Dynamic analyses of a masonry building tested in a shaking table

Guang Yang

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Master’s Thesis

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Advanced Masters in Structural Analysis of Monuments and Historical Constructions

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Abstract

It is widely known that earthquakes have been one of the main natural hazards that have damaged the historical built heritage. However, the careful analysis of the historical structures due to seismic activities is a recent endeavor.

The experimental tools adopted for assessment of the dynamic behavior of structures include dynamic identification, pseudo-dynamic tests and shaking table tests. The Finite Element Method (FEM) has also been widely used in the dynamic analysis of historical structures. As computing ability improves, more sophisticated analyses like the nonlinear time-integration analysis of models with thousands of degrees of freedom have enabled engineers to approach the real seismic behavior of structures.

This dissertation addresses the basic experimental and numerical methods in earthquake engineering. A brief state of the art is first presented. Then, the dissertation focuses with detail on the seismic assessment of masonry “Gaioleiro” buildings in Lisbon. Previously, the National Laboratory of Civil Engineering, Lisbon (LNEC), together with University of Minho, carried out a set of shaking table tests with the purpose of evaluating the seismic performance of the “Gaioleiro” buildings. This dissertation performs numerical modeling using nonlinear time-integration analyses and compares the results to the data obtained from the experimental testing. The comparison includes the displacement, velocity and acceleration on 80 typical points on the structure, the damage indicator after each earthquake input and the crack patterns. Furthermore, an updated full scale numerical model is also analyzed using nonlinear time-integration with the objective of discussing the effect of scaling in dynamic analysis.
Dynamic analyses of a masonry building tested in a shaking table
Resumo

Título: Análises dinâmicas de um edifício de alvenaria ensaiado em plataforma sísmica

É sabido que os sismos têm sido uma das principais catástrofes naturais que têm causado danos no património histórico construído. No entanto, a análise cuidada de construções históricas sob acção sísmica é um esforço recente.

As estratégias experimentais utilizadas para avaliação do comportamento dinâmico de estruturas incluem a identificação dinâmica, ensaios pseudo-dinâmicos e ensaios em plataforma sísmica. O Método dos Elementos Finitos (FEM) tem sido também muito utilizado na análise dinâmica de construções históricas. Como a capacidade do cálculo automático tem aumentado, as análises mais sofisticadas, como por exemplo a análise não-linear com integração no tempo de modelos com milhares de graus de liberdade, têm permitido aos engenheiros simular melhor o comportamento sísmico das estruturas.

Esta tese aborda métodos experimentais e numéricos frequentemente utilizados em Engenharia Sísmica. No primeiro capítulo da tese apresenta-se uma breve revisão bibliográfica. Nos capítulos seguintes, a tese aborda detalhadamente o desempenho sísmico dos edifícios Gaioleiros de Lisboa. Na primeira fase do estudo, o Laboratório Nacional de Engenharia Civil em Lisboa (LNEC) em colaboração com a Universidade do Minho realizaram um conjunto de ensaios em plataforma sísmica, tendo por objectivo a avaliar o desempenho sísmico dos edifícios Gaioleiros.

Nesta tese realizou-se a modelação numérica através de análises não-lineares com integração no tempo e comparou-se os resultados com a resposta da estrutura obtida nos ensaios experimentais. A comparação de resultados envolveu os deslocamentos, velocidades e acelerações medidos em 80 pontos da estrutura, os indicadores de dano calculados após aplicação das séries sísmicas e os padrões de fendilhação. Além disso, estudou-se também um modelo calibrado à escala real através da análise não-linear com integração no tempo, tendo por objectivo avaliar o efeito de escala na análise dinâmica.
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摘要

论文题目：砌体结构的振动台动力分析

地震是对古建筑构成损害的重要自然灾害之一。但是，有关古建筑抗震的深入研究仅仅在最近几十年才开始。

实验方法包括动力损伤识别，拟动力测试和振动台试验。另外，有限单元模拟在古建筑动力分析中也得到广泛应用，尤其在大型计算机迅速发展的今天，几十万，甚至几百万自由度模型的非线性时程分析使得工程师们可以更好的模拟结构在地震作用下的反应。

本文首先介绍了地震工程中常用的实验及理论研究方法。首先，对建筑的动力分析做了简要介绍。随后，本文重点放在了“伽莱罗”砌体结构的抗震分析当中。2004年，米尔奥大学协同葡萄牙里斯本的土木工程国家实验室共同进行了一组振动台试验，目的是研究“伽莱罗”砌体结构的抗震性能，本文进行了对实验模型的有限单元模拟，并进行了非线性时程分析及并对比了实验数据。对比主要包括，模型上八十个典型位置的加速度，速度及位移对比，以及裂缝形式及相应的应力应变分析。除此之外，在缩尺模型的基础上，一个足尺的有限单元模型也被建立起来，用以分析尺寸效应在非线性时程分析中的影响。
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Index

Acknowledgements
Abstract
Resumo
摘要

1 Introduction
1.1 Motivation for Dynamic Analysis on Masonry Structures ................................................................. 2
1.2 Focus of the Dissertation .................................................................................................................. 3
1.3 Outline of the Dissertation ............................................................................................................. 4

2 Basic Experimental and Numerical Methods in Earthquake Engineering
2.1 Introduction ................................................................................................................................... 8
2.2 Basic Dynamics ............................................................................................................................. 8
  2.2.1 Single-Degree-of-Freedom Systems ....................................................................................... 10
  2.2.2 Multi-Degree-of-Freedom Systems ....................................................................................... 11
2.3 Experimental Methods ................................................................................................................ 13
  2.3.1 In-situ Vibration Tests ....................................................................................................... 14
    2.2.3.1 Basic Dynamic Identification Method ..................................................................... 14
    2.2.3.2 Description of In-situ Vibration Tests ..................................................................... 16
  2.3.2 Laboratory Dynamic Tests ................................................................................................... 20
    2.3.2.1 Pseudo-dynamic Tests ............................................................................................. 21
      2.3.2.1.1 Introduction of the Pseudo-dynamic Tests ................................................ 21
      2.3.2.1.2 Case of the Pseudo-dynamic Tests .......................................................... 22
Dynamic analyses of a masonry building tested in a shaking table

2.3.2.2 Shaking Table Tests ........................................................................................................ 23
   2.3.2.2.1 Introduction of Shaking Table Tests ........................................................................ 23
   2.3.2.2.2 Case of Shaking Table Tests ................................................................................. 24
2.4 Numerical Methods .................................................................................................................. 27
   2.4.1 Time Integration Analysis .................................................................................................. 27
      2.4.1.1 Description of the Time Integration Analysis .............................................................. 27
      2.4.1.2 Numerical Approximation Procedures of the Method ............................................... 28
      2.4.1.3 Incremental Formulation for Nonlinear Analysis ...................................................... 30
   2.4.2 Pushover Analysis ............................................................................................................ 31
      2.4.2.1 Basic Introduction of Pushover Analysis ................................................................. 31
      2.4.2.2 Basic Steps of Pushover Analysis ............................................................................ 31
2.5 Conclusion .............................................................................................................................. 33

3 Description of the Prototype and the Scaled Model in Shaking Table Tests
3.1 Introduction of the Prototype - Gaioleiro .............................................................................. 36
3.2 Description of the Shaking Table Tests ............................................................................... 36
   3.2.1 Introduction of the Mock-up ........................................................................................... 37
   3.2.2 Description of the Testing Procedures ......................................................................... 39
   3.2.3 Introduction of the Equipment ....................................................................................... 41
3.3 Description of the Numerical Model ..................................................................................... 42
3.4 Calibration of the Numerical Model ...................................................................................... 45
3.5 Conclusion .............................................................................................................................. 52

4 Dynamic Analysis of the Scaled Model
4.1 Description of the Nonlinear Time-Integration Analysis ....................................................... 54
4.2 Model Optimization .............................................................................................................. 55
   4.2.1 Discussion about the Beams ......................................................................................... 56
   4.2.2 Discussion about the Stiffness and Damping Ratio ....................................................... 60
4.3 Results of the analysis ......................................................................................................... 62
   4.3.1 Results of 25% of the PGA .......................................................................................... 62
      4.3.1.1 Acceleration Comparison ....................................................................................... 62
      4.3.1.2 Displacement Comparison ..................................................................................... 64
      4.3.1.3 Principal Strain ...................................................................................................... 66
   4.3.2 Results of 100% of the PGA ........................................................................................ 67
4.3.2.1 Acceleration and Displacement Comparison ........................................................................................................ 67
4.3.2.2 Principal Strain and Crack Patterns .................................................................................................................. 70
4.3.2.3 Piers on the Facade ........................................................................................................................................ 70
4.3 Conclusion ......................................................................................................................................................... 71

5 Dynamic Analysis of the Full Scale Model
5.1 Basic Introduction of Scaling ............................................................................................................................... 74
5.1.1 Size effect on Strength ....................................................................................................................................... 74
5.1.2 Size effect on Shaking Table Tests .................................................................................................................... 76
5.1.3 Discussion in this case ......................................................................................................................................... 77
5.2 Results of the Full Scale Model ............................................................................................................................ 81
5.2.1 Displacement ..................................................................................................................................................... 81
5.2.2 Principal Strain and Piers on the Facade .............................................................................................................. 82
5.3 Conclusion ......................................................................................................................................................... 83

6 Conclusions and the Future Works
6.1 Conclusions ............................................................................................................................................................ 86
6.2 Future Works ......................................................................................................................................................... 87

References
Appendixes
Chapter 1

Introduction
1.1 Motivation for Dynamic Analysis on Masonry Structures

Masonry can be defined as a material usually made with individual units laid in and bonded with mortar. It is widely used in historical constructions, see Figure 1.1. Masonry provides an aesthetic appearance, is durable, and can be relatively easy to apply. However, low tensile strength (almost 1/10 of the compressive strength) is the main shortcoming of masonry structures. As a composite material, masonry contains complicated features, namely: a) Composite character (stone/brick and mortar); b) Quasi-brittle response in tension and almost null tensile strength; c) Frictional response in shear (with the loss of bond between units and mortar); d) Anisotropy (response depends of the stress orientation).

![Figure 1.1 – Examples of the world-famous historical masonry structures:](image)
a) The Great Wall, China; and b) Taj Mahal, India.

The protection of the historical structures from the natural hazards has been an object of many studies and discussions all over the world. Carta di Veneza (1964) has defined the main principles on how the protection should be done in order to keep its most authentic value (Oliveira, 2003).

It is widely known that earthquake has been one of the main natural hazards that has damaged the historical built heritage, see Figure 1.2. Information from the United Nations reveals that the percentage of deaths originated by seismic events was 26% of the total number of casualties caused by natural disasters, with an estimate of more than fourteen million of victims since 1755 (Barbat et al., 2006). To further minimize destruction under the seismic activity, it is necessary to know how the structures perform during earthquakes.

The intent of dynamic analysis of historic structures is to: a) provide technical data for use in design codes; and b) model the structure performance so as to aid in the decision making during the strengthening work. Historical constructions, especially historical masonry, need
more attention in dynamic analysis because of the special features of the material; the
complicated initial constructions and the previous degradation of the structure and its elements
due to natural hazards, including earthquake effects, man-made modifications and many other
effects.

![Image](https://example.com/image1.png)

Figure 1.2 – Some damage to the historical structures (Oliveira, 2003):
a) the Lisbon Earthquake, 1755; and b) the Izmit earthquake, 1999.

The careful analysis of the historical structures due to seismic activities is a rather recent
endeavor. Research in this field began in the past few decades and there are now several
techniques used in the seismic analyses of historical structures. These started with basic
dynamic identification, pseudo-dynamic tests, and shaking table tests. Since the Finite Element
Method (FEM) was invented in the 1960s, it has been widely used in the dynamic analysis of
historical structures. As computing ability improves, more sophisticated analyses also have
developed. For instance, the non-linear, time-integration analyses for models with thousands of
degrees of freedom have enabled engineers to better approach the seismic performances of
structures under real seismic activities.

1.2 Focus of the Dissertation

This dissertation addresses the basic experimental and numerical methods in earthquake
engineering. With respect to the dynamic theories and the analyzing methods, some real cases
are also presented. Then, the dissertation focuses with detail on the seismic assessment of
masonry “Gaioleiro” buildings in Lisbon. The “Gaioleiro” buildings are, usually, four or five
stories high, with masonry walls sand timber floors and roof. The external walls are, usually, in
rubble masonry with lime mortar (Pinho, 2000).

Previously, the National Laboratory of Civil Engineering, Lisbon (LNEC), together with
University of Minho, carried out a set of shaking table tests with the purpose of evaluating the
seismic performance of the so called “Gaioleiro” buildings (Candeias et al., 2004). This dissertation performs numerical modeling using nonlinear time-integration analyses and compares the results to the data obtained from the previous testing. Furthermore, an updated full scale numerical model is also analyzed using nonlinear time-integration with the objective of discussing the effect of the scaling in dynamic analysis.

1.3 Outline of the Dissertation

The dissertation is organized in six chapters by as follows:

- **Chapter 1** is the introduction of the work, including the motivation, the focus and the outline of the dissertation;

- **Chapter 2** presents a state of the art in basic experimental and numerical methods in earthquake engineering. First, a brief introduction on dynamics is carried out, followed by a main review of the basic experimental and numerical methods. Issues about experimental methods addressed include in-situ tests and laboratory tests; then, issues about numerical methods include time-integration method and pushover method. Some examples are also discussed in this part;

- **Chapter 3** presents a description of the prototype, the scaled mock-up and the numerical model. First, a brief introduction on “Gaioleiro” prototype is provided, followed by the description of the shaking table tests previously carried out, including the scaled model, the equipments adopted and the testing procedures. Then the FEM model is introduced, with the calibration of the numerical model in detail;

- **Chapter 4** presents an analysis of the experimental and numerical results of the scaled model. First, for the purpose of optimization, five different numerical models are used for time-integration analysis, and then compared with the experimental data for 25% of the Peak Ground Acceleration (PGA). Then, the selected model is used to carry out 100% of the PGA. Aspects considered in the comparisons include: a) the accelerations and displacements of 80 typical points in four facades; b) the parameters of all the piers on North/South facades; c) the maximum strain on each facade and crack patterns. d) the damage factor after the 25% and 100% of the PGA;
• Chapter 5 addresses the issue of scaling in shaking table tests. As addressed in Chapter 3, additional masses should have been taken into account in the 1:3 models tested in the shaking table. Therefore, a calibrated numerical full scale model will be put into nonlinear time-integration analysis to evaluate the effect of the self weight. The full scale model is again computationally tested for 25% and 100% PGA, to compare the new results with the scaled model results. In the end, some conclusions are provided.

• Chapter 6 presents the main conclusions of each chapter and a proposal for future work.
Dynamic analyses of a masonry building tested in a shaking table
Chapter 2
Basic Experimental and Numerical Methods in Earthquake Engineering

Abstract

In this chapter, a state of the art in basic experimental and numerical methods in earthquake engineering is presented. First, a brief introduction on dynamics is carried out, followed by a main review of the basic experimental and numerical methods. Issues about experimental methods addressed include in-situ tests and laboratory tests; then, issues about numerical methods include time integration method and pushover method. Some examples are also discussed in this part.
2.1 Introduction

The main goal of the present chapter is to review the most important methods in the field of earthquake engineering. Before presenting the experimental and numerical methods, a brief introduction on basic dynamics is carried out, aiming to introduce basic concepts and definitions later mentioned.

