storeys structure), expressed as percentage change in the free field motion caused by the presence of structures; positive values correspond to amplifications, while negative to reductions. It can be seen that the greater the number of buildings and the shorter the inter-building separation, the greater the interaction. Average reductions of 10% to 15% were observed at the base of the buildings for almost every case (Isbiliroglu et al. 2015). In addition, reductions in the order of 40% are evidenced for the case with the non-linear soil model. Among their concluding remarks, the authors highlighted that the SCI has very little effect in the frequency variation when compared with the isolated single structure, as change in natural frequency tend to be more sensitive to SSI. However, changes in amplitude of up to 40% were reported and these changes are 75% attributable to SCI effects while the remaining 25% to SSI. As the authors have pointed out, SCI effects are very likely to decrease for heterogeneous arrangements.
Figure 2.38 Changes in the peak horizontal-magnitude velocity for a 40 storey building type with different number of neighbouring structures and separations for both elastic and softened soil models (Isbiliroglu et al. 2015).

2.3.3 Experimental Approach

As can be inferred by now, experimental studies that address SCI are not abundant. To the author’s knowledge, the only experimental setup designed exclusively to assess the SCI phenomenon was the one carried out by Schwan et al. (2016). This research combined successfully analytical, numerical and experimental approaches to provide insight into the SCI phenomenon. They studied an idealised city made of periodically distributed structures, based on the premise that some districts in certain cities can be represented in this way. Their analytical formulation was based on the analysis of the surface impedance of the city derived through homogenization methods. The numerical application was a replica of the experimental arrangement made with BEM to define the stratum and SDOF oscillators located at the surface of the domain to simulate the structures. The full experimental setup was formed of 37 oscillators (structures) attached to the top of a polyurethane foam block to simulate the stratum, and the block was fully attached to surface of a shaking table. The oscillators were made of an aluminium sheet
0.5 mm thick clamped by two aluminium angles forming a foundation with a total length of 2.65 cm. The height of the oscillator was 18.4 cm, their length was 1.75 m, since the experiment was designed to recreate plane strain conditions and could resonate in bending as a consequence of an out of plane motion while remaining virtually inert while exited by an in-plane motion. Figure 2.39 depict the experimental setup with dimensions. The fundamental resonant frequency of the structures was 8.45 Hz, while the fundamental resonant frequency of the block was 9.36 Hz and 9.11 Hz in the X and Y direction respectively. An array of 3D accelerometers was placed on the oscillators and the foam block. The input excitation consisted of a Ricker wavelet with central frequency around 8 Hz, modal shapes were confirmed using sinusoidal inputs. In order to assess the dependence of the SCI on the number of structures present in the city, experiments were carried out with 1, 5, 9, 19 and 37 oscillators with separations of 40 cm, 20 cm, 10 cm and 5 cm respectively. Figure 2.41 displays the time histories recorded with the accelerometers located on the surface of the block as well as the transfer functions between the bottom and top of the foam block. It can be seen that for the configuration 5 (bottom of Figure 2.41) the effect of the city is negligible and, the peak of the transfer function is located at the resonant frequency of the block. However, some effect is seen for configuration 4 (centre of Figure 2.41), the main frequency is shifted upwards slightly and a small peak becomes noticeable around 7.5 Hz, while no effect can be observed in the time history. For configuration 1 (top of Figure 2.41) the effect is significant in both the time and frequency domain, the time history exhibits more than one frequency. The transfer function reveals a redistribution of the energy in two frequencies, the stratum frequency has been pushed upwards while the fundamental resonant frequency of the site-city system has changed to 6.8 Hz. This behaviour is in agreement with the statement of Boutin and Roussillon (2014), in which they pointed out that when the stratum’s eigenfrequency is higher than the city’s eigenfrequency, this latter behaves as an additional spring, leading to an additional frequency of the system. On the other hand this results are in conflict with Ghergu and Ionescu (2009), as they indicate that SCI does not depends on the number of structures present in the city. It is evident that the number of structures have an influence in the response in the city. This is supported by results reported by Schwan et al. (2016) and analyses presented herein. An additional finding that is worth highlighting is that the number of structures in the system seem to influence the response of the adjacent structures through the soil, this effect can be seen in Figure 2.40 in which
the transfer functions between the surface layer and the top of the oscillators are displayed for all the studied configurations.

Figure 2.39 Experimental setup and dimensions after Schwan et al. (2016).

Figure 2.40 Transfer function between the bottom and top of the oscillators, configuration 1 correspond to 37 oscillators and increases corresponding respectively to the remaining cases studied (Schwan et al. 2016).
2.3.4 Summary

- According to the results presented by Gueguen et al. (2002), in addition to the surface ground motion, due to the wave propagation in the soil, buildings in a city radiates a wavefield with amplitudes comparable to those of the free field motion. Specifically, energy calculations showed that the intensity of the wavefield radiated by the Roma Norte district was equivalent to 70% of the intensity of the input.
• It has been shown by several researchers (Gueguen et al. 2002; Semblat et al. 2002; Kham et al. 2006; Semblat et al. 2008; Sahar et al. 2014) that the influence of the site-city interaction is particularly significant when both soil and structures have a similar resonant frequency.

• According to Kham et al. (2006), the construction of a cluster of structures with similar resonant frequencies to that of the stratum could be beneficial, as the site-city effect could yield reductions in the order of 50% in the surface ground motion.

• The feature of energy redistribution in frequency due to the presence of the city was observed by Clouteau and Aubry (2001), Semblat et al. (2008), Boutin and Roussillon (2014) and Schwan et al. (2016); the same phenomenon was observed by Padrón et al. (2009) and Clouteau et al. (2012) when studying SSSI. This implies that the city can behave as a whole and act as a tuned mass damper to the coupled city-stratum system in the same way that an adjacent buildings can have this effect in neighbouring structures (Alexander et al. 2012).

Finally, it seems that several studies have concurred in the fact that the presence of the city can modify the free field motion, especially when the city’s arrangement tends to be homogeneous and possess a predominant natural frequency in the same range as that of the stratum. Moreover, the existence of the so-called site-city resonance, defined as the coincidence of the city and stratum natural frequencies, seems to be a predominant remark among the scientific community studying SCI. Furthermore, a collective effect seems to take place causing a redistribution of the energy in frequency. This effect has already been observed in the study of adjacent structures in SSSI. It is this feature that has recently been exploited by Cacciola et al. (2015) and Cacciola and Tombari (2015) through the Vibrating Barrier as a vibration control strategy (See section 2.4.4).

From the study of the contributions on site-city interaction, it is evident that there are gaps in the existing knowledge that requires further studies. Namely, there is a lack of experimental models that address SCI. Physical models are of paramount relevance for the understanding of this challenging phenomenon that up to now has mainly been addressed through simplified numerical models. Furthermore, to the author’s knowledge, randomness in both soil and ground motion excitation when studying SCI has not yet been addressed by numerical studies. Moreover, as pointed out by previous studies, the site-city interaction modifies the ground motion in the urban environment. The modification has been found to be either beneficial or detrimental. Nevertheless, the
influence of the buildings in the ground motion has been established. Therefore, this effect might be used following previous studies on the vibrating barriers to develop a novel and ambitious strategy aimed to reduce vibrations of structures in an urban environment for the first time.

2.4 Non-localised vibration control

Vibration control of structures in civil engineering encompasses a series of techniques aimed at reducing the structural response when subjected to external dynamic action such as wind, waves, earthquakes, etc. Vibration control strategies can be broadly classified as active, passive and hybrid. Active vibration control systems are characterised for providing external energy to the system in order to minimise the structural demands. These systems tend to be more versatile but also more costly than the other available solutions. Passive vibration control comprises a wide range of solutions that do not rely on an external energy supply to function. A combination of the aforementioned approaches are often called hybrid systems. A detailed review on vibration control strategies state of the art can be found in Saaed et al. (2013).

In the context of vibration control, an additional distinction can be made. Depending on whether the control devices are directly connected to the structure, in which case they are referred as “localised” solutions; or if on the contrary, they are adjacent to the structure, they can be classified as “non-localised” solutions. This terminology is in agreement with Tombari et al. (2018). There has been a plethora (Hrovat et al. 1983; Symans and Constantinou 1999; Song et al. 2006; Owji et al. 2011; Ozbulut et al. 2011) of research and successful full scale implementation of localised vibration control devices, and their study is beyond the scope of the present study. Instead, this section is devoted to provide a review on the different non-localised techniques that have been proposed in literature.

2.4.1 Trenches as wave barriers

In the effort to mitigate the effects of incoming seismic waves on structures Woods (1968) conducted field tests to evaluate the effectiveness of open trenches in reducing the amplitude or vertical ground motion. This approach seeks to somehow disrupt the propagation of the waves travelling through the soil. In the majority of cases these solutions are designed to reduce the vibrations of structures in the vicinities of discrete sources of vibrations like building sites or railways. In this context, Yang and Hung
(1997) carried out a parametric study on the effectiveness of different types of trenches to mitigate the vibrations induced by the passage of trains. Open trench, infilled trench and elastic foundations systems were investigated. Figure 2.42 presents a graphic representation of the three proposed wave barriers. According to their findings, the effectiveness of each barriers largely depends on the barrier dimension relative to the wavelength of the ground-borne vibration. Using these types of barriers was proven unfeasible for long wavelengths. Further, the performance of infilled trenches is better when the stiffness of the barrier is greater than that of the soil, while for the case of elastic foundation the opposite is true. Among the main parameters governing the behaviour of the trenches, the depth seems to be of primary importance.

Figure 2.42 Sketch of problem with the three studied scenarios, open trench, infilled trench and elastic foundation (Yang and Hung 1997).

The isolation capabilities of trenches was assessed experimentally by Çelebi et al. (2009) through a series of full-scale field tests were undertaken by the authors. The experimental setup was made of two neighbouring concrete foundations, one of which was vertically excited by an electrodynamic shaker. A trench was dug in between foundations and two scenarios were assessed, namely open and infilled trench. Three filling materials were considered, namely water, bentonite and concrete. Vibration reductions were reached in all cases; nevertheless, open trenches are generally more effective than infilled trenches. When infilled trenches were used, softer filling materials were shown to produce better results. Despite the advantages of using open trenches to minimise the effect of dynamic waves propagated through the soil, in practice this solution is limited to a few meters, which limits the application and efficiency of the barrier. To add stability to the excavation and allow for greater depths either fillings (e.g. bentonite muds) or side walls are necessary. Tsai and Chang (2009) studied the effects of trench wall slidings, sheet of
piles or diaphragm walls through 2D BEM. The results indicated that the screening effectiveness of the stabilised excavation is significantly different than the pure open trench case, being the latter more effective. For shallow depths the diaphragm wall option performs best while, for greater depths sheets of piles deliver greater vibrations reductions. Additionally, Ekanayake et al. (2014) explored the effectiveness of expanded polystyrene (EPS) geofoam as a filling material by means of 3D finite element models validated through full scale field experimental data. They concluded that EPS geofoam is the most efficient filling material. Increase in the efficiency of the EPS filled trench was found to be proportional to the trench depth.

According to Coulier et al. (2015), if properly designed, stiff wave barriers are capable of reducing vibration levels significantly. Design, installation, testing and numerical analyses of a 55 x 1 m cross section and 7.5 m depth concrete filled trench in soft soil were carried out by Coulier et al. (2015). The wall was intended to mitigate the vibrations generated by the passing of a train through a conventional railway located in the vicinities of the wall. They reported that vertical velocities records indicated reductions in the range of 5 dB for frequencies above 8 Hz, with a peak reduction of 12 dB at 30 Hz.

Up to now, most applications of this non-localised strategies has been in the context of waves propagating from a nearby discrete source like a train passing through the railway. Carpinteri et al. (2016) carried out a numerical investigation of the effects of a vertical barrier inside the ground when the soil is subjected to earthquake ground motion. The barriers were aimed to protect heritage buildings when subject to earthquake excitation. They reported acceleration reductions of about 59% attributed to the presence of the barrier. Following a numerical approach, Rezaie et al. (2018) introduced a novel approach for the mitigation of earthquake induced vibrations, consisting of buried concrete barrier aimed to intersect the incoming wave field; limiting the amount of energy that reaches the system. The idea is to somehow obstruct the vertically propagating shear waves using concrete blocks. Figure 2.43 display some of the barriers configurations analysed by Rezaie et al. (2018). They concluded that the performance of the buried barriers depends on the complex interaction of the layout, natural frequencies of the structure, and frequency content of the input. Further, they reported drift reductions for all the analysed cases, and maximum reductions for the case of stiff structures subject to high frequency earthquake loadings.
There seems to be a point of convergence in the literature presented herein and is that a better performance of the barriers can be achieved for high frequency dynamic loadings. This, combined with the comments of Yang and Hung (1997), namely that using this types of barriers was proven unfeasible for long wavelengths, seems to indicate that for barriers to be effective against vertically propagating shear waves they must be of dimensions that might not be feasible from a construction point of view.

2.4.2 Periodic piles barriers

One possible non-localised vibration control strategy consists of shielding the soil around the structure of interest by means of arrangements of piles. The piles are intended to act a physical barrier to disrupt the propagation of the waves through the soil. Avilés and Sánchez-Sesma (1983) proposed a theoretical study on the usefulness of rows of rigid piles as isolating barriers for elastic waves propagating through the soil. Exact solutions to the two-dimensional problem of multiple scattering of elastic waves by a row of rigid piles were given. Among their main conclusions it stands out the relationship between the effectiveness of the barrier with the wavelength of the incident wave field, namely, good results are obtained if the radius of the piles lies between two and eight times the wavelength of the incident wave field. They highlight that, for shorter wavelengths, a continuous barrier would perform better.

Using 3D frequency domain boundary element models, Kattis et al. (1999) analysed the influence of tubular and solid piles with circular or square cross section, in the effectiveness of rows of piles as vibration isolation systems. The vibration source was a vertically loaded rigid footing with a harmonic load that varies with time. Kattis et al. (1999) stated that the screening behaviour of the piles is similar to that of trenches, being the piles always less effective than the trenches. Nevertheless, in cases where the wavelength of the incident waves is too large, pile barriers might be the only option. Finally, they concluded that the most important parameter controlling the effectiveness
of the piles row is the inter-pile spacing, while the cross section of the piles does not play an important role. Following a similar approach, Tsai et al. (2008) analysed the screening effectiveness of piles made of different materials, namely, steel, concrete hollow, concrete solid and timber piles. In contrast to what was said by Kattis et al. (1999), Tsai et al. (2008) concluded that the pile length had a significant effect on the screening effectiveness of the barrier, while the net spacing and distance from the source has a less important role. In addition, they stated that the screening effectiveness of pile barriers is independent of frequency.

More recently, Huang and Shi (2013) used the theory of solid-state physics to analyse the reduction effects of pile barriers and provide a new concept for designing pile barriers. The screening effectiveness of finite periodic pile barriers was numerically evaluated. They found that within the periodic structures there were attenuation zones. This observation was validated using experimental data; furthermore, vibrations with frequencies in the attenuation zones can be significantly reduced.

2.4.3 Metamaterials

Metamaterials are engineered materials with designed properties that materials found in nature do not possess. Metamaterials have been widely used and still remains an open research field, with a broad range of applications. The main idea behind metamaterials, lies on the ability to manipulate waves that pass through the material not by changing the material geometry only (e.g. in the field of optics) but instead altering its properties as well. The ability to modify both geometry and material properties significantly expands the applications of metamaterials in wave manipulation. Inspired by this idea, the concept of metamaterials has also expanded into civil engineering problems, particularly those involving wave propagation. All civil engineering structures are funded on the ground and eventually subject to the dynamic action of waves that propagate thought the soil stratum. The application of the metamaterial concept in civil engineering applications hinges in the modification of the media in which the dynamic waves propagate, that is to say the soils stratum surrounding the structures. In this endeavour Brule et al. (2014) studied this phenomenon, both experimentally and numerically. Their experiment was made on an alluvial basin 200 m deep. The seismic metamaterial was created by drilling a series of cylindrical empty boreholes. Velocities were recorded by an array of sensors when the site was subject to seismic waves generated using a monochromatic
vibrocompaction probe. Figure 2.44 depicts the field test site setup, displaying the seismic waves source, the metamaterial arrangement and the sensors array. They observed a modification of the seismic energy distribution in the presence of the metamaterial that was congruent with numerical results obtained using an approximate plate model, hence validating the use of plate models to analyse this type of problems.

A different approach was taken by Achaoui et al. (2016) who proposed the implementation of arrays of inertial resonators hosted in the soil to act as a seismic shield. Each resonator was formed of 0.74 m diameter iron spheres connected to a bulk of concrete through either rubber or steel ligaments. They reported that some damping was achieved in the 8–49 Hz frequency range.

More recently, Colombi et al. (2016b) introduced the idea of trees acting as locally resonant metamaterials with the ability of attenuating Rayleigh surface waves. They state that locally resonant metamaterials made of vertical resonators on a subwavelength scale induce large frequency bandgaps for Rayleigh waves at tens of Hz. This arrangement possess dynamic properties that are similar to those of forest trees and therefore they could be somehow used as means to mitigate the propagation of Rayleigh waves as long as the longitudinal resonators have a resonant frequency in the range of interest. A similar approach was used by Colombi et al. (2016a) in the development of the metawedge metamaterial aimed at controlling seismic Rayleigh waves. Further, Palermo et al. (2016) study the potential of buried resonant structures under the soil surface to act as a wave...
shield. This arrangement was referred by the authors as a metabarrier. They stated that meter sized structures can control surface seismic waves with wavelengths ranging from 10 to 100 m. Moreover, an analytical framework to solve the problem of metasurfaces formed of arrays of rods that act as resonators is given in Colquitt et al. (2017), they stress the need to solve the canonical problems in order to rigorously study the seismic metasurfaces.

One of the limitations of seismic metamaterials is to provide protection in a broad range of frequencies. Achaoui et al. (2017) proposed the use of zero-frequency stop band materials to provide effective broadband protection. They identified zero-frequency stop bands located at the connection between concrete columns and the bedrock, as depicted in Figure 2.45. Analyses on a realistic 15 m deep sedimentary basin yielded zero frequency bands covering the broad frequency range of 0-30 Hz.

Finally, taking into account the effect of vertically propagating body waves, Haupt et al. (2017) proposed a novel approach to mitigate ground motion caused by earthquakes implementing subsurface seismic barriers. The subsurface was formed of a combination of boreholes and trenches. They addressed the issue of vertically propagation body waves by adding inclined opposite boreholes that form a muffler structure. They concluded that subsurface borehole arrays can significantly reduce the impact of hazardous seismic waves arriving at given location.
2.4.4 Vibrating barriers

Up to now, most of the studies presented in this section, except Haupt et al. (2017), were aimed at the protection of structures against surface waves, especially Rayleigh waves. However, in real cases the wave propagation is highly influenced by site effects, which not always favour the formation of a particular type of surface waves. Furthermore, a vast majority of construction sites are founded on shallow soil deposits, characterised by shear wave velocities that increase with depth. An inclined wavefront that strikes pseudo-horizontal strata, with decreasing shear wave velocity towards the surface, is usually reflected to a more vertical direction. This causes shear waves to propagate vertically when they reach a given site, and then the ground response is dominated by vertically propagating shear waves (Kramer 1996). It is widely known that body waves travel faster than surface waves, hence, in the case of an earthquake shear waves will strike before surface waves. Moreover, these solutions are highly dependent on the direction of the ground motion, and to be effective in all possible scenarios, need to be implemented all around the structures they intend to protect. Additionally, the solution proposed by Palermo et al. (2016) is aimed at transforming surface waves into shear bulk waves that propagate away from the soil surface. This waves that propagate away from the surface might be reflected and refracted as they encounter different geological materials, and in the case of urban areas, underground facilities. Under this scenario, it is possible that the waves that are being redirected in one area increase the seismic energy elsewhere.

Moreover, the practicality of the solution proposed by Colombi et al. (2016b) is questionable. Indeed, according to Haupt et al. (2017), in order to access frequencies in the range of 1 – 10 Hz, the vertical resonator will need to be several hundred meters long. There seems to be a lack of substantial strategies to protect structures against the influence of vertically propagating body waves. This need for new solutions has driven the development of a novel device called the Vibrating Barrier (ViBa). Cacciola and Tombari (2015) proposed for the first time a device hosted in the soil that exploits the SSSI phenomenon able to absorb part of the seismic energy to attenuate the vibration in neighbouring structures. They developed closed form solutions for the design of the ViBa in the case of harmonic excitation and experimental tests yielded a reductions of up to 87% in structural acceleration. Furthermore, Cacciola et al. (2015) studied the effectiveness of the ViBa as a vibration control strategy for monopole structures founded on a linear elastic stratum, subject to synthetic ground motion, modelled as zero-mean
quasi-stationary spectrum-compatible Gaussian stochastic process. The ViBa was designed following an optimisation process and reached response reductions of up to 44%.

More recently, Tombari et al. (2018) expanded the application of the ViBa to more than one structure and proposed closed form solutions for the optimal design of the ViBa for two structures when subject to harmonic excitation. Shaking table experiments were undertaken on two structures founded on silicon rubber simulating the soil. They reported reductions of up to 46.2% of the maximum acceleration for both structures.

The ViBa is adopted in the present study as a non-localised vibration control strategy for being both versatile and technically sound. Further, its application is expanded to protect several structures in an urban environment taking into account both site effects and site-city interaction. The ViBa design relies heavily in the SSI and SSSI phenomena and its appropriate characterisation is of paramount importance. Hence the following two Chapters are devoted to develop a methodology for identifying the parameters governing both SSI and SSSI phenomena.
3 Soil-structure interaction identification

With the increased development in computers and numerical methods, particularly the finite element method, the study of SSI escalated significantly and expanded into a great number of disciplines in civil engineering. However, the ability to consider more realistic scenarios came at a price, high computational expense, especially when analysing relatively big problems.

The need for models that are able to account for the complex phenomenon of SSI while keeping computational expenses affordable gave way to the introduction of low order discrete models (LODM) or lumped parameter models, as referred in the literature (Alexander et al. 2013). LODM have been widely used by many researchers (e.g. Wolf and Hall 1988; Kramer 1996; Harte et al. 2012; Damgaard et al. 2014a; Damgaard et al. 2014b; de Silva et al. 2018) to solve SSI related problems. These models are often formed by lumped masses and discrete stiffness and damping coefficients. Nonetheless, the problem of selecting the right set of model parameters to represent the real structure conditions and response persists. Even though closed form solutions are available for the determination of the constants involved in SSI (Kramer 1996), these are limited to certain foundation geometries and idealised soil-foundation interfaces.

One solution that has been extensively adopted to close the gap between numerical estimations and real structural behaviour has been to implement model identification techniques (Friswell and Mottershead 1995; Fassois 2001; Petsounis and Fassois 2001; Yu et al. 2007; Hu et al. 2014; Lu et al. 2018). Model identification techniques are classified as parametric and non-parametric. Parametric identification of vibrating systems comprises the development of parametrised models based on the response of dynamically excited systems. Conversely, non-parametric identification involves the determination of the transfer function of the system in terms of analytical representations (Fassois 2001). Least-square related methods are to date one of the most employed approaches in parametric identification. The main idea behind least square methods pivots around the minimisation of the so-called penalty function representing the error between the measured and estimated data. In the framework of parametric identification, it has been customary to use modal response data to define the penalty function. Generally, modal data encompasses modal frequencies, modal damping and modal shapes. This
approach presents the difficulty that experimentally just low order modes can be accurately estimated. Additionally, the measurement of modal shapes depends on the sensors arrangement and are obtained with at least 10% of error (Friswell and Mottershead 1995). If the number of unknown model parameters is greater than the number measured data, the system is underdetermined, and the system of equation will definitely be rank deficient. There are many alternatives to overcome this obstacle e.g. adopt the frequency response function (FRF), in the range of interest, as penalty function or use weighting matrices to introduce additional information on the model and overcome both rank deficiency and ill-conditioning. Nonetheless, for lightly damped structures the accurate prediction of the peaks in the FRF can be difficult (Cacciola et al. 2011), moreover, the implementation of weighting matrices requires information on the variance of the different parameters, and even though the ability to set levels of uncertainty in the parameters is very powerful, estimating the variance can be arduous (Friswell and Mottershead 1995). Driven by the need to improve rank deficiency and ill conditioning problems, several time-domain strategies have been proposed; a thorough comparison of the most important ones is given in Petsounis and Fassois (2001). More recently, an innovative time-domain identification method was proposed by Cacciola et al. (2011), in which a bounded variable least-square method is suitably employed. One of the main features of this method is the definition of the penalty function from the response time-history of dynamically excited systems. In this way problems of rank deficiency are avoided.

Identification techniques to address SSI have been used by several researchers (Stewart and Fenves 1998; Ghahari et al. 2016; Pioldi et al. 2017; Shirzad-Ghaleroudkhani et al. 2018). Stewart and Fenves (1998) developed numerical models of Laplace domain transfer functions and used least-square techniques to minimise the error between the model and the recorded output of strong ground motion in buildings. Ghahari et al. (2016) implemented blind identification techniques to update a FEM considering SSI effects of the Millikan library building, of the California Institute of Technology, using the modal response data. Furthermore, Pioldi et al. (2017) proposed an innovative algorithm that includes SSI effects in non-parametric identification of structural systems based on frequency domain decomposition algorithms. Moreover, Shirzad-Ghaleroudkhani et al. (2018) adopted a LODM made of a Timoshenko beam resting on both sway and rocking
springs to account for SSI effects and proposed a probabilistic method for model identification using modal response parameters.

In this Chapter, the bounded variable time-domain least-square identification procedure proposed by Cacciola et al. (2011) is employed to identify the parameters of a series of LODM able to account for SSI effects. This is achieved by measuring the response time histories of HODM (high-order discrete models) under forced excitation.

3.1 Bounded variable time-domain identification procedure

Penalty function methods normally use a truncated Taylor series expansion of the modal data in terms of the unknown parameters (Friswell and Mottershead 1995), however, a different response quantity can be used. Consider the Taylor series expansion of the response \( x(a) \) of order \( r \times 1 \), in terms of unknown model parameters \( a \) of order \( p \times 1 \).

\[
x(a) = x(a_o) + S_{a_o}(a - a_o) + \frac{S_{a_o}'}{2!}(a - a_o)^2 + \ldots
\]  

(3.1)

where \( S_{a_o} \) is the sensitivity matrix, of order \( r \times p \), containing the first partial derivative of the response \( x(a) \) with respect to the unknown parameters \( a \) determined for an initial set of parameters \( a_o \) :

\[
S_{a_o} = \left[ \frac{\partial}{\partial a_1} x(a) \ldots \frac{\partial}{\partial a_p} x(a) \right]
\]  

(3.2)

If the Taylor series is restricted to the first two terms, the following linear approximation can be cast:

\[
\delta x = S_{a_o} \delta a
\]  

(3.3)

where \( \delta x = x(a) - x(a_o) \) is the change in the response and \( \delta a = a - a_o \) is the change in the parameters. Considering that \( x(a) = x_m(a) \) corresponds to the target response measured either experimentally or through a high order finite element model and \( x(a_o) \) as the estimated response quantity through a low order discrete model for an initial set of parameters \( a_o \). Equation (3.3) represents a set of linear equations with the unknown parameters of vector \( \delta a \). If the sensitivity matrix is square and invertible the solution is trivial. However, if the sensitivity matrix is rectangular the system could be either over-determined \((r>p)\) or under-determined \((r<p)\). In which case the problem can be solved by
the usage of the Moore-Penrose pseudo inverse procedure as in Friswell and Mottershead
(1995) and presented below:

\[
\delta \alpha = \left[ S_{m}^{-T} S_{m} \right]^{-1} S_{m}^{-T} \delta x
\]

Equation (3.4) can be casted in a form that would suit an iterative procedure as follows:

\[
a_{i+1} = a_{i} + \left[ S_{m}^{-T} S_{m} \right]^{-1} S_{m}^{-T} \left( x_{m} - x_{i} \right)
\]

where \(a_{i}\) represent the vector of unknown parameters at the iteration \(i\) and \(x_{m} - x_{i}\) is the
difference between the target response and that estimated for the parameter set \(a_{i}\). It can
also be shown that equation (3.5) represents the iterative solution (Friswell and
Mottershead 1995), according to the Newton-Gauss method, of the penalty function \(J(\alpha)\)
defined as

\[
J(\alpha) = \varepsilon^{T} \varepsilon
\]

where

\[
\varepsilon = \delta x - S_{m} \delta \alpha
\]

It is important to mention that any parameter that has an influence on the model response
can be included in the unknowns parameter vector \(a\). However, two important aspects
must be considered, namely, the parameters must be in the same order of magnitude for
the problem not to become ill-conditioned; for example, if both modulus of elasticity and
Poisson ratio were being updated scaling factors must be introduced to improve the
numerical conditioning of the problem. Secondly, the procedure is based in the
assumption that equation (3.3) represents a combination of independent linear equations;
if a parameter has none or very little influence in the measured model response vector \(x_{i}\)
, the problem becomes ill-conditioned and additional measures need to be taken (Friswell
and Mottershead 1995). In this study the target response \(x_{m}\) will be considered as either
the acceleration or displacement time histories at a given measuring point on the structure.
This approach was chosen over the traditional modal data to circumvent the issue of
under-determinacy of the problem in the case in which the number of unknown
parameters is greater than the number of equations. Considering the time histories ensures
over-determinacy of the problem by having a greater number of equations than
unknowns (Cacciola et al. 2011).

The structure to implement the identification procedure using equation (3.5) is as follows:

1. Determine the measuring points on the high order/experimental model.
2. Collection of the data that comprises the vector $x_m$ from the points selected in (1).
3. Selection of the structural parameters to be sought and assemblage of the parameter vector $\alpha_o$ of order $p \times 1$ with the initial values.
4. Calculation of the vector $x(\alpha_o)$ corresponding to the response of the system for the set of initial parameters.
6. Compute $\alpha_i$ using equation (3.5) and calculate $x(\alpha_i)$ and $S_\alpha$.
7. Repeat step (6) until convergence is reached.

Initially the measured response vector must be defined, since all digital signals are processed or generated in a discrete fashion the vector $x_m$ containing the measured response data at different $s$-points of the structure can be written as follows:

$$
x_m = \begin{bmatrix}
x_m^{(1)}(\Delta t) \\
\vdots \\
x_m^{(1)}(n\Delta t) \\
\vdots \\
x_m^{(s)}(\Delta t) \\
\vdots \\
x_m^{(s)}(n\Delta t)
\end{bmatrix}
$$

(3.8)

where the superscript $1,2,\ldots,s$, represents the measuring point in the structure; $\Delta t$ is the sampling time; $n$ is the total number of time steps and $n\Delta t$ is the total duration of the time history. The dimension of $x_m$ will be $r \times 1$, with $r = sn$. It is worth mentioning that, depending on the nature of the target model, either a high order finite element or physical model, the vector $x_m$ could be formed of different response parameters. This is due to the fact that extraction of displacement, velocities and acceleration time histories out of a high order finite element model (HOFEM) is straightforward, contrary to the case for physical models. The most common tool to measure the response of physical models under dynamic excitation is the uniaxial accelerometer. Such tool will produce an
acceleration time history that can then be integrated to obtain both velocity and
displacement time histories; however, during this process the data can suffer numerical
corruption and consequential bias of the solution. Nevertheless, the procedure can be used
to match any response parameter of the structure, as long as consistent quantities are used,
this will be shown in the following sections.
Consider an m-degree of freedom (DOF) low order linear system, whose dynamic
response is governed by the following equation of motion:

\[ M(\alpha)\ddot{u}(\alpha,t) + C(\alpha)\dot{u}(\alpha,t) + K(\alpha)u(\alpha,t) = f(\alpha,t) \]  

(3.9)

where \( M(\alpha) \), \( C(\alpha) \) and \( K(\alpha) \) are the mass, damping and stiffness matrices of order m x m of the structure; \( u(\alpha,t) \) is the vector of nodal displacement and \( f(\alpha,t) \) is the forcing
everror. A dot over the variable denotes differentiation with respect to time. It is worth
highlighting the dependency of the mass, damping, stiffness matrices and the forcing
vector on the unknown parameter vector \( \alpha \) to consider the most general case. In order to
determine the response of the system described by equation (3.9) needs to be integrated,
the following time step method is implemented

\[ \frac{\partial u(\alpha,t)}{\partial t} = \dot{u}(\alpha,t) \]  

(3.10)

\[ \frac{\partial \dot{u}(\alpha,t)}{\partial t} = M^{-1}(\alpha)\tau f(t) - M^{-1}(\alpha)C(\alpha)\dot{u}(\alpha,t) - M^{-1}(\alpha)K(\alpha)u(\alpha,t) \]  

(3.11)

Where \( \tau \) is the incidence vector to apply the force at a given DOF. Introducing the vector
of state variables

\[ y(\alpha,t) = \begin{bmatrix} u(\alpha,t) \\ \dot{u}(\alpha,t) \end{bmatrix} \]  

(3.12)

Equations (3.10) and (3.11) can be cast in the following form

\[ \dot{y}(\alpha,t) = D(\alpha)y(\alpha,t) + V(\alpha)f(\alpha,t) \]  

(3.13)

where
\[ D(\alpha) = \begin{bmatrix} Z_{\alpha m} & I_{\alpha m} \\ -M^{-1}(\alpha)K(\alpha) & -M^{-1}(\alpha)C(\alpha) \end{bmatrix} \] (3.14)

\[ V(\alpha) = \begin{bmatrix} Z_{\alpha 1} \\ -M^{-1}(\alpha)\tau \end{bmatrix} \] (3.15)

where \( Z \) is a matrix of zeros and \( I \) is the identity matrix. A step-by-step procedure can be expressed as follow:

\[ y(\alpha_j, t)_{j=1} = \Theta(\alpha_j, \Delta t)y(\alpha_j, t)_{j=1} + \gamma_s(\alpha_j, \Delta t)V(\alpha_j)f(t)_{j=1} \]

\[ + \gamma_t(\alpha_j, \Delta t)V(\alpha_j)f(t)_{j=1} \] (3.16)

where

\[ \Theta(\alpha_j, \Delta t) = \exp(D(\alpha_j)\Delta t) \] (3.17)

\[ \gamma_s(\alpha_j, \Delta t) = \left( \Theta(\alpha_j, \Delta t) - \frac{L(\alpha_j, \Delta t)}{\Delta t} \right)D(\alpha_j)^{-1} \] (3.18)

\[ \gamma_t(\alpha_j, \Delta t) = \left( \frac{L(\alpha_j, \Delta t)}{\Delta t} - I_{2m \times 2m} \right)D(\alpha_j)^{-1} \] (3.19)

\[ L(\alpha_j, \Delta t) = \left( \Theta(\alpha_j, \Delta t) - I_{2m \times 2m} \right)D(\alpha_j)^{-1} \] (3.20)

with the operator \( \exp() \) representing the exponential matrix.

Both displacement and acceleration time histories can be extracted from the \( y \) and \( \dot{y} \) matrices respectively. The sensitivity matrix of the system can be derived in a similar fashion, consider the definition of the sensitivity as

\[ S(\alpha, t) = \frac{\dot{u}(\alpha, t)}{\dot{\alpha}} \] (3.21)

Differentiating with respect to time the following equations hold
\[
\dot{S}(a,t) = \frac{\partial}{\partial t} \left( \frac{\partial u(a,t)}{\partial a} \right) = \frac{\partial}{\partial a} \left( \frac{\partial u(a,t)}{\partial t} \right) = \frac{\partial \ddot{u}(a,t)}{\partial a}
\]

(3.22)

\[
\ddot{S}(a,t) = \frac{\partial}{\partial \eta t} \left( \frac{\partial u(a,t)}{\partial a} \right) = \frac{\partial}{\partial a} \left( \frac{\partial u(a,t)}{\partial \eta t} \right) = \frac{\partial \dddot{u}(a,t)}{\partial a}
\]

(3.23)

Differentiating equation (3.12) and (3.13) with respect to the parameter vector \(\alpha\)

\[
\frac{\partial y(a,t)}{\partial a} = \begin{bmatrix} \frac{\partial u(a,t)}{\partial a} \\ \frac{\partial \dot{u}(a,t)}{\partial a} \end{bmatrix} = \begin{bmatrix} S(a,t) \\ \dot{S}(a,t) \end{bmatrix} = S_y(a,t)
\]

(3.24)

\[
\frac{\partial \dot{y}(a,t)}{\partial a} = \begin{bmatrix} \frac{\partial \ddot{u}(a,t)}{\partial a} \\ \frac{\partial \dddot{u}(a,t)}{\partial a} \end{bmatrix} = \begin{bmatrix} \dot{S}(a,t) \\ \ddot{S}(a,t) \end{bmatrix} = \dot{S}_y(a,t)
\]

(3.25)

\[
\frac{\partial y(a,t)}{\partial a} = \frac{\partial D(a)}{\partial a} y(a,t) + D(a) \frac{\partial y(a,t)}{\partial a} + \frac{\partial V(a)}{\partial a} f(t)
\]

(3.26)

where

\[
\frac{\partial D(a)}{\partial a} = \begin{bmatrix} Z_{num} \\ -\frac{\partial M^{-1}(a)}{\partial a} K(a) + M^{-1}(a) \frac{\partial K(a)}{\partial a} \end{bmatrix} \quad \frac{\partial V(a)}{\partial a} = \begin{bmatrix} Z_{num} \\ -\frac{\partial M^{-1}(a)}{\partial a} C(a) + M^{-1}(a) \frac{\partial C(a)}{\partial a} \end{bmatrix}
\]

(3.27)

Equation (3.26) can be expressed in a more condensed fashion as follows:

\[
\dot{S}_y(a,t) = A(a) y(a,t) + D(a) S_y(a,t) + B(a) f(t)
\]

(3.29)
with
\[ A(a) = \frac{\partial D(a)}{\partial a}; B(a) = \frac{\partial V(a)}{\partial a} \]  \hspace{1cm} (3.30)

In a similar manner as in (3.16), the step-by-step procedure can be expressed as follow:
\[
S_j(\alpha_j, t)_{j=1} = \Phi(\alpha_j, \Delta t)S_j(\alpha_j, t)_{j=1} + \gamma_1(\alpha_j, \Delta t)A(\alpha_j)y(t)_{j=1} + B(\alpha_j)f(t) 
\]  \hspace{1cm} (3.31)

In the subsequent sections the step-by-step procedure described in equations (3.5) to (3.31) is suitably employed to minimise the penalty function \( J(a) \) formed of the difference between the selected measured response quantities stored in the vector \( x \) and the vector \( x(a) \) listing the data evaluated in the LODM, determined as in equation (3.32)
\[
x(a) = Py(a) 
\]  \hspace{1cm} (3.32)

where \( P \) is a matrix whose elements are either null or unitary so to extract from the vector \( y(a) \) the relevant response quantities. Minimising the penalty function \( J(a) \) in this way will allow the identification of the parameters dominating the behaviour of the structure coupled with the soil.

The identification procedure is considered to have terminated when parameter changes between iterations are inferior to 0.1%.

The outlined bounded variable time-domain identification procedure has the limitation of not ruling out, fully, the existence of local minima within the established boundaries of the problem. The use of different initial conditions could be a way of minimising the possibility of convergence into local minima, however, this will not guarantee that the solution has converged to the absolute minimum.
3.2 Structural model identification

Consider an isolated structure coupled with the soil as depicted in Figure 3.1. The dynamic response of this system is governed by equation (3.9), characterised by the mass, damping and stiffness matrices of the system. The analysis of coupled soil-structure systems can be computationally arduous since a significant portion of the soil domain needs to be considered. One way to reduce the computational effort is to use a combination of finite elements and boundary elements. These models are called FEM-BEM models. Despite the reduction in computational demands achieved by their, the analysis of several load cases or the performance of stochastic analyses still remains time consuming. The representation of coupled soil-structure systems by LODM arises as an alternative strategy to overcome this drawback. The use of LODM to represent coupled soil-structure systems has been proven to be a viable approach by several researchers (Wolf and Hall 1988; Kramer 1996; Harte et al. 2012; Damgaard et al. 2014a; Damgaard et al. 2014b). Nevertheless, the adequate selection of the model formulation, degrees of freedom (DOF), number of elements, and specially the interaction constants, is not trivial and depends on the type of problem being analysed. There are many formulations that can be used to represent the super-structure. In this study three models are used, namely, a two degree of freedom (TDOF) system, a shear type building and a beam element that takes into account shear effects (Timoshenko beam). Furthermore, the interaction spring parameters will be determined following time-domain identification techniques.

![Figure 3.1 Sketch of an isolated structure coupled with the soil.](image)

With the aim of identifying the importance of the chosen DOF, particularly the influence of rotations, a parametric study was carried out using a HODM made with of 3D solid elements. The analyses were conducted employing the finite element software ADINA. Two scenarios were considered, namely, 3D (x, y and z DOFs were free) and just x DOF case. It is important to highlight that when 3D 8 nodes quadrilateral elements are being
used, rotations of the elements are given by combinations of the x, y and z DOFs, and there are not rotational DOFs as such. Moreover, the coupled soil-structure system considered three different types of soil, whose properties are summarised in Table 3.1. The soil domain was 68 m by 68 m with a depth of 4 m. This represents 8 times the structure’s base dimension in each side to avoid boundary conditions effects. The structure has a square base with dimension b = 4 m, and four different aspect ratios (Ar) are considered {0.5; 0.75; 1.0; 2.0}, defined as in Alexander et al. (2013), Ar = h/b, where h is the building height. The structure’s foundation was embedded 1 m in the soil, which was kept constant for all cases. The structure’s material is defined by a modulus of elasticity E = 2.55 N/m², Poisson ratio μ = 0.2 and mass density ρ = 420 kg/m³.

Table 3.1 Soil stratum parameters

<table>
<thead>
<tr>
<th>Soil class (granular)</th>
<th>ρs [kg/m³]</th>
<th>μ</th>
<th>Vs [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>1300</td>
<td>0.3</td>
<td>150</td>
</tr>
<tr>
<td>Medium</td>
<td>1600</td>
<td>0.3</td>
<td>250</td>
</tr>
<tr>
<td>Dense</td>
<td>2000</td>
<td>0.35</td>
<td>350</td>
</tr>
</tbody>
</table>

Figure 3.2 to Figure 3.5 display the FEM mesh of the different models. All models were excited by the same input signal at the top of the structure. Care was taken to impose constraints on the top nodes of the structures as to avoid errors caused by the stress concentrations created by a point load.

Figure 3.2 FEM model coupled soil-structure system Ar = 0.5.
Figure 3.3 FEM model coupled soil-structure system $Ar = 0.75$.

Figure 3.4 FEM model coupled soil-structure system $Ar = 1.0$.

Figure 3.5 FEM model coupled soil-structure system $Ar = 2.0$. 
The model was excited by a force applied at the top of the structure in the \( x \) direction. The input signal was a sample of white noise with a bandwidth of 0-50 Hz, total duration of 1.0 s and time discretisation \( \Delta t \) of 1E-3 s. White noise was selected for its characteristic flat spectrum allowing to deliver the same amount of energy at all frequencies in the frequency range of interest. The numerical integration of the input time history was performed using modal superposition making sure that the frequency range of the solution was the same for both cases (that is, all DOFs and \( x \)-only DOFs). The modal damping was set constant for all modes and equal to 5%.

All four models were assessed for three types of soils and the results are presented in Figure 3.6 to Figure 3.8. In order to evaluate the degree of agreement between the two cases the Euclidean norm \( N \) defined in equation (3.33) of the vector \( \delta \) representing the difference between time histories (equation 3.31) was used. For ease of interpretation and comparison, the norm \( N \) was divided by the time history maximum for the case of all DOF.

\[
N = \sqrt{\sum_{k=1}^{n} |\delta_{k}|^2}
\]  

(3.33)

and

\[
\delta_{k} = z_{k}^{(i)} - w_{k}^{(i)}
\]

(3.34)

where \( k \) is the number of time steps; \( n \) is the total time of time steps; \( i \) is the measuring point in the model, \( z \) is the response of the system when all \( x, y \) and \( z \) DOF are considered while \( w \) is the response of the system when just \( x \) DOF is being considered.

Figure 3.6 present the displacement time histories of the system measured at the top and base of the structure for a soil stratum with a shear wave velocity \( V_s = 150 \) m/s. As can be seen from Figure 3.6, it appears that neglecting the contribution of the \( Z \) and \( Y \) DOF, and hence rotations, has an important effect for all aspect ratios but \( Ar = 0.75 \). In which case some peaks are slightly underestimated.
Figure 3.6 Parametric analysis results soil stratum $V_s = 150$ m/s.

In an analogous way Figure 3.7 present the displacement time histories of the system measured at the top and base of the structure for a soil stratum with a shear wave velocity $V_s = 250$ m/s. Results in Figure 3.7 suggest that for medium soils structures with aspect ratios greater than 0.5 must be analysed considering the effect of rotations, not doing so might incur in an underestimation of the systems response.
Finally, the results of the case in which the soil stratum shear wave velocity $V_s$ is equal to 350 m/s are depicted in Figure 3.8. It seems that the simplification of using just the X DOF gives satisfactory results for aspect ratios of up to 1 in the case of dense soil.
It is intuitive to deduce from the results of the parametric analysis depicted in Figure 3.6 to Figure 3.8 that as the soil stiffness increases, the influence of the rotations on the lateral displacements of the structure becomes less important for aspect ratios of up to 1. For aspect ratios greater than 1 the rotations seem to have an important impact on the lateral displacement of the structure regardless of the soil stratum stiffness.

The following subsections elaborate on the different models studied herein to represent the superstructure as well as the time-domain identification of the relevant parameters necessary to define such models. As the main objective of this study is the design of the
ViBa device to enhance the seismic resilience of urban environments simplified models of structures that are mainly governed by a shear type behaviour. Therefore, for this reason and to minimise the complexities in the identification procedure, all the LODM considered in this study neglect the in-plane rotational degrees of freedom.

3.2.1 Two-DOF system

The simplest way to represent the structure coupled with the soil with a Two-DOF (TDOF) system. Figure 3.9 displays a representative sketch of the system in which just horizontal translations are allowed. The soil-foundation system is represented by a lumped mass and a spring. This model is suitable to reproduce the response of structures that are mainly governed by their fundamental frequency or situations in which the expected input is likely to excite just the fundamental mode of the structure.

![Figure 3.9 Sketch of the TDOF model of coupled soil-structure system.](image)

The response of the system depicted in Figure 3.9 is governed by equation (3.9) and characterised by its mass, stiffness and damping matrices. The mass matrix $\mathbf{M}$ of the system is as follows,

$$
\mathbf{M} = \begin{pmatrix}
    m_{f,1} & 0 \\
    0 & m_i
\end{pmatrix}
$$

(3.35)

where $m_{f,1}$ is foundation mass and $m_i$ is the mass of the super structure.

The stiffness matrix of the system can be cast as follows
\[ K = \begin{pmatrix} k_{\text{st},i} + k_i & -k_i \\ -k_i & k_i \end{pmatrix} \]  

(3.36)

With the knowledge of both the mass \( M \) and stiffness \( K \) matrices, the damping matrix \( C \) can be found either considering modal or Rayleigh damping.

**Validation**

In order to validate the identification procedure, a model containing the same number of DOF was created in ADINA with an arbitrary set of parameters, namely, \( m_j = 4E3 \) kg, \( m = 2E3 \) kg, \( \zeta_n = 0.05 \) for all modes, \( k_i = 1E8 \) N/m and \( k_{\text{st},i} = 5E7 \) N/m. These parameters are representative of an ancient masonry building and were developed after the detailed building model studied in section 6.10. Figure 3.10 shows an illustration of the ADINA model. The system was exited at the top node of the structure with a force time history. The input signal was a sample of a white noise with a bandwidth of 0-50 Hz, total duration of 1.0 s and time discretisation \( \Delta t \) of 1E-3 s. At this point is worth mentioning that when analysing coupled soil-structure systems, site effects might have a significant impact on the systems response, particularly at foundation level. The foundation response will be influenced by both the dynamic properties of the structure and those of the soil. To include the site effects, the identification of a pseudo mass at the foundation level is proposed. This mass, combined with the interaction spring, will be aimed to reproduce ground response effects. It is important to highlight that the identified mass does not represent the real mass of the foundation. Instead, it has no physical meaning and its magnitude will depend on the interaction stiffness. Since the mass of the superstructure, in most cases, can be estimated with a fair degree of accuracy, it is considered to be a known quantity and it is not part of the variables to be identified.

The identification procedure was carried out with initial values of \( k_i = 5E7 \) N/m and \( k_{\text{st},i} = 2.5E7 \) N/m and \( m_j = 2E3 \) kg using as target vector \( x_m \) the acceleration time history at both top and base of the building. The variable boundaries were set to a minimum of 0.01 kg and a maximum of 1e5 kg for the mass parameters and a minimum of 1E5 N/m and a maximum of 1E12 N/m for the stiffness parameters to constraint the procedure into yielding realistic solutions. From this point onwards in this section these boundaries were used for all the cases analysed. Figure 3.11 presents the target time history (blue markers) as well as the first and final iteration of the identification procedure (yellow and red line respectively). As it is observed from the figure there is a satisfactory degree of agreement.
between the target and the response of the system at the final iteration. This is further evidenced by observing the improvement in the final norm \( N_f \) with respect to the initial norm \( N_i \). The norm has been calculated as in equation (3.33) and divided by the maximum of the target time history. Furthermore, convergence plots are displayed in Figure 3.12 showing that the procedure is able to arrive to the correct values initially set as targets. Hence, the formulation of the system as well as the time integration technique seem to be correct.

Figure 3.10 ADINA model of the TDOF system structure coupled with the soil.

Figure 3.11 Acceleration time histories of both top and base of the TDOF, model validation.
Figure 3.12 Convergence of the TDOF model parameters.

Application

Consider a more realistic scenario, the FE model depicted in Figure 3.13 is a numerical representation of the small scale soil-structure model built for the LOSSVAR project (Zentner I. 2016). Detailed information on the model are given in Chapter 5. The soil stratum possess a mass density $\rho = 1680$ kg/m$^3$, a shear wave velocity of $V_s = 630$ m/s, and a Poisson ratio of $\mu = 0.46$. The structure has a mass density $\rho = 420$ kg/m$^3$, a modulus of elasticity $E = 2.55E8$ and a Poisson ratio of $\mu = 0.2$. This model has the particularity that poses an aspect ratio $Ar = 0.6$ in the X direction. According to the results presented in the previous section this structure can be represented by a model that only takes into account horizontal translations without incurring in significant oversimplification. The building is subject to a time history force excitation applied at the top in the X direction (see Figure 3.13). The input signal comprises a sample of clipped white noise with a total duration of 0.16s, time discretisation interval of 1.95E-5 s and a frequency bandwidth of 800-5120 Hz. It is important mentioning that this signal represents the force time history that has been recorded during the experiments on the physical model. In order to identify the lateral stiffness of the structure, the coupling spring stiffness and the foundation mass, the vector $x_m$ is composed of the lateral acceleration time history at the top and base of the structure. Finally, the analysis was carried out considering all, X, Y and Z DOF. It is important to mention that the results obtained in this HOFEM analysis will be used for the identification of the two remaining models, namely the shear type building and the beam element that takes into account shear effects.
Figure 3.13 HOFEM of soil-structure system of the small scale LOSSVAR model. Acceleration time histories of both HODM and LODEM are presented in Figure 3.14 measured at the top and base of the models. While convergence plots are displayed in Figure 3.15. From Figure 3.14 it is noticeable the improvement from the initial conditions as well as the fair representation of the HOFEM response. This is further observed by the decrease in the final norm Nf with respect to the initial norm Ni. The norm has been calculated as in equation (3.33) and divided by the maximum of the target time history.

Figure 3.14 Acceleration time histories of both top and base of the building. Comparison between the HOFEM and LODEM using TDOF to model the super structure.
3.2.2 Shear type building

An alternative, more general way to represent the super structure is through a shear type building. This model combines a simple formulation with the ability to reproduce a more realistic structural behaviour with a wider range of applications when compared with the TDOF system. It can accommodate scenarios in which the response of the structure is not dominated by a single frequency or cases in which the input signal is broadband instead of narrowband.

When a shear type building formulation suits the problem, one can start from the diagram depicted in Figure 3.16 to define the mass $\mathbf{M}$ and stiffness $\mathbf{K}$ matrices. In this particular case the mass matrix $\mathbf{M}$ is diagonal, with its elements being the storey masses, as indicated in equation (3.37):

$$
\mathbf{M} = \begin{pmatrix} 
m_{f,1} & 0 & 0 & 0 \\
0 & m_2 & 0 & 0 \\
0 & 0 & \ddots & 0 \\
0 & 0 & 0 & m_n
\end{pmatrix}
$$

(3.37)

where $m_n$ is equal to the storey mass with $n = (1, 2, 3\ldots)$. Similarly, the stiffness matrix $\mathbf{K}$ is a tridiagonal matrix defined as follows:
where \( k_i \) is the first inter-storey stiffness, \( k_{sui,j} \) is the soil-structure interaction stiffness and \( j = (1,2,3 \ldots) \) is the storey number.

\[
\mathbf{K} = \begin{pmatrix}
    k_i + k_{sui,1} & -k_i & 0 & 0 & 0 \\
    -k_i & k_i + k_2 & -k_2 & 0 & 0 \\
    0 & -k_2 & \ddots & \ddots & 0 \\
    0 & 0 & \ddots & k_j + k_{j+1} & -k_{j+1} \\
    0 & 0 & 0 & -k_{j+1} & \ddots & -k_n \\
    0 & 0 & 0 & 0 & -k_n & k_n
\end{pmatrix}
\] (3.38)

Figure 3.16 Sketch of the simplified model of shear type building coupled with the soil.

Validation

Following a similar approach as in the case of the TDOF, the shear type building model was evaluated by comparing it with an identical model created in ADINA and depicted in Figure 3.17. The geometrical properties are those of the LOSSVAR structure. The lateral stiffness is defines as in equation (3.39) and characterised by the shear modulus \( G = 1E8 \text{ N/m}^2 \). The SSI is taken into account through a linear spring \( k_{sui,1} \), with a stiffness of \( 1E8 \text{ N/m} \). The inter-storey mass was set equal to \( 5E2 \text{ kg} \) and the foundation mass \( m_{f1} = 4E3 \text{ kg} \):

\[
k_k = \frac{GA}{l}
\] (3.39)
where $k_s$ is the lateral inter-storey stiffness; $G$ is the shear modulus; $A$ is the cross sectional area and $l$ is the inter-storey height.

The identification procedure was carried out with initial values of $G = 6 \times 10^7$ N/m² and $k_{s{2,1}} = 6 \times 10^7$ N/m and $m_{f,1} = 2 \times 10^3$ kg using as target vector $\mathbf{x}_m$ the acceleration time history at both top and base of the building. Figure 3.18 presents the target time history (blue markers) as well as the first and final iteration of the identification procedure (yellow and red line respectively). As it is observed from the figure, there is a satisfactory degree of agreement between the target and the response of the system at the final iteration. This is further evidenced by observing the improvement in the final norm $N_f$ with respect to the initial norm $N_i$. The norm has been calculated using equation (3.33) and divided by the maximum of the target time history. Furthermore, convergence plots are displayed in Figure 3.19 showing that the procedure is able to arrive to the correct values initially set as targets, except for the shear modulus $G$ value in which there is a slight discrepancy between the ADINA and the model proposed herein. This discrepancy might be due to the use of different integration approaches for the equation of motion. Nonetheless, the differences are negligible, and therefore it can be said that the formulation of the LODM is in agreement with that of the commercial software ADINA.

![Figure 3.17 ADINA model of the shear type building superstructure and soil-foundation system.](image-url)
Figure 3.18 Acceleration time histories of both top and base of the shear type building super structure, validation.

Figure 3.19 Convergence of the parameter shear type building model parameters.

Application

In this subsection the output data of the analysis performed on the HOFEM depicted in Figure 3.13 of section 3.2.1 is used to identify the parameters of the shear type building model. The vector $x_m$ is composed of the lateral acceleration time history at the top and base of the structure. Acceleration time histories of both HODM and LODM are presented in Figure 3.20 measured at the top and base of the models. While convergence plots are displayed in Figure 3.21. From Figure 3.20. From these plots it is noticeable the improvement from the initial conditions as well as the fair representation of the HOFEM response. This is further observed by the decrease in the final norm $N_f$ with respect to the initial norm $N_i$. The norm has been calculated as in equation (3.33) and divided by the maximum of the target time history.
3.2.3 Beam element model

An additional approach to represent the superstructure is by using beam elements able to take into account shear effects. It can be said that in comparison with the two previous alternatives this model is able to capture more complex structural behaviour. However, the formulation is more complicated than the previous two, and for certain structures there are not significant advantages in using beam elements as a way to model the superstructure. Nevertheless, if the effect of rotations is important, this is the only model that is able to account for them.
For simplicity just 2D scenarios are considered. The beam element is defined as a Timoshenko beam, able to account for the effect of shear deformations. Both the stiffness matrix \( K \) and mass matrix \( M \) have been taken from Przemieniecki (1985).

Starting from the diagram depicted in Figure 3.22 the consistent mass matrix of the system is as follows:

\[
M = \rho A l
\]

where \( \rho \) is the mass density of the element; \( A \) is the cross sectional area of the element; \( l \) is the length of the element and \( I \) is the moment of inertia.

The stiffness matrix \( K \) can be cast in the following form:

\[
K = \begin{pmatrix}
\frac{EA}{l} & 0 & 0 & -\frac{EA}{l} & 0 & 0 \\
0 & \frac{12EI}{l^3(1+\Phi)} & \frac{6EI}{l^2(1+\Phi)} & 0 & \frac{12EI}{l^3(1+\Phi)} & \frac{6EI}{l^2(1+\Phi)} \\
0 & \frac{6EI}{l^2(1+\Phi)} & \frac{(4+\Phi)EI}{l(1+\Phi)} & 0 & \frac{6EI}{l^2(1+\Phi)} & \frac{(2-\Phi)EI}{l(1+\Phi)} \\
-\frac{EA}{l} & 0 & 0 & \frac{EA}{l} & 0 & 0 \\
0 & \frac{12EI}{l^3(1+\Phi)} & \frac{6EI}{l^2(1+\Phi)} & 0 & \frac{12EI}{l^3(1+\Phi)} & \frac{6EI}{l^2(1+\Phi)} \\
0 & \frac{6EI}{l^2(1+\Phi)} & \frac{(2-\Phi)EI}{l(1+\Phi)} & 0 & \frac{6EI}{l^2(1+\Phi)} & \frac{(4+\Phi)EI}{l(1+\Phi)}
\end{pmatrix}
\]

where \( E \) is the modulus of elasticity and \( \Phi \) is defined as follows:

\[
\Phi = \frac{12EI}{GA_l^2}
\]

where \( G \) is the shear modulus, and \( A_s \) is the modified shear area equal to 5/6. The full system consist of a succession of this elements and the coupling with the soil is achieved by adding the corresponding stiffness, rotation and translational, to the respective DOF.
of the first element. In this study, the in-plane rotational DOFs have not been considered nor static condensation of the DOF has been applied either. The sketch presented in Figure 3.22 corresponds to the assembled system limited to horizontal translations only.

Figure 3.22 Sketch of the simplified model of beam elements coupled with the soil.

Validation

In a similar manner as it was shown in the case of the TDOF system and the shear type building, the formulation will be validated by comparing it with an equivalent model made in the software ADINA and presented in Figure 3.23. The geometrical properties of the model are those of the LOSSVAR structure, the modulus of elasticity $E = 1E9$ N/m$^2$, the SSI is taken into account through a linear spring with a stiffness of $k_{SSI,1} = 1E9$ N/m. The mass density $\rho$ was set equal to $2E6$ kg/m$^3$ and the foundation mass $m_{f,1} = 4E3$ kg.

The identification procedure was carried out considering just horizontal translations with initial values of $E = 7E8$ N/m$^2$, $k_{SSI,1} = 4E8$ N/m and $m_{f,1} = 2E3$ kg using as target vector the acceleration time history at both top and base of the building. Figure 3.24 presents the target time history (blue markers) as well as the first and final iteration of the identification procedure (yellow and red line respectively). As it is observed from the figure there is a satisfactory degree of agreement between the target and the response of the system at the final iteration. This is further evidenced by observing the improvement in the final norm $N_f$ with respect to the initial norm $N_i$. The norm has been calculated using equation (3.33) and divided by the maximum of the target time history.
Furthermore, convergence plots are displayed in Figure 3.25 showing that the procedure is able to arrive to the correct values initially set as targets. Therefore, the formulation of the system as well as the time integration technique seem to be correct. Nonetheless, a greater number of iterations, compared with the two previous cases, as well as better initial conditions are needed for the solution to converge.

Figure 3.23 ADINA model of the beam element superstructure model and soil-foundation system.

Figure 3.24 Acceleration time histories of both top and base of the beam element superstructure, validation.
Application

In this subsection the output data of the analysis performed on the HOFEM depicted in Figure 3.13 of section 3.2.1 is used to identify the parameters of the beam element model. The vector is composed of the lateral acceleration time history at the top and base of the structure. Acceleration time histories of both HODM and LODM are presented in Figure 3.26 measured at the top and base of the models. While convergence plots are displayed in Figure 3.27. From Figure 3.26 it can be seen the improvement from the initial conditions as well as an acceptable representation of the HOFEM response. Nevertheless, the response of the top of the structure is not captured with a satisfactory degree of accuracy. While there is a good improvement in the norm for the response of the base of the building, this does not seem to be the case for the top of the structure.

Comparison between the HOFEM and LODM using beam elements to model the superstructure.
Figure 3.27 Convergence of the model parameters. Results of the LODM using beam elements to represent the numerical data.

3.3 Summary

Finally, when comparing the three models, it can be said that both the TDOF system and the shear building are able to provide a satisfactory representation of the HOFEM model. The former being closer to the target response than the latter. Nevertheless, the shear type building model seems to be less dependent on the initial conditions providing a norm average perceptual improvement of 750% which is twice the one reported for the TDOF model.