2.2 Basic Dynamics

In opposite to static, dynamic can be simply defined as time varying, meaning that a dynamic load is any load with magnitude, direction, and/or position varying with time. Similarly, the structural response to a dynamic load, i.e. the resulting stresses and deflections, is also time varying, or dynamic.

There are two basically different approaches for evaluating the structural response to dynamic loads: deterministic and nondeterministic. If the time variation of loading is fully known, even though it may be highly oscillatory or irregular in character, it will be referred to herein as a prescribed dynamic loading; and the analysis of the response of any specified structural system to a prescribed dynamic loading is defined as a deterministic analysis. On the other hand, if the time variation is not completely known but can be defined in a statistical sense, the loading is termed a random dynamic loading; and its corresponding analysis of response is defined as a nondeterministic analysis. Generally, structural response to any dynamic loading is expressed basically in terms of the displacements of the structure. Thus, a deterministic analysis leads directly to displacement time-histories corresponding to the prescribed loading history; other related response quantities, such as stresses, strains, internal forces, etc., are usually obtained as a secondary phase of the analysis (Clough et al., 1995).

The prescribed loading may be divided into two categories, Periodic and Nonperiodic load. The simplest periodic loading is shown in Figure 2.1 a, termed simple harmonic; other kind of periodic loading is more complex with more frequencies, see Figure 2.1 b. Using Fourier analysis, any periodic loading can be represented as a sum of a series of simple harmonic components, which can be dealt using in a general procedure. Nonperiodic loadings can be short-duration impulsive loadings like a blast or explosion, see Figure 2.1 c, or long-duration general forms of loads like typical earthquake loading, see Figure 2.1 d. Generally, long-duration loading can be treated only by completely general dynamic analysis procedures.
Chapter 2 – Basic Experimental and Numerical Methods in Earthquake Engineering

The three main characteristics of dynamic systems are stiffness, damping and mass. For damped systems, energy is dissipated, resulting in the reduction of the amplitudes of motion. For undamped systems, once the movement is started, the body or the system remains in harmonic motion indefinitely.

Structural systems can be considerate as discrete or continuous, depending on the level of analysis sought. Normally, civil engineering structures are discretized in a series of key points, being their characterization of movement enough to understand the entire system. Each of these points may have a maximum of six degrees of freedom (three translations and three rotations) (Chopra, 2001).

In most cases, especially for analysis in civil engineering, an approximate analysis involving only a limited number of degrees of freedom will provide sufficient accuracy. The mathematical expressions defining the dynamic displacement are called the equations of motion of the structure. Three methods are used to get the equations of motion: a) Direct Equilibration Using d'Alembert's Principle; b) Principle of Virtual Displacements; and c) Variational Approach. All of these methods can be used generally, but one may be simpler than other according to different application.

Then, some classic equations of motion in Single-Degree-of-Freedom Systems (SDOF) and Multi-Degree-of-Freedom Systems (MDOF) will be presented.
2.2.1 Single-Degree-of-Freedom Systems

In the simplest model of a SDOF system, each of these properties discussed above is assumed to be concentrated in a single physical element, see Figure 2.2 a. The entire mass m of this system can move only in simple translation; thus, the single displacement coordinate v(t) completely decides its position. The elastic resistance to displacement is provided by the weightless spring of stiffness k, while the energy-loss mechanism is represented by the damper c. The external dynamic loading producing the response of this system is the time-varying force p(t), see Figure 2.2 b.

![Figure 2.2 - Idealized SDOF system: (a) basic components; (b) forces in equilibrium.](image)

Using d'Alembert's principle, the forces acting in the direction of the displacement degree of freedom are the applied load p(t) and the three resisting forces resulting from the motion, i.e., the inertial force $f_I(t)$, the damping force $f_D(t)$, and the spring force $f_S(t)$. The equation of motion is merely an expression of the equilibrium of these forces as given by:

$$f_I(t) + f_D(t) + f_S(t) = p(t)$$  \hspace{1cm} (2.1)

Changing the $f_I(t)$, $f_D(t)$, $f_S(t)$ according to their own equations, the equation of motion is found to be:

$$m \ddot{v}(t) + c \dot{v}(t) + k v(t) = p(t)$$  \hspace{1cm} (2.2)

Assume the load to be harmonic load having an amplitude $p_o$ and circular frequency $\omega$, the equation can be changed into:

$$m \ddot{v}(t) + c \dot{v}(t) + k v(t) = p_o \sin \omega t$$  \hspace{1cm} (2.3)
A straightforward mathematical process provides the response of system under harmonic load, found as:

\[
v(t) = \left[ A \cos \omega_D t + B \sin \omega_D t \right] \exp(-\xi \omega t) \\
+ \frac{p_o}{k} \left[ \frac{1}{(1 - \beta^2)^2 + (2 \xi \beta)^2} \right] \left[ (1 - \beta^2) \sin \omega_D t - 2 \xi \beta \cos \omega_D t \right]
\]  
(2.4)

Here, \( A, B \) depend on the initial conditions: initial displacement \( v(t) \) and initial velocity \( v'(t) \), given by:

\[
\beta \equiv \frac{\omega}{\omega_D}
\]  
(2.5)

\[
\omega_D \equiv \omega \sqrt{1 - \xi^2}
\]  
(2.6)

and \( \xi \) is the damping ratio:

\[
\xi \equiv \frac{c}{c_c} = \frac{c}{2m\omega}
\]  
(2.7)

For an arbitrary force acting in the system, the solution of this second order differential equation can be obtained by the Duhamel’s integral (Chopra, 2001), valid for linear systems and given by the following expression, in which \( q(t) \) is equal to \( v(t) \) in equation (2.2):

\[
q(t) = \frac{1}{m\omega_D} \int_0^t p(\tau) e^{-\xi \omega_D (t-\tau)} \sin[\omega_D (t-\tau)] d\tau, \quad t > \tau
\]  
(2.8)

2.2.2 Multi-Degree-of-Freedom Systems

The equation of motion of the system of a MDOF system can be formulated by expressing the equilibrium of the effective forces associated with each of its degrees of freedom. So the general expression can be made in matrix form:

\[
f_I + f_D + f_S = p(t)
\]  
(2.9)

where \( k, c, m \) are defined as stiffness matrix, damping matrix and mass matrix:

\[
f_S = k \, v
\]  
(2.10)

\[
f_D = c \, \dot{v}
\]  
(2.11)

\[
f_I = m \, \ddot{v}
\]  
(2.12)

Thus, Equation (2.9) can be changed into:

\[
m \ddot{v}(t) + c \, \dot{v}(t) + k \, v(t) = p(t)
\]  
(2.13)
Then the concepts of eigenvalue or characteristic value problems are required. For a freely vibrating undamped system MDOF system, the equations of motion can be obtained by omitting the damping matrix $c$:

$$\mathbf{m} \ddot{\mathbf{v}} + \mathbf{k} \mathbf{v} = \mathbf{0} \quad (2.14)$$

Let:

$$\mathbf{v}(t) = \hat{\mathbf{v}} \sin(\omega t + \theta) \quad (2.15)$$

and the following equation can be obtained as:

$$[\mathbf{k} - \omega^2 \mathbf{m}] \hat{\mathbf{v}} = \mathbf{0} \quad (2.16)$$

The solution of Equation (2.16) will be the eigenvalue vector $\mathbf{\omega}$, see Equation (2.17) and matrix in Equation (2.18) provides the mode shape vectors for the free vibration.

$$\mathbf{\omega} = \begin{Bmatrix} \omega_1 \\ \omega_2 \\ \omega_3 \\ \vdots \\ \omega_N \end{Bmatrix} \quad (2.17)$$

$$\mathbf{\Phi} = [\phi_1 \phi_2 \phi_3 \cdots \phi_N] = \begin{bmatrix} \phi_{11} & \phi_{12} & \cdots & \phi_{1N} \\ \phi_{21} & \phi_{22} & \cdots & \phi_{2N} \\ \phi_{31} & \phi_{32} & \cdots & \phi_{3N} \\ \phi_{41} & \phi_{42} & \cdots & \phi_{4N} \\ \vdots & \vdots & \ddots & \vdots \\ \phi_{N1} & \phi_{N2} & \cdots & \phi_{NN} \end{bmatrix} \quad (2.18)$$

The displacement vector $\mathbf{v}$ may be also expressed by summing the modal vectors as:

$$\mathbf{v} = \phi_1 Y_1 + \phi_2 Y_2 + \cdots + \phi_N Y_N = \sum_{n=1}^{N} \phi_n Y_n \quad (2.19)$$

So the equation of motion can be obtained as:

$$\phi_n^T \mathbf{m} \phi_n \ddot{Y}_n(t) + \phi_n^T \mathbf{k} \phi_n Y_n(t) = \phi_n^T \mathbf{p}(t) \quad (2.20)$$

and the Duhamel integral will be given by:

$$Y_n(t) = \frac{1}{M_n \omega_n} \int_0^t P_n(\tau) \exp \left[-\xi_n \omega_n (t - \tau)\right] \sin \omega_n (t - \tau) \ d\tau \quad (2.21)$$
2.3 Experimental Methods

In the previous section, some basic reviews of the basic dynamics were presented in order to better understand the experimental methods and numerical methods later addressed. As referred, for the equation of motion of any system, three main characteristics are stiffness, damping and mass. The object of the simplest dynamic testing is to measuring these dynamic characteristics of the structures, however, the purposes of the experimental methods of the dynamic analysis of historical structures now has been spreaded for: a) assessment of safety and definition of reliability; b) understanding existing damage; c) the evaluation of strengthening and/or repair efficiency; d) measuring dynamic characteristics of buildings, e.g. natural frequencies, damping values, hysteretic response.

In short, dynamic testing can be used to obtain modal characterization, but also to check and evaluate the statistical parameters of the recorded response signals, supplying quantitative information. The influence factors, the properties and the response involved in structural dynamic tests are provided in Figure 2.3.

![Figure 2.3 – Influence, properties and response involved in structural dynamic tests.](image-url)
2.3.1 In-situ Vibration Tests

Vibration testing is now a mature and widely used tool in the analysis of structural systems, as a non-destructive test that can be valuable for historic structures. Vibration tests on historic buildings might have the following goals: a) understand how the structure behaves; b) find when a damage failure threshold occurs; c) locate damage in the structure.

Typically two types of vibration tests may be applied: sinusoidal and random. The sinusoidal test consists of a certain acceleration level (amplitude, energy output) combined with a frequency sweep at a range from an initial frequency to a final frequency. The sinusoidal tests can illustrate the resonant frequencies of the structure and allow often an easy mathematical treatment. A structure in a random vibration test is exposed to energies at all frequencies in the bandwidth selected and the analysis requires a statistical description (Pospíšil, 2002)

Before presenting the vibration tests, some basic introduction of the dynamic identification are made.

2.2.3.1 Basic Dynamic Identification Method

The purpose of dynamic identification is to identify structural damage and deterioration according to the measured modal parameters. The conventional modal identifications is based on structural response induced by known excitation such as a simple harmonic vibration imposed by an artificial excitation. The structural parameters are identified either from the free vibration decay traces, after the excitation is removed, or simply as a transfer function between input and output of the system.

Various techniques have been developed to determine the structural parameters. The basic idea is to assume mathematical models for the system and then to minimize the prediction errors by fitting the models against the obtained output data.

In what concerns in situ modal identification tests there are two groups of experimental techniques: (a) the input-output vibration tests, where the excitation forces and the vibration response are measured; (b) the output-only tests, where only the response of the system is measured (from ambient vibration or from the free vibration tests, where an initial deformation and then are quickly released).
The input-output vibration tests are based on the control of the input excitation and the measurement of the structural time history response in a set of selected points, see Figure 2.4. The modal parameters (natural frequencies, mode shapes and damping coefficients) are then calculated by estimating the FRFs or the Impulse Response Functions (IRF), either in frequency or time domain (Ramos, 2007).

![Figure 2.4 – Principle of the input-output vibration tests (Ramos, 2007).](image)

The output-only tests are based on the dynamic response measurements of a virtual system under natural (ambient or operational) conditions, and the assumption that the excitations are of random nature in time and in the physical space of the structure, see Figure 2.5.

![Figure 2.5 – Principle of the output-only vibration tests (Ramos, 2007).](image)

Damage on masonry structures are mainly cracks, foundation settlements, material degradation and excessive deformations. When cracks occur, generally they are localized, splitting the structures in macro-blocks. The use of dynamic based methods to identify the damage is an attractive tool to use in this type of structures due to the modern requirements of unobtrusiveness, minimum physical intervention and respect of the original construction. The assumption that damage can be associated with the decrease of stiffness seems to be reasonable to this type of structures (Ramos, 2007).

The free vibration tests, where the systems are induced with an initial deformation and then are quickly released. The response of the structure will be measured to obtain the modal parameters of the system.
2.2.3.2 Description of In-situ Vibration Tests

Vibration testing requires the following successive tasks: acquisition of data with a sensing system; communication of the information gathered; intelligent processing and analyzing of data; storage of processed data; diagnostics (e.g. damage detection or comparison with numerical simulation).

Generally, an irregular or unexpected property of the dynamic response can reveal a “symptom” of structural weakness. The presence of several symptoms might make a direct and deterministic interpretation difficult, meaning that “pattern recognition” or artificial intelligence techniques may be effective. Alternative methods are those founded on updating numerical models: model correction is based on the results of a previous structural identification (model updating techniques).

2.2.3.3 Examples of In-situ Vibration Tests

Selected examples of in situ vibration tests are briefly described next, to illustrate how these techniques are applied to cultural heritage buildings:

a) Church of Monastery of Jeronimos, Lisbon, Portugal

The Monastery of Jeronimos, located in Lisbon, is one of the most famous Portuguese monuments, with a length of 70m and a width of 40m. The main nave of the church, see Figure 2.6, was tested using output-only modal identifications techniques, which provided the modal parameters: resonant frequencies, mode shapes and damping coefficients.

![Figure 2.6](image1.jpg)

**Figure 2.6** – The main nave of the church: a) front view; b) side view.
Two techniques were applied to compare the experimental dynamic parameters obtained and to obtain more accurate results, namely the Enhanced Frequency Domain Decomposition (EFDD) and the Stochastic Subspace Identification (SSI) methods. Figure 2.7 illustrates details of the in situ vibration tests, namely locations of fixed and moving sensors to identify the vibration modes and adopted sensors. Table 2.1 gives six natural frequencies, damping ratios and MAC values estimated by two different output-only system identification techniques. The natural frequencies range from 3.7 to 12.4Hz. Moreover, from the data of the accelerometer, the experimental mode shapes can be obtained, see Figure 2.8.

<table>
<thead>
<tr>
<th>Mode shape</th>
<th>$\omega_i$ (Hz)</th>
<th>$\zeta$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EFDD</td>
<td>SSI</td>
<td>EFDD</td>
</tr>
<tr>
<td>Mode 1</td>
<td>3.69</td>
<td>3.68</td>
</tr>
<tr>
<td>Mode 2</td>
<td>5.12</td>
<td>5.04</td>
</tr>
<tr>
<td>Mode 3</td>
<td>6.29</td>
<td>6.30</td>
</tr>
<tr>
<td>Mode 4</td>
<td>7.23</td>
<td>7.29</td>
</tr>
<tr>
<td>Mode 5</td>
<td>9.67</td>
<td>9.65</td>
</tr>
<tr>
<td>Mode 6</td>
<td>12.45</td>
<td>12.51</td>
</tr>
</tbody>
</table>

Figure 2.7 – Details of in situ dynamic tests:
   a) location of moving and fixed sensors; (b) strong motion accelerometers used.