Furthermore, the beam elements model, is the least satisfactory of the three models, in terms of both complexity of the formulation and quality of the HODM response representation. However, this is the only model that is able to account for the potential influence of rotations. Thus, it can be concluded that while the TDOF model provides a better representation of the HODM response the shear type building mode is more robust and less dependent on the quality of the initial conditions.

In this chapter, a methodology for using a bounded variable time-domain least-square identification procedure to define LODMs able to account for SSI has been introduced. The LODMs are representative of more computationally expensive HODMs. This represents an alternative strategy to account for the model’s complex behaviour while significantly reducing computational expenses without significant loss of accuracy.
4 Structure soil structure interaction identification

Structures are rarely found in isolation. On the contrary, they are often surrounded by neighbouring buildings, especially in urban environments, where the gap between buildings can be as small as a few meters. One might wonder if the presence of nearby structures could influence the dynamic behaviour of adjacent buildings. There has been a significant amount of research aimed not just to confirm the existence of the interaction between adjacent structures, but to somehow quantify it and determine in which cases it might be beneficial or detrimental. In this endeavour, researchers have tackle this problem following analytical, numerical and experimental approaches. A detailed treatment of the progress of SSSI is given in Lou et al. (2011).

Most of the challenges encountered during the study of SSI remain when analysing SSSI problems. One of the difficulties is the computational demands as a consequence of using HOFM to solve this type of problems. The use of LODM arises as an alternative to overcome the setback of high computational demands, as they are able to capture the main effects of SSSI while keeping computational cost low. The use of LODM has been proven to be a valid approach when studying SSSI (Kobori et al. 1973; Mulliken and Karabalis 1998; Behnamfar and Sugimura 1999; Alexander et al. 2012; Naserkhaki and Pourmohammad 2012).

This Chapter is devoted to the application of the bounded variable time-domain least-square identification procedure to identify the parameters of a series of LODM able to account for both SSI and SSSI effects. To the author’s knowledge, time-domain identification techniques have not been used before in the study of SSSI. Furthermore, a novel two-stage approach is introduced to breach the problem of disproportionate sensitivities. In addition, a novel discrete model is proposed to account for wave propagation effects. Finally, both parameter normalisation and weighting matrices are successfully employed to improve the numerical conditioning of the problems.
4.1 SSSI identification: problem position

Following a similar approach as in the case of a single isolated building, consider a two-building system, in which, each building is individually coupled with the soil (SSI) and coupled with each other through the soil (SSSI). A general sketch of this system is depicted in Figure 4.1. In agreement with (Kobori et al. 1973; Alexander et al. 2013; Cacciola et al. 2015; Cacciola and Tombari 2015), this system can be represented by the simplified lumped parameter model presented in Figure 4.2, where the coupling through the soil between structures is accounted for by means of a linear spring.

![Figure 4.1 Sketch of two adjacent buildings coupled with and through the soil.](image1)

![Figure 4.2 Simplified lumped parameter model of two adjacent buildings coupled with and through the soil.](image2)

The dynamics governing equation of the coupled system takes the form:

\[ M(a)\ddot{u}(a,t) + C(a)\dot{u}(a,t) + K(a)u(a,t) = f(t) \]  

(4.1)
where $M(\alpha)$, $C(\alpha)$ and $K(\alpha)$ are the global mass, viscous damping and stiffness matrix, respectively; $\ddot{u}(a,t)$, $\dot{u}(a,t)$ and $u(a,t)$ are respectively the acceleration, velocity and displacement vector. The dependence of the dynamic behaviour of the system on the vector of unknown parameters $\alpha$ is stressed. The matrices of the global system are partitioned in the sub-matrices defined for the individual structures coupled with the soil, with the off-diagonal sub-matrices related to the dynamic coupling between structures. Therefore, the global mass matrix $M(\alpha)$ is as follows:

$$M(\alpha) = \begin{bmatrix}
M_1(\alpha) & 0 & \ldots & 0 \\
0 & M_2(\alpha) & \ldots & 0 \\
\vdots & \vdots & \ddots & \vdots \\
0 & 0 & 0 & M_p(\alpha)
\end{bmatrix} \quad (4.2)$$

in which the $i$th sub-block includes the mass of the $i$th structure given by

$$M_i(\alpha) = \begin{bmatrix}
m_i(\alpha) & 0 & \ldots & 0 \\
0 & m_j(\alpha) & \ldots & 0 \\
\vdots & \vdots & \ddots & \vdots \\
0 & 0 & 0 & m_n(\alpha)
\end{bmatrix} \quad (4.3)$$

The global stiffness matrix $K(\alpha)$ are block-matrices partitioned in the following form:

$$K(\alpha) = \begin{bmatrix}
K_{11}(\alpha) & K_{12}(\alpha) & \ldots & K_{1p}(\alpha) \\
K_{21}(\alpha) & K_{22}(\alpha) & \ldots & K_{2p}(\alpha) \\
\vdots & \vdots & \ddots & \vdots \\
K_{p1}(\alpha) & K_{p2}(\alpha) & \ldots & K_{pp}(\alpha)
\end{bmatrix} \quad (4.4)$$

for the stiffness matrix, the off-diagonal sub-matrices $K_{ij}$ ($i, j = 1, \ldots, p$) are related to the dynamic coupling between the $i$th and the $j$th structures. With the knowledge of both the mass $M(\alpha)$ and stiffness $K(\alpha)$ matrices, the damping matrix $C(\alpha)$ can be found either considering modal or Rayleigh damping. The foundation masses as well as all the stiffness parameters are listed in the vector $\alpha$ and represents the unknowns of the problem. The mass of the superstructure are assumed to be known quantities, since in most cases they can be determined with a fair degree of accuracy. In this regard, the bounded variable time domain identification procedure described in section 3.1 can be used to minimise the penalty function $J(\alpha)$ defined as in equation (3.6).
4.2 Proposed two stage time-domain identification procedure

Recalling the definition of the sensitivity matrix presented in equation (3.2), it can be seen that it represents the derivative of the response with respect to the unknown parameters vector \( \alpha \). Depending on the type of model, excitation point, measuring points and type of excitation, the influence of a given parameter could be more important than others. According to Friswell and Mottershead (1995), this can potentially cause the sensitivity matrix to be badly scaled, and therefore the solution of the Moore-Penrose pseudo inverse (equation 3.5) is bound to have numerical problems. Friswell and Mottershead (1995) have suggested to use scaling factors when dealing with parameters that are different in magnitude, and the use of weighting matrices to incorporate additional information to problems characterised by rank deficiency or noisy data. However, there are some cases in which the numerical conditioning cannot be improved by the use of the aforementioned approaches. Hence, an alternative is proposed herein. It consists of solving the bounded variable time-domain least-square identification procedure in two stages. In the first stage, generally parameters that have greater impact and comparable sensitivities will be sought; while the remaining parameters are found in the second stage. To this aim, the vector of unknown parameters \( \alpha \) is redefined in block form as follows:

\[
\alpha_{i,j} = \begin{bmatrix} \beta_i \\ \chi_j \end{bmatrix}
\]  

(4.5)

where \( \beta_i \) is the vector of unknown parameters updated in the first stage, while \( \chi_j \) is the vector of unknown parameters updated in the second stage. It is important to mention that during the first stage the parameters stored in \( \chi_j \) remain constant, while during the second stage the parameters stored in \( \beta_i \) are not updated. For the first stage of the procedure equation (3.5) can be recast in the following form:

\[
\begin{bmatrix} \beta_{i+1,l} \\ \beta_{i+1,\ell} \end{bmatrix} = \begin{bmatrix} \beta_i \\ \beta_l \end{bmatrix} + \left[ \begin{bmatrix} S_{\beta} & T_{S_{\beta}} \end{bmatrix} \right]^{-1} \begin{bmatrix} S_{\beta} \\ T_{S_{\beta}} \end{bmatrix} \left( x_{m,\beta} - x_{l,\beta} \right)
\]  

(4.6)

while for the second stage is conveniently expressed in equation (4.7)

\[
\begin{bmatrix} \chi_{j+1,l} \\ \chi_{j+1,\ell} \end{bmatrix} = \begin{bmatrix} \chi_j \\ \chi_l \end{bmatrix} + \left[ \begin{bmatrix} S_{\chi} & T_{S_{\chi}} \end{bmatrix} \right]^{-1} \begin{bmatrix} S_{\chi} \\ T_{S_{\chi}} \end{bmatrix} \left( x_{m,\chi} - x_{l,\chi} \right)
\]  

(4.7)
where the difference between the vectors $\mathbf{x}_{n,\beta}$ and $\mathbf{x}_{n,\chi}$ listing the measured data is stressed to suit the most general case in which the measuring points are different for each stage.

The structure to implement the two-stage identification procedure using equations (4.6) and (4.7) is as follows:

1. Determine the measuring points on the FE/experimental model.
2. Collection of the data that comprises the vectors $\mathbf{x}_{n,\beta}$ and $\mathbf{x}_{n,\chi}$ from the points selected in (1).
3. Perform a sensitivity analysis to identify parameters that have greater or lesser impact on the model’s dynamic behaviour.
4. Assemble the vector $\mathbf{\beta}_o$ containing the initial values of the parameters that have greater impact as well as the vector $\mathbf{\chi}_o$ containing the initial values of the parameters that have lesser impact on the response of the system, and list them in the vector $\mathbf{a}_{i,j}$. It has been observed from the analysis conducted in the present study, that the parameters governing SSSI systems tend to be easily classified in two groups, namely, high and low incidence in the response. To determine the parameters with greater and lesser influence in the response, a threshold of one order of magnitude was established. Those parameters with a sensitivity at least ten times smaller than the parameter with the maximum sensitivity were listed in the vector $\mathbf{\chi}_o$ while the remaining parameters were listed in the vector $\mathbf{\beta}_o$.
5. Numerical evaluation of the vector $\mathbf{x}_i(\mathbf{\beta}_i)$ along with the pertinent sensitivity matrix $\mathbf{S}_{\mathbf{\beta}_i}$.
6. Model updating, evaluation of vector $\mathbf{\beta}_i$ using equation (4.6) and calculate $\mathbf{x}_i(\mathbf{\beta}_i)$ along with the sensitivity matrix $\mathbf{S}_{\mathbf{\beta}_i}$.
7. Repeat step (6) until convergence is reached.
8. Update the vector $\mathbf{a}_{i,j}$ with the values of $\mathbf{\beta}_i$.
9. Numerical evaluation of the vector $\mathbf{x}_j(\mathbf{\chi}_j)$ along with the pertinent sensitivity matrix $\mathbf{S}_{\mathbf{\chi}_j}$.
10. Model updating, evaluation of vector $\mathbf{\chi}_j$ using equation (4.6) and calculate $\mathbf{x}_j(\mathbf{\chi}_j)$ along with the sensitivity matrix $\mathbf{S}_{\mathbf{\chi}_j}$.
11. Repeat step (10) until convergence is reached.
12. Update the vector $\mathbf{a}_{i,j}$ with the values of $\mathbf{\chi}_j$.
13. Repeat steps (5) to (12) until convergence is reached.
Implementing the proposed two-stage bounded-variable time-domain least-square identification procedure adds versatility to the solution, allowing its application to be expanded to a greater range of problems that cannot be solved otherwise.

4.3 Weighted constrained optimisation problem

In many practical cases, particularly when complex formulations are used to describe the system, the estimation of the different parameters might be done with different degrees of accuracy. For instance, if both the elastic modulus of a beam element and the spring representing the soil-structure coupling are being updated the estimation of the discrete spring will be more accurate than that of the modulus of elasticity, simply because the stiffness provided by the first element of the beam depends on more parameters than just the modulus of elasticity. Hence, the parameter change per iteration is not proportional. Moreover, the quality of the measured data might not be equally rich in information, depending on the measured points and type of excitation. Consider the lumped parameter model of two adjacent structures coupled at foundation level through a linear spring depicted in Figure 4.2. Assuming the system is excited at the top of the first structure by a flat spectrum input force time history, and measurements are taken at the top and base of both structures. While the first structure is being excited in a wide range of frequencies the excitation arriving at the second structure has been modified in amplitude, phase and frequency content, hence the data associated with the first structure is of better quality than that recorded in the second structure. One might want to include this effects in the constrained optimisation problem. An alternative proposed by Friswell and Mottershead (1995) is to explicitly weight the parameter changes and the error in the measurement directly in the penalty function as follows:

\[
J(\delta \mathbf{a}) = \mathbf{e}^T \mathbf{W}_{ee} \mathbf{e} + \delta \mathbf{a}^T \mathbf{W}_{oo} \delta \mathbf{a}
\]  

(4.8)

where \( J(\delta \mathbf{a}) \) is the penalty function, \( \mathbf{e} \) is the error in the measurements equal to \( \delta \mathbf{x} - \mathbf{S}_a \delta \mathbf{a} \), \( \mathbf{W}_{ee} \) is the measurements weighing matrix, \( \delta \mathbf{a} \) is the change in parameters and \( \mathbf{W}_{oo} \) is the parameters weighting matrix. Substituting \( \mathbf{e} = \delta \mathbf{x} - \mathbf{S}_a \delta \mathbf{a} \) gives,

\[
J(\delta \mathbf{a}) = \delta \mathbf{x}^T \mathbf{W}_{ee} \delta \mathbf{x} - \delta \mathbf{x}^T \mathbf{W}_{ee} \mathbf{S}_a \delta \mathbf{a} - \delta \mathbf{a}^T \mathbf{S}_a^T \mathbf{W}_{ee} \delta \mathbf{x} + \delta \mathbf{a}^T \mathbf{S}_a^T \mathbf{W}_{ee} \mathbf{S}_a \delta \mathbf{a} + \delta \mathbf{a}^T \mathbf{W}_{oo} \delta \mathbf{a}
\]  

(4.9)
Since $\delta x^T W_{oo} S_a \delta a$ and $\delta a^T S_a^T W_{oo} \delta x$ are scalar quantities, the following simplifications hold

$$J(\delta a) = \delta x^T W_{oo} \delta x - 2\delta a^T S_a^T W_{oo} \delta x + \delta a^T \left[ S_a^T W_{oo} S_a + W_{oo} \right] \delta a$$

(4.10)

Minimising the penalty function with respect to the parameter change $\delta a$ gives

$$\delta a = \left[ S_a^T W_{oo} S_a + W_{oo} \right]^{-1} S_a^T W_{oo} \delta x$$

(4.11)

One of the problems caused by badly scaled sensitivity matrices is the disproportionate parameter changes per iteration which occasionally leads to numerical problems in the identification procedure. To avoid this issue the weighting matrix $W_{oo}$ can be determined as follow:

$$\left[ S_a^T W_{oo} S_a + W_{oo} \right]^{-1} S_a^T W_{oo} = \eta \left[ S_a^T S_a \right]^{-1} S_a^T$$

(4.12)

where $\eta$ is a factor that regulates the parameter change per iteration, solving equation (4.12) for $W_{oo}$ gives

$$W_{oo} = \left\{ \eta \left[ S_a^T S_a \right]^{-1} S_a^T (W_{oo} S_a) \left( \left[ S_a^T W_{oo} \right] (W_{oo} S_a) \right)^{-1} \right\}^{-1}$$

(4.13)

To solve this equation, the measurements error matrix $W_{ee}$ can be defined as the identity matrix of the same order of the larger dimension of the sensitivity matrix. This approach is used in this Chapter to improve the numerical conditioning of the problem assuming $W_{ee}$ to be the identity matrix and $W_{oo}$ calculated as in equation (4.13). Values of the $\eta$ factor equal to 1, 1E-1, 1E-2 and 1E-3 were tested. It was found that values of 1 and 1E-1 would not improve the numerical conditioning. A $\eta$ equal to 1E-3 would improve the numerical conditioning but requires a large number of iterations to converge. Finally, it was concluded that a value of $\eta$ equal to 1E-2 provided an improvement of the numerical conditioning without significantly increasing the amount of iterations required to reach a solution.
4.4 Proposed novel simplified discrete model

One of the complications that arises in the development of a LODM able to account for the cross interactions between two adjacent buildings is capturing the physical phenomenon that takes place through the soil. That is to say, wave propagation effects. According to Kramer (1996), unlike structures, geological materials must be considered as continua, and their dynamic response must be treated in the context of wave propagation. In a LODM two nodes are connected through a discrete source of stiffness; for instance, a spring, which is unable to faithfully mimic the reality. Consider the two TDOF model depicted in Figure 4.3. If a force acts on the foundation of the first building, the effect will be immediately transmitted to the foundation of the second building. However, in a real scenario the arrival of the stress waves to the second foundation will depend on both the distance between structures and the compressive wave velocity of the soil \( v_p \). Recalling the one-dimensional longitudinal wave equation for a constrained rod presented in equation (4.14)

\[
\frac{\partial^2 u}{\partial t^2} = v_p^2 \frac{\partial^2 u}{\partial x^2}
\]  

(4.14)

with

\[
v_p = \frac{E}{\rho}
\]  

(4.15)

where \( E \) is the elastic modulus and \( \rho \) the mass density. The elastic modulus \( E \) is associated with the soil, stiffness while the mass density \( \rho \) is related to its mass properties. An analogy can be made with the discrete model depicted in Figure 4.3, where the soil is solely accounted for through a discrete source of stiffness \( k_{\text{soil}} \), leading to a virtually infinite compressive wave velocity, since in accordance with equation (4.15) there is no representation of the mass term. Therefore, in order to account for wave propagation effects in a simplified way, a novel LODM is proposed herein, able to account for wave propagation effects. This model consider the addition of a DOF with a mass located in between the two structures (see Figure 4.4). The inclusion of this mass indirectly account for the distance between structures, by affecting the velocity but also by decreasing the amplitude of the particle acceleration as this mass increases.
To illustrate the modifications introduced by the newly developed model proposed in this study, Figure 4.5 displays the comparison of the dynamic response of the simple model and the proposed model with the inclusion of the mass in between buildings. Same arbitrary properties where given to both systems, setting all the stiffness to $1E8$ N/m, mass of the superstructure equal to $2E3$ kg, mass of the foundation equal to $4E3$ kg, and in the case of the proposed model, the intermediate mass was equal to the foundation mass. Both
systems were excited by a unitary impulse input at the foundation of the first structure (B1). The input consisted of a unitary single spike at time 0.015 s, the total duration of the input was 1.5 s and the time discretization was 1.5E-3 s.

![Impulse response graph](image1)

Figure 4.5 Comparison of the impulse response of the simple model and the proposed model able to account for wave propagation effects.

![Impulse response graph](image2)

Figure 4.6 Effect of the additional mass magnitude in the response at the foundation of the second building.

It is observed from Figure 4.5 that at the beginning of the response there is very little difference between the responses at the base of both buildings when the simple model is used. This points towards an infinite compressive wave velocity of the soil stratum that allows the stress waves to arrive at exactly the same instant on both foundations. Contrary to this, the response of the proposed model shows a delay between the response signals
as well as a difference in amplitude. The effect of the intermediate mass between structures if further explored in Figure 4.6, where the value of the intermediate mass is gradually increased proportionally to the foundation mass. The two phenomena that the proposed model is able to account for, namely delay in the arrival of the stress waves and amplitude reduction, are evidenced in this plot. As the mass increases in magnitude, the apparent compressive wave velocity should decrease, leading to a greater delay in the arrival of the stress waves, this could be representative of a greater distance between structures. Furthermore, the increase in the mass translates into a decrease of the acceleration amplitude. This also could be considered a consequence of a greater distance between structures.

In the context of parameter identification, the use of the model proposed herein is of particular importance when impulse-like input signals are being used. Contrary to continuous signals, where the delay can be masked as a phase change. Nevertheless, the appropriate consideration of the SSSI effects requires the inclusion of wave propagation effects.

One of the limitations of the proposed model is that it does not account for the higher damping of the soil materials with respect to the structure’s materials. In fact, all the analyses presented in this research have been conducted assuming constant damping ratios for all the model materials. It is acknowledged that this simplification could have an impact in the models response, as in general geological materials have higher damping capacities with respect to commonly used structural materials.

From this point onwards, just the improved model will be used. It is worth mentioning that regardless of the superstructure’s formulation the inclusion of the mass in between buildings has a similar effect and will also be considered for both shear type building and beam elements superstructure models.
4.5 SSSI identification

The following subsections elaborate on the implementation of the bounded-variable time-domain identification procedure to define a series of simplified models able to account simultaneously for the SSI and SSSI phenomena between adjacent structures. To this aim, three different simplified models are studied, namely, a two TDOF systems, two uniform shear type buildings and two Timoshenko beams. The variable boundaries were set for all cases to a minimum of 0.01 kg and a maximum of 1e5 kg for the mass parameters and a minimum of 1E5 N/m and a maximum of 1E12 N/m for the stiffness parameters to constraint the procedure into yielding realistic solutions.

4.5.1 Two TDOF systems

The simplest way to represent both super structures is by means of a two TDOF systems, Figure 4.4 displays a representative sketch of the system in which just horizontal translations are allowed. This model is suitable to reproduce the response of structures that are mainly governed by their fundamental frequency, or situations in which the expected input is likely to excite just the fundamental mode of the structures.

At this stage it is important to highlight the particular issues that are faced when dealing with two structures simultaneously. As it has already been pointed out, the identification procedures hinges on the premise that the variables being sought have a similar effect on the response of the system, if any set of variables has relatively different effect with respect to the others, the problem becomes ill-conditioned. This issue becomes particularly relevant when dealing with systems in which the soil is significantly softer or stiffer than the structure, since the response will be dominated by either the soil or the structures stiffness, making it more sensitive to changes in these quantities. Moreover, the type of excitation also has an impact. If the excitation is applied at only one of the two structures, as it has been done in this study, the response of the system will be more sensitive to parameters associated to the first structure compared to those of the second one. When excited by a ground motion, both extreme cases will have different effects. If the soil is significantly softer with respect to the structures, all the parameters associated with the soil will have greater influence on the response of the buildings. If on the other hand the soil is significantly stiffer than the structures, the structures stiffness will have greater influence then the soil parameters; furthermore, the interaction spring and mass might have an almost null effect and the structures could be considered decoupled.
The first step is to identify which parameter dominates the response or if they all have a comparable impact on the output. One way of doing this is to plot the vector \( x_m \) formed of interest measuring points, like base and top of each structure. Consider a system like the one depicted in Figure 4.4. Figure 4.7 shows the displacement time history of the top and base of the first structure for the three possible cases, namely: response is dominated by the soil parameters; response dominated by both structures and soil parameters (comparable influence scenario); response dominated by the structural parameters. When considering Figure 4.7 (a) the fact that there is very little difference between the top and base displacement indicates that there is no relative displacement of the structure with respect to its base. This type of situations can be solved using a two-step identification approach, namely: first, identifying the soil stiffness parameters using as \( x_m \) vector either, the displacements, velocities or accelerations at the foundation level of the structures; the second step is aimed to identify the stiffness of the structures using as vector \( x_m \) either the displacements, velocities or accelerations of the top of the structures.

An intermediate case is presented in Figure 4.7 (b), in which all the parameters have a comparable effect. This is considered an ideal case, since most of the time the problem can be solved in a single stage. Finally, the other extreme case is that in which the response is dominated by the lateral stiffness of the structure, this is equivalent to having a rigid soil stratum. The response of both base and top of the first structure in such case is presented in Figure 4.7 (c). This case can have the particularity of showing no interaction between structures, especially in the case in which just one of the structures is
being excited. If the response is dominated by the stiffness of the structure but there is still an effect on the second one, two-stage approach is needed, similarly to the first case (response dominated by the soil).

Validation

In order to illustrate how to tackle the three types of scenarios, the formulation validation of each LODM (TDOF, shear type building and beam elements) will be done for one of the possible cases, namely: the TDOF model will be validated for a case in which the response is dominated by the soil; the shear type building validation will be carried out for a comparable influence case; and, finally, the beam elements model will be validated for a case dominated by the structures.

To validate the TDOF formulation, a model containing the same number of DOF was created in ADINA. This model is depicted in Figure 4.8. To evaluate the case in which the response is dominated by the soil (see Figure 4.9), the following set of parameters were used: foundation mass of the first structure \( m_{f,1} = 4\text{E3} \); mass of the first structure \( m_1 = 2\text{E3} \); foundation mass of the second structure \( m_{f,2} = 4\text{E3} \); mass of the second structure \( m_2 = 2\text{E3} \); intermediate mass \( m_s = 4\text{E3} \); modal damping \( \zeta_n = 0.05 \) for all modes; SSI stiffness \( k_1 = k_2 = 1\text{E8} \) and \( k_{s,1} = k_{s,2} = k_{s,ss} = 5\text{E6} \). The model was excited at the base node of the first structure with a force time history. The input signal was a sample of white noise with a bandwidth of 0-50 Hz, total duration of 1.0 s and time discretisation \( \Delta t \) of 1E-3. The identification procedure was carried out in two stages. The first stage is aimed to identify the soil parameters, that is to say the three stiffness (\( k_{s,1}, k_{s,2}, k_{s,ss} \)) and the three masses (\( m_{f,1}, m_{f,2}, m_s \)). The initial values of \( k_1 \) and \( k_2 \) were 6E7, \( k_{s,1} = k_{s,2} = k_{s,ss} = 3\text{E6} \) started at 3E6 while \( m_{f,1} = m_{f,2} = m_s \) initial values were 2.4E3. The second stage of the procedure is aimed to identify the lateral stiffness of the structures, using as initial values those reached in the first stage of the procedure for the soil parameters, and \( k_1 = k_2 = 6\text{E7} \) in the first iteration. This two-stage procedure must be carried out several times until convergence is reached. Figure 4.10 presents the target acceleration time history (blue markers), with both acceleration time histories of the LODM formulation with initial (dashed yellow line) and final iteration parameters (red line). Figure 4.11 display the convergence of the sought parameters at the end of every two-stage iteration.
Figure 4.8 ADINA model of two coupled TDOF.

Figure 4.9 Acceleration time history measured at the top and base of both buildings, case dominated by the soil.

It can be observed from both Figure 4.9 and Figure 4.10 that the frequency content of the response signal of both building one and two are very different. This occurs since the input signal that arrives at the second building has already been filtered by both the first building and the soil mass in between buildings. Moreover, from Figure 4.10 it can be concluded that there is an excellent degree of agreement between the LODM formulation and ADINA models. In addition, the identification procedure has been proved successful in the identification of the correct parameters evidenced in both a reduction of the initial norm and in the convergence of all the parameters to the defined values (see Figure 4.11).
Figure 4.10 Comparison between initial and final iteration acceleration time histories with results obtained in ADINA for equivalent TDOF superstructure models.
Figure 4.11 Covergene of the two-stage procedure for equivalent TDOF superstructure models.

Application

Consider the FE model depicted in Figure 4.12 composed of a soil stratum and two identical structures. The model constitutes a modification of the small-scale soil-structure model built for the LOSSVAR project (Zentner I. 2016) by including an additional structure. The soil stratum possess a mass density $\rho = 1680 \text{ kg/m}^3$, a shear wave velocity of $V_s = 630 \text{ m/s}$, and a Poisson ratio of $\mu = 0.46$. The structures have a mass density
ρ = 420 kg/m³, a modulus of elasticity E = 2.55E8 and a Poisson ratio of μ = 0.2. Both structures have the particularity of having an aspect ratios Ar = 0.6 in the X direction. The model is excited by force time history applied in the X direction at either the top or base of the first structure (located on the left). The input signal comprises a sample of white noise with a total duration of 0.16s, time discretisation interval of 1.95E-5s and a frequency bandwidth of 800-5120 Hz.

With the aim of gaining insight into the influence of the parameters, the top and base acceleration time histories are analysed and depicted in Figure 4.13 when the system is excited by a dynamic force applied at the top of the first structure. It seems that the response could be in the limit of being either comparable or dominated by the structural parameters (stiff soil). As it is more convenient a single stage approach is followed to attempt the identification of the system depicted in Figure 4.12.

The mass of the superstructures is assumed to be the same for both structures and equal to 0.3 kg. The remaining parameters of the model are to be identified, and their initial values are as follow, lateral stiffness of both structures $k_1 = k_2 = 1E8$ N/m; SSI and SSSI parameters $k_{s1,1} = k_{s1,2} = k_{s1,1} = k_{s1,2} = 1E8$ N/m; foundation masses $m_{f,1} = m_{f,2} = 0.6$ kg; intermediate soil mass $m_s = 0.6$ kg. To avoid problems of ill-conditioning, scaling factor of 1E8 was applied to all the masses.

Selecting the appropriate measuring points is of paramount importance if one seeks to obtain a well-conditioned problem. Depending on the force excitation point, different measurement points will reflect more the influence of certain parameters. In this case, the aim is to find the appropriate set of measuring points that provide a balanced representation of the influence of all the sought parameters. When the force is being applied at the base of either structure, the response at foundation level is bound to depend on all parameters, provided that the case being analysed follows in the category of comparable influence.

In this application, the vector of measured response $\mathbf{x}_m$ consisted of the acceleration time history at the base of both structures when the system is under a dynamic force excitation applied at the base of the first structure. For this case, all the sought parameters have a comparable influence and their sensitivities have the same order of magnitude.
Figure 4.12 HOFEM of Two-structure coupled with soil domain.

Figure 4.13 Response of the FEM, top and base acceleration time history of both structures when subject to a dynamic force excitation applied at the top of the first structure.

The analyses results are summarised in Figure 4.14 and Figure 4.15. The response acceleration time histories when the model is excited at the base are presented in Figure 4.14 for the top and base of both structures. It is seen that a fair degree of agreement is achieved after 100 iterations, particularly for the top and base of the first structure, while the response of the second building is captured with less accuracy. Convergence of the parameters is achieved after approximately 60 iterations (see Figure 4.15) which points to a well-conditioned system equations. Significant improvement from the initial conditions is archived in all four measuring points. This is evidenced both graphically
and through the decrease of the final norm with respect to the initial norm of the vector representing the difference between the target and estimated response.

Figure 4.14 Acceleration time histories of both top and base of the building. Comparison between the HOFEM and LODM using TDOF to model both superstructures.
Figure 4.15 Convergence of the model parameters. Results of the LODM using TDOF to model both superstructures.
4.5.2 Two shear-type buildings

This subsection is devoted to develop a two structures model coupled with and through the soil using shear-type buildings to represent the superstructures. This model combines a simple formulation with the ability to reproduce a more realistic structural behaviour with a wider range of applications when compared with the TDOF systems. It can accommodate scenarios in which the response of the structure is not dominated by a single frequency, or cases in which the input signal is broadband instead of narrowband. Figure 4.16 presents the sketch of the proposed model. The presence of an intermediate DOF between structures with a concentrated mass to account for wave propagation effects in the model must be stressed.

Figure 4.16 Sketch of the LODM formed of two structures coupled with and through the soil using shear type building to represent the superstructure.