Table 2.1 – Comparison of the estimated modal parameters of the main nave.

Figure 2.8 – Mode shape results from EFDD method: (a) 1st mode (3.7Hz); (b) 2nd mode (5.1Hz).
As shown in Table 2.1, MAC values are calculated for the eight mode shape vectors obtained from two experimental techniques. The MAC criterion is the most well known procedure to study the correlation between to sets of mode shape vectors. The results vary from 0 to 1, i.e. from bad to good correlation. Observing the table values, the two first mode shapes are highly correlated (values closed to the unit), but for the rest of the modes the values decreases, to a minimum of 0.36.

Nevertheless, the modal identification seems to be acceptable if the structural complexity of the main nave is taken into consideration. Even if the mode shape and damping coefficients estimation is not very accurate for the higher modes, the resonant frequencies were accurately calculated by the two experimental techniques for all estimated modes. (Ramos and Lourenço, 2005)

b) Mexico City Cathedral, *Mexico City, Mexico*

The Mexico City Cathedral is one of the most important colonial monuments of the Americas, see Figure 2.9. It has five longitudinal naves; the central one is covered by a barrel vault, and the lateral ones are covered by spherical domes. Two rows of eight columns support the lateral nave and its heavy central dome. A close array of robust masonry walls divides the extreme naves into small chapels constituting, along with the main(south) facade, the apse and some buttresses, a very stiff system that provides great lateral strength in the transverse direction of the structure. In the longitudinal direction, the lateral strength and stiffness are primarily provided by the exterior walls along the east and the west facades of the cathedral. The building foundation consists of a grid of foundation beams and a thick masonry mat over a dense array of short timber piles.
The main objective of the research project is to provide quantitative information during the evaluation of the seismic safety of the Mexico City Cathedral through the interpretation of a set of instrumental records of its vibration under several moderate ground motions recorded by its seismic network. All seismic records were analyzed in the frequency domain, obtaining Fourier amplitude spectra and response spectra as well as spectral ratios among motions recorded at different points. From the analyses in the time domain, histories of absolute and relative displacements between different points were also obtained. The network is composed of three-directional digital 18 bits accelerographs that are simultaneously triggered to provide a continuous record with a common time basis. The initial array was composed of eight accelerographs, see Figure 2.10.

Figure 2.10 – Schematic view of the initial array of accelerographs.

Twenty seismic events were recorded by the network from 1997 to 2003. In all the cases epicenters were located at least 100 km away. Some data of the biggest earthquakes recorded are shown in Table 2.2.

<table>
<thead>
<tr>
<th>Event</th>
<th>Mag. (Mc)</th>
<th>Comp.</th>
<th>AC</th>
<th>AN</th>
<th>AS</th>
<th>AW</th>
<th>SC</th>
<th>SN</th>
<th>SS</th>
<th>CL</th>
<th>TB</th>
<th>TA</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1</td>
<td>7.3</td>
<td>N–S</td>
<td>14.99</td>
<td>14.31</td>
<td>17.48</td>
<td>13.46</td>
<td>12.64</td>
<td>12.40</td>
<td>12.69</td>
<td>14.84</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>01/11/97</td>
<td>7.3</td>
<td>V</td>
<td>4.74</td>
<td>-3.72</td>
<td>-4.37</td>
<td>4.89</td>
<td>-3.81</td>
<td>-3.54</td>
<td>-4.08</td>
<td>-5.75</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>04/22/97</td>
<td>5.9</td>
<td>E–W</td>
<td>15.89</td>
<td>13.65</td>
<td>15.43</td>
<td>18.09</td>
<td>12.47</td>
<td>12.15</td>
<td>12.54</td>
<td>-14.21</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>E2</td>
<td>5.9</td>
<td>N–S</td>
<td>-3.67</td>
<td>3.45</td>
<td>-6.02</td>
<td>3.56</td>
<td>-2.78</td>
<td>-2.71</td>
<td>-2.76</td>
<td>3.86</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>07/19/97</td>
<td>6.3</td>
<td>V</td>
<td>2.73</td>
<td>2.23</td>
<td>2.05</td>
<td>1.81</td>
<td>1.86</td>
<td>1.92</td>
<td>1.91</td>
<td>2.09</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>E3</td>
<td>6.3</td>
<td>N–S</td>
<td>-2.09</td>
<td>-2.05</td>
<td>-2.37</td>
<td>-1.99</td>
<td>-1.88</td>
<td>-1.89</td>
<td>-1.91</td>
<td>-2.01</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>04/20/98</td>
<td>5.4</td>
<td>E–W</td>
<td>-1.88</td>
<td>-1.68</td>
<td>-1.88</td>
<td>-2.03</td>
<td>-1.40</td>
<td>-1.54</td>
<td>-1.40</td>
<td>1.86</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>E4</td>
<td>5.4</td>
<td>N–S</td>
<td>2.87</td>
<td>2.57</td>
<td>4.09</td>
<td>2.59</td>
<td>1.77</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>2.87</td>
<td>-3.71</td>
</tr>
<tr>
<td>06/21/00</td>
<td>5.9</td>
<td>V</td>
<td>-2.47</td>
<td>-1.68</td>
<td>-1.56</td>
<td>-1.54</td>
<td>-1.08</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>2.04</td>
<td>-1.63</td>
</tr>
<tr>
<td>E5</td>
<td>5.9</td>
<td>N–S</td>
<td>-18.39</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>-5.93</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>8.35</td>
<td>11.61</td>
</tr>
<tr>
<td>06/21/00</td>
<td>5.9</td>
<td>E–W</td>
<td>-7.15</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>-5.42</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>10.16</td>
<td>5.26</td>
</tr>
</tbody>
</table>

Table 2.2 – Max accelerations recorded in the events (unit: gal).
For engineering purposes, characteristics of the seismic ground motion are usually studied through acceleration response spectra for 5% damping. A set of these spectra is shown in Figure 2.11 a, for the N-S direction at the ‘free-field’ station. It can be seen that spectral shapes change with earthquake magnitude, and the earthquakes of highest intensity show a significant amplification of response for long periods. This is attributed to the higher content of long period waves in large magnitude earthquakes compared with that in moderate magnitude events.

![Figure 2.11 – Results in N-S direction:](image)

a) Acceleration response spectra; (b) the ratio of Fourier spectral amplitudes.

To better identify site effects, spectral ratios of the Fourier spectral amplitudes were obtained between the cathedral site and a station located on firm ground in Mexico City, where the same events were recorded (CU station). These functions, shown in Figure 2.11 b, for the five most significant events and for N-S direction, allow one to clearly identify the first mode of vibration of the soil deposits at the site, at approximately 0.37 Hz for the N–S direction. The seismic network installed at the cathedral has been successful in terms of the number and quality of records gathered in a limited time span and of the knowledge derived from them (Riveral and Meli et al., 2008).

2.3.2 Laboratory Dynamic Tests

In the previous section, the in-situ vibration tests were presented. For the in-situ tests, the purposes are normally evaluating the safety, monitoring the health and analyzing for the strengthening. For the purpose of achieving the further, and more accurate, information on seismic behavior of historical structures, laboratory dynamic tests are necessary. In the following section, two basic laboratory dynamic tests are presented.
2.3.2.1 Pseudo-dynamic Tests

2.3.2.1.1 Introduction of the Pseudo-dynamic Tests

The pseudo-dynamic (PSD) test method is a displacement-based experimental technique that is used to simulate the seismic response of structures. It utilizes feedback signals from a test structure in a numerical integration algorithm to sequentially solve the equations of motion to determine command displacements.

The pseudo-dynamic testing procedure is a simultaneous simulation and control process in which inertia and damping properties are simulated and stiffness properties are acquired from the structure. The procedure calculates a set of dynamic displacements based on active control theory, utilizing the simulated inertia and damping properties and acquired stiffness properties under a hypothetical ground motion, and simulates the response of the structure under seismic motion in a quasi-static fashion (Aktan, 1986). The calculations are finished by computer. The whole working principle is presented in Figure 2.12.

![Figure 2.12 – The working principle of the pseudo-dynamic tests.](image)

The pseudo-dynamic test seems to be a reliable test to simulate the response of the structures under seismic activities. The tests can be applied for the nonlinear dynamic analysis with full-scaled tests, and compared to other dynamic tests. The pseudo-dynamic tests can also monitor the performance of the structures under a relatively slow pseudo-dynamic load, which provides a more reliable and accurate result.

However, the disadvantages are also apparent: a) the tests can not show the real response of the structure under earthquake in time history; b) material properties cannot be adjusted with the time history; c) the calculation of displacements is complex and they depend on the load,
meaning that the accuracy is hard to control; d) the load is applied in certain locations, and so
the method can only be applied on the simplified mass concentration structure (MDOF); e) the
tests also has high requirements of the equipments, and it is usually rather costly.

2.3.2.1.2 Case of the Pseudo-dynamic Tests – the Pseudo-dynamic testing of unreinforced
masonry building with flexible diaphragm

A full-scale one-story unreinforced brick masonry specimen, see Figure 2.13 a, having a
wood diaphragm was subjected to earthquake excitations using pseudo-dynamic testing. The
specimen was designed to better understand the flexible-floor/rigid-wall interaction, the impact
of wall continuity at the building corners and the effect of a relatively weak diaphragm on the
expected seismic behavior.

![Figure 2.13 – Pseudo-Dynamic test of a masonry wall:](image)

The unreinforced brick masonry specimen was subjected to a first series of tests under an
earthquake of progressively increasing intensity. The test set-up is shown in Figure 2.13 b.
Non-linear inelastic analyses were conducted to determine an appropriate seismic input motion
that would initiate significant pier rocking from the diaphragm response. The selected input
motion is a synthetic ground motion for La Malbaie, Canada with a peak ground acceleration of
0.453g, see Figure 2.14.

As for the results, Figure 2.15 illustrates the behavior observed during the tests. A stable
combined rocking and sliding mechanisms formed and large deformations developed without
significant strength degradation. The diaphragm remained, however, essentially elastic
throughout. The difference in the wall’s response due to the presence of continuous or
discontinuous corners was somehow negligible during high intensity seismic excitation producing inelastic wall response. (Paquette and Bruneau, 2006)

![Acceleration time history for La Malbaie](image)

Figure 2.14 – Acceleration time history for La Malbaie (peak ground acceleration of 0.453g).

![Response of the structure](image)

Figure 2.15 – The response of the structure:
  a) Door pier rocking response; b) hysteretic response of wood diaphragm at center span

2.3.2.2 Shaking Table Tests

2.3.2.2.1 Introduction of Shaking Table Tests

Among the several different experimental techniques that can be used to test the response of structures to verify their seismic performance, shaking table tests are the most direct and accurate way to detect the seismic performance of the structures. This is a device for shaking structural models or building components with a wide range of simulated ground motions, including reproductions of recorded earthquakes time-histories. For the test, the shaking table is given an excitation of a calibrated earthquake input, and many accelerometers are setup onto
the structures to obtain the data of the response during the dynamic loading. After analyzing the data of the accelerometer, the general parameters of response can be obtained and analyzed furthermore. While modern tables typically consist of a rectangular platform that is driven in up to six degrees of freedom (DOF) by servo-hydraulic or other types of actuators, the earliest reported uses of shake tables date back more than a century (Omori, 1900).

Generally, test specimens are fixed to the platform and shaken, often adding load to the point of failure. Using video cameras and data from transducers, accelerometers normally, it is possible to interpret the dynamic behaviors of the specimen. Shaking table tests are used extensively in seismic research, as they provide the ways to excite structures in such a way that the structures are subjected to conditions representative of true earthquake ground motions.

Shaking table tests results are also the main way used in this dissertation and will be presented in detail in the following chapters. To better understand the tests, an example of a real case is introduced here.

2.3.2.2.2 Case of Shaking Table Tests

In many European countries like France, Greece, Italy, Portugal, Romania, Slovakia and Spain there is a good tradition for using natural stone masonry in buildings and structures. They are long lasting constructions very convenient in severe conditions of climate and foundations. Stone masonry can be used either partially for foundation and first storey walls or for the whole structure, being the high cost of labor, the most limiting condition for their use during the last decades. Recently, rapid rising of energy costs and increasing of losses due to natural catastrophes draw back the attention to natural stone masonry. Originally, stone masonry was made with lime mortar that, despite its low tensile resistance, presents a considerable ductility that contributes to reduce the effects of stress concentration. For improving the general characteristics of the stone masonry constructions, including their response to seismic actions, strengthening the masonry structural members with polymer grids were studied with shaking table tests.

The model tested is a one floor squared rubble masonry building with an asymmetric plan. The walls are made of natural limestone with mortar bed joints reinforced with a polymeric grid, see Figure 2.16 a. This type of construction is representative of a building typology common in Eastern European countries (Romania).
The plan view of the model is shown in Figure 2.16 b. The walls are 3.6m high and 0.24m thick. The design of the structure did not include any slab on the top. Openings can be found in three walls: (a) on the principal façade, with one door with 1.0m×2.0m; (b) on the right lateral view, with two windows with 1.0m×1.0m; and (c) on the other lateral view, with one window with 1.0m×1.0m. The model was built on a reinforced concrete slab with 0.2m of thickness for easy transportation in the testing room and fastening to the shaking table.

The accelerometers are set on masonry, see Figure 2.17. High frequency ENDEVCO, model 7290-A with variable capacitance, CROSSBOW LF series with high precision and PCB Piezotronics, model 337A26, were used for acceleration measurements.
The input signals for the shaking table consisted in semi-artificial series (Coelho et al., 1998, 2004), (Bairrão et al., 2004). The components were applied to the model in both horizontal directions. The use of such type of signals allows an easier approach to the complexities of the behavior of structures. The displacement imposed in the shaking table for each one of the steps resulted from a previous adaptation of the proposed signal for the total mass of the physical model. In Figure 2.18 presents examples of the input motion used in some of the steps of the first phase of the experimental program. The damage are observed after the tests, see Figure 2.19 (BAIRRÃO, SILVA et al., 2006).

![Figure 2.18 – Input motions: (a) Test 01; and (b) Test 06 from the first phase.](image)

![Figure 2.19 – Damages observed in the specimen without vertical reinforcement: (a) East side, (b) West side and (c) South side.](image)
2.4 Numerical Methods

For analyzing the dynamic response of the structure in numerical methods, two means are normally used: Step-by-step Analysis (Time Integration Analysis) and Pushover Analysis. The Step-by-step Analysis or so called Time Integration Analysis is a classic approach of the time domain analysis, which can accurately represent the response of the structure at any given time. And this way can be also conveniently used in the nonlinear analysis (Material Nonlinear or Geographic Nonlinear), because the properties can be updated anytime during the calculation. In order to get stable results, the time steps (time intervals) must be small enough, which causes a large processing time. Thus the analysis is accurate but quite time-consuming. A static procedure is often adopted to overcome this. Pushover analysis has been proved to be a reliable way to analysis the response of the structure, especially during large earthquakes, particularly to test the locations of plastic hinges and the ultimate capacity of the structure (Chopra, 1999). Pushover analysis is also recommended in many codes, as an adequate tool for the seismic safety evaluation of structures.

Next, these two methods will be briefly presented.

2.4.1 Time Integration Analysis

The step-by-step procedure is a general approach to dynamic response analysis, and it is well suited to analysis of nonlinear response because it avoids any use of superposition.

2.4.1.1 Description of the Time Integration Analysis

There are many different step-by-step methods, but in all of them the loading and the response history are divided into a sequence of time intervals or “steps”. The response during each step then is calculated from the initial conditions (displacement and velocity) existing at the beginning of the step and from the history of loading during the step. Thus the response for each step is an independent analysis problem, and there is no need to combine response contributions within the step.

Nonlinear behavior may be considered easily by this approach merely by assuming that the structural properties remain constant during each step and causing them to change in accordance with any specified form of behavior from one step to the next; hence the nonlinear analysis actually is a sequence of linear analyses of a changing system. Any desired degree of
refinement in the nonlinear behavior may be achieved in this procedure by making the time steps short enough; also it can be applied to any type of nonlinearity, including changes of mass and damping properties as well as the more common nonlinearities due to changes of stiffness.

Step-by-step methods provide the only completely general approach to analysis of nonlinear response; however, the methods are equally valuable in the analysis of linear response because the same algorithms can be applied regardless of whether the structure is behaving linearly or not. Moreover, the procedures used in solving (SDOF) systems can easily be extended to deal with MDOF systems merely by replacing scalar quantities by matrices. In fact, these methods are so effective and convenient that time-domain analyses almost always are done by some form of step-by-step analysis regardless of whether the response behavior is linear; the Duhamel integral method seldom is used in practice. (Clough, 1995)

2.4.1.2 Numerical Approximation Procedures of the Method

The simplest step-by-step method for analysis of SDOF systems is the so-called “piecewise exact method”, which is based on the exact solution of the equation of motion for response of a linear structure to a loading that varies linearly during a discrete time interval. But here attention will be focused on the MDOF system and the Numerical Approximation Procedures relatively. (Chopra, 2001)

The step-by-step methods employ numerical procedures to approximately satisfy the equations of motion during each time step - using either numerical differentiation or numerical integration. Taking the most common approach, Newmark-β Method, as an example, some aspects will be reviewed next.

A general step-by-step formulation was proposed by Newmark, which includes the preceding method as a special case, but also may be applied in several other versions.

Equations of motion for a linear MDOF system excited by force vector \( p(t) \) or earthquake-induced ground motion are given by:

\[
\begin{align*}
\mathbf{m}\ddot{\mathbf{u}} + \mathbf{c}\dot{\mathbf{u}} + \mathbf{k}\mathbf{u} &= \mathbf{p}(t) \quad \text{or} \quad -\mathbf{m}\ddot{u}_g(t) \\
\text{initial conditions} \quad &\mathbf{u}(0) = \mathbf{u}_0 \quad \text{and} \quad \dot{\mathbf{u}}(0) = \dot{\mathbf{u}}_0
\end{align*}
\]  

(2.22)

Here, the time scale is divided into a series of time steps, usually of constant duration.
The implicit integration method provides the dynamic equilibrium conditions as:

\[
\Delta t = t_{i+1} - t_i
\]
\[
p_i \equiv p(t_i) \quad \mathbf{u}_i \equiv \mathbf{u}(t_i) \quad \dot{\mathbf{u}}_i \equiv \dot{\mathbf{u}}(t_i) \quad \ddot{\mathbf{u}}_i \equiv \ddot{\mathbf{u}}(t_i)
\]

\[
\mathbf{m}\ddot{\mathbf{u}}_i + \mathbf{c}\dot{\mathbf{u}}_i + \mathbf{k}\mathbf{u}_i = \mathbf{p}_i \Rightarrow \mathbf{m}\ddot{\mathbf{u}}_{i+1} + \mathbf{c}\dot{\mathbf{u}}_{i+1} + \mathbf{k}\mathbf{u}_{i+1} = \mathbf{p}_{i+1}
\]

unknown vectors \( \mathbf{u}_{i+1} \quad \dot{\mathbf{u}}_{i+1} \quad \ddot{\mathbf{u}}_{i+1} \)

The implicit integration method provides the dynamic equilibrium conditions as:

\[
\text{for direct solution} \quad x_n \rightarrow \mathbf{u}_i \quad f_n \rightarrow \mathbf{p}_i \quad M, C, K \rightarrow m, c, k
\]
\[
\text{for modal analysis} \quad x_n \rightarrow \mathbf{q}_i \quad f_n \rightarrow \mathbf{P}_i \quad M, C, K \rightarrow M, C, K
\]

Next, the factor \( \gamma \) provides a linearly varying weighting between the influence of the initial and the final accelerations on the change of velocity; the factor \( \beta \) similarly provides for weighting the contributions of these initial and final accelerations to the change of displacement.

\[
x_{n+1} = x_n + \Delta t\ddot{x}_n + (0.5 - \beta)\Delta t^2\dddot{x}_n + \beta\dddot{x}_{n+1}\Delta t^2
\]
\[
\dot{x}_{n+1} = \dot{x}_n + (1 - \gamma)\Delta t\dddot{x}_n + \gamma\Delta t^2\dddot{x}_{n+1}
\]

(Newmark’s equations)

\[
\begin{align*}
\mathbf{M}\dddot{x}_{n+1} + \mathbf{C}\dddot{x}_{n+1} + \mathbf{K}\dddot{x}_{n+1} &= \mathbf{f}_{n+1} \\
(\text{"discrete" equations of motion})
\end{align*}
\]

\[
\dddot{x}_{n+1} = \dddot{x}_n + (1 - \gamma)\Delta t\dddot{x}_n - \gamma\Delta t^2\dddot{x}_{n+1} - K\dddot{x}_n - \Delta t\dddot{x}_n + (0.5 - \beta)\Delta t^2\dddot{x}_n
\]

As a final conclusion, the full procedure reads:

\[
\text{effective stiffness matrix} \quad \hat{\mathbf{K}} = [M + \gamma\Delta t\mathbf{C} + \beta\Delta t^2\mathbf{K}]
\]

\[
\text{effective load vector} \quad \hat{f}_{n+1} = f_{n+1} - C\dddot{x}_n - K\dddot{x}_n - (1 - \gamma)\Delta t\dddot{x}_n - (0.5 - \beta)\Delta t^2\dddot{x}_n
\]

\[
\text{system of coupled equations} \quad \hat{\mathbf{K}}\dddot{x}_{n+1} = \hat{f}_{n+1} \rightarrow \dddot{x}_{n+1}
\]

unconditionally stable method
for \( \gamma = 1/2 \) and \( \beta = 1/4 \) (average acceleration method)

recommended time step depends on the shortest period of interest \( \Delta t = \frac{T_p}{10} \)
2.4.1.3 Incremental Formulation for Nonlinear Analysis

In this dissertation, nonlinear time-integration method is applied, thus, it is necessary to introduce the description of the incremental formulation for nonlinear analysis for better understanding.

The step-by-step procedures described above are suitable for the analysis of linear systems in which the resisting forces are expressed in terms of the entire values of velocity and displacement that have been developed in the structure up to that time. However, for nonlinear analyses, it is assumed that the physical properties remain constant only for short increments of time or deformation; accordingly it is convenient to reformulate the response in terms of the incremental equation of motion, see Figure 2.21 (Clough, 1995):

![Figure 2.21 – The nonlinear update of the increment](image)

The update of the nonlinear increment goes as:

\[
\begin{align*}
\Delta f_i &= f_i - f_0 = m \Delta \dot{v} \\
\Delta f_D &= f_{D1} - f_{D0} = c(t) \Delta \dot{v} \\
\Delta f_S &= f_{S1} - f_{S0} = k(t) \Delta v \\
\Delta p &= p_1 - p_0
\end{align*}
\]

To avoid this iteration, it is common practice to use the initial tangent slopes instead:

\[
t_{0} \equiv \left( \frac{d f_{D}}{d \dot{v}} \right)_{0} \\
k_{0} \equiv \left( \frac{d f_{S}}{d v} \right)_{0}
\]

So using that way, the time-integration method can be applied on the complicated nonlinear cases. In this dissertation, the nonlinear time integration methods are used, which may provide a reliable and accurate result, the details of the analysis will be presented in detail in the following chapters.
2.4.2 Pushover Analysis

The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

2.4.2.1 Basic Introduction of Pushover Analysis

The pushover analysis can be applied in the following steps: a) appropriate lateral load patterns are applied to a numerical model of the structure and their amplitude is increased in a stepwise fashion; b) a non-linear static analysis is performed at each step, until the structure becomes unstable (and fails) or a specified limit consideration is attained; c) a pushover curve (or capacity curve) – usually base shear against top displacement – is plotted; d) this is then used together with the design response spectrum to determine the top displacement under the design earthquake – the target displacement; e) the method has no rigorous theoretical basis – may be inaccurate if assumed load distribution is incorrect, i.e., the use of load pattern based on the fundamental mode shape may be inaccurate if higher modes are significant; the use of a fixed load pattern may be unrealistic if yielding is not uniformly distributed, so that the stiffness profile changes as the structure yields

The main differences between the various proposed pushover methods are:

• The choice of load patterns to be applied.
• The method of simplifying the pushover curve for design use.

2.4.2.2 Basic Steps of Pushover Analysis

To better understand the application of the pushover analysis, the recommended pushover analysis in EC8 will be introduced bellow.
a) Pushover analysis – apply the following two load patterns:

Modal pattern – the acceleration distribution is assumed proportional to the fundamental mode shape \( \phi_i \), the inertia force on the mass \( k \) is then:

\[
F_k = F_b \frac{\phi_k m_k}{\sum_{j=1}^{n} \phi_j m_j}
\]

Base shear \( F_b \) is increased steadily from zero until failure.

Linear pattern – the acceleration is assumed to have a linear variation with height, the inertia force on the mass \( k \) is then:

\[
F_k = F_b \frac{z_k m_k}{\sum_{j=1}^{n} z_j m_j}
\]

\( z_k \) is the height of the mass \( m_k \)

Plot the pushover curve (base shear \( F_b \) vs. top displacement \( d \)) with maximum displacement \( d_m \).

b) Convert the pushover curve into an equivalent SDOF system using:

\[
F^* = \frac{F_b}{\Gamma} \quad d^* = \frac{d}{\Gamma} \quad \Gamma = \frac{\sum_{j=1}^{n} \phi_j m_j}{\sum_{j=1}^{n} \phi_j^2 m_j}
\]

\( \Gamma \) is the transformation factor from MDOF to SDOF and vice versa

c) Simplify to elastic-perfectly plastic pushover curve, see Figure 2.22.

Set \( F^*_y \) equal to maximum load, choose \( d^*_y \) to give equal areas under actual and idealized curves (\( F^*_y \) and \( d^*_y \) are yield strength and displacement)

Figure 2.22 –Simplify to elastic-perfectly plastic pushover curve.
d) Determine the period of idealized equivalent SDOF system

\[ T^* = 2\pi \sqrt{\frac{m^* d^*_j}{F_y}} \quad m^* = \sum_{j=1}^{n} \phi_j m_j \]  

(2.38)

e) Calculate target displacement of SDOF system under design earthquake

\[ d^*_{t} = S_e \left( \frac{T^*}{2\pi} \right)^2 \quad T^* \geq T_c \]
\[ d^*_{t} = S_e \left( \frac{T^*}{2\pi} \right)^2 \frac{1}{q} \left[ 1 + \left( q - 1 \right) \frac{T^*}{T_c} \right] \quad T^* < T_c \]

\[ q = \frac{S_e m^*}{F_y} \geq 1 \]

(2.39)  

(2.40)

Check that \( d_t \leq d_m/1.5 \). Check member strengths and storey drifts are acceptable at the value of \( d_t \).

f) Transform the target displacement back to that of the original MDOF system

\[ d_t = \frac{d^*_{t}}{1} \]

(2.42)

2.5 Conclusion

In this chapter a state of the art of basic experimental and numerical methods in earthquake engineering is presented. First, a brief introduction about the classic dynamic theory is carried out. Then a review of the basic experimental and numerical methods is presented. Issues about experimental methods addressed include in-situ vibration tests and laboratory tests. In the vibration tests section, the basic techniques of the dynamic identification are also introduced, followed by two typical cases of the in-situ vibration tests in historical structures; in the laboratory tests section, the pseudo-dynamic tests and the shaking table tests are presented, with the real cases addressed. After the experimental part, aspects related to numerical methods are discussed, including time integration method and pushover method, which are the main means to analyze the seismic behavior of the structures.

In this dissertation, the shaking table test results are used and time-integration methods are adopted for the numerical analysis. In the following chapters, the detailed experimental descriptions and analysis results will be presented.
Dynamic analyses of a masonry building tested in a shaking table
Chapter 3
Description of the Prototype and the Scaled Model in Shaking Table Tests

Abstract
In this chapter, a description of the prototype, the scaled mock-up and the numerical model is presented. First, a brief introduction on “Gaioleiro” prototype is provided, followed by the description of the shaking table tests previously carried out, including the scaled model, the equipments adopted and the testing procedures. Then the FEM model is introduced, detailing the calibration of the numerical model.
3.1 Introduction of the Prototype - Gaioleiro

As a typical case, the Gaioleiro building typology developed between the mid 19th century and beginning of the 20th century, mainly in Lisbon and it still remains much in use nowadays. These buildings characterize a transition period from the anti-seismic practices used in the “pombalino” buildings originated after the earthquake of 1755 (Ramos and Lourenço, 2004), and the modern reinforced concrete frame buildings.

The “Gaioleiro” buildings are, usually, four or five stories high, with masonry walls and timber floors and roof. The external walls are, usually, in rubble masonry with lime mortar (Pinho, 2000), see Figure 3.1. In the urban areas these buildings are usually semi-detached and belong to a block of buildings. Although it is not an objective of this dissertation, it is noted that pounding can be relevant when the adjacent buildings present different heights or the separation distance is not large enough to accommodate the displacements (Gulkanetal, 2002; Viviane, 2007). It is noted the “block” effect is usually beneficial and provides higher strength of the building (Ramos and Lourenço, 2004).

Figure 3.1 – Pictures of the typical Gaioleiro buildings.

3.2 Description of the Shaking Table Tests

The National Laboratory of Civil Engineering, Lisbon (LNEC), carried out a set of shaking table tests with the purpose of evaluating the seismic performance of the “Gaioleiro” buildings (Candeias et al., 2009). A review of the tests will be presented in the followed sections.
3.2.1 Introduction of the Mock-up

In order to study the seismic performance through experimental tests, a prototype of an isolated building representative of the “Gaioleiro” buildings was defined. This is constituted by four stories with an interstory height of 3.60 m and 9.45 m × 12.45 m in plan, two opposite façades with a percentage of openings equal to 28.6% of the façade area, two opposite gable walls (with no openings), timber floors, and a gable roof.

The mock-up was prepared to reproduce the geometrical, physical and dynamical characteristics of the prototypes of buildings typologies. However, mock-ups are usually simplified due to difficulties related to its reproduction in laboratory, namely the geometrical properties of the prototype or individual structures and the size of the facilities. In fact, it is difficult to fulfill the similitude laws using very small scales, such as the preparation of masonry units and reinforcement elements.