Validation

The validation of the LODM formulation considering shear-type buildings to model the superstructures is carried out following a similar approach as in the case of the two-TDOF system. A model containing the same number of DOF was created in ADINA. This model is depicted in Figure 4.17. To evaluate the case in which the all the parameters have a
comparable influence (see Figure 4.18), the following set of parameters were used: foundation mass of the first structure $m_{f,1} = 4E3$ kg; mass density of the first structure $\rho_1 = 3.48E6$ kg/m$^3$; foundation mass of the second structure $m_{f,2} = 4E3$ kg; mass density of the second structure $\rho_2 = 3.48E6$ kg/m$^3$; intermediate mass $m_s = 4E3$ kg; modal damping $\zeta_n = 0.05$ for all modes; SSI stiffness $k_{ss,1} = k_{ss,2} = 3E8$ N/m; SSSI stiffness $k_{sssi,1,2} = 5E8$ N/m; shear modulus $G_1 = G_2 = 5E8$ N/m$^2$. The lateral stiffness of each structure was calculated using equation (3.39). The geometrical properties of the structures are those of the LOSSVAR structure. The model was exited at the base node of the first structure with a force time history. The input signal was a sample of white noise with a bandwidth of 0-50 Hz, total duration of 1.0 s and time discretisation $\Delta t$ of 1E-3. The identification procedure was carried out in a single stage. The initial values of $G_1$ and $G_2$ were 3.5E8 N/m$^2$, $k_{ss,1} = k_{ss,2} = 2.1E8$ N/m, $k_{sssi,1,2}$ initial value was 3.5E8 N/m while $m_{f,1} = m_{f,2} = m_s$ initial values were 2.8E3. Figure 4.19 presents the target acceleration time history (blue markers), with both acceleration time histories of the LODM formulation with initial (dashed yellow line) and final iteration parameters (red line). Figure 4.20 displays the convergence of the sought parameters.

Figure 4.17 ADINA model of two coupled shear type buildings.
The comparable influence of the model parameters is put in evidence in the plots presented in both Figure 4.18 and Figure 4.19, where the soil parameters are such that both the frequency content and the amplitude of the response at the second structure is somehow similar to that of the first structure. This is in contrast with the case analysed in the validation of the TDOF, where the response is dominated by the soil parameters (in that case, both frequency content and amplitude of the response of the second building are significantly smaller than those of the first structure).

Furthermore, it seems from Figure 4.19 that a satisfactory agreement exists between the LODM proposed herein and those found using the FEM software ADINA. The proposed procedure has been proved successful in the identification of the correct parameters observed in both a reduction of the initial norm of the vector containing the difference between the target and estimated response, as well as in the convergence of all the parameters to the defined values (see Figure 4.20).
Figure 4.19 Comparison between initial and final iteration acceleration time histories with results obtained in ADINA for equivalent shear type building superstructure models.
Figure 4.20 Convergence of the single-stage procedure for equivalent shear type building superstructure models.

**Application**

The same problem considered in the validation of the two-TDOF system will be used in this subsection. Section 4.5.1 offers additional details on the HOFM.

The initial properties of the LODM using shear type buildings to represent the superstructure are the following: mass density of both superstructures is the same as that used in the HODM and equal to 420 kg/m$^3$, the remaining parameters of the model are to be
identified. Their initial values are: shear modulus $G_1$ and $G_2$ was 1E8 N/m$^2$; $k_{ss1} = k_{ss2}$ started at 3E8 N/m; $k_{mni,2}$ initial value was 3E8 N/m; while $m_{f,1} = m_{f,2} = m_i$ initial values were 0.8 kg. To avoid problems of ill-conditioning, scaling factor of 3E8 was applied to all the masses to be identified.

In this application the vector of measured response $x_m$ consisted of the acceleration time histories at the base of both structures as well as the relative displacement between top and base of each structure, when the system is under a dynamic force excitation applied at the base of the first structure. Four sets of acceleration time histories form the vector of measured response. The numerical conditioning of the problem is somehow linked to the type of formulation that is employed to represent the target model. The more complex the formulation, the greater the risk that the identification procedure becomes ill-conditioned. This occurs due to the different sources of stiffness that coincide at the foundation level, namely, that provided by discrete springs and that provided by the lateral stiffness of the first storey of either a shear type building or beam elements. One way to overcome this setback is by implementing the weighted constraint optimisation procedure described in Section 4.3.

The analyses results are summarised in Figure 4.21 and Figure 4.22. The response acceleration time histories when the model is excited at the base are presented in Figure 4.21 for the top and base of both structures. It is seen that a fair degree of agreement is achieved after 200 iterations, particularly for the base of the first structure, while the response of the second building is captured with less accuracy. It is important to highlight that the amplitude of the response of the first building is underestimated. Convergence of the parameters is achieved after approximately 140 iterations (see Figure 4.22) which points to a well-conditioned system equations. Significant improvement from the initial conditions is archived in all four measuring points, this is evidenced both graphically and through the decrease of the final norm with respect to the initial norm of the vector representing the difference between the target and estimated response.
Figure 4.21 Acceleration time histories of both top and base of the building.
Comparison between the HOFEM and LODM using TDOF to model both superstructures.
Figure 4.22 Convergence of the model parameters. Results of the LODM using TDOF to model both superstructures.

It is worth mentioning that, unlike the case of the single building presented in Chapter 3, the shear-type building model seems to overall provide results that are less satisfactory than those provided by the simpler model using TDOF systems to represent the superstructure. This might be caused by the number of measuring points that are limited to the top and base of the structures, which coincide with the DOF of the TDOF systems, it is possible that increasing the number of measuring points might improve the accuracy of the shear-type building models. Furthermore, the possibility of assigning different
stiffnesses to each storey of the building’s model might also contribute to the improvement of the solution.

4.5.3 Two beam elements model

In this section, a model in which both superstructures are described using beam elements that take into consideration shear effects is presented. Coupling with and through the soil is accounted for using linear springs. Wave propagation effects are included by means of an intermediate mass in between buildings. This model has the potential to accommodate scenarios in which the response of the structure is not dominated by a single frequency or cases in which the input signal is broadband instead of narrowband. Figure 4.23 presents the sketch of the proposed model. The individual elements formulation is presented in section 3.2.3, and the assemblage of the full system follows the same approach as in equations (4.8) and (4.9).

![Figure 4.23 Sketch of the LODM formed of two structures coupled with and through the soil using beam elements to represent the superstructure.](image-url)
Validation

The validation of the LODM formulation considering beam elements to model the superstructure is carried out following a similar approach as in the case of the two-TDOF system. A model containing the same number of DOF was created in ADINA. This model is depicted in Figure 4.24. For this validation of the model the case in which the response of the system is dominated by the structures is considered, that is to say that the soil is very stiff compared to the structure. The following set of parameters were used:

- Foundation mass of the first structure $m_{f,1} = 4E3$ kg; mass density of the first structure $\rho_1 = 3.0E6$ kg/m$^3$;
- Foundation mass of the second structure $m_{f,2} = 4E3$ kg; mass density of the second structure $\rho_2 = 3.0E6$ kg/m$^3$;
- Intermediate mass $s = m$
- Modal damping $\zeta_n = 0.05$ for all modes;
- SSI stiffness $k_{ss1,1} = k_{ss1,2} = 1E8$ N/m; SSSI stiffness $k_{ss1,2} = 1E8$ N/m;
- Elastic modulus $E_1 = E_2 = 1E7$ N/m$^2$.

The geometrical properties of the structures are those of the LOSSVAR structure. Figure 4.25 displays the acceleration time histories at the top and base of each structure when the system is excited at the top of the first structure. It can be observed that there is a significant difference in the response of the top with respect to the base of the first structure. Moreover, in the second structure, the amplitudes are significantly smaller with respect to the first building, indicating that the response will be more dependent on the structural parameters than the soil parameters.

The model was excited at the base node of the first structure with a force time history. The input signal was a sample of white noise with a bandwidth of 0-50 Hz, total duration of 1.0 s and time discretisation $\Delta t$ of 1E-3. The identification procedure was carried out in a single stage. The initial values of $E_1$ and $E_2$ were 7E6 N/m$^2$; $k_{ss1,1} = k_{ss1,2}$ started at 7E7 N/m; $k_{ss1,2}$ initial value was 7E7 N/m, while $m_{f,1} = m_{f,2} = m_s$ initial values were 2.8E3. Figure 4.26 presents the target acceleration time history (blue markers), with both acceleration time histories of the LODM formulation with initial (dashed yellow line) and final iteration parameters (red line). Figure 4.27 display the convergence of the sought parameters. As it has been pointed out, this problem has the particularity that a given set of parameters has a greater impact on the system response, which may lead to problems of ill-conditioning. To overcome this problem, the identification procedure can be split in two stages, where parameters are grouped in either stage according to their influence in the overall system response. The appropriate selection of both excitation and measuring points is crucial to obtain a well-conditioned problem.
Figure 4.24 ADINA model of two coupled beam elements.

Figure 4.25 Acceleration time history measured at the top and base of both buildings, comparable influence case.

Figure 4.26 displays the comparison between the results obtained using the LODM at the beginning and end of the identification procedure with the response obtained using the software ADINA. A satisfactory agreement is found between ADINA and the LODM result at the end of the identification procedure. Is important to highlight that when, compared with the previous two alternative superstructure models, the beam elements formulation requires better initial conditions are required to obtain a well-conditioned problem. Convergence plots of each parameter are displayed in Figure 4.27. The identification procedure has been proved successful in the identification of the correct parameters observed in both a reduction of the initial norm of the vector containing the difference between the target and estimated response as well as in the convergence of all
the parameters to the defined values (see Figure 4.27). Therefore, the formulation of the system as well as the time integration technique seem to be correct.

Figure 4.26 Comparison between initial and final iteration acceleration time histories with results obtained in ADINA for equivalent shear type building superstructure models.
Figure 4.27 Convergence of the single-stage procedure for equivalent shear type building superstructure models.

Application

The same problem considered in the validation of the two-TDOF system will be used in this subsection. Section 4.5.1 provides additional details on the HOFM.

The initial properties of the LODM using beam elements to represent the superstructure are the following: mass density of both superstructures is the same as that used in the HODM and equal to 420 kg/m$^3$; the remaining parameters of the model are to be
identified, and their initial values are: elastic modulus $E_1$ and $E_2$ was $3 \times 10^9$ N/m$^2$; $k_{m,1} = k_{m,2}$ started at $3 \times 10^8$ N/m; $k_{m,1,2}$ initial value was $3 \times 10^8$ N/m while $m_{f,1} = m_{f,2} = m_s$ initial values were 0.6 kg. To avoid problems of ill-conditioning, scaling factor of $1 \times 10^9$ was applied to all the masses to be identified.

In this application, the vector of measured response $x_m$ consisted of the acceleration time history at the base of both structures as well as the relative displacement between top and base of each structure, when the system is under a dynamic force excitation applied at the base of the first structure. Four sets of acceleration time histories form the vector of measured response. Due to problems of ill-conditioning, the weighted constraint optimisation procedure described in section 4.3 was used during the identification procedure.

The analyses results are summarised in Figure 4.28 and Figure 4.29. The response acceleration time histories when the model is excited at the base are presented in Figure 4.29 for the top and base of both structures. A fair level of matching is achieved the top of the first structure. The remaining three measuring locations do not display a satisfactory agreement between the LODM and the HOFM. In addition, as it has occurred in the validation stage of the beam elements model, good initial conditions are required to achieve convergence of the parameters. The inadequacy of the model is further evidenced in the reduction of the norm of the vector containing the difference between the estimated and target response. Figure 4.29 display the convergence plots for all the parameters. Convergence of all the parameters is achieved after approximately 200 iterations.

It can be concluded that among the three alternative models using beam elements to represent the superstructure produces the less satisfactory results.
Figure 4.28 Acceleration time histories of both top and base of the building. Comparison between the HOFEM and LODM using TDOF to model both superstructures.
Figure 4.29 Convergence of the model parameters. Results of the LODM using TDOF to model both superstructures.
4.6 Summary

In this chapter, a novel discrete model able to account for wave propagation effects has been presented. This model considers the addition of a DOF with a mass located in between the two structures (see Figure 4.4). The inclusion of this mass indirectly account for the distance between structures, by affecting the velocity but also by decreasing the amplitude of the particle acceleration as this mass increases.

It is noted that one of the limitations of the proposed model is that it does not account for the higher damping of the soil materials with respect to the structure’s materials. It is acknowledged that this simplification could have an impact in the model’s response, as in general geological materials have higher damping capacities with respect to commonly used structural materials.

The application of the bounded variable time-domain least-square identification procedure to identify the parameters of a series of LODM able to account for both SSI and SSSI effects has been undertaken. To this aim, three different simplified models were studied, namely, a two TDOF systems, two uniform shear type buildings and two Timoshenko beams.

It was seen that, unlike the case of the single building presented in Chapter 3, the shear-type building model seems to overall provide results that are less satisfactory than those provided by the simpler model using TDOF systems to represent the superstructure. Moreover, using beam elements to represent the superstructure produces the less satisfactory results and significantly increase the complexity of the models. In both the shear-type buildings and beam elements models, this might be caused by the number of measuring points that are limited to the top and base of the structures, which coincide with the DOF of the TDOF systems, it is possible that increasing the number of measuring points might improve the accuracy of the shear-type building models. Furthermore, the possibility of assigning different stiffnesses to each storey of the building’s model might also contribute to the improvement of the solution.

Finally, a novel two-stage approach has been successfully introduced to breach the problem of disproportionate sensitivities. It has been observed from the analysis conducted in the present study, that the parameters governing SSSI systems tend to be easily classified in two groups, namely, high and low incidence in the response. To
determine the parameters with greater and lesser influence in the response, a threshold of one order of magnitude was established.
5 Physical models on SSI and SSSI

In the study of both SSI and SSSI, physical models have played an important role, providing robust experimental evidence to uphold the observations made in both analytical and numerical studies. A more detailed review of the main experimental setups that have been used for this purpose can be found in Section 2.2.3.

This Chapter focuses on the development of a methodology to both make and test small scale models exhibiting SSI and SSSI effects. The models are used as a validation and verification (V&V) tool for the proposed identification procedures presented in Chapters 3 and 4.

The SSI models construction and testing campaign was part of the NUGENIA+ pilot project LOSSVAR and has benefitted from financial support by FP7 NUGENIA+. The aim of the experimental tests was to provide a validation and verification tool for a benchmark study. The study was aimed to analyse the influence of soil variability on the study of SSI. The models used in the LOSSVAR project were limited to a single isolated structure on several soil profiles.

The main testing technique implemented was impact testing, complemented with modal analysis. Experiments were performed separately for both the soil and structure to characterize their individual response and identify their material parameters. With the aim of generating experimental data that can be further used to V&V the proposed identification procedures presented in Chapters 3 and 4, two additional models were made. The first model to V&V the SSI scenario was identical to those used for the LOSSVAR project, consisting of an individual structure founded on a uniform soil stratum. The second model was used to V&V the SSSI identification procedure, and it was formed of two identical structures founded on a uniform soil stratum. Both SSI and SSSI models were subjected to dynamic force excitation to generate records of acceleration time histories that could be fed into the proposed time-domain identification procedures.
5.1 LOSSVAR project

The accurate modelling of soil properties and soil-structure interaction (SSI) are an important issue for the seismic margin and safety assessment of industrial plants and critical infrastructures. Concurrently, it has been shown in the past that the spatial variability can have a major impact on extended and multi-supported structures. LOSSVAR implements probabilistic numerical SSI models and evaluate the impact on the expected seismic margin and safety factors in probabilistic risk assessment. To this aim, a model of an industrial buildings is employed for the benchmark study, whose details are presented in Figure 5.1.

![Finite element model and details of the industrial building](image)

Figure 5.1 Finite element model and details of the industrial building used for benchmarking (Trevlopolos et al. 2016).

The dynamic behaviour of the industrial building model founded on a stratum with a shear wave velocity $V_s$ of 600 m/s is displayed in terms of amplification factor in Figure 5.2. Measurements have been taken at the centre of the third storey, as indicated in Figure 5.1. Two scenarios were considered, namely: one with spatial soil variability, referred as the incoherent case in the figure; one considering a homogeneous soil profile, indicated as the coherent case.
In the context of this Chapter, the LOSSVAR project also considered the creation and testing of scaled physical models of both the structure and the soil stratum. The aim of the experimental tests was to provide a validation and verification tool for the benchmark study. The following subsections are dedicated to elaborate on the methods and materials used for the creation and testing of the models. Full details on the experimental campaigned can be found in Coronado-Jimenez and Cacciola (2016).

5.1.1 Scaling relationships

There are two main approaches to develop scaling laws, namely Froude and replica scaling. The main difference consists in which variables are selected as base variables for the scaling process. For instance, Froude scaling approach considers length \((L)\), mass density \((\rho)\) and acceleration \((a)\) as base variables, while replica scaling considers length, mass density and stress \((\sigma)\). The principle to derive the scaling laws is illustrated below, where subscripts \(p\) and \(m\) refer to prototype and model, respectively:

\[
\lambda_l = \frac{L_p}{L_m}; \lambda_{\rho} = \frac{\rho_p}{\rho_m}; \lambda_a = \frac{a_p}{a_m}
\]

(5.1)
where \( \lambda_l \) is the scaling factor for length, \( \lambda_\rho \) is the scaling factor for mass density and \( \lambda_a \) is the scaling factor for acceleration equal to unity for Froude scaling method.

Applying dimensional analysis, and using the relationships of equation (5.5), the scaling factor \( \lambda_m \) for mass \( m \) can be derived as follow:

\[
\lambda_m = \frac{\rho_m}{\rho_p} = \frac{m_p}{m_m} \left( \frac{L_p}{L_m} \right)^3 = \frac{m_p L_m^3}{m_m L_p^3} = \frac{\lambda_m L_p^3}{\lambda_m L_m^3} = \lambda_m = \lambda_p \lambda_l^3
\]  

(5.2)

Analogously the scaling factor \( \lambda_t \) for time \( t \) is defined

\[
\lambda_t = \frac{a_p}{a_m} = \frac{\frac{L_p}{t_p}}{\frac{L_m}{t_m}} = \frac{\lambda_m t_m^2}{\lambda_l t_p^2} = \frac{\lambda_m}{\lambda_l^2} = 1 \rightarrow \lambda_t = \sqrt[2]{\lambda_l}
\]  

(5.3)

while the scaling factor \( \lambda_\sigma \) for stress \( \sigma \) is found from the definition of stress as force \( F \) divided by area \( A \) and presented in equation (5.4)

\[
\lambda_\sigma = \frac{\sigma_p}{\sigma_m} = \frac{\frac{F_p}{A_p}}{\frac{F_m}{A_m}} = \frac{m_p a_p L_m}{m_m a_m L_p} = \frac{\lambda_m \lambda_a}{\lambda_l^2} = \frac{\lambda_p \lambda_l^3}{\lambda_l^2} \rightarrow \lambda_\sigma = \lambda_p \lambda_l
\]  

(5.4)

For the case of replica scaling method, the same approach can be followed to derive the respective scaling laws. The scaling relationships to relate the full scale model (the prototype) and scaled physical model (the model) for the LOSSVAR project were developed as in equations (5.2) to (5.4) choosing length \( L \), mass density \( \rho \) and velocity \( v \) as base variables.
Table 5.1 Scaling relationships for the LOSSVAR models

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Dimensions</th>
<th>Scaling factor</th>
<th>Scaling value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass [M]</td>
<td>m</td>
<td>(\lambda_p^3)</td>
<td>6.4E7</td>
</tr>
<tr>
<td>Force [F]</td>
<td>F</td>
<td>(\lambda_p^2)</td>
<td>1.6E5</td>
</tr>
<tr>
<td>Acceleration [a]</td>
<td>(Lt^{-2})</td>
<td>(\lambda_i^{-1})</td>
<td>2.5E-3</td>
</tr>
<tr>
<td>Time [t]</td>
<td>t</td>
<td>(\lambda_t)</td>
<td>400</td>
</tr>
<tr>
<td>Linear dimension [L]</td>
<td>L</td>
<td>(\lambda_L)</td>
<td>400</td>
</tr>
<tr>
<td>Frequency [f]</td>
<td>(t^{-1})</td>
<td>(\lambda_f^{-1})</td>
<td>2.5E-3</td>
</tr>
<tr>
<td>Modulus of elasticity [E]</td>
<td>(FL^{-2})</td>
<td>(\lambda_i^{-1})</td>
<td>2.5E-3</td>
</tr>
<tr>
<td>Stress [(\sigma)]</td>
<td>(FL^{-2})</td>
<td>(\lambda_i^{-1})</td>
<td>2.5E-3</td>
</tr>
<tr>
<td>Poisson's [(\nu)]</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

Besides all the benefits of small-scale modelling there are drawbacks that need to be taken into account when experiments are being design and analysed. Issues like the inability to reproduce stress states in the model’s soil, the necessity of weaker simulant materials and the difficulty to satisfy simultaneously all the scaling laws, are some of the main complications that are faced when scaled models are being implemented. Since in this case the velocity \(v\) has been chosen as a base variable and its scaling factor \(\lambda_v\) set equal to 1, both the full scale and scaled models have the same shear wave velocity. This is particularly helpful for the selection of the scaled model materials in the case in which hard soils are being studied. A scaling factor for length \(\lambda_L\) equal to 400 has been adopted for the LOSSVAR models.

5.1.2 Soil testing

In order to determine the soil material properties and characterize the response of the isolated soil strata, experiments were performed on the soil models without any structure. The soil stratum was modelled using a mixture of Fraction C sand and plaster. The fraction C sand is characterised by having at least 75% of its particles within the sizes range of 150-300 µm. Table 5.2 presents the three considered mixtures and their proportions.

Table 5.2 Sand-Plaster mixtures materials proportions

<table>
<thead>
<tr>
<th>Mixture</th>
<th>P/S *</th>
<th>W/P±</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.04</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>0.12</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>0.20</td>
<td>1</td>
</tr>
</tbody>
</table>

*Plaster/ Sand ratio
±Water/Plaster ratio
Three homogenous sand-plaster blocks of 393 mm length, 246 mm wide and 130 mm height were casted (according to the proportions given in Table 5.2) Figure 5.3 display a sample of the block casted with sand-plaster ratio of 0.04. An attachment mechanism was placed at the base of the soil stratum (see Figure 5.4 ) to provide a strong bond between the base and the block. The attachment mechanism was 30 mm high, leaving an effective stratum of 100 mm equivalent to 40 m in the full-scale model units.

![Figure 5.3 Sand/Plaster block with 0.04 P/S ratio.](image)

![Figure 5.4 Attachment mechanism for blocks 30 mm high.](image)

The testing campaign to identify the properties of each soil stratum and establish a relationship with the sand-plaster ratio, consisted of the measurement the compressive wave velocity $V_p$ and the identification of the shear wave velocity $V_s$. The velocity of the compressive waves was measured on each soil stratum sample utilising an ultrasonic pulse equipment, as depicted in Figure 5.5. The results are summarised in Table 5.3.
Figure 5.5 Compressive wave velocity measurement using ultrasonic pulse equipment.

On the other hand, the shear wave velocity of the samples was identified indirectly, as impact hammer tests were conducted on each sample and the response of the block was recorded by two uniaxial accelerometers located at the centre and corner top of the soil stratum in its long direction (see Figure 5.14). Figure 5.6 displays the plots of the FRF for the three different homogenous soil models (P-S ratios of 0.04, 0.12 and 0.20) each plot presents two sets of curves, namely, the FRF calculated with the data recorded by the accelerometer located at the centre and on the corner of the block. Translational modes should be recorded by both accelerometers while torsional modes should be identified by significant contrast in terms of amplitude between both accelerometers. Based on this, the first and second translational modal frequencies as well as the first torsional can be identified in the plots bellow. Applying the scaling factor defined in Section 5.1.1, the FRF can be expressed in full scale units. These are depicted in Figure 5.7.

Figure 5.6 FRF of the three soil samples in scaled model units.
Figure 5.7 FRF of the three soil samples in full scale units.

From the results presented in Figure 5.6 and Figure 5.7, a consistent increase in the resonant frequencies proportional to the plaster-sand ration is observed. Unlike the compressive wave velocity, the shear wave velocity is estimated by the knowledge of the natural frequency of the soil stratum. Assuming one dimensional ground response, equation (5.5) can be used to calculate the shear wave velocity \( V_s \):

\[
f = \frac{V_s}{4H} [Hz]
\]  

where \( f \) is the fundamental frequency of the soil stratum measured experimentally, \( V_s \) is the shear wave velocity of the soil, \( H \) is the depth of the soil stratum.

<table>
<thead>
<tr>
<th>Translational Modes</th>
<th>Frequencies</th>
<th>Soil Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Impact Test</td>
<td>E [MPa]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1120</td>
</tr>
<tr>
<td>1st</td>
<td></td>
<td>2193.75</td>
</tr>
<tr>
<td>2nd</td>
<td></td>
<td>1340.65</td>
</tr>
<tr>
<td>1st</td>
<td></td>
<td>2596.87</td>
</tr>
<tr>
<td>2nd</td>
<td></td>
<td>1454.68</td>
</tr>
</tbody>
</table>

Where \( E \) is the Young’s modulus, \( \rho \) is the mass density, \( \nu \) is the Poisson’s ratio calculated as in equation (5.6), \( G \) is the shear modulus, \( V_s \) is the shear wave velocity and \( V_p \) is the compressive wave velocity.
\[ v = \frac{\left( V_p^2 - 2V_s^2 \right)}{2\left( V_p^2 - V_s^2 \right)} \]  

(5.6)

Figure 5.8 Variation of the shear wave velocity of the soil stratum with the plaster-sand ratio.

Finally, the data gathered during the testing of the soil stratum can be used to establish a relationship between the plaster-sand ratio and the shear wave velocity. This correlation is displayed in Figure 5.8 and can be used to reproduce a range of shear wave velocities in the scaled model.

5.1.3 Structure testing

The structure model was a scaled version of the industrial model depicted in Figure 5.1. The structure was designed to have the first translational resonant frequency in the same range of the soils’ resonant frequency (see Figure 5.6 and Figure 5.13). For convenience, the structure was made of acrylic sheets of different thicknesses. The design process of the scaled structure model involved the determination of the acrylic properties to serve as input parameters in a numerical model of the scaled industrial building model so to confirm that the structure had its fundamental frequency in the vicinities of the soil’s fundamental frequency. The construction of the model was made using three different thicknesses, namely, 2 mm, 4 mm and 5 mm. Samples of these materials were tested as cantilever beams using impact testing, as shown in Figure 5.9. The FRFs for each thickness are displayed in Figure 5.10. From these plots the fundamental frequency can be determined and, by means of equation (5.7), the modulus of elasticity can be estimated.
\[ \omega_1 = \frac{3.516}{L^2} \sqrt{\frac{EI}{m}} \]  \hspace{1cm} (5.7)

where \( \omega_1 \) is the first natural frequency of the continuum, measured experimentally; \( L \) is the length of the continuum; \( E \) is the sought modulus of elasticity; \( I \) is the moment of inertia of the cross section, and \( m \) is the mass per unit length. Table 5.4 summarizes the properties of the acrylic for the three different thicknesses estimated with equation (5.7).

Figure 5.9 Testing methodology for the structures’ materials, cantilever conditions.

Figure 5.10 FRF for 2 mm acrylic element.
Table 5.4 Translational modal frequencies and soil properties

<table>
<thead>
<tr>
<th>Thickness [mm]</th>
<th>L [m]</th>
<th>W [m]</th>
<th>ρ [Tonnes/m³]</th>
<th>F [Hz]</th>
<th>E [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.12</td>
<td>0.052</td>
<td>1.14</td>
<td>25.5</td>
<td>1.48</td>
</tr>
<tr>
<td>4</td>
<td>0.12</td>
<td>0.051</td>
<td>1.16</td>
<td>57.6</td>
<td>2.04</td>
</tr>
<tr>
<td>5</td>
<td>0.12</td>
<td>0.051</td>
<td>1.18</td>
<td>84.5</td>
<td>2.33</td>
</tr>
</tbody>
</table>

Where L is the length of the specimen, W is the width of the specimen, ρ is the mass density, F is the frequency of the first mode, E is the modulus of elasticity.

The differences in the modulus of elasticity presented in Table 5.4 are attributed to the fact that each thickness of acrylic is manufactured individually and they are not obtained from the same casting of acrylic.

The assembled physical model of the industrial building, as well as the sensors arrangement during the impact test, is depicted in Figure 5.11. A special base was manufactured to anchor the structure at its base and test it under fully fixed conditions. The FRF computed with the data collected by each sensor is presented in Figure 5.12, while Figure 5.13 presents the same data in full model units.

Figure 5.11 Physical scaled model of the industrial building analysed in the LOSSVAR project.
Figure 5.12 FRF of the physical scaled model of the industrial building analysed in the LOSSVAR project.

Figure 5.13 FRF of the industrial building model in full scale units.

5.1.4 Soil-Structure Systems

The coupled soil-structure system is considered in this section. The structure was embedded 15 mm into the homogeneous soil profile made of a mixture of plaster and sand, with plaster-sand ratio of 0.20 to represent a soil stratum with an approximate shear wave velocity $V_s$ of 600 m/s. The model as well as the monitoring sensors arrangement are depicted in Figure 5.14. Impact testing was conducted on the model and the FRF computed with the data gathered by the accelerometer placed at the centre top of the structure are displayed in Figure 5.15 for both scaled model and full scale units.
Figure 5.14 Physical model of the scaled industrial building coupled with the soil stratum.

![Physical model of the scaled industrial building coupled with the soil stratum.](image)

Figure 5.15 FRF of the accelerometer on the centre top of the structure for both scaled model (left) and full scale (right) units.

![FRF of the accelerometer on the centre top of the structure for both scaled model and full scale units.](image)

Finally, when comparing the result obtained for the scaled physical model of the soil-structure system presented in Figure 5.15 with those reported in Figure 5.2 for the full scale model, they are in good agreement, demonstrating the suitability of both the testing methodology and the physical models to describe the behaviour of full scale models. The following sections elaborate on the testing of both SSI and SSSI systems under dynamic force excitation and the implementation of the time domain identification procedures developed in Chapters 3 and 4 to define simplified models able to account for the SSI and SSSI phenomena.
5.2 Physical model of soil structure interaction

This section focuses on the implementation of the LODM identification procedure, developed in Chapter 3, to determine the parameters of a LODM able to reproduce the response of a small-scale physical model subject to dynamic force excitation. Based on the conclusions presented in Chapter 3, a shear type building model will be used. The testing setup is depicted in Figure 5.16. The small-scale soil-structure model is that built for the LOSSVAR project (Zentner I. 2016). Detailed information on the model is given in Section 5.1.

The model was excited using a shaker able to deliver a force excitation time history with a frequency content of up to 10 kHz. According to Fassois (2001), the force excitation needs to be sufficiently rich to excite the system over the frequency range of interest. The most commonly used excitations include chirp and random signals. The input signal comprises both a periodic chirp and a white noise with a total duration of 0.16 s, time discretisation interval of 1.95E-5s and a frequency bandwidth of 800-5120 Hz. Each excitation signal was applied at the top of the structure and the response of the system was measured using uniaxial accelerometers, placed at the top and base of the structure (see Figure 5.16).
The acceleration recorded at both top and base of the physical model, as well as the initial and final iteration response acceleration of the LODM, are presented in Figure 5.17 and Figure 5.19 for white noise and periodic chirp input, respectively. The convergence plots of the parameters are shown in Figure 5.18 and Figure 5.20 for each excitation, respectively.