Generally, in dynamic problems to be solved by experimental methods, the usual similitude laws are the Cauchy Similitude and the Froude Similitude, which read, respectively,

\[
\text{Cauchy Value} = \frac{\rho \nu^2}{E} \tag{3.1}
\]

and

\[
\text{Froude Value} = \frac{\nu^2}{Lg} \tag{3.2}
\]

in which

- \(\rho\) = specific mass
- \(\nu\) = velocity
- \(E\) = modulus of elasticity
- \(L\) = length
- \(g\) = acceleration of gravity

The Cauchy similitude is adequate for the phenomena in which the restoring forces are derived from the stress-strain constitutive relations. Alternatively, the Froude similitude is adequate for phenomena in which the gravity forces are important as, for instance, the pendular motion, or the flexural resistance of RC columns which depends on applied axial force (Carvalho, 1998).
In the case study, due to size and payload capacity of the shaking table the mock-up had to be geometrical reduced. Thus, a 1:3 reduced scale taking in account Cauchy’s law of similitude was adopted. The Cauchy values are the same in prototype and mock-ups. Table 3.1 lists the factors for the satisfaction of the Cauchy’s similitude law.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Scale factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>L</td>
<td>( L_p/L_m = \lambda = 3 )</td>
</tr>
<tr>
<td>Young’s Modulus</td>
<td>E</td>
<td>( E_p/E_m = \lambda = 1 )</td>
</tr>
<tr>
<td>Specific mass</td>
<td>( \rho )</td>
<td>( \rho_p/\rho_m = \lambda = 1 )</td>
</tr>
<tr>
<td>Area</td>
<td>A</td>
<td>( A_p/A_m = \lambda^2 = 9 )</td>
</tr>
<tr>
<td>Volume</td>
<td>V</td>
<td>( V_p/V_m = \lambda^3 = 27 )</td>
</tr>
<tr>
<td>Mass</td>
<td>m</td>
<td>( m_p/m_m = \lambda^3 = 27 )</td>
</tr>
<tr>
<td>Displacement</td>
<td>d</td>
<td>( d_p/d_m = \lambda = 3 )</td>
</tr>
<tr>
<td>Velocity</td>
<td>v</td>
<td>( v_p/v_m = \lambda = 1 )</td>
</tr>
<tr>
<td>Acceleration</td>
<td>a</td>
<td>( a_p/a_m = \lambda^{-1} = 1/3 )</td>
</tr>
<tr>
<td>Weight</td>
<td>W</td>
<td>( W_p/W_m = \lambda^3 = 27 )</td>
</tr>
<tr>
<td>Force</td>
<td>F</td>
<td>( F_p/F_m = \lambda^2 = 9 )</td>
</tr>
<tr>
<td>Moment</td>
<td>M</td>
<td>( M_p/M_m = \lambda^3 = 27 )</td>
</tr>
<tr>
<td>Stress</td>
<td>( \sigma )</td>
<td>( \sigma_p/\sigma_m = \lambda = 1 )</td>
</tr>
<tr>
<td>Strain</td>
<td>( \varepsilon )</td>
<td>( \varepsilon_p/\varepsilon_m = \lambda = 1 )</td>
</tr>
<tr>
<td>Time</td>
<td>t</td>
<td>( t_p/t_m = \lambda = 3 )</td>
</tr>
<tr>
<td>Frequency</td>
<td>f</td>
<td>( f_p/f_m = \lambda^{-1} = 1/3 )</td>
</tr>
</tbody>
</table>

Table 3.1 – Scale Factors of the Cauchy similitude (Carvalho, 1998). (where p and m designate prototype and experimental model, respectively).

Figure 3.2 – Geometry of the 1:3 Mock-up: a) the north facade with openings; and b) the gable wall; c) the beams and walls.
Figure 3.2 shows the geometric properties of the mock-up that were obtained directly from the application of the scale factor to the prototype, resulting in a model 3.15 m wide and 4.8 m deep, with 0.17 m of wall thickness. The interstory height is equal to 1.2 m. The mock-up has only the top ceiling, due to difficulties in reproducing the gable roof at reduced scale. The external walls have a single leaf of stone masonry (limestone and lime mortar) and were built by specialized workmanship.

In the construction of the timber floors, medium-density fiberboard (MDF) panels connected to a set of timber joists oriented in the direction of the shortest span were used. The panels were cut in rectangles and stapled to the joists, keeping a joint of about 1 mm for separating the panels. The purpose was to simulate flexible floors with very limited diaphragmatic action, see Figure 3.3.

3.2.2 Description of the Testing Procedures

The test was based on previous experience from the National Laboratory for Civil Engineering (LNEC). The methodology includes seismic tests on shaking table with increasing input excitations and characterization tests of the dynamic properties of the mock-ups before the first seismic test and after each of the seismic tests (Degée et al., 2007, Bairrão and Falcão Silva, 2009, Candeias, 2009). The dynamic properties give inherent information of the mock-up and their evolution is related to the damage induced by a given seismic input.
The seismic tests were performed at the LNEC 3D shaking table by imposing accelerograms compatible with the design response spectrum defined by the Eurocode 8 (EN 1998-1, 2004) for Lisbon, with a damping ratio equal to 5% and a type A soil (rock). The accelerograms were imposed with increasing amplitude in two uncorrelated orthogonal directions that should present approximately the same PGA.

Due to costs involved, the mock-up does not have the same initial conditions, i.e. before the application of the seismic input the mock-up presents (cumulative) damage, with exception of the first one. 25%, 50%, 75% and 100% of the Code amplitude (PGA) are made as input in the shaking table tests. For each step, dynamic identifications are made to detect the damages of the structures.

The dynamic properties of the mock-ups were identified through forced vibration tests at the shaking table (Mendes et al., 2010) and its evolution is based on the experimental transfer functions (e.g. Frequency Response Function, FRF) obtained along the tests (Coelho et al., 2000).

The reduction of the natural frequencies is related to the stiffness variation and, consequently, to the evolution of the damage. Equation (3.3) presents a simplified damage indicator $d_{k,i}$ based on the variation of the natural frequencies $f_{k,i}$ ($f_{k,0}$ is the natural frequency of the mode shape “k” before the application of the first seismic test). This damage indicator assumes that the global mass of the mode shape “k” does not change meaningfully in the different tests and presents different values for each mode shape (Mendes, 2009).

$$d_{k,i} = 1 - \left( \frac{f_{k,i}}{f_{k,0}} \right)^2$$  \hspace{1cm} (3.3)

In this procedure, the experimental vulnerability curves of the mock-ups are defined relating the seismic excitation parameters (energy/intensity accumulated, PGA$_\text{eqi}$) and the damage indicator “d”. Furthermore, the seismic performance of the mock-ups is assessed through the results of the seismic tests (maximum displacement, drifts, crack patterns, etc).
3.2.3 Introduction of the Equipment

The equipment adopted in the test involves the measurement of several signals necessary for the quantification of the mock-ups behavior. Besides the shaking table and the instrumentation necessary for its control, accelerometers were used in the masonry walls to characterize the response of the mock-ups.

The 3D LNEC shaking table is composed by a rigid platform, where the mock-ups are fixed, which is moved by four servo-controlled hydraulic actuators (one longitudinal, two transversal and one vertical). This equipment has six degrees of freedom, i.e. three translational and three rotational, which require a very sophisticated control system. In plan the rigid platform has 4.6 m × 5.6 m and the maximum load capacity is equal to 392 kN (Coelho and Carvalho, 2005). In the case study only transversal and longitudinal actuators were used and the vertical component of the earthquake was not considered while input signals were measured by the accelerometers and displacement transducers installed on the shaking table.

In each façade twenty piezoelectric accelerometers (five per floor) with different sensibilities (10 V/g, 1 V/g and 0.1 V/g) were used to obtain a detailed acceleration field of the walls, see Figure 3.4. In total, the simultaneous recording of 84 signals (4 input and 80 output signals) involved the use of two acquisition systems connected through a trigger.

![Figure 3.4](image_url)

Figure 3.4 – Locations of the accelerometers: (a) North facade; (b) West gable wall.
3.3 Description of the Numerical Model

The finite element model used to perform the non-linear time-integration analysis was developed with the software DIANA (TNO, 2008). The numerical model is made up of 5816 elements with 15176 nodes, resulting in 75412 degrees of freedom (DOF). The geometry of the model respects the 1:3 mock-up tested on the shaking table.

Walls and floors are simulated with 8-nodes shell elements (4736 CQ40S elements), while for the timber joists three dimensional 3-nodes beam elements are used (1080 CL18B elements), see Figure 3.5.

![Elements used for the FE model](image)

Figure 3.5 – Elements used for the FE model:
(a) shell elements CQ40S and (b) beam elements CL18B.

For the connections between floors and facades, three translation degrees of freedom are tied, see Figure 3.6, as this was found to the better reproduce the linear behavior of the structure (Mendes, 2007). As for the boundary conditions, the nodes on the base of the model are fully fixed.

![Meshed numerical model](image)

Figure 3.6 – The meshed numerical model:
(a) a globe view of the elements and (b) floor elements
For the material properties, laboratory tests have been made, the axial and diagonal compression tests (Mendes, 2009), see Figure 3.7.

![Test setups for specimens in (a) axial and (b) diagonal compression tests.](image)

Table 3.2 presents the results obtained in the axial compression tests. The compressive strength is, on average, equal to 6 MPa and was determined assuming a uniform stress in the cross-section of the wallets. The Young’s modulus and Poisson ratio were calculated from the variation of the strains (average of the vertical and horizontal data) between 0.05 and 0.2 of the compressive strength. The average of the Young’s modulus is equal to 3.37 GPa. The last three specimens presented unexpected values of Poisson ratio. Thus, due to lack of data, this parameter was not statically analysed.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specific mass (kg)</th>
<th>Compressive strength (MPa)</th>
<th>Young’s modulus (GPa)</th>
<th>Poisson Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>2182</td>
<td>5.81</td>
<td>4.07</td>
<td>0.23</td>
</tr>
<tr>
<td>W2</td>
<td>2135</td>
<td>5.55</td>
<td>3.32</td>
<td>0.20</td>
</tr>
<tr>
<td>W3</td>
<td>2171</td>
<td>6.17</td>
<td>3.97</td>
<td>0.09</td>
</tr>
<tr>
<td>W4</td>
<td>2141</td>
<td>5.91</td>
<td>3.00</td>
<td>0.44</td>
</tr>
<tr>
<td>W5</td>
<td>2182</td>
<td>6.56</td>
<td>2.51</td>
<td>0.05</td>
</tr>
<tr>
<td>Average</td>
<td>2162</td>
<td>6.00</td>
<td>3.37</td>
<td>-</td>
</tr>
<tr>
<td>CV (%)</td>
<td>1.1</td>
<td>6.4</td>
<td>19.5</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 3.2 – Results of the compression tests (Mendes, 2009).

The Young’s modulus presents a significant coefficient of variation (19.5%) and the value of the last specimen W5 (2.51 GPa) appears to deviate markedly from other specimens of the sample. Thus, the Grubbs and Dixon criteria for testing outliers (ASTM E178-02, 2002) were used. Both tests indicated that the Young’s modulus of the specimen W5 should not be considered as an outlier (Mendes, 2009).
In the standard interpretation of the diagonal compression test, the diagonal tensile strength is obtained by assuming that the specimen collapses when the principal stress, $\sigma_1$, at its centre achieves its maximum value. According to Frocht theory (Calderini et al., 2009), the principal stresses at the centre of the specimen are equal to: $\sigma_1$ (tensile strength) = 0.5 $P/A$ and $\sigma_{II} = -1.62 P/A$, in which the $P$ is the load and $A$ is the transversal area of the specimen.

Table 3.3 presents the principal stresses obtained through the diagonal compression tests. The average of the tensile strength is equal to 100 KPa, leading to the conclusion that, as expected, this value is significantly lower than the compressive strength (6.00 MPa). It is noted that according to Grubbs and Dixon criteria the principal stresses of the specimen W6 are outliers and were not considered in the average of the results (Mendes, 2009).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specific mass (kg)</th>
<th>Tensile strength ($\sigma_1$) (KPa)</th>
<th>Principal stress $\sigma_{II}$ (KPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W6</td>
<td>2118</td>
<td>130†</td>
<td>-422†</td>
</tr>
<tr>
<td>W7</td>
<td>2129</td>
<td>104</td>
<td>-338</td>
</tr>
<tr>
<td>W8</td>
<td>2153</td>
<td>96</td>
<td>-310</td>
</tr>
<tr>
<td>W9</td>
<td>2159</td>
<td>103</td>
<td>-332</td>
</tr>
<tr>
<td>W10</td>
<td>2141</td>
<td>98</td>
<td>-318</td>
</tr>
<tr>
<td>Average*</td>
<td>2140</td>
<td>100</td>
<td>-325</td>
</tr>
<tr>
<td>CV* (%)</td>
<td>0.8</td>
<td>3.9</td>
<td>3.9</td>
</tr>
</tbody>
</table>

Table 3.3 – Results of the diagonal compression tests (Mendes, 2009). † outlier according to Grubbs and Dixon criteria * discarding outliers

The physical nonlinear behaviour of the masonry walls was simulated using the Total Strain Crack Model detailed in Diana (TNO, 2008). Figure 3.8 shows the hysteretic behavior of masonry, which includes a parabolic stress-strain relation for compression, where the compressive strength, $f_c$, is equal to 6 N/mm$^2$ and the respectively fracture energy, $G_c$, is equal to 9.6 N/mm. In tension, an exponential tension-softening diagram was adopted, where the tensile strength, $f_t$, is equal to 0.1 N/mm$^2$ and the fracture energy, $G_t$, is equal to 0.12 N/mm. The crack bandwidth, $h$, was determined as a function of the finite element area, $A$, see Equation (3.4). In terms of shear behaviour, a constant retention factor equal to 0.01 was adopted.

$$h = \sqrt{A}$$  \hspace{1cm} (3.4)

The damping, $C$, is simulated according to Rayleigh viscous damping (Chopra, 2001), which is a linear combination of the mass, $M$, and the stiffness, $K$, matrices, see Equation (3.5).
\[ C = \alpha \cdot M + \beta \cdot K \] (3.5)

Figure 3.8 – Adopted hysteretic behavior of masonry (Mendes, 2009).

Constants \( \alpha \) (1.68) and \( \beta \) (0.000319) were determined from the results obtained in the dynamic identification tests, see Table 3.4. In the damping identification, a curve fitting of the Frequency Response Function (FRF) for a Single Degree of Freedom was used.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency</th>
<th>Damping Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Transversal</td>
<td>4.93</td>
<td>3.2</td>
</tr>
<tr>
<td>3rd Transversal</td>
<td>28.53</td>
<td>3.33</td>
</tr>
</tbody>
</table>

Table 3.4 – Experimental damping ratio.

3.4 Calibration of the Numerical Model (Scappaticci, 2010)

The calibration of the numerical model is based on the comparison between experimental and numerical results conducted with reference to the first six modes. From the material tests, the initial material properties are assumed as in Table 3.5.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Masonry</th>
<th>Wood Beams</th>
<th>Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus (GPa)</td>
<td>3.4</td>
<td>12.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Poison</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Density (t/m³)</td>
<td>2.15</td>
<td>0.58</td>
<td>0.76</td>
</tr>
</tbody>
</table>

Table 3.5 – Initial material properties.
The results obtained from the model with initial material properties are listed in Table 3.6. In this first stage a high difference in both frequencies and mode shapes is noticed, as shown by Figure 3.9, Figure 3.10 and Figure 3.11.