**Figure 5.17** Acceleration time histories of both top and base of the building. Comparison between the physical model and LODM using shear type building to model the super structure subjected to white noise force excitation.

**Figure 5.18** Convergence of the model parameters. Results of the LODM using shear type building to represent the physical model subjected to white noise force excitation.
Figure 5.19 Acceleration time histories of both top and base of the building.
Comparison between the physical model and LODM using shear type building to model the super structure subjected to periodic chirp force excitation.

The results displayed in Figure 5.17 and Figure 5.19 indicate that the LODM is able to account fairly well for the response of the physical model in the analysed frequency range. Significant improvement is seen in the degree of matching of the final iteration with respect to the initial values. Furthermore, it looks like the conclusions derived from the parametric analysis, intended to evaluate the impact of rotations on the horizontal translations, are being sustained by experimental evidence. That is to say, structures with aspect ratios below 1 can be evaluated neglecting the contribution of the rotations without

Figure 5.20 Convergence of the model parameters. Results of the LODM using shear type building to represent the physical model subjected to periodic chirp force excitation.
incurred in significant error. This allows the use of a shear type building model to represent the LOSSVAR structure. Additionally, these results suggest that coupled soil-structure systems can be represented using simplified LODM superstructure models coupled with linear springs to account for both SSI and site effects.

Despite the significant improvement with respect to the initial conditions and the fair degree of agreement between the experimental response and the LODM, there are some discrepancies, especially, in the response at the base of the structure. One of the reasons for the discrepancies is the fact that the system is being excited just at the top of the structure, making the system very dependent on the elastic properties of the building. Moreover, the signal that arrives at the base of the structure is smaller in amplitude and has been filtered as it passes through the structure. Nevertheless, the ability to reproduce the complex behaviour of 3D structures by simplified LODM is very powerful. It is worth stressing the practical applicability of the proposed procedure, as performing forced vibration tests and measuring the response of existing structures does not involve major endeavours.

It is noted that there are significant differences in the identified parameters of the LODM (see Figure 5.18 and Figure 5.20) when the model is subject to two different types of input signals, namely, white noise and periodic chirp. This is a limitation of the proposed procedure. Nevertheless, in order to assess which of the two models represent best the actual experimental set-up each identified model was subject to a different input signal than the one that was used to identify the model in the first place. Figure 5.21 presents the comparison of the acceleration time histories, measured at the top and base of the structure, between the experimental set-up and the LODM identified using a white noise signal, when subject to a periodic chirp force excitation. Analogously, Figure 5.22 displays the comparison of the acceleration time histories, measured at the top and base of the structure, between the experimental set-up and the LODM identified using a periodic signal, when subject to a white noise force excitation.
Figure 5.21 Acceleration time histories of both top and base of the building. Comparison between the LODM identified using a white noise signal and the physical model each subject to a periodic chirp force excitation signal. Based on the norm of the vector containing the differences between the LODM and the physical time histories (displayed in Figure 5.21 and Figure 5.22), the model identified using the white noise signal provides a better representation of the physical experiment with respect to the model identified using the periodic chirp signal. It is concluded that there are local minima within the boundaries of the problem and that in this case different inputs yield two different solutions.
5.3 Physical model of structure soil structure interaction

In this section, a physical model of two adjacent buildings is analysed. The model studied herein constitutes a modification of the models described in Section 5.1; indeed, instead of a single structure, two adjacent identical structures are considered (see Figure 5.23). The two structures are separated by a gap of 1 cm in model units (this is equivalent to 4 m in prototype units). The soil stratum was homogeneous. The model was subjected to dynamic force excitation by means of a shaker, and the response was measured at different points of the model using uniaxial accelerometers. The measured data was used to identify a discrete system able to reproduce the measured data using the time-domain identification procedure proposed in Chapter 4. In particular, the shear type building formulation was used. The shaker is able to deliver a force excitation time history with a frequency content of up to 10 kHz. The input signal comprises both a periodic chirp and a white noise with a total duration of 0.16 s, time discretisation interval of 1.95E-5 s and a frequency bandwidth of 800-5120 Hz. Each excitation signal was applied at the top and base in two different tests, while the response of the system was measured at several locations. This type of experimental setup is often referred as Single Input Multiple Output (SIMO).

Figure 5.23 Physical model of two adjacent buildings.
The acceleration time histories recorded at both top and base of each structure, as well as the initial and final iteration response acceleration of the LODM, are presented in Figure 5.24 and Figure 5.26 for white noise and harmonic chirp excitation signals, respectively. The convergence of the parameters as well as the number of iterations required are shown in Figure 5.25 and Figure 5.27 for both types of excitations.

The results displayed in Figure 5.24 show a fair agreement between the experimental data and the output of the LODM. The level of matching is particularly good for the data measured at the top of each structure, where both amplitude and frequency are well captured. In contrast, the response of the LODM at the base underestimates the amplitude of the signal while frequency is captured fairly well. Further, significant improvement is seen in the degree of matching of the final iteration with respect to the initial values. Additionally, the norm of the vector containing the difference between experimental and estimated data is reduced from the initial conditions in every measured point. Overall, it can be said that for certain structures their complex behaviour when subject to dynamic actions can be reproduced using LODM proposed herein.

The convergence of the parameters plots is displayed in Figure 5.25. A total of 300 iterations were set in the analysis. After approximately 250 iteration the model reaches stability and the convergence was reached for all the parameters. This points towards a well-conditioned problem. However, when compared with the single building case, a significantly greater number of iterations is required to reach convergence of all the parameters, indicating, that the greater the number of parameters, the greater the number of iterations required to reach parameters convergence.
Figure 5.24 Acceleration time histories of both top and base of each building when subject to white noise input excitation. Comparison between the physical model and LODM using shear type building to represent structures.
Figure 5.25 Convergence of the model parameters when subject to white noise input excitation. Results of the LODM using shear type building to represent both structures.

The experimental results for the case in which the input was a periodic chirp are presented in Figure 5.26. A portion of the acceleration time histories measured at both top and base of each building as well as their counterparts in the LODM for both initial and final iteration can be seen in Figure 5.26. The initial conditions where different to those used for the case of white noise excitation signal. For this case a fairly good agreement is found for the top of both structures, while not as good as that found in the case of white noise
excitation. The degree of matching for the base of the first building is excellent regardless of a slight overestimation of the amplitude. The worst results are obtained for the response at the base of the second structure, where the amplitude is significantly underestimated, and the frequency is not captured entirely well.

The convergence plots of the model parameters are depicted in Figure 5.27. It can be said that after approximately 200 iterations the final parameters are found. It is also noted that the SSI parameter of the first structure descends to very low values while that of the second building is significantly higher. This is caused since the stiffness of the base is not solely given by the SSI spring. Instead, it is also connected to the SSSI spring, which can provide additional support. In the case of multiple structures, the overall system is being optimised to match a certain target response, hence, the individual value of a parameter is not necessarily linked to the physical model characteristics.

At this stage it can be said that the selection of the input signal might have an effect in the time domain identification procedure, particularly when dealing with physical models, where the frequency content of the signal is dependent on the mounting of the excitation source-model system. Despite the complications that can be found during the testing of small-scale models, it can be said that the use of LODM is a valid alternative to significantly reduce the computational expenses that are demanded by HOFM when studying SSSI problems. It has been shown experimentally that the proposed LODM is able to reproduce the response of small-scale physical model when subject to dynamic force excitation with fair degree of agreement.
Figure 5.26 Acceleration time histories of both top and base of the building. Comparison between the physical model and LODM using shear type building to model the super structure.
Figure 5.27 Convergence of the model parameters. Results of the LODM using shear type building to represent the physical model.

In a similar way as the case of the single structure presented in Section 5.2 discrepancies exist between the LODM identified using a white noise and periodic chirp signals (See Figure 5.25 and Figure 5.27). To evaluate the accuracy of both LODMs, each model was
subjected to a different force excitation input signal than that imposed during the identification stage.

Figure 5.28 Acceleration time histories of both top and base of each building. Comparison between the LODM identified using a white noise signal and the physical model each subject to a periodic chirp force excitation signal.
Figure 5.29 Acceleration time histories of both top and base of each building.

Comparison between the LODM identified using a periodic chirp signal and the physical model each subject to a white noise force excitation signal.

Contrary to the case of the single structure presented in Section 5.2, both models, however different, seem to represent the experimental set-up fairly well. As pointed out in Chapter 3 Section 3.1 the proposed procedure does not rule out the existence of local minima. In this case both solutions seem to satisfy the requirements of the solution indicating the existence of at least two possible solutions (local minima) within the bounded problem. This obviously poses a limitation to the proposed procedure and further improvements
are required to guarantee the convergence to the absolute minimum of the penalty function.

5.4 Summary

In this Chapter a methodology to both make and test small scale models exhibiting SSI and SSSI effects has been developed. The models were used as a validation and verification (V&V) tool for the proposed identification procedures presented in Chapters 3 and 4.

An individual structure coupled with the soil model was subjected to dynamic force excitation using two different input signals: white noise and periodic chirp. The time-domain bounded variable identification procedure described in Chapter 3 was implemented using a shear type building for the LODM. The results displayed in Figure 5.17 and Figure 5.19 indicate that the LODM is able to account fairly well for the response of the physical model in the analysed frequency range. Significant improvement is seen in the degree of matching of the final iteration with respect to the initial values. Despite the significant improvement with respect to the initial conditions and the fair degree of agreement between the experimental response and the LODM, there are some discrepancies, especially, in the response at the base of the structure. One of the reasons for the discrepancies is the fact that the system is being excited just at the top of the structure, making the system very dependent on the elastic properties of the building. Moreover, the signal that arrives at the base of the structure is smaller in amplitude and has been filtered as it passes through the structure. Nevertheless, the ability to reproduce the complex behaviour of 3D structures by simplified LODM is very powerful.

It is noted that there are significant differences in the identified parameters of the LODMs (see Figure 5.18 and Figure 5.20) when the model is subject to two different types of input signals, namely, white noise and periodic chirp. This is a limitation of the proposed procedure. To evaluate the accuracy of both LODMs, each model was subjected to a different force excitation input signal than that imposed during the identification stage.

In the case of the single structure, the model identified using the white noise signal provided a better representation of the physical experiment with respect to the model identified using the periodic chirp signal. It is concluded that there could be local minima within the boundaries of the problem and that in this case different inputs yield two different solutions.
Similarly, a model with two adjacent structures coupled with and through the soil was subjected to dynamic force excitation at one of the structures. Two different input signals were used, namely, white noise and periodic chirp. The LODMs identified seem to provide a fair representation of the physical experiment behaviour with the response measured at the base being captured with less accuracy, particularly, for the model identified using the periodic chirp signal.

In a similar way as the case of the single structure presented in Section 5.2 discrepancies exist between the LODMs identified using a white noise and periodic chirp signals (See Figure 5.25 and Figure 5.27). To evaluate the accuracy of both LODMs, each model was subjected to a different force excitation input signal than that imposed during the identification stage.

Contrary to the case of the single structure presented in Section 5.2, both models, however different, seem to represent the experimental set-up fairly well. As pointed out in Chapter 3 Section 3.1 the proposed procedure does not rule out the existence of local minima. In this case both solutions seem to satisfy the requirements of minimising the penalty function indicating the existence of at least two possible solutions (local minima) within the bounded problem. This obviously poses a limitation to the proposed procedure and further improvements are required to guarantee the convergence to the absolute minimum of the penalty function.
6 Non-Conventional Vibration Control in the Urban Environment

Recent disasters in Mexico (2017), Ecuador (2016), Italy (2016) and Japan (2016) manifest the clear need to address the seismic resilience of existing buildings in a more efficient and affordable way. Structures are very seldom found in complete isolation. In fact, the opposite seems to be the most common case. Buildings are often surrounded by adjacent structures in most scenarios, particularly in densely populated urban environments. While the effect of the specific site conditions is widely known, recent research (Gueguen et al. 2002; Semblat et al. 2008; Kham et al. 2013; Schwan et al. 2016) has also indicated the possible modifications of the ground motion due to the presence of the vibrating structures. In light of new research and better understanding of the complex phenomena of multiple interactions that take place the urban environment, innovative solutions can now be envisaged. Given the pressing need to develop techniques that improve the seismic resilience of the urban environment a non-localised vibration control strategy applied to urban environment is proposed in this Chapter.

The urban environment is characterised by a diverse range of structures densely arranged in limited space. Additionally, buildings with heritage value tend to be preserved, while the built environment around them might change drastically over time. Due to the different materials, construction methods, design philosophy and structures use, the analysis of structures in the urban environment under earthquake ground motion is significantly more complicated than structures in isolation. Clusters of buildings undergoing dynamic excitation are characterised by complex interactions that take place over a wide range of frequencies. Hence, non-localised vibration control strategies that aim to address the buildings response reduction within the urban environment in a global manner should be designed following a holistic approach that encompasses the broadband response nature of the urban environment. Furthermore, in a reliability and resilience context, stochastic approaches are preferred for providing a probability-based appraisal of the potential damage caused by a given natural event.

The recently developed Vibrating Barrier (ViBa) (Cacciola and Tombari 2015) device shown promising results in its implementation to protect several structures undergoing
dynamic loading (Tombari et al. 2018). The ViBa is a massive structure buried in the soil consisting of a foundation and a vibrating spring-mass system. Importantly, the whole arrangement is completely detached from surrounding buildings. The device suitably exploits the SSSI phenomenon to minimise the response of neighbouring structures when subject to dynamic actions. The novel use of the ViBa device in the urban environment is explored herein.

In order to validate the capabilities of the software ADINA version 9.2.1 (Bathe 2016) as a tool to evaluate SSI, SSSI, and SCI problems, a comparative study to reproduce the results reported by Schwan et al. (2016) using ADINA version 9.2.1 (Bathe 2016) was undertaken. The comparisons reported a satisfactory agreement between the data reported by Schwan et al. (2016) and those generated using ADINA version 9.2.1 (Bathe 2016).

Furthermore, the urban environment model proposed by Semblat et al. (2008) is considered. The model consists of a 2D basin located in the centre of the city of Nice, France, with an idealized 2D city arrangement. Dynamically independent groups of buildings within the urban environment are identified according to relevant criteria described herein. A simplified LODM is developed using the time-domain identification procedure explored in Chapters 3 to 5. Forced vibration tests are used to generate the target data that feed into the time-domain identification procedure.

The stochastic design of the ViBa device is undertaken using the LODM. The design principle consists in the minimisation of the penalty function depending on the second-order statistical moments of the response. The ViBa device is appropriately implemented into the urban environment and its performance assessed for the 2001 Nice earthquake.

Finally, an additional case study focused on the improvement of the seismic resilience of the historical village of Vathia, in Greece, using the ViBa device is studied. Particularly, a simplified design approach of the ViBa device is considered. To this aim, a 2D FE model of a section of the Vathia village is suitably employed. It is well known that both the frequency content of earthquake ground motions and the response of urban environments are broadband in nature. Nevertheless, in some cases, as the one studied in this Chapter, given the local site conditions, the response of the studied system can be predominantly dominated by a few frequencies. In this Chapter, the seismic protection of the Vathia village is investigated following simplified approach in which the main frequency of the response is targeted, and the response of the whole village is effectively
reduced by tuning the ViBa to this particular frequency. The protection system was formed of an array of ViBa devices spread throughout the village, all of them calibrated at the same frequency.

6.1 Design of the ViBa parameters: Problem position

Consider the global system depicted in Figure 6.1 under ground motion excitation at the bedrock \( \ddot{u}_g(t) \). The ViBa devices are included aiming to reduce the vibration of the surrounding buildings.

Figure 6.1 Sketch of the simplified model of structure in urban environment protected by the ViBa.

The governing equations of the coupled dynamic system are derived in terms of absolute displacements, as it is conventional in soil-structure interaction, namely the dynamics of the problem take the form:

\[
M(\alpha)\dddot{u}(\alpha,t) + C(\alpha)\ddot{u}(\alpha,t) + K(\alpha)u(\alpha,t) = Q_e(\alpha)u_g(t) + Q_d(\alpha)\dot{u}_g(t)
\]  \hspace{1cm} (6.1)

where \( M(\alpha) \), \( C(\alpha) \) and \( K(\alpha) \) are the global mass, viscous damping and stiffness matrix, respectively; \( \dddot{u}(\alpha,t) \), \( \ddot{u}(\alpha,t) \) and \( u(\alpha,t) \) are respectively the absolute acceleration, velocity and displacement vector. Note that the ViBa dynamic behaviour is embedded in the global equation of motion through its structural parameters listed in the vector \( \alpha \) that represent the unknowns of the problem.

The matrices of the global system are partitioned in the sub-matrices defined for the individual buildings and the ViBa devices; therefore, the global mass matrix is as follows:
\[
\mathbf{M}(\alpha) = \begin{bmatrix}
M_i & 0 & \cdots & 0 & 0 \\
0 & M_i & \cdots & 0 & 0 \\
\vdots & \vdots & \ddots & \vdots & \vdots \\
0 & 0 & \cdots & M_n & 0 \\
0 & 0 & \cdots & 0 & M_V(\alpha)
\end{bmatrix}
\]  
(6.2)

in which the ith sub-block includes the mass of the ith structure, while \( M_V \) is the mass matrix of the ViBa devices distributed in the urban environment given by

\[
\mathbf{M_V}(\alpha) = \begin{bmatrix}
M_{V_1}(\alpha) & 0 & \cdots & 0 & 0 \\
0 & M_{V_2}(\alpha) & \cdots & 0 & 0 \\
\vdots & \vdots & \ddots & \vdots & \vdots \\
0 & 0 & \cdots & M_{V_j}(\alpha) & 0 \\
0 & 0 & \cdots & 0 & M_{V_m}(\alpha)
\end{bmatrix}
\]  
(6.3)

with

\[
\mathbf{M}_{V_j}(\alpha) = \begin{bmatrix}
m_{i_1}(\alpha) & 0 & \cdots & 0 & 0 \\
0 & m_{i_2}(\alpha) & \cdots & 0 & 0 \\
\vdots & \vdots & \ddots & \vdots & \vdots \\
0 & 0 & \cdots & m_{i_j}(\alpha) & 0 \\
0 & 0 & \cdots & 0 & m_{i_f}(\alpha)
\end{bmatrix}
\]  
(6.4)

where \( M_{V_j} \) is the mass matrix of an individual ViBa device, \( m_{i_j} \) are the masses of the internal structure of the ViBa and \( m_{i_f} \) is the mass of the ViBa’s foundation.

The global damping matrix \( \mathbf{C}(\alpha) \) and the global stiffness matrix \( \mathbf{K}(\alpha) \) are block-matrices partitioned in the following form:

\[
\mathbf{C}(\alpha) = \begin{bmatrix}
C_1 & C_{i_1} & \cdots & C_{i_n} & C_{i_V} \\
C_{i_1} & C_i & \cdots & C_{i_n} & C_{i_V} \\
\vdots & \vdots & \ddots & \vdots & \vdots \\
C_{i_n} & C_{n_i} & \cdots & C_n & C_{n_V} \\
C_{i_V} & C_{i_V} & \cdots & C_{V_n} & C_V(\alpha)
\end{bmatrix}
\]  
(6.5)

for the damping matrix, while:
for the stiffness matrix. The sub-matrices \( C_i \) and \( K_i \) (\( i = 1, \ldots, n \)) describe the viscous damping and stiffness matrix of the \( i \)-th structure and its interaction with the soil. The matrices \( C_v \) and \( K_v \) define the damping and stiffness matrix of the ViBa and its interactions to the other buildings through the soil. Lastly, the off-diagonal sub-matrices \( C_y \) and \( K_y \) (\( i, j = 1, \ldots, n \)) are related to the dynamic coupling between the \( i \)-th and the \( j \)-th structures while \( C_{iv} \) and \( K_{iv} \) (\( i, j = 1, \ldots, n \)) are related to the dynamic coupling between the \( i \)-th and the ViBa.

In the previous formulation, the structural parameters of the ViBa indicated by the generic vector \( \alpha \) represent the unknowns of the problem to be determined. Therefore, various optimization criteria can be used to determine the unknown parameters. In this study the optimization is defined by the minimization of the penalty function \( J(\alpha) \) in a least square sense, that is

\[
\min \{ J(\alpha) \}
\]

\[
\alpha = \{ K_v, M_v, C_v \} \in \mathbb{R}_0^+
\]  

where \( J(\alpha) \) is defined in terms of either relative displacements, internal forces, energy etc. The solution of equation (6.14) involves defining a FE model of the full urban environment. This is clearly impractical and computationally demanding, a viable alternative to overcome this is to define LODM as has been shown in Chapters 3 and 4. Therefore, the proposed procedure to design the ViBa device in an urban environment is summarised in the following steps

1. Dynamic response measurements of the structures of interest under forces excitation, either conducted directly in the field, physical experiments or using FE models.
2. Implementation of the bounded-variable time-domain identification procedure described in Chapters 3 and 4 to define a LODM representing the structures of interest.
3. Minimisation of the penalty function $J(\alpha)$, including the coupling terms of the ViBa with the respective structures.

4. Starting from the identified coupling parameters between the ViBa and the structures, define an optimum location of the ViBa.

5. In the case in which FE models are available, implementation of the ViBa design in the urban environment model and assessment of its performance under earthquake ground motion.

In the following, the above procedure is suitably employed for the design of the ViBa to reduce the response of a cluster of buildings in an urban environment.

### 6.2 Analysis tool validation

Finite element models (FEM) are widely used in engineering applications and recently have been proven a successful approach in the study of SCI (see section 2.3.2). This section intends to validate the usage of FE software ADINA (Bathe 2016) through comparisons with both closed form solutions and experimental tests data. The stratum was modelled under fully fixed conditions and its transfer functions was compared with the closed form solution presented in Kramer (1996). In the case of SSI, SSSI and SCI systems, the results of the study published by Schwan et al. (2016) were used.

The experimental setup of Schwan et al. (2016) consisted of a polyurethane foam block 1.76 m long, 2.13 m wide and 0.76 m high, to simulate the soil stratum (Figure 2.39). The foam properties were: mass density $\rho = 49$ kg/m$^3$; shear modulus $G = 55$ kPa; Poisson ratio $\nu = 0.06$; damping $\zeta = 4.9$ %; shear wave velocity $v_s = 33$ m/s. The fundamental frequency of the block was 9.36 Hz and 9.11 Hz in the X and Y direction (Figure 2.39), respectively. The structures were made of aluminium sheets 0.5 mm thick clamped by two aluminium angles forming a foundation, with a total length of 2.65 cm. The height of the oscillator was 18.4 cm, and the experiment was designed to recreate plain strain conditions (2D). The oscillators were 1.75 m long and could resonate in bending as a consequence of an out of plane motion, while remaining virtually inert when exited by an in-plane motion (Figure 2.39). The fundamental frequency of the structures was 8.45 Hz the experimental arrangement was attached to a shaking table responsible of providing the excitation. The motion consisted of a Ricker wavelet with central frequency of approximately 8 Hz. The details of the different cases analysed and main findings of Schwan et al. (2016) are presented in section 2.3.3.
A FEM of a 2D stratum was generated using the software ADINA and the results compared with the solutions presented by Kramer (1996), particularly for the case of one dimensional ground response of an uniform, damped stratum on rigid rock. Starting from the solution of the wave equation of an stratum with the shearing characteristics of a Kelvin-Voigt solid, the transfer function $F(\omega)$ presented in equation (6.8) was derived (Kramer 1996).

$$F(\omega) = \frac{1}{\sqrt{\cos^2(\omega H / v_s) + [\xi(\omega H / v_s)]^2}}$$

(6.8)

where $H$ is the depth of the stratum, $v_s$ is the shear wave velocity and $\xi$ is the damping ratio. The model was 2.13 m wide and 0.76 m deep. The stratum properties were: mass density $\rho = 49$ kg/m$^3$; shear modulus $G = 55$ kPa; Poisson ratio $\nu = 0.06$; damping $\xi = 4.9\%$; shear wave velocity $v_s = 33$ m/s. The model was made of 2D-solid elements with the addition of horizontal constraints to guarantee a 1D behaviour. Fully fixed conditions were also imposed at the base to simulate a rigid rock base. The model was subject to an acceleration time history represented by a sample of white noise signal with 0-40 Hz bandwidth and a time step $\Delta t = 0.0025$ s, leading to a sampling frequency of 400 Hz. The transfer function was computed dividing the Fourier transform of the absolute acceleration at the top of the stratum by the Fourier transform of the input signal. Figure 6.2 display the plots of the transfer function obtained using equation (6.8) (yellow curve) and that obtained with ADINA (blue curve). Despite the irregular peaks of the transfer function generated with ADINA, a satisfactory agreement can be observed between the two solutions.
Soil-Structure system

Given the proven capability of ADINA of reproducing the behaviour of a simplified arrangement for both stratum and structure, a 2D analysis of a stratum-structure system is studied. The results generated with ADINA are compared with those presented by Schwan et al. (2016). The FEM was made using 2D quadrilateral 9-nodes solid elements for both stratum and structure and is displayed in Figure 6.3. The model was meshed in a way that creates coincident nodes in the stratum-structure interface to guarantee a perfect bond between the two elements. The model was fully fixed at the base of the stratum and was subject to a mass proportional load through an acceleration time history defined by a sample of white noise time history with 0-40 Hz bandwidth and a time step $\Delta t = 0.0025$ s. The material properties were the same as those reported in Schwan et al. (2016).

![FE mesh for the case of 1 oscillator.](image)

The transfer function was computed dividing the Fourier transform of the acceleration at the top of the stratum, specifically at the location of the designated point ‘1’ in the Schwan et al. (2016) study (see Figure 6.4), by the Fourier transform of the input signal. Figure 6.5 displays the plots of the transfer function obtained with ADINA (orange dots) and those presented by Schwan et al. (2016) for the same arrangement, it can be observed that the results generated in ADINA can mimic fairly well the trends, both in frequency and amplitude, of the experimental setup studied by Schwan et al. (2016).
Figure 6.4 Instrumentation and measuring points of the Schwan et al. (2016) experimental setup.

Figure 6.5 Comparison between the modulus of the transfer functions of the experimental model after Schwan et al. (2016) and those reproduced in ADINA for the case of 1 structure.

Structure-Soil-Structure system

In a similar way to the case of the single structure, a multi-structure system was also analysed. Schwan et al. (2016) reported results of experiments performed with 5 structures, and Figure 6.6 depicts the FEM for this case. The modelling assumptions, materials and input are the same as the single structure case. Figure 6.7 displays the transfer functions generated with ADINA and those presented by Schwan et al. (2016). It
is evident that a perfect match is not achieved; however, the trend of the response is captured with a fair agreement, with a slight offset in frequency of around 0.2 Hz.

![FE mesh for the case of 5 oscillator.](image)

**Figure 6.6 FE mesh for the case of 5 oscillator.**

![Comparison between the modulus of the transfer functions of the experimental model after Schwan et al. (2016) and those reproduced in ADINA for the case of 5 structures.](image)

**Figure 6.7 Comparison between the modulus of the transfer functions of the experimental model after Schwan et al. (2016) and those reproduced in ADINA for the case of 5 structures.**

**Site-City system**

Finally, a system with 37 structures was also analysed, following the same modelling assumptions, materials and input as in the two previous cases (single structure and 5 structures). Figure 6.8 depicts the FEM with 37 structures. As in the previous two cases, the results were measured at the location of the point identifies as ‘1’ by Schwan et al.
Figure 6.9 presents the results of the transfer function calculated with ADINA and those reported by Schwan et al. (2016).

Figure 6.8 FE mesh for the case of 37 oscillator.

Figure 6.9 Comparison between the modulus of the transfer functions of the experimental model after Schwan et al. (2016) and those reproduced in ADINA for the case of 37 structures.

As in the case of 5 structures, it can be observed that a perfect match is not achieved, nevertheless, a good agreement is found, particularly for the second peak of the function. For the first resonant frequency of the system the FEM underestimates both the amplitude, by approximately 10%, and the frequency, by approximately 5%, of the response.
6.3 Urban environment model

In this section, the urban environment model proposed by Semblat et al. (2008) is considered. The model consists of a 2D basin located in the centre of the city of Nice, France, 2500 m wide, and the deepest part of the basin is 60 m. Figure 6.10 displays the FEM of the 2D basin. The built environment is formed by two types of buildings, B1S and B2S, arranged in a realistic fashion. Figure 6.11 depicts the FE model of the city, in which buildings of the type B1S are indicated in red, while buildings of the type B2S are depicted in blue. The model strata properties are summarised in Table 6.1, while the properties of the B1S and B2S buildings are presented in Table 6.2. The model was evaluated using modal superposition with a constant damping ratio of 5% for all the modes.

Table 6.1 Urban environment model stratum parameters

<table>
<thead>
<tr>
<th>Layer</th>
<th>Mass density [kg/m³]</th>
<th>Poisson ratio</th>
<th>Shear modulus [MN/m²]</th>
<th>Shear wave velocity [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basin</td>
<td>2000</td>
<td>0.25</td>
<td>80</td>
<td>200</td>
</tr>
<tr>
<td>Bedrock</td>
<td>2300</td>
<td>0.25</td>
<td>4328</td>
<td>1372</td>
</tr>
</tbody>
</table>

Table 6.2 Urban environment model building parameters

<table>
<thead>
<tr>
<th>Building type</th>
<th>Height [m]</th>
<th>Base [m]</th>
<th>Mass density [kg/m³]</th>
<th>Poisson ratio</th>
<th>E [MN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1S</td>
<td>40</td>
<td>15</td>
<td>250</td>
<td>0.3</td>
<td>166.4</td>
</tr>
<tr>
<td>B2S</td>
<td>30</td>
<td>10</td>
<td>250</td>
<td>0.3</td>
<td>374.4</td>
</tr>
</tbody>
</table>
Figure 6.10 FEM mesh of the 2D basin located in the centre of Nice, France.

Figure 6.11 FEM mesh of the 2D basin and realistic city located in the centre of Nice, France.

In order to demonstrate the validity of the FE representation made in ADINA of the site-city model proposed by Semblat et al. (2008), the transfer functions between the base of the bedrock stratum and the basin’s surface on the six different locations defined by Semblat et al. (2008) are compared with the those measured in the ADINA FE model at the same positions (see Figure 6.10). These results are summarised in Figure 6.12 and Figure 6.13. It is observed from both figures that for points P1 and P6 the transfer function is slightly overestimated, while for the remaining points they are in the same order of magnitude. Furthermore, it seems that the results generated in ADINA do not capture all the peaks of the response, especially for frequencies beyond 3 Hz. Nevertheless, a fair agreement is noticed between both models, particularly in the location of the main peaks of the response. Moreover, a similar trend of the effects caused by the built environment in the modifications of the free-field ground motion is evidenced by both models.
Figure 6.12 Comparison of the transfer functions at points P1 to P3 for the case of free field and realistic city arrangement between Semblat et al. (2008) and the FE ADINA model.
Figure 6.13 Comparison of the transfer functions at points P4 to P6 for the case of free field and realistic city arrangement between Semblat et al. (2008) and the FE ADINA model.