<table>
<thead>
<tr>
<th>Modes</th>
<th>Experimental [Hz]</th>
<th>FEM [Hz]</th>
<th>Error [%]</th>
<th>MAC</th>
<th>NMD</th>
<th>$f_{exp}/f_{FEM}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Transversal</td>
<td>4.927</td>
<td>10.130</td>
<td>106</td>
<td>0.99</td>
<td>0.12</td>
</tr>
<tr>
<td>2</td>
<td>Distorsion</td>
<td>8.446</td>
<td>21.232</td>
<td>151</td>
<td>0.76</td>
<td>0.57</td>
</tr>
<tr>
<td>3</td>
<td>Longitudinal</td>
<td>12.076</td>
<td>22.916</td>
<td>90</td>
<td>0.86</td>
<td>0.40</td>
</tr>
<tr>
<td>4</td>
<td>Longitudinal</td>
<td>14.306</td>
<td>47.949</td>
<td>235</td>
<td>0.87</td>
<td>0.39</td>
</tr>
<tr>
<td>5</td>
<td>Transversal</td>
<td>16.270</td>
<td>25.943</td>
<td>59</td>
<td>0.95</td>
<td>0.27</td>
</tr>
<tr>
<td>6</td>
<td>Longitudinal</td>
<td>18.649</td>
<td>43.095</td>
<td>131</td>
<td>0.56</td>
<td>0.88</td>
</tr>
</tbody>
</table>

Table 3.6 – Comparison between the initial model and the experimental model.

Figure 3.9 – Frequency Comparison.                                Figure 3.10 – MAC comparison.

Figure 3.11 – Mode shape comparison.
Figure 3.12 shows how far is the representative point of the Model A from the target point, in the plane Average MAC – Average frequency ratio.

Figure 3.12 – Representative point of the Initial Model by means of the average frequency ratio and average MAC.

As the performance analysis is mainly focused on the walls, the masses of the floors are transferred directly to the masonry walls as concentrated masses. Another advantage of this is avoiding many invalid modes created by floors, making the comparison more convenient. Two new models, namely Model C and Model D, are made according to the concentrated masses. In Model C the mass density was set equal to 0 and the mass was replaced by point mass elements (PT3T) placed in correspondence to the end nodes of the floor beams. The Model D is an evolution of Model C, resulting in a model in which the perimetric wood beams into the walls are modelled with CL18B beam elements and their mass is uniformly distributed along the perimeter of the floors through PT3T translation point elements, see Figure 3.13.

Figure 3.13 – PT3T elements used for point masses: (a) topology and (b) displacements.
Table 3.7 – Frequencies of the two new models.

Table 3.7 lists the frequencies of Model C and Model D, and their difference is rather small. However the difference between the experimental and numerical model remains high, especially in frequencies. Assuming that the geometric definition of the model is correct, the source of difference can be attributed to an inappropriate estimation of the elastic modulus of masonry.

From the orthogonal test shown in Table 3.8, the sensitivity of the results to the Young’s modulus of the floor elements is obtained. It is shown that the Young’s modulus of the wood beams has little influence on the variability of frequencies, while decreasing the Young’s modulus of panels can produce a significant increase in the ratio $f_{exp}/f_{FEM}$, see Table 3.9. Subsequently, sensitivities of other components will also be obtained.

<table>
<thead>
<tr>
<th>MODEL</th>
<th>FACADES</th>
<th>GABLE WALLS</th>
<th>WOOD BEAMS</th>
<th>PANELS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.4</td>
<td>3.4</td>
<td>10.0</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>3.4</td>
<td>3.4</td>
<td>12.0</td>
<td>2.0</td>
</tr>
<tr>
<td>3</td>
<td>3.4</td>
<td>3.4</td>
<td>14.0</td>
<td>2.0</td>
</tr>
<tr>
<td>4</td>
<td>3.4</td>
<td>3.4</td>
<td>10.0</td>
<td>3.0</td>
</tr>
<tr>
<td>5</td>
<td>3.4</td>
<td>3.4</td>
<td>12.0</td>
<td>3.0</td>
</tr>
<tr>
<td>6</td>
<td>3.4</td>
<td>3.4</td>
<td>14.0</td>
<td>3.0</td>
</tr>
<tr>
<td>7</td>
<td>3.4</td>
<td>3.4</td>
<td>10.0</td>
<td>4.0</td>
</tr>
<tr>
<td>8</td>
<td>3.4</td>
<td>3.4</td>
<td>12.0</td>
<td>4.0</td>
</tr>
<tr>
<td>9</td>
<td>3.4</td>
<td>3.4</td>
<td>14.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

Table 3.8 – Material Properties of the models analysed in the first Step of the Sensitivity Analysis.

<table>
<thead>
<tr>
<th>MODE</th>
<th>MODEL 1</th>
<th>MODEL 2</th>
<th>MODEL 3</th>
<th>MODEL 4</th>
<th>MODEL 5</th>
<th>MODEL 6</th>
<th>MODEL 7</th>
<th>MODEL 8</th>
<th>MODEL 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.01</td>
<td>10.03</td>
<td>10.06</td>
<td>10.16</td>
<td>10.18</td>
<td>10.19</td>
<td>10.23</td>
<td>10.24</td>
<td>10.25</td>
</tr>
<tr>
<td>3</td>
<td>25.35</td>
<td>25.40</td>
<td>25.45</td>
<td>25.50</td>
<td>25.52</td>
<td>25.53</td>
<td>25.54</td>
<td>25.54</td>
<td>25.54</td>
</tr>
<tr>
<td>4</td>
<td>25.40</td>
<td>25.45</td>
<td>25.50</td>
<td>25.55</td>
<td>25.56</td>
<td>25.57</td>
<td>25.58</td>
<td>25.58</td>
<td>25.58</td>
</tr>
<tr>
<td>5</td>
<td>44.19</td>
<td>44.24</td>
<td>44.29</td>
<td>44.34</td>
<td>44.36</td>
<td>44.37</td>
<td>44.38</td>
<td>44.38</td>
<td>44.38</td>
</tr>
<tr>
<td>6</td>
<td>44.24</td>
<td>44.29</td>
<td>44.34</td>
<td>44.39</td>
<td>44.41</td>
<td>44.42</td>
<td>44.43</td>
<td>44.43</td>
<td>44.43</td>
</tr>
</tbody>
</table>

Table 3.9 – Frequency Comparison in the first Step of the Sensitivity Analysis.
The final model was adopted after several trials and the Table 3.10 and Table 3.11 show the parameters and the comparison between experimental and numerical frequencies for the final model. Figure 3.14 shows the path followed to the target, where it is shown that a better average MAC value can still be possibly found.

<table>
<thead>
<tr>
<th>FACADES</th>
<th>GABLE WALLS</th>
<th>WOOD BEAMS</th>
<th>PANELS</th>
</tr>
</thead>
<tbody>
<tr>
<td>YOUNG (GPa)</td>
<td>0.9</td>
<td>1.5</td>
<td>12.0</td>
</tr>
<tr>
<td>POISON</td>
<td>0.2</td>
<td>0.2</td>
<td>0.3</td>
</tr>
<tr>
<td>DENSIT (t/m³)</td>
<td>2.15</td>
<td>2.15</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Table 3.10 – The parameters of the final model.

<table>
<thead>
<tr>
<th>Modes</th>
<th>Experimental [Hz]</th>
<th>Final Model [Hz]</th>
<th>Error [%]</th>
<th>MAC</th>
<th>NMD</th>
<th>fexp/fFEM</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Transversal</td>
<td>4.927</td>
<td>5.302</td>
<td>8</td>
<td>0.98</td>
<td>0.13</td>
<td>0.929</td>
</tr>
<tr>
<td>2 Distorsion</td>
<td>8.446</td>
<td>9.511</td>
<td>13</td>
<td>0.81</td>
<td>0.48</td>
<td>0.888</td>
</tr>
<tr>
<td>3 Longitudinal</td>
<td>12.076</td>
<td>12.607</td>
<td>4</td>
<td>0.92</td>
<td>0.29</td>
<td>0.958</td>
</tr>
<tr>
<td>4 Longitudinal</td>
<td>14.306</td>
<td>12.840</td>
<td>-10</td>
<td>0.88</td>
<td>0.37</td>
<td>1.114</td>
</tr>
<tr>
<td>5 Transversal</td>
<td>16.270</td>
<td>15.970</td>
<td>-2</td>
<td>0.91</td>
<td>0.32</td>
<td>1.019</td>
</tr>
<tr>
<td>6 Longitudinal</td>
<td>18.649</td>
<td>17.152</td>
<td>-8</td>
<td>0.60</td>
<td>0.82</td>
<td>1.087</td>
</tr>
</tbody>
</table>

Table 3.11 – The comparison of the modes (final model).

Figure 3.14 – Approaches by means of average frequency ratio and average MAC.

With the aim of performing a model updating, several models alternative to the initial one were developed, in order to assess which analytical model showed behavior closer to that expressed by the experimental model. For all these models, it was decided to fix the mechanical properties of wood, based on values provided by literature, defining the Young’s modulus of the other materials employed as variables to calibrate. After the comparison, the model below was adopted due to the closer result to the target value (Scappaticci, 2010).
The model intends to simulate the interaction between the walls through the introduction of corners with a Young's modulus different from both façades and gable walls. The additional variables are represented by the Young’s modulus of panels, façades and gable walls. The initial values of variables were changed, as well as global modes considered, until the most satisfactory solution was found. Figure 3.16 and Figure 3.17 provide the final model of the updating, where it is shown that very good agreement is found in terms of frequencies found and better average MAC value.
Effect of modification

Figure 3.17 – Final model of the automatic updating by means of average frequency ratio and average MAC.

Table 3.12 shows, from an analytical point of view, the comparison between the Experimental Model and the Final Model of the updating, both in terms of frequency and mode shapes (through $MAC$ and $NMD$). Here note that $NMD$ is Normal Maximum Deflection which indicates the proportion of the model that shows a deflection exceeding the maximum deflection tolerance. The global mode shapes of the Experimental Model and the Numerical Model show a good correlation ($MAC_{\text{average}} = 0.89$ and $NMD_{\text{average}} = 0.33$) and a frequency error satisfactorily low ($FreqError_{\text{average}} = 2.6\%$), see Figure 3.18. Finally, Table 3.13 shows the calibrated variables of the Final Model.

<table>
<thead>
<tr>
<th>MODES</th>
<th>EXPERIMENTAL [Hz]</th>
<th>FINAL MODEL UP. [Hz]</th>
<th>ERROR [%]</th>
<th>MAC</th>
<th>NMD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transversal</td>
<td>4.927</td>
<td>4.785</td>
<td>-2.89</td>
<td>0.987</td>
<td>0.116</td>
</tr>
<tr>
<td>Distorsion</td>
<td>8.446</td>
<td>8.200</td>
<td>-2.91</td>
<td>0.812</td>
<td>0.480</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>12.076</td>
<td>12.314</td>
<td>1.97</td>
<td>0.892</td>
<td>0.348</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>14.306</td>
<td>13.913</td>
<td>-2.75</td>
<td>0.878</td>
<td>0.373</td>
</tr>
</tbody>
</table>

Table 3.12 – Comparison Experimental Model, Final Model Updating.

Figure 3.18 – Final Model Updating: (a) MAC matrix and (b) NMD values.
3.5 Conclusion

In this chapter, a description of the prototype, the scaled mock-up and the numerical model is presented, including a brief introduction on Gaioleiro prototype and the description of the shaking table tests previously carried out. The calibration of the numerical model is also addressed in detail, and the final calibrated model will be used next for the nonlinear time-integration analysis.
Chapter 4

Dynamic Analysis of the Scaled Model

Abstract

This chapter presents an analysis of the experimental and numerical results of the scaled model. First, for the purpose of optimization, five different numerical models are used for time-integration analysis, and then compared with the experimental data for 25% of the Peak Ground Acceleration (PGA). Then, the selected model is used for the 100% of the PGA. Aspects considered in the comparison include: a) the accelerations and displacements of 80 typical points in four façades; b) piers on North/South facades; c) the maximum strain on each façade and the crack patterns; d) the damage factor after the 25% and 100% of the PGA.
4.1 Description of the Nonlinear Time-Integration Analysis

The nonlinear time-integration analysis for masonry structures is more complicated than steel-concrete or steel structures because of the low tensile strength of the material. In the case of steel-concrete or steel structures the plastic hinges obviously occur in the overstressed parts, whereas, in masonry, distributed cracks appear. The use of softening tends to lead to convergence problems even for static problems, due to the fact that an ill-conditioned problem appears, and matrices are usually rebuilt in every step, making the analyses rather time-consuming. The quasi-brittle behaviour of the material in tension also introduces numerical noise, due to the fast transition from linear elastic behaviour to a fully cracked state involving almost zero stiffness. The quasi-instantaneous changes in the displacement field tend to originate the propagation of high frequency spurious vibrations (Cervera et al., 1995). Therefore, it is important to adopt the Hilbert-Hughes-Taylor time integration method (DIANA, 2005) (also called the $\alpha$-method), which was used here with $\alpha$ equal to $-0.1$. With this method it is possible to introduce numerical dissipation without degrading the accuracy (Mendes and Lourenço, 2010). The Hilbert-Hughes-Taylor method uses the same finite difference equations as the Newmark method with:

\[
\gamma = \frac{1}{2}(1 - 2\alpha) \tag{4.1}
\]

and

\[
\beta = \frac{1}{4}(1 - \alpha)^2. \tag{4.2}
\]

For $\alpha = 0$ the method reduces to the Newmark method (see Chapter 2). For $\alpha$ between $-1/3$ and $1/2$ the scheme is second-order accurate and unconditionally stable. Decreasing $\alpha$ means increasing the numerical damping, and the adopted damping is low for low-frequency modes and high for the high-frequencies modes.

The seismic tests were performed at the LNEC 3D shaking table by imposing accelerograms compatible with the design response spectrum defined by the Eurocode 8 (EN 1998-1, 2004) for Lisbon, with a damping ratio equal to 5% and a type A soil (rock). Figure 4.1 shows the response curve of the input and the Eurocode 8. Using the 1:3 reduced scale, the accelerograms have a total duration of 12s, and a PGA equal to $0.475g = 4.66 \text{ m/s}^2$. The
accelerograms were imposed with increasing amplitude in two uncorrelated orthogonal directions that should present approximately the same PGA. And the earthquake input of the numerical model is adopted based on the data collected by the accelerometers on the base. In the experimental tests, 25%, 50%, 75% and 100% of the PGA were used. But for the numerical input, only 25% and 100% was used, due to time limitations.

The time step $\Delta t$ was determined using the following equation below, in order to account for the lowest period with relevance in the structural behaviour $T_i$, with an error lower than 5%. It was assumed that $T_i$ was equal to 0.04s, meaning than $\Delta t$ is equal to .002s.

$$\Delta t = \frac{1}{20} T_i$$

(4.3)

About the iteration method, the regular Newton-Raphson Method, in which the tangential stiffness is set up before each iteration, was used. In the equilibrium iteration process, a convergence criterion based on the internal energy and a tolerance equal to $10^{-3}$ was used.

### 4.2 Model Optimization

In the previous chapter, the final model after calibration and updating was introduced. However, all the calibrations are based on the linear mode identifications, so the nonlinear performance of the numerical model is unknown. In the following sections, issues on the model optimization will be addressed.
4.2.1 Discussion about the Beams

In the masonry walls of the calibrated model, horizontal beams are created with full connections to the walls and the wood beams of each floor, see Figure 4.2.

Noticing that many vertical cracks occurred in the lintels in the experimental test of 100% PGA (see Figure 4.3), the effect of the beams inside the walls needs to be discussed. Table 4.1 lists the stiffness of the lintels and the beams (1 unit length), which indicates that the stiffness of the beams is rather considerable.

<table>
<thead>
<tr>
<th></th>
<th>E (kN/m²)</th>
<th>A (m²)</th>
<th>K (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lintel</td>
<td>$12 \times 10^6$</td>
<td>$0.075 \times 0.03 = 0.00225$</td>
<td>27000</td>
</tr>
<tr>
<td>Beam</td>
<td>$5.763 \times 10^5$</td>
<td>$0.15 \times 0.3 = 0.045$</td>
<td>259334</td>
</tr>
</tbody>
</table>

Table 4.1 – Stiffness of lintels and beams.
Three models are used for nonlinear time-integration tests to detect the effect of the beams in the response. The calibration is based on the comparison of natural frequencies. In order to keep the frequencies of the calibrated model constant, the first updated model is made by setting the tensile strength of the beams equal to zero, meaning that the frequencies will be the same of the calibrated model because the mass and the stiffness matrices are not changed. The second updated model is made by deleting all these beams. The change of the natural frequencies is acceptable (below 3% in the first 5 modes), see Table 4.2. The simplified models plan views are shown in Figure 4.4.

<table>
<thead>
<tr>
<th>Model with elastic beams</th>
<th>Model with zero tensile strength beams</th>
<th>Model without beams</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>Frequency (Hz)</td>
<td>Mode</td>
</tr>
<tr>
<td>1</td>
<td>4.7845</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>8.2006</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>12.3140</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>13.9130</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>13.9130</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 4.2 – Comparison of frequencies in each model.

Figure 4.4 – Three models plane view:
  a) original calibrated model  b) model without beams  c) model with beams of zero tensile strength.

These three models are used for the 25% PGA analysis, and the frequencies before and after running are collected. The damage factor is used as the damage indicator, defined by the Equation (3.3) in the previous chapter.

Figure 4.5 – Two modes with highest damage factors.
The comparison of the different models is shown in Figure 4.6. It is shown the damage factors of the zero-tensile-strength beam model and the without-beam model are much higher than the original calibrated one, especially in the first mode. The error of the damage factor of the updated models is 14.2%, which is much lower than the calibrated model, 34.3% (first mode). Also in the 4th mode, the peak value of the updated models is higher than the original calibrated one, i.e., closer to the experimental data, see Figure 4.5. The two updated models exhibit similar performances because the zero-tensile-strength beams are not helpful when the earthquake starts.

Besides damage factors, the accelerations and displacements of 80 typical points are also compared. For the 25% PGA, the values of acceleration and displacement are not so high, so obtaining comparable results is rather difficult. Due to the bigger displacements occurring in transversal direction (the stiffness in this direction is smaller), the acceleration and displacement of the points in the east gable wall are used for comparison.

The comparison of the maximum displacement is presented in the Figure 4.7 in next page. The graphs show that the maximum displacements of the updated models are closer to the experimental results. And another interesting result is that the displacement of the numerical model is higher than the experimental one, especially in the middle line of the wall, which can be explained by a weak connection between the east facade and the floors/beams in the numerical model. In conclusion, the updated model performed much better in both natural frequency comparison and the displacement comparison.
Chapter 4 – Dynamic Analysis of the Scaled Model

Figure 4.7 – Comparison of Maximum Displacement

* $\epsilon$ is the average error
4.2.2 Discussion about the Stiffness and Damping Ratio

As listed in Table 4.3, the natural frequency of Without-Beam model is not accurate, even if the error is below 3%. Noticing the fact that increasing the stiffness of the masonry walls can increase the natural frequency, the model 1 with walls of higher stiffness was built for comparison with the Without-Beam model, see Table 4.3.

<table>
<thead>
<tr>
<th>Model</th>
<th>E(MPa) Facade</th>
<th>E(MPa) Gable wall</th>
<th>Frequency of 1st mode (Hz) Numerical</th>
<th>Frequency of 1st mode (Hz) Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without-Beam Model</td>
<td>5.76E+05</td>
<td>3.17E+06</td>
<td>4.6599</td>
<td>4.3</td>
</tr>
<tr>
<td>New Model 1</td>
<td>6.42E+05</td>
<td>3.82E+06</td>
<td>4.9292</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.3 – The new model 1 with higher stiffness wall.

Besides, to detect the effect of the damping ratio in the model, a new model 2 with damping ratio equal to 5% (instead of 3%) is introduced, Table 4.4 lists the new damping ratio and new $\alpha$ and $\beta$ values in Rayleigh Damping.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency</th>
<th>Damping Ratio (%)</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Transversal</td>
<td>4.93</td>
<td>5</td>
<td>2.641208</td>
<td>0.000476</td>
</tr>
<tr>
<td>3rd Transversal</td>
<td>28.53</td>
<td>5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.4 – The new damping ratio.

Different parameters were compared, including the acceleration, velocity, displacement and also the damage factor. Among them, the maximum displacement shows the biggest differences. The comparison of acceleration is presented in the Figure 4.8 on next page.
Figure 4.8 – Comparison of Maximum Acceleration

* ε is the average error

Higher stiffness

3% damping

5% damping

<table>
<thead>
<tr>
<th>Material</th>
<th>Experiment</th>
<th>Numerical</th>
<th>ε</th>
</tr>
</thead>
<tbody>
<tr>
<td>VA1</td>
<td>Exp</td>
<td>Num</td>
<td>44.69%</td>
</tr>
<tr>
<td>VA2</td>
<td>Exp</td>
<td>Num</td>
<td>54.53%</td>
</tr>
<tr>
<td>VA3</td>
<td>Exp</td>
<td>Num</td>
<td>52.64%</td>
</tr>
<tr>
<td>VA4</td>
<td>Exp</td>
<td>Num</td>
<td>67.66%</td>
</tr>
<tr>
<td>VA5</td>
<td>Exp</td>
<td>Num</td>
<td>89.61%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>Experiment</th>
<th>Numerical</th>
<th>ε</th>
</tr>
</thead>
<tbody>
<tr>
<td>VA1</td>
<td>Exp</td>
<td>Num</td>
<td>41.98%</td>
</tr>
<tr>
<td>VA2</td>
<td>Exp</td>
<td>Num</td>
<td>54.53%</td>
</tr>
<tr>
<td>VA3</td>
<td>Exp</td>
<td>Num</td>
<td>52.64%</td>
</tr>
<tr>
<td>VA4</td>
<td>Exp</td>
<td>Num</td>
<td>67.66%</td>
</tr>
<tr>
<td>VA5</td>
<td>Exp</td>
<td>Num</td>
<td>89.61%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>Experiment</th>
<th>Numerical</th>
<th>ε</th>
</tr>
</thead>
<tbody>
<tr>
<td>VA1</td>
<td>Exp</td>
<td>Num</td>
<td>32.35%</td>
</tr>
<tr>
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</tr>
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<td>Exp</td>
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</tr>
<tr>
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<td>Exp</td>
<td>Num</td>
<td>67.66%</td>
</tr>
<tr>
<td>VA5</td>
<td>Exp</td>
<td>Num</td>
<td>89.61%</td>
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</tbody>
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<tr>
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<td>Exp</td>
<td>Num</td>
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</tr>
<tr>
<td>VA4</td>
<td>Exp</td>
<td>Num</td>
<td>67.66%</td>
</tr>
<tr>
<td>VA5</td>
<td>Exp</td>
<td>Num</td>
<td>89.61%</td>
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<table>
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<th>Experiment</th>
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<td>Num</td>
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<td>Exp</td>
<td>Num</td>
<td>31.59%</td>
</tr>
<tr>
<td>VA5</td>
<td>Exp</td>
<td>Num</td>
<td>31.25%</td>
</tr>
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</table>
The acceleration increases with the increase of the stiffness, and the decrease of the damping, which leads to the conclusion that the Without-Beam model with 5% damping ratio performs the best, among the three updated model. After the discussions, the final model was fixed, the parameters are listed in Table 4.5 (the specific mass of panel and wood joints are zero due to the use of the concentrated mass). The results of this model are detailed next and the model is used for the further time-integration analysis of 100% PGA.

![Table 4.5 – The material properties of the final model.](image)

4.3 Results of the analysis

The results comparison includes a) the accelerations and displacements of 80 typical points in four facades; b) the parameters of all the piers on North/South facades; c) the maximum strain on each facade and crack patterns.

4.3.1 Results of 25% of the PGA

The structure mainly shows the linear performance up to 25% of the PGA, so the attention is focused here on the acceleration and displacement comparison with experimental tests results.

4.3.1.1 Acceleration Comparison

Due to the complexity of the nonlinear time-integration analysis, the acceleration of the numerical and experimental does not exactly fit each other, especially in the peak value part. Taking the N11 point as an example (Figure 4.9), the acceleration goes up and down more frequently than the experimental one from 5 s to 8 s. This can be explained as: 1) the input during that period is changing fast, so some numerical noise of the acceleration may be created; 2) the sensitivity of the accelerometers is limited, so some acceleration signals may be ignored during the collection of the data.
Figure 4.10 shows the maximum acceleration in the horizontal line and in vertical line on the North Facade. The accelerations are not quite accurate, with errors larger than 200% on top floor. And from the horizontal direction, peak values are shown in the middle, which do not reproduce the experimental results very well. The reason for this may be the connection between the floors and the facade, or the use of inappropriate stiffness.

The North facade is on the longitudinal direction, so the errors could be high because of the higher stiffness, especially for the 25% input. In the transversal direction, the results are much better, see Figure 4.11 and 4.12. So the main comparable facades are East/West gable wall in this case.
Dynamic analyses of a masonry building tested in a shaking table

4.3.1.2 Displacement Comparison

The displacement of the numerical model fits the experimental results very well on both facade walls, see Figure 4.13. Here Point N4.3 and E4.3 are taken as examples since the values on these two points are the highest. In the graph, besides the displacement graphs, the Root Mean Square (RMS) of the displacement is also shown for comparison. The RMS is defined by Equation (4.4):

\[ D_{\text{RMS}} = \sqrt{\frac{1}{T} \int_0^T [d(t)]^2 dt} \]  

(4.4)
Figure 4.13 – Comparison of Displacement
4.3.1.3 Principal Strain

Figure 4.14 – Principal stain $\varepsilon_1$ on the numerical model.

Figure 4.14 shows the scan during dynamic analysis of maximum principal strain $\varepsilon_1$ on the outside surface of the North facade and the gable wall, which is lower than 0.005 even at peak values, with some strain concentration areas near the connection between lintels and columns; and another area with strain concentration is the middle pier on the top floor. Figure 4.15 shows the crack pattern of the experimental tests after the 25% PGA input and the cracks are few as expected. The width of the crack is calculated by Equation (3.4) and the width is about 0.3 mm, which is small and possibly remained invisible in the tested stone masonry walls.

Figure 4.14 – Crack patterns of the experimental model.
4.3.2 Results of 100% of the PGA

4.3.2.1 Accelerations and Displacement Comparison

The accurate comparison of the accelerations and displacement would be rather difficult for the 100% of the PGA. Figure 4.15 and Figure 4.16 show that the accelerations of the north facade are similar to the one for 25% PGA, in which the numerical values are higher than the experimental values, especially on the top floors. Still, reasonable agreement is again found in terms of general trend.

![Figure 4.15 – Acceleration comparison of selected points of North facade.](image1)

![Figure 4.16 – Acceleration comparison for Vertical line VA1.](image2)
For the East Facade, the accelerations of the numerical model are also higher than the experimental values, but the accelerations show some irregularity, see Figure 4.17. The acceleration curve along the vertical line show a “Z” shape, which may indicate the high frequency modes are excited more due to the damages of the structure.

![Figure 4.17 – Acceleration comparison and the Vertical line VA12 on East gable wall.](image)

The displacement comparison is present on Figure 4.18, which shows that the simulation of the North facade is better than the East gable wall. In principle, the displacement on the top should be larger than the one on the base, but numerical model did not simulate this very well. This may be caused by two reasons: a) the experimental tests show that the first mode should be the main mode excited, but the numerical results indicate higher frequency modes are playing a main role in the response. Taking the 25% PGA results into consideration, the irregularity can be explained by the decrease of frequencies of the higher modes with the damage, so the higher modes are possibly easier to be excited, which makes them to affect more on the displacement and the acceleration; b) the model may be not proper for the 100% PGA, especially the stiffness of the gable wall and the connections between the floors and the gable walls, which needs more trials to get the best results.
Figure 4.18 – Comparison of displacement on East Gable Wall
4.3.2.2 Principal Strain and Crack Patterns

Figure 4.19 a) shows the crack pattern of the experimental tests. Figure 4.19 b) shows the maximum principal strain $\varepsilon_1$ on the outside surface of the North facade and the gable wall, the maximum strain reaches now about 0.046, and there are some strain concentration areas near the connection between lintels and columns, and in the middle column on the top floor. The maximum width of the crack is about 2.7 mm. The Figure 4.19 shows that the simulation of the crack patterns is rather acceptable.

![Comparison of crack patterns (experimental) and maximum strain (numerical).](image)

4.3.2.3 Piers on the Facade

The piers on the facade are important because from Figure 4.19 we see that a large horizontal crack occurred on the pier of the top floor. This indicates that the out-of-plane drift of the piers might be also very large, which may affect the collapse of the whole structure.

By scanning the maximum strain of each step, the maximum strain of the top pier in the middle was found in step 3388, see Figure 4.20. The maximum drift is calculated and it is
almost equal to 3.25%. This top pier may have some out of plane damages during the bigger earthquake input.

![Deformation for the time step providing the maximum strain in the structure.](image)

**Figure 4.19 – Deformation for the time step providing the maximum strain in the structure.**

### 4.4 Conclusion

This chapter presented the comparison between the experimental results and the numerical results for the 25% and the 100% of the PGA. For 25% of the PGA, the numerical model can simulate the acceleration and displacement well, especially in the transversal direction. For the 100% of the PGA, the numerical model cannot simulate the acceleration and displacement well, because the nonlinear performance is complex and there are many characteristics of the model that may affect the seismic performance of the masonry structure. Possibly some better tuning of the material properties (namely with an increase of tensile strength) is needed. The result of the crack patterns fit the experimental results well, which may give us a possibility to do further analysis of the collapse mechanism of the structure in the future.
Dynamic analyses of a masonry building tested in a shaking table
Chapter 5
Dynamic Analysis of the Full Scale Model

Abstract

In this chapter, issues about the scale factor in this shaking table tests of masonry structures will be presented. As addressed in Chapter 3, more gravity should have been taken into account in the 1:3 models tested in the shaking table. Therefore, a calibrated numerical full scale model will be put into nonlinear time-integration analysis to evaluate the effect of the self weight. The full scale model is again computationally tested for 25% and 100% PGA, to compare the new results with the scaled model results. In the end, some conclusions are provided.
5.1 Basic Introduction to Scaling

Testing of complete structures is normally used to enable the understanding of the global behaviour of a construction and to capture the interplay of the response of its different components. This provides quite realistic information on the expected response of the specific structures under testing, but corresponds to rather expensive tests. So the scaled model tests are often used to keep costs acceptable and allow the consideration of more testing variations. But the due to the geometrical scaling, many laws should be respect in order to keep the same performance between reduced models and the prototypes. In this section, a basic discussion of size effect will be presented.