Despite these discrepancies, it can be said that the FE model developed using ADINA (Bathe 2016) is able to capture the global behaviour fairly well. This model will be used in the following section to design the ViBa to protect a cluster of buildings in the urban environment.
6.4 Cluster selection criteria

In order to design the vibrating barrier to protect a given group of buildings within the urban environment, dynamically independent clusters of buildings need to be selected. In the framework of SSSI, it is understood that a “dynamically independent cluster” is a group of buildings that possess weak SSSI coupling between buildings at the boundaries of the unit and adjacent structures outside the cluster.

Consider the group of buildings depicted in Figure 6.14. the selected assemblage comprises five buildings, three B1S and two B2S type structures. In order to designate the boundaries of the cluster different types of criteria could be established to account for the strength of the SSSI coupling, namely, distance, frequency range, aspect ratio of the structures, height ratio between structures, etc. This section focuses on the use distance and type of building as parameters to determine the limits of dynamically independent clusters.

Figure 6.14 Detail of the observed cluster in the FE model of the 2D basin located in the centre of Nice, France.

A parametric study of the three possible cluster boundaries scenarios was undertaken. Models containing two B1S, two B2S and a combination of B1S-B2S structures were studied, considering different inter-building distances. To this aim, the models are evaluated in the frequency domain and results reported in terms of power spectral densities.

The different models are subject to ground motion modelled as stationary Gaussian zero-mean processes with unitary power spectral density. Results are compared against the isolated structure case in order to assess the variation of the SSSI coupling with distance and type of building. Figure 6.15 presents the three different possible scenarios for an
inter-building distance of 10 m, additional cases with 15 m, 20 m and 30 m were also studied.

Figure 6.15 Three different configurations used in the parametric study.

The models depicted in Figure 6.15 were subject to steady state analysis under ground motion simulated by a unitary flat spectrum signal with a frequency range of 0 – 10 Hz. Results are reported in terms of the power spectral density (PSD) of the acceleration in the X direction.

The results of the case of two identical B1S buildings are summarised in Figure 6.16. The PSD functions are calculated at the base and top of both buildings. It can be seen that very little change occurs in either structure at the top when the second building is present. On the other hand, results reported at the base display some differences in the amplitude of the PSD, especially for close distances. As the distance increases the coupling decreases as well, up to a point where the response PSD when the inter-building distance is 30 m becomes very similar to the isolated case. In addition, results for both structures seem to be very similar. Finally, it can be concluded that should the boundary of the cluster be formed of two adjacent B1S buildings the distance criteria could be applied, considering weak SSSI coupling for distances greater than 30 m.
Analyses results for the case of two adjacent B2S type buildings are displayed in Figure 6.17. Also, in this case, effects are mainly seen at the base of both structures, while very little change is observed at the top of the buildings. It is noted that the coupled frequency is slightly lower than the frequency of the isolated structure. The PSD at the base is modified in both frequency and amplitude as a function of the distance. Distances above 30 m seem to show weak SSSI coupling between buildings, and this could therefore be considered as a criteria to define dynamically independent clusters. Moreover, it looks like, when dealing with structures of the same type on a medium homogeneous soil stratum characterised by a shear wave velocity Vs of 250 m/s, the SSSI coupling is governed solely by the inter-building distance. Hence, for the soil conditions considered in this study, a cluster can be considered dynamically independent from neighbouring buildings if these are of the same type and are at a distance greater than 30 m.
Figure 6.17 Acceleration PSD for the case of two adjacent B2S buildings.

The results of the final case in which two adjacent B1S and B2S buildings are considered are depicted in Figure 6.18. Again, no observable changes can be detected at the top of either structure. On the other hand, the presence of building B2S is easily detected by looking at the PSD of the B1S building where in the vicinities of the resonant frequency of building B2S an increase in power is noticeable. However, very little change occurs in the PSD of building B2S due to the presence of building B1S. Therefore, an additional criteria to delimit the boundaries of a clusters can be established as follow: if the boundaries are formed by sequence of a B2S and B1S building, the building B2S can be considered the limit of the cluster, since the presence of building B1S will have very little effect on B2S. In addition, it is important to highlight that the opposite does not hold. Moreover, it is worth mentioning that this parametric study is case specific, and it has been undertaken with the sole purpose of delimiting dynamically independent clusters for design purposes. Detailed parametric studies are out of the scope of the present study. More general results are given in Alexander et al. (2013) in which the SSSI coupling is accounted for using rotational springs.
Finally, it can be concluded that from the group of buildings depicted in Figure 6.14, the three central structures (B2S-B1S-B2S) can be considered a dynamically independent cluster of buildings in an SSSI framework, and these results are in agreement with those reported by Padrón et al. (2009) when analysing similar configurations.

6.5 LODM Time-domain cluster identification

With the aim of designing the ViBa following a stochastic approach and to keep computational demands low, a LODM is defined from the HODM depicted in Figure 6.14 using time-domain identification techniques described in Chapter 4. To this aim, the dynamically independent cluster formed of a succession of B2S-B1S-B2S buildings (see Figure 6.19) is used. To aid the understanding of results, buildings are numbered from 1 to 3 from left to right.
The HODM was subject to a force excitation applied at the base of the central B1S building of the selected cluster. The presence of the urban environment during the test is stressed, so to mimic a realistic case scenario, in which dynamic tests can be conducted in existing building within a urban environment. The input signal consists of a sample of white noise with a 0–10 Hz bandwidth, time discretisation interval of 5E-3 and total duration of 5 s. Displacement time histories from the base and top of the three buildings were extracted from the HODM model. The response of the buildings was used as target for the time-domain identification procedure proposed in Chapter 4. The use of the weighted procedure with the parameter $\eta$ proposed in Section 4.3 was successfully implemented to improve the numerical conditioning of the problem. Given the greater number of parameters involved in this problem, the scope was limited and, the proposed model presented in Chapter 4 to account for wave propagation effects has not been considered.

The simplified model proposed in Chapter 4 using TDOF systems to model the structures was adopted for simplicity. The full LODM assemblage is depicted in Figure 6.20. The only parameters assumed to be known are the masses of the superstructure, while all the remaining parameters are identified to match the target response of the HODM system. Emphasis is given to the determination of the foundation mass to accounts for site effects. The mass and stiffness matrices of the system are given in equation (6.9) and (6.10).
Figure 6.20 LDOM of the cluster of buildings in the city of Nice.

\[
M = \begin{bmatrix}
  m_1 & 0 & 0 & 0 & 0 & 0 \\
  0 & m_{f,1} & 0 & 0 & 0 & 0 \\
  0 & 0 & m_2 & 0 & 0 & 0 \\
  0 & 0 & 0 & m_{f,2} & 0 & 0 \\
  0 & 0 & 0 & 0 & m_3 & 0 \\
  0 & 0 & 0 & 0 & 0 & m_{f,3}
\end{bmatrix}
\]

(6.9)

\[
K = \begin{bmatrix}
  k_1 & -k_1 & 0 & 0 & 0 & 0 \\
  -k_1 & k_{c,1} & 0 & -k_{s_u1,2} & 0 & 0 \\
  0 & 0 & k_2 & -k_2 & 0 & 0 \\
  0 & -k_{s_u1,2} & -k_2 & k_{c,2} & 0 & -k_{s_u2,3} \\
  0 & 0 & 0 & 0 & k_3 & -k_3 \\
  0 & 0 & 0 & -k_{s_u2,3} & -k_3 & k_{c,3}
\end{bmatrix}
\]

(6.10)

with

\[
k_{c,1} = k_{s_u1,1} + k_{s_u1,2} + k_1 \\
k_{c,2} = k_{s_u2,2} + k_{s_u2,3} + k_{s_u2,2} + k_2 \\
k_{c,3} = k_{s_u3,3} + k_{s_u2,3} + k_3
\]

(6.11)

In order to find which parameters dominate the response, and hence decide whether a single stage or a multistage solution needs to be applied, the displacement time histories at the top and base of every structure are analysed. Figure 6.21 presents these results. Both soil and structural parameters have influence in the response of the system, this
scenario is referred as comparable influence of the parameters, and it can be solved following a single-stage approach (see Chapter 4).

Figure 6.21 Displacement time history at the top and base of every building in the cluster when subject to forced excitation applied at the base of building 2, HODM results.

The single-stage time-domain identification procedure, described in Chapter 3, is implemented using the measured data from the numerical model. The resultant parameters defining the model can be found in Table 6.3 to Table 6.5. Response displacement time histories of both HODM and LODM are presented in Figure 6.22. Convergence plots of the sought parameters are depicted in Figure 6.23 and Figure 6.24.

Table 6.3 Mass parameters of the lumped discrete model of cluster of buildings

<table>
<thead>
<tr>
<th>Units/Parameter</th>
<th>( m_1 )</th>
<th>( m_2 )</th>
<th>( m_3 )</th>
<th>( m_{f,1} )</th>
<th>( m_{f,2} )</th>
<th>( m_{f,3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>kg</td>
<td>7.50E4</td>
<td>1.50E5</td>
<td>7.50E4</td>
<td>8.32E5</td>
<td>4.08E5</td>
<td>2.91E6</td>
</tr>
</tbody>
</table>

Table 6.4 Stiffness parameters of the lumped discrete model of cluster of buildings coupled

<table>
<thead>
<tr>
<th>Units/Parameter</th>
<th>( k_1 )</th>
<th>( k_2 )</th>
<th>( k_3 )</th>
<th>( k_{ssi,1} )</th>
<th>( k_{ssi,2} )</th>
<th>( k_{ssi,3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/m</td>
<td>7.68E6</td>
<td>5.90E6</td>
<td>1.02E7</td>
<td>6.61E7</td>
<td>3.70E7</td>
<td>1.15E8</td>
</tr>
</tbody>
</table>

Table 6.5 Cross-Interaction Stiffness parameters of the lumped discrete model of cluster of buildings coupled

<table>
<thead>
<tr>
<th>Units/Parameter</th>
<th>( k_{ssi1,2} )</th>
<th>( k_{ssi2,3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/m</td>
<td>2.05E7</td>
<td>6.64E7</td>
</tr>
</tbody>
</table>
Figure 6.22 Displacement time history at the top and base of every building in the cluster when subject to forced excitation applied at the base of building 2. Comparison between initial and final iteration of the LODM and the target response from the HDOM.
Figure 6.23 Convergence of the structure’s lateral stiffness and interaction springs.

It is observed from Figure 6.22 that there is a fair degree of agreement between the target response and the final iteration of the identification procedure results. Significant improvement from the initial iteration is achieved. This is further evidenced in the significant reduction in the norm of the vector containing the difference between the target and LODM response. Moreover, the convergence of the parameters in Figure 6.23 and Figure 6.24 indicates a well-conditioned problem achieved by the incorporation of weighing matrices.
Figure 6.24 Convergence plots of the foundation mass of the three structures.

In Figure 6.25 the validity of the LODM is corroborated by comparing the results of the HODM when subject to ground motion excitation in the presence of the urban environment and the response of the LODM when subject to the same actions. The ground motion comprises the acceleration records of the 2001 Nice earthquake.

Relative displacement between top and base of each structure in the cluster calculated using the HODM and the LODM are depicted in Figure 6.25. It is observed that a fair agreement between both models is achieved, especially for structure 2. Less satisfactory results are reported in the case of building 1 and 2 in which both frequency and amplitude seem to be underestimated. Given the significant reduction in complexity of the LODM compared to the HODM without a significant loss of accuracy, the LODM is used for the stochastic design of the ViBa in the urban environment.
6.6 Proposed ViBa design procedure

This section is devoted to the development of a procedure to design a single ViBa device to protect a cluster of buildings subject to ground motion modelled as stationary Gaussian zero mean processes with unitary power spectral density. To this aim, the penalty function $J(\alpha)$ is defined in terms of the second order statistical moments. The minimisation of the penalty function can be made in a least-square sense by appropriately modifying the Moore-Penrose pseudo-inverse presented in equation (3.5). If the vector $\mathbf{x}_w$ listing the target data is selected as a null vector, aiming to converge to an optimal solution where most of the input energy is absorbed by the ViBa, so to protect the whole cluster, the following holds

$$a_{i+1} = a_i - \left[ S_{u_i}^T S_{u_i} \right]^{-1} S_{u_i}^T (x_i)$$

(6.12)

where $a_i$ represent the vector of unknown parameters at the iteration $i$ and $x_i$ is the vector listing the model response parameters for the parameter set $a_i$ to be minimised. It is important to mention that any parameter that has an influence in the model response can be included in the unknowns parameter vector $a_i$. However, two important aspects need to be considered, namely: the parameters must have the same order of magnitude for the problem not to become ill-conditioned, otherwise scaling factors should be introduced to improve the numerical conditioning of the problem; secondly, the procedure is based on the assumption that equation (6.12) represents a combination of independent linear equations. If a parameter has none or very little influence in the measured model response vector $x_i$, the problem becomes ill-conditioned and additional measures need to be taken (see Friswell and Mottershead (1995)). In this study the vector $x$ is defined in terms of the second order stationary response statistical moments, that is:

$$x = P m_y^{(2)}(\alpha)$$

(6.13)

where $Y(\alpha,t)=[u(\alpha,t) \dot{u}(\alpha,t)]^T$ is the $2n \times 1$ state space vector, $P$ is a matrix whose elements are either null or unitary so to extract from the second order statistical moments of the response vector $m_y^{(2)}(\alpha)$ the relevant quantities. By considering the ground motion at the bedrock as zero-mean Gaussian stationary white noise input process, with power spectral density $S_w$, the evolution of the in the nodal space of the second-order statistical moments of the response is ruled by the following ordinary set of differential equations:
\[ m_{v}^{(2)}(a,t) = D_2(a)m_{v}^{(2)}(a,t) + 2\pi S_w \bar{V}^{[2]}(a) \]  

(6.14)

where

\[ m_{v}^{(2)}(a,t) = E[Y(a,t) \otimes Y(a,t)] \]  

(6.15)

\[ D_2(a) = D(a) \otimes I_{2n} + I_{2n} \otimes D(a) \]  

(6.16)

and \( I_{2n} \) is the identity matrix of order \( 2n \times 2n \). \( D(a) \) is defined in equation (3.14) and

\[ \bar{V}^{[2]}(a) = V(a) \otimes V(a) \]  

(6.17)

In equations (6.15) to (6.17), the \( \otimes \) operator stands for the Kronecker product.

The stationary response of the second order statistics is given by the following equation:

\[ m_{v}^{(2)}(a,t) = m_{v}^{(2)}(a) = -D_2^{-1}(a)2\pi S_w \bar{V}^{[2]}(a) \]  

(6.18)

The sensitivity of the stochastic response respect to the vector of the parameters \( a \) is determined as follows:

\[ S_{y}^{(2)}(a,t) = \frac{\partial m_{v}^{(2)}(a,t)}{\partial a} = E[S_{y}(a,t) \otimes Y(a,t)] + E[Y(a,t) \otimes S_{y}(a,t)] \]  

(6.19)

Therefore, the evolution of the sensitivity can be written in the form

\[ \dot{S}_{y}^{(2)}(a,t) = A_2(a)m_{v}^{(2)}(a,t) + D_2(a)S_{y}^{(2)}(a,t) + B_2(a)2\pi S_w \]  

(6.20)

with

\[ A_2(a) = A(a) \otimes I_{2n} + I_{2n} \otimes A(a) \]  

(6.21)

\[ B_2(a) = B(a) \otimes \bar{V}(a) + \bar{V}(a) \otimes B(a) \]  

(6.22)

where \( A(a) \) and \( B(a) \) are defined as

\[ A(a) = \frac{\partial D(a)}{\partial a} ; \quad B(a) = \frac{\partial V(a)}{\partial a} \]  

(6.23)

The stationary solution of equation (6.20) can be then written in the following form:

\[ S_{y}^{(2)}(a) = -D_2^{-1}(a)\left[A_2(a)m_{v}^{(2)}(a,t) + B_2(a)2\pi S_w \right] \]  

(6.24)

therefore,
Finally, the procedure also allows for the pertinent selection of boundaries to the solution to avoid unrealistic results.

6.7 SDOF Vibrating Barrier for cluster of buildings

This section focuses on the stochastic design of the ViBa aimed to protect a cluster of buildings. The selected group of buildings corresponds to the system that has been identified through time-domain identification techniques in Section 6.5. The procedure of the stochastic design of the ViBa has been presented in Section 6.1. Figure 6.26 depicts the discrete model of a cluster of buildings coupled with a SDOF vibrating element ViBa. The dynamic response of this system is governed by equation (6.1), where the mass, damping and stiffness matrix are of the form expressed in equations (6.2), (6.5) and (6.6) respectively.

![Figure 6.26 LODM of a cluster of buildings coupled with a SDOF ViBa device.](image-url)
For this case both the mass and damping ratios are assumed to be known. The mass and stiffness matrix are presented in explicit form in equations (6.26) and (6.27) while the damping matrix is calculated as a back transformation from the modal space assuming all the modes have the same damping equal to $\zeta_n = 0.05$. The vector of unknown parameters $\alpha$ is formed solely of the unknown stiffness parameters, namely, $k_1, k_{ss1}, k_{ss2}$ and $k_{ss3}$. A parametric study is conducted to identify the influence of the parameter $k_{ss,v}$ and selected accordingly.

$$M = \begin{bmatrix} m_1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & m_{j,1} & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & m_2 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & m_{j,2} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & m_3 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & m_{j,3} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & m_{j,v} \\ 0 & 0 & 0 & 0 & 0 & 0 & m_n \end{bmatrix}$$  

(6.26)

$$K = \begin{bmatrix} k_1 & -k_1 & 0 & 0 & 0 & 0 & 0 \\ -k_1 & k_{c,1} & 0 & -k_{ss1,2} & 0 & 0 & 0 & -k_{ssv,1} \\ 0 & 0 & k_2 & -k_2 & 0 & 0 & 0 & 0 \\ 0 & -k_{ss1,2} & -k_2 & k_{c,2} & 0 & -k_{ss2,3} & 0 & -k_{ssv,2} \\ 0 & 0 & 0 & 0 & k_3 & -k_3 & 0 & 0 \\ 0 & 0 & 0 & -k_{ss2,3} & -k_3 & k_{c,3} & 0 & -k_{ssv,3} \\ 0 & 0 & 0 & 0 & 0 & 0 & k_y & -k_y \\ 0 & -k_{ssv,1} & 0 & -k_{ssv,2} & 0 & -k_{ssv,3} & -k_y & k_{c,4} \end{bmatrix}$$  

(6.27)

with

$$k_{c,1} = k_{ss1} + k_{ss1,2} + k_{ssv,1} + k_1$$
$$k_{c,2} = k_{ss2} + k_{ss1,2} + k_{ss2,3} + k_{ssv,2} + k_2$$
$$k_{c,3} = k_{ss3} + k_{ss2,3} + k_{ssv,3} + k_3$$
$$k_{c,4} = k_{ss4} + k_{ssv,1} + k_{ssv,2} + k_{ssv,3} + k_4$$  

(6.28)
6.7.1 Boundaries of the design parameter

In order to determine the boundaries of the variables to be optimised, the impact of the static contribution of the ViBa is assessed. It is understood by static contribution the provision of additional stiffness to the cluster by the presence of the ViBa, which causes modification in the frequency distribution of the system. Evaluation of the static effects is conducted by varying the stiffness of the SSSI and SSI coupling springs linked to the ViBa. Three different values of the \( k_{SSI,SS} \) are considered, namely, 1E6, 1E7 and 1E8 N/m. For each of this cases four different values of the parameters \( k_{SSI,1}, k_{SSI,2} \) and \( k_{SSI,3} \) were studied, namely, 1E6, 1E7, 1E8 and 1E9 N/m. The mass properties of the ViBa were kept constant and can be found in Table 6.6. To this aim, the models are evaluated in the frequency domain and results reported in terms of power spectral densities. The governing equation of motion (equation 6.1) can be rewritten in the frequency domain as follows:

\[
K_{dyn}(\omega)U(\omega) = \left( Q_e(a) + i\omega Q_d(a) \right) U_{\bar{g}}(\omega) \tag{6.29}
\]

where \( K_{dyn}(\omega) = K(a) + i\omega C(a) - \omega^2 M(a) \) is the dynamic stiffness matrix, \( a \) is the design parameters vector and \( i = \sqrt{-1} \) is the imaginary unit. The response in the frequency domain for a single realization \( U_{\bar{g}}(\omega) \) can be readily determined as follows

\[
U(\omega,\omega) = H(\omega,\omega) U_{\bar{g}}(\omega) \tag{6.30}
\]

where the transfer function \( H(\omega,\omega) \) is given by the following equation

\[
H(\omega,\omega) = K^{-1}_{dyn}(\omega)(\omega^2 M(a)) \tag{6.31}
\]

The power spectral density function is then determined

\[
G_{UU}(\omega,\omega) = H(\omega,\omega) H^*(\omega,\omega) G_{U\bar{g}U\bar{g}}(\omega) \tag{6.32}
\]

where the asterisk means transpose complex conjugate.

The different models are subjected to ground motion modelled as stationary Gaussian zero-mean white noise process with unitary power spectral density function. Results are depicted in Figure 6.27 to Figure 6.29. In all the analyses, both the mass of the foundation
and resonant element of the ViBa was assumed to be negligible, to focus just on the static contribution provided by the springs connecting the ViBa to the different elements of the cluster. The static effects can be assessed in terms of the change in the area underneath the PSD, shifts in frequency and changes in shape. It is observed from Figure 6.27 that coupling springs between the ViBa and the buildings of the cluster with stiffness greater or equal to 1E8 N/m produce important modifications in the dynamic response of building 1 while just moderate changes are introduced for building 3 and very little impact is caused in the central structure building 2.

<table>
<thead>
<tr>
<th>Units/Parameter</th>
<th>$m_v$</th>
<th>$m_{f,v}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>kg</td>
<td>5.75E5</td>
<td>1.00E3</td>
</tr>
</tbody>
</table>

Furthermore, a stiffness of 1E6 N/m for the coupling springs between the ViBa and the buildings of the cluster has imperceptible effects on the dynamic response of all the buildings in the cluster. Finally, a coupling stiffness of 1E7 N/m seems to have a negligible effect on both building 2 and 3, while noticeable changes are introduced in the response PSD of building 1.

The results of the case in which the $k_{ss,v}$ parameter is equal to 1E7 are presented in Figure 6.28. The trend in the results is very similar to the case in which $k_{ss,v}$ is equal to 1E6 with slightly greater impact in buildings 2 and 3 for coupling stiffness equal or greater than 1E8 N/m. Finally, the response PSD of the $k_{ss,v}$ equal to 1E8 case are depicted in Figure 6.29. Further modifications are observed in all structures for coupling stiffness equal or greater than 1E7 N/m, significant changes are seen in all structures for values above 1E8.
Figure 6.27 Power spectral density of the relative displacement of each building for a $k_{sv} = 1E6$ N/m.
Figure 6.28 Power spectral density of the relative displacement of each building for a $k_{s,v} = 1E7N/m$. 
Figure 6.29 Power spectral density of the relative displacement of each building for a \( k_{ssi,v} = \text{1E8 N/m} \).

Consider now the response of building 1 for the second set of coupling stiffness values particularly \( \text{1E8 N/m} \) displayed in the right side of the three previous figures as the red dotted line. This particular case illustrates one of the consequences of not setting appropriate boundaries for the sought variables, since as it can be seen, important reductions in the power of the response for building 1 are achieved by modifying the frequency response of the system adding additional source of stiffness to it, hence
providing stronger coupling with it neighbouring structures. This is a fictitious effect that cannot be reproduced by the ViBa in reality. Hence, it is not beneficial for the design. It is often the case that when there is stronger coupling between structures power decrease in one structure translates into increase of power in adjacent structures. This is noticeable, for instance, in the plots of the left in Figure 6.27 to Figure 6.29.

6.7.2 Design of the Vibrating Barrier device

The ViBa design was undertaken aimed to protect three buildings within the urban environment. The design criteria encompass minimising the static modification of the built environment. To this aim, a bound-variable least-square minimisation procedure is suitably implemented. Considering the particular aspects of the dynamic response of the cluster, it can be observed the significant difference that exists between the amplitude of the PSD of building 2 compared with the remaining two structures. This introduces additional constraints into the design process, since the unbounded minimisation of the response parameters will unavoidably aim to reduce the response of building 2 rather than that of buildings 1 and 3. If the appropriate boundaries are set in place, the solution can be shifted towards one direction or the other. Additionally, the frequency gap between structures also plays a primary role in finding the optimum solution. Buildings that have their resonant frequencies in the same range of influence of the ViBa may benefit from the protection provided in that band. Nevertheless, if like in this case the three structures have different resonant frequencies, additional modifications need to be introduced to the internal configuration of the ViBa device to be effective in a wider range of frequencies. These modifications are explored in the following subsection.

To study further the effect of the disproportionate difference in power among the buildings in the cluster, consider a scenario in which the coupling between buildings and the ViBa is identical and in this case equivalent to a spring with a stiffness of 1E7 N/m.

The objective of this numerical study is to apply the procedure presented in Section 6.6 to determine the optimal parameters of the ViBa so to minimize the stochastic response (in terms of second-order statistical moments) of the three buildings. In this regard, the selected elements of the response 3x1 vector $\mathbf{x}$ are the statistical moments of the relative displacements between the top and base of the structure, that is:
\begin{align*}
x(1) &= m_{11}^{(2)}(\alpha) + m_{22}^{(2)}(\alpha) - m_{12}^{(2)}(\alpha) - m_{21}^{(2)}(\alpha) \\
x(2) &= m_{33}^{(2)}(\alpha) + m_{44}^{(2)}(\alpha) - m_{34}^{(2)}(\alpha) - m_{43}^{(2)}(\alpha) \\
x(3) &= m_{55}^{(2)}(\alpha) + m_{66}^{(2)}(\alpha) - m_{56}^{(2)}(\alpha) - m_{65}^{(2)}(\alpha)
\end{align*}

(6.33)

in which \( m_{ii}^{(2)}(\alpha) \) \((i = 1 \ldots 6)\) are the second order response statistical moments of the \( i \)th degree of freedom while \( m_{jk}^{(2)}(\alpha) \) \((j, k = 1 \ldots 6 \ j \neq k)\) are the cross statistical moments of the response between the \( j \)th and \( k \)th degrees of freedom.

Table 6.7 lists the mechanical parameters of the ViBa assigned a priori and not included in the optimization procedure.

Table 6.7 Soil-structure interaction and cross interaction stiffness parameters of the ViBa device

<table>
<thead>
<tr>
<th>Units/Parameter</th>
<th>( k_{x,m} )</th>
<th>( k_{x,m1} )</th>
<th>( k_{x,m2} )</th>
<th>( k_{x,m3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/m</td>
<td>1.00E7</td>
<td>1.00E7</td>
<td>1.00E7</td>
<td>1.00E7</td>
</tr>
</tbody>
</table>

Due to the simple configuration of the SDOF ViBa device the vector parameter \( \alpha \) reduces to a scalar value representing the single unknown to be determined i.e. the stiffness parameter \( k_x \).

Modification factors \( \gamma_k \), where the index \( k \) is the building identifier, are introduced in order to shift the solution towards a particular structure within the cluster. In other words, they can make all the buildings to have the same influence on the solution or can be appropriately selected to make a particular structure more important in the solution procedure. To illustrate this, four cases are analysed, namely: original case, in which no modification factors are applied, followed by three cases in which each structure is targeted individually.

Figure 6.30 and Figure 6.31 present the results in terms of displacement PSD, for the four scenarios considered. The effect of the ViBa is assessed in terms of the percentage difference in area of the relative displacement PSD function, referred as \( \Delta \), between the case with and without the ViBa.

\[
\frac{\int_{0}^{\infty} G_{\text{ViBa}}(\alpha, \omega) - \int_{0}^{\infty} G_{\text{ViBa},0}(\alpha, \omega)}{\int_{0}^{\infty} G_{\text{ViBa},0}(\alpha, \omega)}
\]

(6.34)
where $G_{0tr} (a, \omega)$ is the power spectral density function of the system without the ViBa and $G_{0tr,s} (a, \omega)$ is the power spectral density function of the system with the ViBa.

As expected, it is noticed in Figure 6.30 that the solution of the minimisation problem with no modification factors results in significant reductions in the central structure (Figure 6.30 left). Nevertheless, the design yields results that are beneficial for all three structures. The solution when a modification factor is applied to shift it towards protecting building one are displayed at the right in Figure 6.30. It is observed that a higher level of protection is achieved compared to the previous case. In addition, reductions in building three are also noticed even though the solution is mainly intended for building one. This effect is due to the inability of the coupled ViBa system to achieve the frequency required to protect only building one. Regardless of the increase in the stiffness of the vibrating mass within the ViBa, the frequency of the coupled system does not increase. In this particular case, the whole ViBa is vibrating at the same frequency. Furthermore, detrimental effects occur in building two, where the power of the relative displacement in the presence of the ViBa is greater than the unprotected system.

Similar results are depicted in Figure 6.31. On the left the solution for building two is presented. It seems that significant reductions are achieved in the central structure, while no detrimental effects are observed in the remaining structures. Further, the case of building three as main target of the design are depicted at the right of Figure 6.31, a significant reduction is achieved in building three, moderate improvement in building one and the effects are detrimental in building two. It appears that, if the aim is to protect both the buildings one and three, detrimental effects occur in building two. Finally, a case in which the modification factors $\gamma_i$ have been chosen to generate a scenario in which all the response statistical moments have the same weight in the penalty function was analysed and the results are presented in Figure 6.32. This solution yields the overall most satisfactory results, causing simultaneous reductions in all the three structures. The optimum values of the parameter $k_i$ for each case analysed are summarised in Table 6.8.

An additional dimension of possible static contributions needs to be considered, particularly when LODMs are being used for the design. Static contributions can be provided by additional coupling through the ViBa. Consider the model depicted in Figure 6.26. Initially, before the ViBa is added to the system, the foundations of building one and building three are not directly coupled; however, once the ViBa is added to the system
and provided that the coupling between the ViBa and the buildings is stiff enough, the foundations of building one and three could be coupled through the ViBa. Moreover, let assume that the foundation of building one is significantly stiffer than that of building three. Regardless of the value of $k_{w_{1,1}}$, additional coupling can be provided through $k_{w_{1,2}}$ and $k_{w_{2,2}}$.

Figure 6.30 Optimum ViBa design case with no modification factors (left) and modification factor $\gamma_1$ to shift solution towards building 1 (right).

Bearing these considerations in mind the boundaries of the variables are suitably established to avoid the influence of static effects; however, no additional constraints are
introduced as to force the solution to be driven toward the protection of any particular structure within the cluster. Furthermore, it is important to highlight that the mass of the ViBa’s foundation needs to be selected considering that the vibrating element of the ViBa is able to reach the frequencies required in order to protect the adjacent structures. If the coupled foundation frequency of the ViBa is lower than that required for protection, the device will not be effective.

Figure 6.31 Optimum ViBa design with modification factor $\gamma_2$ to shift the solution towards building 2 (left) and modification factor $\gamma_3$ to shift solution towards building 3 (right).
Figure 6.32 Optimum SDOF ViBa design with all the buildings having the equal weight in the solution.

<table>
<thead>
<tr>
<th>Case</th>
<th>Units/Parameter</th>
<th>k_v</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original</td>
<td>N/m</td>
<td>3.24E7</td>
</tr>
<tr>
<td>Target 1</td>
<td>N/m</td>
<td>5.67E8</td>
</tr>
<tr>
<td>Target 2</td>
<td>N/m</td>
<td>3.67E7</td>
</tr>
<tr>
<td>Target 3</td>
<td>N/m</td>
<td>1.44E8</td>
</tr>
<tr>
<td>Equal influence</td>
<td>N/m</td>
<td>4.44E7</td>
</tr>
</tbody>
</table>

Additionally, the protection capabilities of the ViBa are directly linked to its inertial properties. Normally, within an appropriate range, the larger the mass, the greater the effectiveness of the device in reducing the response of adjacent structures. The suitable definition of the upper boundary of the resonant element mass within the ViBa depends on the foundation stiffness, since the coupled system needs to be able to reach the frequency range of the neighbouring built environment and, as pointed out previously, there are limitations on the coupling stiffnesses. Optimum design results of the ViBa for two different masses of 5.75E4 and 5.75E5 kg are presented in Figure 6.33. The ViBa’s foundations in virtually massless, while the $k_{x,v}$ was fixed at 1E7 N/m and the sought parameters were limited at 1E6 for $k_{x,1}$ and 5E7 for $k_{x,2,3}$ and $k_{x,3,3}$. The response reduction is evaluated in terms of area underneath the PSD curve.

ViBa with a mass of 5.75E4 kg achieves an area reduction of 8.9% and 24% for buildings 1 and 2 respectively while it causes detrimental effects in building 3, increasing the power by 8.3%. On the other hand, the 5.75E5 kg ViBa yields more satisfactory results, causing power reductions in all buildings, with a remarkable value of 64% in building two. Nevertheless, the ViBa is not significantly effective in the reductions of the response of
building one and three. This has to do with the major difference in power that exists between building two and the rest.

Figure 6.33 SDOF ViBa optimum design for two different masses of 5.75E4 and 5.75E5 kg.
6.8 MDOF Vibrating Barrier for cluster of buildings

Despite the beneficial effect of the SDOF ViBa in the response of the cluster of buildings studied herein, consistent response reduction in all the buildings was not achieved. This section focuses on the stochastic design of a MDOF ViBa device aimed to improve the level of protection provided by the SDOF ViBa. The selected group of buildings corresponds to the system that has been identified through time-domain identification techniques in Section 6.5. The design procedure is that proposed in Section 6.1. This section studies the performance of two configurations of the MDOF ViBa device, namely, TDOF and two SDOF in parallel ViBa devices.

6.8.1 TDOF ViBa

Consider the cluster of buildings identified from the HODM of the Nice model coupled with a TDOF ViBa device, Figure 6.34 displays a sketch of the LODM of the group of buildings coupled with the ViBa.

![Discrete model of a cluster of buildings coupled with a TDOF ViBa device.](image)

Figure 6.34 Discrete model of a cluster of buildings coupled with a TDOF ViBa device.
The mass $\mathbf{M}$ and the stiffness $\mathbf{K}$ matrixes assume the following form:

$$
\mathbf{M} = \begin{bmatrix}
m_{1} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & m_{f,1} & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & m_{2} & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & m_{f,2} & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & m_{3} & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & m_{f,3} & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & m_{f,v} & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & m_{v,1} \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & m_{v,2}
\end{bmatrix}
$$

with

$$
\mathbf{K} = \begin{bmatrix}
k_{1} & -k & 0 & 0 & 0 & 0 & 0 & 0 \\
-k & k_{v,1} & -k_{suv,1,2} & 0 & 0 & 0 & -k_{suv,v,1} \\
0 & 0 & k_{2} & -k_{2} & 0 & 0 & 0 & 0 \\
0 & -k_{suv,1,2} & -k_{2} & k_{v,2} & 0 & -k_{suv,2,3} & 0 & -k_{suv,v,2} \\
0 & 0 & 0 & k_{3} & -k_{3} & 0 & 0 & 0 \\
0 & 0 & 0 & -k_{suv,2,3} & -k_{3} & k_{v,3} & 0 & -k_{suv,v,3} \\
0 & 0 & 0 & 0 & 0 & k_{v,1} & -k_{v,1} & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & -k_{v,1} & k_{v,1} + k_{v,2} \quad -k_{v,2} \\
0 & -k_{suv,1,2} & 0 & -k_{suv,2,3} & 0 & -k_{suv,v,1} & -k_{v,2} & k_{v,4}
\end{bmatrix}
$$

and

$$
k_{v,1} = k_{suv,1} + k_{suv,1,2} + k_{suv,v,1} + k_{1}
k_{v,2} = k_{suv,2} + k_{suv,1,2} + k_{suv,2,3} + k_{suv,v,2} + k_{2}
k_{v,3} = k_{suv,3} + k_{suv,2,3} + k_{suv,v,3} + k_{3}
k_{v,4} = k_{suv,v} + k_{suv,1} + k_{suv,2,3} + k_{suv,v,3} + k_{v,2}
$$
The procedure described in the previous Section 6.6 is employed for the design of the TDOF ViBa. Table 6.9 shows the parameters used for the vibrating mass system, while the parameters pertinent to the cluster of buildings are reported in Table 6.3 to Table 6.5.

<table>
<thead>
<tr>
<th>Units/Parameter</th>
<th>( m_{v1} )</th>
<th>( m_{v2} )</th>
<th>( m_{fr} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>kg</td>
<td>2.875E5</td>
<td>2.875E5</td>
<td>1.00E3</td>
</tr>
</tbody>
</table>

Due to the configuration of the ViBa designed as TDOF system, the vector parameter \( \alpha \) reduces to 2x1 vector listing the unknown stiffnesses of the vibrating structure \( k_{v,1} \) and \( k_{v,2} \), i.e. \( \alpha = \begin{bmatrix} k_{v,1} & k_{v,2} \end{bmatrix}^T \).

The introduction of an additional parameter has an important impact on the penalty function and in the procedure to find its minimum. To illustrate this, Figure 6.35 displays the variation of the norm of the vector containing the collection of second order statistical moments of the relative displacement of each structure with both \( k_{v,1} \) and \( k_{v,2} \) parameters. The presence of two indentations in the surface corresponding to the lower values of the function is observed. These channels correspond to areas of the penalty function that are mainly dependant on a single variable, while the other remains virtually constant. This causes the sensitivity matrix to be badly scaled leading to an ill-conditioned problem. To circumvent this problem, a two-stage approach is adopted, in line with the procedure presented in Section 4.2. is proposed, in which the variables are found individually while the remaining variables are kept constant. The results of the minimisation of the penalty function for the ViBa’ design following both a single and two stage approach are presented in Figure 6.36. It is evident that the results obtained following the two-stage approach yield a greater improvement overall.
Figure 6.35 Penalty function minimisation surface dependant on $k_v$ and $k_{v,2}$ for the TDOF ViBa device.
Similarly, to the case of the SDOF ViBa device the disproportionate weight of the central structure’s response poses the same problem, namely from a minimisation point of view, the optimum solution is shifted towards protecting the structure with the greater energy. To circumvent this and be able to direct the solution, modification factors of the response $\gamma_k$ are introduced. In order to assess the benefits or disadvantages of shitting the solution towards protecting a particular structure, four cases are analysed: an original scenario, in
which no modification factors are applied; two cases, in which the solution is shifted towards structures one and three; and a final case in, which both structures one and three should be protected simultaneously.

Figure 6.37 Optimum ViBa design case with no modification factors (left) and modification factor $\gamma_i$ to shift solution towards building 1 (right).
The results of the original case and that with the solution shifted towards building one are presented in Figure 6.37, while Figure 6.38 displays the results of the solution shifted towards building three and the case in which both building one and three are prioritised.

Figure 6.38 Optimum ViBa design case with modification factor $\gamma_1$ to shift solution towards building 3 (left) and modification factor $\gamma_1$ and $\gamma_3$ to shift solution towards building 1 and 3 (right).
The results of the original case without the use of modification factors are as expected the solution prioritises the protection of the central structure for being the one that carries greater weight in the solution. On the other hand, the results obtained when a modification factor is applied to the response of the first building show a significant reduction in the response of building one while the effect in the remaining buildings is very small. It is important to mention that this solution is characterised for a very small value of $k_{v,2}$, which translates into a system similar to a SDOF with half of the mass of the full system. This is an interesting feature of using a TDOF ViBa, since it provides a more comprehensive analysis compared to the SDOF alternative. The solutions of the two remaining cases are as expected. In the case of modification factors applied to building three, significant reductions are achieved in this structure, while a moderate reduction is observed in building one; furthermore, when both building one and three are prioritised for protection improvement in the response is achieved in both buildings. Nevertheless, both cases are characterised by an increase in the power of the relative displacement of the central structure. This behaviour is very similar to the one reported in the case of the SDOF ViBa. Finally, a case in which the modifications factors $\gamma_k$ have been chosen to generate a scenario in which all the second order statistical moments of the response have the same weight in the solution was analysed and the results are presented in Figure 6.39. This solution yields the overall most satisfactory results, causing simultaneous reductions in all the three structures. The values of the parameters $k_{v,3}$ and $k_{v,2}$ for all the cases considered are summarised in Table 6.10.

![Figure 6.39 Optimum TDOF ViBa design with all the buildings having the equal weight in the solution.](image)
Table 6.10 Optimal stiffness of the TDOF ViBa device for all the cases considered

<table>
<thead>
<tr>
<th>Case</th>
<th>Units/Parameter</th>
<th>$k_{v,1}$</th>
<th>$k_{v,2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original</td>
<td>N/m</td>
<td>4.77E7</td>
<td>4.09E7</td>
</tr>
<tr>
<td>Target 1</td>
<td>N/m</td>
<td>1E3</td>
<td>8.81E8</td>
</tr>
<tr>
<td>Target 3</td>
<td>N/m</td>
<td>5E8</td>
<td>1.41E8</td>
</tr>
<tr>
<td>Target 1-3</td>
<td>N/m</td>
<td>5E8</td>
<td>5E8</td>
</tr>
<tr>
<td>Equal influence</td>
<td>N/m</td>
<td>2.60E7</td>
<td>5.96E7</td>
</tr>
</tbody>
</table>

6.8.2 Two SDOF in parallel ViBa

This section is devoted to the study of a two SDOF in parallel ViBa device. The LODM of the group of buildings isolated from the urban environment coupled with a two SDOF in parallel ViBa formed of two individual vibrating elements is depicted in Figure 6.40.

Figure 6.40 Discrete model of a cluster of buildings coupled with a two SDOF in parallel ViBa device.
The mass \( M \) of the system is given in equation (6.35) while the stiffness \( K \) matrixes assume the following form:

\[
K = \begin{bmatrix}
  k_1 & -k & 0 & 0 & 0 & 0 & 0 & 0 \\
  -k & k_{c,1} & 0 & -k_{sv1,2} & 0 & 0 & 0 & -k_{svv,1} \\
  0 & 0 & k_2 & -k_2 & 0 & 0 & 0 & 0 \\
  0 & -k_{sv1,2} & -k_2 & k_{c,2} & 0 & -k_{sv2,3} & 0 & 0 & -k_{svv,2} \\
  0 & 0 & 0 & 0 & k_3 & -k_3 & 0 & 0 & 0 \\
  0 & 0 & 0 & -k_{sv2,3} & -k_3 & k_{c,3} & 0 & 0 & -k_{svv,3} \\
  0 & 0 & 0 & 0 & 0 & k_{v,1} & 0 & -k_{v,1} \\
  0 & 0 & 0 & 0 & 0 & 0 & k_{v,2} & -k_{v,2} \\
  0 & -k_{svv,1} & 0 & -k_{svv,2} & 0 & -k_{svv,3} & -k_{v,1} & -k_{v,2} & k_{c,4}
\end{bmatrix}
\]  

(6.38)

with

\[
\begin{align*}
  k_{c,1} &= k_{sv1,1} + k_{sv1,2} + k_{svv,1} + k_1 \\
  k_{c,2} &= k_{sv2,1} + k_{sv2,2} + k_{svv,2} + k_2 \\
  k_{c,3} &= k_{sv3,1} + k_{sv3,2} + k_{svv,3} + k_3 \\
  k_{c,4} &= k_{sv4,1} + k_{sv4,2} + k_{svv,4} + k_{v,1} + k_{v,2}
\end{align*}
\]  

(6.39)

The introduction of additional parameters into the problem entails an increment in the complexity of the problem, particularly in the case in which both vibrating masses have weak coupling between them. In the framework of least square strategies for the minimisation of functions badly scaled matrices are associated with problems of ill-conditioning and inaccurate solutions. This scenario is particularly prone to this type of problems given the little interdependency of both \( k_{c,1} \) and \( k_{c,2} \) variables. The norm of the vector containing the second order statistical moments of the structure relative displacements is plotted against both \( k_{c,1} \) and \( k_{c,2} \) in Figure 6.41.

From Figure 6.41 it can be seen the existence of two grooves in the surface that intersect at a common minimum and from them diverge to solutions that are independent from the variation of the second variable. These particularities make the minimisation of the penalty function non-trivial and both a two-stage approach as well as criteria in the selection of the initial conditions is required. The minimum of the function corresponds to the maximum energy reduction in the system, nevertheless, from an engineering point of view this might not be the optimum solution in a system, since the building with the greater maximum response might not be the most vulnerable structure within the cluster.
Figure 6.41 Penalty function minimisation surface dependant on $k_{v_1}$ and $k_{v_2}$ for the two SDOF in parallel ViBa device.
The same four scenarios with modification factors presented in section 6.8.1 are considered in this section, results are depicted in Figure 6.42 and Figure 6.43.

Figure 6.42 Optimum ViBa design case with no modification factor (left) and modification factor $\gamma_i$ to shift solution towards building 1 (right).
Finally, a case in which the modifications factors $\gamma_k$ have been chosen to generate a scenario in which all the second order statistical moments of the response have the same weight in the solution was analysed and the results are presented in Figure 6.44. The values of the parameters $k_{r,1}$ and $k_{r,2}$ for all the cases considered are summarised in Table 6.11.
After the analyses presented in this section it can be concluded that the overall reduction capacity of both the TDOF and the two SDOF in parallel ViBa devices is greater than the SDOF ViBa device. However, it is noted that depending on the particular scenario and problem constraints one design could prove to be more advantageous than another.

### 6.9 FEM design validation

This section is devoted to the implementation of the ViBa design back into the urban environment model with numerical evaluation of the ViBa effectiveness using HOFM. To this aim, the first milestone is to generate a suitable location of the ViBa that somehow correlates to the interaction parameters found in the stochastic design of the system for an optimum performance. A static approach similar to the one used in Alexander et al. (2013) is suitably employed to determine a relationship between distance and the SSSI stiffness of the ViBa with the neighbouring structures. The analysis was carried out for the two type of structures namely B1S and B2S buildings. Figure 6.45 displays a sample of the cases considered to the ViBa location study using HODM implemented using the FE software ADINA. The ViBa foundation is modelled using a buried cylindrical
structure with a massless rigid foundation. The vertical location of the ViBa was kept constant and is equal to 5 m below the ground surface measured to the centre of the ViBa casing structure. Interaction stiffness were estimated by imposing a unitary displacement of the building’s foundation while fixing the displacement of the ViBa’s foundation equal to zero and recording the horizontal reaction.

![Figure 6.45 ViBa location study using HODM, sample cases.](image)

![Figure 6.46 Coupling stiffness between the ViBa an adjacent structures for the two types of buildings.](image)

The variation of the coupling stiffness against the separation horizontal distance is presented in Figure 6.46 for both types of buildings, namely B1S and B2S. A distance equal to zero represents that the ViBa has been located directly underneath the building. The horizontal distance is measured between the centre of the building and the centre of the ViBa. Further, in order to determine if the type of structure has a significant importance, the coupling stiffness is plotted in Figure 6.47 against the normalised distance.
“zB” with respect to the foundation dimension “B” (see Table 6.2 for buildings characteristics). It is observed that the two curves are fairly similar, hence the influence of the type of structure is negligible.

![Coupling stiffness](image)

**Figure 6.47** Coupling stiffness between the ViBa an adjacent structures for the two types of buildings.

Considering the parameters obtained after the stochastic minimisation procedure presented in the previous section, and making use of the coupling stiffness-horizontal distance relationship displayed in Figure 6.46 and Figure 6.47, the ViBa is suitably located in the HODM created in ADINA and displayed in Figure 6.48. The three interaction stiffness obtained for the optimum design of both the SDOF and two SDOF in parallel ViBa devices were $k_{int,1} = 1E6$, $k_{int,2} = 1E6$ and $k_{int,3} = 5E7$. Locating the ViBa underneath the central structure of the structure gives the minimum value of the interaction of the ViBa for both building 1 and 2.

![Interaction stiffness](image)

**Figure 6.48** Coupling stiffness between the ViBa an adjacent structures for the two types of buildings.
The models with the single and the two SDOF in parallel ViBa devices were both subject to the 2001 Nice earthquake ground motion and compared to the model without any ViBa is present. The relative displacement time histories of each building are displayed in Figure 6.49 for the SDOF ViBa, and the results for the two SDOF in parallel ViBa are presented in Figure 6.50. The percentage of reduction is expressed in terms of reduction of the peak displacement with respect to the case in which the ViBa is not present. Results of the SDOF ViBa report a peak reduction of 3.9, 31 and 1.3 % for buildings one to three respectively, while the two SDOF in parallel ViBa yields reductions of 4.8, 26 and 1.7 %. It seems that the main difference between the SDOF and two SDOF in parallel ViBa that the latter yields a slightly higher reduction in buildings 1 and 3, but the reduction in the central building is smaller by almost 5% with respect to the SDOF ViBa.

![Figure 6.49 Coupling stiffness between the ViBa and adjacent structures for the two types of buildings.](image)

![Figure 6.50 Coupling stiffness between the ViBa and adjacent structures for the two types of buildings.](image)
6.10 Case study Vibrating Barrier in the City of Vathia: a simplified approach

This section focuses on the improvement of the seismic resilience of the historical village of Vathia, in Greece, using the ViBa device. In particular, a simplified design approach of the ViBa device is considered. To this aim, a simplified Finite Element (FE) model was created from the 2D-section of the village taken in accordance with the plan representation presented in Figure 6.51. All the analyses were performed using the FE software ADINA 9.2.1 (Bathe 2016). Figure 6.52 displays the FE model mesh with indication of building labels considered in the analyses presented herein. It is important to highlight that all the materials considered in the FE model (stratum, buildings, etc.) have isotropic linear elastic behaviour and the only global degree of freedom considered was horizontal translation (X-Direction).

Figure 6.51 Plan representation of Vathia village with indication of the studied section. (Sario et al. 2015).

Figure 6.52 FE mesh of the Vathia village section.
According to Higgins and Higgins (1996), the predominant rock type in the vicinities of the Vathia village is shale. A shear wave velocity \((V_s)\) of 1100 m/s and mass density of 2200 kg/m\(^3\) were considered to be a realistic assumption for the rock stratum. To determine the pertinent 2D parameters of the model, a detailed model of building seven (B7), depicted in Figure 6.53, was used. Assuming a density of 2300 kg/m\(^3\) for the structural elements (shear walls and arches) and 1000 kg/m\(^3\) for the floors, the mass of the building was determined and a constant mass per unit area (2D-model) of 4896.03 kg/m\(^2\) was calculated. Furthermore, the determination of the modulus of elasticity was performed by an identification process to match a target value of the fundamental frequency of B7 computed according to EUROCODE 8, that is:

\[
T_i = C_i \frac{H^{3/4}}{t_C}
\]

(6.40)

where \(T_i\) is the fundamental period of the structure; \(H\) is the height of the building, in m, from the foundation; \(C_i\) is a factor depending on the type of structure. For structures with masonry shear walls the latter value can be estimated as follows:

\[
C_i = 0.075 / \sqrt{A_c}
\]

(6.41)

where \(A_c\) is the total effective area of the shear walls in the first storey of the building, in m\(^2\), calculated as follows:

\[
A_c = \sum A_i \left(0.2 + \left(\frac{l_{wi}}{H} \right)^2\right)
\]

(6.42)

In equation (6.42), \(A_i\) is the effective cross-sectional area of the shear wall \(i\) in the direction considered in the first storey of the building, in m\(^2\); \(l_{wi}\) is the length of the shear wall \(i\) in the first storey in the direction parallel to the applied forces, in m.
Given the fact that Vathia is an ancient village, it was assumed that all the buildings will be made of similar materials. Therefore, all the parameters corresponding to B7 can be applied to the rest of the structures in the city.

Both harmonic analyses and ground excitation through white noise process were undertaken, taking B7 as the control building (see Figure 6.52). To determine the amplification characteristics (FRF) of the control building in isolation, both under fixed base and coupled SSI conditions, as well as surrounded by the building environment, steady state analyses using harmonic inputs with a Δf equal to 0.02 Hz in the 0–10 Hz range was used. These results are expressed in terms of relative acceleration and are depicted in Figure 6.54, the FRF of the stratum is also presented to indicate the origin of the first resonant peak of the building. From Figure 6.54, the following observations can be made: the resonant frequency of the soil-structure system (both the isolated structure and that in the full city) is smaller than the fixed-base frequency, as stated by Kramer (1996); the existence of a resonant peak that coincides with the resonant peak of the soil; and, finally, the amplitude of the full site-city model seems to be smaller than that of the isolated building. In other words, the consideration of the building environment has a beneficial effect for this particular case. Moreover, these findings are in agreement with Bard et al. (2006), Isbiliroglu et al. (2015) and Semblat et al. (2008).

The system was then forced by ground motion white noise process to identify the relevant peak responses. The response PSD functions of the absolute top displacement of each building are presented in Figure 6.55. As can be seen from Figure 6.55, all the buildings...
have a predominant peak at 2.50 Hz, which corresponds to the fundamental frequency of the stratum. This behaviour is a direct consequence of the soil-structure cross interaction phenomena, in which if the soil-structure system natural frequency is in the higher range with respect to the stratum fundamental frequency, the fundamental frequency of the soil-structure system will be in the vicinities of the fundamental frequency of the stratum (Kobori et al. 1973).

Figure 6.54 FRF of the control building in isolation under both fixed base and coupled SSI condition as well as surrounded by the building environment.

Figure 6.55 Power spectral density functions of the buildings top absolute displacements.
Given the results presented in Figure 6.55, a first design attempt of an array of ViBa devices can be made to minimise the city’s response. To protect all the buildings in the model, the ViBa devices were tuned to match the predominant frequency, namely 2.50 Hz. Figure 6.56 depicts the proposed arrangement of three ViBa devices with an internal mass of 600 Tonnes, each calibrated at the system’s fundamental frequency.

The PSD of the city protected with the array of ViBa devices was calculated in the same manner as the city’s response without the ViBa devices and results are depicted in Figure 6.57. As can be seen from Figure 6.57, there is an obvious and significant effect of the array of ViBa devices in the city, as they seem to reduce the amplitude of the peak at the
frequency that the devices were tuned to mitigate (the fundamental frequency of the system). Figure 6.57 also indicates a redistribution of the ground motion energy to other frequencies away from the site-city system.

Finally, a pertinent Monte Carlo study has been performed generating 50 quasi-stationary ground motion time histories with power spectral density

\[
G_{ug}(\omega) = \frac{\left(\frac{\omega_k^4 + 4\zeta_k^2\omega_k^2\omega^2}{\omega_k^2 - \omega^2}\right)^2 + 4\zeta_k^2\omega_k^2\omega^2\left(\frac{\omega_p^2 - \omega^2}{\omega_k^2 - \omega^2}\right)^2 + 4\zeta_p^2\omega_p^2\omega^2}{\omega_k^2 - \omega^2}
\]

(6.43)

with

\[
G_w = \frac{0.141\zeta_k\ddot{u}_g^2}{\omega_k\sqrt{1 + 4\zeta_k^2}}
\]

(6.44)

where \(\ddot{u}_g\) is the peak ground acceleration, taken as 0.3g and \(\omega_k = 15.0\), \(\zeta_k = 0.6\), \(\omega_p = 1.5\), \(\zeta_p = 0.6\).

Figure 6.58 Band plot of maximum absolute displacements (m) of the city without the ViBa.
Figure 6.59 Band plot of maximum absolute displacements (m) of the city with the array of ViBa devices.

Figure 6.58 and Figure 6.59 show the band plots of the city model for a selected sample of simulated ground motion time history with and without the ViBa devices. It can be observed the beneficial effects of the ViBa devices in the reduction of the maximum displacements. Also Figure 6.59 shows that the ViBa is experiencing, as expected, the maximum displacements, undergoing “resonance” to absorb part of the seismic energy. It was evidenced form the Monte Carlo study that each building had an average beneficial effect by the inclusion of the ViBa devices in the soil, yielding a minimum average reduction of 12.22% and maximum reduction 26.15% of the maximum relative displacement. It is noted that additional masses/ViBa devices will further improve the beneficial effects.

6.11 Summary

In the endeavour to improve the seismic resilience of urban environments and given the heterogeneity of cities nowadays, it is of paramount importance to identify the boundaries of dynamic cross interaction between structures. This allows for the design of vibration control solutions that are particularly tailored to enhance seismic resilience for this so-called dynamically independent units.

In this Chapter a general framework for the identification of the “dynamically independent” units was proposed. In the case of the urban environment model studied herein, it was concluded that for distances greater than 30 m the dynamic cross interaction coupling between structures is weak. Moreover, it was noticed that the type of structure
participating in the dynamic coupling is relevant. If the neighbouring structures of a building have higher natural frequency they act as a shield limiting the dynamic coupling with structures beyond. While the cluster identification criteria deduced in section 6.4 is tailored to the cases analysed in this study, the strategy can be further used to develop more general outlines.

The design process of a SDOF ViBa coupled with a cluster of buildings in the urban environment involves the consideration of the possible static contributions caused by the addition of the ViBa to the system, the appropriate selection of the SSI parameter of the ViBa and the magnitude of the ViBa’s mass. Parametric analyses to identify the influence of each aspect were conducted. Furthermore, provided the difference in influence of the different structures in the cluster, modifications factors able to prioritise a given structure over the rest were successfully introduced. These factors enabled to shift the optimum design towards protecting a particular structure within the cluster. Finally, a case in which all the structures were assigned the same weight in the solution was carried out and energy reductions ranging from 10 to 51% percent were achieved simultaneously in the cluster.

In order to improve the overall performance of the ViBa device in the urban environment two different internal configuration of the ViBa were studied, namely, TDOF and two SDOF in parallel. It was observed that, in addition to all the design implications involved in the case of the SDOF ViBa using a MDOF ViBa device introduces other complexities. In particular, those involved in the minimisation of the penalty function and the combined effect of disproportionate sensitivities regarding the parameters of the ViBa. A two-stage minimisation approach was applied to overcome this difficulty. Optimum designs were proposed for both two MDOF arrangements, and it can be concluded that the overall reduction capacity of both MDOF ViBa devices is greater than the SDOF ViBa device. Starting from the optimum cross interaction parameters between the ViBa and the different structures in the cluster, identified from the minimisation of the penalty function, a parametric study was conducted to identify the optimum location of the ViBa in the full-scale model of the urban environment. Relationships between the coupling stiffness and the distance between the ViBa and the structures were established. Making use of these relationships enabled the implementation of the optimum design of both the SDOF, and the two SDOF in parallel ViBa devices. These were implemented into the full-scale model of a relevant case-study urban environment, and their performance was assessed under earthquake ground motion. To this aim, the 2001 Nice earthquake time history was used.
The SDOF ViBa device yielded simultaneous reductions in the maximum relative
displacement ranging between 1.3% and 31%, while the two SDOF in parallel ViBa under
the same excitation caused simultaneous reductions between 1.7% and 26%.
7 Physical models on SCI

Physical models play a pivotal role in the understanding of engineering phenomena. Given their relative high cost compared with the development of analytical or numerical tools, physical models are often undertaken to validate both analytical formulations and numerical models, where greater insight can be gained by the analysis of a wider range of scenarios without incurring in significant expenses. In the subject of dynamic cross interaction among several structures, various physical models have been studied (Niwa et al. 1988; Kitada et al. 1999; Yano et al. 2000; Kitada et al. 2001; Kitada and Iguchi 2004; Trombetta et al. 2012; Aldaikh et al. 2016). Nevertheless, experimental studies that address SCI are not abundant. To the author’s knowledge, the only experimental setup designed exclusively to assess the SCI phenomenon was the one carried out by Schwan et al. (2016), their model was formed of a uniform soil stratum and different configurations of identical structures. This is certainly a simple configuration; however, it provided one of the first experimental evidences of the importance of SCI phenomenon.

One of the main difficulties of addressing SCI experimentally arises from the extent at which this phenomenon takes place. Unlike SSI and SSSI, where a single or two buildings need to be considered, the analysis of SCI requires a significant number of structures interacting with the stratum on which they are founded. There are two main alternatives to obtain further experimental evidence of the SCI phenomenon, namely: heavily monitored cities, and scaled models of prototypes cities. The former has the main disadvantage of depending on earthquakes of moderate to high magnitudes to strike the monitored site in order to generate usable data; moreover, the cost of the city characterisation in terms of both soil stratum and built environment as well as the deployment of a dense array of sensors can be important.

In addition to the already limited amount of experimental data on the subject of SCI, experiments that address the non-localised reduction of vibrations in urban environments in a holistic fashion are even scarcer. In the effort to mitigate the effects of incoming seismic waves on structures, Woods (1968) conducted field tests to evaluate the effectiveness of open trenches in reducing the amplitude or vertical ground motion. Çelebi et al. (2009) performed a full scale study on the effectiveness of two type of trench barriers, namely, open and infilled trenches, as a solution to mitigate the effects of ground
motion. Brule et al. (2014) investigated the so-called seismic metamaterials in a full-scale arrangement of periodic boreholes aimed to act as a seismic shield against surface waves generated by monochromatic vibrocompaction probe. Nonetheless, to the author’s knowledge, experimental data on the non-localised vibration control of structures in an urban environment has not yet been generated. This Chapter aims, for the first time, to suitably employ the ViBa device as vibration control strategy in an urban environment through a scaled physical model of a realistic city. To this aim, a scaled physical model of a city environment after the model proposed by Semblat et al. (2008) was used. Both free-field and city-like environment were considered. Furthermore, the ViBa device was successfully implemented in the physical model as a non-localised vibration control strategy, yielding response reductions in a group of buildings within the city.

7.1 Scaled physical model

The use of scaled models from larger prototypes has been proven to be an advantageous approach to both increase the understanding of a given phenomenon and add robustness to numerical models addressing the same problem. Nevertheless, besides all the benefits in scale modelling, there are drawbacks that need to be taken into account when experiments are being designed and analysed. Issues like the inability to reproduce stress states in the model’s soil, the necessity of weaker simulant materials and the difficulty to satisfy simultaneously all the scaling laws are some of the main challenges that are faced when scale models are being implemented. Given the large dimensions involved in site-city problems, scaled models often have to compromise on several scaling laws. For instance, consider a Froude scaling laws, in which the frequency scaling factor is equal to \( \lambda_f = 1/\sqrt{\lambda_l} \) where \( \lambda_l \) is the length scaling factor. If a city extension of 5000 m is being considered and the maximum size of the model is 2 m, \( \lambda_f \) will be equal to 0.02, which means that 1 Hz is prototype units is equivalent to 50 Hz in model units. Provided that excitations with frequency bandwidth of 0–10 Hz wish to be explored will require a shaking table able to drive a payload of several hundred kilos (depending on material) up to a frequency of 500 Hz. From a technical point of view, this is up to date not achievable.