5.1.1 Effect of Size effect on Strength

Discussion about the size effect of masonry structure is expected to be similar to concrete structures, despite the fact the masonry is a composite material with mortar and masonry units. The fracture toughness of concrete as determined by the work-of-fracture method depends on the size of specimens. This is the so-called size effect. It appears in all structures, but it is more pronounced in structures made of cementitious materials due to their large fracture process zone (FPZ), which plays a vital role in the analysis of growth of the crack. The size effect is defined in terms of the nominal stress $\sigma_N$ at maximum (ultimate) load of geometrically similar structures of different sizes. The nominal stress in a structure need not to represent an actual stress and is defined as $\sigma_N = \frac{P_u}{bd}$ or $\sigma_N = \frac{P_u}{d^2}$ for a two or three dimensional structure, respectively, where $d$ is a characteristic dimension of the structure (e.g. the depth or the span of a beam, the length of the FPZ, etc.) and $b$ is the thickness of the two dimensional structure. A dependence of $\sigma_N$ on the size of the structures called the size effect. If $\sigma_N$ does not depend on the size of the structure we say that there is no size effect. The size effect in concrete structures can be easily established if we consider as characteristic length the length of the FPZ. If we consider that this length is approximately five to six times the size of the aggregates, then the length of the FPZ is constant, whereas the size of the structure changes (Gdoutos, 2005).

Thus, for large structures the size of the FPZ is negligible, while for small structures it is appreciable. This explains the rather brittle behaviour of large structures, as opposed to the ductile behaviour of small structures. Classical theories, such as elastic analysis with allowable stress or plastic limit analysis cannot take into consideration the size effect. Contrary, linear
elastic fracture mechanics exhibits a strong size effect dependence described by the dependence of stress intensity factor on the crack length. An approximate formula for predicting the size effect was proposed by Bazant (equation 5.1), as

\[ (\sigma_N)_u = Af_t \left(1 + \frac{W}{B}\right)^{-1/2} \]  

where \( (\sigma_N)_u \) = Nominal stress at failure of a structure of specific shape and loading condition.  
\( W \) = Characteristic length of the structure.  
\( A, B \) = Positive constants that depend on the fracture properties of the material and on the shape of the structure, but not on the size of the structure.  
\( f_t \) = Tensile strength of the material introduced for dimensional purposes.

Equation (5.1) combines limit analysis for small structures and linear elastic fracture mechanics for large structures. A typical size effect curve is shown in Figure 14.6. The horizontal dashed line represents the failure status according to the strength or yield criterion. The inclined dashed line exhibits a strong size effect predicted by linear elastic fracture mechanics. The solid curve between the two limiting curves represents the real situation for most structures. From Figure 5.1 we can observe that for very small structures the curve approaches the horizontal line and, therefore, the failure of these structures can be predicted by a strength theory. On the other hand, for large structures the curve approaches the inclined line and, therefore, the failure of these structures can be predicted by linear elastic fracture mechanics (Gdoutos, 2005).

![Figure 5.1– Size effect law on the strength in a bilogarithmic plot (Gdoutos, 2005).](image-url)
5.1.2 Effect of Size effect on Shaking Table Tests

As introduced in Chapter 4, the maximum dimensions and payload capacity of the table which in most cases call for the use of reduced scale specimens (typically in the range of 1:3 to 1:8 geometric scale for large shaking tables if a multi-storey structural system is to be tested).

The need for reduced scales in shaking table tests of complete structural systems is one of the major drawbacks of this type of testing, due to the similitude problems that it raises (Okada and Bertero et al., 1978). In fact, in order to be able to test an undistorted reinforced concrete model to its ultimate capacity, it is necessary to obtain a true simulation of

- The geometry;
- The stress-strain relationship of the materials;
- The mass and the gravity forces;
- The initial and boundary conditions.

Geometric similitude is normally achieved by a direct application of a geometrical scale and does not present any difficulty, besides that associated with the construction of small specimens (Carvalho, 1998).

The accurate simulation of the stress-strain relationships is much more difficult (Bedell and Abrams, 1982), even when using essentially the same material in the prototype and in the model. Small geometric scales require for example the use of micro-concrete and specially manufactured reinforcing bars or scaled masonry units. Thus, it is extremely difficult to satisfy by scaling models, throughout the complete spectrum of stresses, strains, amplitude of load reversals, strain rates and strain gradients. For concrete, a correct simulation of the compressive and tensile strengths, together with the ultimate strain is of critical importance. The bond characteristics are an important aspect conditioning the correct simulation of the structural behaviour. In this respect a large scatter of results is to be expected, with cases being known of superior bond behaviour of reinforced concrete structures in small scale (Abrams and Tangkijingamvong, 1984).

The Cauchy value and Froude value are listed again, given by
Chapter 5 – Dynamic Analysis of the Full Scale Model

\[ \text{Cauchy Value} = \frac{\rho v^2}{E} \]  \hspace{1cm} (5.2)

and

\[ \text{Froude Value} = \frac{v^2}{Lg} \]  \hspace{1cm} (5.3)

in which
\[
\begin{align*}
\rho &= \text{specific mass} \\
v &= \text{velocity} \\
E &= \text{modulus of elasticity} \\
L &= \text{length} \\
g &= \text{acceleration of gravity}
\end{align*}
\]

The equations are derived from the relationship between the inertia forces and the elastic restoring forces:

\[ (\rho L^3 v^2 / L) / E L^2 = \frac{\rho v^2}{E} \]  \hspace{1cm} (5.4)

and the Froude value is the ratio between inertia forces and gravity forces:

\[ (\rho L^3 v^2 / L) / \rho L^3 g = \frac{v^2}{Lg} \]  \hspace{1cm} (5.5)

It is evident that for the realistic modeling of strongly non-linear dynamic response of structures both similitude laws must be respected. However, assuming the mechanical properties are the same both in the prototype and in the model, i.e., the same material is used, such a requirement asks for the specific mass scale to be inversely proportional to the geometric (assume the acceleration of gravity cannot be changed), which is very hard to apply. The difficulty is usually overcome by adding distributed masses to the model, so that the desired specific mass is obtained in an equivalent way. But the added mass should be linked rigid to the model and cannot affect the structural stiffness (Carvalho, 1998).

5.1.3 Discussion in this case

The size effect on the strength of masonry structures is more complicated than of concrete or steel structures. Masonry structures strength depends on the strength of masonry units and the mortar, but also on the connections between walls and between walls and floors. Theoretically the 1:3 model used in the shaking table test should be stronger than the prototype due to the size effect law introduced in the first section. As a result, the experimental findings
can be unsafe and should be taken into consideration when it comes to the collapse mechanism analysis or to extend the results to practice.

And for the simulation law, the 1:3 scaled model used in this case respects the Cauchy Law, but does not respect the Froude Law. The gravity should be increased according to the Equation (5.3), but the effect of this added gravity to the dynamic performance of the structure is unknown. This can be detected here using the numerical model, in which it is possible to conveniently change any parameter. Normally, these two rules are respected by changing the specific mass (Change 1), but in that way, the input has to be changed, see Table 5.1. Here, in order to keep the input of earthquake and to better compare the results with the original one, the new parameters are adopted by directly increasing the acceleration of gravity by three times (Change 2).

<table>
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<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Scale factor</th>
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<th>Change 2</th>
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<td>tp/im=λ^2=3/2</td>
<td>tp/im=λ=3</td>
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</table>

* The red part provides the parameters that should be changed from the original.

The model with Change 2 was used with 25% of the PGA as loading, and the comparison of the results is given in Figure 5.2, from which we can find that the effect of the gravity is positive for the 25% PGA.
Chapter 5 – Dynamic Analysis of the Full Scale Model

Figure 5.2 – Comparison of displacement between original and added-gravity model under 25% PGA
This can be explained as the movement of the structures are small and the response is mostly elastic, meaning that the gravity will add normal stress to the structure, which will increase the strength for the low earthquake input. For higher input the influence remains unclear, as compressive failure might play a role. Due to time limitations, the 100% PGA loading results could not be completed and cannot be shown here. It is expected that the results are similar with the full scale tests shown below.

The damage factor in Figure 5.3 can also prove that the damage of the model with additional gravity is smaller.

![Figure 5.3 – The comparison of the damage factor between two models.](image)

Additionally, in order to prove the correctness of the Cauchy and Froude law, the model with added-gravity is also used to compare with the full scale model with 25% PGA loading. If the behaviour is elastic, the displacement of the added-gravity model should be 3 times as much as the displacement of the full scale model. Figure 5.4 indicates that the results fit very well and the law is accurate (Examples: N1.2 and N4.2), despite the fact that some non-linearity is present.
Chapter 5 – Dynamic Analysis of the Full Scale Model

5.2 Results of the Full Scale Model

As stated before, the displacements of the full scale model for 25% PGA loading are smaller than 3 times the displacement of scaled model without added masses, which indicates that the gravity can create a positive effect on the performance of the structures. So attention here will mainly be focused on the comparison of the 100% PGA between these two models.

5.2.1 Displacement

The performance of the full scale model for 100% PGA is very much similar with the scale model for 100% of the PGA. Again, the full scale model did not simulate the experimental results very well. From the results, it is noticed that the displacement of the full scale model is significantly smaller than the 3 times the displacement of the scaled model, see Figure 5.5.

Figure 5.4 – Comparison of displacement between added-gravity model and full scale model (N1.2 and N4.2).

The added-gravity model is now running for the 100% PGA, but due to the time limitation, this part can not be put in the dissertation, but the addition data will be put as attach at the end of the dissertation and be presented on the defence.
Dynamic analyses of a masonry building tested in a shaking table

5.2.2 Principal Strain and Piers Analysis

Figure 5.6 shows the maximum principal strains for the full scale model, which are smaller than the scaled model. Table 5.2 shows the comparison of the maximum principal strain and the slope in the middle of the top pier in the middle. It is noticed that all these values are smaller.
Table 5.2 – Comparison of Maximum strain, damage factors and maximum slope in two models.

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<tr>
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<th>Full scale model</th>
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<tr>
<td>Maximum Strain (peak value)</td>
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<td>0.265E-01</td>
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<td>Damage factor of 1st Mode</td>
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<td>0.59</td>
</tr>
<tr>
<td>Damage factor of 2nd Mode</td>
<td>0.53</td>
<td>0.51</td>
</tr>
<tr>
<td>Damage factor of 3rd Mode</td>
<td>0.47</td>
<td>0.45</td>
</tr>
<tr>
<td>Maximum Slope</td>
<td>3.25%</td>
<td>1.17%</td>
</tr>
</tbody>
</table>

Note that, all the conclusions are based on the numerical model analysis only and the same material properties are adopted, so the results only address scaling effect. As mentioned in the last chapter, the numerical simulation does not simulate the experimental results perfectly, especially for the 100% PGA. So the numerical model needs to be improved to get a closer agreement with the experiments. Still, this remark does not affect the conclusion found that not including the additional masses and testing at reduced scale is beneficial for this structure.

5.3 Conclusion

The analyses of the full scale model are presented in this chapter. The following conclusions can be drawn as:

· For the size effect law, the mock-up used in the shaking table tests is stronger than the real building because of the size effect on the strength. So from this point of view, the analysis would be more favourable than the real situation.

· However, the mock-up also needs more gravity to be added according to the Froude Similitude Law, and the additional gravity was found to be helpful when the earthquake happens, when compared to the absence of additional loading. By comparing to a full scale building, which already includes size effect, it was shown that testing a small scale building without additional masses provides a more severe response, meaning that the obtained results are conservative and can be extrapolated to larger sizes.
Dynamic analyses of a masonry building tested in a shaking table
Chapter 6

Conclusions and the Future Works
6.1 Conclusions

This dissertation addresses the basic experimental and numerical methods in earthquake engineering, focusing with detail on the seismic assessment of masonry “Gaioleiro” buildings, and performing numerical modeling using nonlinear time-integration analyses and comparing the results to the data obtained from the previous testing. The comparison includes the displacement and acceleration on 80 typical points on the structure, the damage indicator after each earthquake input and the crack patterns. Furthermore, an updated full scale numerical model is also used for the nonlinear time-integration analyses with the objective of discussing the scaling in dynamic analysis.

The conclusions of the whole dissertation can be summarized as:

- **Scaled model:** According to the comparison between the experimental results and the numerical results for the 25% and the 100% of the PGA, the numerical model can simulate the acceleration and displacement well, especially in the transversal direction. For the 100% of the PGA, the numerical model cannot simulate the acceleration and displacement well, because the nonlinear performance is complex and there are many characteristics of the model that may affect the seismic performance of the masonry structure. Possibly some better tuning of the material properties (namely with an increase of tensile strength) is needed. The result of the crack patterns fit the experimental results well, which may give us a possibility to do further analysis of the collapse mechanism of the structure in the future.

- **Full Scale model:** For the size effect law, the mock-up used in the shaking table tests is stronger than the real building because of the size effect on the strength. So from this point of view, the analysis would be more favourable than the real situation. However, the mock-up also needs more gravity to be added according to the Froude Similitude Law, and the additional gravity was found to be helpful when the earthquake happens, when compared to the absence of additional loading. By comparing to a full scale building, which already includes size effect, it was shown that testing a small scale building without additional masses provides a more severe response, meaning that the obtained results are conservative and can be extrapolated to larger sizes.
6.2 Future Works

Due to the time limitation, several possible trials that may improve the numerical model could not be made. The future work to be done may include:

- **Model Optimazition:** As stated in the conclusions, the numerical model is not perfect. The possible improvement could be: a) changing the connections between the floors and the masonry walls, especially the gable walls. The comparison of the acceleration and displacement shows the numerical results are larger than the experimental results; b) increasing the stiffness of the gable walls. And the natural frequencies may increase by increasing the stiffness, which may improve the results of the 100% of the PGA; c) increasing the tensile strength of the masonry; d) Newton-Raphson Method is adopted in this dissertation, in which the stiffness matrices need to be updated before every iteration. The calculation time for the scale model of 100% PGA load is about one week, and some steps did not converge. Other iteration methods could be adopted to compare.

- **Collapse Mechanism:** Collapse mechanism is an important aspect of the masonry dynamic analyses. In the dissertation, no such issues are addressed. Future work can be done by increasing the input of the earthquake and analyzing the possible collapse mechanism of the structures. The attention should be focused on the out-of-plane mechanism of the piers on the North/South facades.

- **Size Effect on the Strength of Masonry:** In this dissertation, the full model was used to detect the effect of the scaling in the shaking table tests, but the size effect on the strength of masonry is not taken into consideration. Further analyses could be made to discuss this issue.
Dynamic analyses of a masonry building tested in a shaking table
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Appendixes

For Chapter 4 and Chapter 5