It is customary in the context of scale modelling to designate the full-scale model scenario as the ‘prototype’ while its scaled version is referred as the ‘model’. However, for simplicity, full scale and scaled model designations are used accordingly herein. The full scale model of the urban environment used in this study is that proposed by Semblat et
al. (2008). The model represents a 2D section of the city of Nice, France. The full-scale model has been described and validated in Chapter 6 following a numerical approach. Detailed information about the model can be found in Section 6.3. The greatest dimension of the full-scale model is 2500 m which represents the horizontal extension of the bedrock-alluvial basin section. The available testing equipment is limited to a dimension of 1 m, hence a scaling factor for length $\lambda_l$ equal to 2500 is defined. Given the fact that the model is being tested under the same gravitational field as the prototype (1g model), Froude scaling laws can be used. The velocity scaling factor $\lambda_v$ is equal to $\sqrt{\lambda_l}$. Given the shear wave velocity of the full-scale basin material of 200 m/s a material with a shear wave velocity of 4 m/s would be required to represent the basin in the scaled model. Furthermore, the mass density scaling factor $\lambda_\rho$ is equal to $\lambda_l^3$. Considering the mass density of the alluvial basin of the full-scale model to be equal to 2000 kg/m$^3$ the scaled material should have a mass density of 1.28E-7 kg/m$^3$. A material with this combination of properties is virtually non-existing. Based on previous models proposed by Niwa et al. (1988); Yano et al. (2000); Aldaikh et al. (2016) and Schwan et al. (2016), silicon rubber and polyurethane foam were chosen as suitable materials to model the basin and the bedrock respectively. Moreover, a combination of polyurethane foam and acrylic was used to represent the structures in the city arrangement. The development of the scaled model was initiated by Bedran (2018) under the undergraduate research grant of the IStructE (Institution of Structural Engineers), and further developed. Detailed information on the model materials is given in the subsequent sections.

7.1.1 Materials and construction

The model was formed by two main materials, i.e., silicon rubber and polyurethane foam. The former material was used to model the alluvial basin, while the latter one to represent the bedrock. The polyurethane foam was tested following the same methodology presented in Section 5.1.3 using modal analysis through impact hammer testing. The testing setup is presented in Figure 7.1, while the test results, expressed in terms of the frequency response function (FRF), are presented in Figure 7.2. The elastic properties of the material were determined creating an equivalent FE model in ADINA and varying the elastic properties until the modal response measured in the experimental setup was achieved. Figure 7.3 displays the FE model. The mass density of the polyurethane foam is equal to 45.7 kg/m$^3$ determined in a trivial manner by dividing the mass of the sample by its volume.
The properties of the silicon rubber were determined following the same methodology. Table 7.1 summarises the properties of the model’s materials.
Table 7.1 Scale model material’s properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Mass density [kg/m³]</th>
<th>Poisson ratio</th>
<th>Shear modulus [MN/m²]</th>
<th>Shear wave velocity [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyurethane foam</td>
<td>45.7</td>
<td>0.40</td>
<td>15.97</td>
<td>590</td>
</tr>
<tr>
<td>Silicon rubber</td>
<td>1330</td>
<td>0.47</td>
<td>0.16</td>
<td>11</td>
</tr>
</tbody>
</table>

Figure 7.3 FE model of the polyurethane foam under testing conditions.

Figure 7.4 Soil-bedrock model.

The soil stratum was created by carving the basing shape into a block of polyurethane foam with the dimensions of the scaled model and a thickness of 100 mm. The model is intended to reproduce plain strain conditions. The basin material, represented by the silicon rubber was poured in the cavity created between the polyurethane foam contour and the sides of an acrylic box, where the model was placed to serve as lateral boundaries.
during the setting of the silicon rubber. The soil-bedrock model is displayed in Figure 7.4. A second model consisting of the soil stratum and the urban environment was also made following the same procedure with the particularity that two 6 mm diameter steel pipes were included in the soil (see Figure 7.5) to act as and the housing of the ViBa. The buildings of the city were made of polyurethane foam, following the same arrangement of the realistic city proposed by Semblat et al. (2008). Given the high shear modulus of the foam, a strip of acrylic was glued to the top of each structure to reduce their fundamental frequency to the range of the basin’s fundamental frequency, to favour the SCI (Semblat et al. 2008). The city model is depicted in Figure 7.6.

Figure 7.5 Steel tubes embedded in the basin to serve as housing for the ViBa (Bedran 2018).

Figure 7.6 Site-city model with embedded ViBa’s housing.
7.1.2 ViBa set-up

Starting from the definition of the ViBa, considered as a vibrating element encased in its foundation, buried in the soil and completely detached from the structures that is intended to protect, a scaled device was developed. The scaled device was formed of a 6 mm aluminium pipe, that represents the foundation of the ViBa. The vibrating element consisted of a mass attached to a steel cable, and the source of stiffness is provided by tension in it. A fixed attachment mechanism was placed at one extreme of the wire, while the other extreme was secured by a removable wire clamp. The tension in the wire was provided by the rotation of an adjustment mechanism that, in combination with the wire clamp, pulled the cable and supplied tension to the wire proportional to the rotation of the adjuster. A sketch of the parts and the assembled device is presented in Figure 7.7, while the assembled device and its separate parts are displayed in Figure 7.8. The tuning of the device was performed experimentally by rotating the adjuster and measuring the instantaneous transfer function at the target structure when the system is subject to an input motion at the base of the model.

Figure 7.7 Scaled ViBa device sketch.
Figure 7.8 Assembled scaled ViBa device and its separated parts.

7.1.3 Model Testing setup

All the configurations were tested using a tailored made accessory to a shaking table able to hold the model along all its extension. The shaking table was driven by a shaker to deliver a force excitation time history with a frequency content up to 10 kHz. The testing setup is depicted in Figure 7.9. The input force into the system was measured using a force transducer model 208C01, manufactured by PCB. The motion at the base of the model was measured using a uniaxial accelerometer model 333B30, manufactured by PCB. The response at the top of the structures was measured using a miniature single axis accelerometer, model 352C23, manufactured by PCB. The total mass of the sensor is 0.2 gm; so, by using this type of sensors, the impact that additional mass has on the response of the structures was minimal. The frequency range of the sensors was 0–3000 Hz, providing very good resolution results in the range of interest of 0–300 Hz.
7.2 Model results

This section is devoted to present and discuss the experimental results of the scaled site-city models with the inclusion of the experimental design of the ViBa. The results presented herein constitute the first attempt to design the ViBa in an urban environment following an experimental approach, as well as the first experimental evidence on the efficiency of the ViBa in protecting a cluster of buildings in an urban environment, taking into account SCI effects. Experimental results of the ViBa aimed to protect one and two structures can be found in (Cacciola et al. 2015; Cacciola and Tombari 2015; Tombari et al. 2018). Initially, the bedrock-soil model without the built environment was tested. Results are compared with the scaled numerical model and by comparing both the full scale and the scaled numerical models similitude relationships between the prototype and model were established. Moreover, the physical model of the free-field is validated by comparing the response in full scale units with the results presented by Semblat et al. (2008). Subsequently, the model that includes the realistic city arrangement was tested and results are compared with its numerical counterpart. It is important to mention here that no vibration control device was included during this set of tests. Finally, the ViBa was incorporated into the model and designed following an experimental approach. The experimental design of the ViBa was implemented in the full-scale model of the city of
Nice according to the similitude relationships developed between the numerical prototype and scaled models. The full-scale model is subject to an input time history, and results are reported in the frequency domain as the Fourier transform of the measured acceleration at the top of several buildings. Significant reductions are achieved.

7.2.1 Free-field validation

Initially, tests were conducted on the model made of both the bedrock and the basin, excluding the built environment (see Figure 7.10). Measurements were taken at the same six points along the surface of the alluvial basin as reported by Semblat et al. (2008). A numerical model of the scale model was also developed for the sake of creating a link between the full scale and the scaled model, this model and the measuring points are displayed in Figure 7.11.

Figure 7.10 Free-field numerical model with measuring points on the basin’s surface.

Figure 7.11 FEM mesh of the scaled physical model of the 2D basin located in the centre of Nice, France.

Both the physical and numerical models were subject to the same input signal, while the position of the accelerometers was changed. The input signal comprised a periodic sinusoidal chirp with a frequency bandwidth of 0–300 Hz. The data is compared in the frequency domain by applying the Fast Fourier Transform (FFT) to the acceleration time histories measured in both the physical and numerical models. Figure 7.12 displays the
FFT amplitudes for the six points defined in Figure 7.11. With the exception of the measuring point P5, it can be observed that a fair agreement is achieved between both models in terms of amplitude and frequency.

The comparison between the numerical results of both the full scale and scaled model yielded a frequency similitude relationship $\lambda_f$ of 116. That is to say that 1 Hz in the full-scale model is equivalent to 116 Hz in the scaled model. In order to validate the response
of the physical model and making use of the frequency similitude relationship, the transfer functions of the response at the six measured points are compared in Figure 7.13 and Figure 7.14 with the results reported by Semblat et al. (2008).

![Figure 7.13 Comparison of the transfer functions measured on the physical model with the results reported by Semblat et al. (2008) at points 1 to 3.](image1)

It can be said from the results depicted in Figure 7.13 and Figure 7.14 that, despite the out of proportion peaks observed in the response of the physical model for point P1 and P6, that the general trend of the results is satisfactory, with a good degree of agreement...
found for point P2, P3 and P4. Regardless of the limitations of the physical model, it has been evidenced that the scaled model captures the behaviour of the full-scale model of the Nice basin, allowing the solutions developed for the scaled model to be extrapolated into the full-scale scenario.

Figure 7.14 Comparison of the transfer functions measured on the physical model with the results reported by Semblat et al. (2008) at points 4 to 6.
7.2.2 Urban environment model

The physical model that includes the realistic city arrangement was subject to an input motion at the base of the model, the input signal was a periodic sinusoidal chirp, with a frequency bandwidth of 0–300 Hz. It is worth mentioning that the arrangement of the structures within the city corresponds to a realistic case scenario proposed by Semblat et al. (2008). Acceleration time histories were recorded at the top of B1, B2, B4 and B6 buildings indicated in Figure 7.15, given the small size of the model it was not possible to attach the miniature accelerometer to B5 structure. Figure 7.16 display the detail of the studied cluster in the physical model. Furthermore, the recorded input acceleration was used as excitation in the scaled numerical model in order to compare the degree of agreement between the solutions. The acceleration amplitudes in the frequency domain are presented in Figure 7.17. A remarkably good agreement is found between the physical and the numerical model for structures B1 and B6. On the other hand, a fair agreement can be observed in the case of the structure B4 while the comparisons between both models for B2 show a moderate matching. It is worth mentioning that despite many efforts to reproduce the model as accurately as possible during the construction stage, imperfections in the physical model remain. These imperfections do not exist in the numerical model, trying to reproduce them can lead to further complications and discrepancies. Among the possible sources of differences between both models stands the issue of the connection between the buildings and the basin material, in the numerical model a perfect bond between both materials exist this is in contrast with the physical model where a perfect connection between both materials cannot be guaranteed.

Figure 7.15 Scaled city numerical model model with detail of the studied cluster.
Figure 7.16 Scaled city physical model model with detail of the studied cluster.

Figure 7.17 Amplitudes measured in both the physical and numerical models for different structures in the city.

An additional disadvantage of the scaled model arises when assessing the influence of the city environment in modifying the free-field ground motion. Since the prototype has such a large extension, and given the size limitations of the testing arrangement, the inter-building distances are but a few millimetres, limiting the installation of the measurement
sensors without compromising the integrity of the model in the installation process. Nevertheless, in order to provide measurements at all points, and provided the good agreement between the physical and numerical scaled models, the influence of the city in the scale model is also evaluated numerically. Figure 7.18 displays the acceleration amplitudes at six different points on the surface of the basin for the cases of free field motion and the motion in presence of the urban environment for the scaled model.

Figure 7.18 FFT amplitudes at the surface of the basin at points P1 to P6 for both free field and city case.
It is observed from Figure 7.18 the impact of the presence of the built environment in the modification of the free-field ground motion, as pointed out by Kham et al. (2006); Semblat et al. (2008) and Schwan et al. (2016). The most important effects are observed for points P1, P2, and P3, while lesser modifications are seen in point P4, P5 and P6. These results are in accordance with the findings presented by Semblat et al. (2008). Since the model response is dominated by the deepest part of the basin, greater interactions take place in the vicinities of P1, P2 and P3.

7.2.3 Seismic resilience enhancement in the urban environment

The focus of this section is twofold: to establish for the first time an experimental approach for the design of the ViBa in an urban environment, as well as to provide experimental evidence of the ViBa’s effectiveness as a non-localised vibration control solution for clusters of buildings in urban environments. The impact of the ViBa on buildings that extend beyond the target cluster is explored both experimentally and numerically. Furthermore, the experimentally designed ViBa is successfully implemented in the full-scale numerical model and reductions are achieved in all the buildings in the cluster.

The experimental setup is depicted in Figure 7.19, with indication of the structures in the cluster where measurements were taken. In comparison with the results presented in Figure 7.15, the response of structures B8 and B9 was monitored to analyse the area of influence of the ViBa device. The scaled ViBa device is assembled as in Figure 7.8 in one of the aluminium pipes placed in the model during casting. The design process of the ViBa device consisted in the gradual tuning of the vibrating element frequency by means of the adjustment mechanism. The tension in the wire holding the mass in the ViBa was increased by the rotation of the adjustment screw. The system was subject to a periodic sinusoidal chirp signal, while the response of the structure B1 was continuously monitored in terms of acceleration amplitude at the top of the structure. By gradually increasing the frequency of the ViBa its presence starts to have a noticeable effect in the response of the structure B1 creating a sharp decrease of the acceleration amplitude at the ViBa’s frequency. The ViBa’s frequency is increased until the peak response acceleration at the top of the B1 structure is significantly reduced. At this stage the locknut of the device is used to prevent the adjustment mechanism from moving and the response at the remaining buildings is measured. Results are reported in the frequency domain in Figure 7.20.
The response recorded in structure B2 seems anomalous (Figure 7.20), as it differs significantly from that of structures B1, B6 and B9, which in principle have the same characteristics. Nevertheless, the detrimental effects observed in structure B8 are below 5% and thus they are negligible. The frequency domain response of the structures presented in Figure 7.20 are evaluated in terms of the percentage change in the area under the frequency response curve. It can also be noticed from Figure 7.20 the significant amplitude reduction of the acceleration in structures B1 and B6 while moderate reductions in the vicinities of the ViBa’s frequency can be seen in B2 and B4. Less significant changes are observed in structures B8 and B9 located at the boundaries of the cluster. However, the overall reduction in terms of energy reduction are moderate. This effect has to do with the low damping that the scaled device possess, which limits the effectiveness of the ViBa to a very narrow frequency range. This is explored further in the implementation of the ViBa using the full-scale numerical model.

Following the similitude relationships established between the full scale and scaled numerical models, the ViBa design is transferred into full scale units in terms of mass and stiffness of the vibrating element. The mass of the ViBa in the physical model is approximately 2.5 times the mass of a building, this ratio is used to determine the total mass of the ViBa in full scale units. The stiffness of the resonant element is found combining the knowledge of the mass and frequency of the ViBa. The acceleration response in the time and frequency domain measured in the full scale numerical model at
the top of the monitored buildings are presented in Figure 7.21 and Figure 7.22 respectively.

Figure 7.20 Acceleration amplitudes in the frequency domain at the top of the buildings in the protected cluster, experimental results.

A similitude in the trend between the experimental results and the full-scale numerical model can be evidenced in Figure 7.21. Relevant reductions are achieved in both structures B1 and B4, while small reductions characterise the response of structures B6 and B9, with a negligible effect in B8. The greatest difference exists for the measurements of structure B2 where a reduction in the same order of those obtained for B1. The
hypothesis that the response of structure B2 in the physical model is anomalous and that some imperfections must exist, either in the connection of the material of this particular building is further reinforced by the response of the cluster displayed in the frequency domain presented in Figure 7.22. It is observed that since B1 and B2 are identical structures their response is very similar, this is in contrast with the results reported for the physical model.

Figure 7.21 Acceleration time histories measured at the top of the buildings in the protected cluster, full scale numerical model results.
Figure 7.22 Acceleration amplitudes in the frequency domain at the top of the buildings in the protected cluster, full scale numerical model results.

In addition, Figure 7.22 indicates the effectiveness of the ViBa to reduce the amplitude of the acceleration at the top of the monitored structures. The modifications in the response of the structures caused by the presence of the ViBa are characterised by a sharp reduction of the acceleration amplitude at the frequency of the ViBa. Moreover, the observed gradual reduction of the ViBa’s influence in the response of the structures with distance is in accordance with the observations of Cacciola et al. (2015).
Finally, in order to illustrate the influence of the ViBa’s damping in its effectiveness and characteristic modifications of the structure’s response, an additional analysis with and increased damping (from 0.5% to 10%) in the ViBa device was undertaken. The results are depicted in Figure 7.23. It is observed that the sharp reduction at the ViBa’s resonant frequency is not noticeable, nonetheless, significant reductions are still achieved over a slightly wider range of frequencies. It can therefore be concluded that ViBa is able to
effectively reduce the response amplitude of groups of structures in urban environments improving the overall seismic resilience of the urban environment, particularly if the ViBa devices are equipped with relatively high damping.

Range of influence of the ViBa

As it has been observed, the level of reduction caused by the presence of the ViBa in a cluster of buildings decreases as the distance from the ViBa increases. At this stage one might wonder what are the limits of influence of the ViBa. In this endeavour, measurements at different positions along the basin’s surface were taken in both the scaled physical model and the full scale numerical model. Acceleration time histories at the top of the nearest structure to the six points on the basin’s surface designated by Semblat et al. (2008) were recorded, with and without the presence of the ViBa. The location of the points along the basin’s surface in full scale units were as follow: point P1 was 295 m from the left extreme of the model and the subsequent points (P2-P6) were distributed at uniform intervals of 250 m. Figure 7.24 displays an image of the physical model with indication of the six measuring locations.

![Figure 7.24 Scaled physical model with indication of the six measuring points on the basin’s surface.](image)

Initially, comparisons were made between the response of the structures B8, B1 and B9 (see Figure 7.19) in order to designate the range of action, in terms of distance, of the ViBa. These results are expressed in terms of the transfer functions between the motion at the base of the model and the top of the three structures and are presented in Figure 7.25. Structure B8 is located 90 m to the left of the ViBa while structure B9 is located 70 m to the right of the ViBa. According to Cacciola et al. (2015), some effects are expected within this range. Furthermore, besides the distance criteria there is an additional factor that plays a role in the extension of the ViBa’s influence, namely the dynamic interference created by different types of structures. Figure 7.22 and Figure 7.23 show the reductions
of structures B6, B8 and B9 to be very similar, which correlates with the fact that in the trajectory from the ViBa to those structures there is always a structure of the type B2S in between. These observations are in agreement with the results presented in section 6.4 regarding the identification of the so called dynamically independent units in an urban environment.

Figure 7.25 Acceleration amplitudes in the frequency domain at the top of the buildings B8, B1 and B9 located in the vicinities of point P3 with and without the ViBa.

The assessment of the effects of the ViBa throughout the city was made by measuring the acceleration time history at the top of the structures located in the vicinities of points P1 to P6, with and without the presence of the ViBa. Results are expressed in the time and frequency domain in Figure 7.26 and Figure 7.27 for the physical model, while those corresponding to the full scale numerical model are displayed in Figure 7.28 and Figure 7.29. Data collected from the physical model seems to indicate that no significant changes are introduced at points P1, P2 and P5 while noticeable modifications are evidenced at points P3, P4 and P6. While modification in P3 are expected, changes at P6 are not consistent with the fact that no modifications are observed at point P5. Due to the availability of only one miniature accelerometer, the sensor had to be moved from structure to structure for the case of tested with and without the ViBa. Although care has been taken during the testing stage to ensure that the measuring points are the same in both scenarios, it is likely that the position of the sensor was different by a few millimetres. These apparent small changes can have an important impact on the recorded response since 1 millimetre in the scale model units is equivalent to 2.5 meters in the full-scale model.
Figure 7.26 Acceleration time histories measured at the top of the nearest structure to points P1 to P6 with and without the ViBa in the physical model.
Figure 7.27 Acceleration amplitudes in the frequency domain at the top of the nearest structure to points P1 to P6 with and without the ViBa in the physical model.

With the aim to aid the interpretation of the results obtained during the physical model testing campaign, the influence of the ViBa in the urban environment was explored using the full-scale numerical model. Measurements were taken at the same locations, and acceleration time histories are displayed in Figure 7.28, while the frequency domain response is presented Figure 7.29.
Figure 7.28 Acceleration time histories measured at the top of the nearest structure to points P1 to P6 with and without the ViBa in the full scale numerical model.

The trend in the results presented in both Figure 7.28 and Figure 7.29 is as expected, with important modifications recorded in the vicinities of point P3, while a negligible influence of the ViBa is evidenced elsewhere in the urban environment. It can hence be concluded that, in accordance with the findings of Cacciola et al. (2015), experimental and numerical data seem to indicate that the range of influence of the ViBa is limited to a few hundreds
of meters. Beyond this area, the modifications in the ground motion induced by the ViBa can be considered negligible.

Figure 7.29 Acceleration amplitudes in the frequency domain at the top of the nearest structure to points P1 to P6 with and without the ViBa in the full scale numerical model.
7.3 Summary

In this Chapter, an experimental and numerical investigation on the performance of the ViBa as a non-localised holistic approach for the seismic protection of clusters of buildings in an urban environment has been conducted. A design framework of the ViBa in an urban environment following an experimental approach has been presented. Experimental evidence highlights the capabilities of the ViBa of protecting more than one structure in an urban environment yielding reductions of up to 21% in the maximum response acceleration. Furthermore, making use of similitude relationships between the scaled and full-scale models the ViBa is suitably implemented in the full-scale numerical model and response reductions are achieved for all the buildings in the cluster when subject to earthquake ground motion. Percentage acceleration reductions ranging from 24% to 1% are achieved in the group of six monitored buildings, with no detrimental effects. Moreover, analyses on the area of influence of the ViBa were also carried numerically and experimentally. Acceleration time histories at the top of the structures in six different points along the basin’s extension were recorded and analysed. Results indicate that the range of influence of the ViBa is limited to a few hundreds of meters. Beyond this area the modifications in the ground motion induced by the ViBa can be considered negligible.
8 Conclusions

In the case of heritage buildings, critical facilities and urban areas, especially in developing countries, traditional localized solutions of structural control might be impractical. Therefore, what we are witnessing nowadays is the lack of substantial actions to protect existing cities in seismic prone areas with fatalities and loss of historic and artistic heritage. In the present study, effort has been devoted to design and evaluate the recently developed ViBa device as a viable solution for the enhancement of seismic resilience in urban environments in a holistic fashion.

8.1 Concluding remarks

After the examination of the literature regarding the study of structure-soil-structure interaction (SSSI) and site-city interaction (SCI) an understanding of the main governing aspects of these phenomena was gained. The revision of the available non-localised vibration control techniques exposed the gap in knowledge concerning viable strategies for the vibration control in urban environments that address the problem in a global manner. Furthermore, the limited amount of experimental evidence on SCI was observed as an opportunity to enrich the experimental insight available in this regard.

In order to minimise the computational effort involved in the study of SSSI and SCI systems, following numerical approaches low order discrete models (LODM) able to capture with sufficient detail the dynamic behaviour of both SSI and SSSI systems, including the impact caused by site effects and wave propagation, were developed. These models were limited to horizontal translations only, and the scope of this simplification was assessed in terms of the aspect ratio of the structures being identified and soil stiffness, it was concluded that as the soil stiffness increases the influence of the rotations on the lateral displacements of the structure becomes less important for aspect ratios of up to 1. Furthermore, in the identification of SSSI systems under a single input multiple output (SIMO) framework, the issue of disproportionate sensitivities becomes important. In order to overcome this issue, an alternative approach was proposed. It consists of solving the bounded-variable time-domain least-square identification procedure in two stages. In the first stage, parameters that have greater impact and comparable sensitivities will be sought, while the remaining parameters are found in the second stage. In addition,
a model LODM able to account simultaneously for site and wave passage effects was developed, with the potential to further account for the influence of distance between structures.

The effectiveness of the LODM formulations was validated through the implementation of high order discrete models (HDOM) and scaled physical models. Among the three proposed LODM, it was concluded that the two degree of freedom (TDOF) model provided the better balance between simplicity and accuracy. The application of the time domain identification procedure to define a LODM of both an SSI and SSSI physical models indicated that the LODM is able to account fairly well for the response of the physical model in the analysed frequency range. Significant improvement is seen in the degree of matching of the final iteration with respect to the initial values.

In the endeavour to improve the seismic resilience of urban environments and given the heterogeneity of cities. it is of paramount importance to identify the boundaries of dynamic cross interaction between structures. This allows for the design of vibration control solutions that are particularly tailored to enhance seismic resilience for so-called “dynamically independent units”. A general framework for the identification of these units was proposed. In the case of the urban environment model studied herein, comprised of a medium soil characterised by a shear wave velocity Vs of 250 m/s, it was concluded that for distances greater than 30 m the dynamic cross interaction coupling between structures is weak. Moreover, it was noticed that the type of structure participating in the dynamic coupling is relevant. If the neighbouring structures of a buildings have higher natural frequency they act as a shield limiting the dynamic coupling with structures beyond. While the cluster identification criteria deducted in Section 6.4 is tailored to the cases analysed in this study, the strategy can be further used to develop more general outlines.

The design process of a SDOF vibrating barrier (ViBa) coupled with a cluster of buildings in the urban environment involves the consideration of the possible static contributions caused by the addition of the ViBa to the system, the appropriate selection of the SSI parameter of the ViBa and the magnitude of the ViBa’s mass. Parametric analyses to identify the influence of each aspect were conducted. Furthermore, provided the difference in influence of the different structures in the cluster, modifications factors able to prioritise a given structure over the rest were successfully introduced. These factors enabled to shift the optimum design towards protecting a particular structure within the
cluster. Finally, a case in which all the structures were assigned the same weight in the solution was carried out and energy reductions ranging from 10 to 51 percent were achieved simultaneously in the cluster.

In order to improve the overall performance of the ViBa device in the urban environment two different internal configuration of the ViBa were studied, namely, TDOF and two SDOF in parallel. It was observed that, in addition to all the design implications involved in the case of the SDOF ViBa, using a MDOF ViBa device introduces other complexities into the design procedure. In particular, those involved in the minimisation of the penalty function and the combined effect of disproportionate sensitivities regarding the parameters of the ViBa. A two-stage minimisation approach was applied to overcome this difficulty. Optimum designs were proposed for both MDOF arrangements, and it can be concluded that the overall reduction capacity of both MDOF ViBa devices is greater than the SDOF ViBa device.

Starting from the optimum cross interaction parameters between the ViBa and the different structures in the cluster, identified from the minimisation of the penalty function, a parametric study was conducted to identify the optimum location of the ViBa in the full-scale model of the urban environment. Relationships between the coupling stiffness and the distance between the ViBa and the structures were established. Making use of these relationships enabled the implementation of the optimum design of both the SDOF, and the two SDOF in parallel ViBa devices. These were implemented into the full-scale model of a relevant case-study urban environment, and their performance was assed under earthquake ground motion. To this aim, the 2001 Nice earthquake time history was used. The SDOF ViBa device yielded simultaneous reductions in the maximum relative displacement ranging between 1.3% and 31%, while the two SDOF in parallel ViBa under the same excitation caused simultaneous reductions between 1.7% and 26%.

In the context of SCI experimental data is not abundant. In this research, a scale model after the full scale model of a realistic city proposed by Semblat et al. (2008) was developed. The model was tested under ground motion excitation and validation of the results was conducted against those reported by Semblat et al. (2008). The ViBa was added into the urban environment model and designed following an experimental approach. The ViBa’s performance, in terms of vibration reduction in a group of adjacent buildings, under ground motion excitation, was undertaken and results analysed in both the time and frequency domain. Reduction in the absolute amplitude of the acceleration...
time histories in some buildings of the cluster reached values of up to 21%. A negative impact of the ViBa was also noticed, nevertheless, as better discussed in Section 7.2.3, this may have been caused by imperfections in the model. The ViBa parameters defined experimentally were suitably scaled, and the ViBa was implemented into the full-scale model, where reductions were observed in all the buildings of the protected cluster. Furthermore, the capacity of the ViBa to modify the ground motion in the whole urban environment and the definition of the area of influence for the ViBa was explored both experimentally and numerically. Results indicate that the area of influence of the ViBa is limited to no more than 100 m. From an engineering point of view, therefore, the effects of the ViBa can be deemed negligible for distances greater than 100 m.

8.2 Recommendations for future work

Dynamic cross interaction, whether limited to two structures or extended to entire cities, is a complex phenomenon that takes place in three dimensions, on highly non-linear soils, involving irregular structures, under spatially non-stationary ground motion. These are some of the real complications that are often simplified in order to gain helpful insight to aid engineers produce more affordable and reliable designs. Nevertheless, it should always be the aim to incorporate as much of the real complexity as possible in the research of today that will shape the designs of tomorrow. In this regard, most of the analysis undertaken in this research involve the consideration of dynamic interaction in a two-dimensional space, providing a significant amount of ground to explore in the incorporation of additional degrees of freedom. The bounded-variable time-domain identification strategies applied to simplified models, that considered just horizontal translations, could be further expanded to take into account in plane rotations, and scenarios in which all the 6 DOF are considered is highly desirable. Moreover, the interaction with and through the soil has always been accounted for through linear elastic springs, so that the utilisation of non-linear springs is an area of possible improvement.

In the context of vibration control in urban environments, further studies are needed in the assessment of the boundaries of the so-called “dynamically independent” units of buildings to produce guidelines that are more general than those presented herein. In addition, the stochastic design of the ViBa, particularly in the case of TDOF ViBa and two SDOF in parallel devices, is not trivial and is characterised by the simultaneous effect of several parameters. Studies to evaluate the importance of the frequency range
characterising the structures in the cluster, their relative difference, and the directivity of
the solution towards a particular type of structure within the cluster are some of the
questions that remain unanswered. Furthermore, following the approaches outlined in the
present research to improve the seismic resilience of structures in an urban environment,
namely, to design an array of ViBa devices aimed to protect different sections of the city,
could be undertaken and their global effect in the urban environment assessed in terms of
overall reduction of the city’s response.

In terms of further developments of the ViBa device the issues associated with the
practical implications of its characteristics stand out, for instance the magnitude of the
mass, the size of the device, the conditions that need to be met to guarantee a rigid
foundation, and the ability to make this structure tuneable, as the urban environment
characteristics might change over time.
9 References


